Lateral capacity and Seismic characteristic of hybrid cold formed and hot rolled steel systems

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DECLARATION BY AUTHOR

The work presented in this thesis is, to the best of my knowledge and belief, original except as acknowledged in the text. I hereby declare that I have not submitted this material, either in full or in part, for a degree at this or any other institution.
**Published and Submitted Works Based on this Thesis**


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### Acronyms

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<th>Acronym</th>
<th>Definition</th>
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<tbody>
<tr>
<td>BMG</td>
<td>Bolivian Magnesium Board</td>
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<tr>
<td>CFS</td>
<td>Cold Formed Steel</td>
</tr>
<tr>
<td>CSB</td>
<td>Calcium Silicate Board</td>
</tr>
<tr>
<td>DSM</td>
<td>Direct Strength Method</td>
</tr>
<tr>
<td>FCB</td>
<td>Fibre-Cement Boards</td>
</tr>
<tr>
<td>FE</td>
<td>Finite Element</td>
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<tr>
<td>FSM</td>
<td>Finite Strip Method</td>
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<tr>
<td>GWB</td>
<td>Gypsum Wall Board</td>
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<tr>
<td>HRS</td>
<td>Hot Rolled Steel</td>
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<tr>
<td>HWP</td>
<td>Hybrid Wall Panel</td>
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<tr>
<td>HWPS</td>
<td>Hybrid Wall Panel System</td>
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<tr>
<td>IHWPS</td>
<td>Improved Hybrid Wall Panel System</td>
</tr>
<tr>
<td>LFRS</td>
<td>Lateral Force Resisting System</td>
</tr>
<tr>
<td>LSF</td>
<td>Light Steel Framing</td>
</tr>
<tr>
<td>OSB</td>
<td>Oriented Strand Board</td>
</tr>
<tr>
<td>SBMF</td>
<td>Special Bolted Moment Frames</td>
</tr>
<tr>
<td>SHS</td>
<td>Square Hollow Sections</td>
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<tr>
<td>SHWPS</td>
<td>Stiffened Hybrid Wall Panel System</td>
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This thesis addresses the application of hybrid cold-formed steel (CFS) - Hot-Rolled Steel (HRS) structures, as a new lateral force resisting system for light weight steel framed buildings in seismic regions. The study considers hysteretic behaviour, as well as maximum lateral load resisting capacity through comprehensive testing and advanced numerical analyses. The study identifies the advantages and disadvantages of the proposed hybrid system and provides in depth knowledge about performance characteristics of this innovative structural system, in order to facilitate the use of this system in earthquake-prone regions. The project is divided into three main parts: experimental, numerical and analytical studies.

A comprehensive literature review is performed as a part of this study, in order to discover the existing gaps in the current knowledge regarding the structural performance of CFS structures and the methods for lateral performance enhancement. The literature review suggests that although CFS walls are not new, and have been used as non-structural components for many years, their application as main load-bearing structural frames is relatively new. That is, appropriate guidelines that address the seismic design of CFS structures have not yet been fully developed in the literature. In addition, the lateral design of these systems is not adequately detailed in the available standards of practice. There have been several attempts to improve the seismic performance of such structural system by different bracing or sheathing configurations. However, there is minimal background information available on hybrid systems such as hot rolled-cold formed structures.

In this study, a series of CFS-HRS hybrid shear walls are constructed in order to investigate the lateral behaviour of the walls with different configurations to obtain the
optimum combination of HRS and CFS. Different configurations are considered to provide the most efficient load transfer pattern from cold formed steel part of the wall to the Hot Rolled section, which is responsible for withstanding the lateral loads. The CFS part is aimed to transfer lateral loads to HRS part without any internal local failure. The ideal failure condition is the HRS yielding. Therefore, the optimum rigidity of the HRS part is of great importance to prevent any local failure happening prior to reaching the maximum lateral capacity of the HRS. For each experimental specimen, the hysteretic envelope curve is plotted, and different characteristics are evaluated. Since the failure mode of such systems is very complicated, the test results will provide the possible failure modes to be utilised for any further investigation or any optimisation analysis in numerical and analytical studies.

In addition, Non-linear finite element (FE) analysis is employed using the ABAQUS software [1], in order to investigate the seismic performance of the proposed hybrid shear walls in multi-storey light steel frames. The nonlinear analysis accounts for different structural characteristics, including material non-linearity, geometric imperfection and residual stresses. The numerical models are verified based on experimental test results. The principal objective of this part of the study is aseismic optimisation of the proposed hybrid system and finding the corresponding dimensions and configurations to improve the strength and stiffness to achieve the objective. Using the hybrid wall panel system, a 4-storey building in an earthquake prone region is designed as per the relevant codes of practice. For the designed 4-storey building, the CFS part of the panel only bears the gravity loads, while a hot rolled steel collector transfers the lateral load to the HRS part acting as the main lateral load resisting system. Finally, the building is designed using different lateral load resisting systems
and the results are compared with those from the proposed hybrid system in terms of cost.

Furthermore, based on the real failure mode shapes obtained from test specimens, a Finite Strip Method program is developed to evaluate the elastic buckling mode shapes of a single stud with an arbitrary section detail. The code is helpful for design of CFS studs as explained in Chapters 3 and 5.
**KEYWORDS**

Cold-formed Steel, Hot-rolled steel, Light Steel Frames, hybrid systems, Finite Element Modelling, Finite strip method, structural design.
Chapter 1 Introduction
1.1. Problem overview

Lateral load resisting systems are aimed at transferring the total applied lateral load to the base of the structure. In a steel structure, the most popular lateral load resisting systems are generally classified into two main categories: moment resisting frames and shear walls or a combination of both, a hybrid system. In moment resisting frames, the beams are rigidly connected to columns and the lateral load resistance is provided basically by the bending moment and shear force developed in rigid joints. On the other hand, shear walls can provide lateral load resistance for simple frames. In this case, the wall system (which can be selected among a large variety of systems) is responsible for resisting and transferring the loads to the foundation. Each of the above mentioned lateral load resisting systems are applicable depending on various parameters such as size and shape of structure, lateral load effects, weight of the structure and economic constraints.

With respect to the material used, two main types of steel structural members are hot rolled profiles and cold formed sections. Hot Rolled Steel (HRS), which is more common, includes hot rolled shapes and the sections made of individual plates. The second group, which is fast growing in construction, is Cold Formed Steel (CFS). CFS structural sections are composed of steel sheet, plate or strips cold formed in roll forming machines, press brake or bending brake process.

The appropriate application of both cold formed steel and hot rolled steel in a structural system can combine the advantages of both elements and compensate for the deficiencies of each other. This research is conducted on hybrid CFS and HRS systems useable in residential construction. This is a new concept that increases the lateral capacity, hence addressing the main deficiency of solo cold-formed systems, thereby
increasing the extent of use for residential construction, in particular in seismic and cyclonic wind regions where the lateral capacity demand is high.

The project is a combination of experimental, numerical and analytical work with an emphasis on detailing of the elements connecting the two systems (CFS and HRS) together in addition to the energy absorbing mechanism in the hot rolled component, which shall take major responsibility for resisting the lateral loads. The bulk of the project will be conducted on shear walls such as the one shown below in Figure 1-1. It can be seen that the proposed hybrid system is composed of a cold-formed component (shown on the left) which is composed of studs, top plate and bottom plate in addition to a hot-rolled framed component on the right. The two systems are connected in such a way that the load transfers between the two works adequately and appropriately, when it comes to resisting lateral loads.

![Figure 1-1: An overview of a HRS-CFS hybrid shear wall](image)

The application of cold formed steel for structural and building purposes dates back to 1850s in the US and the Great Britain. However, the use of this type of steel members was not widely developed till 1940s. By issuance of the editions of “Specification for Design of Cold-Formed Steel Structural Members” by the American Iron and Steel
Institute in 1946, the application of thin-walled steel members in construction industry entered into an accelerated development phase.

The main general advantages of cold-formed steel members are as follows:

1. CFS provides a lightweight structure which is about 35% to 50% lighter than an equivalent timber structure.
2. CFS structural elements can be prefabricated off-site and under high quality control and the only onsite operation is the assembly. Thus, CFS provides speed of construction as well as superior construction quality.
3. Cold forming provides economically acceptable production of unusual section configurations which also leads to a favourable strength to weight ratio.
4. Since CFS structural members are termite and rot proof, structures made of CFS remain durable.
5. Cold formed steel is favoured by the environmentalist as it can be recycled and re-used easily.

Alongside the above-mentioned advantages, the following qualities are also displayed by cold formed steel members: ease of mass production, rapid erection and installation, considerable elimination of weather induced delays, accurate detailing, shrinkage and creeping elimination at ambient temperatures, no formwork needed, uniform quality, and the ease of transportation and handling.

Because of these advantages, the use of CFS is dramatically increasing worldwide. The only drawback to the system is its small lateral load resisting capacity which does not allow the system to be used in cyclonic wind regions or highly seismic zones comfortably, in particular in midrise construction. In those regions, one is limited to buildings of only few stories tall, thereby, losing a big portion of the market to other
systems. This research aims to combine the many advantages of CFS with the high lateral load capacity that can be gained from a HRS and creating a hybrid system that can work effectively well in all loading zones.

1.2. Aims and objectives

The main objective of the project is to improve the lateral load resisting capacity and the energy absorption of Cold-Formed steel framed panels by introducing a Hot Rolled-Cold Formed hybrid action in individual panels. In this regard, the project shall address the optimum load transfer pattern from the cold formed part of the panel to the hot rolled elements, which are mainly bearing the lateral load. The load transfer pattern in CFS members is a complicated issue, due to the complexity of the thin walled action. Both CFS and HRS are responsible for carrying vertical gravity loads. However, with regard to lateral loads, the entire system shall mainly rely on the hot rolled component. Therefore, the CFS share of lateral load should be transferred to HRS so that the system demonstrates the maximum possible capacity. To achieve this aim, the system should be designed in a way not to allow the CFS component to fail prior to reaching HRS’s desirable performance objective. This will cause a more uniform hysteretic loop, better ductility capacity, and consequently a more reliable seismic response. This issue can be addressed by designing the CFS collector element and its connections to the HRS part. However, by increasing the height of the structure, the storey shear increases, and it raises the need for a more reliable collector element. In that case a HRS collector can replace the CFS one to transfer the entire storey shears to the base of the structure, while the CFS part only contributes to resisting the gravity loads.

The objectives of this research study are, therefore as follows:

- Enhancing the hysteretic response of CFS panels by introducing an HRS component;
• Conducting necessary full-scale experimental tests to evaluate the response of the hybrid CFS-HRS panels, and analyse the results;

• Hot-rolled panel design and energy absorption capacity evaluation so that the load transfer between the two components of the system path is optimised and the maximum capacity of the system is obtained;

• Developing numerical tools including finite element (FE) models using commercial software such as ABAQUS plus a finite strip code in MATLAB to account for all possible failure modes determined in experimental tests. By calibrating these, and optimizing the system to obtain higher capacity of the proposed system, extrapolating and expanding on the experimental findings; and finally

• Conducting a finite element (FE) study on a multi storey building applying the proposed lateral load resisting system and compare the results with a traditional cold formed steel bracing system.

1.3. Research methodology

The research is to be conducted according to the following steps:

a) A comprehensive background study on the theories and standards related to cold-formed steel design.

b) A thorough literature review of the state-of-the-art with respect to the lateral load resisting systems, currently available for CFS structures.

c) Design and fabrication of a testing rig that allows cyclic loading of the hybrid system while vertical loads being applied.

d) Conducting experimental studies on the proposed hybrid wall panel system (HWPS) where a CFS section is acting as the lateral load collector element and detecting the deficiencies and suggesting the possible improving amendments.
e) Conducting experimental studies on the improved hybrid wall panel system (IHWPS) with the CFS collector element.

f) Developing numerical models with finite element and finite strip methods and validating the results with their experimental counterparts.

g) Conducting experimental studies on laterally stiffened hybrid wall panel system (SHWPS) which is applicable for higher number of stories where a HRS profile acts as a collector element.

h) Finite element analysis of a multistorey structure with the proposed lateral load resisting system.

i) Design of the multistorey structure with the proposed lateral load resisting system according to relevant codes of practice.

j) Developing a finite strip method code to determine the axial capacity of cold formed steel sections to be used in the multistorey structural design.

k) A cost analysis on the multistorey building and comparing the traditional and proposed systems

l) Presenting conclusions

1.4. **Organisation of the thesis**

In Chapters 1 and 2, the concept of the proposed hybrid system and a literature review is provided. Chapter 3 is an introduction and review on finite strip method and provides the details of the developed code to be used later in Chapter 5 for multistorey structural design. Chapter 4 includes the testing rig design and fabrication process. In addition, the results of wall panel tests and design of improved panels according to each test result is provided. Using Finite Element method program, ABAQUS, a model of the tested wall panel is developed and verified in Chapter 4. The results of multistorey structural analysis using an equivalent lateral force procedure are presented in Chapter
5. A comparative cost analysis between the traditional and proposed lateral load resisting system is provided in Chapter 6. Chapter 7 is assigned to concluding remarks and any suggestions for further research.

The following is a brief overview of the chapters of this thesis:

Chapter 1: An introduction to the proposed system and a general overview of the research program explaining the aims and objectives. This chapter also briefs the methodology and the steps for the thesis.

Chapter 2: A comprehensive literature review on CFS lateral load resisting systems and the relevant codes of practice is provided in this chapter. The review is not restricted to the main objectives of the present research as there was a scarce number of studies on CFS-HRS hybrid action. The review represents various CFS lateral load resisting systems such as CFS shear walls with (wood, steel or other materials) face sheathing, CFS strap braced wall systems and mixed shear wall systems. The chapter also introduces the relevant standards.

Chapter 3: In this chapter CFS design tools are introduced and explained. A finite strip code is developed in MATLAB software to evaluate the buckling loads of an arbitrary CFS section with general boundary conditions.

Chapter 4: This chapter starts with the design and manufacturing process of a testing rig which was particularly conducted for the present research project. The preliminary analysis and design results along with fabrication drawings are presented later in the chapter. The chapter is continued by test and FE results of the HWPS and IHWPS with the cold formed steel top chord acting as the load collector element.

Chapter 5: An introduction to the SHWPS with a HRS profile acting as the collector element and the relevant experimental and FE results are provided in this chapter. It is
followed by analysis and design of a 4-storey building according to relevant codes.

The CFS studs are also manually designed using different methods and the results are compared. A brief cost analysis is provided in this chapter and a comparison among different applicable lateral force resisting systems (LFRSs) is provided.

Chapter 6: This chapter is dedicated to a summary of the thesis and the drawn conclusions. Some recommendations for future research are also presented.
Chapter 2 Literature review
2.1. Cold formed steel lateral force resisting systems

The great progress in the knowledge of cold formed steel structures achieved in the past two decades, together with the modern design and fabrication methods supported by progressively improved specifications, have prepared the light steel construction industry to play an important part, with confidence, in the future of building construction. Despite the increasing hopes of expanding the use of cold formed steel (CFS) framing into more complex, robust and taller structures, the lateral performance of CFS framed structures has remained one of the major concerns, in particular with the applications in mid-rise residential construction. The following chapter reviews and summarises the major research developments in the area of major lateral force resisting systems in light steel frames (LSF) as published in leading journals and codes’ provisions in the area. Research advances in shear walls with face sheathings, CFS frame strap-braced wall systems and some frame-connection systems are reviewed here.

2.2. Introduction

Light steel framing (LSF) systems are being widely used in a variety of low rise housing construction, while targeting a major share in the mid-rise residential construction. The increased demand for mid-rise light weight steel frames in recent years has resulted in targeted research activities being undertaken in order to enhance the performance of these systems and make them a better match for mid-rise construction. The lateral performance of cold formed steel (CFS) framed structures is one of the major concerns over the use of these systems in mid-rise residential construction. Recent advances in the understanding of LSF’s behaviour, and ongoing research related to the design of lateral force resisting systems (LFRS) are hoped to
expand the use of CFS framing into more complex, robust and taller structures. Yet, it should be noted that research activities would be of little consequence, unless they address the issues arising from the practical applications, and result in continual increase in the market share of cold-formed sections. This in turn, would make demands on design procedures and requires parallel development in design codes, which are not adequately developed in the area of LFRS in LSF systems at the moment.

Unlike hot-rolled steel structures, where a variety of braced and moment frame systems are available, CFS traditional braced and moment frame systems have proven challenging. The thinness of CFS sections and rather low rigidity, i.e. partially restrained connections make most CFS moment frame systems relatively inefficient.

A bare standard CFS framing panel, consisting of studs and track, has very little to no lateral resistance [2]. On the other hand, since close spacing of vertical members is an efficient arrangement for gravity loading in CFS framing, concentric bracing through the webs becomes more complex. Consequently, the lateral resisting systems for CFS framing follow more of the traditions found in timber construction than in hot-rolled steel construction [3].

The US national institute of standards and technology [3] defines LFRS as the structural elements and connections required to resist racking and overturning, because of wind, seismic, or other predominantly horizontal forces, and/or combinations thereof, imposed upon the structure in accordance with the applicable code. LFRSs for LSF typically fall into one of the following categories: (1) shear walls with face sheathings such as plywood, plasterboard or steel sheets; (2) CFS frame strap-braced wall systems; (3) some frame-connection systems such as special bolted moment frames; (4) podium-type structures, where a complete load bearing CFS light-frame is built atop lower levels of other construction, such as concrete or structural steel; and
(5) mixed (hybrid) systems where CFS joists, trusses, and load-bearing walls are used for the primary gravity system, diaphragms, and collectors, and concrete shear walls or structural steel braced or moment-resisting frames are used for the vertical elements of the LFRS. Determination of an appropriate LFRS for a given building is dependent upon architectural and structural considerations. Moreover, other factors like the hold-downs, shear bolts, strap anchorage details (for example gusset plates), and bearing of panels on the foundation beam may also influence the overall shear resistance of the light-gauge steel framed wall [4].

This chapter reviews and summarises the major research developments, and provides an up to date review of references on LSF LFRSs with regard to the above-mentioned categories (1) through (3). The two latest categories, i.e. podium-type structures and mixed systems are beyond the scope of this study, as the LSF does not play an important role as an LFRS in those systems.

2.3. Lateral behaviour of light steel frames

Lateral design provisions for cold-formed steel light-frame shear wall structures were first introduced into building codes in the released version of Uniform Building Code in 1997 [5]. Extensive research, testing and analysis led to substantial progress in the lateral design provisions for CFS framing as reflected in the 2004 American Iron and Steel Institute Standard for Cold-Formed Steel Framing-Lateral Design (AISI, 2004). Nowadays, there are several provisions for lateral design of CFS frames, such as FEMA-450 [6], TI 809-07 [7], ASCE 7 [8], which refers to AISI standards for lateral design [9-17], AS/NZS 4600 [18] and IBC [19]. AISI S240-15 (covering structural systems) and AISI S400-1 (covering seismic force resisting systems) have evolved from older AISI standards: S110, S200, S210, S211, S212, S213 and S214. Table 2-1 shows this evolution path over time [20].
The factors affecting LSFs behaviour under lateral loading, as well as factors affecting the behaviour of CFS wall panels have been classified by Gad et al. [21]. They include factors such as properties of frame (members’ material properties, spacing, etc.); cladding (material, thickness and orientation, number of cladded sides, type and configuration of fasteners); bracings (material, type of bracing and member properties, fixity details and initial tension level); cladding and bracing interactions; aspect ratio (length to height ratio); boundary conditions (set corner joints, ceiling cornices, skirting boards and vertical loads); and size and location of openings. These factors are shown in Table 2-2.

<table>
<thead>
<tr>
<th>Structural Systems</th>
<th>Seismic Force Resisting</th>
</tr>
</thead>
<tbody>
<tr>
<td>S200</td>
<td>S210</td>
</tr>
<tr>
<td>S211</td>
<td>S212</td>
</tr>
<tr>
<td>n/a</td>
<td>n/a</td>
</tr>
<tr>
<td>S214</td>
<td>S213</td>
</tr>
<tr>
<td>Lateral</td>
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</tr>
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<td>S110</td>
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</tr>
</tbody>
</table>

Table 2-2: Factors that influence the lateral behaviour of framed structures

When studying the lateral performance of a structural system, their seismic behaviour
is of critical importance. Many studies have approached the problem of lateral or seismic response of LSF structures by characterising the performance of wall panels. The performance of LSF wall panels, on the other hand as a whole, is dependent on the performance of sheeting-to-sheeting connectors, and sheeting-to-framing connectors [22]. Moreover, the global behaviour of buildings made of LSF is significantly influenced by non-structural elements.

One major drawback of LSF structures, with regard to the seismic behaviour, is the relatively small seismic ductility. As a result, low levels of response modification coefficient are deemed to be applicable. In contrast, thanks to rather low seismic weight of this type of structure, low levels of seismic forces allow the design to be carried out within the elastic range of response, i.e. with relatively low values of the strength reduction factors. Results of an experimental and numerical study [23-25] on the seismic behaviour of CFS stud shear walls, sheathed with Oriented Strand Board (OSB), and gypsum-board panels (also called gypsum wall) demonstrated that these systems can be constructed in the low to moderate seismicity zones, when they are designed according to relevant standards [26]. Many standards such as ASCE7’s equivalent lateral force procedure use response modification coefficients and system over-strength factor. Table 2-3 shows these factors for different seismic force resisting systems. An overview of the seismic requirements for various design standards and explanation of how these factors are determined in different design standards are presented by Zhao and Rogers [27].
In parallel with some attempts for the efficient use of CFS frames in mid-rise construction [28], in 2012 Cold Formed Steel Network for Earthquake Engineering Simulation (CFS-NEES) project incorporated full scale shaking table tests together with advanced modelling of CFS framed buildings in an effort to address multi-storey CFS lateral force resisting systems for modern performance-based seismic design. Some outcomes of this project have been published in [29-32], and some will be published in the near future [33]. The tests were conducted on buildings with both structural components such as sheathed shear walls and floors and roof diaphragms, as well as non-structural components such as partition walls, staircase and ceilings. In a similar study, Shamim et al. [34, 35] evaluated the seismic performance of single- and double-storey steel-sheathed CFS framed shear walls to obtain measures of damping and natural period of vibration, investigate the influence of the second storey, and validate and improve the accuracy of the numerically predicted force-deformation response. The results revealed that the general strength-versus-drift hysteretic behaviour, failure modes as well as ultimate and nominal shear resistances under dynamic lateral loading did not significantly differ from those tested using reversed-cycle displacement-based loading protocols. Their results also indicated that a reasonable prediction of the frequency and damping is possible, by using the existing empirical expression and modelling, if the wall stiffness and tributary mass are known.
In another attempt, Shamim and Rogers [36] proposed a multi-phase approach to develop appropriate seismic design provisions for steel-sheathed CFS framed shear walls for inclusion in the new AISI S400 Standard [15]. They showed that improvement to the predicted seismic response of CFS framed buildings could be forthcoming, provided the contribution of the non-structural gypsum wall panels is considered.

In the design of LFRSs, in particular in a seismic resistant design, an important consideration is the distribution of lateral load demands in proportion to the in-plane relative stiffness of diaphragms and wall bracings, thereby restricting the inter-storey drift ratios within an acceptable range. Serrette and Chau [37] had a review and a discussion on the AISI provisions in order to estimate lateral displacement of conventional CFS shear walls. In a study by Dubina [22] on the seismic performance of LSF structures, monotonic and cyclic tests on full-scale shear panels, tests on connection details, as well as in-situ ambient vibration tests on a house under construction were conducted. The study concluded that in terms of performance, for “fully operational” and “immediate occupancy” levels according to Eurocode 8-1 [38], LSF houses can be designed as “Low Dissipative Structures” by taking the corresponding R factor of 1.5-2.0, while, for “life safety” and “collapse prevention”, these structures could be deemed as “Moderate Dissipative” with a corresponding R factor of 2.0–3.0.

In order to investigate the non-linear dynamic behaviour of CFS frames, Kim et al. [39] documented the results of shake-table tests of a full-scale two-storey structure, in which as shown in Figure 2-1, the cross-bracing straps showed very ductile but highly pinched hysteretic behaviour.
The CFS columns, on the other hand, showed a small contribution to the shear capacity of the structure, while performed well in energy dissipation. Gad et al. [21] investigated the performance of domestic structures with CFS frames when subjected to earthquake loading to identify the critical components and assess the influence of non-structural components. They concluded that the steel frames perform very well under earthquake loads, and non-structural components, such as plasterboard lining, make a significant contribution to the lateral bracing of the frames. Wang et al. [40] performed a full-scale shake-table test on a five-storey building to study the seismic performance of two types of CFS wall systems: an exterior architectural façade, and an interior partition wall system.

With respect to the effects of CFS elements in earthquake resistant moment frame buildings, Bagheri et al. [41, 42] studied the development of these sections as energy dissipative elements for seismic moment-resisting multi-storey frame buildings. Likewise, Weng and Ye [43] conducted cyclic testing of two- and three-storey CFS shear-walls with reinforced end studs. The results showed that (i) similar to the single-storey shear wall test results, failure modes mainly occurred in the underlying walls;
(ii) two-storey specimens with an aspect ratio of less than 2.0 had shear type failures, whereas the failure mechanism of three-storey specimens with an aspect ratio larger than 3.0 tended to be bending failure; (iii) both shear strength and non-deformability were enhanced by adopting reinforced end studs, and the specimens revealed better energy-dissipating capacity after the yielding point, as shown in Figure 2-2.

Experimental tests and finite element modelling were employed by De Matteis [44, 45] to assess the shear behaviour of CFS shear walls both in single-storey and multi-storey buildings, and then an equivalent numerical model for hysteretic behaviour of wall panels was developed. In order to prevent failure at the bottom track in the anchor bolt region, they suggested strengthening of the corner details, so that the uplift force is directly transmitted from the brace (or corner stud) to the anchor bolt, without imposing extra bending moment to the bottom track.

Some research studies have also been conducted on the racking performance of CFS partition walls. An experimental investigation was conducted by Restrepo and Bersofsky [46], where the configuration of the walls, the effects of metal doorframe,
openings, wallboards, screw spacing, stud thickness and spacing, top tracks and the wallboard thickness were assessed. They proposed damage limit-states at similar drift ratios for each case. Moreover, a series of full-scale system-level experiments using a two-storey steel braced-frame structure was conducted by Jenkins et al. [47], and behaviour of CFS framed partition wall systems were critically assessed using similar design variables. Xu and Martinez [48] presented an analytical method to determine the ultimate lateral strength of the shear wall panels and its corresponding displacement, by taking material properties, geometrical dimensions and construction details into account. They also presented a simplified approach for analysing cold-formed steel buildings by using finite element methods, in which the nonlinear behaviour of shear wall panels was simulated, and a lesser number of elements were used for modelling a structure.

2.4. CFS shear walls with face sheathing

Sheathing can provide adequate bracing for axial load bearing CFS stud walls. In sheathed wall systems, the primary stability resistance for the studs is provided by translational (lateral) stiffness supplied by the sheathing at the stud-to-sheathing fastener locations. CFS sheathed panel systems include CFS frames sheathed with wood structural panels, steel sheet, gypsum board, etc. The sheathings usually have adequate strength and stiffness to resist shear in its plane, and can act as a web, in resisting in-plane shear. Vieira and Schafer [49] separate the source of this translational stiffness into two parts: local and diaphragm. That is, the sheathing bracing derives from both local fastener deformations and global shear diaphragm behaviour. In the literature, investigating the lateral performance of CFS sheathed panel systems has been done using three different methods: empirical relations
obtained from experimental data; simplified mathematical derivations; and the finite element modelling. The CFS framing deforms in shear under lateral loads, while the sheathing rotates and creates differential demands at all the fastener locations, thus, developing the primary mechanism resisting lateral loads. The attachment of sheathed panels also enhances the capacity of the studs under gravity load.

In an AISI sponsored research report [50], shear values for plywood, steel, OSB and gypsum wall board (GWB) sheathed LSF walls, as well as X-braced wall assemblies were developed. This research report presents the applied load, lateral displacement at the top plate, uplift, and mode of failure for various wall assembly configurations, tested in the program. Lange and Naujoks [51] developed a design procedure that allows for the design of CFS shear walls with sheathing on one or both sides, carrying horizontal and vertical loads. Also, the behaviour of CFS wall frames with different sheathing systems (expanded polystyrene, OSB, calcium silicate and gypsum board) affected by angular distortions simulating ground differential settlements due to land subsidence were studied by Castillo et al. [52]. According to their results, the OSB wall frame offered the greatest stiffness of all the systems, while polystyrene wall frame presented the lowest stiffness as a structural system. On the other hand, the cold-formed steel wall frame with polystyrene sheathing presented a greater ductility in comparison with other systems.

Most design provisions for CFS shear walls with wall height aspect ratios greater than 2:1, and less than 4:1 require a reduction in nominal strength based on the wall aspect ratio, which is attributed to increased wall flexibility. According to the results of an extensive experimental program, Nava and Serrette [53] offered a more conservative and consistent strength reduction factor for OSB sheathed CFS shear walls. In another research study on the seismic capacity of sheathed CFS walls, Fiorino
et al. [54] evaluated seismic design parameters such as behaviour factors and inter-storey drift limits as well as some dynamic characteristics such as vibration periods and damping ratios for various wall configurations, sheathing panel typology, wall geometry, external screw spacing, seismic weight and soil type. They also developed an approach for the seismic design of sheathed CFS frame structures, as well as a model to predict the whole pushover response curve of sheathed CFS shear walls [55]. Another seismic design procedure for CFS structures employing sheathed shear wall panels, compatible with the framework of the Eurocodes, was also proposed by Kechidi et al. [56]. The CFS structural system evaluated in their study was shown to meet the acceptance criteria for a behaviour factor equal to 2 for low- and moderate-seismicity. Their results also revealed that the lateral over-strength has a relevant influence on the probability of collapse and that an improved performance could be achieved if continuity of the chord studs along the height is enforced. Kechidi et al. [57] also developed smooth hysteresis models that take into account strength and stiffness degradation, as well as pinching effect for wood and steel sheathed walls. In another attempt, Serrette [58] developed a method for estimating the available strength level (load and resistance factor design) seismic resistance of CFS shear walls that accounted for the early onset of inelastic behaviour in light-frame shear walls.

Bracing systems can be categorised, by function, performance, or the method of interaction between elements [59]. Here, the literature review on CFS shear walls with face sheathing is categorised as the ones whose sheathing materials are specifically recognized by AISI S400 [15], such as shear walls with wood structural panels (plywood or OSB) attached to cold-formed steel studs and tracks; or shear walls with steel sheet sheathing attached to cold-formed steel studs and tracks; and those not specifically recognized by AISI S400-15, such as shear walls with gypsum board.
2.4.1 Shear walls with wood structural panels sheathing

Shear walls with wood structural panel sheathing, which are also discussed as category E1 in seismic force resisting systems in AISI S400 [15], are one of the most common LFRS used with CFS framing. A considerable body of research has confirmed that the deformation in the member-to-sheathing connections, and the wood panels structural characteristics are the primary lateral force resisting mechanism for this system [3]. The nominal shear strength of shear walls constructed with structural sheathing and oriented strand board with a variety of fastener spacing, as well as stud and track thickness is given in AISI S400 [15]. However, it has been shown that the lateral performance of CFS framed, wood-sheathed, walls are dominated by the local response of the sheathing-to-steel connections. This is mainly because of a complex interaction between the fastener and the sheathing and steel sheet that are connected together and is considered highly variable [60].

AISI S400 [15] provides design requirements for Type I and Type II shear walls with wood structural panels, where Type I shear walls are fully sheathed with hold-downs at each end but are allowed to have openings where detailing is provided for force transfer around the openings; while Type II shear walls are allowed to have openings without specific details for force transfer around openings. In either case, CFS shear walls with wood structural panels require all sheathing edges to be attached to framing members or panel blocking, which is used to transfer shear between adjacent panels. Expected strength of CFS-wood structural panel shear walls is subject to uncertainties such as construction techniques and/or variability in timber materials that are accounted for by applying an over-strength factor. The basis of design of wood structural panel shear walls matches the procedures for Steel sheathed shear walls.
Pan and Shen [61] conducted an experimental study on the structural strength of CFS wall frames with three different kinds of sheathing material (gypsum board, calcium silicate board, and oriented-strand board, with two different thicknesses (9 and 12 mm)), under monotonic shear loading. The ultimate strength, stiffness, energy absorption, and ductility ratio were studied for each test specimen, and the ductility ratios of the CFS wall frames were proposed. They concluded the bearing failure of sheathing around the self-drilling screw area and the separation between sheathing and screw are the main reasons to induce failure. The wall frame with OSB sheathing showed the greatest ultimate shear strength, with calcium silicate board providing secondary strength, and gypsum boards making the smallest contribution. Also, the ultimate strengths of specimens having an aspect ratio of 2.0 were about 35% less than those of the specimens having an aspect ratio of 1.0 for the wall frames with the same sheathing configuration. Regarding the energy absorption, the specimen with calcium silicate board had the highest value, those with OSB had at smaller value, and those with gypsum had the lowest value. While the ductility ratio of the specimens showed dependency to the screw arrangement and anchor condition, the specimens with one-sided sheathing were more ductile than those with two-sided sheathing. In another experimental study on CFS wall panels with OSB sheathing, Bran and Alica [62] concluded that the geometry of hold-down attachment used at the base of CFS wall panels to transfer tensile forces has a major effect on the overall behaviour of the panels. They also showed that existence of diagonal struts inside CFS frame, as shown in Figure 2-3, causes a slight increase in load capacity and initial stiffness of the panels.
Serrette et al. [63] explored the potential benefits of using a continuous adhesive bond for load transfer between the framing and the steel and OSB sheathing. Their results showed that the peak wall resistance significantly exceeded the nominal values in the current building codes and standards for similarly sheathed walls without adhesives, while the specimens with adhesive exhibited more linear responses with higher stiffness and generally larger degradation in strength after the peak resistance. In another study, Swensen et al. [64] showed that while enhanced screws can increase connection and wall strength modestly relative to conventional screws, a much more significant increase in strength and stiffness can be obtained by using construction adhesives, as shown in Figure 2-4.
2.4.2 Shear walls with steel sheet sheathing

Use of steel sheets as a sheathed material for CFS wall frames has gained popularity in the building construction due to its high shear resistance, high ductility, lower combustibility and good construction feasibility. In general, the use of closely spaced sheathing panel fasteners and thicker panels will lead to a higher shear resistance if the stud members are designed to carry the additional force. Nominal shear strength values for the steel sheets with a thickness of 0.46 mm to 0.84 mm together with framing thickness and various screw spacing are presented in AISI S400-15 [15], inspired by the works of Yu and Chen [65-67]. Additional research seems to be required though, to expand the available data to thicker steel sheets and framing members that enable higher strength shear walls for use in mid-rise construction. Results of an experimental investigation [68] on single and double-sided steel sheathing CFS shear walls under cyclic loading showed the walls developing sheathing connection failure show higher energy dissipation than the walls imposing chord stud buckling. Using double-sided sheathings increases the energy dissipation, shear strength and elastic stiffness by up to 70%, 63% and 115%, respectively, compared to those of single-sided sheathed walls, provided the chord stud failure is avoided. With regard to double sided steel
sheathing or thicker steel sheathing CFS shear wall panels, Niari et al. [69] reported the results of an experimental investigation on the maximum lateral load capacity and failure modes of walls when the steel sheathing thickness and the number of layers of sheathings vary. They showed by using double sided steel sheathing, shear resistance of CFS shear wall doubled, while increasing the thickness of the sheets by 50%, resulted in 42% increase in the shear resistance. The ultimate shear resistance of the CFS walls was directly related to the failure of the connections of the steel sheathings to the framing.

In order to investigate the performance of single-storey steel-sheathed CFS framed shear walls, a set of displacement based loading tests was carried out by DaBreo et al. [70] on various framing thickness, sheathing thickness, screw fastener detailing, aspect ratio and framing reinforcement. They concluded the use of blocked studs and a capacity-based design approach allowed for an increased shear resistance and protection of the gravity load carrying framing members. These are shown in Figure 2-5. The shear resistance was also dependent on the shear buckling response of the panel, and the performance of the walls was affected by the damage caused to the chord studs.
Figure 2-5: (a) Details of typical walls with blocked, bridging and unblocked framing. (b) Comparison of normalized monotonic resistance vs. displacement curves. (c) Comparison of normalized reversed cyclic resistance vs. displacement curves [70]

Using the database of monotonic and reversed cyclic shear wall tests from this study, Balh et al. [71] produced a simplified bilinear elastic–plastic force–displacement curve for the CFS walls, and developed a method for the design of steel sheathed CFS framed shear walls for improved design provisions in the AISI standards. In order to predict the structural behaviour of CFS framed shear wall sheathed with steel sheet sheathing according to AISI standards, an analytical design method using the effective strip method was developed by Yanagi and Yu [72] for predicting the nominal strength of walls.

Attari et al. [73] proved that the reduction factor, used in most LSF design provisions for the shear walls’ strength with a height to width aspect ratio greater than 2:1, is conservative and requires some modifications. In another study [74], they compared the behaviour of one and two sided steel sheeting, and the effect of the nominal
thickness of steel sheet and boundary elements, as well as height to width aspect ratios of the wall. The tests exhibited that the capacity of two-sided steel-sheathed walls is more than twice of those for one sided steel sheathed, provided that the boundary elements are strong enough to sustain imposed forces. They concluded the performance of CFS shear walls is highly dependent on the ratio of boundary element thickness to steel sheet thickness, and there is a linear relationship between the nominal sheet thickness and the ratio of ultimate strength to nominal frame thickness.

Ronagh et al. [75] conducted an experimental investigation on CFS frames sheathed by thin galvanized steel plates, under cyclic loading with different configurations of studs and screws. According to the test results, they suggested an increase in the AISI’s R factor from 6.5 to 7 for these frames. They also concluded that decreasing the screw spacing from 150 mm to 100 mm enhanced the shear resistance capacity by around 16–18% in the single end studs. No enhancement was seen for panels with double studs at the end. Yet, more decrease in the screw spacing (from 150 mm to 75 mm) in the quarter length of the boundary studs did not result in a notable effect on shear resistance. In another experimental investigation on 1.83 m wide, 2.44 m high stud framed shear walls sheathed by steel sheets, Yu and Chen [76] studied different wall configurations through monotonic and cyclic tests. Their test results indicated that besides the sheet buckling and screw pull out, the interior studs may buckle under cyclic lateral forces if the minimum framing required by AISI S213 [77] is used without additional detailing. In this study special detailing was also developed to prevent the failure in the studs and increase both the shear strength and the ductility of the shear walls.

Corrugated steel sheathing has been shown to significantly increase the strength of the CFS shear walls, but with rather relatively low contribution to the ductility of the wall
for seismic applications. Stojadinovic and Tipping [78] developed an alternative lateral bracing system comprising corrugated sheet steel shear walls that enjoyed the additional shear strength provided by the corrugated sheet steel. Their system is suggested to be added to Table 12.2-1 of ASCE 7-05 as a bearing wall system utilizing light-framed CFS walls sheathed with corrugated sheet steel with the response modification factor of 5.5, system over-strength factor of 2.5, and deflection amplification factor of 3.25. In another study, Yu et al. [79] investigated possible solutions for improving ductility by creating openings in the corrugated sheets. They concluded that by having circular holes in the corrugated sheathing, the walls could fail in sheathing ruptures instead of screw failures. That is, although shear walls did gain ductility due to the new failure mode, the stiffness and strength of the shear walls were significantly reduced. In a recent attempt, Zhang et al. [80] presented the results of some experiments and numerical simulations, and proposed some seismic performance factors for CFS framed walls sheathed with corrugated steel sheets. Their test results indicated that gravity load at the service load level led to an increase of shear strength and initial stiffness. They suggested 7% drift, as the collapse drift limit for their corrugated steel sheathed walls.

#### 2.4.3 Shear walls with other sheathing materials

To satisfy the requirements of fire resistance and provide lateral and rotational supports to the studs in the plane of the wall, shear walls sheathed with various leaning materials such as GWB, Bolivian magnesium board (BMG) and calcium silicate board (CSB) are commonly used for CFS wall systems. Most standards allow using such alternate sheathing materials as acceptable LFRS, provided that they meet the standards’ requirements. For example, ASCE 7 [81] allows for all seismic force resisting systems that their dynamic strength and ability to dissipate energy are equivalent to a listed
system having the same response modification factor, over-strength factor, and deflection amplification factor [3].

The behaviour of CFS wall frames with different sheathing systems (expanded polystyrene, OSB, calcium silicate and gypsum board) affected by angular distortions simulating ground differential settlements due to land subsidence were studied by Castillo et al. [52] (Figure 2-6 (a)). They made a comparative study of the results of previously conducted research, highlighting specifications of wall frames, and types of sheathing as shown in Table 2-4, and in Figure 2-6. According to their collection, shown in Figure 2-6(b), the OSB wall frame offered the greatest stiffness of all the systems, while polystyrene wall frame presented the lowest stiffness as a structural system. On the other hand, the cold-formed steel wall frame with polystyrene sheathing presented a greater ductility in comparison with other systems.
### Table 2-4: Details of studies on CFS wall frames with different sheathing materials in Figure 2-6

<table>
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<tr>
<th>Author</th>
<th>Nomenclature</th>
<th>Type of sheathing</th>
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<th>Studs-tracks-separation</th>
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<td>UAA S/POL</td>
<td>Panel without polystyrene</td>
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<td>CB8.70×43.20×11, 1×0.9, U88.7×22.8×0.70, S-405 mm</td>
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<td>FFM-C09-HD</td>
<td>Calcium S. Panel (Simple) 9 mm</td>
<td>1.22×2.44</td>
<td>C92×65×12×1.6, U95,2×45×1.6 S-400 mm</td>
</tr>
<tr>
<td>Chi-Ling Pan</td>
<td>FFM-C12-FO</td>
<td>Calcium S. Panel (Simple) 12 mm</td>
<td>2.40×2.40 m²</td>
<td>C92×65×12×1.6, U95,2×45×1.6 S-400 mm</td>
</tr>
<tr>
<td>Chi-Ling Pan</td>
<td>FFM-C09-HT</td>
<td>Calcium S. Panel (Doble) 9 mm</td>
<td>1.22×2.44</td>
<td>C92×65×12×1.6, U95,2×45×1.6 S-400 mm</td>
</tr>
<tr>
<td>Chi-Ling Pan</td>
<td>FFM-C12-HO</td>
<td>Calcium S. Panel (Simple) 12 mm</td>
<td>1.22×2.44</td>
<td>C92×65×12×1.6, U95,2×45×1.6 S-400 mm</td>
</tr>
<tr>
<td>Nithyadharan</td>
<td>WP-12/M/25A</td>
<td>Calcium S. Panel (Doble) 12 mm</td>
<td>1.20×2.40</td>
<td>C100×50×20×1.2, U103×50×1.2 S-600 mm</td>
</tr>
<tr>
<td>Nithyadharan</td>
<td>WP-10/M/25A</td>
<td>Calcium S. Panel (Doble) 10 mm</td>
<td>1.20×2.40</td>
<td>C100×50×20×1.2, U103×50×1.2 S-600 mm</td>
</tr>
<tr>
<td>Nithyadharan</td>
<td>WP-08/M/25A</td>
<td>Calcium S. Panel (Doble) 8 mm</td>
<td>1.20×2.40</td>
<td>C100×50×20×1.2, U103×50×1.2 S-600 mm</td>
</tr>
<tr>
<td>Nithyadharan</td>
<td>WP-10/M/10A</td>
<td>Calcium S. Panel (Doble) 10 mm</td>
<td>1.20×2.40</td>
<td>C100×50×20×1.2, U103×50×1.2 S-600 mm</td>
</tr>
<tr>
<td>Nithyadharan</td>
<td>WP-10/M/10B</td>
<td>Calcium S. Panel (Doble) 10 mm</td>
<td>2p 0.6×2.4</td>
<td>C100×50×20×1.2, U103×50×1.2 S-600 mm</td>
</tr>
<tr>
<td><strong>OSB Panel</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Zhou Xuhong</td>
<td>SSG</td>
<td>OSB Panel (Simple)</td>
<td>3×2.40 m²</td>
<td>CB9×44.5×12×1, U92×40×1 S-600 mm</td>
</tr>
<tr>
<td>Chi-Ling Pan</td>
<td>FFM-009-FO</td>
<td>OSB Panel (Simple)9 mm</td>
<td>2.40×2.40 m²</td>
<td>C92×65×12×1.6, U95,2×45×1.6 S-400 mm</td>
</tr>
<tr>
<td>Chi-Ling Pan</td>
<td>FFM-012-FO</td>
<td>OSB Panel (Simple)12 mm</td>
<td>2.40×2.40 m²</td>
<td>C92×65×12×1.6, U95,2×45×1.6 S-400 mm</td>
</tr>
<tr>
<td>Baran</td>
<td>C140-11-15-E</td>
<td>OSB Panel (Simple)11 mm</td>
<td>1.22×2.44 m²</td>
<td>C140×47×12×0.8, U140×47×0.8 S-600 mm</td>
</tr>
<tr>
<td>Baran</td>
<td>C90-11-15-E</td>
<td>OSB Panel (Simple)11 mm</td>
<td>1.22×2.44 m²</td>
<td>C90×47×12×0.8, U90×47×0.8 S-600 mm</td>
</tr>
<tr>
<td>Baran</td>
<td>C90-11-30-E</td>
<td>OSB Panel (Simple)11 mm</td>
<td>1.22×2.44 m²</td>
<td>C90×47×12×0.8, U90×47×0.8 S-600 mm</td>
</tr>
<tr>
<td>Baran</td>
<td>C90-18-15-E</td>
<td>OSB Panel (Simple)18 mm</td>
<td>1.22×2.44 m²</td>
<td>C90×47×12×0.8, U90×47×0.8 S-600 mm</td>
</tr>
<tr>
<td>Baran</td>
<td>C90-18-30-E</td>
<td>OSB Panel (Simple)18 mm</td>
<td>1.22×2.44 m²</td>
<td>C90×47×12×0.8, U90×47×0.8 S-600 mm</td>
</tr>
</tbody>
</table>
Figure 2-6: (a) Moment–angular distortion curves for wall frames with different sheathing materials, (b) Load–displacement curves for cold-formed steel wall frames with different sheathing materials [52]
Gypsum plasterboard is a common lining material used in bearing and non-load bearing CFS wall frame systems. The behaviour of CFS walls lined with plasterboard is thoroughly studied by Telue and Mahendran [82, 83]. They studied unlined, both sides lined and one-side lined wall frames and studs and compared the results with predictions from the Australian standard and the American specification. In another study, Morgan et al. [84] presented the results of some shear wall tests, conducted to evaluate the performance of wall configurations not permitted in the building codes in the previous version of AISI specifications. Conducting an experimental program on the seismic response of steel sheathed CFS shear walls, using gypsum and fibre cement board claddings, Mohebbi et al [85] concluded that the shear strength and secant stiffness of the CFS steel sheathed walls can be increased by up to 31% and 32% for single-sided and 80% and 67% for double-sided claddings, respectively. They showed (Figure 2-7) when using double sided claddings, compared to single-sided and walls with no cladding, the ductility is significantly improved, and the hysteretic energy dissipation increased by 37% and 76% on average, respectively.

![Figure 2-7: Cumulative hysteretic energy dissipation’s comparison [85]](image)

Ye et al. [86] carried out a set of cyclic tests on CFS walls sheathed with double layer GWB, BMG and CSB on both sides of the walls, to study the effects of aspect ratio, sheathing material, screw connections and stud section and spacing on the shear
performance. The results indicated that BMG significantly exceeded the nominal value of the current standard, while CSB exhibited brittleness damage with poor ductility. The results showed significant improvement in shear strength of the walls by changing the interior stud section (using identical steel grade), but no meaningful difference by changing the stud spacing from 400 mm to 600 mm, nor by increasing the wall length. They also concluded that differences in sheathing materials influence the shear behaviour of the screw connections. The results also suggest walls sheathed with GWB and CSB should be used in areas of low seismicity, due to the shear-wall drift angle limit. A summary of their results are illustrated in Figure 2-8, where G12 and B12 stand for 12mm thick GWB and BMG, respectively; H indicates that the loading direction is parallel to the entire wallboard width; 15, 20 and 25 are the edge distances in mm; 0.9 and 1.2 represent the thicknesses of the steel plate in mm. Peterman and Schafer [87] characterized the behaviour of CFS studs sheathed by sheathed with OSB, GWB, or combinations, under axial and lateral load. According to their results, the failure modes for studs sheathed on only one face include torsion and/or fastener pull-through; and for studs sheathed on both faces include torsion, local buckling, fastener pull-through, and bearing.

![Figure 2-8: Typical load–deformation curves of the screw connections: (a) for GWB and BMG; (b) for CSB [86]](image-url)
When gypsum panels are applied to the bare wall frame, they were shown to significantly improve the shear strength (over 130% increase), but under seismic loading, the static values should be reduced to compensate for the opening of holes around the screw shank [4]. For CFS lined frames, Gad et al. [21] showed that while plasterboard fixed as a non-structural component provides higher stiffness, load carrying capacity and damping than strap braces, when plasterboard and strap braces are combined, the overall stiffness and strength of the system is a simple addition of individual contributions from plasterboard and strap brace. Though investigating load sharing and failure mechanisms of the various components, their tests also concluded that the plasterboard resisted about 60–70% of the applied racking load whereas the strap braces resisted 30–40%, while in-plane brick veneer walls attached to the frames had no contribution to the stiffness of the system. Gad et al. [88] also conducted a detailed investigation into the contribution of plasterboard to the lateral resistance of CFS frames, by studying the effect of boundary conditions, wall length, the spacing of plasterboard screws, load transfer between frame and plasterboard, and the effects of addition of strap bracing. Their results showed that in residential structures, walls with ceiling cornices, corner return walls, and skirting boards had more than three times the lateral capacity of an identical isolated wall panel. They also showed that the presence of these boundary conditions could influence the relationship between the wall length and the ultimate lateral load-carrying capacity of the wall system. In a similar study by Peck et al [89] on the performance of CFS framed gypsum shear walls under monotonic and reversed cyclic loading, it was concluded that increased strengths and wall toughness may be achieved with closer intermediate panel fastener spacing. In addition, they showed aspect ratio, and abutting supported vertical panel joints, had little effect on shear wall performance.
An experimental study was carried out by Zeynalian and Ronagh [90] to examine the lateral performance of currently in-use CFS frames sheathed by fibre-cement boards (FCB) under cyclic lateral loading. The results suggested that an overall performance of the currently in-use FCB sheathed lateral resisting system under cyclic loads is not satisfactory with a small average R factor of 2.5. They proposed an FCB lateral resisting system that can be considered as a more reliable system with a higher value of R factor of 5. Their proposed system included concurrent use of fibre-cement board on one side and X strap-braced system on the other side. Figure 2-9, illustrates their specimens configurations and the hysteretic curves.

![Figure 2-9: (a) Specimens configurations, (b) Hysteretic envelope curves [90]](image)

In another study, the behaviour and strength of the CFS shear wall panels with calcium silicate board as sheathing, subjected to monotonically increasing and reversed cyclic in-plane shear deformation, was studies by Nithyadharan and Kalyanaraman [91]. The study investigated the influence of board thicknesses, screws spacing and wall board configurations, as the parameters influencing the load-deformation behaviour, and determined different limit states in the failure of the screws connecting the board and the CFS framing. The study of structural strength and behaviour of cold-formed steel wall frame sheathed with calcium silicate board were continued by Lin et al. [92], by conducting monotonic shear and cyclic loading tests. It is noted that the failure mostly
occurred at the bottom track of wall specimens due to the large deformation or tearing failure of track. They showed that the wall strength is not affected by the change of sheathing’s thickness significantly, but wall frames attached with two-sided calcium silicate board sheathing provide higher resisting strength and stiffness than those attached with one-sided sheathing.

Some innovative CFS sheathed walls having shown some advantages over the conventional configurations have also been discussed in the literature. Tian et al. [93] developed a CFS truss-like shear wall used with various steel sheathings that shows higher ultimate strength. Table 2-5 shows the specifications of their specimens. Their testing results also showed that corrugated steel sheathing yielded less lateral deformation than plain steel sheathing under the same loading condition. Figure 2-10 illustrates shear-deformations curves for their walls.

Table 2-5: Detail of CFS truss-like shear walls used with various steel sheathing [93]

<table>
<thead>
<tr>
<th>Shape</th>
<th>Configuratio</th>
<th>Skeleton</th>
<th>Sheathing</th>
<th>Chord studs type</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>Truss</td>
<td>Truss</td>
<td>Truss</td>
<td>Truss</td>
</tr>
<tr>
<td>Sheathing configuration</td>
<td>Corrugated steel sheet</td>
<td>Plain steel sheet</td>
<td>Plain steel sheet</td>
<td>No sheathing</td>
</tr>
<tr>
<td>Chord studs type</td>
<td>Built-up section</td>
<td>Built-up section</td>
<td>Back-to-back section</td>
<td>Back-to-back section</td>
</tr>
</tbody>
</table>
Gao and Xiao [94] used glued laminated bamboo as a sheathing material for CFS shear walls and conducted an experimental study on the monotonic and cyclic lateral loading behaviour of this system. Results showed that the new system had similar failure modes as shear walls sheathed by conventional wood-based materials and can provide comparable nominal shear strength. They concluded, therefore, that the tested ply-bamboo sheathed CFS shear walls were sufficiently qualified as a seismic resistant system in terms of strength capacity and deformability.

Wang and Ye [95] proposed a CFS shear wall with concrete-filled rectangular steel tube columns as end studs and sheathed with autoclaved lightweight concrete slabs and Bolivian magnesium boards. They conducted cyclic loading tests on the proposed CFS shear walls to study the influence of stud type, sheathing material and openings. Mowrtage et al. [96] proposed shotcreted ribbed steel sheets, where they sheathed the outer side of CFS structure external walls with thin ribbed steel sheets, then shotcreted the sheets with cement or gypsum mortars. Test results, shown in Figure 2-11, indicate that the increased lateral load carrying capacities by about two times compared with the walls sheathed with traditional boards. Under ultimate loads, this new sheathing system fails in local failure modes rather than common overall buckling failure modes.
Mowrtage et al. [97] also studied the effects of some sheathing materials (trapezoidal steel sheet, steel sheet, reinforced cement board, and thin ribbed steel sheet shotcreted with cement mortar) on the cyclic lateral load behaviour of CFS walls, as well as the effect of wall foundation connection details on the lateral load response of the walls. Test results, shown in Figure 2-12, indicate that specimens sheathed with steel sheets have almost the same lateral load carrying capacities of specimens sheathed with Trapezoidal steel sheets, while the hysteresis damping is higher in the steel sheet sheathed specimens. That means corrugating the steel sheets as Trapezoidal will not
increase the strength and damping characteristics of the walls. Walls sheathed with 12 mm thick reinforced cement boards show higher lateral load carrying capacity than all the tested specimens. In a similar study, a variety of bracing methods including no bracing, bracing with cement particle board or oriental strand board, and bracing with single or double X-strap were investigated by Tian et al. [98], and the deformation behaviour and failure modes of each frame under shear were recorded. Their results indicated that frames with 2 sided X-straps had the best racking performance.

![Figure 2-12: The effects of different sheathing materials on the cyclic lateral load behaviour [97]](image)

2.4.4 The effect of fasteners

CFS shear walls may be integrally connected to frames, foundations, floors, or other walls through a variety of means including hold downs, straps, diaphragm chords and collectors. Shear walls’ lateral resistance in CFS framed buildings varies because of the randomness in the components and connections that comprise the wall [60]. The interaction between fasteners and sheathing is particularly important because sheathing-to-steel fastener response is the main source of shear wall nonlinearity, and there is high variability in this fastener response. The lateral behaviour of sheathed CFS framed shear walls also depends considerably on the complex behaviour that occurs at each fastener location. Relative motion of screw fasteners attaching the
sheathing material to the CFS framing creates local damage and can result in non-linearity at the scale of the entire shear wall. The strength and stiffness of the screw connections between sheathings and CFS framing members play a governing role in CFS wall panel performance. The connections also provide the key energy dissipating behaviour in sheathed CFS shear walls as well as bracing to the studs under gravity and out-of-plane loads.

A set of experiments were conducted by Peterman et al. [99] to characterize the hysteretic behaviour of the connection between CFS studs and sheathing when subject to in-plane lateral demands, in which the variations of sheathing configuration, fastener spacing, steel thickness, and fastener types were considered. Their results suggested that while fasteners’ spacing had no significant effect on the failure mode, steel thickness and sheathing type impacted the failure mode considerably: pull-through was dominant for OSB and bearing was dominant for GWB.

The effects of screw fasteners attaching the sheathing material to the CFS framing, as well as the impact of relative motion of these components on local damage and non-linearity of the shear wall was studied by Buonopane et al. [100]. Fulop and Dubina [101] attempted to characterize the behaviour and provide design criteria for the connections of CFS wall panels with various sheathing systems and to provide some design criteria. They concluded that, provided the failure of the bottom track in the region of the anchor bolts is prevented, the seam fasteners and the sheeting-to-frame fasteners are the components of the LSF wall panels most sensitive to damage. An experimental program with different steel thicknesses, sheathing types, fastener schedules, and stud spacing was conducted by Serrette and Nolan [102] to evaluate the performance of wood panel sheathed cold-formed steel frame shear walls attached together by one type of pneumatically driven steel pins.
Nithyadharan and Kalyanaraman [103] used a constitutive model to study the hysteretic energy dissipating behaviour of the wall panels and screw connections by capturing the deteriorating behaviour, such as the strength and stiffness degradation with severe pinching, observed in the screw connections between the CFS framing members and sheathing under cyclic loading. Fiorino et al. [104] conducted some experimental tests on the screw connections between OSB or GWB, and CFS stud profiles. They compared the response of different types of panels; studied the effect of sheathing orientation; and examined the effect of the loaded edge distance under different cyclic loading protocols and loading rates. They concluded that the sheathing type had a significant effect on the shear response of connections: OSB sheathings showed larger strength and absorbed energy, while GWB sheathings reveal larger stiffness and ductility. Fiorino et al. also characterised solutions for panel-to-CFS connections commonly used in common practice, with reference to gypsum and cement-based solutions in a recent study [105].

In an experimental and analytical study by Karabulut and Soyos [106] on CFS structures sheathed with different gypsum boards, they concluded that the main factor providing high ductility is the screws, which supply connectivity between the board and CSF framing system, and contribute to the energy absorption capacity of the walls sheathed with different configurations and types of gypsum wallboard. The shear transfer, parallel to the nearest free edge in screw connection between CFS framing member and calcium silicate board under static and cyclic loading, was studied by Nithyadharan and Kalyanaraman [107]. They investigated the effect of the edge distance of the screws and the thickness of the boards on the ultimate strength and the energy dissipated in the screw connection. Their results showed that the peak strength
and initial stiffness increased with the increase in the board thickness and the edge distance.

2.5. **CFS strap-braced wall systems**

Strap-braced walls resist lateral loads primarily through truss (axial) action by employing diagonal flat strap connected on one or both faces of the wall panel. Strap braced walls with high lateral strength can be designed by using relatively wide, thick straps and/or some specific structural shapes. The response modification factor, deflection amplification factor and over-strength factor, proposed for CFS strap-braced wall systems, differ significantly from those for CFS wood structural panel or steel sheet shear walls [3]. Also, specific limitations on the strap connections are placed by codes to ensure that net section fracture of the tension straps does not occur prior to yielding of the strap gross cross-section. Providing an appropriate load path for transferring the strap load to the supports is also vital for preventing stud-to-track connection failure.

In North America, the seismic design of strap-braced CFS shear walls used to be carried out using the AISI S213 Standard, which was recently replaced by the seismic specific standard AISI S400 [15]. Straps can be used on one or both sides of the walls. One sided strapped walls, in particular, the ones of high aspect ratio, may be subject to a significant eccentric moment in the chord studs, compression, weak-axis bending, and strong-axis bending, all of which need to include consideration of the expected strength of the strap [3]. In an effort to investigate the seismic response of strap-braced stud walls, Macillo et al. [108, 109] defined some criteria for the seismic design of strap braced CFS structures, and carried out a critical analysis of the requirements for CFS systems provided by the AISI S213, by comparing it with those given by Eurocodes for traditional braced steel frames.
Most Standards use capacity-based design procedure in which the tension-only diagonal braces are assumed to act as the inelastic fuse elements in the pin connected seismic force resisting system, while all other elements remain essentially undamaged under loading. Experimental work has shown this assumption to be valid for walls with low aspect ratios; however, the testing of high aspect ratio walls has revealed that large moments develop in the frame members, which can result in their failure prior to yielding of the braces [110].

Strap-braced walls can generally be designed using principles of mechanics. Results of an experimental research [111] however, indicates that strap-braced walls with aspect ratios over 1.9:1 may generate a significant moment in the chord studs at the strap connection location. Pastor and Ferran [112] presented a differential model of the hysteretic behaviour of unsheathed x-braced frames that considered perfect or hardening plasticity of diagonal straps under tension as well as buckling of diagonal straps under compression.

Gad et al. [21] showed that the behaviour of domestic unlined LSFs is governed by the strap bracing system, while the failure load and mechanism are governed by the type of fixity of the strap bracing to the top and bottom plates and the presence of the tensioner unit. They also showed that the initial tension in the strap braces increases the frame stiffness and consequently affects the initial dynamic characteristics of the frame. The results of an experimental study by Fiorino et al. [113] showed that the CFS strap-braced stud walls’ inelastic behaviour of structural elements can be affected by non-ductile phenomena, such as the gusset-to-track connection failure and combined bending and compression axial load failure of the chord studs, as shown in Figure 2-13, so they suggested wall corners to be carefully designed, since their behaviour could significantly affect the overall wall response.
The lateral seismic characteristics of light-weight knee-braced CFS frames were investigated by Zeynalinan and Ronagh [114, 115]. They suggested limiting the use of knee-stud bracing CFS systems to the low seismic regions; because, despite their relatively high maximum drifts, their strengths were not as high as strap bracing systems. The experimental results showed that using brackets at four interior corners of a knee-braced wall panel improved the shear strength and the panel ductility of the panel considerably. It was also seen that shorter knee-elements led to greater shear strength for the wall, but at the expense of a lower ductility. The bracing configurations and their hysteretic envelope curves are shown in Figure 2-14.
Zeynalian et al. [116] also investigated the lateral performance of K-braced CFS structures, maximum lateral load capacity, failure modes, deformation behaviour and their response modification coefficients (R factor). Comparing the results to their other experiments, they concluded that despite their relatively high maximum drifts, the strength of K-braced walls is not as high as the currently-in-use strap bracing systems. Hence, they suggest that the use of a K-stud bracing system is possible only in low seismic regions. Figure 2-15, illustrates their bracing configurations and results.

![Figure 2-15: (a) Different K-bracing configurations (b) Hysteretic envelope curves [116]](image)

In another work [117], using a non-linear FEM analysis verified based on experimental tests, the authors optimised the seismic characteristics of strap-braced CFS shear walls enhanced with brackets in the four interior corners of the wall. The performance of light-weight K-braced CFS shear panels with improved connections under cyclic loading was also studied by Pourabdollah et al. [118]. They observed that proper modification of the K-braced connections (shown in Table 2-6), and using gusset plate in the braced to stud connection, as illustrated in Figure 2-16, can significantly increase their shear strength, energy dissipation and ductility capacities in comparison to the CFS shear panels with regular connections.
Table 2-6: modification of the K-braced connections

<table>
<thead>
<tr>
<th>Configuration</th>
<th>Specimen</th>
<th>K1</th>
<th>K2</th>
<th>K3</th>
<th>K4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Description</td>
<td></td>
<td>Ordinary braced CFS</td>
<td>Change in brace configuration, double noggings and extra screws.</td>
<td>Gusset plates at the K-element to stud connections.</td>
<td>Gusset plates at connection with double chord studs.</td>
</tr>
</tbody>
</table>

Another study with rather different outcomes by Al-Kharat and Rogers [119] evaluated the inelastic performance of some CFS strap braced walls experimentally, whose results regarding the response modification coefficient suggested rather low ductility levels, in particular for heavy walls, which were not adequate to warrant the use of a seismic response modification coefficient of 4.0 in design.

Berman et al. [120] conducted a comparative study to contrast CFS braced frames and steel plate shear wall in terms of their stiffness, maximum displacement, ductility, cumulative hysteretic energy dissipation, and energy dissipation per cycle for a given strength. According to their test results, braced frames showed larger initial stiffness; steel plate shear wall with flat infill exhibited the larger ductility, while both had similar energy dissipated per cycle and cumulative energy dissipation. Trembley et al. [121] carried out an experimental study on the seismic performance of X-bracing and
single diagonal bracing systems. According to the results, for X-bracing, the effective length of the braces can be used to determine their compression strength and to characterize their hysteretic response, including energy dissipation capability.

The failure modes of different strap bracing systems and the main factors contributing to ductile response of the CFS walls were discussed by Moghimi and Ronagh [122] to ensure that the diagonal straps yield and respond plastically with a significant drift, thereby preventing any risk of brittle failures such as connection failure or column buckling. They also proposed some arrangements to provide a reliable lateral performance even in large lateral deformations by using perforated straps and/or bracket members in four corners of the wall.

There are also other innovative bracing systems proposed in the literature that have shown some advantages. The performance of an innovative system including V-braced panels under cyclic loading was investigated by Dao and Van de Lindt [123], whose results showed that the building performed well based on the ASCE Standard 41 criteria at the global level.

### 2.6. Mixed shear wall systems

It is common to use a mixed LFRS, say sheathing and strap bracing, for CFS structures. Flat strap tension bracing and sheathing martials possess high shear strength, but the use of straps plus wall panels (e.g. gypsum boards) may face some practical problems in fabrication [4]. The failure modes of some CFS strap-braced walls with different strap arrangements (shown in Table 2-7) sheathed with gypsum board were studied by Moghimi and Ronagh [124]. By investigating the main factors contributing to the ductile response of the walls, their designs aimed to ensure that the diagonal straps yield and respond plastically with a significant drift and without any risk of brittle failure, such as connection failure or stud failure. They concluded that although
reliance on gypsum board cladding alone is not a good idea, strap-braced walls without gypsum board or bracket members present severe pinching in their hysteretic loops due to plastic slack of strap braces and lack of redundancies. Their results, shown in Figure 2-17, indicated that double-side bracing does not offer a great deal of advantage over single-side bracing when a wall panel is designed to allow straps to develop their full plastic capacity. Moreover, adding brackets at four corners of the wall panel improved the lateral performance (strength, stiffness and ductility) of the wall panel considerably.

Table 2-7: Different arrangements of CFS strap-braced walls sheathed with gypsum board

<table>
<thead>
<tr>
<th>Shear wall scheme</th>
<th>Types</th>
<th>Specimen number</th>
<th>Bracing type</th>
<th>Framing and cladding</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>BA1</td>
<td>-</td>
<td>One side Gypsum board</td>
<td></td>
</tr>
<tr>
<td>I</td>
<td>AC1</td>
<td>One side strap bracing connected to interior studs and tracks</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td></td>
<td>AB1</td>
<td>One side strap bracing connected to interior studs and tracks</td>
<td>One side Gypsum board</td>
<td></td>
</tr>
<tr>
<td>II</td>
<td>CC2</td>
<td>One side strap bracing connected to strong brackets and corners frame</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td></td>
<td>CB1</td>
<td>One side strap bracing connected to strong brackets and corners frame</td>
<td>One side Gypsum board</td>
<td></td>
</tr>
<tr>
<td>III</td>
<td>DA2</td>
<td>One side strap bracing connected to frame corners</td>
<td>Back-to-back double stud at the cords</td>
<td></td>
</tr>
<tr>
<td>IV</td>
<td>DB1</td>
<td>One side strap bracing connected to interior frame corners</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>V</td>
<td>EA1</td>
<td>Strap connected to frame corner with gusset plate</td>
<td>-</td>
<td></td>
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</table>
Serrette and Ogunfunmi [4] studied the shear behaviour of typical light-gauge steel walls for three different shear resisting systems: flat strap X-bracing on the face (Type A); framed walls with single-ply gypsum wallboard on the back and single-ply gypsum sheathing board on the face (Type B); and framed walls with single-ply gypsum wallboard on the back and the face together with flat strap X-bracing on face (Type C). The results showed that the behaviour of different types of walls was governed by the yield strength of the straps with practically no resistance provided by flexure in the studs. In type B and type C tests, the measured maximum load was controlled by the breaking of the wallboard along its edges. The failure mechanism was initiated by a rotation of the screws at the edges. They also showed that in type B and type C, the use of stronger tension bracing prevented cracking of the boards at the perimeter, reduced the lateral displacement, and increased the maximum load capacity of the wall by as much as 28%. In a similar study on the behaviour and performance of the CFS framed shear walls sheathed with a composite panel which contains a 0.0686 mm steel sheet and a 15.9 mm gypsum board bonded together. Yu et al. [125] showed that the composite panel provided considerably higher shear strength than the traditional wood based sheathing. Their results also indicated that the composite panel demonstrated similar failure mechanism and post-peak behaviour as the steel sheet sheathing.
Recently, Liu et al. [126] tested sheathing with sprayed lightweight mortar for CFS walls and checked the effects of adding joint-strengthened knee elements or X-shaped steel-strap bracings. The test results showed increased strength and stiffness and restricted crack propagation. Also, knee elements or steel-strap bracings increased the load-bearing capacity and reduced the ductility of the specimens. In a similar study, a variety of bracing methods including no bracing, bracing with cement particle board or oriental strand board, and bracing with single or double X-strap were investigated by Tian et al. [98], and the deformation behaviour and failure modes of each frame under shear were recorded. Their results indicated that frames with 2 sided X-straps had the best racking performance.

2.7. CFS frame connections

CFS Special Bolted Moment Frames (SBMF) is a one-storey framing system where C-section beams are connected to hollow structural column sections by bearing-type high-strength bolts. This system is commonly used in industrial platform construction. It withstands inelastic deformations through friction and bearing at their bolted connections [3]. Seismic design provisions of CFS–SBMF in the proposed AISI Seismic Standard were developed on the basis that ductility capacity is provided through bolt slippage and bearing in bolted beam-to-column moment connections, and that beams and columns are to remain elastic at the design storey drift to resist the maximum force that can be developed in the connections [127]. The main source of energy dissipation in CFS–SBMF is the inelastic action in the form of bolt slippage and bearing in the bolted moment connection. Therefore, beams and columns are designed to remain elastic at the design storey drift for the maximum force that can be developed in the connection region [3].
Sato and Uang [127] provided background information for the development of capacity design provisions in the AISI’s first Standard for Seismic Design of CFS-SBMF, and provided an iterative flowchart to demonstrate the proposed capacity design procedure for the seismic design of CFS-SBMF. Their test results showed that a bolted connection would first slip and produce a pseudo-yield behaviour. Typical bolted moment connections together with their slip-bearing response are shown in Figure 2-18.

Sato and Uang [128] also presented the CFS-SBMF’s seismic performance factors such as response modification coefficient, deflection amplification factor, and system over-strength factor for an SBMF. They suggested a value of 3.5 for the response modification coefficient, based on the large ductility capacity observed from the cyclic testing of beam-column subassemblies. In a similar attempt, Uang et al. [129] showed that the conventional strong column-weak beam design philosophy is not appropriate for this system, and columns should be protected to remain elastic by the capacity design principles.

According to the results of an extensive experimental investigation on bolted moment connections between CFS sections, Wong and Chung [130] identified four different modes of failure: (a) bearing failure in section web around bolt hole; (b) lateral torsional buckling of gusset plate, (c) flexural failure of gusset plate; and (d) flexural
failure of connected cold-formed steel section. These failure modes are shown in Figure 2-19.

![Figure 2-19: Four different failure modes of bolted moment connections between CFS sections](image)

In another study, Chung and Wong [131] encouraged structural engineers to implement CFS structures with bolted moment connections to achieve efficient construction. They presented an investigation for predicting the structural behaviour of bolted moment connections between cold-formed steel sections and a set of design rules for section failure of connected sections under combined bending and shear was proposed.

Due to the thinness of CFS sections, the strength of the joints may often dictate the strength of a member or assembly. On the other hand, the form of typical bolted connections in cold formed steel is such that complete rigidity is difficult to achieve. Therefore, consideration of joint flexibility is of fundamental importance in the design of bolted connections for CFS structures. Zadanfarrokh, and Bryan [132] thoroughly
investigated the factors influencing the strength and rigidity of bolted connections in CFS sections and proposed design expressions for calculating the bearing strength and estimating the flexibility of such connections.

Experimental tests have previously shown that the strength of bolted moment connections between cold-formed steel members, where the connections are formed by an array of bolts in the web, is dependent on the length of the bolt group [133]. An experimental investigation on the structural performance of CFS members with bolted moment connections was conducted by Chung and Lau [134] where a number of connection configurations with both hot rolled steel and CFS gusset plates were proposed to form bolted moment connections to accommodate members in practical orientations. They identified four modes of failure: bearing failure in section web around bolt hole; lateral torsional buckling of gusset plate; flexural failure of connected member; and combined compression and bending failure of column member. The test results showed that only bearing failure is a ductile mode with large deformation capacity and the other three failure modes may cause sudden collapse.

The sway stiffness of light steel frames with flexible bolted joints was investigated by Lee et al. [135], by studying the lateral sway behaviour for light steel frames with bolted top-seat flange cleat connections, comprising slender cold-formed steel sections. Their results showed that geometrics, joint flexibility and base conditions affect the sway stiffness of light steel frame. Beam-to-column bolted connections showed 34–88% contribution to the overall frame elastic lateral stiffness for pinned bases and 17–33% for rigid base connections. A finite element model with three-dimensional solid elements was also established by Chung and Ip [136, 137] to investigate the structural performance and bearing failure of CFS bolted connections.
Various research projects have been undertaken on the CFS connection in portal frames indicating that the main factors of the connection performance are the thickness and the shape of the mild steel connection elements [138]. Lim et al, [139] conducted a set of laboratory tests on cold-formed steel portal frame buildings and showed that internal frame with roof sheeting resisted approximately three times more horizontal load than the bare frame and the deflection of the internal frame was reduced by 90% relative to the bare frame as a result of skin diaphragm action. These results are shown in Figure 2-20. They also showed that the joint flexibility of the frame has a significant effect on the load transfer between frames through the roof sheathing, and the ‘true’ loads transferred to the gable frames are between three and seven times higher than the loads derived from a tributary area.

![Figure 2-20: (a) Test set up for the building with and without roof sheathing. (b) Lateral building resistance comparison [139]](image)

The results of another study by Lim et al. [133] demonstrated that the Direct Strength Method (DSM) of the design of CFS structural members can be used to predict the strength of bolted moment connections in cold-formed steel portal frames. An extensive experimental study was conducted to investigate the structural performance of connections for cold-formed steel portal frames. Regarding the CFS portal frames connections, Mills and LaBoube [140] proposed self-drilling screw joints in order to
address some concerns about the joint designs widely used in practice, and to match
them with the moment capacity of the sections and prevent sudden failures.

Sabbagh et al. [141] conducted an investigation on the potential use of CFS sections
in moment-resisting frames for seismic applications. They proposed beam–column
connections which potentially limit the out-of-plane action of the transferred forces,
and exhibited a good ductile behaviour. They also simulated the hysteretic moment–
rotation behaviour and failure deformations of bolted CFS moment connections
[141]. They showed a type of through plate type connection between cold-formed
beams and columns can provide sufficient strength and ductility if appropriately
detailed and stiffened to allow development of plasticity in the beams. In another
attempt for the enhancement of movement resisting CFS frames, Chung and Lawson
[142] investigated the structural performance of shear resisting connections between
CFS sections where web cleats of CFS strips are used to attach beams to supporting
beams and columns. They demonstrated that CFS web cleats may be used with bolts
or self-drilling self-tapping screws as practical shear resisting connections in building
construction.

2.8. Cold formed steel standards

2.8.1 AISI standards and ASCE

The American Iron and Steel Institute, AISI, is one of the pioneering centres working
on CFS framing systems. Although the first steel design standards were written in the
1930s, they did not include CFS structures. Since 1946, the use of CFS sections in the
construction of residential houses in the United States started to grow and several
standards came into being. At first, there were different editions of the “Specification
for the Design of Cold-Formed Steel Structural Members” which were provided by the American Iron and Steel Institute. These early editions were mostly based on the research performed by Professor George Winter, who is referred to as “the father of Cold Formed Steel” at Cornell University, USA, since 1939 [143]. These standards have since been regularly revised to reflect the latest technical developments, with the most recent being the AISI STANDARD, Standard for Cold-Formed Steel Framing [144].

By the early 1990s, the CFS business faced a massive residential demand due to a shortage of timber and the availability of steel coil at relatively lower prices. The growth led to the formation of a Residential Advisory Group in 1991 to facilitate the further growth of the residential market. After a few years, in the mid-1990s, the first revision of the “Prescriptive Method for One and Two Family Dwellings” was adopted into Council of American Building Officials code. In 1996, the Residential Advisory Group was combined into the North American Steel Framing Alliance, which was later renamed as the Steel Framing Alliance in 1998. The group then started looking into the light commercial markets. Also in 1998, the American Iron and Steel Institute, AISI, established a Committee on Framing Standards with the mission to "To eliminate regulatory barriers and increase the reliability and cost competitiveness of cold-formed steel framing in residential and light commercial building construction through improved design and installation standards" [143]. Consequently, more AISI design standards were adopted in 2001, followed by regular updates until now (AISI 2004, 2007), which are [145]: North American Specification for the Design of Cold-Formed Steel Structural Members [146], General Provisions [147], Header Design [148], Lateral Design [149], Truss Design [150], Wall Stud Design [151], Prescriptive Method for One and Two Family Dwellings [152].
ASCE7 [153] stipulates that the design of lightweight cold-formed steel structures to resist seismic loads shall be in accordance with the requirements of AISI. However, it requires that for those systems (e.g. a K-braced system) which are not detailed in accordance with AISI, the designer should use the R factor designated for “Structural steel systems not specifically detailed for seismic resistance” which is equal to 3.

2.8.2 NEHRP provisions, FEMA 450 and P750

The National Earthquake Hazard Reduction Program, NEHRP, is an American entity which has published a few seismic provisions covering CFS contexts, such as FEMA 450 [154] and FEMA P750[155]. They specify that the design of cold-formed carbon or low-alloy steel members to resist seismic loads shall be in accordance with the requirements of AISI Specifications and AISI General Provisions. However, the allowable stress and load levels in AISI are incompatible with the force levels calculated in accordance with FEMA provisions. Therefore, it is essential to adjust the provisions of AISI for use with the FEMA provisions.

2.8.3 TI 809-07

Another US standard on the cold-formed steel structures is the Technical Instructions, TI 809-07 [156]. This code was originally developed for the design and construction of cold-formed steel military constructions and is used extensively by the US Army Corps of Engineers, USACE. The code is primarily based on FEMA 302 [157], though with some modifications in the design load considering over-strength of straps.

2.8.4 UBC 97 and IBC 2000

The Uniform Building Code, UBC 97, [158] and the International Building Code, IBC, [159] highlight that the design, installation and construction of CFS structural and non-structural framing shall be in accordance with AISI. Also, the R factor shall be based
on ASCE 7 for the appropriate steel systems which are designed and detailed in accordance with the provisions of AISC. Although UBC allows a maximum height of five storeys for steel stud wall systems in seismic zones, provided that they comply with some specifications, IBC limits the use of CFS systems to up to two storeys in height considering AISI provisions.

2.8.5 **Australian/New Zealand Standard, AS/NZS 4600**

This standard was developed through the cooperation of Standards Australia and Standards New Zealand, via a body called Committee BD-082, Cold-formed Steel Structures. The latest edition of Cold-formed Steel Structures, AS/NZS 4600 [18] was released in 2005. “The objective of this Standard is to provide designers of cold-formed steel structures with specifications for cold-formed steel structural members used for load-carrying purposes in buildings and other structures”. The standard has eight sections and six appendices and covers a wide range of design and testing criteria. Although the earthquake loading standard of Australia, AS1170.4 [160], does not cover cold-formed steel structures, the Australian cold-formed steel structures standard, AS/NZS 4600[18], requires that when cold-formed steel members are used as the primary earthquake resisting element, the selected response modification factor shall not be greater than 2, unless specified otherwise. However, as Australia is located in a low seismic zone, wind loads often dominate the design of low-rise cold-formed steel buildings and, therefore, such a low value for R factor does not affect designs. Little research attention has been paid to the evaluation of R factors in Australia for the same reason. The standard also sets out the maximum thickness for load-carrying CFS members as 25 mm.
2.8.6 Eurocode, Part 1.3 – Cold-formed steel

Part 1.3 of Eurocode 3 [161] is the standard that covers cold-formed steel structures. This standard has been superseded by British Standard, BS 5950-5 [162], which is called “Code of practice for design of cold formed thin gauge sections”. The code takes into account only the effective width method for design of CFS sections. Although the Eurocode and AISI provisions are similar in many cases, there are some fundamental differences between them which cause the use of Eurocode provisions for CFS section design to be more onerous than the North American Specification [163]. However, Eurocode involves a wider range of materials, and other provisions which provide richer and more sophisticated solutions for engineers.

Scrutinising the above review of the standards shows that there is no universal agreement on the value for the response modification factor. As an example, while AISI standards [146, 149, 152], NEHRP and FEMA 450 [154] specify a seismic response modification factor of 4 for diagonal strapping system, albeit with conditions attached to it, UBC [164] prescribes an R=2.8 for the same lateral resistance system and the Australian cold-formed steel structures standard, AS/NZS 4600 [18], requires that when cold-formed steel members are used as the primary earthquake resisting element, the selected response modification factor shall not be greater than 2, unless specified otherwise. Also, it is necessary to mention that there is no reference in the codes specifically for the R factor of some lateral bracing systems such as K-braced, knee-braced and fibre-cement board systems. Therefore, more studies are required in order to clarify this matter.
2.8.7 National Association of Steel-Framed Housing (NASH) standard

NASH is an industry based association focused on light steel structural framing systems for residential and commercial construction. NASH has provided education and training materials for steel frame construction industry; materials including: trainers’ syllabus and assessment manuals for teachers, training resource kit for students and suppliers, training videos for floor, wall and roof framing.

In 2005, NASH issued a comprehensive standard named “Residential and Low-rise Steel Framing”

2.9. Conclusions

The growth in the use of light steel framing in low rise building construction and the increased intention of using them in mid-rise residential construction throughout the world, have resulted in a relatively significant amount of research on the development of lateral force resisting systems using cold-formed steel members in the past decade, in order to address the need to provide structurally reliable, as well as highly economical design solutions. This chapter reviews and summarises the major research developments, and provides an updated review of references on LSF LFRSs which have appeared in the leading journals and standards in the area regarding shear walls with various face sheathings; CFS frame strap-braced wall systems; and some frame-connection systems such as special bolted moment frames. A summary of the solutions suggested for improving the lateral performance of LSF walls in the literature, are stated in Table 2-8.
Table 2-8: Solutions suggested for improving the lateral performance of LSF walls

<table>
<thead>
<tr>
<th>Solutions</th>
<th>Reference</th>
</tr>
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<tbody>
<tr>
<td>Applying double-sided sheathing or bracing</td>
<td>[68, 69, 85, 86, 122, 124, 165, 166]</td>
</tr>
<tr>
<td>Increasing the thickness of sheathing</td>
<td>[69, 70, 87, 96, 98, 102, 107]</td>
</tr>
<tr>
<td>Increasing stud thickness</td>
<td>[43, 65-67, 74, 76, 95]</td>
</tr>
<tr>
<td>Restricting buckling in wall studs</td>
<td>[70, 75, 89]</td>
</tr>
<tr>
<td>Decreasing the screw spacing in connections</td>
<td>[34, 70, 89, 106, 167]</td>
</tr>
<tr>
<td>Using coupled C section studs</td>
<td>[92, 116, 118, 122, 124, 168-170]</td>
</tr>
<tr>
<td>Changing the sheathing type. OSB; Bolivian magnesium resistance; calcium</td>
<td>[52, 61, 86, 87, 171]</td>
</tr>
<tr>
<td>silicate resistance; gypsum resistance</td>
<td></td>
</tr>
<tr>
<td>Using gusset plates in the corners</td>
<td>[116, 118, 122, 124, 168-170]</td>
</tr>
<tr>
<td>Presence of gravity axial loads on walls</td>
<td>[80, 106, 126]</td>
</tr>
<tr>
<td>Enhancing hold-down connections</td>
<td>[62, 172]</td>
</tr>
<tr>
<td>Enhancing screw types or materials</td>
<td>[64]</td>
</tr>
<tr>
<td>Adding diagonal struts inside CFS frame</td>
<td>[62]</td>
</tr>
<tr>
<td>Using adhesives in fasteners</td>
<td>[37, 63, 64]</td>
</tr>
<tr>
<td>Using low aspect ratio walls,</td>
<td>[61]</td>
</tr>
<tr>
<td>Using X strapped bracing system instead of knee elements and K elements</td>
<td>[116, 168-170]</td>
</tr>
<tr>
<td>Not using vertical or horizontal panel seam</td>
<td>[173]</td>
</tr>
<tr>
<td>Increasing the size of straps</td>
<td>[122, 124, 167]</td>
</tr>
<tr>
<td>Considering the effect of upper level weight on the walls of lower levels</td>
<td>[43, 95]</td>
</tr>
<tr>
<td>Strengthening the corners and their detailing</td>
<td>[45, 174]</td>
</tr>
<tr>
<td>Enhancing diaphragm action</td>
<td>[49]</td>
</tr>
<tr>
<td>Using corrugated steel sheathing instead of flat sheathing</td>
<td>[78]</td>
</tr>
<tr>
<td>Employing concrete filled end studs</td>
<td>[95]</td>
</tr>
<tr>
<td>Avoiding openings in the walls</td>
<td>[95]</td>
</tr>
<tr>
<td>Adding ledger tracks for attaching the interior faces of studs</td>
<td>[173]</td>
</tr>
</tbody>
</table>

A comprehensive literature review is performed as a part of this study in order to discover the existing gaps in the available knowledge regarding the structural performance of CFS structures and the possible methods of its lateral performance enhancement. It was found that although CFS walls are not new and have been used as structural and non-structural components for many years, their application as main load-bearing structural frames is relatively new, and as a result, appropriate guidelines that address the seismic design of CFS structures have not yet been fully developed. In addition, the lateral design of these systems is not adequately detailed in the
available standards of practice. There have been several attempts to improve the seismic performance of such structural systems by different bracing or sheathing configurations. However, there is minimal background available on hot rolled-cold formed steel hybrid structures which is the focus of this PhD program.
Chapter 3  Cold Formed Steel design tools- Development of a Finite Strip Method code
3.1. Design of thin elements

To design a hybrid steel structure, the most critical design criterion is the local buckling of thin walled members, which may occur at lower stress levels than the yield stress of steel. When a two-dimensional plate reaches the critical local buckling stress, it does not fail like a column member. Regarding the edge condition, the plate continues the load bearing due to stress redistribution in compression elements. This post buckling capacity of plates may be much larger than the critical local buckling capacity. According to this fact, such elements of cold formed steel structures should be designed on post buckling strength basis rather than critical local buckling stress.

There are basically two main design methods for the design of CFS members that are adopted in design standards in North American [175] and Australian/New Zealand standards [176]. These two methods are the Traditional Effective Width Method and the Direct Strength Method. Although the former is in use nearly worldwide for the design of CFS sections, the latter has been adopted only in North America and in Australia/New Zealand [177].

The basic idea for the Traditional Effective Width Method is that, compared to the CFS widths, the thicknesses of individual plate elements of CFS sections are normally small. Therefore, local buckling should occur before the yielding of the section. However, the occurrence of element local buckling does not essentially imply that the load capacity of the section has been reached. If such an element is stiffened by other elements on its edges, it will continue to resist the load and past the buckling load into a range named the Post Buckling Strength. It is expected that local buckling occurs in most CFS sections and they offer greater economy compared to heavier sections that do not buckle locally. In other words, due to the post buckling behaviour, local plate buckling causes a reduction in the effectiveness of the plates that comprise a cross
section. This reduction from the gross cross section to the effective cross section, as indicated in Figure 3-1, is fundamental to the application of the Traditional Effective Width Method. This method has some advantages. For example, it gives a clear sense of the physical phenomenon for ineffective locations in the cross section, where material does not work efficiently.

The Traditional Effective Width Method clearly illustrates the shift of the neutral axis of the section due to local buckling. It clarifies the incorporation of local to global interaction, where properties of reduced cross section affect global buckling.

On the other hand, the Traditional Effective Width Method has some disadvantages, such as:

- This method neglects interaction between the section’s flange and web for specifying the elastic buckling behaviour.

- Interaction of different buckling modes, e.g. local, distortional and global buckling, may be ignored.

- The Traditional Effective Width Method is cumbersome, as it requires substantial work to determine the effective width and to calculate the section properties and finally in design, even for common basic sections.

![Figure 3-1: The effective cross sectional area](image)

Figure 3-1: The effective cross sectional area [177]
To overcome these disadvantages, another method called the Direct Strength Method (DSM) has been introduced by Schafer and Pekoz [178]. The DSM uses elastic buckling analysis and strength curve for the whole cross section, instead of for the individual elements. The biggest advantage of the DSM is its simplicity of use, even for complicated sections.

The DSM uses finite strip analysis for elastic buckling calculation. Finite strip analysis is a special form of the finite element analysis using the displacement approach. In contrast to the standard finite element method, which uses polynomial shape functions in all directions, the finite strip method employs simple polynomials in some directions and continuously differentiable smooth series in other directions, which satisfy boundary conditions at the ends of the strips [179]. The finite strip method has a few limitations, such as the constancy of the cross section along the length. This limitation, however, only apply to a minority of cases and the majority of cases encountered can be analysed with finite strip. Due to its simplicity, this method is preferred over the finite element plate elastic buckling analysis or the manual determination of plate buckling coefficients (k’s) that are used in conjunction with the Traditional Effective Width Method.

In the finite strip method, a reference stress distribution is used for the loading of members. The reference stress is calculated under pure compression loading for finding the member’s local elastic buckling load, $P_{cr}$, and under pure bending for finding the member’s local elastic buckling moment, $M_{cr}$ (see Figure 3-2). The mode shape and the half-wavelengths of the members are used to determine the type of buckling mode, i.e. local, distortional and global.
Schafer and Adany [181-184] presented a new approach for the definition of CFS section buckling modes, including local, distortional and global buckling, which is based on mechanical behaviour of the cross section and which may easily be used in any numerical method. Firstly, they illustrated the available definition for different buckling classes, i.e. local, distortional and global. Then, using two examples, they showed that these definitions were not efficient for all different CFS sections. After that, they explained the mode definitions based on the Generalized Beam Theory and classified different criteria for different modes. Next, they tried to apply these criteria to the finite strip method. In this way, they presented the application of the proposed definitions to the FSM, and provided some numerical examples to validate the application of the proposed classification approach. They claimed that their results are efficient enough to be used in standardised calculations, and can be considered as the starting point for developing more professional design procedures.

**Local Buckling**

Local buckling occurs at half-wavelengths that are smaller than the maximum dimension of the cross section under compressive stresses. This corresponds to a local minimum in the buckling curves. Buckling modes corresponding to longer lengths may be distortional or global. The reason that half-wavelength is limited for local
buckling to a value smaller than the largest outside dimension under compressive stresses is that a simply supported plate’s local buckling under pure compression occurs in square waves, which have a half-wavelength equal to the plate width that is the largest outside dimension of the cross section. The critical half-wavelength decreases when there exists a stress gradient or a lateral restraint.

**Distortional Buckling**

Distortional buckling happens at half-wavelengths that are in between local and global buckling modes. The distortional half-wavelength is normally several times larger than the local half-wavelength and the largest dimension of the section. Loading and geometry of the section affect the buckling half-wavelengths. Distortional buckling includes both translation and rotation along the member’s length. It involves distortion of individual elements of the cross section and a rigid rotation/translation of part or whole of the section. As an example, the flanges with stiffened edge of the lipped C and Z shapes are mainly responding as one rigid piece while the web is distorting.

**Global Buckling**

Global buckling modes for a column can be flexural, torsional or coupled flexural-torsional. For a beam which is bending about its strong axis, lateral-torsional buckling is the main global buckling mode. Global buckling mode is sometimes called Euler buckling and often occurs at long half-wavelengths. Global buckling mode includes translation and rotation of the whole cross section.

### 3.1.1 Terms and definitions

There are some general terms in cold formed steel design which is useful to note:

1. **Unstiffened compression element** is a flat compression element with only one stiffened edge, parallel to the stress direction.
2. *Stiffened/partially stiffened compression element* is also a flat compression element with both edges stiffened parallel to stress direction.

3. *Multiple stiffened element* is an element with intermediate stiffeners.

4. *Flat width* \( w \), is the width of straight unbent portion of the element.

5. *Flat width to thickness ratio*

6. *Thickness* \( t \) is the thickness of base steel which is applied in cold formed design.

7. *Safety factor* \( \Omega \), *Resistance factor* \( \phi \). According to the type of structural system, different safety factors and resistance factors are applicable in design provisions.

### 3.2. Structural Behaviour

**Stiffened compression elements**

- **Yielding**
  
  If the \( \frac{w}{t} \) ratio of a compression member is relatively small, the strength of the member is governed by yielding. However, if the \( \frac{w}{t} \) ratio is larger than a specific amount, local buckling may govern the strength of the member.

- **Elastic local buckling**
  
  The governing equation for simply supported rectangular plates subjected to uniform compressive load is as following:

  \[
  D\left( \frac{\partial^4 w}{\partial x^4} + 2 \frac{\partial^2 w}{\partial x^2 \partial y^2} + \frac{\partial^4 w}{\partial y^4} \right) + N_s \frac{\partial^2 w}{\partial x^2} = 0
  \]

  where

  \[
  D = \frac{E t^3}{12(1 - \mu^2)}
  \]

  And \( w \) is the plate deflection perpendicular to surface. This deflection may be expressed as a double series (Equation 3-3) considering \( m \) and \( n \) as the number of half waves in \( x \) and \( y \) direction, respectively:
\[ W = \sum_{m=1}^{\infty} \sum_{n=1}^{\infty} A_{mn} \sin(m\pi \frac{x}{a}) \sin(n\pi \frac{y}{b}) \]  

in which “b” is the width (perpendicular to stress direction) and “a” is the length (parallel to stress direction) of the plate. This equation is satisfied by boundary conditions which is zero deflection and moments in all edges.

By substituting the corresponding derivatives into the above mentioned equation, Equation 3-4 is obtained.

\[ \sum_{m=1}^{\infty} \sum_{n=1}^{\infty} A_{mn} \left( \frac{m^4 \pi^4}{a^4} + 2 \frac{m^2 n^2 \pi^4}{a^2 b^2} - \frac{n^4 \pi^4}{b^4} \right) \sin\left(\frac{m\pi x}{a}\right) \sin\left(\frac{n\pi y}{b}\right) = 0 \]  

The non-trivial solution for Equation 3-4 is given in Equation 3-5 to 3-7:

\[ [\pi^4 \left(\frac{m^2}{a^2} + \frac{n^2}{b^2}\right)^2 - \frac{N_x m^2 \pi^2}{a^2}] \sin\left(\frac{m\pi x}{a}\right) \sin\left(\frac{n\pi y}{b}\right) = 0 \]  

\[ N_x = \frac{D a^2 \pi^2}{m^2} \left(\frac{m^2}{a^2} + \frac{n^2}{b^2}\right)^2 \]  

\[ f_{cr} = \frac{N_x}{\frac{D}{tw^2} \left[m \frac{w}{a} + \frac{n^2}{m w}\right]^2} \]  

It is obvious that \( N_x \) increases as \( n \) goes up. Therefore, \( n=1 \) is corresponding to the minimum \( N_x \) value.

\[ f_{cr} = \frac{D a^2 \pi^2}{\frac{D}{tw^2} \left[m \frac{w}{a} + \frac{1}{m w}\right]^2} = k \frac{D \pi^2}{tw^2} \]  

![Image](image.jpg)

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According to Figure 3-3, in case of the Equation 3-9 condition, the transition from $m^{th}$ to $m+1^{th}$ half sine wave takes place:

$$m \frac{w}{a} + \frac{1}{m} \frac{a}{w} = (m+1) \frac{w}{a} + \frac{1}{m+1} \frac{a}{w} \rightarrow \frac{a}{w} = \sqrt{m(m+1)} \quad 3-9$$

According to Equation 3-9, for a long plate, with a higher value of $a/w$ ratio:

$$a \approx m \quad 3-10$$

$$\lambda \approx \frac{a}{w} \approx m \quad 3-11$$

According to the Equation 3-11, it is seen that number of half sine waves is more for higher $a/w$ ratios. Furthermore, it is found that the half sine wave length, $\lambda$, is approximately equal to the width of the plate which causes the square wave formation. Figure 3-3 illustrates that for plate aspect ratios of higher than 4, to determine the critical buckling load of a simply supported plate, a value of $k = 4$ is considered which results in the Equation 3-12:

$$f_{cr} = \frac{\pi^2 E}{3(1-\mu^2)(w/t)^2} \quad 3-12$$

Table 3-1 [185] illustrates the values for $k$ in plates with higher values of $a/w$ ratio under a variety of boundary conditions.
In some cases, stress value exceeds the steel’s yield stress in only one direction. Consequently, steel becomes an orthotropic material with different modulus of elasticity in two perpendicular directions.

Differential Equation 3-13, proposed by Bleich [186], expresses the inelastic buckling of plates:

\[
\left( \frac{\partial^4 w}{\partial x^4} + 2\sqrt{\frac{\partial^4 w}{\partial x^2 \partial y^2}} + \frac{\partial^4 w}{\partial y^4} \right) + \frac{f_{ct}}{D} \frac{\partial^2 w}{\partial x^2} = 0
\]

where:

\[
f_{ct} = k \frac{\pi^2 E}{12 (1 - \nu^2)(w/t)^2}
\]
\( \tau = \frac{E}{E_t} \) and \( E_t \) is the tangent modulus of steel. Considering a modified boundary condition, the critical buckling stress associated with inelastic buckling of the plate is expressed as Equation 3-14:

\[
f_{cr} = \frac{k \pi^2 E \sqrt{\tau}}{12(1-\mu^2)(w/t)^2}
\]

Therefore, the following wavelength is obtained for a long plate:

\[
\lambda = \sqrt{\tau}w
\]

In all above-mentioned equations, \( \sqrt{\tau} \) is the reduction factor associated with plasticity for a simply supported plate under one directional uniform stress (Table 3-1: case (a)). Considering different loading and boundary conditions, this factor may be different.

- **Post buckling Strength- Effective Width Design**

Two dimensional stiffened elements will not collapse by reaching the buckling stress. Due to the stress redistribution, additional load carrying capacity is provided for such element.

Prior to buckling, when small deformations are considered, a uniform stress distribution as shown in Figure 3-4 (a) exists [185]. When buckling occurs, the centre strip transfers a portion of its pre-buckling load to the adjacent strips (Figure 3-4 (b)). This redistribution process continues until the edge strips reach the yield stress when the plate starts failing (Figure 3-4(c)).
Applying the large deformation theory, one can analyse the post buckling behaviour of a plate. Karman introduced Equation 3-16 for large deformation behaviour of plates in 1910:

\[
\frac{\partial^4 w}{\partial x^4} + 2 \frac{\partial^4 w}{\partial x^2 \partial y^2} + \frac{\partial^4 w}{\partial y^4} = \frac{t}{D} \left( \frac{\partial^2 F}{\partial y^2} \frac{\partial^2 w}{\partial x^2} - 2 \frac{\partial^2 F}{\partial x \partial y} \frac{\partial w}{\partial x} \frac{\partial w}{\partial y} + \frac{\partial^2 F}{\partial x^2} \frac{\partial^2 w}{\partial x^2} \right)
\]

Equation 3-16

and

\[
fs = \frac{\partial^2 F}{\partial y^2} \quad fy = \frac{\partial^2 F}{\partial x^2} \quad \tau_{xy} = -\frac{\partial^2 F}{\partial x \partial y}
\]

Equation 3-17

However, due to its complexity, solution of this differential equation is hardly applicable in practical design. In this regard, von Karman et al. introduced an “effective width” concept in 1932 [187]. According to this approach, an imaginary effective width \(b\) as shown in Figure 3-5 is considered to be subjected to a uniformly distributed load and the rest of the width of the plate is assumed to be unloaded. To determine the effective width \(b\), the area under the real non-uniform stress distribution curve should be equal to the sum of the rectangular areas i.e. \(b\) times the edge stress \(f_{max}\), as follows:

\[
\int_0^w f\,dx = bf_{max}
\]

Equation 3-18

![Figure 3-5: Effective width of a plate [185]](image)
In other words, the effective width \( b \) is considered as the buckling part of the plate when the yield stress of steel is reached. Thus, the effective width \( b \) for a long plate is obtained as follows:

\[
f_{cr} = F_y = \frac{\pi^2 E}{3(1 - \mu^2)(b / t)^2}
\]

Solving this equation for \( b \) results in Equation 3-20:

\[
b = Ct \sqrt{\frac{E}{F_y}} = 1.9t \sqrt{\frac{E}{F_y}}
\]

where

\[
C = \frac{\pi}{\sqrt{3(1 - \mu^2)}} = 1.9, \mu = 0.3 \quad \text{(where } \mu \text{ is the Poisson ratio)}
\]

The resulting equation for \( b \) developed in 1932 by von Karman, is the stiffened element design formula.

For \( w > b \), the following equations are obtained:

\[
f_{cr} = \frac{\pi^2 E}{3(1 - \mu^2)(w / t)^2}
\]

\[
w = Ct \sqrt{\frac{E}{f_{cr}}}
\]

Therefore, the relation between \( b \) and \( w \) is as follows

\[
\frac{b}{w} = \sqrt{\frac{f_{cr}}{F_y}}
\]

Winter [188] conducted a comprehensive research on CFS sections. According to his extensive investigations, he suggested that the proposed equation for \( b \) is also applicable to the sections in elastic range. This means that considering the maximum edge stress to be \( f_{max} \), the following equation can express \( b \):
\[ b = C_1 \frac{E}{\sqrt{f_{\text{max}}}} \quad \text{Equation 3-25} \]

### 3.3. Finite strip method

Finite element method is known as the most powerful method for accurate analysis of structures. However, in case of structures with regular sections and simple boundary conditions, this well established method is unnecessary and due to its time consuming nature, in some cases even impossible to use. Therefore, an alternative method to reduce the computational complexity and analysis run time is desirable for such structures.

The recently developed method of analysis, the finite strip method, satisfies the above mentioned requirements. This method considers two-dimensional (strips) or three-dimensional (prisms) sections having shared sides with the boundaries of the structure. This method is mostly beneficial for structures with a constant width along the length. In this regard, the width of each strip (or cross section of each prism) is considered as constant along its whole length. Figure 3-6 illustrates a diagrammatic view of finite strip method for some practical structures.

Figure 3-6: Some structures and typical mesh divisions: (a) Plate strips (b) Shell strips (c) Quadrilateral finite prisms (d) Finite layers[189]
Finite strip method is a particular form of Finite Element Method considering the displacement approach. FEM uses polynomial displacement shape functions in all directions; however, FSM applies polynomial shape functions in some directions but continuously differentiable series along other directions. The resulting displacement shape function is a product of series and polynomials. Therefore, in a strip which is reduced from a two-dimensional problem to a one dimensional problem, \( W \) is defined as Equation 3-26.

\[
w = \sum_{m=1}^{r} f_m(x)Y_m
\]

Where, \( f_m(x) \) is the polynomial expression for the \( m^{th} \) term of series and \( Y_m \) is the series satisfying the boundary conditions in \( y \) direction which specifies the displaced shape in that direction. Table 3-2 provides a comparison between FEM and FSM.
Table 3-2: Comparison between finite element and finite strip method [189]

<table>
<thead>
<tr>
<th>Finite Element</th>
<th>Finite Strip</th>
</tr>
</thead>
<tbody>
<tr>
<td>Applicable to any geometry, boundary conditions and material variation.</td>
<td>In static analysis, more often used for structures with two opposite simply supported ends and with or without intermediate elastic supports, especially for bridges. In dynamic analysis it is used for structures with all boundary conditions but without discrete supports.</td>
</tr>
<tr>
<td>Extremely versatile and powerful</td>
<td>Usually large number of equations and matrix with comparatively large bandwidth. Can be very expensive and at times impossible to work out solution because of limitation in computing facilities.</td>
</tr>
<tr>
<td></td>
<td>Usually much smaller number of equations and matrix with narrow bandwidth, especially true for problems with an opposite pair of simply supported ends. Consequently much shorter computing time for solution of comparable accuracy.</td>
</tr>
<tr>
<td></td>
<td>Large quantities of input data and easier to make mistakes. Requires automatic mesh and load generation schemes.</td>
</tr>
<tr>
<td></td>
<td>Very small amount of input data, because of the small number of mesh lines involved due to the reduction in dimensional analysis.</td>
</tr>
<tr>
<td></td>
<td>Large quantities of output because as a rule all nodal displacements and element stresses are printed. Also many lower order elements will not yield correct stresses at the nodes and stress averaging or interpolation techniques must be used in the interpretation of results.</td>
</tr>
<tr>
<td></td>
<td>Easy to specify only those locations at which displacements and stresses are required and then output accordingly</td>
</tr>
<tr>
<td></td>
<td>Requires a large amount of core and is more difficult to program. Very often, advanced techniques such as mass condensation or subspace iteration have to be resorted to for eigenvalue problems in order to reduce core requirements</td>
</tr>
<tr>
<td></td>
<td>Requires smaller amount of core and is easier to program. Because only the few lowest eigenvalues are required (in most cases anyway), the first two to three terms of the series will normally yield sufficiently accurate results. Matrix can usually be solved by standard eigenvalue subroutines.</td>
</tr>
</tbody>
</table>

The application of finite strip method requires continuum discretization so that the number of existing unknowns becomes finite in the resulting formulation. The adopted procedure is as below.

1) The continuum problem is sub-divided into individual strips. The ends of the resulting strips partly constitute the boundary conditions of the continuum.

2) It is assumed that the strips are connected along a number of nodal lines coinciding with longitudinal strip boundaries.

Nodal displacement parameters (degrees of freedom at each nodal line), are normally associated with displacements and rotations (first derivative of displacements) consid-
ering the x polynomial variable in transverse direction. Furthermore, they can also refer to non-displacement terms like strains (covering shear strains, direct strains, bending and twisting curvatures).

Since continuous functions are applied in longitudinal direction, comparing with an element node, a strip nodal line contributes to less DOFs. For example the DOFs associated with an element node are \( w, \theta_x \) and \( \theta_y \), while at each strip line, \( w \) and \( \theta_x \) are the only existing DOFs (Figure 3-7).

![Figure 3-7](image)

**Figure 3-7- Coordinates, Degree of Freedom, and loads on a typical strip**

3) To introduce displacements, and consequently the strain and stress fields within each element, a displacement function according to nodal displacement parameters is chosen.

4) Having chosen the displacement function, one can obtain a stiffness matrix and load matrices equilibrating various load types acting on the strip by applying either minimum potential energy or virtual work principles.

5) By assembling the stiffness and load matrices of all contributing strips, the overall stiffness equations are formed.
3.3.1 Displacement function assumption

According to previous discussion, it is revealed that displacement function selection for a strip is the most critical step of analysis. An improper choice of shape function not only causes erroneous results but may also lead to convergence of wrong results with successively refined meshes.

The following simple rules should be considered to ensure the answers converge to correct values.

1) The series \( Y_m \) contributing to displacement function should basically satisfy the end conditions of the strip.

2) The constant strain state along the transverse direction must be presented by \( f_m(x) \) as the polynomial term of the displacement function. If this is not considered, then the strain is not necessarily converging to the correct strain distribution by subdividing the mesh to smaller elements.

3) The displacement compatibility along the boundaries of neighbouring strips must be satisfied by the displacement function. In this regard, besides displacement values, the first partial derivative continuity must be included as well.

3.3.2 A review on finite strip method [190]

The concept of finite strip method was basically introduced in late 1960s. A product of trigonometrical or hyperbolic series and a polynomial represents the displacements of a conventional strip.

In 1996, Cocchi applied trigonometrical-hyperbolic functions to represent the displacements of plates with different boundary conditions. He found that for all support conditions the orthogonality condition was satisfied; therefore, a great improvement in computational efficiency was achieved [191].
In 1997, Gagnon et al. applied spline finite strip method for rectangular plate analysis. They used spline functions of linear, quadratic and cubic order for longitudinal interpolation. The proposed formulation is applicable to any boundary condition and covers all support types. The other advantage of their proposed method is validity of the method where plate thickness varies in different directions. They compared obtained results of clamped-free case for thick plates with the results derived by two-dimensional finite element analysis and found that they are in a good agreement [192].

By applying natural shape functions, Kong et al. tried to achieve a better convergence. The natural shape functions were derived by forth order differential equation solution [193]. Applying the determined shape functions, they conducted the solution process in a conventional way. The proposed strips overcame some of difficulties associated with standard strips including constant strains, rigid body modes and zero energy modes.

In 1996, by applying the U transformation, Li et al uncoupled the finite strip equations leading to a simplified single strip equivalent problem [194].

By dividing each strip into smaller intervals, Zhong et al conducted a precise integration method to develop the shape functions [195]. They found that the approach can deal with effects of supports and concentrated loads with an improved accuracy.

3.4. Developing a FSM code for DSM design of CFS members

DSM uses the buckling loads obtained from FSM analysis of CFS sections, as explained earlier in this chapter. To be able to design the cold formed studs with DSM, an engineer requires the values of local, distortional and global buckling loads. A computer code in MATLAB software is developed to calculate the buckling loads for every corresponding half wavelength of a particular section. The FSM code applies to any
boundary condition at the two ends of the CFS column, namely: pin-pin, fixed-fixed, fixed-pin, fixed-free. The solution is based on the shape functions of the work done by Bradford and Azhari [196]. Later in Chapter 5 of this thesis the provided code is verified against shell finite element analysis implemented in ABAQUS [1] and the well-known FSM tool, CUFSM [180]. The developed FSM code is to be combined with an optimisation algorithm to determine optimum CFS sections according to various criteria. The optimisation process is not included in this thesis.

3.4.1 Degree of freedom and shape functions

Unlike the finite element method which discretises the section in both longitudinal and transverse directions, finite strip method assumes a shape function to represent the longitudinal displacement field only. Three translations of $U$, $V$, $W$ and a rotation ($\theta$) are considered as global displacements. Local displacements are associated with the deformation of a strip, three translations of $u$, $v$, $w$ and a rotation ($\theta$).

The membrane (in-plane) shape functions are assumed to be identical linear polynomial in transverse direction for all boundary conditions. On the other hand, in the longitudinal direction, regarding the pre-set boundary conditions, trigonometric functions are selected. A cubic polynomial shape function is set for the out of plane displacement in transverse direction for all boundary conditions while for longitudinal direction, trigonometric functions are assumed to correspond to the pre-set boundary condition of loaded edges. The definitions of the general displacements, $u$, $v$ and $w$, are addressed in terms of the displacement at the nodes and selected shape functions as below:

$$u = \sum_{p=1}^{m} \left(1 - \frac{x}{b} \right) b \left[ u_{1p} \right] Y_p$$  \hspace{1cm} 3-27

$$v = \sum_{p=1}^{m} \left(1 - \frac{x}{b} \right) b \left[ v_{1p} \right] Y_p' \frac{a}{\mu_p}$$  \hspace{1cm} 3-28
\[ w = \sum_{p=1}^{m} \left[ 1 - \frac{3x^2}{b^2} + \frac{2x^3}{b^3} \right] x(1 - \frac{2x}{b} + \frac{x^2}{b^2}) \left( \frac{3x^2}{b^2} - \frac{2x^3}{b^3} \right) x(\frac{x^2}{b^2} - \frac{x}{b}) \right] \{ w_{1p} \} \{ \theta_{1p} \} \{ w_{2p} \} \{ \theta_{2p} \} Y_p \]  

3-29

Where ‘a’ is the strip length, ‘b’ is the strip width, \( \mu_{p=p\pi} \), ‘p’ is the half-wave number, ‘m’ is the maximum half-wave number considered, \( Y_m \) is the shape function in longitudinal direction representing the longitudinal displacement field [196]. In addition, for different boundary conditions, the shape functions are given in Table 3-3.

<table>
<thead>
<tr>
<th>( Y_p )</th>
<th>Boundary condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \sin \frac{p\pi y}{a} )</td>
<td>Simple-Simple (S-S)</td>
</tr>
<tr>
<td>( \sin \frac{p\pi y}{a} \sin \frac{\pi y}{a} )</td>
<td>Clamped-Clamped (C-C)</td>
</tr>
<tr>
<td>( \sin \frac{(p + 1)\pi y}{a} + \left( \frac{p + 1}{p} \right) \sin \frac{p\pi y}{a} )</td>
<td>Simple-Clamped (S-C)</td>
</tr>
<tr>
<td>( 1 - \cos \frac{(p - \frac{1}{2})\pi y}{a} )</td>
<td>Clamped-Free (C-F)</td>
</tr>
</tbody>
</table>

The general vector shown in Equation 3-30 comprises the expressions of \( u \), \( v \) and \( w \).

\[
\begin{bmatrix}
{u} \\
{v}
\end{bmatrix} = \sum_{p=1}^{m} [N_{uv}] \begin{bmatrix}
{u}_{1p} \\
{v}_{1p} \\
{u}_{2p} \\
{v}_{2p}
\end{bmatrix} = \sum_{p=1}^{m} [N_{uv}] d_{uv}^p
\]

\[
w = \sum_{p=1}^{m} [N_{w}] \begin{bmatrix}
{w}_{1p} \\
{\theta}_{1p} \\
{w}_{2p} \\
{\theta}_{2p}
\end{bmatrix} = \sum_{p=1}^{m} [N_{w}] d_w^p
\]

3.4.2 Elastic stiffness matrix (k)

For a section with constant thickness of \( t \), applying the membrane constitutive relation, \( \{ \sigma_M \} = [D_M] \{ \varepsilon_M \} \), and stress-generalised strain relation, \( \{ \sigma_B \}_e = [D_B] \{ \varepsilon_B \}_e \), the internal strain can be written as following:
\[ U = \frac{1}{2} t \int_{a}^{b} (\varepsilon_M)^T [D_M] (\varepsilon_M) dxdy + \frac{1}{2} t \int_{a}^{b} (\varepsilon_B)^T [D_B] (\varepsilon_B) e dxdy \]  

Where \( [D_M] = \begin{bmatrix} E_1 & v_\gamma E_2 & 0 \\ v_\gamma E_1 & E_2 & 0 \\ 0 & 0 & G \end{bmatrix} \) and \( [D_B] = \begin{bmatrix} D_x & D_1 & 0 \\ D_1 & D_y & 0 \\ 0 & 0 & D_{xy} \end{bmatrix} \) with \( E_1 = \frac{E_x}{1-\nu_{xy}} \).

\[ E_2 = \frac{E_y}{1-\nu_{xy}}, \quad D_x = \frac{E_x t^3}{12(1-\nu_{xy})}, \quad D_y = \frac{E_y t^3}{12(1-\nu_{xy})}, \quad D_1 = \frac{v_\gamma E_x t^3}{12(1-\nu_{xy})} = \frac{v_x E_y t^3}{12(1-\nu_{xy})} \] and \( D_{xy} = \frac{G t^3}{12} \).

Therefore, the elastic stiffness matrix can be readily derived from the short statement of the internal strain energy such that:

\[ U = \sum_{p=1}^{m} \sum_{q=1}^{m} \frac{1}{2} d_p^T k_{eq} d_q \]  

Where \( d_p = \begin{bmatrix} d_{u \nu}^p \\ d_{w}^p \end{bmatrix} \), \( d_q = \begin{bmatrix} d_{u \nu}^q \\ d_{w}^q \end{bmatrix} \) and \( k_{eq} \) is the elastic stiffness matrix for the half-wave numbers \( p \) and \( q \) and can be separated for membrane and bending.

\[ k_{eq} = \begin{bmatrix} k_{eM}^{pq} & 0 \\ 0 & k_{EB}^{pq} \end{bmatrix} \]

\[ \begin{cases} 
  k_{eM}^{pq} = t \int_{a}^{b} (B_M^p)^T [D_M] B_M^p dxdy \\
  k_{eB}^{pq} = t \int_{a}^{b} (B_B^p)^T [D_B] B_B^p dxdy 
\end{cases} \]

Therefore, the elastic stiffness can be expressed as follows:

\[ k = \begin{bmatrix} k_{e}^{pq} \end{bmatrix}_{m \times m} \]  

By substitution and integration, the following expressions for the membrane and bending elastic stiffness matrices are obtained:
\[ k_{eM}^{pq} = t \left[ \begin{array}{ccc}
\frac{E_1 I_1}{b} + \frac{Gb_5}{3} & -\frac{E_2 v I_3}{2c_2} & -\frac{E_1 I_1}{b} + \frac{Gb_5}{6} & -\frac{E_2 v I_3}{2c_2} + \frac{Gl_5}{2c_2} \\
-\frac{E_2 v I_2}{2c_1} & \frac{E_2 b I_4}{2c_1} + \frac{Gl_5}{3c_1 c_2} & -\frac{E_2 v I_2}{2c_1} - \frac{Gl_5}{2c_1} & \frac{E_2 b I_4}{2c_1} + \frac{Gl_5}{bc_1 c_2} \\
-\frac{E_2 v I_3}{2c_4} + \frac{Gb_5}{2c_1} & \frac{E_2 v I_3}{6c_1 c_2} + \frac{Gl_5}{bc_1 c_2} & -\frac{E_2 v I_3}{2c_4} + \frac{Gl_5}{2c_1} & \frac{E_2 b I_4}{3c_1 c_2} + \frac{Gl_5}{bc_1 c_2} \\
-\frac{E_2 v I_2}{2c_1} + \frac{Gb_5}{2c_1} & \frac{E_2 b I_4}{6c_1 c_2} + \frac{Gl_5}{bc_1 c_2} & -\frac{E_2 v I_2}{2c_1} + \frac{Gb_5}{2c_1} & \frac{E_2 b I_4}{bc_1 c_2} + \frac{Gl_5}{bc_1 c_2}
\end{array} \right] 
\]

3-36

Where \( c_1 = \frac{pr}{a} \), \( c_2 = \frac{qr}{a} \); \( I_1 = \int_0^a Y_p Y_q dy \); \( I_2 = \int_0^a Y''_p Y_q dy \); \( I_3 = \int_0^a Y''_p Y_q dy \); \( I_4 = \int_0^a Y''_p Y_q dy \); \( I_5 = \int_0^a Y'_p Y'_q dy \).
\[ k_{eq}^b = \frac{1}{420b^3} \begin{bmatrix}
5040D_1J_1 - 504b^2D_2I_2 \\
-504b^2D_1J_3 + 156b^4D_4I_4 \\
+2016b^2D_xI_5 \\
(2520D_1J_1 - 462b^3D_4I_2) \\
-42b^3D_1J_3 + 22b^5D_yI_4 \\
+168b^3D_{xy}I_5 \\
-2520D_1J_4 + 462b^3D_4I_2 \\
-504b^2D_1J_3 + 13b^5D_yI_4 \\
-2016b^2D_{xy}I_5 \\
-504D_1J_1 + 504b^2D_2I_2 \\
+504b^2D_1J_3 + 54b^4D_yI_4 \\
+168b^3D_{xy}I_5 \\
-2520bD_1J_1 + 42b^3D_4I_2 \\
+24b^3D_1J_3 + 13b^5D_yI_4 \\
-168b^3D_{xy}I_5 \\
2520D_1J_4 - 42b^3D_4I_2 \\
-42b^3D_1J_3 - 13b^5D_yI_4 \\
+168b^3D_{xy}I_5 \\
\end{bmatrix} \]

Where \( I_1 = \int_0^a Y_p Y_q dy; \ I_2 = \int_0^a Y_{p} Y_{q} dy; \ I_3 = \int_0^a Y_{p} Y_{q} dy; \ I_4 = \int_0^a Y_{p} Y_{q} dy; \ I_5 = \int_0^a Y_{p} Y_{q} dy. \)
3.4.3 Geometric stiffness matrix ($k_g$)

Considering the linearly varying edge traction loads ($T_1, T_2$) to be applied to the edges of the strip; two methods are applicable to determine the geometric stiffness matrix; in terms of additional potential energy due to in-plane forces ($T_1, T_2$) or by considering higher order strain. The former is the method used in this section. The potential energy $V_p$ associated with $T_1$ and $T_2$ during the buckling is expressed as [181, 196]:

$$V_p = \frac{1}{2} \int_0^a \int_0^b \left[ (T_1 - (T_1 - T_2)) \left( \frac{\partial u}{\partial y} \right)^2 + \left( \frac{\partial v}{\partial y} \right)^2 + \left( \frac{\partial w}{\partial y} \right)^2 \right] dx dy$$

The displacement derivatives can be rewritten as appropriate derivatives of shape functions, $N_u$, $N_v$, and nodal displacements. Therefore, the potential energy can be written as:

$$V_p = \frac{1}{2} \int_0^a \int_0^b \left( T_1 - (T_1 - T_2) \right) \left[ \sum_{p=1}^m \sum_{q=1}^m d_p^T [G_p]^T [G_q] d_q \right] dx dy$$

Where $[G_p] = \begin{bmatrix} G_p^M & 0 \\ 0 & G_p^B \end{bmatrix}$ and $[G_q] = \begin{bmatrix} G_q^M & 0 \\ 0 & G_q^B \end{bmatrix}$. Regarding the half-wave numbers $p$ and $q$, $k_g^{pq}$ is broken into membrane, $k_g^{pq}_M$, and bending, $k_g^{pq}_B$, portions. The following equations express the membrane, $k_g^{pq}_M$, and bending, $k_g^{pq}_B$, geometric stiffness matrices corresponding to half-wave numbers $p$ and $q$:

$$k_g^{pq} = \begin{bmatrix} \frac{(3T_1+T_2)bl_5}{12} & 0 & \frac{(T_1+T_2)bl_5}{12} & 0 \\ \frac{(3T_1+T_2)ba^2I_4}{12\mu_p\mu_q} & 0 & \frac{(T_1+T_2)ba^2I_4}{12\mu_p\mu_q} & 0 \\ \frac{(T_1+T_2)bl_5}{12} & 0 & \frac{(T_1+T_2)bl_5}{12} & 0 \\ \frac{(3T_1+T_2)ba^2I_4}{12\mu_p\mu_q} & 0 & \frac{(T_1+T_2)ba^2I_4}{12\mu_p\mu_q} & 0 \end{bmatrix}$$

**Symmetric**
\[
\begin{bmatrix}
(10T_1 + 3T_2)bl_5 & (15T_1 + 7T_2)b^2l_5 & 9(T_1 + T_2)bl_5 & 7(T_1 + 6T_2)b^2l_5 \\
35 & 420 & 140 & 140 \\
(5T_1 + 3T_2)b^3l_5 & (6T_1 + 7T_2)b^2l_5 & (3T_1 + 10T_2)bl_5 & (7T_1 + 15T_2)b^2l_5 \\
840 & 420 & 35 & 280 \\
(3T_1 + 5T_2)b^3l_5 & & & 420 \\
& & & 840 
\end{bmatrix}
\]

Symmetric

Where \( \mu_p = p\pi; \mu_p = q\pi; \)

\[
I_1 = \int_0^a Y_p Y_q dy; I_2 = \int_0^a Y''_p Y_q dy; I_3 = \int_0^a Y_p Y''_q dy; I_4 = \int_0^a Y''_p Y''_q dy; I_5 = \int_0^a Y'_p Y'_q dy.
\]

### 3.4.4 Assembly of the global stiffness matrices and finite strip solution

As previously mentioned, the discretisation only applies to cross section forming strip elements along the length of the section. Having assigned the boundary condition for every individual strip, a transformation matrix is required to transform the local stiffness matrices to global ones. Thus, by summation of global stiffness and geometric matrices the global elastic, \( K_e \), and geometric, \( K_g \), stiffness matrices can be assembled and by solving the following typical eigenvalue problem, the elastic buckling loads can be determined as the diagonal matrix of eigenvalues (\( \Lambda \)):

\[ K_e \Phi = \Lambda K_g \Phi \]

Where \( \Phi \) is the matrix with corresponding Eigen modes (or buckling modes).

### 3.4.5 Structure of the code

A free shape thin walled section with uniform wall thickness in x-y plane can be introduced to the code. A Node/Coordinates matrix defines the nodes and their coordinates in x-y plane. The section is considered to be a cold formed open section which means that there is one start node and one end node and every node is connected only
to the two adjacent nodes and the path does not self-cross under any condition, Figure 3-8. The number of strips is equal to the number of nodes minus one and the strip width, b, is calculated according to the nodal coordinates.

The required input data for the code is the material properties (yield stress, modulus of elasticity and Poisson’s ratio), Node/Coordinates matrix, half-wave lengths to be considered, matrix of connectivity which determines the order of nodal connections and the thickness of the cross section. The requested output is the local, distortional and global buckling loads and finally the code computes the nominal buckling loads using the Direct Strength Method (DSM). The calculation details are further discussed in Chapter 5 of this thesis.

3.5. Conclusions

DSM is a powerful tool to design a CFS section. The section buckling loads (local, global, and distortional) are required for calculation of nominal buckling loads using
DSM. To evaluate the section buckling loads, FSM is applied and a code is developed in MATLAB software for this purpose. The results of the developed code is verified using the well-known FSM code, CUFSM [180]. The developed FSM code is used later in Chapter 5 of this thesis to design the CFS studs of the 4-storey building.
Chapter 4  Experimental investigation on the Lateral Behaviour of a Hybrid Cold-Formed and Hot-Rolled Steel Wall System
4.1. Introduction

During the past few decades, the use of cold-formed steel (CFS) frames as the main load bearing system of low to mid-rise structures has become a common practice. Due to its light weight, construction flexibility, prefabrication options and ease of installations, in comparison with hot-rolled steel frame; this structural system is becoming a popular option for residential construction [197, 198]. However, unlike hot-rolled steel structures, it is well recognised that the implication of CFS for lateral bearing systems has been challenging. Low rigidity of CFS sections alongside partially restrained screw or rivet fasteners leads to limited or no lateral resistance for CFS frames [199]. There have been various efforts to combine other structural systems with CFS frames to improve its seismic performance and remedy the existing deficiencies.

Relying on face sheathings is the first common approach to improve the lateral load performance of CFS wall systems. Face sheathing elements such as steel, plywood, oriented strand board (OSB) and gypsum wall board (GWB) are the most popular bracing elements being evaluated to improve the lateral behaviour of CFS frames [50, 51, 54, 165, 200]. The second approach being developed to improve the CFS lateral capacity is to apply strap bracings through lateral load bearing spans. Different configurations of strap bracings such as K bracing, knee bracing or diagonal bracing have been considered in a number of research studies [114, 116, 170].

Mixed shear walls are the next alternative for CFS lateral load resisting systems. A combination of face sheathing panels with strap bracings have been investigated by Moghimi and Ronagh [124]. They evaluated the lateral behaviour of strap braced walls with and without gypsum boards and brackets and concluded that adding brackets at the corners rectifies the lateral performance. They also showed that double-sided bracing does not show any further improvement to the overall behaviour unless straps
are prevented from developing full plastic capacity. Mixed shear wall system can also include face sheathing boards accompanied by sprayed light-weight mortar to act as bracing element[126].

The primary aim of using LSF systems is to minimize the amount of labour and material resources to reduce construction costs and time. One way to minimize labour and the duration of constructional process is panelisation, in which panels are common elements containing tracks and studs. Assembly is done in a controlled interior environment with higher quality control; repetitive and efficient assembly, and reduced erection time are some advantages of such panelised systems [201]. Although the CFS structural wall panels are lightweight and easy to handle, their behaviour as a structural element is still not reliable enough to justify their application as the sole load resisting system for mid- to high-rise construction [114].

In earthquake-prone regions, CFS structures are expected to withstand lateral loads, during seismic events. In the current literature, CFS shear walls with various face sheathings (wood, steel and other materials) and strap braced wall systems or a mix of both are experimented as effective lateral load resisting systems for CFS structures. Regarding seismic design of bare LSF shear walls/panels however, where the effects of sheathing are not considered, strap bracing is the most common system being used for resisting lateral loads. The results of studies have shown that strap braced walls often have large residual displacements, which could be undesirable due to permanent deformation resulting from severe damage and an inability to re-centre [28]. Such large residual displacements and very probable slacks in the wall and rather poor energy dissipation during cyclic loading make the existing strap bracing systems quite ineffective in earthquake-prone regions [114, 116, 124, 170].
Hot-rolled steel frames on the other hand, are reliable lateral load resisting systems, supported by a wide range of studies on their seismic behaviour in low- to mid-rise structures. Therefore, hybrid shear wall panels including CFS and hot-rolled steel to accommodate the advantages of both structural systems are an interesting field for investigating the potential of CFS structures for mid- to high-rise construction.

In the current research, a hybrid wall panel (HWP) system is introduced which consists of a hot-rolled steel framed panel laterally connected to a CFS panel. The CFS panel transfers its share of lateral load to the hot-rolled panel, while the hot-rolled panel is responsible to resist the transferred lateral load. The proposed panel provides the advantages of a light-weight structural system as well as the reliability of a hot-rolled steel structural frame.

The following chapter provides the results of experimental and numerical studies on lateral behaviour of the proposed hybrid wall panel systems (HWPSs) and investigates the possibility of any further improvements in the system. Each hybrid panel consists of a hot rolled steel frame laterally attached to a cold formed panel. The specimens are studied under monotonic and cyclic loads and enhancing changes are considered according to the results.

4.2. Testing rig design

Due to increased need for testing various types of panels with extended heights under multiple load directions, fabrication of a panel testing rig became a necessity in Centre for Infrastructure Engineering (CIE) Structural laboratory. Some pioneering wall panel industries are currently considering two storey wall panels (up to 6 meters height) which need to be structurally tested under probable load conditions. The proposed panel in this research is of 3 meter height, however due to two reasons the rig is designed to support assemblies up to 6 meters: firstly, to apply vertical loads
simultaneously with horizontal load, a 1 meter gap is required for assembling the vertical hydraulic jacks; secondly, this rig is meant to be utilized for other potential and extended panels in future. The vertical load which represents the share of gravity load borne by the panel should be well distributed along the length. Each part resists the gravity load according to its portion of length which would be 2/3 for the cold formed steel part and 1/3 for the hot rolled steel part. Three lateral frames are used to mount three vertical load cells each of them applying 22.5 kN load downward.

In addition to vertical load, the top chord of the panel is under push, pull or cyclic load. Therefore, a mechanism is required to let the top chord move in the direction of horizontal load while the vertical load is being applied. In this regard, a top beam is designed to distribute the three point loads to the top chord and each load cell is equipped with a roller to facilitate the horizontal displacement. The distribution method of horizontal load among the wall elements depends on the selected top beam.

For the panels with simple HRS frame (HWPS and IHWPS), the top beam is a CFS section; while for the panels with laterally stiffened HRS frame (SHWPS), a HRS section is acting as the top beam.

As previously explained, the testing rig might be used for other tests with different load protocols in the future, therefore, the whole frame (beam, column and connections) are designed for a 30 kN vertical load to be applied either upward or downward. The stress and strain distribution in each lateral frame is shown in Figure 4-1.
4.2.1 Components of the testing rig

Lateral frames: as previously mentioned, three lateral frames are designed to mount the hydraulic jacks responsible for vertical loads. Each frame is composed of two columns, one beam, one jack mounting plate and a base connection setup (Figure 4-3).

Longitudinal frame: An inclined frame to mount the horizontal hydraulic jack in the wall panel longitudinal direction (Figure 4-3).

Strong floor: These frames are complemented and supported by multi-configurable strong floor of 16m x 8m.

Top beam: Top beam is a hot rolled channel section covering the top chord for the following reasons:

- Proper distribution of vertical load along the length of the top chord
- Distributing the horizontal load proportionally to bearing elements
- Facilitating horizontal displacement for the panel while being vertically loaded.

Bottom beam and clamps: Bottom beam is a hot rolled channel to prevent any out of plane displacement at the bottom of the panel. In addition, the bottom chord is clamped to the strong floor using the bottom chord to prevent any possible uplift.
Lateral struts with rollers: These struts are keeping the top chord on the right track and prevent any overall out of plane deformation. A horizontal roller is attached to the tip of these struts to allow horizontal displacement.

Hydraulic jacks: A hydraulic jack with the capacity of 1000 kN, made by HESCO is installed to apply horizontal load. Three similar hydraulic jacks are used for vertical load. Each jack has a roller mounted on the loading tip to allow for horizontal load.

The final fabrication drawings and specimen setup are illustrated in Figure 4-3.

4.3. Panelised hybrid cold-formed-hot-rolled steel system

There have been a great deal of research studies on the lateral load resisting capacity of LSF systems [202]. Different factors such as sheathing properties [106], framing details, fastener types and spacing [171], geometry and construction approach might be considered as the main contributing factors. In a CFS structural system, the structural lateral performance is affected by both horizontal and vertical elements as well as connections.

The proposed prefabricated hybrid panel here is formed of two individual panels: a hot-rolled steel portal panel made of square hollow sections (SHS) and a CFS panel made of top and bottom chords and studs. The panels can be transported to construction site separately and assembled on site using the same fastener options as for pure CFS systems. The weight and size of the panels are kept in a range that can be safely handled i.e. lifted, installed, transported and assembled by two workers. The length of the CFS part of the panel can vary according to the architectural demand, while the hot-rolled part maintains the same size according to the amount of shear force required to be resisted.
In case of lateral excitation, the CFS parts of the HWPS carry the vertical load while the lateral load is primarily resisted by the hot-rolled panels. Screw connections provide the energy dissipation through hysteresis, essentially combining to make up the total panel hysteresis. The hot-rolled panel behaviour, therefore, governs the seismic behaviour of the HWPS.

As shown in Figure 4-2, the hot-rolled frame is designed to carry the lateral load both in tension and compression, while one side of the hot-rolled panel is connected to the CFS panel. This design allows the hot-rolled panel to dissipate energy for the entire HWPS. The panel is designed to only allow hot-rolled part to take the lateral loads and this provides relatively small residual displacements under lateral cyclic loading. Reversed cyclic tests were performed to aid in understanding the behaviour of the hybrid panel to provide accurate modelling of its hysteresis for integration at a structural system, such as a building level. In this chapter, the behaviour of bare panels
is investigated and the effect of sheathing elements on the lateral behaviour of the wall is not considered.

4.4. First set of tests arrangement and specimen details

The experiments are based on two sets of tests. For the first set, a lateral load is applied to the specimen and the effect of vertical gravity loads is not considered. The purpose of this experiment is to evaluate and improve the capacity of cold formed steel top chord to act as a collector. The second set of tests accounts for both lateral and vertical loads in which the cold formed part does not play a role in transferring the lateral load since a relatively rigid hot rolled top chord is acting as a collector. The latter set of tests aims to evaluate the hysteretic response of the hybrid panel with cold formed steel bearing a major portion of vertical load and hot rolled steel acting as the only lateral load resisting system. The first set of specimens is investigated here in Chapter 4 while the second set is explained in Chapter 5 of this thesis.

The lateral behaviour of the proposed HWPS is investigated using full-scale physical experiments. Hex flange head self-drilling screws of 12 gauge diameter with 14 thread/inch are used as fastener element for the CFS parts and also between cold-formed and hot-rolled steel parts. Lab View Signal Express software [84] is used to analyse and transfer the data obtained from LVDTs and load-cells. The experimental program was conducted in Structural Laboratory of Centre for Infrastructural Engineering of Western Sydney University using the specifically designed testing rig illustrated in Figure 4-3.
4.4.1 Specimen fabrication and material

The specimens were designed to accommodate the panelised system characteristics; each HWPS is of 3.6m width (2.4 m cold-formed, and 1.2m hot-rolled frame) and 3m height as shown in Figure 4-2. The hot-rolled profile is made of a rectangular hollow section (SHS89x89x3.5). In Australia, small hollow sections are usually made through cold forming process and welded to form a closed section. However, since the thickness of the used profile is more than 3mm and it complies with AS 1163, it can be treated as a Hot Rolled profile based on clause 1.1.1 of AS 4100. Therefore, in this thesis regardless of manufacturing process, the profile is referred to as Hot- Rolled Section rather than Cold- Formed section. The cold-formed studs are WSL92-075-30 C sections and the bottom chord is a 94-055-30 C channel. The top chord is not chosen from currently existing products since it should accommodate enough space for a higher number of screws with a wider flange, in case any improvements are required. Therefore, a 94-075-100 C channel was bent out of a CFS coil for this purpose. Two rows of noggins were connected to one-third (1/3) and two-third (2/3) of the studs’ height on both sides to reduce the free buckling length of the studs to 1m.
The material properties for both hot-rolled and CFS were provided by the manufacturer. However, 3 coupons were tested for each element of the wall to verify the material properties to be utilised for numerical simulation. Figure 4-5 illustrates the coupon test results for hot-rolled and CFS materials.

Four Linear Variable Differential Transformers (LVDTs) of 170 mm capacity were attached to measure the lateral displacements at top left, top right, mid height left and mid height right of the specimen (Figure 4-2-LVDTs A to D, respectively). The lateral load and lateral displacement were also measured via the actuator’s load cell.

As shown in Table 4-1, a total of 5 types of specimens were tested in the first part of this study. Firstly, the monotonic and cyclic behaviour of the hot-rolled steel part of the panel was investigated (HWPS-I and HWPS-II). Then, the maximum capacity of the HWPS was evaluated through monotonic tests under push and pull loading protocol (HWPS-III and HWPS-IV), and the maximum capacity of the HWPS was improved according to the results of specimens III and IV. Finally, the improved HWPS was tested under cyclic load to study the hysteretic behaviour (IHWPS).
Figure 4-4- Hold-down devices: a) top chord bolted to top beam under cyclic load b) Hot-rolled steel frame fixed at both ends to top beam under cyclic load, C&D) Hot-rolled steel frame and bottom chord clamped to strong floor
Table 4-1: Specimens and loading protocol

<table>
<thead>
<tr>
<th>Specimen</th>
<th>HWPS-I</th>
<th>HWPS-II</th>
<th>HWPS-III</th>
<th>HWPS-IV</th>
<th>IHWPS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Title</td>
<td>Hot-rolled panel</td>
<td>Hybrid wall panel</td>
<td>Improved hybrid wall panel</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Schematic</td>
<td><img src="image1" alt="Schematic HWPS-I" /></td>
<td><img src="image2" alt="Schematic HWPS-II" /></td>
<td><img src="image3" alt="Schematic HWPS-III" /></td>
<td><img src="image4" alt="Schematic HWPS-IV" /></td>
<td><img src="image5" alt="Schematic IHWPS" /></td>
</tr>
<tr>
<td>Load protocol</td>
<td>Push</td>
<td>Cyclic</td>
<td>Push</td>
<td>Pull</td>
<td>Cyclic</td>
</tr>
</tbody>
</table>

Figure 4-5: Material properties of G350 cold-formed steel and hot-rolled steel parts
4.4.2 Experimental program

First, a monotonic push test set was conducted to evaluate the lateral behaviour of the bare hot-rolled frame under push and pull load. The study was followed by two monotonic test sets (push and pull) to evaluate the lateral capacity of the HWPS with the same fasteners with no particular improvements. Afterwards, an improved connection between cold-formed and hot-rolled steel parts of the panel was designed, and the cyclic behaviour of the improved HWPs was evaluated.

This experimental program was designed to provide the following information: (i) the reliability of CFS wall panels to act as lateral load transferring elements along with hot-rolled steel frames as the lateral load resisting part; (ii) comparison between monotonic and cyclic response of the HWPS; (iii) providing information to calibrate the finite element models of the proposed system for application in mid-rise construction. Since the purpose of this study is to investigate and improve the lateral behaviour of the HWPS, the effect of vertical load is ignored at this stage of the study.

4.5. Monotonic tests

The lateral behaviour of a wall panel can be expressed as the relationship between the shear resistance and lateral displacement of the panel. In order to study the behaviour of the panel under monotonic loading, first the monotonic behaviour of the hot-rolled frame was investigated, and then the entire panel (hot-rolled frame + cold-formed panel) was tested under push and pull loading. For the monotonic loading protocol, the specimens were subjected to incremental lateral displacement at the rate of 0.1 mm/s.
4.5.1 Monotonic push of the hot-rolled steel panel

The overall behaviour of the HWPS under monotonic horizontal loads is represented as a relationship between the lateral shear resistance and the lateral displacement measured by LVDTs or the load actuator. The lateral load - displacement curve for the hot-rolled panel is illustrated in Figure 4-6. At the lateral displacement of 60 mm and the lateral load of 8.7 kN, the first yield was observed. The load cell ran out of stroke and the loading process stopped at the load of 14.4 kN and 185 mm lateral displacement. However, the load-displacement curve started flattening at about 150 mm drift. Local yielding occurred around the bottom connections of the hot-rolled panel to the strong floor as illustrated in Figure 4-7. As Figure 4-6 is showing, the stiffness of the current hot-rolled frame was relatively low, which resulted in a lateral deflection of 185mm (drift of 6.26%), which needed some improvements.

![Figure 4-6- Monotonic pull and push results of specimens HWPS-I, HWPS-III and HWPS-IV](image)
4.5.2 Monotonic push test of the HWPS

The lateral load was applied to the CFS top chord to investigate its ability and performance for load transfer. As shown in Figure 4-8, at a maximum horizontal load of 4.4 kN, the failure happened, where the screws connecting the cold-formed top chord to the cold-formed stud, directly attached to the hot-rolled column, were pulled out. The hot-rolled steel panel remained in the elastic deformation zone.

As per AISI S100 [203], the shear strength for each screw attaching cold-formed members, limited by tilting and bearing, is shown in Table 4-2, where $t_1$ and $F_{u1}$ are the thickness and tensile strength of members in contact with screw head, $t_2$ and $F_{u2}$ are the thickness and tensile strength of members not in contact with screw head, $d$ is nominal screw diameter and $P_{ns}$ is the nominal shear strength per screw.
According to Table 4-2, two pairs of screws attaching the top chord to the last stud, and connected to the hot-rolled part can contribute to 3.1kN of the lateral load to be transferred to the hot-rolled panel. However, the friction between cold-formed surfaces in addition to the interaction of top chord edge with hot-rolled panel is also expected to transfer a portion of the lateral load. Accordingly, the maximum achieved lateral load capacity of 4.4kN was predictable.

<table>
<thead>
<tr>
<th>$t_2/t_1$</th>
<th>Equation</th>
<th>$P_{ns}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.78&lt;1</td>
<td>$4.2(t_2^2d)/2 F_{u2}$</td>
<td>0.77kN</td>
</tr>
<tr>
<td></td>
<td>$2.7t_1d F_{u1}$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$2.7t_2d F_{u2}$</td>
<td></td>
</tr>
</tbody>
</table>

### 4.5.3 Monotonic pull test of the HWPS

Regarding the monotonic pull test, basically a different behaviour is expected in comparison with the push test. In the push analysis the local and global buckling of top chord and the failure of screw connections between top chord and the last stud attached to the hot-rolled steel panel are the possible failure mechanisms. During the pull test, however; the top chord buckling is not a concern anymore, and the pull out
of screws connecting the first CFS stud to the hot-rolled steel column could be the alternative possible failure mechanism.

The pulling monotonic load was applied to the top chord in two points, while no lateral load was directly applied to the hot-rolled part. As shown in Figure 4-9, although the deformation of the cold-formed stud was excessive in the screw regions attaching it to the hot-rolled steel column, the failure did not happen there. The same failure mechanism in pull test was as in push test i.e. the failure of screw connections between the top chord and the last stud.

In Figure 4-6, the observations of the monotonic evaluation of the panel reveal the following observations:

- The hot-rolled frame remained almost in elastic zone with no plastic deformations.

- The two rows of screws connecting the cold-formed stud to the hot-rolled column remained intact; however, there are major deformations around the screw area separating the cold-formed stud from hot-rolled column in the vicinity of the top screws (Figure 4-9)

- The major deformations of the panel were localised in two main areas: the connection point of the actuator tip to the cold-formed top chord; and the connection between hot-
rolled and cold-formed parts. The failure both in push and pull occurred in the screws connecting the top chord to the last stud attached to the hot-rolled part.

- As previously explained, the push and pull test results for the panel are different and that is due to two different failure modes of the panel under tension and compression. Under compressive loads, buckling of the top chord and pull out of screws connecting the top chord to the last stud is the failure mode; while, under tensile loads, CFS to HRS screw failure is observed.

### 4.6. Detection of deficiencies and improving the panel

The results of the monotonic push and pull tests revealed that an adequate connection between the hot-rolled and cold-formed part needs to be provided to increase the capacity of the HWPS. A new screw configuration, therefore, was designed in accordance with AISI S100 [203]. In the first set of tests, the connections between the cold-formed and hot-rolled parts attachment point are the governing factors of lateral load strength. The screwed connections of CFS members under shear conditions are designed in different steps. Firstly, a minimum spacing between the screws shall be at least three times as much as the screw diameter. Gauge 12 screws with 14 threads per 25.4 mm are used for the specimens. Gauge 12 screws represent a nominal diameter of 5.5 mm. Therefore, $S_{\text{min}}$, which is the minimum spacing between the centre of screws is $S_{\text{min}} = 3 \times d_f = 16.5$ mm; where $d_f$ is the nominal screw diameter. In addition, the minimum distance between the centre of screws and edges should be $1.5d_f$, which is 8.25 mm. According to the calculations provided in Table 4-2, in order to meet the maximum target capacity, which is the maximum lateral capacity of the hot-rolled panel (15kN according to Figure 4-6), a minimum of 20 screws on CFS part are required. Since the common surface of the top chord with vertical stud cannot accommodate the required
space for 20 screws, a linking part is designed to be attached on both top chord and hot-rolled steel beam (Figure 4-10). As a consequence, the screws attaching this linking part to the hot-rolled steel beam are also designed as per Table 4-3; where \( t_1 \) and \( F_{u1} \) are the thickness and tensile strength of member in contact with screw head (CFS link), \( t_2 \) and \( F_{u2} \) are the thickness and tensile strength of member not in contact with screw head (hot-rolled steel beam), \( d \) is nominal screw diameter and \( P_{ns} \) is the nominal shear strength per screw. Accordingly, 5 screws are required to accommodate the 15kN lateral load resisting target capacity. However, to maintain the symmetry of the panel, six screws (3 on each side of the hot-rolled steel beam) are used (Figure 4-10).

<table>
<thead>
<tr>
<th>( t_2/t_1 )</th>
<th>Equation</th>
<th>( P_{ns} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>5&gt;2.5</td>
<td>( 2.7t_1dF_{u1} )</td>
<td>3.27kN</td>
</tr>
<tr>
<td></td>
<td>( 2.7t_2dF_{u2} )</td>
<td></td>
</tr>
</tbody>
</table>

In addition to the improvements in connection point, the axial capacity of the cold-formed top chord should also be improved according to the target lateral load capacity. Therefore, in the top chord profile, the space between two studs is reinforced with a cold-formed member with the same profile as studs. The improvement details are shown in Figure 4-10.
4.7. Cyclic tests

According to the results of the existing literature regarding the effect of loading type (monotonic or cyclic) on the lateral behaviour of CFS stud shear walls [23, 50, 204-206], cyclic loading degrades the shear performance of the walls by up to 90% of that of monotonic loading condition. In the first part of this study, the monotonic behaviour of the bare hot-rolled panel and HWPS was investigated. Having determined the maximum lateral load capacity and ultimate displacement of the hot-rolled panel, the HWPS is improved so that it can achieve the hot-rolled panel characteristics. In the following part of this study, the improved HWPS would be investigated under cyclic loading protocol, according to the hot-rolled steel ultimate displacement.
4.7.1 Loading protocol

In the cyclic test stage, the specimens were subjected to a cyclic load based on method B of ASTM E2126 [27]. This method implements gradually increasing displacement cycles. As illustrated in Figure 4-11, this loading protocol includes two consequent patterns. Five fully reserved cycles of 1.25%, 2.5%, 5%, 7.5%, and 10% of the ultimate displacement, form the first pattern while the second one consists of the phases with three equal cycles of 20%, 40%, 60%, 80%, 100%, and 120% of the ultimate displacement. In this study, based on method B of ASTM E2126, the ultimate displacement is supposed to be extracted from a monotonic push test of the panel until the failure point. Since the ultimate displacement of the hot-rolled part of the panel is extracted, and the panels are optimised at each level of tests, and are not going to be all identical, the ultimate displacement of the HWPS is considered as $\Delta_m$ with the value of 150mm. This is the point that the hot-rolled steel panel’s push curve started flattening, as previously discussed.

![Cyclic displacement schedule (Method B)](Figure 4-11- cyclic load protocol)
4.7.2 Results

The hysteretic behaviour of the hot-rolled panel and HWPS are compared here to investigate the extent of reliability of the cold-formed panel for being employed as the lateral load transferring system in HWPS. The target for improved HWPS was to present a hysteretic response similar to that for the hot-rolled steel panel. Following is the discussion on the hot-rolled steel panel and HWPS cyclic test results.

**Hysteretic response:** Figure 4-12 shows the hysteretic response of the specimen Type II, which is a bare hot-rolled panel. As predicted according to the monotonic push test results, the maximum resisted load, whether in push or pull direction was lower than 14.4 kN. The maximum lateral capacity achieved under push cyclic load was 13.85 kN, which occurred at the lateral displacement value of 113.1mm; and the one under pull cyclic load is 13.55 kN occurring at the lateral displacement of 141.4mm. In addition, the results are confirming that the panel has insufficient stiffness regarding the serviceability limit states. Considering a maximum allowable deflection of H/300 for the wall panel, the maximum lateral displacement would be 10mm. This leads to a lateral load of about 2.8 kN which is considered to be very small for a wall panel. This again confirms the requirement for improving the panel for stiffness purposes. Since specimen HWPS-II does not include any cold-formed elements or connections, no hysteretic pinching effect is detected in the hysteretic response.
Figure 4-12- Cyclic response of the hot-rolled steel panel (Specimen Type II)

Figure 4-13-Cyclic response of the improved HWPS (IHWPS)

Figure 4-13 illustrates the hysteretic response of the specimen IHWPS. It can be observed that the maximum lateral load resisted in compression mode is 13.3 kN (at the lateral displacement of 182.3 mm) and for tension mode it is 12.56 kN (at the lateral displacement of 147 mm). These results reveal the fact that by adding the linking part with an adequate number of fasteners, the governing failure mode for specimens HWPS-III and HWPS-IV (which was connection failure at the attachment point of the
two panels) has been alleviated in the improved HWPS. Consequently, the maximum lateral load capacity equals 92.4% of the target capacity. On the other hand, as illustrated in Figure 4-14 (a), a significant pinching behaviour can be observed in the lateral deformations of the HWPS. Although the linking part enhanced the lateral capacity of the panel, no improvement has been considered to account for pinching behaviour and this leads to lower energy dissipation in case of any seismic events. The pinching behaviour is the result of local buckling of cold-formed studs as well as the characteristics of the (CFS to CFS and HRS to CFS) fasteners’ behaviour.

A comparison between the envelopes of the load-displacement curves for specimens HWPS-II and IHWPS is shown in Figure 4-14-b. Compared to the specimen HWPS-II, the lateral stiffness of the IHWPS was reduced by 16%.

![Figure 4-14- (a) Hysteretic curves; and (b) hysteretic envelope curves for bare hot-rolled steel panel and HWP](image)

Failure modes: Since the lateral stiffness of the hot-rolled steel panel, and consequently the improved HWPS is not so high, and due to serviceability limitations, there was no need to continue the test until ultimate failure of the panels. As a result, no significant failure mode was observed, influencing the hysteretic load application process before
reaching the service limits. However, there were a number of local failure modes in both specimens HWPS-II and IHWPS. Regarding specimen HWPS-II, the plastic deformation in the hot-rolled steel columns and around the points clamped to strong floor were observed (Figure 4-15). For specimen IHWPS, the main failure areas were the two ends of the top chord. The end zone connected to the loading jack is shown in Figure 4-16(a). The initial rivets (which failed in specimens HWPS-III and HWPS-IV) remained intact in specimen IHWPS, because of the load transfer pattern forming through the linking element and its screws. The linking element was locally buckled between the two adjacent screws attaching hot-rolled steel part to the last CFS stud (Figure 4-16(c)). The screws in the vicinity of the linking element’s buckled area demonstrate an excessive tilting (Figure 4-16(b)). However, none of the screws failed under the cyclic load. In addition, as previously mentioned, at both ends of the top chord, where it was connected to the load cell and hot-rolled steel, a significant buckling occurred in top chord flange and the vertical stud adjacent to the hot-rolled steel column. It indicates that these areas may require some improvements.
Figure 4-15: failure modes for hot-rolled panel under cyclic load (Specimen HWPS-II)

Figure 4-16: deformations for improved HWP (Specimen IHWPS)

4.8. Conclusions

An experimental investigation, considering the monotonic and cyclic behaviour of a Hybrid hot-rolled and Cold-formed steel wall panel has been provided in this research. This experimental study provides the following conclusions:
1. Since the hot-rolled panel is resisted the lateral load, it is of great importance to assure a complete load transfer path from the cold-formed panel to the hot-rolled frame. To achieve that, preventing any significant local buckling in the CFS parts, and providing adequate number of screw connections between the hot-rolled and CFS sections are of great importance.

2. Significant local deformations in some critical areas of the CFS panel, such as the attachment points to the hot-rolled part, can be reduced by local enhancement approaches to provide an improved global behaviour.

3. By improving the CFS to hot-rolled connections, the maximum lateral capacity of the system is improved. Nevertheless, the pinched behaviour may remain. Such pinched behaviour is a result of tilting and local deformations occurring in the screws.

4. With regard to construction issues, the hybrid wall panel should be delivered as two individual parts: a hot-rolled steel panel and a cold-formed steel stud panel. Then, the two parts are attached on-site. The reason for that is a relatively large discrepancy between their stiffness which can create major deformations in cold-formed steel panel under transportation and lifting loads.

This experimental research does not consider the effect of the vertical loads on the HWPS. In addition, the stiffness of the hot-rolled steel panel was considered to be relatively low to prevent a large stiffness gap between the two parts of the panel. To improve the HWPS stiffness, the hot-rolled steel panel can be improved with respect to stiffness for future studies. However, possible local failures between the two parts should again be considered due to increased stiffness differences between them. The next chapter of this thesis is dedicated to evaluating the application of the HWPS for multi-storey structures. In that case, the hot-rolled panel’s stiffness can be increased
using some laterally bracing elements. A hot-rolled steel top beam can also be an alternative for CFS top chord element to transfer significantly higher lateral loads from CFS panel to the hot-rolled steel panel; while the cold-formed part is only responsible for sustaining the vertical gravity loads.
Chapter 5  Hybrid Cold-Formed and Hot-Rolled Steel

Wall Panel System for Mid-Rise Construction
5.1. Introduction

An alternative lateral load resisting system is required for cold formed steel structures to be seismically reliable for midrise to high rise construction industry. A new system comprising a hot rolled steel panel and a cold formed steel panel laterally attached to each other is proposed in this thesis. This chapter provides a practical engineering analysis and design process of a 4-storey building with the alternative lateral load resisting system for cold formed steel structures. The proposed hybrid panel utilizes a hot rolled steel panel as the main lateral load resisting element connected to cold formed steel panel carrying a major portion of vertical loads. A typical architectural plan is chosen and according to the required openings and architectural limitations, the structure is analysed and designed. The hot rolled steel panel is applicable in all external and internal walls with openings of 1.2m width or less. The hybrid system gives engineers the same design and construction flexibility as ordinary moment resisting steel frame system while it offers the advantage of prefabrication and modular construction process. It is light which gives the benefit of being easy to transport and construct while remaining tough to withstand shipping and strong enough to be relied on as a lateral load resisting element. In other words, the proposed hybrid system provides the advantages of cold formed steel and hot rolled steel structural systems simultaneously. The finite element results, provided in this chapter, facilitates the analysis and design process of the proposed system which is extendable to structures with increased number of stories.

5.2. Background and description

Recently, the use of cold-formed steel (CFS) frames as the main lateral load bearing system for low to mid-rise structures has become a common practice. Compared to hot-rolled steel frame, CFS frame is becoming a popular option for residential construction due to its light weight, construction flexibility, prefabrication options and ease of installations[197, 198]. However, unlike hot-rolled steel structures, the application of CFS systems have been proven challenging with regard to their lateral
load resisting capacity. Low rigidity of CFS sections alongside partially restrained screw or rivet fasteners leads to limited or no lateral resistance for CFS frames [199]. There have been various efforts to combine other structural systems with CFS frames to improve their seismic characteristics and compensate for the existing deficiencies. Relying on face sheathings is the first common approach to improve the lateral load response of CFS wall systems [50, 51, 54, 165, 200].

Lightweight cold formed steel framed structures are typically configured as repetitive members developing a floor-to-floor framed system. Face sheathings and strap bracings can provide reasonable lateral load resisting capacity for CFS stud walls utilised in one or two storey structures acting as the main or part of the lateral force resisting systems (LFRS). There have been various studies investigating the lateral performance of CFS LFRS with strap braced or sheathed walls [54, 116, 166, 168, 170, 172, 202, 207]. These common LFRSs utilise floor diaphragms and load bearing walls as the main system-level load paths[208]. As a concept, the lateral force is delivered from the diaphragm to the shear wall to be transferred to the foundation. A completed load path from the diaphragm to the foundation should be secured to prevent any local failure occurrence.

From constructional point of view, in a conventional multi-storey structure, the cold formed studs are used to form the interior partition walls or exterior facades; while, the main lateral force resisting system can be a steel or concrete moment resisting frame or shear wall system. Therefore, the CFS studs are not structurally connected to the frame in order not to carry any axial forces. The studs instead are aligned laterally every 600mm to provide sufficient support for finishing surfaces of the wall. However, every individual stud offers an axial force resisting capacity which can be utilised as a gravity force resisting system. Taking the currently unused capacity of the stud into
account may lead to a lighter gravity force resisting system and consequently a lighter LFRS.

In case of a seismic event in earthquake prone regions, CFS structures are expected to provide a lateral load resistance. CFS shear walls with various face sheathings (wood, steel and other materials) and strap braced wall systems or a mix of both have been proven as effective lateral load resisting systems in the current literature. The studies have shown that strap braced walls often have large residual displacements, which could be problematic due to permanent deformation resulting from severe damage and an inability to re-centre [28]. Such large residual displacement and very probable slack in the wall and rather poor energy dissipation during lateral cyclic loading make the existing strap bracing systems quite ineffective in earthquake prone regions [114, 116, 124, 170].

Hot-rolled steel frames on the other hand, have proven to be reliable lateral load resisting systems via a wide range of studies on their seismic behaviour in low-rise to mid-rise structures. Therefore, the concept of a hybrid system comprising CFS and hot-rolled steel to accommodate the advantages of both systems is an interesting field of research in order to experiment the possibility of CFS structures extension to mid-rise to high-rise construction.

Earlier in this study, a HWPS was introduced and evaluated for low-rise (one and two storey) buildings. However, even after applying the improvements to the Hot-rolled to Cold-Formed steel connection, relatively low lateral stiffness of the system remained as a concern. Due to insufficient stiffness, to be able to apply the system to mid to high rise structures, the need for further improvements still exists. In this chapter, the hybrid panels with improved lateral stiffness are individually evaluated and then their
application in a 4-storey building is investigated considering the effect of axial load bearing capacity of wall studs.

Like previous set of HWPSs, every panel is made of two parts, namely: 1) cold formed steel part and 2) hot rolled steel part. The CFS part of the panel has no lateral force resisting capacity while the hot rolled part accounts for the whole lateral force transferred to the panel. The gravity load is resisted by every individual span of the panel, either CFS or HRS, depending on the loaded span. Assuming a rigid diaphragm in every storey eliminates the need for design of the collector element at this stage of the study. The previous chapter of this thesis presented an investigation on a CFS channel acting as a collector element of the HWPS in a one or two-storey building. However, upgrading from low-rise to mid to high-rise structures with a higher range of lateral forces to be transferred, a hot rolled profile can be an alternative acting as the collector element.

5.3. HWPS characterisation

There is a vast amount of literature on the lateral load resisting capacity of LSF systems [202]. Sheathing properties [106], framing details, fastener types and spacing [171], geometry and construction approach might be considered as the main contributing factors to the CFS frame’s lateral resisting capacity. In a CFS structural system, the lateral performance of the structure is affected by both horizontal and vertical elements as well as connections. The behaviour of cold-formed steel stud walls is governed by local failures of fasteners or individual stud profiles due to local, distortional or global buckling. These reductions must be captured prior to applying the hybrid system to a multi-storey building. Using Finite Element Analysis and full-scale experimental tests,
the lateral behaviour and local failure of an individual panel is identified and is applicable to a full system created by the hybrid panels.

The details of the hot rolled and cold-formed steel parts are given in Figure 5-1. For both of the laterally stiffened panels, the cold formed part is made of WSL92-075-30 C sections preassembled into a top chord (C94-075-100) and bottom chord (C94-075-30) CFS sections. The hot rolled steel profile is made of SHS89x89x3.5 profiles, once braced with two gusset plates (350x350x4mm) at every corner (Figure 5-1-a); and once braced with a SHS89x89x3.5 knee element with 495mm length in every corner (Figure 5-1-b). The material properties for both hot rolled and cold formed steel are presented in Figure 4-5.

![Diagram of HWPS and set up details](image)

(a) general overview with gusset plates  
(b) with knee element

The vertical load representing the top floor gravity loads is 22.5 kN per each vertical load cell (a total gravity of 67.5 kN). The 22.5 kN floor dead load is calculated considering a tributary width of 6 meters for panel; that would be $6m^2$ per meter of length of panels. Each vertical jack is provided with a roller head to facilitate the
horizontal displacement while applying vertical load. Four linear variable differential transformers (LVDTs) of 170 mm were attached to measure the lateral displacements at top left, top right, mid height left and mid height right of the specimens (Figure 4-2: LVDTs A to D respectively). The lateral load is measured via the actuator’s load cell. Figure 5-2 shows the test set up details.

A monotonic push finite element analysis was conducted to evaluate the maximum lateral load resisting capacity and ultimate displacement of the panel to be used for cyclic load regim calculations. The method of analysis is Static general. Since both the HRS and CFS members are thin profiles, shell element with reduced integration (S4R element) is used to simulate their behaviour. The optimum mesh size is selected by conducting a sensitivity analysis with regards to size of the mesh vs analysis runtime; a maximum mesh size of 20 mm is considered for the FEM analyses accordingly. In the experimental study Hex flange head self-drilling screws of 12 gauge diameter with 14 thread/inch are used as fastener element for the CFS parts and also between cold-formed and hot-rolled steel parts. In FE model the connections are tied for translational degrees of freedom (pinned). In addition, a hard surface- to – surface hard contact is
also introduced between all surfaces in direct contact. The connection of the HRS panel to the strong floor is modeled using two strips of elements tied in all translational degrees of freedom to represent two hold down ties used in experimental study. Loading protocol is induced to the HRS column on a 80x80mm plate as displacement control loading condition. This is how the hydraulic jack applies the displacement regimen to the experimental specimen.

The finite element results determine the maximum lateral load capacity of the stiffened panels to be considered for the tests set up. The lateral load- displacement curve for the laterally stiffened HWPS (SHWPS) with a bare hot rolled steel panel is compared to the one with stiffened hot rolled steel panel (with gusset plate and knee element stiffeners) in Figure 5-3. The developed stresses in the hot rolled parts are illustrated in Figure 5-4.

Figure 5-3- Monotonic pull FEA results for plain and stiffened HWPS
Figure 5-4 - Mises stress developed in: a) bare and b) stiffened hot rolled steel panel with gusset plates and c) knee elements

Method B of ASTM E2126 is used for calculation of cyclic load protocol to be applied to the test specimens. To compare the cyclic responses of all panels, a constant value of 150mm is assumed for the ultimate displacement of the SHWPS. Therefore, the cyclic displacement schedule is similar to the one for the first set of tests explained in Chapter 4 and illustrated in Figure 4-11.

The hysteretic behaviour of the SHWPSs resulting from the full scale panel tests are illustrated in Figure 5-5. As expected, according to the finite element analysis results, the lateral load resisting capacity is improved to about 27 kN by adding the stiffening elements to the corners. The plastic hinges formed in the vicinity of the bracing element (either the gusset plate or knee element) attachment point to the bottom beam and columns (Figure 5-6). The gusset plates also buckled at the ends attached to the
bottom beam while the knee elements remained in elastic zone and did not experience any plastic deformations.

(a)  
(b)  

Figure 5-5: Experimental cyclic response of the HWPS stiffened with: a) gusset plates b) knee elements

The cold formed part of the panel experienced significant buckling failures particularly in the connecting point of the last cold formed stud to the hot-rolled steel column, which was expected due to the rather considerable differences in the stiffness. A
separation between the cold formed stud from the hot rolled steel column happened between the attachment points (Figure 5-7). However, it caused no significant change on the cyclic response of the panel.

Figure 5-7: Separation of CFS and HRS between the screw connected points

5.4. **Building archetype**

This part of the research involves the design of a four-storey building. Firstly, the cyclic behaviour of the proposed SHWPS is experimentally evaluated. As previously shown in Figure 5-1, the SHWPS includes a hot rolled part, a cold formed part and a hot rolled beam profile acting as the collector element. The cold formed part of the panel can vary in length according to architectural demands. Following the full scale tests on wall panels, a 4 storey building is analysed and designed utilising the SHWPS as the main LFRS. The considered
The building is located in greater Los Angeles area and accommodates four residential units in every level (Figure 5-9) of 21m by 16.2m in plan and 3m in height with the total height of 12m and total seismic weight of 2,400kN. The building was loaded and analysed according to ASCE7 [81] and designed according to AISC[9]. The cold formed studs are separately designed according to AISI- S100 [9]

5.4.1 Definitions

Loads: The analysis involves live load, dead load, superimposed dead load and cladding load. The dead load is the self-weight calculated by the FE software according to the material self-weight. The values considered for other load types are provided in Table 5-1.

<table>
<thead>
<tr>
<th>Load type</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Live load( kN/m²)</td>
<td>1.92</td>
</tr>
<tr>
<td>Roof’s live load( kN/m²)</td>
<td>0.96</td>
</tr>
<tr>
<td>Superimposed dead load( kN/m²)</td>
<td>1</td>
</tr>
<tr>
<td>Cladding( kN/m)</td>
<td>1.825</td>
</tr>
</tbody>
</table>
Plan and elevation: As shown in Figure 5-9, the building has six spans in horizontal (X) direction and three spans in vertical (y) direction. However, the SHWPS offers the flexibility of number of spans since by connecting to the rigid diaphragm; every individual hybrid partition wall can act as a shear wall. The key feature of this structural system is that it falls into panelised construction category while having the advantages of framing systems. Once the structure is designed, every cold-formed steel panel is assembled and delivered to construction site along with the hot-rolled part of the panel. The parts will then be assembled to form the system. The major design concern is to minimise the number of hot rolled steel parts while maintaining the allowable storey drifts. Figure 5-10 shows a schematic of a typical elevation of the building. This elevation is also highlighted in the building 3D view shown in Figure 5-12.
Seismic factors: Based on ASCE 7-10, the risk category factor for earthquake loads is risk category I because it is a residential 4-storey building which represents a low risk to human life in the event of a failure. The seismic importance factor, $I_e$, is 1 according to the risk category factor.

The seismic load resisting factors for the proposed system are extracted from Table 12.2-1 ASCE7-10 and the seismic data are considered as bellow:

- Response modification factor, $R$: 4
- Over strength factor, $\Omega$: 2
- Deflection amplification factor, $C_d$: 3.5

And the following parameters are calculated by the FE software:

- Fundamental period of vibration used in the analysis: 0.462 Sec
- Coefficient Used, $C_s$: 0.220967
- Weight used: 2367.2884 kN
- Base shear: 523.09 kN
- Seismic design category: D

Response spectrum: The location of the building is in greater Los Angeles area and the site class is considered to be a very dense soil and soft rock which falls in site class C. The mapped acceleration parameters are $S_s = 1.3258$ and $S_1 = 0.5064$ and the long-
The short-period site coefficient, $F_a = 1$ and long-period site coefficient $F_v = 1.3$. Therefore, the spectral response acceleration parameters at short periods and a period of 1 are defined as below, respectively:

$$SDS = \frac{2}{3} \times F_a \times S_s = 0.8839$$

$$SD1 = \frac{2}{3} \times F_v \times S_1 = 0.4389$$

The response spectrum function obtained for this structural analysis is depicted in Figure 5-11.

![Figure 5-11: Response spectrum to be applied for equivalent lateral force procedure](image)

### 5.5. Full scale building modelling

The full scale building was modelled using FE software to be analysed under gravity and lateral applicable loads according to ASCE 7 [153]. The major goal of this part of the study is firstly to provide a model to predict the overall behaviour of a multi-storey building in a seismic event; and secondly to provide a feasible design procedure to be used by structural engineers. The details of the floor system are not investigated here and it is assumed to act as a rigid diaphragm. This raises a potential research study subject for future studies to check the rigidity of the flooring system according to the floor details.
As shown in Figure 5-12, for both gusset plate and knee element SHWPSs, a 3D model of the building with the CFS studs, structurally connected to the floors, is provided and an equivalent lateral force procedure is used for analysis. The LFRS is a bearing wall system and the suggested R factor for light frame wall system using flat strap bracing, R=4, is used conservatively.

Figure 5-12- Preliminary designed 3D model

5.6. **Notional loads and second order effects in analysis and design of a 4-storey building**

**Notional loads**

Theoretically, buckling occurs suddenly in a perfect slender column under pure axial load. No lateral deflection occurs prior to the failure point where the critical point is attained. Due to material and geometrical imperfections, real columns are inevitably imperfect. Residual stresses are the main sources of material imperfections while out of straightness and out of plumbness are the main causes of geometrical imperfections. Both material and geometrical imperfections are unavoidable during the fabrication of
cold-formed steel profiles and panels. Since the usual analysis performed by design engineers determines forces and moments considering an elastic and perfect profile, the imperfection effects must be taken into account in linear elastic analysis. The most direct and accurate method of embedding the imperfections into the analysis process is explicitly modelling the structure which might not be an efficient approach for a frame of some significant size. Therefore, the code (AISI-S100) considers notional loads to be applied to the lateral framing systems to account for the imperfections. Notional loads are virtual loads assumed to account for the neglected destabilising effects.

Notional loads are lateral loads applied to every individual level of the structure and are determined in terms of the gravity loads of that level. The magnitude of notional loads shall be calculated as \( N_i = 0.002Y_i \), where \( N_i \) is the notional load applied at level \( i \) and \( Y_i \) is the gravity load applied at level \( i \). At any level, \( N_i \) shall be applied in a manner to provide the greatest destabilizing effect.

Second order effects

A first order analysis evaluates the structural forces and drifts assuming the structural elements to behave as linear elastic elements; while the second order analysis, taking the P-delta effect into account, is required to evaluate the sensitivity of the structure to change of geometry of structure and member curvature.

For structures in which the ratio of maximum second-order drift (considering the P – \( \Delta \) effect) to maximum first-order drift in all stories is equal to or less than 1.7, it is permissible to apply the notional load, \( N_i \) only in gravity-only load combinations and not in combinations that include other lateral loads.
5.7. Analysis results

A preliminary design is conducted to approximate the number of hot rolled steel panels in each direction. Figure 5-12 shows the 3D model of the structure with gusset plate SHWPS designed according to AISI using the first order analysis data. The maximum storey drifts are compared for first and second order analysis and as shown in Table 5-2 the ratio of second order to first order storey drifts in both X and Y directions are less than 1.7 and the notional loads shall be applied only to gravity load combinations.

<table>
<thead>
<tr>
<th>Table 5-2- First order and second order storey drifts</th>
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<tbody>
<tr>
<td></td>
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<tr>
<td>Maximum storey drifts</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>X* direction</td>
</tr>
<tr>
<td>1st order</td>
</tr>
<tr>
<td>Knee</td>
</tr>
<tr>
<td>St 4</td>
</tr>
<tr>
<td>St 3</td>
</tr>
<tr>
<td>St 2</td>
</tr>
<tr>
<td>St 1</td>
</tr>
</tbody>
</table>

* X and Y directions are defined in Figure 5-9

According to ASCE 7-10, storey drift is the horizontal deflection at the top of the storey relative to the bottom of the storey. According to Table 12.12-1 of the code [81], for the current 4 storey building in risk category I, the allowable storey drift is calculated as:

\[
\Delta_a = 0.025h
\]

\[
\frac{C_d \times \Delta_{eu}}{h} < 0.025 \rightarrow \frac{\Delta_{eu}}{h} < \frac{0.025}{3.5} = 0.00714
\]

Where h is the storey height and \( \Delta_a \) is the allowable storey drift according to the deflection amplification factor, \( C_d \), of 3.5 specified for the case. The storey drifts obtained from the FE analysis should be compared to \( 0.025/3.5 = 0.00714 \).
As previously explained, for structures supporting gravity loads through vertical columns the effect of initial imperfections shall be considered using notional loads as specified in section C2.2b of the code.

The storey drift results after the application of notional loads to the model is represented in Figure 5-13, showing that for both SHWPSs the maximum storey drifts are within allowable storey drift range of $\Delta_a = 0.025h$ according to ASCE7 [153]:

![Figure 5-13: Storey drifts for second order analysis with application of notional loads to gravity only load combinations](image)

5.8. **Design of CFS studs**

Once the hot-rolled steel LFRS is fully designed according to AISC, the cold-formed studs must be designed manually according to the axial forces obtained from the analysis results and the bending moment developed as a result of local application of wind loads. Two typical CFS studs are considered (Figure 5-14) and their capacity is calculated using Finite Strip analysis where the developed FSM code is utilised. Section one is a typical C92-30-075 stud and section two is a double stud of the same section assuming to act fully compositely. For a double C section to act as an integral section they should be attached at particular points using rivets, clinches or by any
other usual means. The attachment method and configurations are considered to be able to effectively attach the C studs so they act as an integral section. Considering a resistance factor of $\varphi = 0.8$, the hand calculations and design curves for the CFS studs applied to the structure are shown in Figure 5-20. To verify the FSM results, one of the CFS studs (single C section) is also manually designed using effective width and FE methods as follows:

![Figure 5-14: Two typical CFS studs applied to the HWPS: a) single WSL92-075-30 and b) double WSL92-075-30](image)

**Section 1: C92-30-075**

A. Effective width method

In this section, the single C-section cold formed steel stud is designed according to North American Specification for Design of Cold-Formed Steel Structural Members [9].

**Axial and flexural properties (Table 5-3)**

1. Basic parameters (the parameters are shown in Figure 5-14):
\[ A' = 92\text{mm} \quad B' = 30\text{mm} \]
\[ C' = 10\text{mm} \quad t = 0.75\text{mm} \]
\[ R = 3\text{mm} \quad \alpha = 1.0 \quad (\text{Section has stiffener lips}) \]
\[ r = R + \frac{t}{2} = 3.375\text{mm} \quad a = A' - (2r + t) = 84.5 \]
\[ \bar{a} = A' - t = 91.25\text{mm} \quad b = B' - \left(\frac{t}{2} + \alpha(r + \frac{t}{2})\right) = 22.5\text{mm} \]
\[ \bar{b} = B' - \left(\frac{t}{2} + \alpha t/2\right) = 29.25\text{mm} \quad c = \alpha \left(C' - (r + \frac{t}{2})\right) = 6.25\text{mm} \]
\[ \bar{c} = \alpha(C' - \frac{t}{2}) = 9.624\text{mm} \quad u = \frac{\pi r}{2} = 5.3\text{mm} \]

2. Cross sectional area

\[ A = t(a + 2b + 2u + \alpha(2c + 2u)) = 122.4\text{ mm}^2 \]

3. Moment of inertia about the x-axis

\[ I_x = 2t(0.0417a^3 + b \left(\frac{a}{2} + r\right)^2 + u \left(\frac{a}{2} + 0.637r\right)^2 + 0.149r^3 + \]
\[ \alpha \left(0.0833c^3 + \frac{c}{4}(a - c)^2 + u \left(\frac{a}{2} + 0.637r\right)^2 + 0.149r^3\right) \right) = 15,376\text{mm}^4 \]

4. Distance between centroid and web centreline

\[ \bar{x}_c = \frac{2A}{t} \left(b \left(\frac{b}{2} + r\right) + u(0.363r) + \alpha(u(b + 1.637r) + c(b + 2r))\right) = 8.17\text{mm} \]

5. Moment of inertia about the y-axis

\[ I_y = 2t \left(b \left(\frac{b}{2} + r\right)^2 + \frac{b^3}{12} + 0.356r^3 + \alpha((c(b + 2r)^2 + u(b + 1.637r)^2 \]
\[ + 0.149r^3 - A\bar{x}_c^2 = 14,762\text{mm}^4 \]
6. Distance between shear centre and web centre line

\[ m = b \left( \frac{3\bar{a}^2\bar{b} + \alpha\bar{c}(6\bar{a}^2 - 8\bar{c}^2)}{\bar{a}^3 + 6\bar{a}^2\bar{b} + \alpha\bar{c}(8\bar{c}^2 - 12\bar{a}\bar{c} + 6\bar{a}^2)} \right) = 13.58\text{mm} \]

7. Distance between centroid and shear centre

\[ x_o = -(\bar{x}_c + m) = -21.75\text{mm} \]

<table>
<thead>
<tr>
<th>Basic parameters</th>
<th>Section properties</th>
</tr>
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<tbody>
<tr>
<td>A'</td>
<td>92</td>
</tr>
<tr>
<td>B'</td>
<td>30</td>
</tr>
<tr>
<td>C'</td>
<td>10</td>
</tr>
<tr>
<td>t</td>
<td>0.75</td>
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<tr>
<td>\bar{c}</td>
<td>9.625</td>
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<tr>
<td>u</td>
<td>5.301</td>
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</table>

- Effective section modulus, \( S_e \), at initiation of yielding
1. Compression flange

\[ \frac{w}{t} = 29.998 < 60 \text{ OK} \]

\[ S = 1.28\sqrt{\frac{E}{f}} = 30.98 \]

\[ I_a = 399t^4 \left( \frac{w}{t} - 0.328 \right)^3 \leq t^4 \left( 115 \frac{w}{S} + 5 \right) \rightarrow I_a = 33.16 \text{ mm}^4 \]

\[ d = 6.25 \text{ mm} \]

\[ \theta = 90 \text{ degrees} \]

\[ I_s = \left( d^3 t \sin^2 \theta \right)/12 = 15.26 \text{ mm}^4 \]

\[ R_i = I_s/I_a = 0.46 \]

\[ n = \left( 0.582 - \frac{w}{t \sqrt{45}} \right) \geq 1/3 \rightarrow n = 0.34 \]

\[ D/w = 0.444 \]

\[ k = \left( 4.82 - \frac{5D}{w} \right) R_i^n + 0.43 = 2.42 \leq 4 \text{ OK} \]

\[ F_{cr} = k \frac{\pi^2 E}{12(1-\mu^2)} \left( \frac{t}{w} \right)^2 = 499.3 \text{ N/mm}^2 \]

\[ \lambda = \sqrt{\frac{f}{F_{cr}}} = 0.837 > 0.673 \rightarrow \text{flange is subject to local buckling} \]

\[ \rho = (1 - 0.22/\lambda)/\lambda = 0.88 \]

\[ b = \rho w = 19.81 \text{ mm} \]

2. Stiffener lip

\[ w/t = 8.33 \]
Maximum stress in lip by similar triangles:

\[ f_1 = 321.47 \text{ N/mm}^2 \]

\[ f_2 = 273.9 \text{ N/mm}^2 \]

\[ \psi = f_2/f_1 = 0.852 \]

\[ k = 0.442 \]

\[ F_{cr} = k \frac{\pi^2 E}{12(1 - \mu^2)} \left( \frac{t}{w} \right)^2 = 1178.97 \text{ N/mm}^2 \]

\[ \lambda = \sqrt{\frac{f}{F_{cr}}} = 0.522 < 0.673 \rightarrow \text{lip is not subject to local buckling} \]

\[ d'_s = d = 6.25 \text{ mm} \]

\[ d_s = d'_s \times R_l = 2.87 \text{ mm} \]

The calculated parameters are summarised in Table 5-4.
Table 5-4 - Parameters involved in Effective section modulus, $S_e$, calculation

<table>
<thead>
<tr>
<th>Compression flange</th>
<th>Stiffener lip</th>
<th>Web</th>
</tr>
</thead>
<tbody>
<tr>
<td>w/t</td>
<td>w/t</td>
<td>w/t</td>
</tr>
<tr>
<td>S</td>
<td>$f_1 (N/mm^2)$</td>
<td>$\psi$</td>
</tr>
<tr>
<td>$l_a (mm^4)$</td>
<td>$f_2 (N/mm^2)$</td>
<td>$k$</td>
</tr>
<tr>
<td>d (mm)</td>
<td>$\psi$</td>
<td>$\lambda$</td>
</tr>
<tr>
<td>$\theta (degree)$</td>
<td>$k$</td>
<td>$\rho$</td>
</tr>
<tr>
<td>$l_s (mm^4)$</td>
<td>$F_{cr} (N/mm^2)$</td>
<td>$b_e(mm)$</td>
</tr>
<tr>
<td>$R_l$</td>
<td>$\lambda$</td>
<td>$h_0/b_0$</td>
</tr>
<tr>
<td>n</td>
<td>$d'_s (mm)$</td>
<td>$b_1 (mm)$</td>
</tr>
<tr>
<td>D (mm)</td>
<td>$d_s (mm)$</td>
<td>$b_2 (mm)$</td>
</tr>
<tr>
<td>D/w</td>
<td></td>
<td>$b_1 + b_2 (mm)$</td>
</tr>
<tr>
<td>$\kappa$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$F_{cr} (N/mm^2)$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\lambda$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\rho$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>b (mm)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

$$b_1 + b_2 = 50.96 > \frac{w}{2} = \frac{84.5}{2}$$
Table 5-5: Calculation of moment of inertia

<table>
<thead>
<tr>
<th>Element</th>
<th>L mm</th>
<th>y from top fibre (mm)</th>
<th>Ly (mm²)</th>
<th>Ly² (mm³)</th>
<th>I₅ (mm⁴)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top flange</td>
<td>19.8</td>
<td>0.375</td>
<td>7.4</td>
<td>2.8</td>
<td>0.000</td>
</tr>
<tr>
<td>Bottom flange</td>
<td>22.5</td>
<td>91.6</td>
<td>2,061.6</td>
<td>188,890.2</td>
<td>0.000</td>
</tr>
<tr>
<td>Web</td>
<td>84.5</td>
<td>46</td>
<td>3887</td>
<td>178,802</td>
<td>50,279.3</td>
</tr>
<tr>
<td>Negative web element</td>
<td>8.7</td>
<td>16.4</td>
<td>142.7</td>
<td>2,338.2</td>
<td>55.1</td>
</tr>
<tr>
<td>Top inside corner</td>
<td>5.3</td>
<td>1.6</td>
<td>8.5</td>
<td>13.6</td>
<td>5.7</td>
</tr>
<tr>
<td>Bottom inside corner</td>
<td>5.3</td>
<td>90.4</td>
<td>479.2</td>
<td>43,324.1</td>
<td>5.7</td>
</tr>
<tr>
<td>top outside corner</td>
<td>5.3</td>
<td>1.6</td>
<td>8.5</td>
<td>13.6</td>
<td>5.7</td>
</tr>
<tr>
<td>Bottom outside corner</td>
<td>5.3</td>
<td>90.4</td>
<td>479.2</td>
<td>43,324.1</td>
<td>5.7</td>
</tr>
<tr>
<td>Top Lip</td>
<td>2.9</td>
<td>5.2</td>
<td>14.9</td>
<td>77.4</td>
<td>2</td>
</tr>
<tr>
<td>Bottom lip</td>
<td>6.2</td>
<td>85.1</td>
<td>532.0</td>
<td>45,289.2</td>
<td>20.3</td>
</tr>
<tr>
<td>Sum</td>
<td>165.9</td>
<td></td>
<td>7,621.14</td>
<td>502,075</td>
<td>50,379.63</td>
</tr>
</tbody>
</table>

\[
\bar{y} = \frac{\sum Ly}{\sum L} = \frac{7621.14}{165.857} = 45.95\text{mm}
\]

\[
I_x = \left[\sum I_x' + \sum Ly^2 - \bar{y}^2 \sum L\right]t = 151,680\text{mm}^4 \text{ (Parameters summarised in Table 5-5)}
\]

\[
S_e = \frac{I_x}{\bar{y}} = 3,301\text{mm}^3
\]

- Nominal flexural strength

Since the section is not subject to flexural-torsional buckling due to presence of sheathings and noggins, the nominal flexural strength is derived as follows:
\[ M_n = S_e F_y = 3301 \times 350 = 1.155 \text{ kN.m} \]

- Nominal axial strength

Since the member can only buckle perpendicular to the x-axis:

\[ r_x = \sqrt{\frac{I_x}{A}} = 35.2 \text{ mm} \]

\[ F_e = \frac{\pi^2 E (\frac{K L_x}{r_x})^2}{(\frac{1 \times 1000}{35.2})^2} = 2507.2 \frac{N}{mm^2} \]

\[ \lambda_c = \sqrt{\frac{F_y}{F_e}} = \sqrt{\frac{350}{2507.2}} = 0.3736 < 1.5 \]

\[ F_n = (0.658 \lambda_c^2) F_y = 330.136 \frac{N}{mm^2} \]

Effective area at uniform compressive stress of \( f = 330.136 \frac{N}{mm^2} \):

\[ S = 1.28 \sqrt{\frac{29500}{330.136}} = 31.8963 \]

\[ \frac{w}{t} = 29.998 > 0.328 S \rightarrow \text{Check effective width of flange} \]

\[ I_a = 29.016 mm^4 \]

\[ R_l = 0.526 \]

\[ n = 0.347 > 1/3 \]

\[ A_e = 73 mm^2 \]

\[ P_n = A_e F_n = 24 kN \]
B. Finite strip method

As previously explained the developed FSM code was implemented to find the local buckling loads involved. As shown in Figure 5-15, in case of application of pure axial load, the load factor for local buckling (70mm length) is 0.2078 and for distortional buckling (350mm length) is 0.51328. Figure 5-16 presents the local buckling (50 mm length) and distortional buckling (350 mm length) load factors under pure bending moment to be 1.0787 and 1.1964, respectively. Applying the obtained load factors, the local, distortional and global axial forces and bending moments can be calculated as explained later in this chapter.

![Figure 5-15- Buckling load factors under pure axial stress calculated by FSM code](image)

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Figure 5-16- Buckling load factors under pure bending moment calculated by FSM code

Under pure axial load:

\[ P_{cr} = P_y = 44,625N \]

\( L=70\text{mm} \):

\[ \frac{P_{cr}}{P_y} = 0.2078 \rightarrow P_{crl} = 0.2078 \times 350 \times 127.5 = 9,273.075N \]

\( L=350\text{mm} \):

\[ \frac{P_{cr}}{P_y} = 0.51328 \rightarrow P_{crd} = 0.51328 \times 350 \times 127.5 = 22,905.12N \]

Under pure bending moment:

\[ M_y = 1,250,763.9N.\text{mm} = 1.251kN.\text{m} \]

\( L=50\text{mm} \):

\[ \frac{M_{cr}}{M_y} = 1.0787 \rightarrow M_{crl} = 1.0787 \times 1,250,763.9 = 1,349,199.02N.\text{mm} = 1.349kN.\text{m} \]

\( L=350\text{mm} \):

\[ \frac{M_{cr}}{M_y} = 1.1964 \rightarrow M_{crd} = 1.1964 \times 1,250,763.9 = 1,496,413.93N.\text{mm} = 1.496kN.\text{m} \]
C. Direct strength method

By having the buckling loads calculated using the FSM code, the direct strength method is applied as follows to obtain the nominal axial load and bending moments.

To calculate the ultimate loads, a $\varphi = 0.8$ is multiplied by the nominal loads as per the code requirements. The resulting design curve for a C shaped stud is presented in Figure 5-17.

Under pure axial load (Compression member):

\[
P_{ne} = P_y = 350 \times 127.5 = 44,625N = 44.6kN
\]

\[
\lambda_l = \frac{P_{ne}}{P_{crl}} = 2.19 > 0.776 \rightarrow P_{nl} = \left(1 - 0.15\left(\frac{P_{crl}}{P_{ne}}\right)^{0.4}\right)\left(\frac{P_{crl}}{P_{ne}}\right)^{0.4}P_{ne} = 21.9kN
\]

\[
\lambda_d = \frac{P_y}{P_{crd}} = 1.39 > 0.561 \rightarrow P_{nd} = \left(1 - 0.25\left(\frac{P_{crd}}{P_y}\right)^{0.6}\right)\left(\frac{P_{crd}}{P_y}\right)^{0.6}P_y = 24.9kN
\]

\[
P_n = \min(P_{nl}, P_{nd}, P_{ne}) = 21.9kN
\]

\[
P_u = \varphi P_n = 17.52kN
\]

Under pure bending moment (Flexural member):

\[
M_{ne} = M_y = 1.251kN.m
\]

\[
\lambda_l = \frac{M_{ne}}{M_{crl}} = \sqrt{\frac{1.251}{1.349}} = 0.9273 > 0.776 \rightarrow M_{nl} = \left(1 - 0.15\left(\frac{M_{crl}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{crl}}{M_{ne}}\right)^{0.4}M_{ne} = 1.090kN.m
\]

\[
\lambda_d = \frac{M_y}{M_{crd}} = \sqrt{\frac{1.251}{1.349}} = 0.914 > 0.673 \rightarrow M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5}M_y = 1.039kN.m
\]

\[
M_n = \min(M_{nl}, M_{nd}, M_{ne}) = 1.0391kN.m
\]

\[
M_u = \varphi M_n = 0.83 kN.m
\]
D. Finite element method

To verify the buckling loads obtained from FSM code, a FEM model of the C shaped stud is evaluated using a FEM model. The resulting local and distortional buckling loads are 9.35 kN (Figure 5-18(a)) and 20.23 kN (Figure 5-18(b)), respectively, which are consistent with the FSM results.

![Figure 5-18- FEM results of a C shaped stud buckling analysis: a) local and b) distortional buckling](image)

Section 2: double C92-30-075

For double-C section studs, the buckling loads are calculated using the FSM code as follows.
Under pure axial load (Compression member):

\[ P_{cr} = P_y = 89.250N = 89.25\, kN \]

L=80mm:

\[ \frac{P_{cr}}{P_y} = 0.6826 \rightarrow P_{cr} = 0.68 \times 350 \times 255 = 60,922.94N = 60.9\, kN \]

L=300:

\[ \frac{P_{cr}}{P_y} = 1.086 \rightarrow P_{cr} = 1.086 \times 350 \times 255 = 96925.5N = 96.9\, kN \]

\[ \lambda_l = \sqrt{\frac{P_{ne}}{P_{cr}}} = \sqrt{\frac{89250}{60922.94}} = 1.21 > 0.776 \rightarrow P_{nl} = \left(1 - 0.15\left(\frac{P_{cr}}{P_{ne}}\right)^{0.4}\right)\left(\frac{P_{cr}}{P_{ne}}\right)^{0.4}P_{ne} \]

\[ = 66.744\, kN \]

\[ \lambda_d = \sqrt{\frac{P_y}{P_{cr}}} = 0.96 > 0.561 \rightarrow P_{nd} = \left(1 - 0.25\left(\frac{P_{cr}}{P_y}\right)^{0.6}\right)\left(\frac{P_{cr}}{P_y}\right)^{0.6}P_y = 69.144\, kN \]

\[ P_n = \min(P_{nl}, P_{nd}, P_{ne}) = 66.744\, kN \]

\[ P_u = \phi P_n = 53.4\, kN \]

Under pure bending moment (Flexural member):

\[ M_y = 2,501,527.8\, N.\, mm = 2.5\, kN.\, m \]

L=30mm:

\[ \frac{M_{cr}}{M_y} = 1.7779 \rightarrow M_{cr} = 1.7779 \times 2,501,527.8 = 4.444\, kN.\, m \]

L=300mm

\[ \frac{M_{cr}}{M_y} = 1.9029 \rightarrow M_{cr} = 1.9029 \times 2,501,527.8 = 4.757\, kN.\, m \]

\[ M_{ne} = M_y = 2.5\, kN.\, m \]

\[ \lambda_l = \frac{M_{ne}}{M_{cr}} = 0.74997 < 0.776 \rightarrow M_{nl} = M_{ne} = 2.5\, kN.\, m \]
\[
\lambda_d = \frac{M_y}{M_{crd}} = \sqrt{\frac{1.251}{1.349}} = 0.7249 > 0.673 \rightarrow P_{nd} = \left(1 - 0.22 \left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y
\]

\[
= 2.4 \text{kN}.m
\]

\[
M_n = \min(M_{nl}, M_{nd}, M_{ne}) = 2.4 \text{kN}.m
\]

\[
M_u = \varphi M_n = 1.92 \text{kN}.m
\]

The resulted design curve for a double-C CFS stud is presented in Figure 5-19.

![Figure 5-19- Design curve for a double-C CFS stud](image)

To design the CFS studs, a summary of maximum design loads for both sections are presented in Table 5-6 and Figure 5-20.
Table 5-6: The maximum axial bending moment capacity of the CFS studs

<table>
<thead>
<tr>
<th></th>
<th>$P_{nl}$ (kN)</th>
<th>$P_{nd}$ (kN)</th>
<th>$P_{ne}$ (kN)</th>
<th>$P_n$ (kN)</th>
<th>$M_{nl}$ (kN.m)</th>
<th>$M_{nd}$ (kN.m)</th>
<th>$M_{ne}$ (kN.m)</th>
<th>$M_n$ (kN.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sing C stud</td>
<td>21.9</td>
<td>24.9</td>
<td>44.62</td>
<td>21.9</td>
<td>1.09</td>
<td>1.039</td>
<td>1.25</td>
<td>1.039</td>
</tr>
<tr>
<td>Double C stud</td>
<td>66.74</td>
<td>69.14</td>
<td>89.25</td>
<td>66.74</td>
<td>2.5</td>
<td>2.4</td>
<td>2.5</td>
<td>2.4</td>
</tr>
</tbody>
</table>

Once the design curves are obtained, the two typical CFS studs are allocated according to the design wind load and FE results for axial forces obtained from the 4-storey building analysis. For the risk category I building considered in this study, basic wind speed extracted from the code is $V=45$ m/s. The wind directionality factor, $K_d$, is 0.85. An urban area with surface roughness category of B and exposure category of B is considered for the case. Since the openings are going to exceed 1 percent of the gross area of the walls, the structure is classified as a partially enclosed building. As per Table 26.11-1 of ASCE7 [81], the internal pressure coefficient, $G_{pi}$, is $\pm 0.55$. The building is considered to be on a flat area leading to a topographic factor, $K_{zt}$, of 1. According to section 26.2 of the code, the structure is determined
as a low rise building which is a rigid building with gust effect factor of, \( G = 0.85 \). Velocity pressure exposure coefficients, \( K_h \) and \( K_z \) are calculated in Table 5-7. According to the parameters determined above, the velocity pressure, \( q_z \), shall be calculated as:

\[
q_z = 0.613K_x K_z t K_d V^2
\]

<table>
<thead>
<tr>
<th>h(m)</th>
<th>( V ) (m/s)</th>
<th>( K_d )</th>
<th>( GC_{pi} )</th>
<th>( K_{zt} )</th>
<th>( K_h )</th>
<th>( \alpha )</th>
<th>( Z_g )</th>
<th>( q_z ) (N/m²)</th>
<th>( q_z ) (N/m²)</th>
<th>( P ) (N/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Int</td>
<td>Ext</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.5</td>
<td>45</td>
<td>0.85</td>
<td>0.55</td>
<td>0.68</td>
<td>1</td>
<td>0.756</td>
<td>7</td>
<td>365.8</td>
<td>772.2</td>
<td>797.68</td>
</tr>
<tr>
<td>4.5</td>
<td>45</td>
<td>0.85</td>
<td>0.55</td>
<td>0.68</td>
<td>1</td>
<td>0.756</td>
<td>7</td>
<td>606.3</td>
<td>772.2</td>
<td>851.02</td>
</tr>
<tr>
<td>7.5</td>
<td>45</td>
<td>0.85</td>
<td>0.55</td>
<td>0.68</td>
<td>1</td>
<td>0.756</td>
<td>7</td>
<td>698.5</td>
<td>822.9</td>
<td>913.71</td>
</tr>
<tr>
<td>10.5</td>
<td>45</td>
<td>0.85</td>
<td>0.55</td>
<td>0.68</td>
<td>1</td>
<td>0.756</td>
<td>7</td>
<td>769</td>
<td>861.7</td>
<td>961.64</td>
</tr>
</tbody>
</table>

According to loading span of \( S = 0.6 \)m per each cold formed steel stud, the maximum uniformly distributed load on each external wall stud is calculated as (Table 5-8):

\[
q = PS = 961.64 \times 0.6 = 577 \text{ (N/m)} = 0.577 \text{ (kN/m)}
\]

The uniformly distributed load imposes a maximum bending moment in mid span of the studs as follows:

\[
M = 0.577 \times 3^2/8 = 0.649 \text{ kN.m}
\]

The bending moment developed in the free length of the exterior wall studs due to wind load should be compared to the bending moment capacity of the single and double C studs to evaluate the maximum allowable axial compression load on it.
To design the cold formed steel studs, according to ASCE7, the following two combinations including wind loads are applied to the structure while the local wind effect is applied as a uniformly distributed load to every individual stud:

$1.2D + 1W + 1L + 0.5L_R$

$0.9D + 1W$

Where:

$D$ is dead load, $W$ is wind load, $L$ is live load, and $L_R$ is roof live load.

Since the variation of wind load is negligible in stories, the maximum wind load is considered for all studs in every load combination. According to the maximum axial force and bending moment applied to every stud, they are compared to the curves shown in Figure 5.20 for a single stud. The single C studs placing above the graph, are changed to double C studs and finally compared to the upper limit line to evaluate the adequacy of the section’s capacity.

5.9. Design of the 4-storey building

Having assigned the manually designed CFS studs to the structure, the second order effect on shears and moments is considered and the storey drifts are compared to the first order analysis results. As a final control, the maximum storey drifts, taking the

<table>
<thead>
<tr>
<th>P(kN/m²)</th>
<th>Distributed load (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Int</td>
</tr>
<tr>
<td></td>
<td>0.772</td>
</tr>
<tr>
<td></td>
<td>0.772</td>
</tr>
<tr>
<td></td>
<td>0.823</td>
</tr>
<tr>
<td></td>
<td>0.862</td>
</tr>
</tbody>
</table>

Table 5.8- Wind pressure and equivalent uniformly distributed load on each stud
notional loads into account, are compared to the allowable storey drifts. Having assured that the storey drifts are within the allowable range, this can be considered as the final design of the structure with application of HWPS.

Table 5.9 displays the final design in every storey of the building.
Table 5-9- Plan view of every storey with the number of HRS panels in every direction

<table>
<thead>
<tr>
<th>Storey</th>
<th>X direction</th>
<th>Y direction</th>
<th>Plan view for the structure with gusset plates***</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Gusset</td>
<td>Knee</td>
<td>Gusset</td>
</tr>
<tr>
<td>Storey 4</td>
<td>13</td>
<td>12</td>
<td>11</td>
</tr>
<tr>
<td>Storey 3</td>
<td>17</td>
<td>15</td>
<td>15</td>
</tr>
<tr>
<td>Storey 2</td>
<td>20</td>
<td>19</td>
<td>17</td>
</tr>
<tr>
<td>Storey 1</td>
<td>22</td>
<td>20</td>
<td>19</td>
</tr>
</tbody>
</table>

* Hot rolled steel panel:  
** Gravity only walls:  

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5.10. Cost Comparison

Since the HWPS allows for light weight floor system, the total weight of the structure is significantly lower than that of the moment resisting frame. Consequently, the base shear is reduced leading to lighter lateral force resisting elements. On the other hand, since the CFS studs are involved as a part of the gravity load resisting system for HWPS, the second order analysis results in smaller moment in members which again causes reduced size of the members. A comparison between the total amount of steel consumed for the 4-storey building with two different LFRSs (HWPS and intermediate moment resisting frame) is represented in Table 5-10.

<table>
<thead>
<tr>
<th></th>
<th>Intermediate moment resisting frame</th>
<th>HWPS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hot rolled steel weight (kN)</td>
<td>397.5</td>
<td>192.5</td>
</tr>
<tr>
<td>Cold formed steel weight (kN)</td>
<td>23.7</td>
<td>44.4</td>
</tr>
<tr>
<td>Total weight of steel (kN)</td>
<td>421.2</td>
<td>236.9</td>
</tr>
<tr>
<td>Total weight of structure (kN)</td>
<td>3894</td>
<td>2363</td>
</tr>
</tbody>
</table>

5.11. Conclusion

This chapter provided a practical design method for an innovative wall panel system. The lateral behaviour of individual panels was experimentally and numerically evaluated and according the preliminary design and analysis, the final design for a partic-
ular building with a given plan is provided. The proposed system is successfully capable of extending the number of stories to four. Key findings regarding the behaviour of the HWPS system and the design recommendations are as follows:

1. In individual HWPS experiments, the plastic hinges formed in gusset plates of the lower beam which was also observed in the FE analysis. This deformation mode shows that with no plastic hinges in beams and columns, the panel’s stiffness can still be improved by using better designed bracing elements.

2. Although the HWPS gained the maximum lateral strength of the bare HRS panel, due to relatively lower stiffness, the amount of dissipated energy in hysteretic loops is relatively low.

3. Application of the simultaneous vertical and lateral load did not affect the hysteretic behaviour as expected. This is due to relatively higher stiffness of the HRS panel compared to the CFS one. Because the vertical load is being applied through a HRS top beam (which can be assumed as rigid), the vertical load is proportionally distributed between vertical elements, while the entire lateral load is resisted by the HRS part.

4. For the analysis process of the 4 storey building, the cold formed studs are bearing a portion of the vertical loads, which depends on the configuration of HRS and CFS adjacent columns. This means that the size and number of CFS members can be optimised by choosing an optimum configuration of HRS and CFS parts. On the other hand, the main role of HRS panels is to resist lateral loads and provide lateral stiffness. This leads to a constant number of HRS panels required in every storey and every direction. Therefore, by considering proper arrangements of HRS and CFS parts, one can get a better design of CFS members with the same number of HRS members.
For future research, this study is being followed by a research on some construction details for improving energy dissipation of the current system, a new HWPS stiffened with knee elements, and investigation on the detailed criteria to be met for providing the required rigidity for the floors for this system.
Chapter 6 Concluding remarks and future work
6.1. Summary and Conclusion

The main objectives of this study were to investigate the performance of a hybrid Cold-Formed steel/Hot-Rolled steel structural system subjected to cyclic loads and to perform a study into the use of this construction system in mid-rise structures. The study was divided into six chapters with the following briefly described conclusions:

Chapter 1 is dedicated to an introduction to the CFS structures and their increasing application in building industry. This chapter also addresses the research methodology used for the current study and how the structure of this thesis is formed.

Chapter 2 provides a review of the existing research trends in CFS construction and aims to discover the existing gaps in available studies. The chapter summarises the most related literature in a table, representing the improvements suggested by each study for lateral performance of CFS structures. It finally concludes that although this is not a new concept in building industry; the use of CFS sections as a lateral load resisting system is a new concept and still there are gaps in guidelines and seismic design process. In addition, there is a very limited background available on application of HRS and CFS acting as a hybrid lateral load resisting system.

Chapter 3 introduces CFS design tools. Finite Strip Method (FSM) is explained as a method of buckling load evaluation for an arbitrary CFS section and a Code in MATLAB Software is developed and verified based on this method. Direct Strength Method (DSM) is used later in this chapter to calculate the nominal buckling loads using the section buckling loads provided by FSM. Later in Chapter 5 of the thesis, where a 4-storey building is analysed and designed, the FSM code is used to design the CFS studs of the building.
Chapter 4 describes the experimental study, starting from the testing rig design and manufacture process continued by full scale tests and Finite Element Analysis results of the proposed hybrid systems. This part of study considers the monotonic and cyclic response of the system. According to observations during the test and the associated results, following are the extracted key conclusions: Firstly, a complete load transfer path from CFS panel to the HRS frame is to be assured because the HRS frame is the main lateral load resisting element. Secondly, significant local deformations can be avoided by enhancing the vulnerable locations such as CFS to HRS attachment points. This enhancement results in an improved overall lateral behaviour of the panel; however, the pinched hysteretic loop, which is a result of the screw tilting, may not be avoided by this enhancement. Lastly, from construction point of view, the hybrid panel should be delivered in two individual pieces: HRS panel and CFS panel. The two parts then can be attached on site to form a complete system. This also offers a better manoeuvrability since it can be manually handled and no lifting equipment are required.

Chapter 5 provides a feasible design process the proposed hybrid system. Having evaluated the behaviour of individual panels, their application to a multi-storey structure is investigated in this chapter. A structure with a specific given plan is considered here as a case study. The results showed that the system is successfully capable of being extended to four stories.

To summarise, in this thesis the following objectives are attained:

- A comprehensive background study and literature review on the theories and standards related to cold-formed steel design and their lateral load resisting systems.

- Experimental studies on the proposed hybrid wall panel system (HWPS) and suggesting the possible improving amendments.
- Experimental studies on the improved hybrid wall panel system.

- Developing and validating numerical models with finite element and finite strip methods.

- Experimental studies on laterally stiffened hybrid wall panel system for higher number of stories where a HRS profile acts as a collector element.

- Finite element analysis of a multistorey structure with the proposed lateral load resisting system.

- Design of the multistorey structure with the proposed system.

- Finite strip method code development to determine the axial capacity of cold formed steel sections used in the multistorey structural design.

- Cost analysis on the multistorey building comparing the traditional and proposed systems

6.2. Suggestions for future research

The experimental and numerical studies provided in this thesis were able to satisfactorily address the predefined objectives of the research. However, there are still some areas which require further research both in experimental and numerical fields of research.

6.2.1 Lateral load resisting capacity of Cold-Formed Steel in combination with Hot-Rolled steel structures

As previously stated in Chapter 2, there have been several attempts to improve the seismic performance of CFS structural system by different bracing or sheathing configurations. However, there is minimal background available on hot rolled-cold formed steel hybrid structures. Therefore, further research studies with the approach
of CFS-HRS hybrid action, considering all contributing factors are essential. Factors such as: HRS part to CFS part lateral stiffness ratio, HRS part to CFS part maximum lateral load bearing capacity, type and arrangement of CFS to HRS attachments, level of rigidity of floor system.

6.2.2 Optimising the CFS stud sections

As described in Chapter 3 of this thesis, a FSM Code in MATLAB Software is developed to evaluate buckling capacity of the CFS profiles. Two typical CFS profiles are used in this study; however, developing an optimisation code, which evaluates the optimum CFS section according to the required buckling capacity, can be helpful for reducing the size of the CFS sections. The reduction in CFS section size leads to lower weight of structure and consequently reduced lateral forces during seismic events which again causes a reduced HRS lateral load resisting system.

6.2.3 Coupling effects of seismic loading and fire exposure

For a multi-storey building, the response of the proposed hybrid system to the coupling effects of seismic excitation and fire exposure can be studied in a future work by other researchers.

6.2.4 Effects of high axial loads

In a following research, the effects of high axial loads on the supporting structure can be studied.
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