

Department of Civil and Environmental Engineering

Imperial College London

Simulation and behaviour of single-span steel portal frames

Aya Ahmed

A thesis submitted for the degree of

Doctor of Philosophy

September 2023, London

فأقصد إلى قلم الأشياء تدركها

تجري الرياح كما رادت لها السفن

Declaration of originality

I confirm that this thesis is the result of my own work, and I have appropriately cited and referenced all materials sourced from other authors and publications. This thesis is submitted to Imperial College London for the degree of Doctor of Philosophy and has not been submitted for a degree or any other qualification at another university.

Aya Ahmed

Copyright declaration

The copyright of this thesis rests with the author. Unless otherwise indicated, its contents are licensed under a Creative Commons Attribution Non-Commercial 4.0 International Licence (CC BY-NC). Under this licence, you may copy and redistribute the material in any medium or format. You may also create and distribute modified versions of the work. This is on the condition that: you credit the author and do not use it, or any derivative works, for a commercial purpose. When reusing or sharing this work, ensure you make the licence terms clear to others by naming the licence and linking to the licence text. Where a work has been adapted, you should indicate that the work has been changed and describe those changes. Please seek permission from the copyright holder for uses of this work that are not included in this licence or permitted under UK Copyright Law.

Abstract

Portal frames are a common structural system in single-storey industrial and commercial buildings. In recent decades, the design of portal frames has evolved towards larger structures with stiffer cladding systems, introducing structural challenges such as increased frame flexibility and higher demand on connections. These concerns were highlighted by a SCOSS alert in 2018 (SCOSS, 2018), emphasising the need to reevaluate the current design recommendations. The effect of scale on the overall behaviour of steel portal frames and their connecting elements and fasteners is therefore investigated in this study.

Finite element models were employed to conduct the investigation. The modelling approaches for typical components of steel portal frames are described and validated. These models account for the nonlinear material stress-strain responses, the load-displacement characteristics of portal frame connections and the behaviour of portal frame cladding systems. The validated key components were utilised to develop a full portal frame FE model, which was then validated and used in a parametric study to investigate the effect of scale of single-span portal frames. Wind sway was found to be the most critical load case, resulting in substantial shear displacements of the cladding screw connectors, reaching up to 20 mm at 1.5 times the serviceability load levels. As for the effect of scale, the increase in height was found to exacerbate the shear displacements of the cladding screw connectors under wind sway load. In contrast, the increase in span increased the shear displacements of the cladding screw connectors under both gravity load and thermal action. However, the change in frame length had a minimal effect. Overall, the demand on the connector elements in modern steel portal frames have been shown to be high under some realistic loading scenarios, particularly in the end bays, prompting the need to consider suitable means of mitigation in future research.

Acknowledgements

I would like to extend my gratitude to my supervisors, Professor Leroy Gardner and Dr. Fiona Walport, for their exceptional guidance, support, and mentorship throughout this research journey. Their dedication has been invaluable in shaping the course of this work.

I am thankful to Haoyue Yuan, Tong Hu, and Xiaobo Zhu for their remarkable contributions to this research. Their hard work and commitment have significantly enriched the project, and I am grateful for their collaboration.

I would also like to express my appreciation to David Moore, Ana Girao-Coelho, Mike Banfi, John Harrison, and Dan Thompson, who have been providing continuous support and valuable insights as part of the steering committee for this research. Their guidance and feedback have been invaluable in refining the research objectives and methodologies.

Furthermore, I wish to express my heartfelt thanks to my parents, whose unwavering support, encouragement, and sacrifices have been the bedrock of my journey. Without them, I would not be where I am today.

I am also grateful for the financial support provided by the Schlumberger Foundation (Faculty for the Future) and the British Constructional Steelwork Association (BCSA). Their generous support has enabled me to pursue this research and contribute to the field.

I extend my warmest thanks to my husband, Luai, and my daughters, Alaa and Lujain, for their unconditional support, patience, and understanding throughout this challenging journey. Their encouragement has been my constant motivation.

Finally, I will forever be grateful to Ibtiha, who has made me a better version of myself.

Publications

The peer-reviewed journal articles and conference proceeding papers that have originated from the work presented in this thesis are listed below:

Journal publications

- Ahmed, A., Zhu, X., Hu, T., Walport, F. & Gardner, L. (2023). Simulation and behaviour of single-span portal frames: Part I - Model development and validation, (in preperation).
- Zhu, X., Ahmed, A., Walport, F. & Gardner, L. (2023). Simulation and behaviour of single-span portal frames: Part I I – Parametric analysis and practical implications, (in preperation).

Conference publications

- Ahmed, A., Hu, T., Walport, F. & Gardner, L. (2022). Numerical modelling of single-span portal frames. In *The International Colloquium on Stability and Ductility of Steel Structures (SDSS)*, Aveiro, Portugal, September, 2022.

Contents

Declaration of originality	I
Copyright declaration	II
Abstract	III
Acknowledgements	IV
Publications	V
List of Figures	IX
List of Tables	XV

1 Introduction-----	1
1.1 Background to the Research -----	1
1.2 Research Aim and Objectives -----	4
1.3 Thesis Overview -----	5
2 Literature review-----	6
2.1 Introduction-----	6
2.2 Types of steel portal frames -----	6
2.3 Steel portal frame components -----	8
2.3.1 Primary steel frame -----	8
2.3.2 Secondary steelwork -----	9
2.3.3 Cladding systems -----	11
2.4 Portal frame connections -----	14
2.4.1 Primary frame connections-----	15

2.4.2	Secondary steelwork connections-----	17
2.4.3	Cladding connections -----	20
2.5	Design of steel portal frames -----	21
2.6	Effect of stressed skin action in steel portal frames -----	23
2.7	Concluding remarks -----	26
3	Finite element modelling of frame components: fundamentals and validation-----	27
3.1	Introduction-----	27
3.2	Fundamental aspects of finite element modelling -----	27
3.2.1	Material modelling-----	28
3.3	Connections modelling and validation-----	33
3.3.1	General-----	33
3.3.2	Modelling approach for bolts -----	33
3.3.3	Primary frame bolted connections -----	37
3.3.4	Secondary steelwork bolted connections -----	39
3.3.5	Cladding primary connections-----	41
3.3.6	Cladding secondary connections -----	44
3.4	Cladding modelling and validation-----	46
3.4.1	Single-skin trapezoidal sheeting-----	46
3.4.2	Sandwich panel cladding -----	50
3.5	Concluding remarks-----	53
4	Full frame finite element modelling: fundamentals and validation-----	54
4.1	Introduction-----	54
4.2	Fundamental aspects of finite element modelling -----	54
4.2.1	Boundary conditions -----	55
4.2.2	Connection details -----	57

4.3	Validation -----	67
4.3.1	Five bay portal frame-----	67
4.3.2	Two bay portal frame -----	71
4.4	Concluding remarks -----	76
5	Full portal frame parametric study-----	77
5.1	Introduction-----	77
5.2	General -----	77
5.3	Frame cases modelled-----	77
5.3.1	General-----	77
5.3.2	Loading combinations-----	80
5.3.3	Key aspects under investigation-----	82
5.4	Influence of scale -----	83
5.4.1	Effect of span -----	83
5.4.2	Effect of height-----	114
5.4.3	Effect of length-----	139
5.4.4	Discussion and summary -----	147
5.5	Concluding remarks -----	154
6	Conclusions and further research-----	155
6.1	Conclusions -----	155
6.2	Suggestions for further work -----	157
	References -----	158

List of Figures

1.1	Newton Heath rail maintenance depot constructed as a single-span steel portal frame (NSC, 2020) -----	1
1.2	HS2 precast manufacturing facilities constructed as a multi-span steel portal frame (NSC, 2021) -----	2
1.3	Illustration of the typical components of a single-span portal frame -----	2
1.4	Diaphragm action in (a) a pinned rectangular structure and (b) a rigid pitched roof structure (Bryan, 1976) -----	3
2.1	Illustration of a typical single-span portal frame -----	7
2.2	Illustration of a typical "hit and miss" multi-span portal frame -----	8
2.3	Components of the primary steel frame -----	9
2.4	Components of the secondary steelwork -----	10
2.5	A typical two-skin built-up cladding system with Z spacers -----	12
2.6	A two-skin built-up cladding system utilising cassettes -----	13
2.7	A typical two-skin sandwich panel cladding system -----	14
2.8	Typical arrangement of (a) eaves and (b) apex joints in steel portal frames -----	15
2.9	Typical nominally pinned column base -----	17
2.10	Typical arrangement for (a) gable post cleat connection and (b) longitudinal bracing connection -----	18
2.11	Typical arrangement for (a) a purlin cleat connection, (b) a sleeved purlin connection and (c) an eaves beam cleat connection -----	19
2.12	Typical diagonal stay connection -----	19
2.13	Primary cladding screws for (a) two-skin built-up system with Z spacer, (b) two-skin built-up system with "bar and bracket" spacer and (c) sandwich panel system -----	21
3.1	Quad-linear material model and typical experimental stress-strain curve (Yun & Gardner, 2017) -----	30

3.2	Typical cold-formed steel stress-strain curve -----	31
3.3	Generic load-deformation model for (a) bolts in tension and (b) bolts under combined shear and bearing -----	36
3.4	Comparison of load-deformation curves for hot-rolled steel bolted T-stub and lap-shear connections from FE model and tests reported in Coelho et al. (2004) and Draganić, Dokšanović and Markulak (2014), respectively -----	39
3.5	Comparison of load-deformation curves for secondary steelwork bolted lap-shear connections from FE model and tests reported in Chung and Ip (2000) and Rinchen and Rasmussen (2019), respectively -----	41
3.6	(a) Components of a sandwich panel connection, (b) Sandwich panel primary screw connectors, (c) Proposed generic tri-linear model for load-deformation behaviour of sandwich panel primary screw connectors in combined shear and bearing, (d) Failure criteria of the sandwich panel primary screw connectors in the axial direction -----	43
3.7	(a) Proposed generic load-deformation behaviour of cladding secondary screw connectors under combined shear and bearing, (b) comparison of load-deformation curves for cladding secondary screw connectors from FE model and tests reported in Käßlein and Misiak (2011) -----	45
3.8	RVE of corrugated sheet and orthotropic plate in adopted coordinate systems, with the y axis into the page -----	47
3.90	Validation of modelling technique used herein for trapezoidal sheeting -----	50
3.10	Validation of modelling technique used herein for sandwich panel cladding system against experimental results of Kunkel and Lange (2015)-----	52
4.1	Typical components of full frame FE model -----	56
4.2	Primary steel frame connections -----	58
4.3	Typical arrangement for primary frame connections -----	59
4.4	Secondary steelwork connections -----	60
4.5	Typical arrangement for secondary steelwork connections -----	61
4.6	Primary cladding screw connections -----	63
4.7	Summary of cladding primary screw arrangements -----	64
4.8	Cross-section of the exposed gutter system -----	65
4.9	Cleader angle to purlin connection -----	65
4.10	Longitudinal side lap seam screw connections -----	66
4.11	End lap seam screw connections -----	66

4.12	Metal flashings seam screw connections -----	67
4.13	Details of the testing arrangement for the frames presented in Davies and Bryan (1982) and Roberts, Davies & Wang (2021) -----	68
4.14	Comparison of load-displacement curves for load case 1 of clad and bare frames, respectively, obtained from the FE model and the tests reported in Davies and Bryan (1982) and Roberts, Davies & Wang (2021) -----	71
4.15	Details of the testing arrangement for the frames reported in Davies et al. (1990) and Junid (1992) -----	73
4.16	Comparison of load-displacement curve for the 12 m span frame obtained from FE model and the test reported in Davies et al. (1990) -----	75
4.17	Comparison of load-displacement curve for the 24 m span frame obtained from FE model and the test reported in Junid (1992) -----	75
4.18	Failure mode of the 12 m span frame -----	76
4.19	Failure mode of the 24 m span frame -----	76
5.1	Illustration of the variation of (a) span, (b) eaves height and (c) structure length considered in the study -----	78
5.2	Load combinations -----	81
5.3	Influence of portal frame span on gravity load distribution between frames -----	84
5.4	Deflected shape of a portal frame under gravity load -----	85
5.5	Influence of portal frame span on the relative shear displacement of flashing seam screws under gravity load -----	86
5.6	Relative load-displacement behaviour of the critical flashing seam screw under gravity load -----	87
5.7	Influence of portal frame span on the relative shear displacement of longitudinal and end lap seam screws under gravity load -----	88
5.8	Illustration of (a) Relative shear displacement of longitudinal and end lap seam screws and (b) horizontal displacement of roof cladding panels under gravity load --	89
5.9	Relative load-displacement behaviour of the critical longitudinal/end lap seam screw under gravity load -----	90
5.10	Influence of portal frame span on the relative shear displacement of internal sheet connectors under gravity load -----	91
5.11	Relative load-displacement behaviour of the critical internal sheet connector under gravity load -----	92

5.12	Influence of portal frame span on the relative shear displacement of external sheet connectors under gravity load -----	93
5.13	Relative load-displacement behaviour of the critical external sheet connectors under gravity load -----	94
5.14	Influence of portal frame span on wind load distribution between frames -----	96
5.15	Influence of portal frame span on the relative shear displacement of flashing seam screws under wind sway load -----	97
5.16	Influence of portal frame span on the relative shear displacement of longitudinal and end lap seam screws under wind sway load -----	99
5.17	Illustration of (a) Relative shear displacement of longitudinal and end lap seam screws and (b) vertical displacement of gable cladding panels under wind sway load- -----	101
5.18	Relative load-displacement behaviour of the critical longitudinal/end lap seam screw under wind sway load -----	101
5.19	Influence of portal frame span on the relative shear displacement of internal sheet connectors under wind sway load -----	102
5.20	Relative load-displacement behaviour of the critical internal sheet connector under wind sway load -----	104
5.21	Influence of portal frame span on the relative shear displacement of external sheet connectors under wind sway load -----	105
5.22	Influence of portal frame span on the relative shear displacement of flashing seam screws under thermal action -----	107
5.23	Relative shear load-displacement behaviour of the critical flashing seam screw under thermal action -----	108
5.24	Influence of portal frame span on the relative shear displacement of longitudinal and end lap seam screws under thermal action -----	109
5.25	Relative load-displacement behaviour of the critical longitudinal/end lap seam screw under thermal action -----	110
5.26	Influence of portal frame span on the relative shear displacement of internal sheet connectors under thermal action -----	111
5.27	Relative load-displacement behaviour of the critical internal sheet connector under thermal action -----	112
5.28	Influence of portal frame span on the relative shear displacement of external sheet connectors under thermal action -----	113

5.29	Relative load-displacement behaviour of the critical external sheet connector under thermal action -----	113
5.30	Influence of portal frame height on gravity load distribution between frames -----	114
5.31	Influence of portal frame height on the relative shear displacement of flashing seam screws under gravity load -----	115
5.32	Influence of portal frame height on the relative shear displacement of longitudinal and end lap seam screws under gravity load -----	117
5.33	Influence of portal frame height on the relative shear displacement of internal sheet connectors under gravity load -----	120
5.34	Influence of portal frame height on the relative shear displacement of external sheet connectors under gravity load -----	122
5.35	Influence of portal frame height on wind load distribution between frames -----	124
5.36	Influence of portal frame height on the relative shear displacement of flashing seam screws under wind sway load -----	125
5.37	Influence of portal frame height on the relative shear displacement of longitudinal and end lap seam screws under wind sway load -----	127
5.38	Influence of portal frame height on the relative shear displacement of internal sheet connectors under wind sway load -----	129
5.39	Influence of portal frame height on the relative shear displacement of external sheet connectors under wind sway load -----	131
5.40	Influence of portal frame height on the relative shear displacement of flashing seam screws under thermal action -----	132
5.41	Influence of portal frame height on the relative shear displacement of longitudinal and end lap seam screws under thermal action -----	134
5.42	Influence of portal frame height on the relative shear displacement of internal sheet connectors under thermal action -----	136
5.43	Influence of portal frame height on the relative shear displacement of external sheet connectors under thermal action -----	138
5.44	Influence of portal frame structure length on gravity load distribution between frames under gravity load -----	140
5.45	Influence of portal frame structure length on the relative shear displacement of flashing seam screws under gravity load -----	141
5.46	Influence of portal frame structure length on the relative shear displacement of longitudinal and end lap seam screws under gravity load -----	143

5.47	Influence of portal frame structure length on the relative shear displacement of internal sheet connectors under gravity load -----	145
5.48	Influence of portal frame structure length on the relative shear displacement of external sheet connectors under gravity load -----	146
5.49	Influence of scale on flashing seam screws -----	148
5.50	Influence of scale on longitudinal and end lap seam screws -----	149
5.51	Influence of scale on the internal sheet connectors -----	151
5.52	Influence of scale on the external sheet connectors -----	152

List of Tables

3.1	Recommended representative values for strain hardening exponents n and m (Gardner & Yun, 2018) -----	32
3.2	Modelling techniques employed for bolted and screwed connections in steel portal frames -----	34
3.3	Modelling approach employed for the component behaviour of portal frame connections -----	35
3.4	Stiffness properties of an orthotropic plate representing a trapezoidal corrugated sheet -----	48
3.5	Stiffness properties of corrugated and equivalent orthotropic plate -----	48
3.6	Material properties for corrugated sandwich panel (Kunkel & Lange, 2015) -----	51
3.7	Material properties for equivalent orthotropic system -----	51
4.1	Comparison of horizontal displacements at the bottom of the eaves (results taken from frame 3 for the clad frame cases) -----	69
5.1	Summary of member sizes and connection details used in the parametric study -----	79
5.2	Load combinations considered in the parametric study -----	80
5.3	Summary of key aspects under investigation -----	82

Chapter 1

Introduction

1.1 Background to the Research

Portal frames are a common form of structural system that enable the creation of wide-span enclosures. These systems are designed with pitched roofs and are commonly used in single-storey industrial and commercial buildings where expansive areas and versatile functionality are required. Portal frames can be constructed as either a single- or multi- span structure, see Figures 1.1 and 1.2, with multi-span portal frames consisting of up to ten spans in common use in the United Kingdom. Typical multi-span portal frames utilise a “hit and miss” design, in which the number of internal columns is reduced to increase the internal clear space (Henderson, 2015).



Figure 1.1: Newton Heath rail maintenance depot constructed as a single-span steel portal frame (NSC, 2020)



Figure 1.2: HS2 precast manufacturing facilities constructed as a multi-span steel portal frame (NSC, 2021)

Portal frame structural systems consist of primary structural frames, secondary steelwork and a cladding system, with the key components illustrated in Figure 1.3. The primary frames are a series of parallel braced frames comprising rafters and columns, with each end frame commonly referred to as a gable frame. The secondary steelwork, which includes the purlins, side rails, eaves beams, anti-sag members, diagonal tie wires and diagonal stays, is used to support the cladding and provide out-of-plane stability for the primary frame members. Finally, the cladding system is used to create a building envelope and separate the enclosed area from the exterior environment, providing thermal and acoustic insulation, as well as restraint to the flanges of the attached secondary steelwork.

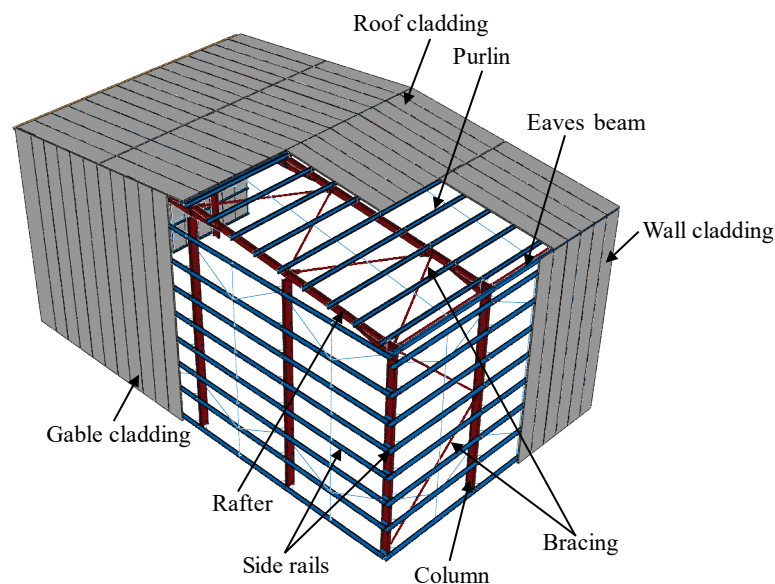


Figure 1.3: Illustration of the typical components of a single-span portal frame

Traditionally, in the design of portal frames, the three components of the structural system are designed independently based on simple assumptions for load transfer. In 1959, Godfrey and Bryan (1959) first reported the stiffening effect of the cladding when they noticed that the bending moments of an on-site full-scale clad steel frame were considerably lower than the design calculations. The observed values indicated that the metal roof sheeting had contributed to a stiffening effect in resisting the sway of the building. This effect was then referred to as stressed skin action. The basic principle of stressed skin action is that the cladding contributes to the in-plane stiffness and strength of a structure, acting as a stressed membrane commonly referred to as a diaphragm (Bryan, 1976). The diaphragm action of the roof panels transmits the load from the internal frames to the gable frames, as illustrated in Figure 1.4.

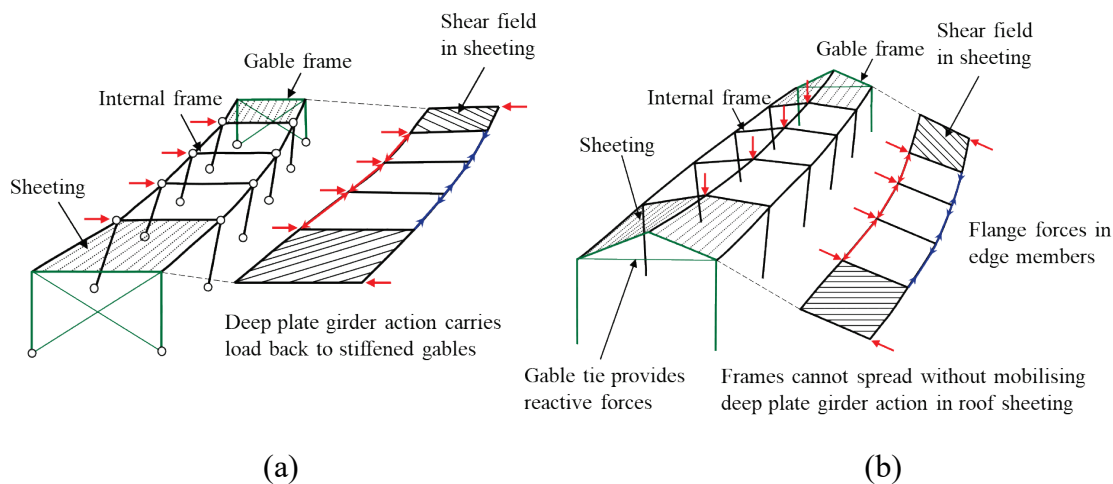


Figure 1.4: Diaphragm action in (a) a pinned rectangular structure and (b) a rigid pitched roof structure (Bryan, 1976)

Bryan investigated the effect of stressed skin action on semi-full scale frames, diaphragms and components (Bryan & El Dakhakhni, 1964; Bryan, 1964; Bates, Bryan & El-Dakhakhni, 1965; Bryan & El-Dakhakhni, 1968; Bryan, 1971a; Bryan, 1971b; Bryan & Mohsin, 1972). This led to the conclusion that utilising stressed skin design could save up to 25% of the weight of the primary steel frame (Bryan & Mohsin, 1972). The tests also led to the identification of potential failures in the connecting frame elements due to the parasitic stressed skin effect, i.e. unwanted stressed skin action (Bryan, 1972). It was concluded that the interaction between the cladding and the frame is always present and, depending on whether the stressed skin action was accounted for in the design or not, can be beneficial or detrimental to the overall structural response.

In 1982, the research into the stressed skin action was published in the book “Manual of stressed skin diaphragm design” (Davies & Bryan, 1982) which was taken as the basis to the current European recommendations (CEN, 2003; ECCS, 1995). However, despite the existence of these recommendations to account for the stressed skin action in the design of portal frames, it is still not currently taken into consideration in common practice. As a result, structural components are still being designed independently.

Over the past few decades, the design of portal frames has evolved in the direction of longer, lighter and taller structures. The cladding systems have also evolved from single-skin corrugated metal sheeting to a range of two-skin cladding systems. These two-skin systems consist of two metal sheets separated by a layer of insulation and are thus significantly stiffer than single-skin corrugated metal sheeting. While bigger frames allow for more versatile spaces, the increased size can also result in exacerbated structural challenges, such as increased frame flexibility resulting in a greater portion of the load being resisted by the stiff cladding. This can cause increased demand on the connecting elements and potentially result in localised failures and serviceability concerns. It can also result in leaking or detachment of the cladding and put the global stability of the structure at risk, as highlighted in a SCOSS (Standing Committee on Structural Safety) alert on the effects of scale in 2018 (SCOSS, 2018).

Although the scale of portal frames has grown, the design recommendations have remained the same. To ensure the safety and efficiency of portal frames, it is crucial for code provisions and industry guidance to adapt to the evolving climate within which the portal frames are created. This requires further understanding and research into the performance of these structural systems.

1.2 Research Aim and Objectives

The aim of this work is to investigate the above concerns, with a focus on the contribution of the cladding to the global stiffness and strength of portal frame structures and the resulting demand on the secondary steelwork and connections. The objectives of this research include:

- Establish a robust parametric FE model to investigate the effect of key parameters, including dimensions, the cladding arrangement and thermal actions on the overall structure performance and the distribution of load sharing between frames.
- In this study, particular attention is given to investigating the impact of scale and the relative flexibility between the frame and cladding on the connecting elements. As the frames

increase in size and flexibility, the cladding system tends to become more rigid, resulting in an increased relative flexibility that places higher demands on the connecting cleats and fasteners. The objective is to assess whether this larger relative flexibility has detrimental consequences on the connecting elements, and to draw informed conclusions based on findings.

- Explain the likely causes of the portal frame roof failures, identify probable causes, and propose mitigation strategies. To investigate the potential causes of portal frame failures associated with portal frame cladding, frames with varying dimensions and subject to different load cases are modelled. The investigation will serve as a basis for mitigation strategies to prevent similar failures from occurring the future.

1.3 Thesis Overview

In Chapter 2, a review of the background literature and research related to the topic is presented. The main components of portal frame systems are detailed with a particular focus on the components and connections of single-span portal frames. Additionally, the typical design approach for single-span single-storey steel portal frames is explained and research into the effect of stressed skin action on the behaviour of steel portal frames is summarised.

In Chapter 3, the fundamental aspects of the finite element modelling (FE) employed to simulate the steel portal frames connections and cladding are presented including element choice, material models and key aspects of the screw fasteners for the secondary elements. The modelling approaches for simulating cladding and different types of typical portal frame connections are then validated.

In Chapter 4, the fundamental aspects of the finite element modelling employed to model the full steel portal frames are presented. The full frame model is then validated against the test results from experimental programmes on full-scale single-span steel portal frames.

In Chapter 5, a parametric study is presented. Various portal frame sizes were modelled under different loading conditions to investigate the effect of stressed skin action, with particular focus on scale, on the overall behaviour of steel portal frames and their connecting elements.

In Chapter 6, the key findings and conclusions drawn from the research are summarised along with recommendations for future work on the topic.

Chapter 2

Literature review

2.1 Introduction

In this chapter, a review of the relevant background literature and research related to the thesis is presented. Firstly, steel portal frames are discussed with a particular focus on the components and connections of single-span portal frames. The typical design approach for single-span steel portal frames is then explained and, finally, research into the effect of stressed skin action on the behaviour of steel portal frames is summarised.

2.2 Types of steel portal frames

Portal frames are single-storey steel structures that are commonly used for buildings where large clear enclosed spaces are needed. The portal frame system is preferred for its efficiency, versatility and cost-effectiveness to achieve the building envelope requirements of a steel framed building. Additionally, portal frame systems provide maximum flexibility of space inside the building with a wide range of spans that can easily be extended in the future. Portal frame systems are frequently used in a variety of settings, including industrial, commercial, and agricultural buildings, as well as sports and leisure facilities, aircraft hangars, infrastructure, and transportation. Portal frames can be constructed either as single- or multi-span structures.

A typical single-span portal frame, as shown in Figure 2.1, is composed of a series of parallel frames, with each frame consisting of two columns and two rafters. The span of each frame is typically between 15 and 50 m with an eaves height between 5 and 10 m. The parallel frames are spaced at intervals of 5 to 8 m with each end frame commonly referred to as a gable frame. Additionally, portal frames are normally designed with a pitched roof, with a slope angle θ ranging between 5° and 10° , with 6° in common use (Hayward & Weare, 2002; Salter, Malik & King, 2004; Davison & Owens, 2012).

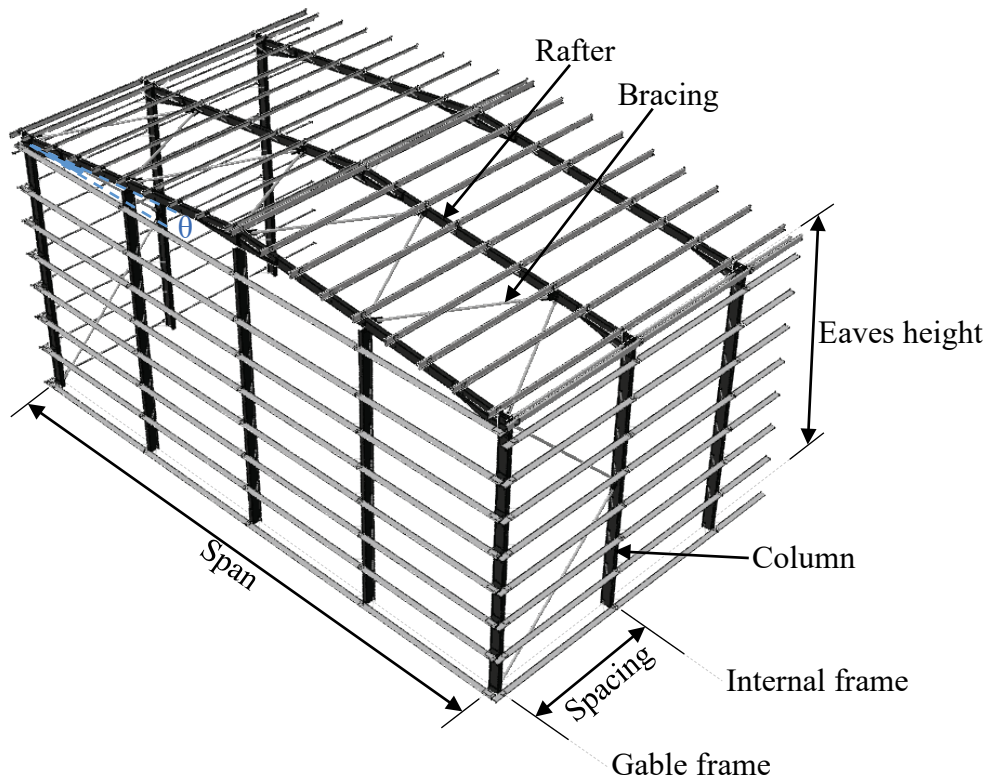


Figure 2.1: Illustration of a typical single-span portal frame

A multi-span portal frame consists of multiple columns, rafters and intermediate supports. A combination of different spans and eaves heights can be achieved utilising this system resulting in a relatively flexible frame (King, 2001). Multi-span portal frames often utilise a “hit and miss” design, as shown in Figure 2.2, where alternate internal columns are removed and a valley beam is used to support the rafters at these locations (Henderson, 2015). The dimensions of a “hit and miss” frame can vary depending on the design requirements and intended use of the building; recent multi-span portal frames for example, the SEGRO Logistics Park in the East Midlands, United Kingdom, have featured spans up to 45 m, heights up to 35 m, widths up to 150 m and a length extending up to 250 m (NSC, 2019).

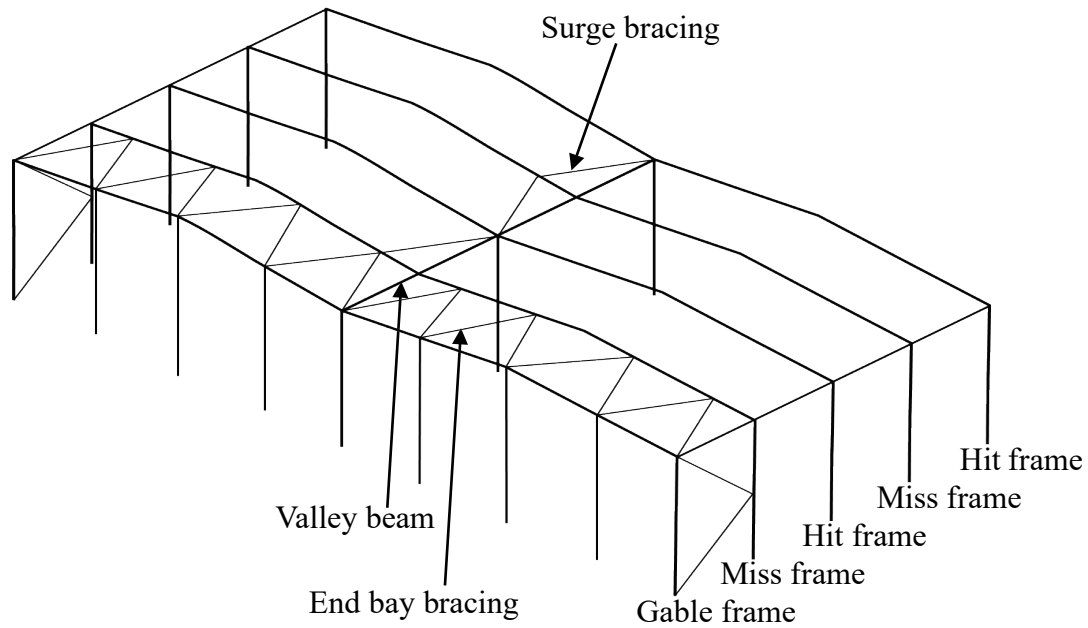


Figure 2.2: Illustration of a typical "hit and miss" multi-span portal frame

2.3 Steel portal frame components

A typical steel portal frame consists of three main structural components; (1) the primary steel frame, (2) the secondary steelwork and (3) the cladding system. Each of these three components is discussed in this section.

2.3.1 Primary steel frame

The primary steel frame is the main structural frame comprising parallel moments-resisting frames, braced longitudinally. The components of the primary steel frame are shown in Figure 2.3. Each frame consists of columns and rafters manufactured from hot-rolled I-sections, offering in-plane stiffness to the frame (Davison & Owens, 2012). Haunches in the rafter are provided at the eaves and apex to increase the stiffness and resistance at these locations of higher bending moment and to facilitate bolted connections. These haunches are typically cut from the same or slightly larger section as the rafter, with the apex haunches being shallower and shorter than the eaves haunches. Eaves haunches typically extend to 10% of the span (Davison & Owens, 2012), while the apex haunch is designed to facilitate the bolted connection and therefore its dimensions are not typically considered in the design process (Salter, Malik & King, 2004).

The end frames, which are commonly referred to as gable frames, can either be identical to the internal frame with additional support from gable posts, or they can be constructed from short

rafters and column offering a non-portal option (Davison & Owens, 2012). Longitudinal bracing contributes to the resistance to lateral loads, predominantly wind loads acting in the longitudinal direction of the building. While angle or flat steel sections can be used for the bracing members, circular hollow sections (CHS) are preferred due to their efficiency in compression and the elimination of the need for cross bracing (Davison & Owens, 2012).

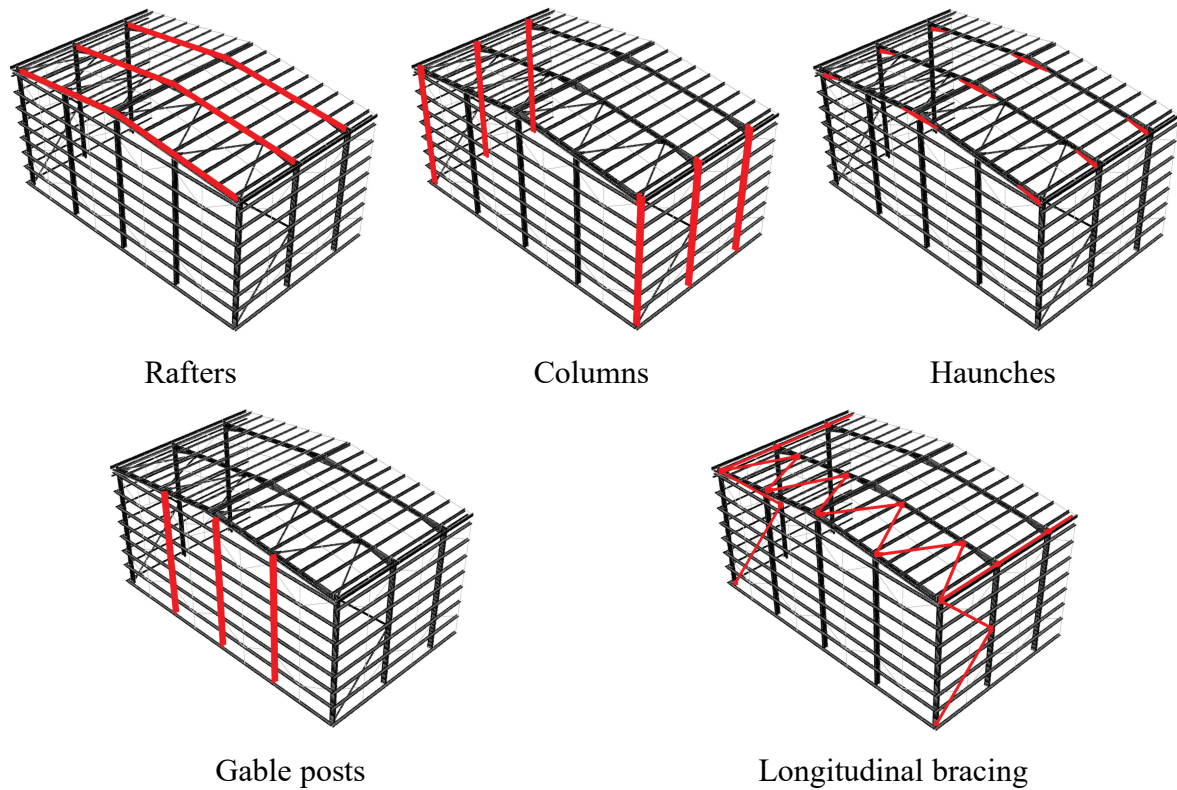


Figure 2.3: Components of the primary steel frame

2.3.2 Secondary steelwork

Secondary steelwork plays a critical role in supporting the cladding, transmitting loads from the cladding to the primary steel frame and providing restraint to the primary members (Heywood, 2006). The components of the secondary steelwork, as shown in Figure 2.4, include the purlins, side rails, eaves beams, cleats, anti-sag rods, diagonal tie wires and diagonal stays (also referred to as fly bracing).

The purlins, side rails and eaves beams (referred to herein as the cladding supporting members) transmit the load from the cladding and provide restraint against buckling to the outer flange of the primary frame members (i.e. the rafters and columns). The roof cladding is supported by the purlins, while the gable and wall cladding are supported by the side rails. These members are most commonly made from cold-formed steel sections, but hot-rolled steel angles can also

be suitable (Hayward & Weare, 2002). Cleats, typically made of hot-rolled steel, are utilised to connect the cladding supporting members to the outer flange of the primary frame members.

Anti-sag rods and diagonal tie wires are employed to provide lateral stability to the cladding supporting members against out-of-plane deflection. The need for anti-sag rods and diagonal tie wires is dependent on a number of factors including the roof slope, the purlin/side rail section, the spacing between the purlins, the magnitude of the applied load and the length of the roof slope (Heywood, 2006). Advice on the type and location of the anti-sag rods and diagonal tie wires is provided in the manufacturer's literature.

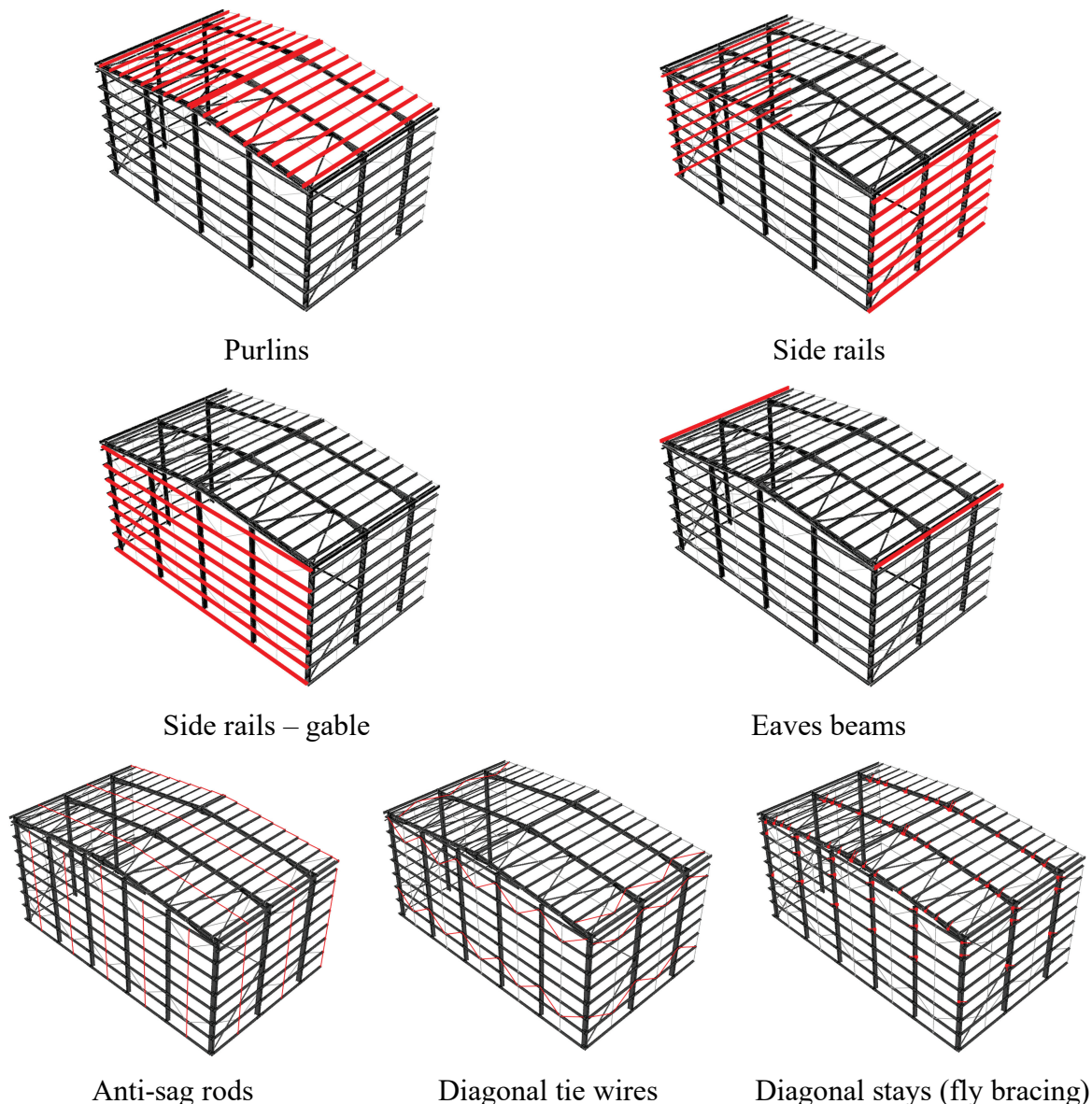


Figure 2.4: Components of the secondary steelwork

Diagonal stays are utilised to provide restraint against lateral torsional buckling of the inner flange of the primary frame members. Angle sections are most commonly used as diagonal

stays in the design of steel portal frames (Heywood, 2006; METSEC, 2017). These diagonal stays are required at the plastic hinge locations and where the inner flange is under compression, typically near the eaves under gravity loads and near the mid-span under uplift load conditions (Davison & Owens, 2012; Henderson, 2015).

2.3.3 Cladding systems

Cladding is used to isolate the interior space from the external conditions. The cladding system should provide thermal and acoustic insulation, seal the building envelope and transfer the load to the secondary steelwork. Additionally, the connections between the cladding and the secondary steelwork should provide restraint to the cladding supporting members (Heywood, 2006).

All cladding systems are sealed using metal flashings manufactured from cold-formed steel, which play a crucial role in sealing the building envelope and enhancing the weathertightness and durability of the structure (MCRMA, 2000). Metal flashings are typically 0.7 mm thick (TATA Steel, 2016). Three flashing components are required: (1) the apex hat, also known as the ridge cap or the ridge flashing, placed along the ridge line where the two roof slopes meet, (2) the barge cap or the barge flashing, used to cover the exposed edge between the roof and the gable cladding and (3) the corner flashing, used where the wall cladding meets the gable cladding. Two additional flashings are installed below the cladding to enhance the weathertightness: an internal apex hat positioned at the ridge line and a cleader angle positioned at the barge flashing location. Additionally, a gutter formed as a trough or a channel is installed along the eaves of the building to collect rainwater from the roof and redirect it away from the building.

Cladding panels can also include windows, vents and rooflights, to allow natural daylight into the building interior. In steel portal frames, rooflights are constructed using translucent sheets with the same profile as the metal sheets to ensure a seamless integration with the metal cladding system (MCRMA, 2022a). To enhance thermal insulation, an additional layer or two of translucent sheets are inserted in the middle of the rooflight panel (MCRMA, 2022a). Translucent sheets can be manufactured from glass reinforced plastic (GRP), polycarbonate, glass or UPVC (MCRMA, 2022b). However, glass is only used in flat roofs, and UPVC tends to be avoided due to its tendency to become brittle when exposed to UV radiation. Consequently, GRP and polycarbonate are most commonly used in the UK (MCRMA, 2022b). GRP is resin-coated, glass reinforced polyester that effectively disperses light, reduces glare,

and minimises localised heat build-up, thereby contributing to an energy-efficient building (MCRMA, 2022a). Polycarbonate is a thermoplastic translucent polymer that provides clear, non-diffused light that can be adjusted with opal tints (MCRMA, 2022b).

Over time, cladding systems have evolved from single-skin sheeting to a variety of two-skin cladding systems. The choice of the appropriate cladding system should align with both the structural requirements and the aesthetic preferences of the building. In this section, two of the most commonly used cladding systems in industrial-type buildings; (1) two-skin built-up cladding systems and (2) sandwich panels (Heywood, 2006), are introduced.

Two-skin built-up cladding system

The two-skin built-up cladding system, illustrated in Figure 2.5, consists of two metal sheets separated by a spacer system and insulation. The liner sheet, positioned at the bottom, is in direct contact with the cladding supporting member (i.e. the purlin). Typically, the liner sheet is manufactured from cold-formed steel or aluminium, with a shallow trapezoidal profile and a sheet width ranging from 900 to 1000 mm (Heywood, 2006; TATA Steel, 2013). The weather sheet, located at the top, protects the building from the external environment by providing a weathertight envelope (TATA Steel, 2013). Additionally, the weather sheet transfers the externally applied load to the secondary steelwork via the spacer system. To achieve adequate bending stiffness, the weather sheet is manufactured from cold-formed steel or aluminium with a deep trapezoidal or sinusoidal profile (Heywood, 2006; MCRMA, 2022b). The steel sheets come in two thickness options: either 0.4 mm or 0.7 mm while the available thicknesses for aluminium sheets are 0.5 mm or 0.9 mm (Heywood, 2006; TATA Steel, 2013).

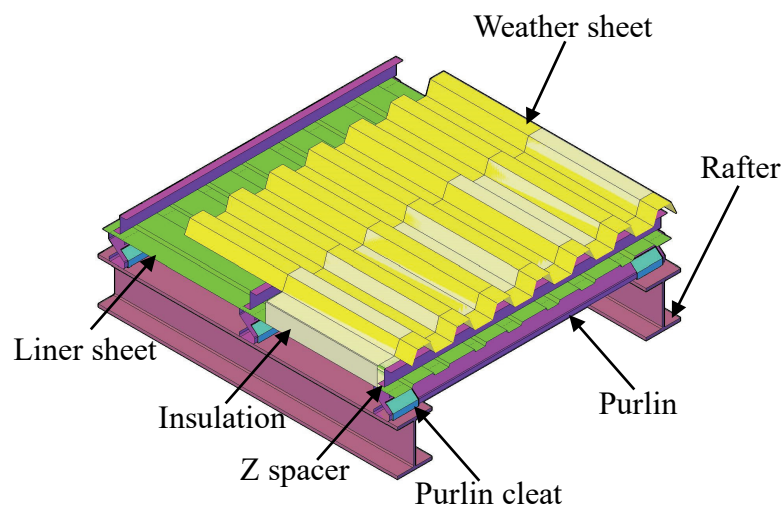


Figure 2.5: A typical two-skin built-up cladding system with Z spacers

The spacer system provides support to the weather sheet, maintains the required distance between the two sheets and transfers the load from the weather sheet to the cladding supporting member. To ensure a reliable and direct load path, the spacer system is always aligned with the cladding supporting members and secured firmly to them. Different types of spacer systems are available, including Z spacers and “bar and bracket” systems with the latter being the most commonly used in the UK (Heywood, 2006). An insulation layer is essential between the metal sheets to prevent the flow of heat between the interior space of the building and the external environment. Commonly used insulation materials include stiff mineral wool slabs and mineral wool quilts. The mineral wool quilts are more deformable than the rigid mineral wools slabs allowing for a tight fit without leaving any air gaps, ensuring enhanced insulation performance (Heywood, 2006).

The two-skin built-up system described above requires the use of a typical cladding supporting system. However, the purlin/side rail cleat system has been identified as a potential plane of weakness where failure can occur at either the interface between the cladding and the support member or at the purlin/side rail cleat assembly (Davies, Roberts & Wang, 2021). Alternative liner trays, known as cassettes, can be used to eliminate the weakness planes associated with the use of purlins/side rails (Davies, Roberts & Wang, 2021). As illustrated in Figure 2.6, cassettes are C-shaped cold-formed sections that can be used to replace both the liner tray and the cladding supporting system (Davies, 2006). Cassettes have yet to be widely employed in construction within the United Kingdom (Davies, Roberts & Wang, 2021).

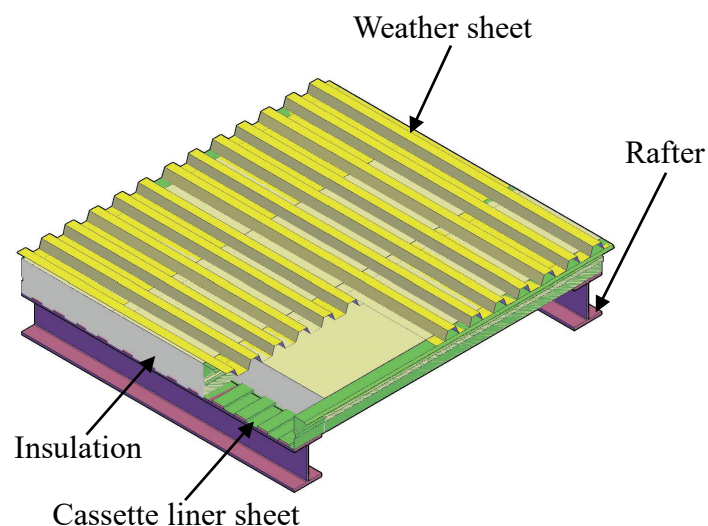


Figure 2.6: A two-skin built-up cladding system utilising cassettes

Two-skin sandwich panel cladding system

Sandwich panels, also known as insulated panels, are the most commonly used cladding system in the United Kingdom (Heywood, 2006). As shown in Figure 2.7, a typical sandwich panel consists of two metal sheets separated by stiff insulation. The metal sheets used in this system are similar to those employed in the two-skin built-up systems, regardless of their profile, as the composite action in the sandwich panel is not affected by the profile (Heywood, 2006). The insulation used in this system should be stiff enough to transfer the load from the weather sheet to the cladding supporting members and maintain the distance between the sheets without the need for a spacer system (Heywood, 2006). Commonly used insulation materials in this system include Polyisocyanurate (PIR) and stiff mineral wool slabs, with PIR being preferred due to its self-adhesive nature and quick constructability (Heywood, 2006; MCRMA, 2022a). Additionally, some manufacturers offer other insulation materials, such as Quadcore® from Kingspan (Kingspan, 2020a; Kingspan, 2022).

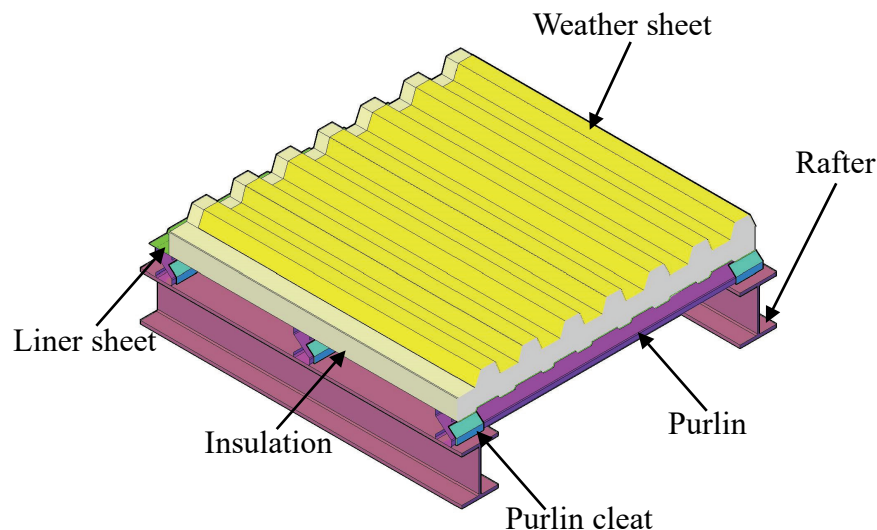


Figure 2.7: A typical two-skin sandwich panel cladding system

2.4 Portal frame connections

Various types of connections are used to join different structural components within steel portal frames. These connections can be welded, bolted, screwed or formed with a combination of these. Connections can be categorised into: (1) primary frame connections, (2) secondary steelwork connections and (3) cladding connections.

2.4.1 Primary frame connections

The primary frame connections include connections at the eaves, apex, column bases, gable posts and bracing. The typical arrangement of the connections at the eaves and apex are illustrated in Figure 2.8. Both connections should be moment-resisting and effectively resist high design moments that are likely to reverse in certain design conditions (Henderson, 2015). To satisfy the high design moment capacity, haunches and end plates are welded to the rafter at both ends to ensure continuous load transfer. The haunches also allow for a longer lever arm between the tension and compression bolt rows, thus facilitating attainment of the required moment resistance.

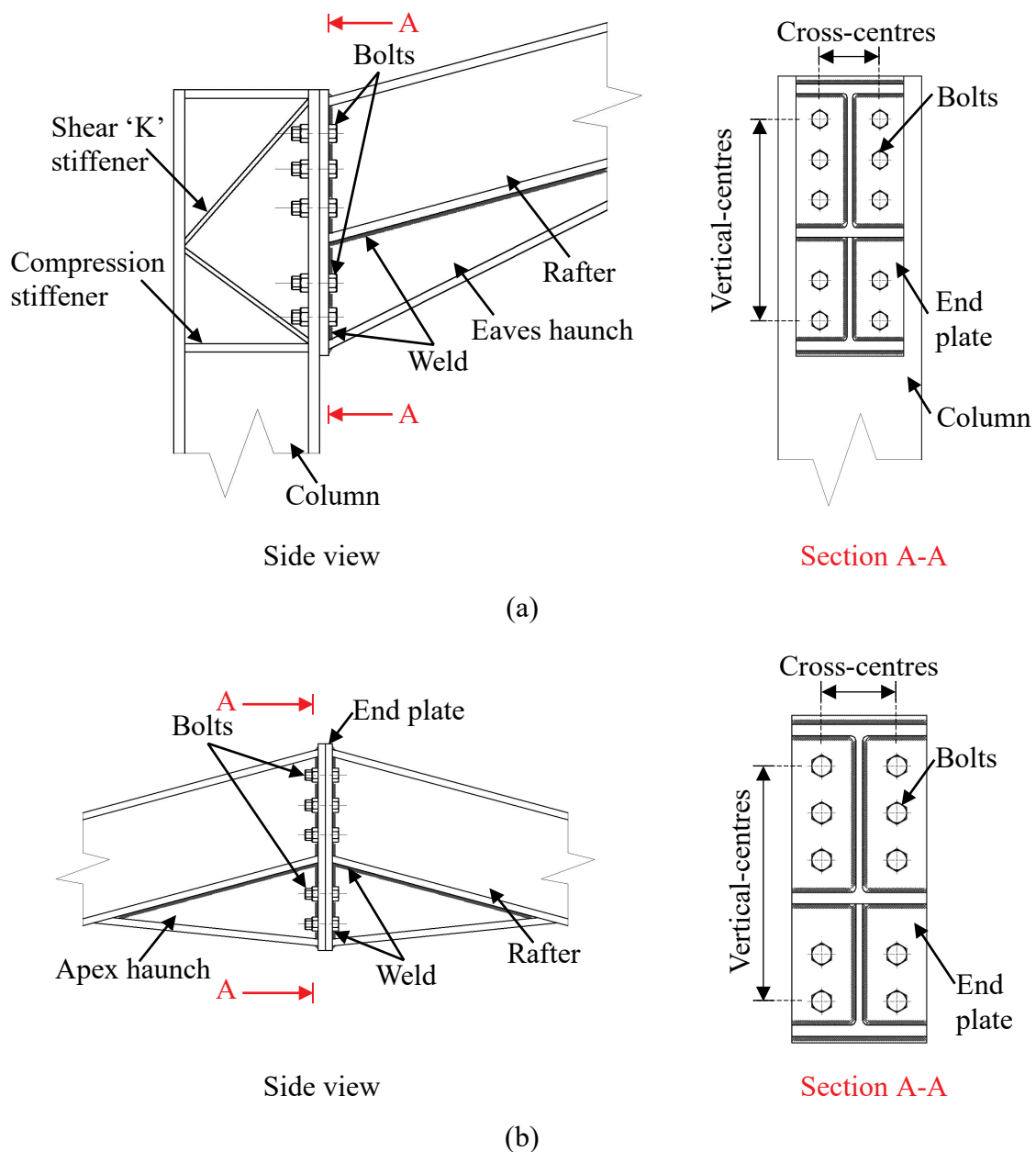


Figure 2.8: Typical arrangement of (a) eaves and (b) apex joints in steel portal frames

For moment resisting joints, it is recommended to use M20 or M24 fully threaded class 8.8 bolts. The recommended cross-centres for bolts are typically 90 to 100 mm, with 90 mm recommended for vertical-centres (Brown et al., 2013). End plates are generally made from S275 or S355 steel with a thickness no less than the bolt diameter (Brown et al., 2013). The location of the tension zone depends on the loading condition. In the eaves connection, the top rows of bolts are in tension under gravity loads and in compression under uplift conditions. On the contrary, the zones are reversed in the apex connection. Compression is transferred by direct bearing at the end of the compression zone, which corresponds, in the eaves connection, to the bottom flange of the haunch under gravity load and the top flange of the rafter under uplift conditions. To reinforce the thin web of the column, a compression stiffener is usually required at the end of the compression zone, and in some cases, a shear stiffener may also be needed to strengthen the web panel (Henderson, 2015).

Column bases in steel portal frames can be classified into three main types: pinned, nominally pinned and rigid. Nominally pinned column bases are typically used in single-span portal frames. These bases are designed to withstand small moments and allow for rotational movement (Henderson, 2015). A nominally pinned column base connection, as shown in Figure 2.9, is achieved through a base plate, holding down bolts and a concrete foundation. This connection is classified as a bearing-type connection and primarily designed to transfer axial loads. To ensure continuous load transfer, the columns are welded to the base plates. Additionally, the tension and compression arising from the small design moment are resisted by holding down bolts and the steel to concrete contact, respectively (Brown et al., 2013; Moreno, Tarbé & Brown, 2014). The holding down bolts are usually M20, M24 or M36 class 8.8 bolts depending on the thickness of the base plate. The base plate is typically made from S275 or S355 steel plate (Moreno, Tarbé & Brown, 2014).

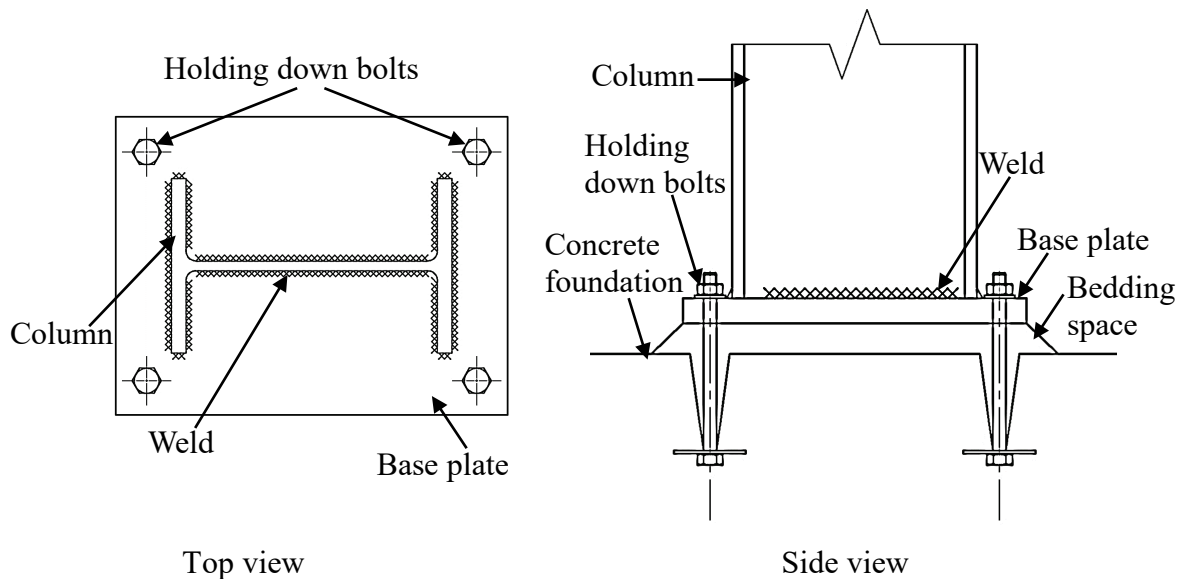


Figure 2.9: Typical nominally pinned column base

The connections at the gable posts and bracing members are designed to transfer lateral loads acting on the frame. The gable posts are connected to the rafter at the end frame (i.e. the gable frame). For a portal gable frame, a cleat is welded to the bottom flange of the rafter and then bolted to the web of the gable post, as shown in Figure 2.10a. Note that the gable posts are designed to resist lateral loads applied on the frame but not to support the vertical load transferred from the rafter. Thus, to prevent premature failure arising from vertical load, a slot is introduced to allow for the vertical displacement of the rafter (Hayward & Weare, 2002). Longitudinal bracing members are typically attached to the web of the primary frame member, as illustrated in Figure 2.10b. The gusset plate is welded to the rafter or column web close to the top flange while the tee cleat is welded to the bracing member (Hayward & Weare, 2002; Henderson, 2015). For both connections, it is recommended to use M20 fully threaded class 8.8 bolts with S275 steel plates (Moreno, Tarbé & Brown, 2014).

2.4.2 Secondary steelwork connections

Secondary steel connections arise at (1) the cleats connecting the cladding supporting member to the outer flange of the primary frame member, (2) the anti-sag rods and diagonal tie wires connection to the cladding supporting members, and (3) the diagonal stays connection to either the cladding supporting member or the inner flange of the primary frame member.

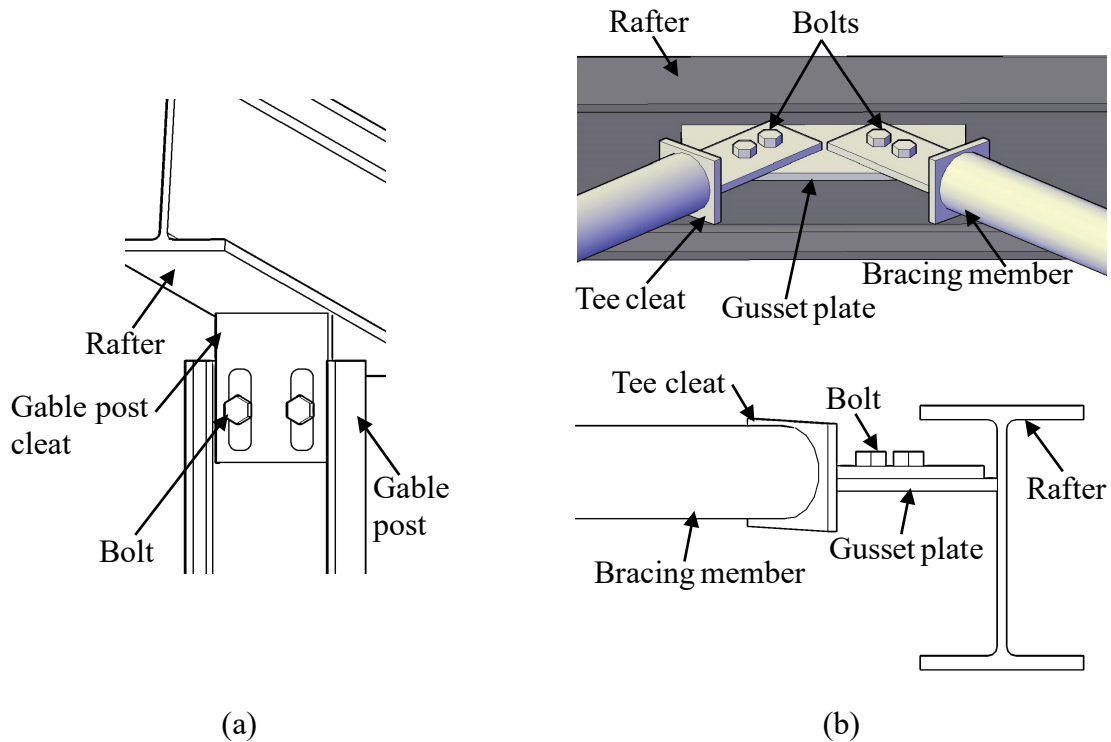


Figure 2.10: Typical arrangement for (a) gable post cleat connection and (b) longitudinal bracing connection

Cladding supporting members are typically bolted to cleats pre-welded to the primary steel member. The purlins are connected to the outer flange of the rafter while the side rails and eaves beams are connected to the column outer flange. Typical cleat connections used in steel portal frames are illustrated in Figure 2.11. M12 or M16 class 4.6 bolts are typically used to connect these light structural components (Moreno, Tarbé & Brown, 2014). Purlins and side rails can be continuous over the frame, overlapped, sleeved or simply supported, with the sleeved system most commonly used in steel portal frames (Hayward & Weare, 2002; Hui, Gardner & Nethercot, 2016; Kyvelou et al., 2018). In a sleeved system, the purlins and side rails span between two or three frames and are made continuous over the next intermediate frame using a short overlapping sleeve section, as shown in Figure 2.11b (Hayward & Weare, 2002).

Anti-sag rods are either bolted or clipped to the cladding supporting members depending on the system offered by the manufacturer. The diagonal tie wires extend diagonally between two purlins/side rails as shown in Figure 2.4 and are typically bolted on both sides. Generally, the manufacturers provide a comprehensive package including the required sizing of the cladding supporting member, anti-sag rods and associated fittings and fasteners, accompanied by a detailed installation manual.

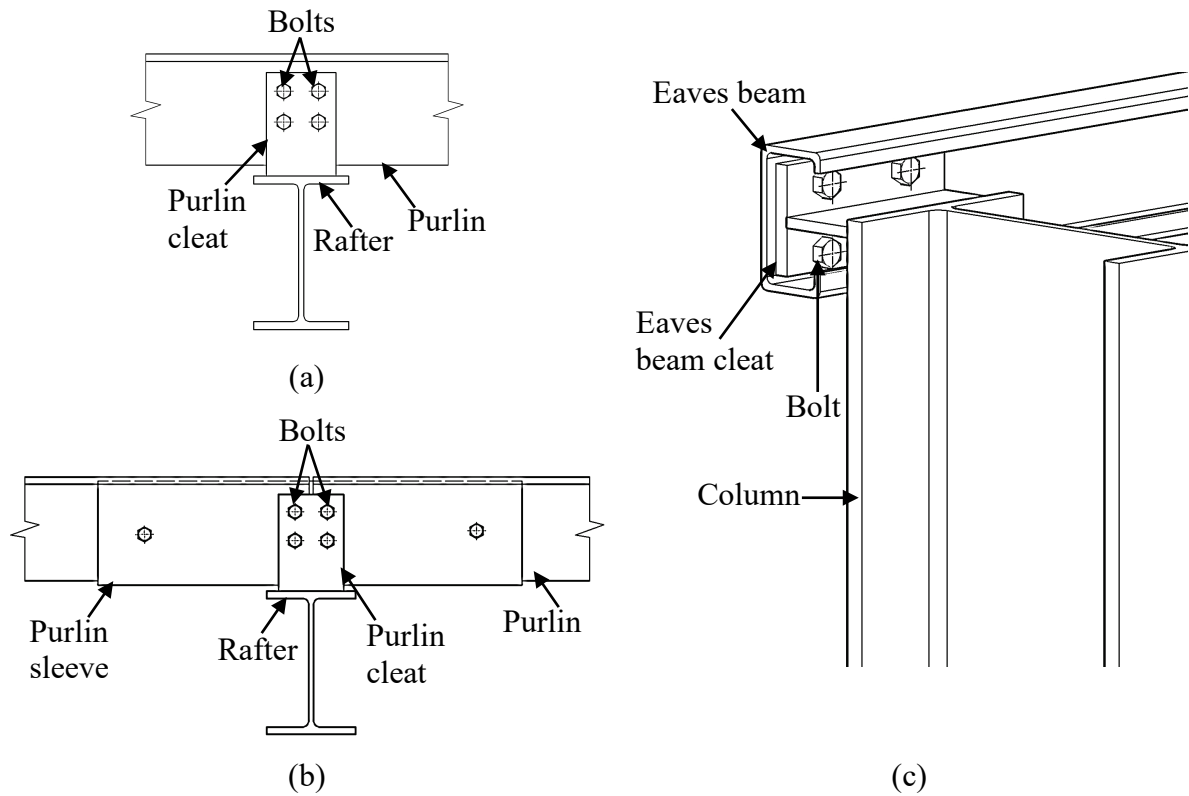


Figure 2.11: Typical arrangement for (a) a purlin cleat connection, (b) a sleeved purlin connection and (c) an eaves beam cleat connection

A typical diagonal stay connection is shown in Figure 2.12, where a plate is welded to the internal flange of the primary steel member (i.e. the rafter or column), to which the stay member is bolted. The stay is then bolted to the purlin or side rail web at a maximum angle of 45° to the horizontal (Davison & Owens, 2012). M16 class 4.6 bolts are typically used for this connection (Hayward & Weare, 2002).

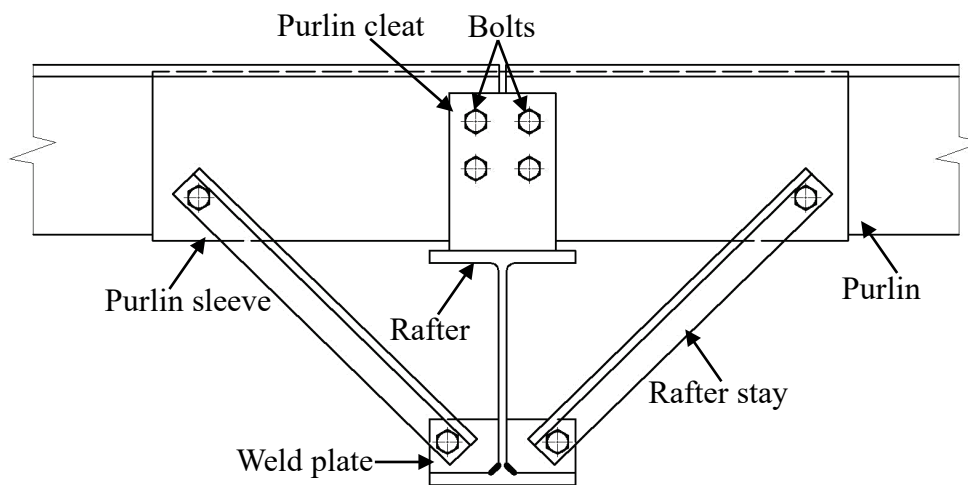


Figure 2.12: Typical diagonal stay connection

2.4.3 Cladding connections

The cladding system is typically connected to the secondary steelwork via mechanical fasteners, including bolts, screws, rivets and pins (MCRMA, 2021). The type of mechanical fasteners used depends on the choice of cladding system, and typically the cladding system suppliers provide instructions specifying the fasteners required for installation. Cladding connections are generally divided into primary and secondary connections (MCRMA, 2022c). The primary connections are the fasteners connecting the cladding to their supporting member and are designed to transfer the load from the cladding to the secondary steelwork (Heywood, 2006; MCRMA, 2000). The secondary connections are the fasteners used to join the adjacent cladding panels as well as the cladding to the metal flashings. The main purpose of the secondary connections is to seal the building and stitch the adjacent panels; thus, they are not typically designed to transfer load (Heywood, 2006; MCRMA, 2000).

For the primary connections, self-drilling self-tapping screws are typically used for metal cladding systems (MCRMA, 2000). For two-skin built-up cladding systems, short self-drilling self-tapping screws are used to build up the cladding components, as shown in Figure 2.13a and b. However, for the sandwich panel cladding system, a single long self-drilling self-tapping screw connects the external sheet through the insulation and internal sheet to the supporting member, as shown in Figure 2.13c. The specifications of the screw length, positioning and spacing are typically provided by the cladding manufacturer.

For the secondary connections, a range of mechanical fasteners are used for metal cladding systems, including bolts, screws, rivets and pins (MCRMA, 2000; MCRMA, 2021). The fasteners used in the secondary connections are purpose designed to provide weathertightness and colour matching to the building exterior. Adjacent cladding panels are connected longitudinally at the side laps and transversely at the end laps. The arrangement for cladding secondary connections is also provided by the cladding manufacturer.

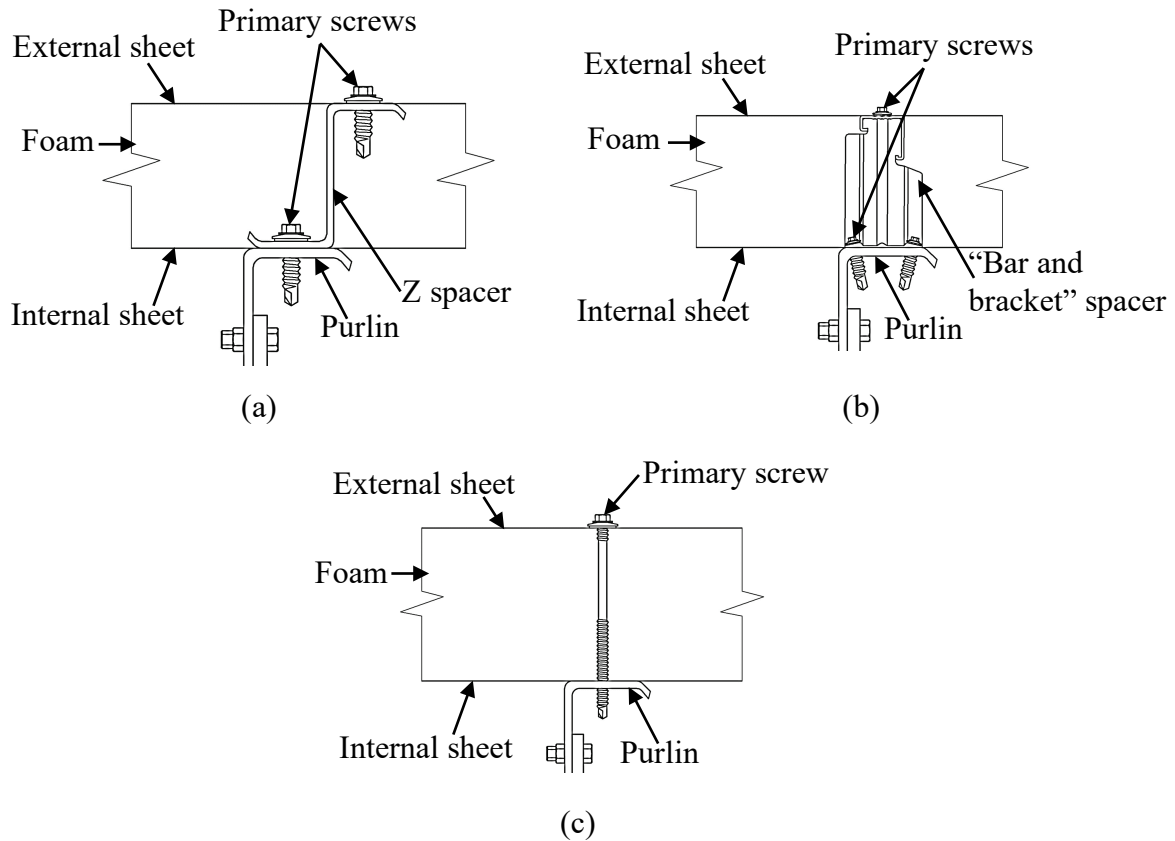


Figure 2.13: Primary cladding screws for (a) two-skin built-up system with Z spacer, (b) two-skin built-up system with “bar and bracket” spacer and (c) sandwich panel system

2.5 Design of steel portal frames

In the traditional design of steel portal frames, each structural component is designed separately based on simple assumptions for load transfer, taking into account the restraint provided by other structural components. The designer must consider all load cases and the design of the restraints and connections (Salter, Malik & King, 2004).

The primary steel frames are designed to transmit the load acting on the building to the ground. The preliminary sizes for primary frame members (i.e. the rafters and columns) can be produced either from tabulated member sizes or design charts (Davison & Owens, 2012). The member sizes provided by the tabulated member sizes are based on the frame span, eaves height, load per rafter and restraint in the column, while the member sizes provided by design charts are based on the frame span, eaves height, roof slope and load per rafter. In both cases, the design needs to be checked for stability at the ultimate limit state and deflection at the serviceability limit state (Davison & Owens, 2012). Longitudinal bracing is designed to transmit the horizontal load acting on the building to the ground and provide out-of-plane

stability to the frames both during and after construction (Salter, Malik & King, 2004; Davison & Owens, 2012).

Cladding supporting members (i.e. the purlins, side rails and eaves beam) are typically proprietary cold-formed steel sections that are developed and tested by the manufacturer. Where applicable, the manufacturer provides the size, type and spacing of these members in terms of load/span tables (Salter, Malik & King, 2004). Additionally, the manufacturer supplies the necessary anti-sag rods and diagonal tie wires along with installation guidance (METSEC, 2017). Diagonal stays connected to the purlins or side rails are designed to provide torsional restraints to the rafters or columns. Precautions should be taken to ensure that the purlin or side rail is adequately sized to provide restraint and that the section or sections used as diagonal stays can resist the forces acting on the flange in both directions (Salter, Malik & King, 2004).

The cladding system is chosen to provide the weathertightness of the building, thermal and acoustic insulation, fire resistance and compliance with aesthetic requirements. Other factors that are also considered when specifying the cladding system include feasibility, durability, maintenance, remedial work, strength and rigidity (Heywood, 2006). The cladding is typically required to support the externally applied load (e.g. gravity and wind loads) without excessive deflection and therefore a cladding profile can simply be chosen from the manufacturer's safe load tables (Heywood, 2006).

While not currently widely adopted in practice, it is also possible to utilise the in-plane shear stiffness of the cladding to enhance the sway stiffness of the frame, known as the stressed skin design (Heywood, 2006). This approach requires the cladding stiffness to be incorporated into the design of the steel frame components. Stressed skin design also requires careful design of the cladding connections and thorough consideration of the placement of the translucent cladding panels, as both aspects can significantly impact the stressed skin action (Heywood, 2006). The additional stiffness provided by the cladding is evident in real structures, with studies showing the horizontal deflections of clad portal frames to be less than 50% of the deflections calculated for an equivalent bare frame (Henderson, 2015; Wrzesien et al., 2015; Lendvai & Joo, 2018; Vacev et al., 2020). This implies that the interaction is always present, regardless of whether it was considered during the design phase or not. This has the potential to either enhance or adversely affect the overall behaviour of the frame.

2.6 Effect of stressed skin action in steel portal frames

The stiffening effect of roof sheeting on the overall behaviour of steel portal frames was first reported by Godfrey and Bryan (1959) during an experimental study on a full-scale clad steel frame. During this study, it was observed that the bending moments in the tested frame were considerably lower than the anticipated values, indicating that the metal sheeting had contributed to the sway resistance of the frame. Following this discovery, Bryan embarked on a series of experimental investigations into the stiffening effect of sheeting on semi-full scale steel portal frames (Bryan, 1964; Bryan & El Dakhakhni, 1964; Bates, Bryan & El-Dakhakhni, 1965; Bryan, 1971a; Bryan, 1971b; Bryan & Mohsin, 1972). It was found that the collapse load of a clad frame was 42% higher than the collapse load of a bare frame (Bryan & El Dakhakhni, 1964) and the deflections of a clad frame were around 50% less than those of a bare frame (Bates, Bryan & El-Dakhakhni, 1965; Bryan, 1971a). Furthermore, it was also concluded that by utilising the stiffening effect of the cladding, the weight of the primary steel frame could be reduced by 25% (Bryan & Mohsin, 1972). Experiments were also conducted on cladding panels to investigate the effect of purlins, connections and the cladding sheet profile and width on their shear flexibility (Bryan & El-Dakhakhni, 1968). In 1972, Bryan published the first book on stressed skin action (Bryan, 1972) summarising all this previous work.

The basic principle of stressed skin action is that the cladding contributes to the in-plane stiffness of the steel portal frame acting as a stressed membrane known as a diaphragm (Bryan, 1972; Bryan, 1976). Each roof panel is considered as a diaphragm transferring the load to the gable frames of the structure. Consequently, the diaphragm action redistributes the load from the internal frames to the stiffened gable frames at the end of the structure, as illustrated in Figure 1.4.

The research was continued to investigate the potential failure mechanisms associated with stressed skin action in clad steel frames (Davies, 1973; Davies & Fisher, 1987). The combined research of Bryan and Davies led to identifying multiple areas of weakness at the roof plane including at the purlin interface with the sheeting and purlin cleats, as well as at the locations of the seam fasteners. In 1982, the findings of the research into stressed skin action were published in the book “Manual of stressed skin diaphragm design” (Davies & Bryan, 1982) which summarised all the research up to that point. This publication consequently formed the basis to the current European recommendations on stressed skin design of portal frames (CEN, 2003; ECCS, 1995).

The effect of stressed skin action on cold-formed steel portal frames was investigated by Wrzesien et al. (2015) and Duoc et al. (2015). Wrzesien et al. (2015) conducted tests on cold-formed steel frames with a 6 m span, 3 m eaves height, 10° roof slope and 3 m length, comparing bare frames to clad frames utilising single-skin trapezoidal sheeting. It was found that the deflection of the internal frame in the clad frame was 90% less than that in the bare frame. Duoc et al. (2015) optimised the design of cold-formed steel frames using stressed skin design principles for frames with a span of 12 m, an eaves height of 3 m, a roof slope of 10° and total length ranging from 8 to 24 m. Single-skin trapezoidal sheeting was utilised in the cladding. It was shown in this study that, by using the stressed skin design principles, the material cost of the internal frames can be reduced by up to 53% compared to the traditional design approaches.

The effect of stressed skin action on the overall behaviour of steel portal frames was more recently investigated by Lendvai, Joo and Dunai (2018) and Lendvai and Joo (2018). Full-scale portal frames with an 18.9 m span, 6 m eaves height and 6 m length were tested in more than 100 configurations. Single-skin trapezoidal sheeting was used in the clad frame configurations which was characterised by the lack of seam fasteners. The aim of the study was to examine the parameters affecting stressed skin design and improve the existing design recommendations. It was found that the shear flexibility of the clad frame increased by up to 109% by increasing the height of the purlin by 25% and the side rails by 33%. Similarly, a 125% increase in the height of the sheeting profile resulted in up to a 53% increase in frame shear flexibility. Additionally, by doubling the number of primary fasteners between the cladding and the secondary steelwork, the frame shear flexibility was reduced by up to 40%. The research also evaluated the effect of other frame components on the frame shear flexibility; the purlins, side rails, and bracing were found to reduce frame shear flexibility by 2-10%, while the presence of cladding reduced the shear flexibility by up to 78% (Lendvai & Joo, 2018). Vacev et al. (2020) also concluded that cladding reduces shear flexibility significantly when comparing the frame stabilisation provided by longitudinal bracing against the stabilisation provided by stressed skin action under horizontal loads. Again, single-skin trapezoidal cladding was used in the clad frame tests. It was observed that, compared to the unbraced bare frame, horizontal deformations of the braced bare frame were reduced by 86.7%, while the horizontal deformations of the unbraced clad frame were reduced by 85.4%. The braced clad frame experienced the most substantial reduction, with horizontal deformations reduced by 93.7% compared to the unbraced bare frame deformations.

The effect of parasitic stressed skin action, i.e. unwanted stressed skin action, was investigated by Gryniewicz, Roberts and Davies (2021). A steel portal frame with 12 m span, 5.5 m eaves height and 20 m length was tested under horizontal load. To study the influence of parasitic stressed skin action, no provisions were made in the design to account for the effect of stressed skin. Three frame configurations were tested: a bare frame, a frame with roof cladding only, and a frame with both roof and wall cladding. Single-skin trapezoidal sheeting was used in the clad configurations. The test results were used to validate finite element models to explore the load path. It was shown that merely 10% of the applied horizontal load was resisted by the internal frames, with the majority of the load transferred to the gable frames through the parasitic stressed skin action.

A recent study by Roberts, Davies and Wang (2021) investigated the behaviour of large span portal frames. The study included the numerical modelling of single- and multi-span portal frames under vertical and horizontal load conditions. A single-span portal frame model with a 12 m span, 3.4 m eaves height, 13° roof slope and 30 m length was validated against the tests conducted in Salford in 1972 (Bryan, 1971b; Bryan & Mohsin, 1972; Davies & Bryan, 1982). Following validation of the model, the number of frames and spans was varied to investigate the effect of scale on the overall behaviour of the frame. It was found that under horizontal loading, the roof transmitted the load to the gable ends. However, under vertical loading, the load was initially resisted by individual frames and once an internal frame failed the roof sheeting transferred a substantial portion of the load, resulting in an increase in the collapse load of the clad frame compared to the bare frame. The findings in this study highlighted that the components of the steel portal frames behave as part of an integrated system rather than independently.

The effect of stressed skin action on sandwich panel cladding was investigated by Roberts and Davies (2023). Numerical models of sandwich panel cladding and the supporting purlins were developed. It was highlighted that seam screws first yield at a relatively low load level. Another study was conducted by Nagy et al. (2023) to investigate the effect of different cladding systems on the behaviour of portal frames. Numerical models of portal frames with a span of 6 m, an eaves height ranging between 2.6 and 5 m and a length ranging between 9 and 21 m were developed. Two cladding types were used: single-skin trapezoidal sheeting and sandwich panel cladding. It was observed that the sandwich panel-clad frames exhibited greater stiffness compared to the single-skin-clad frames, making the sandwich panels more susceptible to damage due to parasitic stressed skin action.

It should be noted that the dimensions considered in the aforementioned studies are considerably smaller than some steel portal frames in current practice with steeper roof slopes than the typical 6° slope used nowadays. Additionally, the cladding systems utilised were mostly a single-skin profiled sheeting, whereas stiffer alternatives, including two-skin built-up cladding systems and sandwich panel cladding systems, are now more widely employed.

The effect of scale on the behaviour of steel portal frames was highlighted in a SCOSS alert (SCOSS, 2018) in 2018, specifically outlining a number of structural issues associated with the increasing size of these structures. For example, increased spans result in longer rafters and therefore the potential for larger deflections. Furthermore, a higher eaves height introduces the potential for wind sway dynamics, which is a factor typically excluded from traditional design considerations. Thus, it was highlighted that the interaction between the increasingly flexible frames and the stiffer cladding systems need to be reconsidered, with the current design recommendations deemed questionable for contemporary portal frame structures.

2.7 Concluding remarks

A general review of the literature review related to this thesis has been presented in this chapter with a particular focus on the traditional approach for the design of steel portal frames and an overview of the research to date into the effect of stressed skin action. The current concerns related to the effect of scale were introduced and will be investigated further herein. It should be noted that further literature is introduced and examined later in the thesis where relevant.

Chapter 3

Finite element modelling of frame components: fundamentals and validation

3.1 Introduction

In this chapter, the fundamental aspects of the finite element modelling employed to simulate steel portal frame connections and cladding are presented. Subsequently, the key behavioural aspects of different portal frame connections are introduced and validated against experimental results. Finally, the modelling approaches for simulating equivalent single- and two- skin cladding systems are described and validated against corrugated cladding models as well as experimental results.

3.2 Fundamental aspects of finite element modelling

The general purpose finite element software ABAQUS (2021) was used to develop the FE models presented in the following sections. The four-noded quadrilateral shell S4R element with reduced integration was chosen for modelling the steel plated components of the frames to accurately capture local instabilities, as widely used in similar literature (Hui, Gardner & Nethercot, 2016; Zhang & Rasmussen, 2014; Zhang, Rasmussen & Zhang, 2015; Kyvelou et al., 2019; Yun & Gardner, 2018; Yuan et al., 2014; Kucukler, Gardner & Macorini, 2015; Meng & Gardner, 2020). Multi-point constraints, MPC tie and MPC beam, were used to represent the welds and ensure continuous load transfer. MPC beam was used when there was a gap between the connected nodes while MPC tie was used in other instances. The mesh size was defined such that an aspect ratio between 1:1 and 1:2 was ensured and was reduced further at the corners and weld zones to approximately 1:10. Each steel plate or steel member was assigned the suitable material stress-strain from the models presented in the following section.

3.2.1 Material modelling

Portal frames are constructed using a combination of hot-rolled and cold-formed steel components. Hot-rolled steel I-sections are used as rafters, columns and haunches while web stiffeners, end plates and cleats are made of hot-rolled steel plates. Bracing members can be manufactured from either hot-rolled or cold-formed steel sections. Secondary steelwork, flashings and gutters are typically made of cold-formed steel members. The material stress-strain characteristics and modelling approaches for these two types of steel elements are discussed in this section. It should be noted that the corrugated steel sheets used for cladding are also typically made from cold-formed steel; however, an equivalent orthotropic material model is subsequently presented and validated in Section 3.4.1 for this specific component of the portal frame.

Hot-rolled material model

The stress-strain behaviour of hot-rolled carbon steel subjected to tensile forces typically exhibits three distinct phases: an elastic phase, a plastic yield plateau, and a phase of strain hardening. The elastic stage is a linear path characterised by the Young's modulus E and limited by the yield stress f_y and corresponding yield strain ϵ_y . Upon reaching the yield point, a phase of plastic deformation known as the plastic yield plateau initiates, characterised by a relatively consistent stress level until the strain hardening strain ϵ_{sh} is attained. As the strain hardening point is reached, stress accumulation occurs at a declining rate, progressing towards the ultimate tensile stress f_u and its corresponding ultimate tensile strain ϵ_u . Subsequently, necking and fracture follow.

The stress-strain behaviour of hot-rolled steel is accurately captured by the models proposed by Yun and Gardner (2017). Two models are proposed: a quad-linear stress-strain model and a bilinear plus nonlinear hardening model. Both models accurately represent the yield plateau and strain hardening behaviour using three input parameters: the Young's modulus E , yield stress f_y and ultimate tensile stress f_u . The quad-linear model is included in the draft Eurocode for the design of hot-rolled steel structures assisted by finite element analysis, prEN1993-1-14 (CEN, 2021), and has been adopted in the present study.

The quad-linear stress-strain curve consists of four stages, as defined by Eq. (3.1) and shown in Figure 3.1. The initial two stages represent the elastic range and the plastic yield plateau, while the subsequent two stages represent the strain hardening phase. The third stage is defined by the strain hardening modulus E_{sh} , as given by Eq. (3.2), and limited by the strain $C_1 \epsilon_u$ which

was calibrated to match the intersection point of the third stage of the model and measured stress-strain curves (see Figure 3.1). The fourth stage starts at $C_1 \varepsilon_u$ and ends at the ultimate tensile strain ε_u and stress f_u .

$$f(\varepsilon) = \begin{cases} E\varepsilon, & \text{for } \varepsilon \leq \varepsilon_y \\ f_y, & \text{for } \varepsilon_y < \varepsilon \leq \varepsilon_{sh} \\ f_y + E_{sh}(\varepsilon - \varepsilon_{sh}), & \text{for } \varepsilon_{sh} < \varepsilon \leq C_1 \varepsilon_u \\ f_{C_1 \varepsilon_u} + \frac{f_u - f_{C_1 \varepsilon_u}}{\varepsilon_u - C_1 \varepsilon_u}(\varepsilon - C_1 \varepsilon_u), & \text{for } C_1 \varepsilon_u < \varepsilon \leq \varepsilon_u \end{cases} \quad (3.1)$$

where f is the stress, ε is the corresponding strain, ε_y is the yield strain, ε_{sh} is the strain hardening strain, as given by Eq. (3.3), and $f_{C_1 \varepsilon_u}$ is the corresponding stress to $C_1 \varepsilon_u$.

$$E_{sh} = \frac{f_u - f_y}{C_2 \varepsilon_u - \varepsilon_{sh}} \quad (3.2)$$

$$\varepsilon_{sh} = 0.1 \frac{f_y}{f_u} - 0.055, \quad \text{but } 0.015 \leq \varepsilon_{sh} \leq 0.03 \quad (3.3)$$

The material coefficients C_1 and C_2 are given by Eqs. (3.4) and (3.5), respectively, which were derived based on the analysis of a large set of measured stress-strain curves. The expression for ultimate strain ε_u given by Eq. (3.6) was originally proposed by Arrayago, Real and Gardner (2015) for ferritic stainless steel and was found by Yun and Gardner (2017) to also be applicable to hot-rolled steel.

$$C_1 = \frac{\varepsilon_{sh} + 0.25(\varepsilon_u - \varepsilon_{sh})}{\varepsilon_u} \quad (3.4)$$

$$C_2 = \frac{\varepsilon_{sh} + 0.4(\varepsilon_u - \varepsilon_{sh})}{\varepsilon_u} \quad (3.5)$$

$$\varepsilon_u = 0.6 \left(1 - \frac{f_y}{f_u} \right), \quad \text{but } \varepsilon_u \geq 0.06 \text{ for hot-rolled steel} \quad (3.6)$$

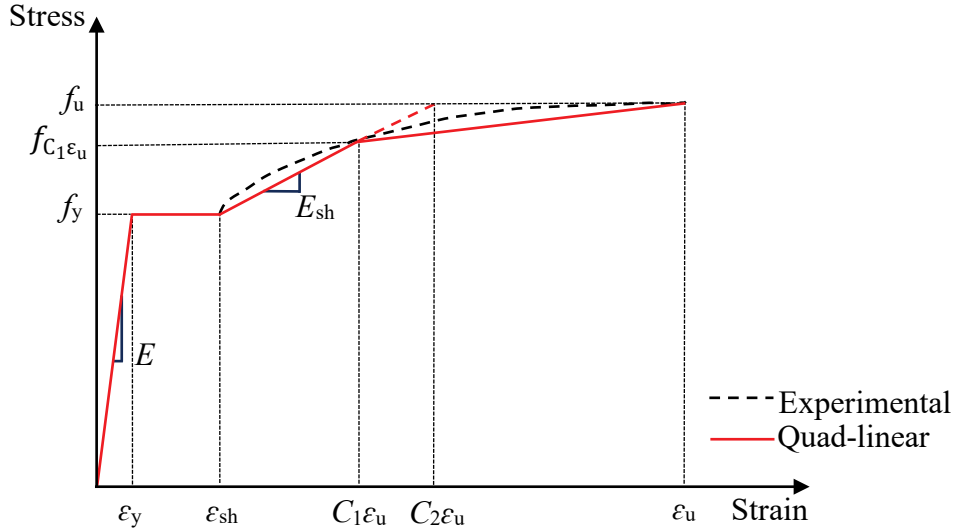


Figure 3.1: Quad-linear material model and typical experimental stress-strain curve (Yun & Gardner, 2017)

Cold-formed material model

The stress-strain behaviour of cold-formed carbon steel is characterised by a rounded response with no distinct yield point, as shown in Figure 3.2. The general expression proposed by Ramberg-Osgood model (1943) forms the basis to the most commonly used models that describes the stress-strain response of nonlinear metallic materials. The Ramberg-Osgood model captures the rounded stress-strain response of metallic materials such as stainless steel and cold-formed carbon steel using the Young's modulus E , the yield stress f_y and the strain hardening exponent n .

Hill (1944) modified the Ramberg-Osgood model, which was subsequently adopted to develop expressions to describe the full stress-strain curves for stainless steel. Mirambell and Real (2000) and Rasmussen (2003) developed two-stage stress-strain models based on the Ramberg-Osgood model. In both models, the basic Ramberg-Osgood formula was employed for the first stage, when $f \leq f_y$, and a modified Ramberg-Osgood formula was developed to capture the stainless steel behaviour in the strain hardening stage, when $f \geq f_y$. A comprehensive assessment of the two-stage material model was conducted by Arrayago, Real and Gardner (2015) against experimental results for austenitic, ferritic and duplex grades of stainless steel. Quach, Teng and Chung (2008) developed a three-stage stress-strain model for stainless steel defined using the three Ramberg-Osgood formulations.

In order to expand application of the model to include a broad range of steel grades, production routes and cross-section types, Gardner and Yun (2018) introduced predictive expressions for the stress-strain response of cold-formed steel based on the Mirambell and Real two-stage stress-strain model (Mirambell & Real, 2000), as given by:

$$\varepsilon = \begin{cases} \frac{f}{E} + 0.002 \left(\frac{f}{f_y} \right)^n, & \text{for } f \leq f_y \\ \frac{f - f_y}{E_{0.2}} + \left(\varepsilon_u - \varepsilon_{0.2} - \frac{f_u - f_y}{E_{0.2}} \right) \left(\frac{f - f_y}{f_u - f_y} \right)^m + \varepsilon_{0.2}, & \text{for } f_y < f \leq f_u \end{cases} \quad (3.7)$$

where f_y is the yield stress, taken as the 0.2% proof stress, $E_{0.2}$ is the tangent modulus at the 0.2% proof stress, ε_u is the ultimate strain, $\varepsilon_{0.2}$ is the strain at the 0.2% proof stress, f_u is the ultimate strength and m is the second strain hardening exponent.

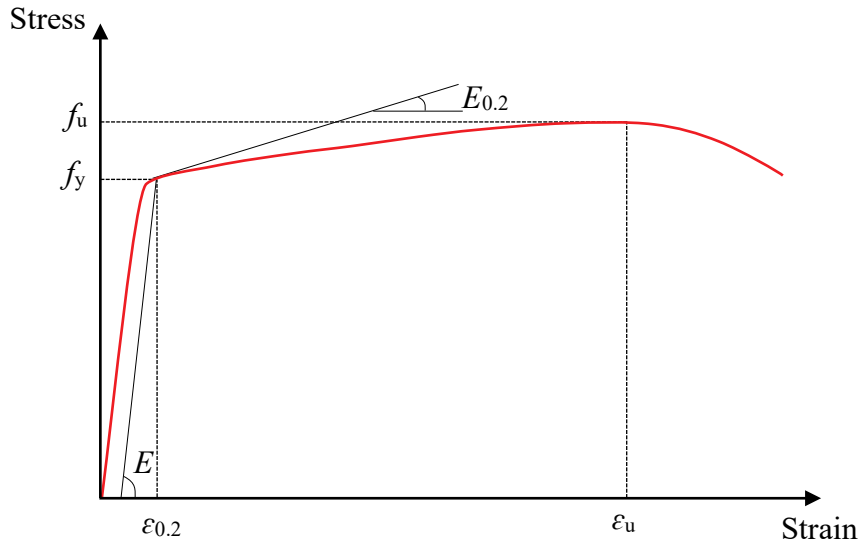


Figure 3.2: Typical cold-formed steel stress-strain curve

The strain hardening exponents n and m for cold-formed steel may be determined either using Eqs. (3.8) and (3.9), respectively, or by using representative values determined from the compiled tests in Gardner and Yun (2018), as given in Table 3.1. The expression for the strain hardening exponent n in Eq. (3.8) was originally derived by Arrayago, Real & Gardner (2015) to provide consistent n values compared to curves fitted to stainless steel stress-strain data by regression analysis. This equation was then further validated by Gardner and Yun (2018) against cold-formed steel test results. The expression used for the second strain hardening exponent m in Eq. (3.9) was originally proposed by Rasmussen (2003) and subsequently refined by Gardner and Yun (2018).

$$n = \frac{\ln(4)}{\ln(f_y/\sigma_{0.05})} \quad (3.8)$$

where $\sigma_{0.05}$ is the measured 0.05% proof stress.

$$m = 1 + 3.3 \frac{f_y}{f_u} \quad (3.9)$$

Table 3.1: Recommended representative values for strain hardening exponents n and m (Gardner & Yun, 2018)

	n	m
Flat coupons	7.6	3.8
Corner coupons	7.0	4.2

The tangent modulus at the 0.2% proof stress $E_{0.2}$ is determined by Eq. (3.10), as defined by Mirambell and Real (2000). The ultimate tensile strength f_u and the corresponding strain ε_u are calculated using Eqs. (3.11) and (3.12) respectively. The expression for the variation of f_u/f_y with yield stress f_y was proposed by Gardner and Yun (2018) following a regression analysis of tensile coupon test results while the expression for the ultimate strain ε_u was proposed and validated by Yun and Gardner (2017):

$$E_{0.2} = \frac{E}{1 + 0.002n \frac{E}{f_y}} \quad (3.10)$$

$$f_u/f_y = 1 + (130/f_y)^{1.4} \quad (3.11)$$

$$\varepsilon_u = 0.6(1 - f_y/f_u) \quad (3.12)$$

This model has shown good agreement with experimental results and is therefore considered to be accurate for use in the numerical modelling of cold-formed steel members and will be used herein. This model is also due to be included in the upcoming new prEN1993-1-14 for design assisted by finite element analysis (CEN, 2021). The same model was also applied to describe the stress strain response of aluminium alloys by Yun, Wang and Gardner (2021).

3.3 Connections modelling and validation

3.3.1 General

In this section, the modelling techniques employed for the range of bolted and screwed connections that arise in steel portal frames, as summarised in Table 3.2, are introduced and validated. The behaviour of the bolts/screws in tension, shear and bearing was defined using fastener elements (*FASTENER, *ELEMENT, TYPE = CONN3D2 (ABAQUS, 2021)) while all other behavioural aspects of the connections (see EN 1993-1-8 (CEN, 2005c) component method) were explicitly modelled, as summarised in Table 3.3. To capture the bolt radius and prevent the occurrence of undue stress concentrations (Ye et al., 2020), the fastener element was assigned a radius of influence defined equal to the washer diameter. Note that the use of fastener elements is not applicable at the interface between the sandwich panel primary screws (modelled using beam finite elements, as detailed in Section 3.3.5) and the connected cladding and supporting steelwork; therefore, a distributing coupling constraint was added to simulate the effect of the radius of influence in these connections.

3.3.2 Modelling approach for bolts

Generic models were developed herein to simulate the behaviour of bolts in tension and combined shear and bearing in line with the provisions of EN 1993-1-8 (CEN, 2005c). The load-extension behaviour of a single bolt in tension is represented by two stages, as shown in Figure 3.3a. The first stage is characterised by the bolt axial stiffness S_t , calculated using Eq. (3.13), where E is the Young's modulus of the bolt, A_s is the tensile area of the bolt and L_b is the bolt elongation length, equal to the grip length $L_{b,g}$ (the total thickness of the connected plates and washers) plus half the sum of the heights of the bolt head and nut, and limited by the yield resistance of the bolt $F_{t,y,Rk}$ (CEN, 2005c; Wang, Zhang & Jiang, 2016; Jaspert & Weynand, 2017), while the second stage extends from the yield to the ultimate tensile resistance $F_{t,Rk}$, calculated using Eq. (3.14), where f_{ub} is the bolt ultimate tensile strength. The yield resistance $F_{t,y,Rk}$ is calculated as the product of the yield strength of the bolt $f_{y,b}$ and the tensile area A_s . The relative displacement of the bolt at failure δ_b is defined as the bolt elongation given by the bolt grip length $L_{b,g}$ multiplied by the bolt ultimate strain ϵ_u .

$$S_t = \frac{A_s E}{L_b} \quad (3.13)$$

$$F_{t,Rk} = 0.9 f_{ub} A_s \quad (3.14)$$

Table 3.2: Modelling techniques employed for bolted and screwed connections in steel portal frames

Connection label	Location	Modelling approach in ABAQUS (2021)	Connection behaviour		
			Shear + Bearing	Axial	Rotation
Primary frame bolts	- Eaves: End plate to column ¹ - Apex: End plate to endplate ¹ - Bracing: Bracing tee cleat to gusset plate ¹	*FASTENER, CONN3D2, RADIAL-THRUST	Section 3.3.2 Figure 3.3b	Section 3.3.2 Figure 3.3a	-
Primary frame bolts with slots	Gable wall: Gable post to gable post cleat ¹	*FASTENER, CONN3D2, CARTESIAN	Section 3.3.2 Figure 3.3b	Section 3.3.2 Figure 3.3a	-
Secondary steelwork bolts	- Purlin to purlin cleat ² - Eaves beam to eaves beam cleat ² - Side rail to Side rail cleat ²	*FASTENER, CONN3D2, RADIAL-THRUST	Section 3.3.2 Figure 3.3b	Section 3.3.2 Figure 3.3a	-
	- Stay to weld plate ² - Stay to cladding supporting member ³	*FASTENER, CONN3D2, RADIAL-THRUST + ALIGN [†] + CARTESIAN ^{††}	Section 3.3.2 Figure 3.3b	Section 3.3.2 Figure 3.3a	-
Cladding primary connections	Sandwich panel to secondary cold-formed steelwork	Interface at external sheet	Section 3.3.5 Figure 3.6c	Rigid Figure 3.6d	Section 3.3.5
		Interface at internal sheet			
		Interface at secondary cold-formed steelwork			
Cladding secondary connections	Longitudinal seam screws: Between the external sheets of adjacent cladding panels – longitudinal side	B31 beam element + *CONN3D2, RADIAL-THRUST + CARDAN + CARTESIAN ^{††} *COUPLING, DISTRIBUTING *CONN3D2, CARTESIAN + CARDAN *COUPLING, DISTRIBUTING	Section 3.3.6 Figure 3.7	Section 3.3.6	-
	End lap seam screws: Between the external sheets of adjacent cladding panels – short side				
	Flashing seam screws:				
	- Apex hat to external sheet of the roof cladding - Corner flashing to external sheet of the wall cladding - Barge cap to external sheet of the wall and roof cladding	*FASTENER, CONN3D2, RADIAL-THRUST			

¹ Hot-rolled steel to hot-rolled steel connection, ² Hot-rolled steel to cold-formed steel connection, ³ Cold-formed steel to cold-formed steel connection

[†] ALIGN connector element to provide restraint for the rotational DOFs at the connections between beam elements (B31) and shell elements (S4R) where contact is not applicable

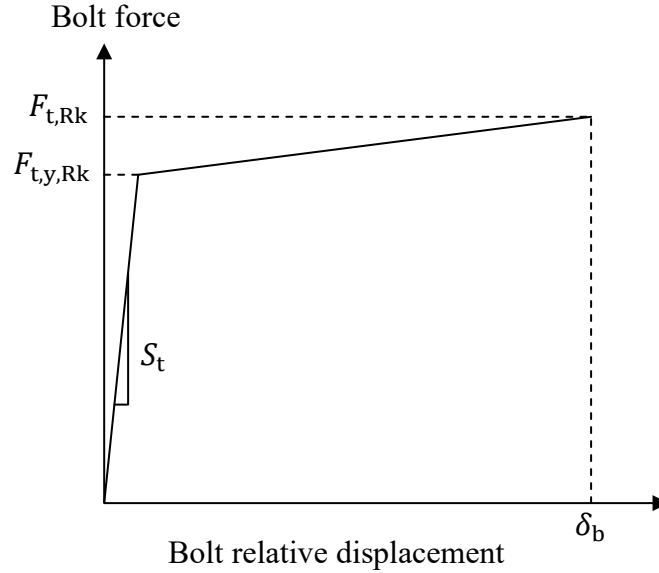
^{††} CARTESIAN connector element with negligible stiffness added for numerical stability.

Table 3.3: Modelling approach employed for the component behaviour of portal frame connections

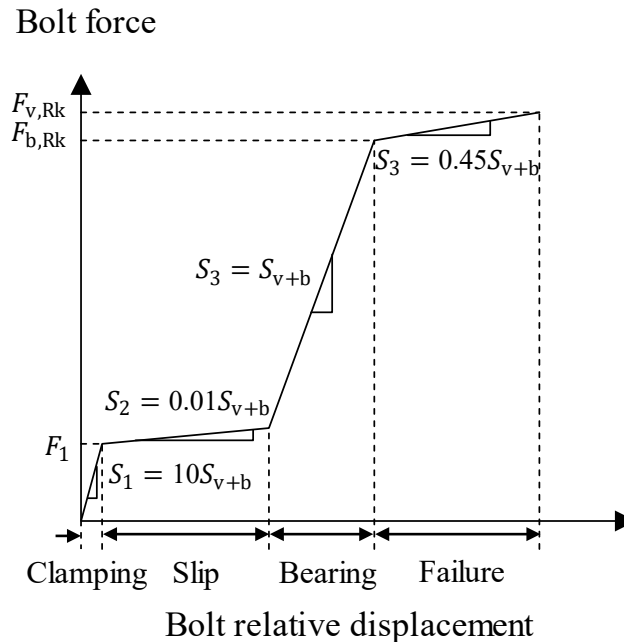
Component	Modelling approach
Column web panel in shear	Explicitly modelled
Column web in compression	Explicitly modelled
Column web in tension	Explicitly modelled
Column flange in bending	Explicitly modelled
Endplate in bending	Explicitly modelled
Flange cleat in bending	Explicitly modelled
Beam web in tension	Explicitly modelled
Beam web in compression	Explicitly modelled
Beam flange in tension	Explicitly modelled
Beam flange in compression	Explicitly modelled
Haunched beam	Explicitly modelled
Bolts in tension	Axial spring with stiffness S_t
Bolts in shear	Transverse spring with stiffness S_v
Bolts in bearing	Transverse springs with stiffness S_b

The load-deformation behaviour of bolts in combined shear and bearing is represented by four stages: clamping, slip, bearing and failure, as shown in Figure 3.3b. In the clamping stage, load is carried by friction arising from the bolt tightening during installation and occurs up to a load level F_1 , equal to the friction coefficient μ (taken as 0.2 for rolled surfaces with no additional treatment (CEN, 2018)) multiplied by the clamping force F_c . For snug-tight bolts (the most commonly used bolts in standard design), the clamping force F_c can be obtained from test results or taken as a proportion of the ultimate tensile resistance of the bolt. The ratios of the clamping force F_c to the tensile resistance $F_{t,Rk}$ assumed herein are 0.35, 0.17, and 0.14 for class 4.6, 8.8 and 10.9 bolts, respectively. These values have been calculated as the average values of the clamping force $F_c (= T/Kd)$ based on the torque values T presented in Stainsby and Pope (2003), a range of K factors (0.12 – 0.3) accounting for a range of means of lubrication, surface conditions etc. and a range of bolt sizes (M16 – M30), and thus bolt diameters d . The degree of slip in the second stage is equal to the bolt-hole clearance which depends on the bolt diameter and can be obtained from EN 20273 (CEN, 1992). Following slip, load is carried by bearing up to the bearing capacity of the steel element $F_{b,Rk}$, as calculated using Eq. (3.15) (CEN, 2005c), where d is the bolt diameter, t is the thickness of the steel onto which the bolt bears, f_u is the ultimate tensile strength of the steel, and k_1 and α_b are calculated from Eqs. (3.16) and (3.17), respectively, where α_d is given by Eq. (3.18), e_1 and e_2 are the end

and edge distances, respectively, d_0 is the diameter of the bolt-hole, and p_1 and p_2 are the spacings between the bolts parallel and perpendicular to the load (CEN, 2005c; Jaspart & Weynand, 2017; Henriques, Jaspart & da Silva, 2014). Finally, the bolt will fracture when its shear strength $F_{v,Rk}$, as calculated using Eq. (3.19) (CEN, 2005c), is attained, where f_{ub} is the ultimate tensile strength of the bolt.



a) Behaviour in tension



b) Behaviour in combined shear and bearing

Figure 3.3: Generic load-deformation model for (a) bolts in tension and (b) bolts under combined shear and bearing

$$F_{b,Rk} = k_1 \alpha_b d t f_u \quad (3.15)$$

$$k_1 = \begin{cases} 2.8 \frac{e_2}{d_o} - 1.7 \leq 2.5 & \text{for edge bolts} \\ 1.4 \frac{p_2}{d_o} - 1.7 \leq 2.5 & \text{for inner bolts} \end{cases} \quad (3.16)$$

$$\alpha_b = \min \left(\alpha_d, \frac{f_{ub}}{f_u}, 1.0 \right) \quad (3.17)$$

$$\alpha_d = \begin{cases} \frac{e_1}{3 d_o} & \text{for end bolts} \\ \frac{p_1}{3 d_o} - \frac{1}{4} & \text{for inner bolts} \end{cases} \quad (3.18)$$

$$F_{v,Rk} = 0.6 f_{ub} A_s \quad (3.19)$$

The model utilises an equivalent stiffness, S_{v+b} , calculated as a system of three springs S_v , S_{b1} and S_{b2} in series, where S_v is the stiffness of a single bolt in shear, as calculated using Eq. (3.20) (CEN, 2005c), where d_{M16} is the nominal diameter of an M16 bolt, and S_{b1} and S_{b2} are the stiffnesses of a single bolt in bearing against each of the two plates, as calculated using Eq. (3.21), where k_b and k_t are defined by Eqs. (3.22) and (3.23), respectively, where e_b is the distance from the bolt-row to the free edge of the plate in the direction of load transfer, and p_b is the spacing of the bolt-rows in the direction of load transfer (CEN, 2005c; Jaspart & Weynand, 2017; Henriques, Jaspart & da Silva, 2014).

$$S_v = 8d^2 f_{ub} / d_{M16} \quad (3.20)$$

$$S_b = 12 k_b k_t d f_u \quad (3.21)$$

$$k_b = \min \begin{cases} 0.25 e_b / d + 0.5 \\ 0.25 p_b / d + 0.5 \\ 1.25 \end{cases} \quad (3.22)$$

$$k_t = \min (1.5 t / d_{M16}, 2.5) \quad (3.23)$$

If the shear strength of the bolt is less than the bearing strength of the connection (i.e. $F_{v,Rk} < F_{b,Rk}$), the connection will fail at the shear strength due to bolt fracture. The stiffnesses of stages 1, 2 and 4 (S_1 , S_2 and S_4) have been calibrated against test data; the resulting validation of the generic models against test results for different bolted connection details is shown in the following sections.

3.3.3 Primary frame bolted connections

Primary frame bolted connections arise at the eaves, apex, bracing and gable posts as detailed in Section 2.4.1. The key behavioural aspects of these connections can be captured through lap

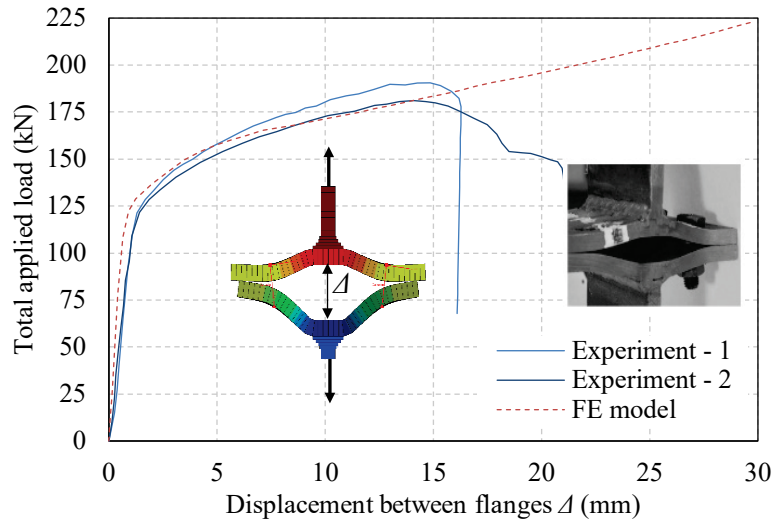
and T-stub connection tests and such tests (Draganić, Dokšanović & Markulak, 2014; Coelho et al., 2004) are used for the validation of the modelling technique employed herein.

The behaviour of bolts in tension was validated against T-stub test results from Coelho et al. (2004). Two tests were considered herein in which the failure occurred by bolt fracture. Each of the tested 12 mm bolts was represented by a fastener element (*FASTENER, CONN3D2, RADIAL-THRUST (ABAQUS, 2021)) in the FE model and assigned the generic bolt model described in Section 3.3.2 and shown in Figure 3.3a. Steel plates were assigned the quad-linear material model for hot-rolled steel described in Section 3.2.1. The welds between the plates were modelled using *MPC tie constraints (ABAQUS, 2021) and the fillet zones were modelled by increasing the thickness of the web plate elements adjacent to the intersection with the flange (Yun, Gardner & Boissonnade, 2018). Four elements were used to model the 8 mm weld with the thickness of the element determined to satisfy the required second moment of area of the welded T-section. Figure 3.4a shows load-displacement curves for the T-stub connections, where close agreement can be seen between the FE model and the test results. The test fails at a lower load level compared with the numerical model because the bending in the bolts is not captured in the spring behaviour assigned to the bolts herein. The adopted modelling approach nonetheless remains valid for use in the current study since none of the bolts in the full frame models reached the nonlinear stage in tension.

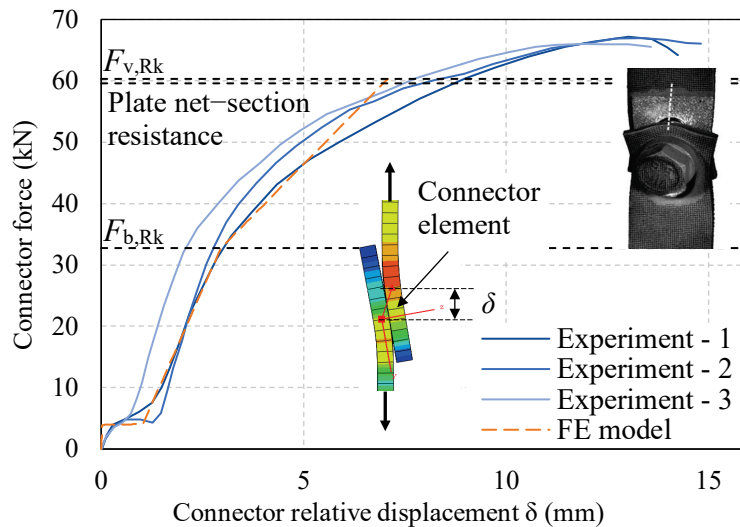
The modelling approach for bolts in combined shear and bearing was validated against three lap-shear tests reported by Draganić et al. (2014). The 16 mm bolt in each test was represented by a fastener element and assigned the generic bolt model as detailed in Section 3.3.2 and shown in Figure 3.3b. The clamping force was assumed to be 25% of the ultimate tensile strength of the bolt with a slip of 1 mm to match that observed during the tests. The steel plates were assigned the quad-linear material model for hot-rolled steel described in Section 3.2.1. A comparison between the numerical and experimental results is shown in Figure 3.4b, demonstrating good agreement.

Note that for the gable post cleat to gable post connections, a slot is introduced in the cleat to reduce the vertical load carried by the gable posts and thus prevent premature failure (Hayward & Weare, 2002). This is modelled herein by omitting the clamping stage of the load deformation response of the bolt and extending the slip stage by extending the bolt-hole diameter in the direction of the slot to ensure that no vertical load is transferred through the bolted connection to the gable post. Additionally, the stiffness of the slip stage is reduced to 10

N/mm (approximately 0.06% of the combined shear and bearing stiffness). Consequently, a cartesian connector element is used (*FASTENER, CONN3D2, CARTESIAN (ABAQUS, 2021)) to allow for the definition of the different slip behaviour in the two shear directions.



(a) T-stub connections



(b) Lap-shear connections

Figure 3.4: Comparison of load-deformation curves for hot-rolled steel bolted T-stub and lap-shear connections from FE model and tests reported in Coelho et al. (2004) and Draganić, Dokšanović and Markulak (2014), respectively

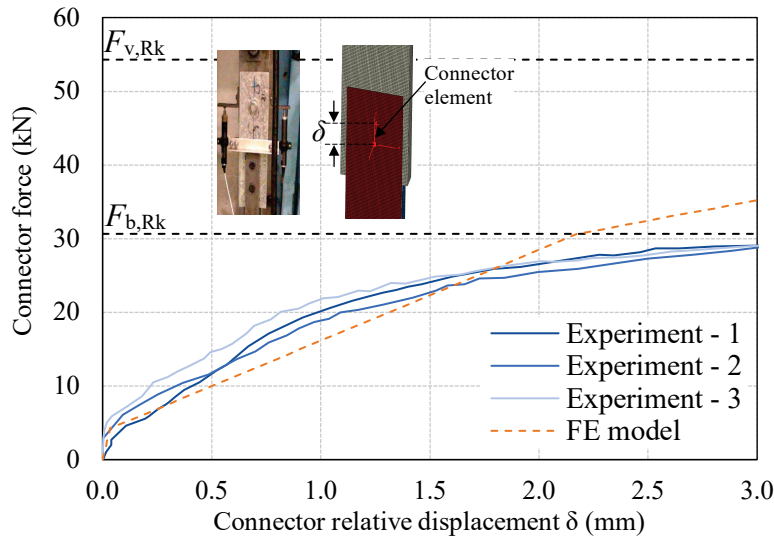
3.3.4 Secondary steelwork bolted connections

Secondary cold-formed steelwork bolted connections arise between cold-formed steel secondary steelwork members as well as between cold-formed steel members and their corresponding hot-rolled steel cleats or weld plates. The key behavioural aspects of these

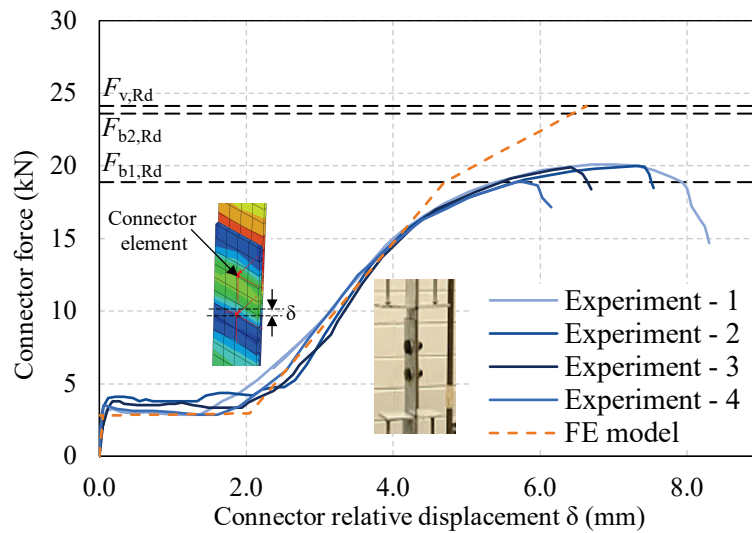
connections can be captured through lap-shear tests and such tests (Chung & Ip, 2000; Rinchen & Rasmussen, 2019) are used for the validation of the modelling technique employed herein.

The key behavioural aspects for bolted connections between secondary steelwork members and their cleats or weld plates are captured through lap-shear tests of a bolted connection between a hot-rolled steel plate and a cold-formed steel strip. The modelling approach for these bolts in combined shear and bearing was validated against three lap-shear tests reported by Chung and Ip (2000). The 12 mm bolt was represented by a fastener element (*FASTENER, CONN3D2, RADIAL-THRUST (ABAQUS, 2021)) and assigned the axial and transverse behaviour detailed in Section 3.3.2 and shown in Figures 3.3a and b, respectively. The clamping force was assumed to be 30% of the ultimate tensile strength of the bolt to match that observed during the tests and no slip was modelled since there was no bolt-hole clearance in the test specimens. The steel plate was assigned the quad-linear material model for hot-rolled steel while the steel strip was assigned the two-stage Ramberg-Osgood material model for cold-formed steel (both described in Section 3.2.1). The measured geometric and material properties were employed in the FE model. Figure 3.5a shows good agreement between the numerical model and the tests. Note that the experiments were stopped at a displacement of 3 mm when they were considered to have failed in bearing.

The key behavioural aspects for bolted connections between secondary steelwork members are captured through lap-shear tests of a bolted connection between cold-formed steel strips. The modelling approach for these bolts in combined shear and bearing was validated against four lap-shear tests performed by Rinchen and Rasmussen (2019). Each 8 mm bolt was represented by a fastener element (*FASTENER, CONN3D2, RADIAL-THRUST (ABAQUS, 2021)) and assigned the axial and transverse behaviour detailed in Section 3.3.2 and shown in Figures 3.3a and b, respectively. The clamping force was assumed to be 45% of the ultimate tensile strength of the bolt to match that observed during the tests and the steel strips were assigned the two-stage material model for cold-formed steel described in Section 3.2.1. The measured geometric and material properties were input into the numerical model. Figure 3.5b shows good agreement between the numerical model and the tests.



(a) Hot-rolled steel to cold-formed steel bolted lap-shear connection



(b) Cold-formed steel bolted lap-shear connection

Figure 3.5: Comparison of load-deformation curves for secondary steelwork bolted lap-shear connections from FE model and tests reported in Chung and Ip (2000) and Rinchen and Rasmussen (2019), respectively.

3.3.5 Cladding primary connections

Cladding primary connections are the fasteners that connect and transfer the load between the cladding systems and their supporting secondary cold-formed steelwork. Different types are used based on the cladding system employed (MCRMA, 2000). Sandwich panels, as utilised in around half of all portal frames constructed in the UK (Davies, Roberts & Wang, 2021) and are thus the focus of this study, predominantly utilise self-drilling self-tapping screws that connect from the external sheet through the internal sheet to the purlin flange, as shown in Figure 3.6a. Each sandwich panel primary screw connects three elements: the external metal

sheet of the cladding, the internal metal sheet of the cladding, and the supporting secondary cold-formed steel member flange. In the FE models, the sandwich panel primary screws were modelled explicitly using beam finite elements (ABAQUS, 2021) to capture their bending stiffness, as shown in Figure 3.6b, with the nonlinear material properties obtained by Vella et al. (2020) for 80 mm long self-drilling self-tapping steel screws. At each interface, two connector elements were then used to represent the interface between the screw and the connected material: 1) a translational connector element (*CONN3D2, RADIAL-THRUST at the interface with the metal sheets or *CONN3D2, CARTESIAN (ABAQUS, 2021) at the interface with the supporting secondary cold-formed member) to represent the stiffness in shear and bearing as well as the axial stiffness, and 2) a rotational connector element (*CONN3D2, CARDAN (ABAQUS, 2021)) to represent the rotational stiffness.

At the interface between the sandwich panel primary screw and each of the metal sheets, the combined behaviour of shear and bearing was represented by a radial-thrust connector element, with the load-deformation behaviour was captured by a three-stage model based on K pplein and Misiek (2011). The model, shown in Figure 3.6c, is defined based on the EN 1993-1-3 (CEN, 2003) bearing resistance equation and extensive testing on a range of sheeting thicknesses, steel grades and screw diameters. The bearing resistance $F_{b,Rk}$ and the screw stiffness S_{v+b} in the linear stage are calculated using Eqs. (3.24) and (3.26), respectively, where d_1 is the minor diameter of the screw thread, calculated by Eq. (3.25), where d_s is the diameter of the unthreaded part of the fastener. In addition to the connectors above, a cartesian connector element (*CONN3D2, CARTESIAN (ABAQUS, 2021)) with negligible stiffness of 10 N/mm (approximately 0.2% of the radial-thrust initial stiffness) is coupled to each radial-thrust element to prevent convergence issues.

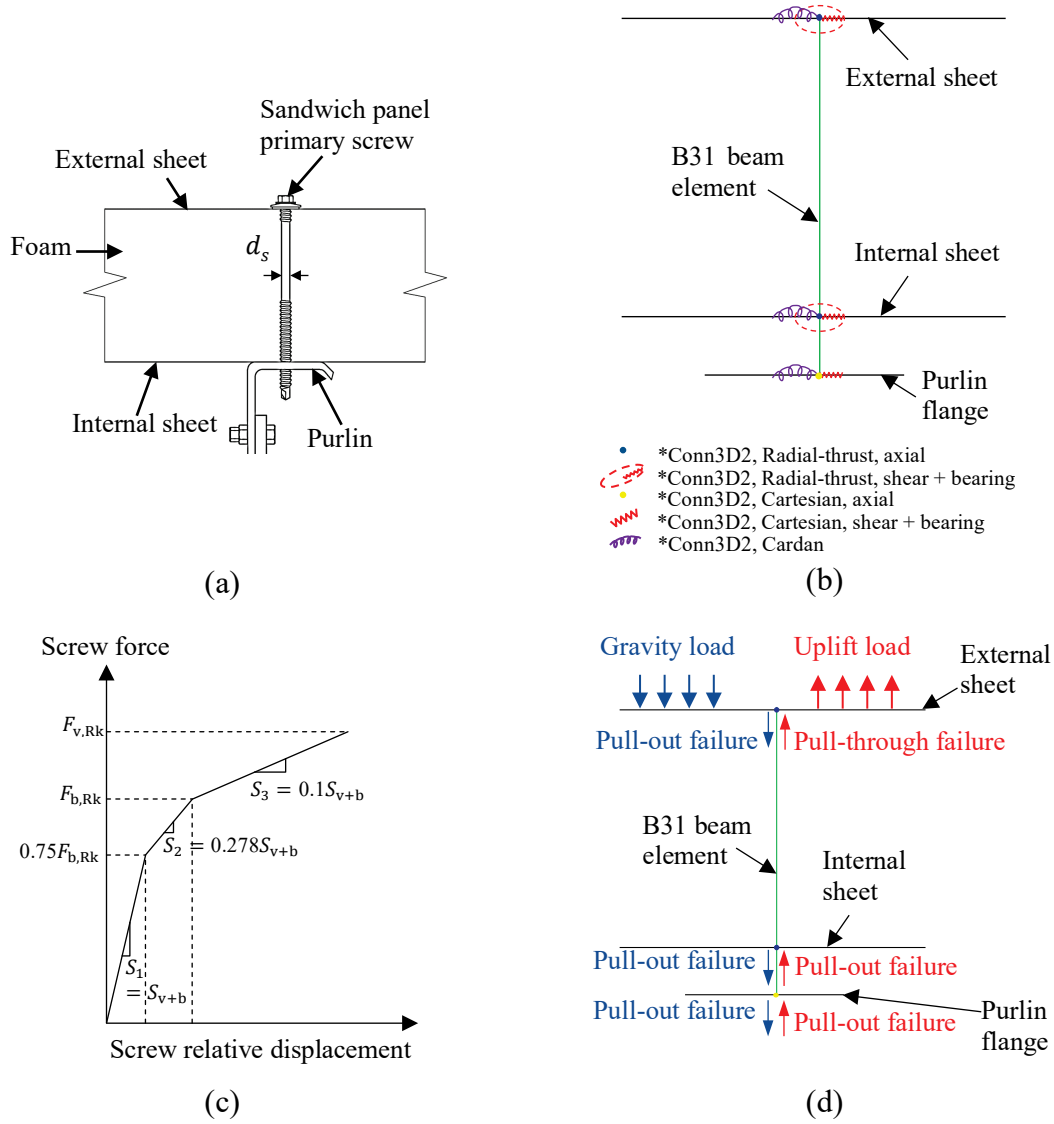


Figure 3.6: (a) Components of a sandwich panel connection, (b) Sandwich panel primary screw connectors, (c) Proposed generic tri-linear model for load-deformation behaviour of sandwich panel primary screw connectors in combined shear and bearing, (d) Failure criteria of the sandwich panel primary screw connectors in the axial direction.

$$F_{b,Rk} = 4.2f_u\sqrt{d_1t^3}, \text{ with } t \text{ and } d \text{ in mm and } f_u \text{ in N/mm}^2 \quad (3.24)$$

$$d_1 = 0.9d_s \quad (3.25)$$

$$S_{v+b} = \begin{cases} 6.93 \frac{f_u\sqrt{d_1t^3}}{0.26 + 0.8t} & 0.4 \text{ mm} \leq t \leq 0.7 \text{ mm} \\ 4.2 \frac{f_u\sqrt{d_1t^3}}{0.373} & 0.7 \text{ mm} \leq t \leq 1.0 \text{ mm} \end{cases} \quad (3.26)$$

The interface between the sandwich panel primary screw and the supporting secondary cold-formed member is assumed to act as the origin of rotation of the sandwich panel primary screw

due to the greater thickness compared to the metal sheets (MCRMA, 2021). Hence, the displacements at this point can be neglected and the stiffness in combined shear and bearing at this interface was assumed to be rigid and replicated by a rigid cartesian element.

The axial clamping of the thread into the metal sheets and the supporting secondary cold-formed member was replicated by a rigid stiffness and a failure criterion based on EN 1993-1-3 (CEN, 2003). The failure criterion at the interface with the external metal sheet was defined as the pull-through resistance $F_{p,Rk}$ in tension, as calculated by Eq. (3.27), where d_w is the diameter of the screw washer and as the pull-out resistance $F_{o,Rk}$ in compression, as calculated by Eq. (3.28), where s is the screw thread pitch, as illustrated in Figure 3.6d. The failure criteria at both the internal metal sheet and the supporting member interfaces were defined as the pull-out resistance $F_{o,Rk}$.

$$F_{p,Rk} = d_w t f_u \quad (3.27)$$

$$F_{o,Rk} = \begin{cases} 0.45 d t f_u & \text{for } t/s < 1 \\ 0.65 d t f_u & \text{for } t/s \geq 1 \end{cases} \quad (3.28)$$

The rotational stiffness was modelled using a cardan connector element with a stiffness equivalent to 10% of the sandwich panel primary screw bending stiffness. Note that this is based on the assumption that the sandwich panel primary screw is nominally pinned due to the clamping of the screw into the steel.

3.3.6 Cladding secondary connections

The cladding secondary connections, also known as cladding stitching connections, are the fasteners between adjacent cladding panels, as well as between the cladding and the metal flashings (i.e. the apex hat, corner flashing and barge cap). A range of mechanical fasteners can be used in these connections such as bolts, screws, pins and rivets.

The key behavioural aspects of the cladding secondary screws in combined shear and bearing can be captured by a four-stage model based on K  pplein and Misiek (2011), as shown in Figure 3.7a. The bearing resistance $F_{b,Rk}$ for screws connecting steel sheets is calculated using Eq. (3.29), where f_u is the ultimate tensile strength of the two connected metal sheets, d is the nominal diameter of the screw, t is the thickness of the thinner of the two connected metal sheets and t_1 is the thickness of the thicker of the two connected metal sheets.

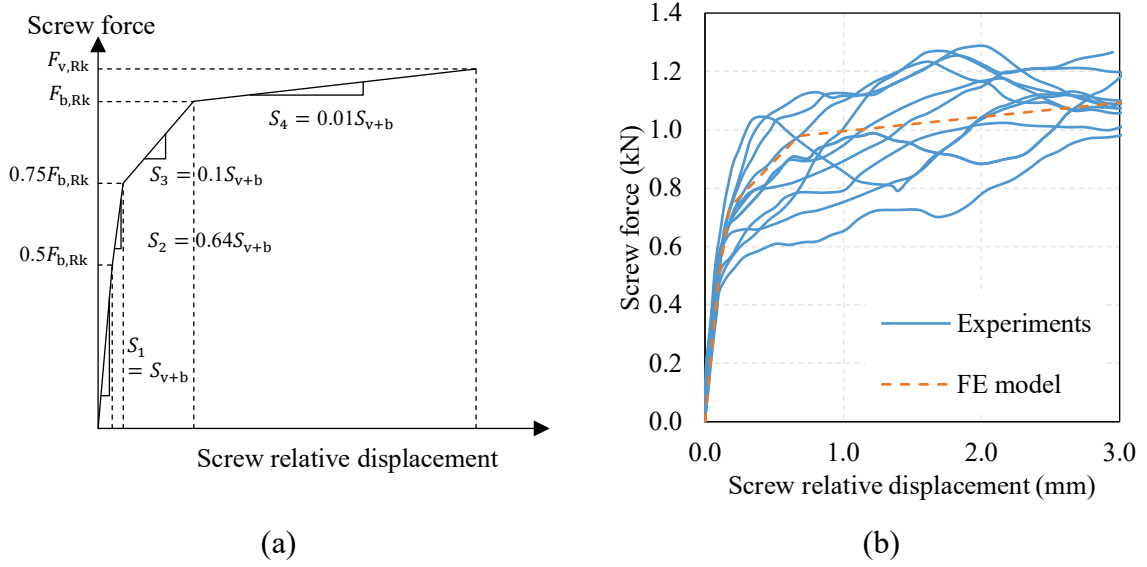


Figure 3.7: (a) Proposed generic load-deformation behaviour of cladding secondary screw connectors under combined shear and bearing, (b) comparison of load-deformation curves for cladding secondary screw connectors from FE model and tests reported in K  pplein and Misiek (2011).

$$F_{b,Rk} = \alpha f_u d t, \text{ with } t \text{ and } d \text{ in mm and } f_u \text{ in N/mm}^2 \quad (3.29)$$

$$\alpha = \begin{cases} 3.2\sqrt{t/d} \leq 2.1 & \text{for } t = t_1 \\ 3.2\sqrt{t/d} \leq 2.1 & \text{for } t_1 \geq 2.5t \text{ and } t < 1.0 \text{ mm} \\ 2.1 & \text{for } t_1 \geq 2.5t \text{ and } t \geq 1.0 \text{ mm} \\ \text{obtained by linear interpolation} & \text{for } t < t_1 < 2.5t \end{cases} \quad (3.30)$$

The screw stiffness in the linear range S_1 (when the screw force is less than $0.5F_{b,Rk}$) is calculated using Eq. (3.31). The second stage stiffness S_2 is defined by K  pplein and Misiek (2011) as $0.64 S_{v+b}$ while the third and fourth stage stiffnesses S_3 and S_4 were calibrated herein based on the results of the lap-shear tests detailed in K  pplein and Misiek (2011) and taken as $S_3 = 0.1 S_{v+b}$ and $S_4 = 0.01 S_{v+b}$, respectively. A comparison between the proposed model and test results for a 0.47 mm thick sheet and a 5.5 mm screw diameter is shown in Figure 3.7b with good agreement achieved.

$$S_{v+b} = 1900 t d, \text{ with } t \text{ and } d \text{ in mm to give } S_{v+b} \text{ in N/mm} \quad (3.31)$$

The behaviour of the cladding secondary screws in tension is characterised by the same axial stiffness as the bolts, calculated by Eq. (3.13), and a failure load taken as the smaller of the pull-through resistance $F_{p,Rk}$ and the pull-out resistance $F_{o,Rk}$, as calculated using Eqs. (3.27) and (3.28), respectively.

3.4 Cladding modelling and validation

In this section, the modelling techniques employed for two cladding systems – single skin trapezoidal sheeting and sandwich panels – commonly utilised in steel portal frames are introduced and validated, as described in Sections 3.4.1 and 3.4.2, respectively. For computational efficiency, the trapezoidal corrugated steel sheeting used in both systems is represented by an equivalent orthotropic plate.

3.4.1 Single-skin trapezoidal sheeting

Equivalent orthotropic plate

Using the equivalent energy method, the constitutive equation of a corrugated sheet can be approximated to an orthotropic classic Kirchhoff (flat) plate that retains the original material properties of the corrugated plate (Xia, Friswell & Flores, 2012). The stiffness properties of the corrugated plate can be calculated by considering one single repeating unit referred to as a representative volume element (RVE), as illustrated in Figure 3.8. Two coordinate systems of the corrugations are defined: (1) the global (x, z) coordinate system, and (2) the local (s, n) coordinate system. It should be noted that the y axis projects into the page.

The constitutive equation of an orthotropic plate in the x-y plane can be expressed by Eq. (3.32), where $\bar{\epsilon}_x$, $\bar{\epsilon}_y$, $\bar{\gamma}_{xy}$, $\bar{\kappa}_x$, $\bar{\kappa}_y$ and $\bar{\kappa}_{xy}$ refer to the strain and curvature components, and $\bar{N}_x$, $\bar{N}_y$, $\bar{N}_{xy}$, $\bar{M}_x$, $\bar{M}_y$ and $\bar{M}_{xy}$ refer to the force and moment components. The $\bar{A}_{ij}$ terms denote the axial and shear stiffnesses, while the $\bar{D}_{ij}$ terms denote the bending and twisting stiffnesses. The A_{ij} and D_{ij} terms refer to the corresponding properties of the corrugated sheet being represented. Utilising the equivalent energy method, Xia, Friswell and Flores (2012) approximated the stiffness terms in Eq. (3.32) as a function of the corrugated geometry, as detailed in Table 3.4, where c is the half period of the RVE (see Figure 3.8), l is the half length, calculated as the sum of the lengths of the inclined and horizontal parts of an RVE ($l_1 + l_2 + 2l_3$), and the integrals I_1 and I_2 are calculated using Eqs. (3.33) and (3.34), respectively.

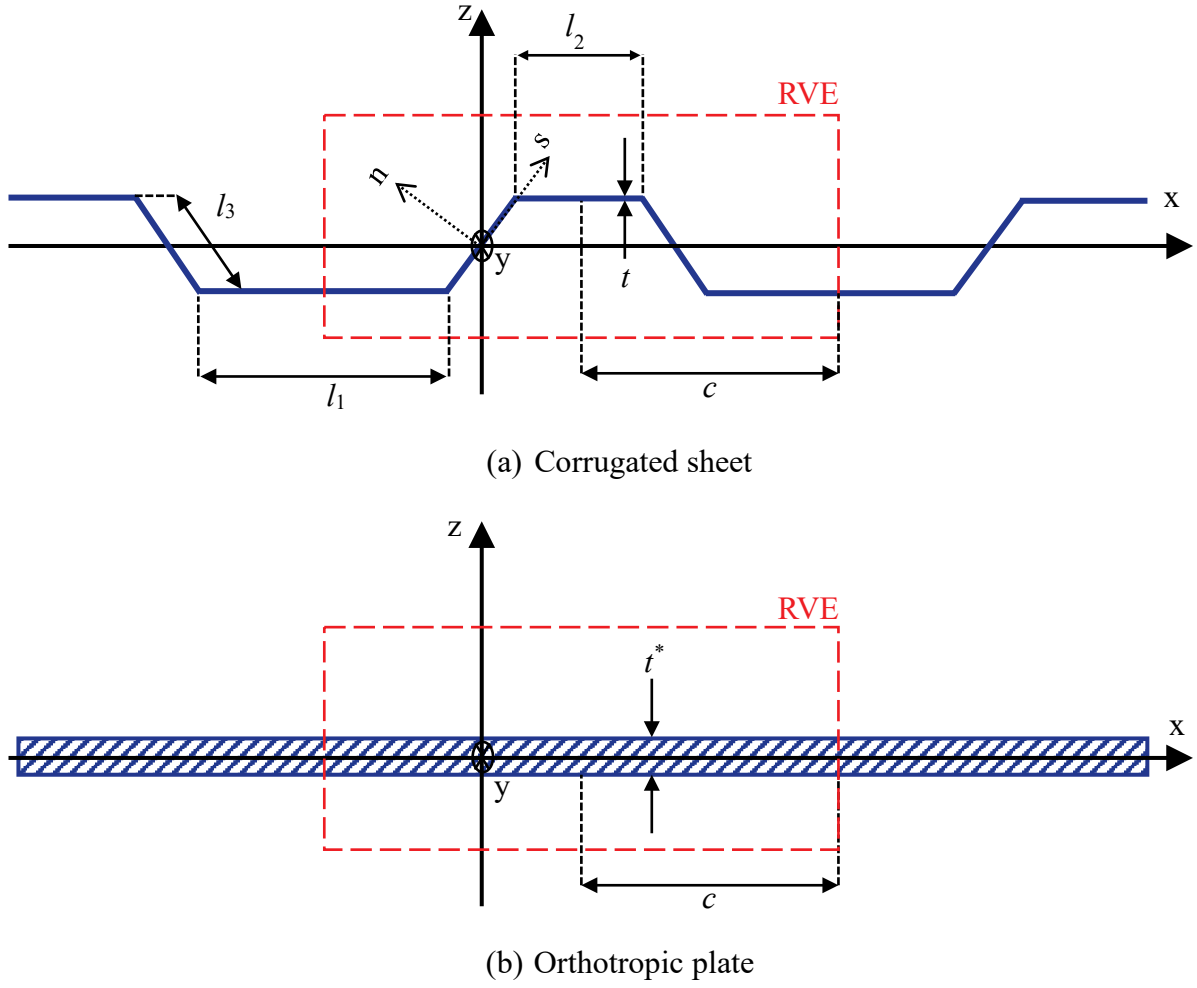


Figure 3.8: RVE of corrugated sheet and orthotropic plate in adopted coordinate systems, with the y axis into the page.

$$\begin{Bmatrix} \bar{N}_x \\ \bar{N}_y \\ \bar{N}_{xy} \\ \bar{M}_x \\ \bar{M}_y \\ \bar{M}_{xy} \end{Bmatrix} = \begin{bmatrix} \bar{A}_{11} & \bar{A}_{12} & 0 & 0 & 0 & 0 \\ \bar{A}_{12} & \bar{A}_{22} & 0 & 0 & 0 & 0 \\ 0 & 0 & \bar{A}_{66} & 0 & 0 & 0 \\ 0 & 0 & 0 & \bar{D}_{11} & \bar{D}_{12} & 0 \\ 0 & 0 & 0 & \bar{D}_{12} & \bar{D}_{22} & 0 \\ 0 & 0 & 0 & 0 & 0 & \bar{D}_{66} \end{bmatrix} \begin{Bmatrix} \bar{\epsilon}_x \\ \bar{\epsilon}_y \\ \bar{\gamma}_{xy} \\ \bar{\kappa}_x \\ \bar{\kappa}_y \\ \bar{\kappa}_{xy} \end{Bmatrix} \quad (3.32)$$

$$I_1 = \int_0^{2l} \left(\frac{dx}{ds} \right)^2 ds \quad (3.33)$$

$$I_2 = \int_0^{2l} z^2 ds \quad (3.34)$$

Validation

In this section, the 0.46 mm Everclad L5 profile trapezoidal corrugated sheeting (later adopted in the full frame model validation (Bryan, 1971b; Davies & Bryan, 1982), as further detailed in Chapter 4), will be used to validate the orthotropic modelling approach used herein. Both the principal stiffness terms of the corrugated sheet as well as those calculated for the orthotropic plate using the expressions given in Table 3.4 are presented in Table 3.5.

Table 3.4: Stiffness properties of an orthotropic place representing a trapezoidal corrugated sheet

Stiffness terms in Eq. (3.32)	Expression from Xia, Friswell and Flores (2012)
$\bar{A}_{11}$	$\frac{2c}{\frac{I_1}{A_{11}} + \frac{I_2}{D_{11}}}$
$\bar{A}_{12}$	$\frac{A_{12}}{A_{11}} \bar{A}_{11}$
$\bar{A}_{22}$	$\frac{\bar{A}_{12} A_{12}}{A_{11}} + \frac{l}{c} \frac{A_{11} A_{22} - A_{12}^2}{A_{11}}$
$\bar{A}_{66}$	$\frac{c}{l} A_{66}$
$\bar{D}_{11}$	$\frac{c}{l} D_{11}$
$\bar{D}_{12}$	$\frac{D_{12}}{D_{11}} \bar{D}_{11}$
$\bar{D}_{22}$	$\frac{1}{2c} (I_2 A_{22} + I_1 D_{22})$

Table 3.5: Stiffness properties of corrugated and equivalent orthotropic plates

Corrugated plate	A_{11} (N/mm)	A_{22} (N/mm)	A_{66} (N/mm)	D_{11} (N/mm)	D_{22} (N/mm)
	95220	95220	36623	1679	1679
Equivalent orthotropic plate	$\bar{A}_{11}$ (N/mm)	$\bar{A}_{22}$ (N/mm)	$\bar{A}_{66}$ (N/mm)	$\bar{D}_{11}$ (N/mm)	$\bar{D}_{22}$ (N/mm)
	28.36	97598	3216	1490	5637085

The stiffness properties are used to determine the equivalent thickness t^* and material properties – the Young's moduli in the two orthotropic in-plane directions E_{11} and E_{22} , the Poisson's ratio ν_{12} and the shear modulus G_{12} .

Firstly, solving for both axial and flexural rigidity in the direction perpendicular to the corrugation (x-axis in Figure 3.8), t^* and E_{11} can be determined as:

$$\begin{cases} \bar{A}_{11} = E_{11} t^* \\ \bar{D}_{11} = \frac{E_{11} t^{*3}}{12} \end{cases} \Rightarrow \begin{cases} t^* = 25.11 \text{ mm} \\ E_{11} = 1.13 \text{ MPa} \end{cases} \quad (3.35)$$

Next, the Young's modulus in the other orthotropic direction in the plane E_{22} can be determined by satisfying the axial term:

$$\bar{A}_{22} = E_{22} t^* \Rightarrow E_{22} = 3886.14 \text{ MPa} \quad (3.36)$$

Thirdly, the Poisson's ratio ν_{12} can be calculated by dividing the coupling term $\bar{A}_{12}$ by its principal term $\bar{A}_{22}$:

$$\nu_{12} = \bar{A}_{12} / \bar{A}_{22} = 8.7 \times 10^{-5} \quad (3.37)$$

Finally, the shear modulus G_{12} is calculated as:

$$\bar{A}_{66} = G_{12} t^* \Rightarrow G_{12} = 1294.70 \text{ MPa} \quad (3.38)$$

These properties were compared against those obtained for the corresponding single RVE of the corrugated sheet under applied axial load, shear force and bending moment at the neutral axis. The principal stiffness terms were all found to be within 0.5% except for the bending stiffness in the direction of the corrugation $\bar{D}_{22}$, which had an error of 8%. The equivalent thickness t^* was calculated in the 11 direction and thus only one of the axial rigidity $\bar{A}_{22}$ and bending rigidity $\bar{D}_{22}$ can be further satisfied; $\bar{A}_{22}$ was chosen herein since it is the most relevant term for a plate deforming predominately in-plane (rather than out-of-plane).

The orthotropic plate model was also validated against a numerical model of a larger corrugated panel (3080×750 mm) loaded in shear, as illustrated in Figure 3.9. The shear force was applied along Purlin 2 with the load transferred directly to the sheet through rigid MPC ties at the fastening locations (at the middle of each trough). Boundary conditions were applied along the bottom flanges of each purlin, with Purlin 2 restrained in U1 and U2 and Purlin 1 restrained in all degrees of freedoms. Figure 3.9 shows the load-displacement response of the two panels; it can be seen that the response of the corrugated sheet in shear is accurately captured by the orthotropic plate model.

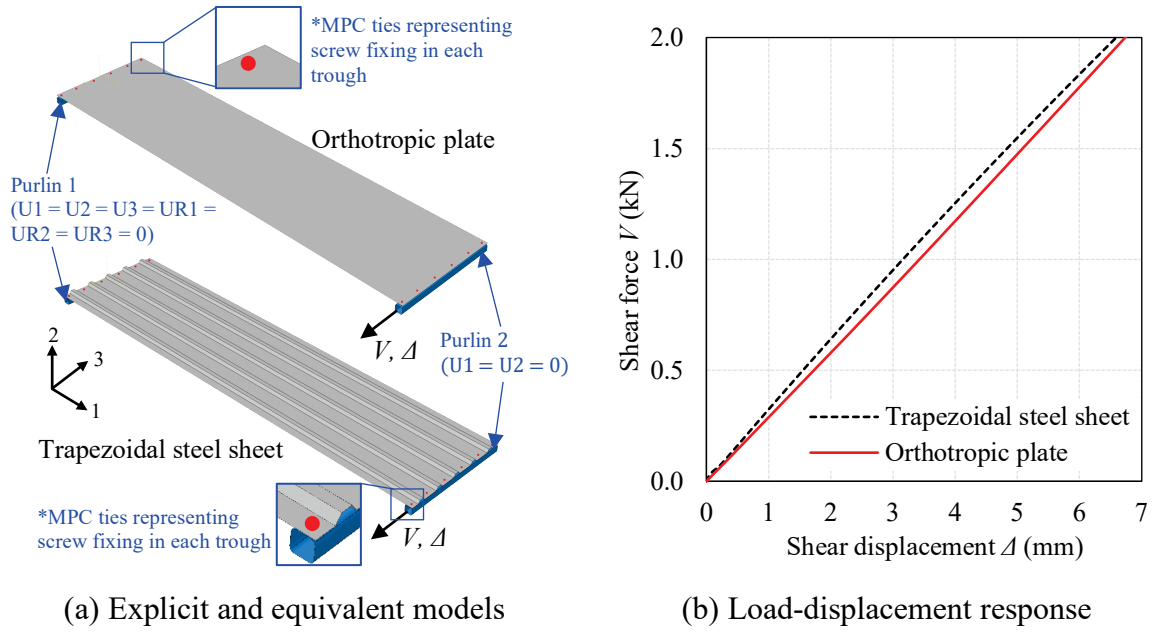


Figure 3.9: Validation of modelling technique used herein for trapezoidal sheeting

3.4.2 Sandwich panel cladding

Equivalent model for sandwich panels

A typical corrugated sandwich panel is formed of three layers – an internal metal sheet, a layer of foam in the middle and an external metal sheet. The two corrugated steel sheets are each modelled herein by an equivalent orthotropic plate following the method detailed in Section 3.4.1. The foam layer indirectly contributes to the stiffness of the cladding through separation of the two sheets and prevention of buckling. However, if the foam is equated to an equivalent steel plate, the thickness is four orders of magnitude smaller than the outer sheets and thus the direct rigidity contribution of the foam can be ignored. Therefore, the foam is simply represented in the model through separation of the two steel sheets with a gap equal to the foam thickness and compression springs connecting perpendicular nodes of the two sheeting layers, with a stiffness k calculated from Eq. (3.39), where A is the surface area of a single cladding finite element, E is the Young's modulus of the insulation foam and D is the sandwich panel depth.

$$k = \frac{AE}{D} \quad (3.39)$$

Validation

In this section, the approach outlined above is validated against a 6000 mm × 1000 mm corrugated sandwich panel tested under shear by Kunkel and Lange (2015). The thickness t , Young's modulus E , shear modulus G and Poisson's ratio ν for each component of the

sandwich panel are given in Table 3.6. The Young's modulus for the insulation was not reported in the tests, so a typical value of 2.5 GPa (Gahlen et al., 2021) was adopted herein. Following the modelling technique outlined in Section 3.4.1, the equivalent thickness t^* and orthotropic properties of each plate of the sandwich panel were determined, as listed in Table 3.7.

Table 3.6: Material properties for corrugated sandwich panel (Kunkel & Lange, 2015)

Component	t (mm)	E (MPa)	ν	G (MPa)
External steel sheet	0.63	210000	0.3	81000
Polyurethane (PU) foam	100	2500*	-	-
Internal steel sheet	0.5	210000	0.3	81000

* (Gahlen et al., 2021)

Table 3.7: Material properties for equivalent orthotropic system

Component	t^* (mm)	E_1 (MPa)	E_2 (MPa)	ν_{12}	G_{12} (MPa)
External steel sheet	5.56	295	22434	0.004	8867
Internal steel sheet	5.55	149	17837	0.002	7053

The panel itself, and a purlin at each edge perpendicular to the corrugation of the sheeting as the substructure, were modelled explicitly – see Figure 3.10a. The sandwich panel primary screws connecting the panel and the purlins were modelled using the approach described in Section 3.3.5 and illustrated in Figure 3.6. A shear force was imposed along Purlin 2 and boundary conditions were applied at the bottom flanges of both purlins, with U1 and U2 restrained along the loading edge (Purlin 2) and all degrees of freedom restrained along Purlin 1. Figure 3.10b shows load-displacement curves for the two FE models (one in which the corrugations were explicitly modelled and the second with the orthotropic plates) and the experimental results, with good agreement achieved.

To investigate the relative contribution of the different components to the overall shear flexibility, the two panels were also modelled with rigid MPC ties to connect the panel and the purlins (in place of the deformable sandwich panel primary screws). These results are also plotted in Figure 3.10b where it can be seen that the two curves are in good agreement, with a marked increase in stiffness compared to the models with the sandwich panel primary screw connections. Therefore, in agreement with the findings of Kunkel and Lange (2015), it can be concluded that the shear flexibility of the sandwich panel is mainly associated with the screw connections. A magnified view highlighting the shear displacement up to 20 mm is also shown

in Figure 3.10b; this corresponds approximately to the maximum level of shear displacements observed in the parametric study conducted herein (see Chapter 5).

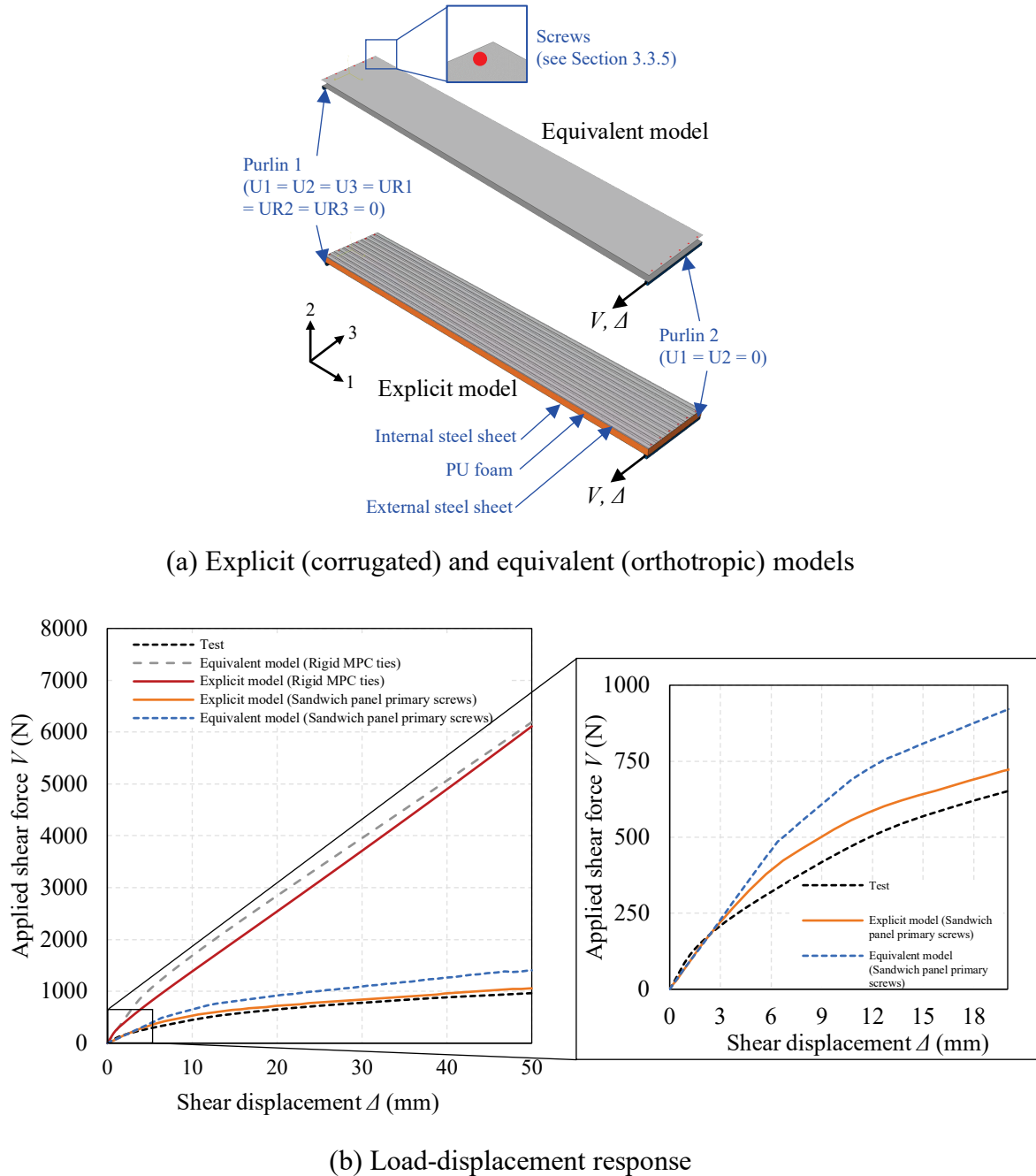


Figure 3.10: Validation of modelling technique used herein for sandwich panel cladding system against experimental results of Kunkel and Lange (2015)

The slight deviation of the equivalent model from the explicit model can be attributed to the following reasons: 1) geometric nonlinearity, as the equivalent orthotropic plate is derived using linear theory, and 2) limitations in the method employed to calculate the equivalent

thickness t^* and the material properties. Following the equivalent orthotropic approach explained in Section 3.4.1, the flexural rigidity along the corrugation direction was not accurately captured. This is justified by the fact that the axial rigidity is more important than the flexural rigidity along the corrugation, especially in the linear stage when the sandwich panel is subject to shear. However, in the direction perpendicular to the corrugations, both axial and flexural rigidities are crucial to capture the localised effects between corrugations.

3.5 Concluding remarks

In this chapter, the modelling approaches for the typical connections and cladding systems employed in steel portal frame design have been described and validated. The models for the material stress-strain response of hot-rolled steel and cold-formed steel were described. Generic models to capture the load-displacement response for typical bolted and screwed portal frame connections were proposed and validated against physical experiments. The modelling approach utilised to model the portal frame cladding systems as orthotropic plates was explained and validated against physical experiments. These validated modelling approaches are incorporated into the full portal frame models in the following chapters.

Chapter 4

Full frame finite element modelling: fundamentals and validation

4.1 Introduction

In this chapter, the fundamental aspects of the finite element modelling employed to model full steel portal frames are presented. The full frame model is validated against the test results from an experimental programme on a five bay portal frame in Section 4.3.1 and a two bay portal frame in Section 4.3.2.

4.2 Fundamental aspects of finite element modelling

The general-purpose finite element (FE) software ABAQUS (2021) was used to develop the full frame FE models. Typical components of the assembled full frame FE model are shown in Figure 4.1. Shell finite elements (type S4R (ABAQUS, 2021)) were used to model the main frame, haunches, end plates, purlins, side rails, eaves beams, cleats, bracing, cladding, metal flashings and gutters to accurately capture local cross-section behaviour. Truss elements (T3D2 (ABAQUS, 2021)) were used to model the anti-sag rods and diagonal tie wires and beam elements (B31(ABAQUS, 2021)) were used to model the rafter and column stays and anti-sag struts.

Suitable material properties were assigned to the different components as described in Section 3.2.1. The material choice and steel grades adopted herein were guided by industrial partners who are members of a steering committee involved in this research. The columns, rafters, haunches, stiffeners, end plates, gusset plates and cleats were assigned the quad-linear material model for hot-rolled grade S355 steel while the tubular bracing members were assigned the two-stage material model for cold-formed grade S355 steel. The purlins, eaves beams, siderails, diagonal stays, anti-sag rods members, metal flashings and gutters were assigned the two-stage

material model for cold-formed grade S450 steel. Note that diagonal tie wires behave only in tension, and thus the material properties were assigned exclusively in tension using the keyword *No compression (ABAQUS, 2021). The cladding was assigned elastic orthotropic material properties as explained in Section 3.4. The mesh size was defined such that an aspect ratio ranging between 1:1 and 1:2 was ensured and was reduced further at the corners and fillet zones to approximately 1:10. The shear stiffness of the orthotropic cladding plate was not sensitive to mesh size, allowing for a larger mesh size in the cladding. A mesh size averaging 180 mm was employed in this case. However, the mesh size of the cladding was reduced near the connections and contact zones to align with the purlins, siderails, and eaves beams to accurately capture the local effects associated with the connectors.

The sandwich panel cladding adopted for the FE models within the scope of this research comprises the KS1000 panels provided by Kingspan® (Kingspan, 2015; Kingspan, 2020b) that utilises rigid QuadCore® polyisocyanurate foam (PIR) as insulation. The insulation was modelled using compression springs following the approach explained in Section 3.4.2. For the spring stiffness, a value of 1×10^6 N/mm was adopted, determined based on an E value of 2.5 GPa for PIR foam (MatWeb, 2023), an average mesh size of 180 mm and a panel depth of 80 mm.

Python scripting was utilised to automate the process of creating the FE models with full parametric capabilities, ensuring, for example, reasonable element aspect ratios, mesh compatibility and numerical stability, while allowing for flexibility of all design parameters.

4.2.1 Boundary conditions

Boundary conditions at column bases are generally either pinned or fixed; however, practical pinned connections have a degree of rotational stiffness, and this is typically considered through the use of an assigned nominal stiffness equal to 10% of the column flexural stiffness (Koschmidder & Brown, 2012; Salter, Malik & King, 2004). Pinned conditions were achieved through the kinematic coupling of the member end cross-section nodes to a reference node in the middle of the cross-section, to which relevant boundary conditions were applied, while nominally pinned conditions (i.e. a 10% base stiffness) were achieved through the addition of a rotational spring to the column base reference node with a stiffness equal to 10% of the column stiffness (equal to $0.4EI/L$, where E is the Young's modulus, I is the second moment of area of the column and L is the length of the column).

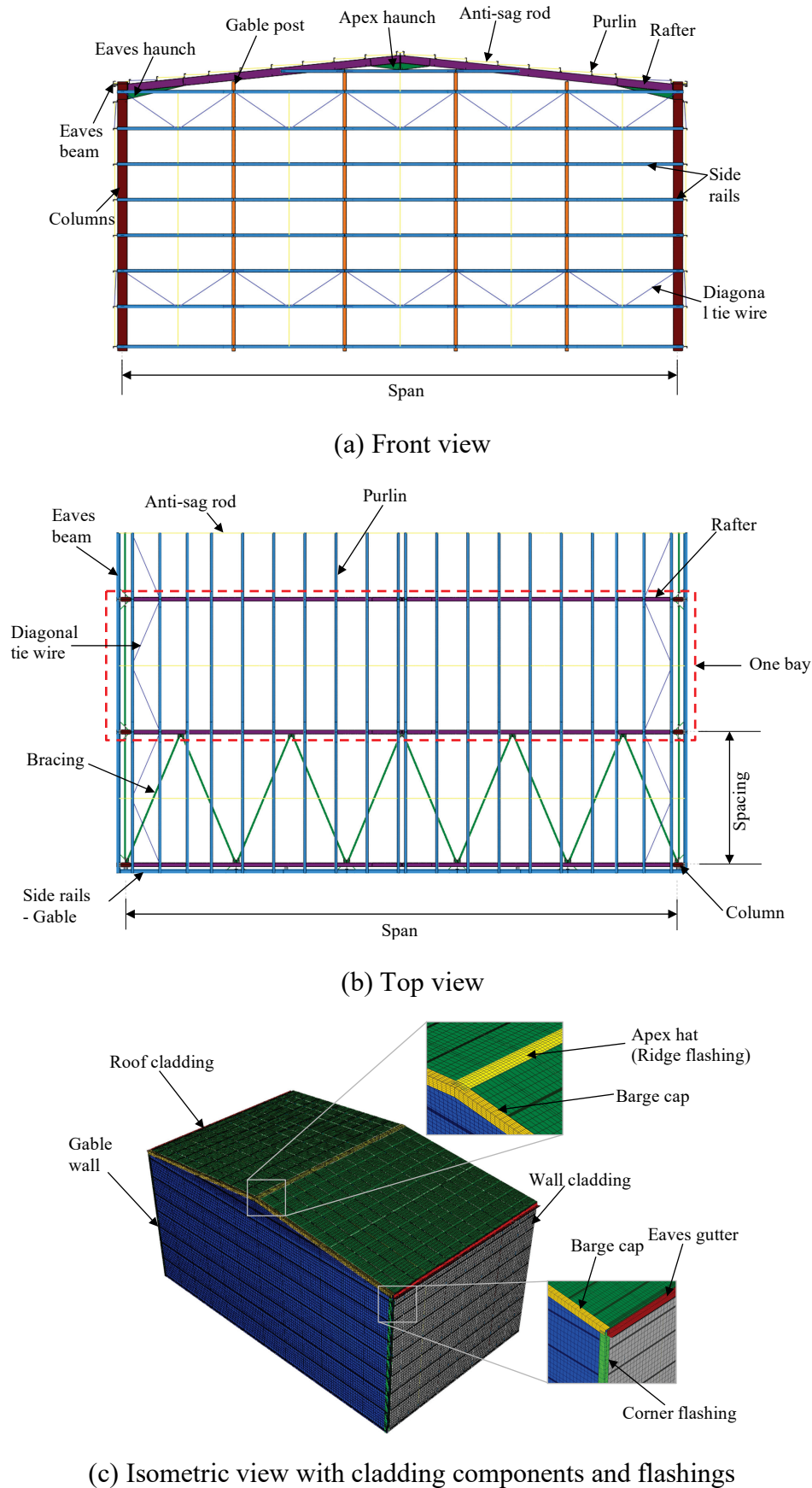


Figure 4.1: Typical components of full frame FE model

In the case of symmetrical load conditions and/or frame arrangements, symmetry boundary conditions were used to increase the efficiency of the simulations. Symmetry boundary conditions are applied in one or two directions depending on the loading condition applied. Symmetry about the z-axis is used in the wind sway loading cases to model half the length of the frame, while symmetry about the x-axis is used in the wind uplift loading cases to model half of the span of the frame. Symmetry about both axes was used in the gravity loading and thermal action cases.

4.2.2 Connection details

A combination of fastener elements, beam elements, multi-point constraints (MPC), tie constraints (TIE), springs and contact surfaces were used to define the interactions and connection details. MPC tie, MPC beam and tie constraints were used to simulate the welded connections from the primary steel members to the haunches, web stiffeners, end plates, gusset plates and cleats, as well as the welded connection between the anti-sag struts and their cleats. MPC tie and MPC beam were used when the mesh of the welded elements was aligned, while tie constraints were used when mesh alignment between the two connected elements was not achievable. MPC beam was used when there was a gap between the nodes representing the connected elements thickness while MPC tie was used where there was no gap. The interaction between bolted surfaces was modelled using hard normal contact and frictional contact with the friction coefficient μ taken as 0.2 (CEN, 1992).

Primary frame connections

The details of the primary frame connections within the full frame finite element models are shown in Figures 4.2 and 4.3. M24 bolts were used for the eaves and apex joints and M20 bolts for the gable post cleat connections and bracing connections. The bolts were positioned to satisfy the minimum edge distance, end distance and spacing as outlined in EN 1993-1-8 (CEN, 2005c), while also following the vertical and horizontal cross-centres recommended by Brown et al. (2013). The number, size and locations of the bolts for the eaves and apex joints were designed using a proprietary estimating system by one of the industrial partners involved in the research. The number, size and locations of the bolts for gable post cleat and bracing connections were chosen based on standard current practice. For the gable post cleat connection, a slot of 100 mm was adopted to prevent vertical load from transferring to the gable post, as explained in Section 2.4.1.

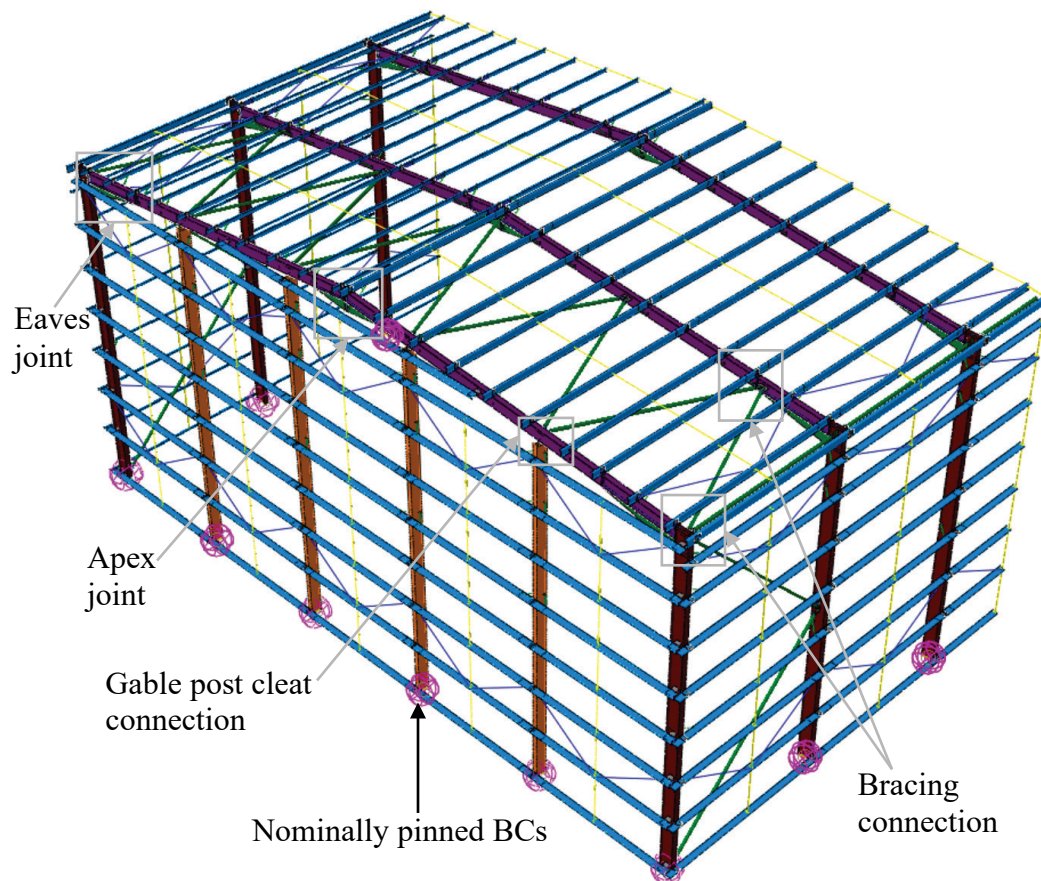
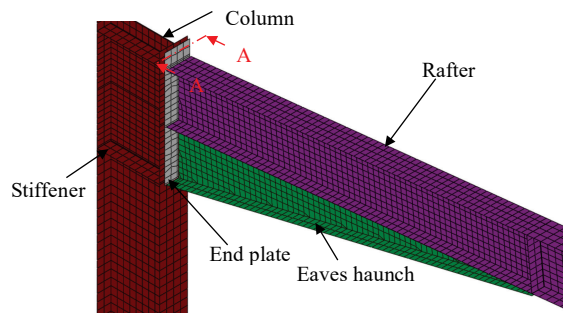
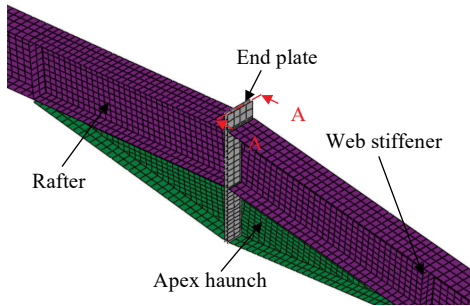
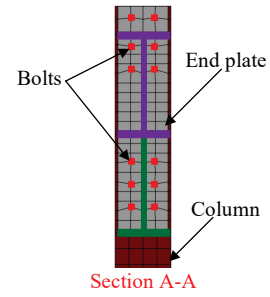


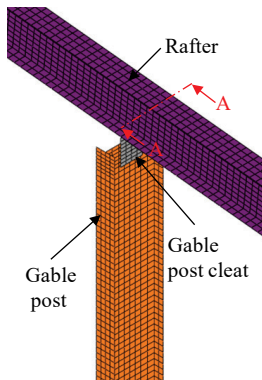
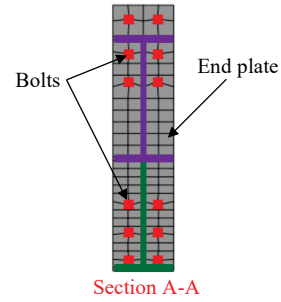
Figure 4.2: Primary steel frame connections



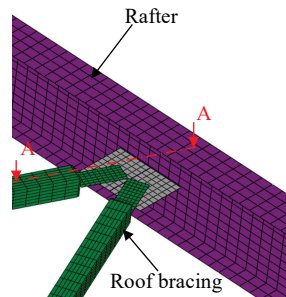
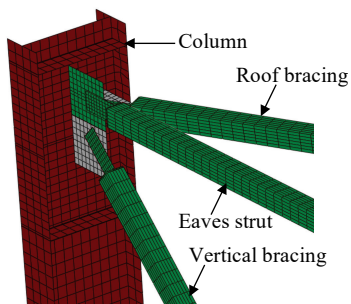
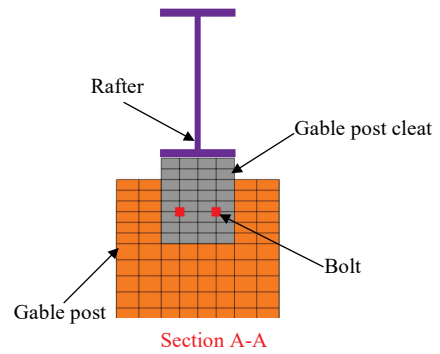
(a) Eaves joint



(b) Apex joint



(c) Gable post cleat connection



(d) Bracing connection

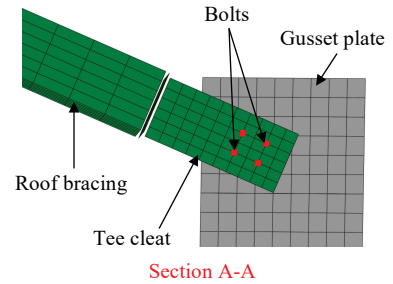


Figure 4.3: Typical arrangement for primary frame connections

Secondary steelwork connections

The details of the secondary steelwork connections for the full frame finite element models are shown in Figures 4.4 and 4.5. M16 bolts were used for the purlin, side rail and eaves beam cleat connections, as well as for the diagonal bracing connections. The number and locations of the bolts are typically provided by the manufacturer and the data sheets provided by METSEC (2017) were adopted for the frames modelled herein.

Anti-sag members can either utilise rods or struts based on the size of the connected purlin or side rail. In the case of anti-sag rods, the connection was simulated with a simple *MPC beam between the rod and the purlin web, as shown in Figure 4.5g, while a mesh tie (*TIE) was used in the case of the struts. Diagonal tie wires are connected to the purlin or side rail web at one end and to the cleat of the next purlin or side rail at the other end, as shown in Figure 4.5h, and in both cases these connections were modelled using an MPC beam.

To ensure the continuity of purlins and side rails over frames, a sleeved system is implemented. The system employed was the single-span heavy end bay sleeved purlin system (METSEC, 2017), with a sleeve required at each internal frame. The length of the sleeve was determined based on the size of the purlin or side rail. At the penultimate frames, the sleeve length ranged from 914 to 1074 mm, while at all other internal frames, it ranged from 714 to 834 mm.

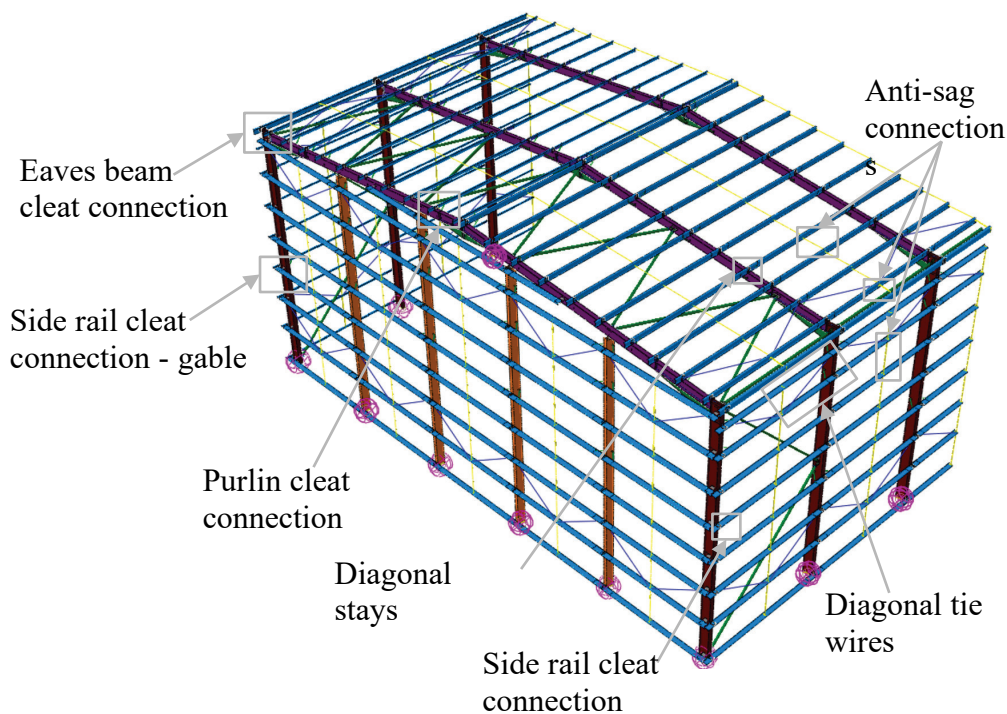


Figure 4.4: Secondary steelwork connections

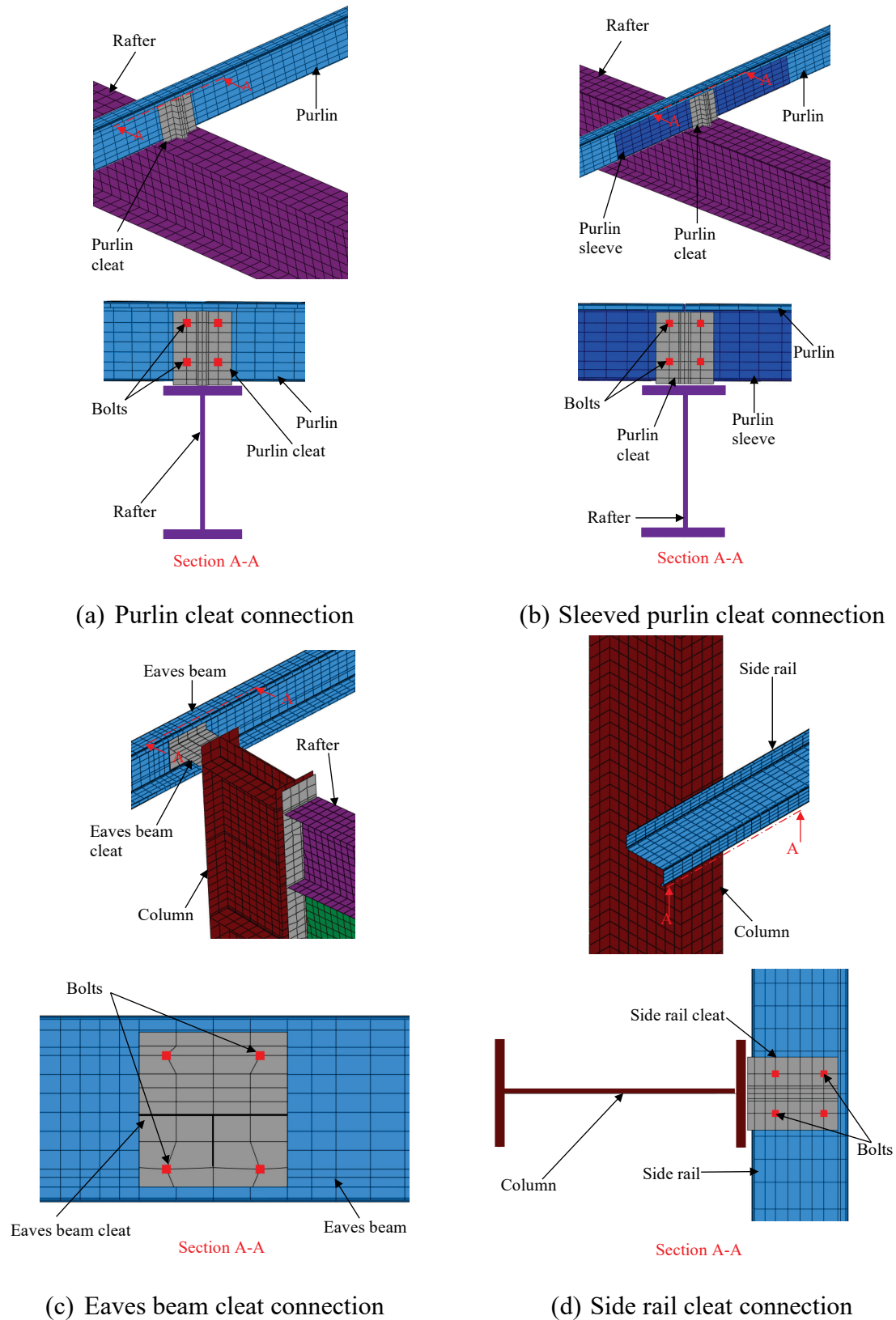


Figure 4.5: Typical arrangement for secondary steelwork connections

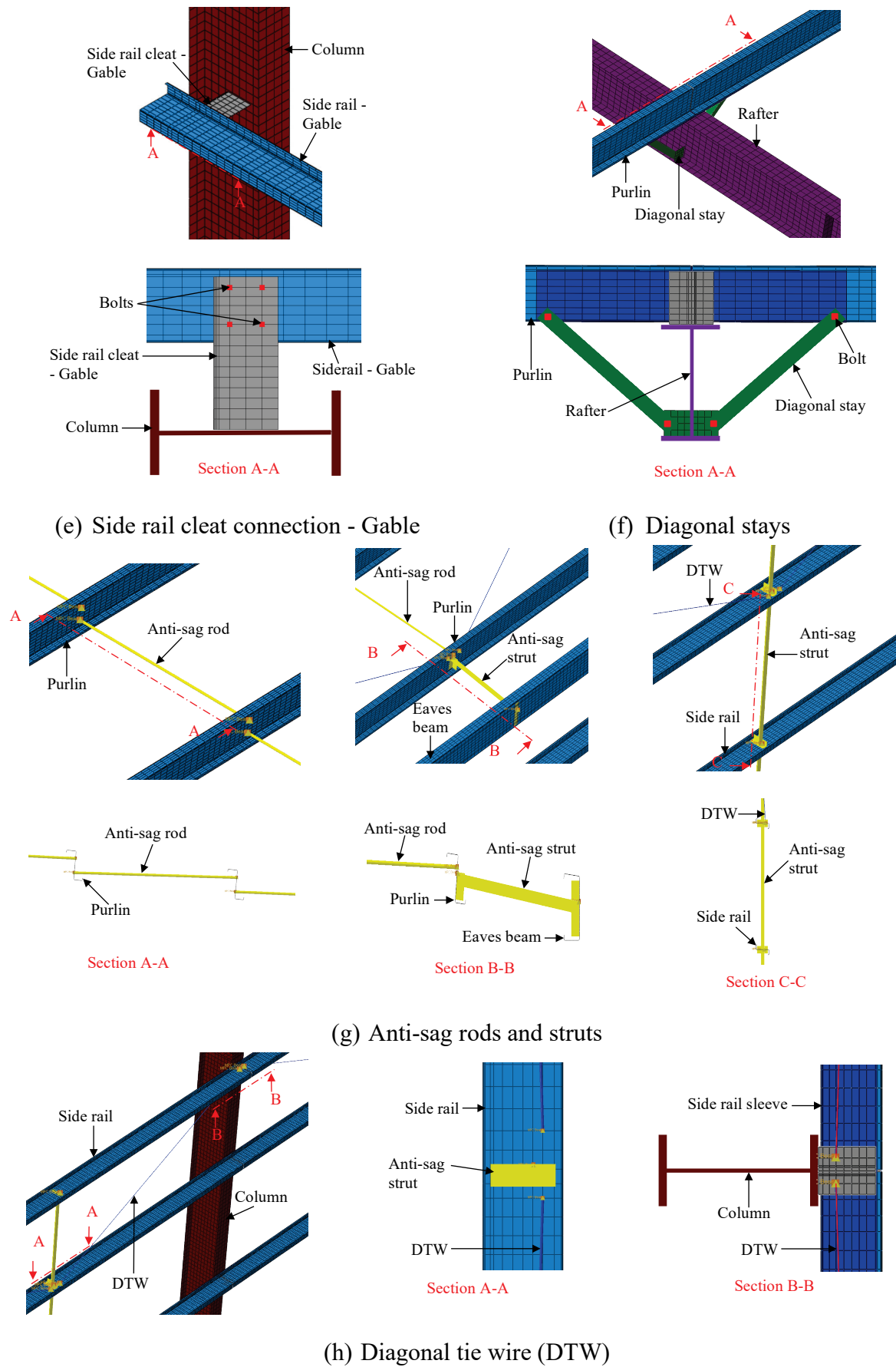


Figure 4.5 (continued): Typical arrangement for secondary steelwork connections

Cladding connections

The cladding primary and secondary connections were modelled following the approach explained in Sections 3.3.5 and 3.3.6, respectively. The diameter of the cladding primary screws was taken as 6.1 mm with a 19 mm washer diameter. Cladding primary screws were spaced at 333 mm which is equivalent to 3 primary screws per panel (Kingspan, 2020a), as shown in Figures 4.6.

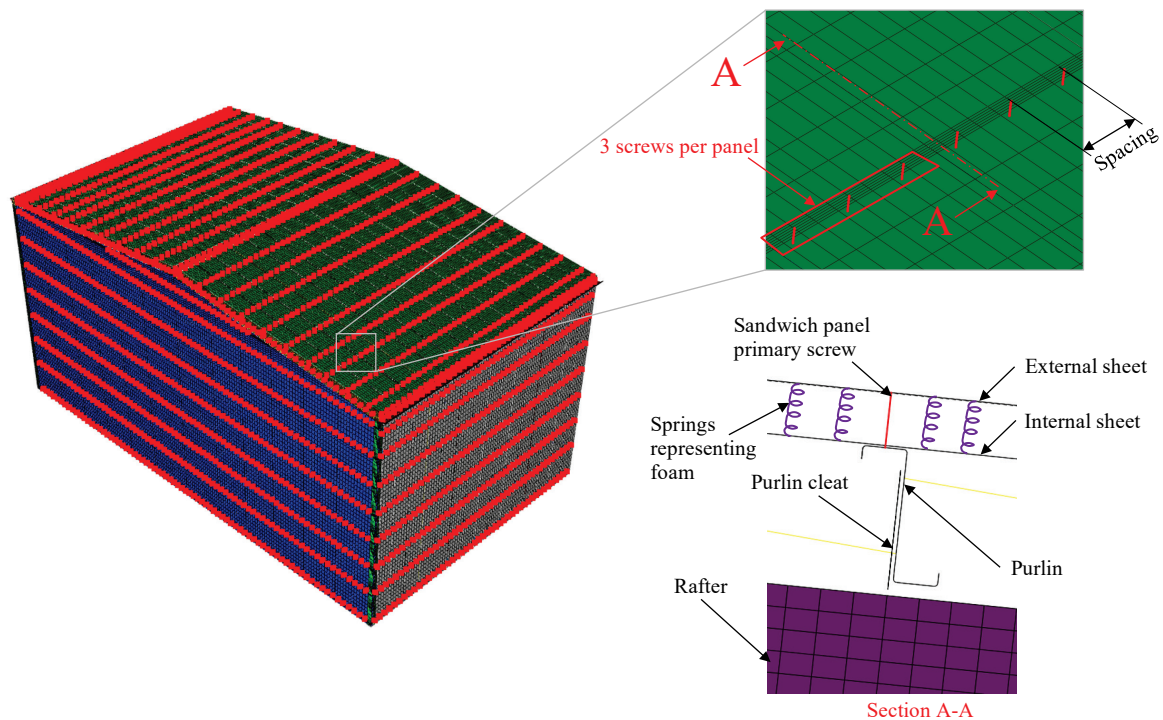


Figure 4.6: Primary cladding screw connections

The primary connections, as shown in Figures 4.7a, typically connect the external sheet to the supporting member through the internal sheet. In a few instances, summarised in Figure 4.7, additional members were also connected via the primary screw connection. For locations involving a sleeved purlin or side rail, the sleeve top flange was joined below the purlin top flange as shown in Figure 4.7b, with the connectors at the sleeve interface assigned the connector behaviour model detailed in Section 3.3.5 for the interface at the secondary cold-formed steelwork. Likewise, at the purlin near the apex, the internal ridge flashing (see Figures 4.7c and 4.12) was placed above the purlin top flange, with the connectors at the interface of the internal ridge flashing assigned the connector behaviour model detailed in Section 3.3.5 and illustrated in Figure 3.6 for the interface at the internal sheet. At the end lap of the sheeting panels, as illustrated in Figures 4.7d and 4.1, two external sheets were connected at the top of the primary screw adopting the connector behaviour model detailed in Section 3.3.5 and

illustrated in Figure 3.6 for the interface at the external sheet. In cases where a sleeved system coincided with the end lap or the internal ridge flashing, five elements were consequently connected via the primary screw connections.

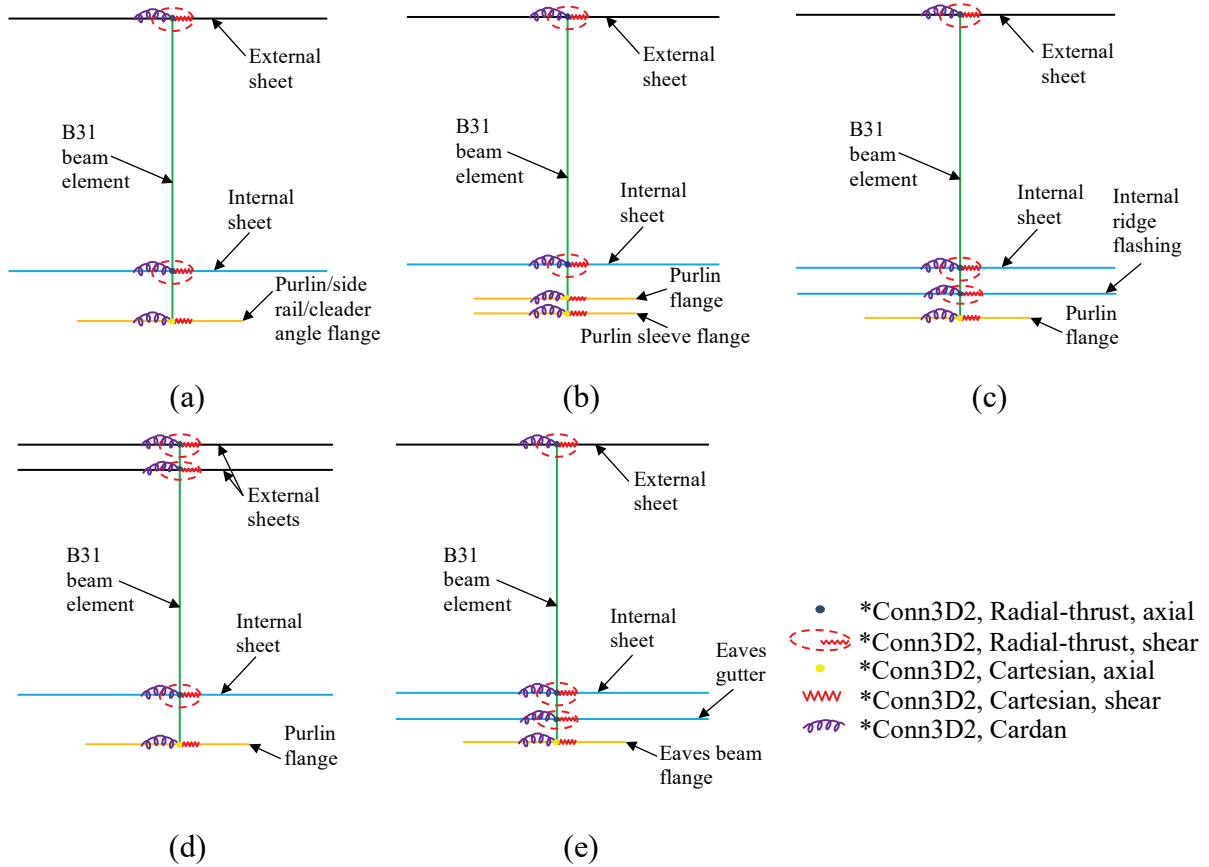


Figure 4.7: Summary of cladding primary screw arrangements

The eaves gutter system implemented in the FE models was an exposed gutter system and was placed between the eaves beam flange and the roof cladding. The cross-section was not specified in the manufacturer guide, so the cross-section shown in Figure 4.8 was used. Note that this implies that the cladding primary screw connection connecting the roof cladding to the eaves beam involved four connected elements as shown in Figure 4.7e. The connectors at the interface of the gutter system were assigned the connector behaviour model detailed in Section 3.3.5 and illustrated in Figure 3.6 for the interface at the internal sheet.

The cleader angle, also known as the internal barge cap, and shown in Figures 4.9 and 4.12, was connected to the purlin flange and the internal sheet of the gable cladding. The cleader angle was connected to the purlin using the approach detailed in Section 3.3.2 and illustrated in Figure 3.3 for secondary steelwork bolted connections, and was connected to the gable

cladding using the primary screw connection with the connector behaviour model detailed in Section 3.3.5 and illustrated in Figures 3.6 and 4.7a.

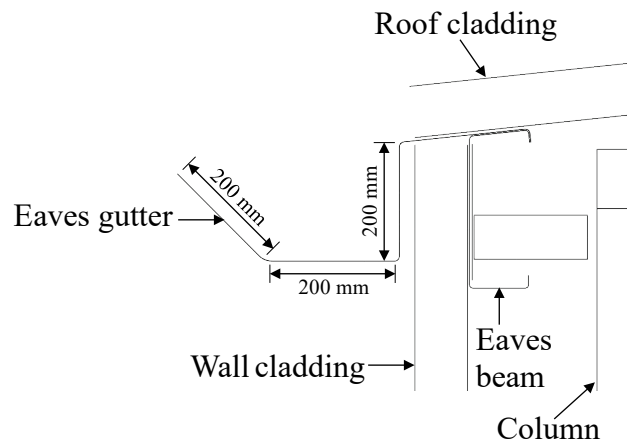


Figure 4.8: Cross-section of the exposed gutter system

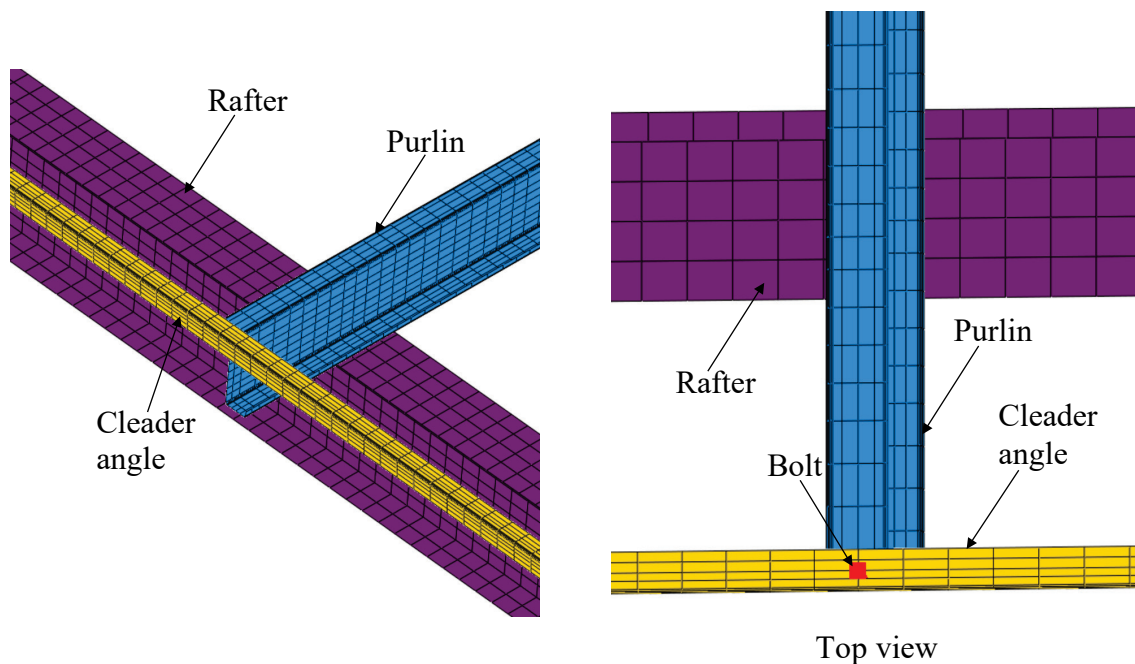


Figure 4.9: Cleader angle to purlin connection

Secondary cladding connections were utilised in the FE models for the longitudinal side laps, end laps and flashing seams, as illustrated in Figures 4.10, 4.11 and 4.12, respectively. For all secondary screws, a screw diameter of 4.8 mm was used with a 14 mm washer diameter. A spacing of 450 mm was adopted between the longitudinal side lap seam screws (see Figure 4.10), which is the maximum spacing allowed for the sandwich panel system utilised in the models (Kingspan, 2020a). As for the end lap seam screws, six screws per panel were used, spaced alternatively at 222 mm and 111 mm (see Figure 4.11), allowing for 3 primary screws

to be placed in the middle of the larger spacing (Kingspan, 2020a). Different spacings were assigned to the flashing seam screws, based on their locations (see Figure 4.12). Apex hat seam screws were positioned at every panel crown with a 333 mm spacing (Kingspan, 2020a). Barge cap seam screws were placed at every other panel crown of the gable wall cladding at a 666 mm spacing. Corner flashing was connected to the longitudinal side of the wall panels at a spacing of 400 mm.

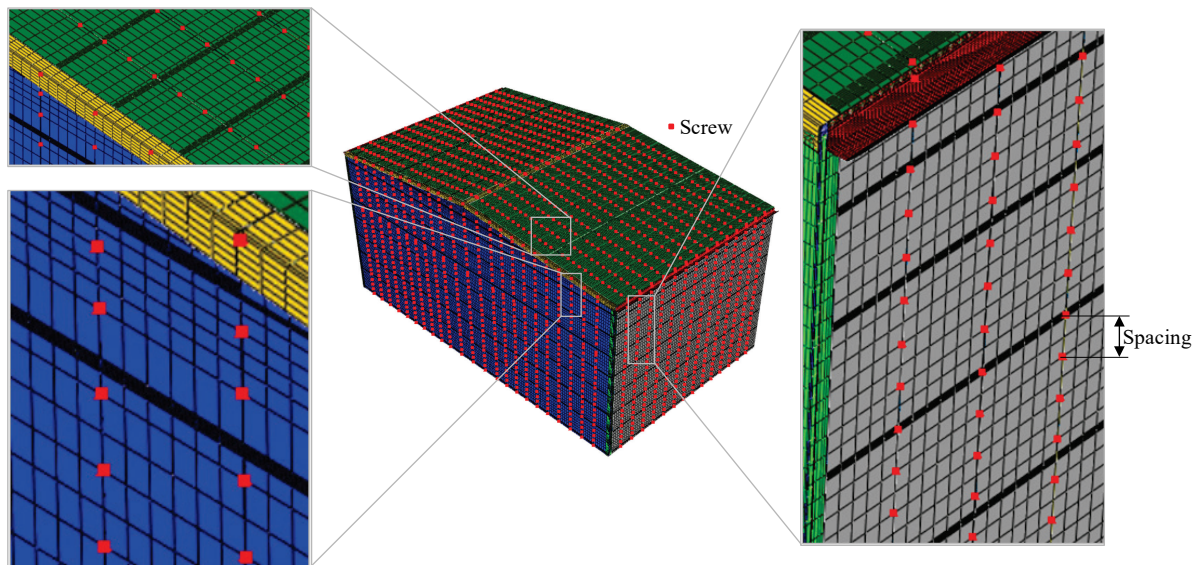


Figure 4.10: Longitudinal side lap seam screw connections

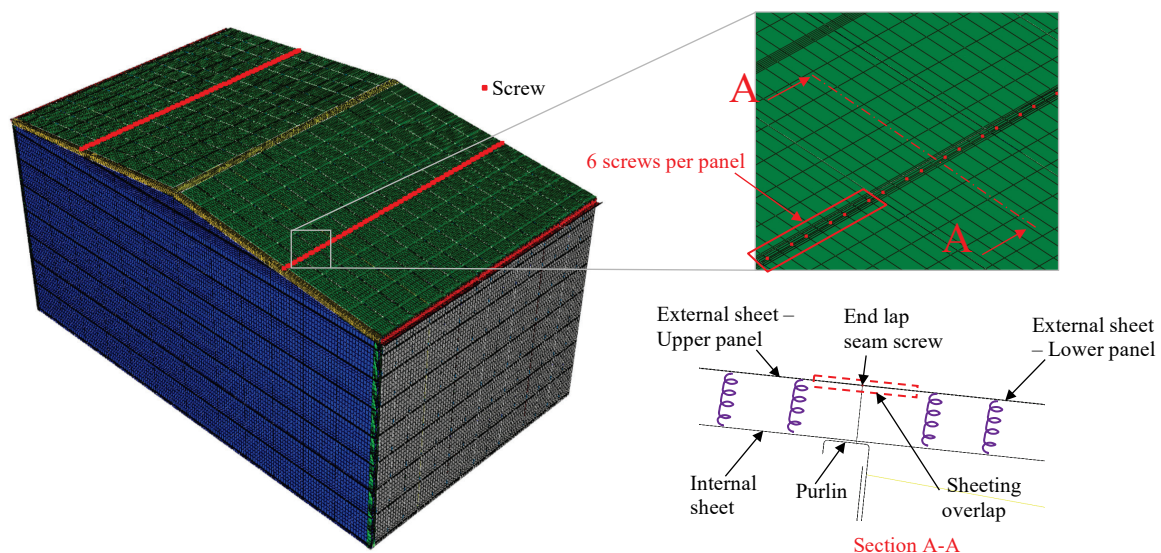


Figure 4.11: End lap seam screw connections

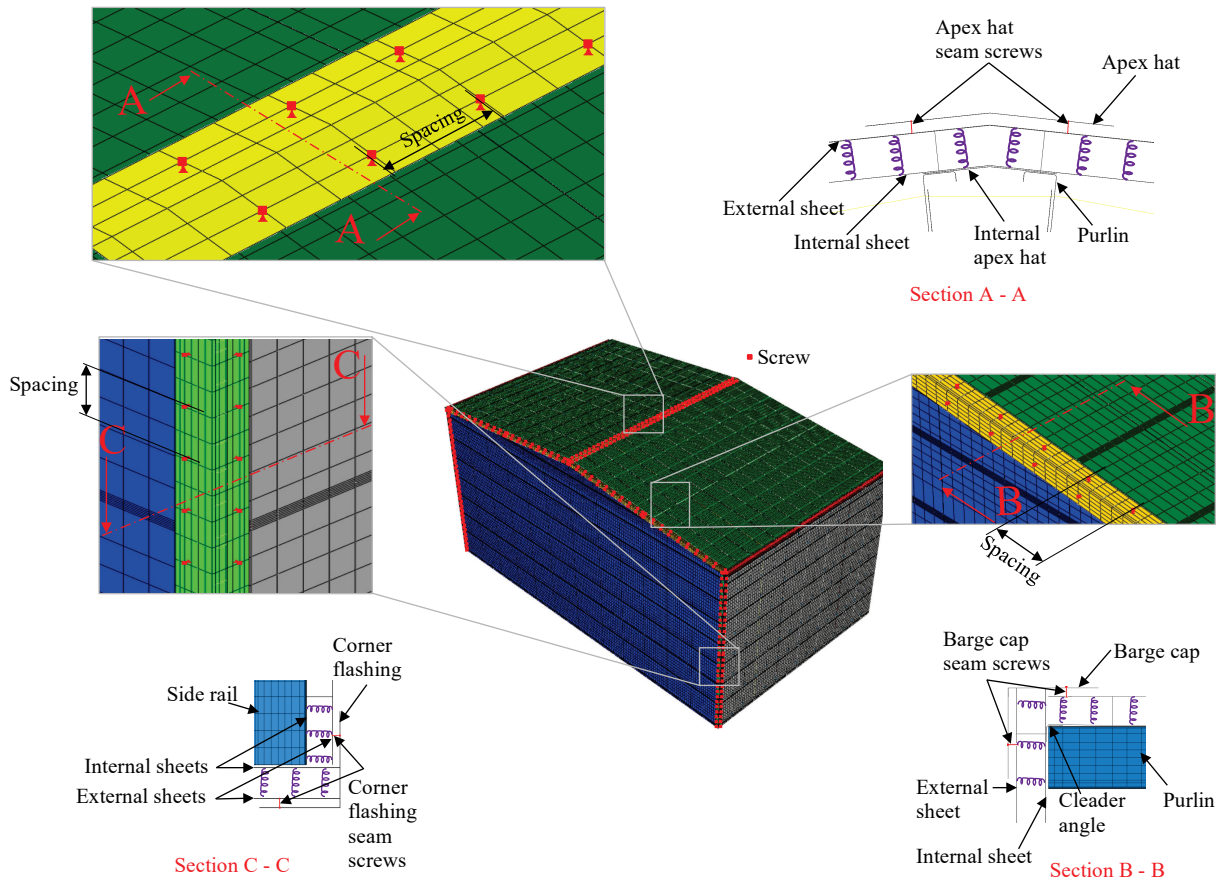


Figure 4.12: Metal flashings seam screw connections

4.3 Validation

4.3.1 Five bay portal frame

In this section, the full frame FE model is validated against the tests reported by Bryan (1971b), as illustrated in Figure 4.13, with further details presented in Davies and Bryan (1982) and Roberts, Davies & Wang (2021). The tests were conducted on a five bay portal frame with single-skin trapezoidal roof sheeting under three load cases: (1) a vertical design load of 0.48 kN/mm^2 applied uniformly on the sheeting, (2) a horizontal load of 5 kN applied at the eaves level of all internal frames and (3) a horizontal load of 5 kN applied at frame 3 only. Additionally, tests were carried out on a bare frame under two load cases: (1) a vertical design load of 42.5 kN applied as a series of point loads at the purlin-to-rafter intersections and (2) a horizontal load of 5 kN . The latter individual unclad test frame was connected to the clad frame by loosely bolted purlins during the tests to ensure adequate lateral stability.

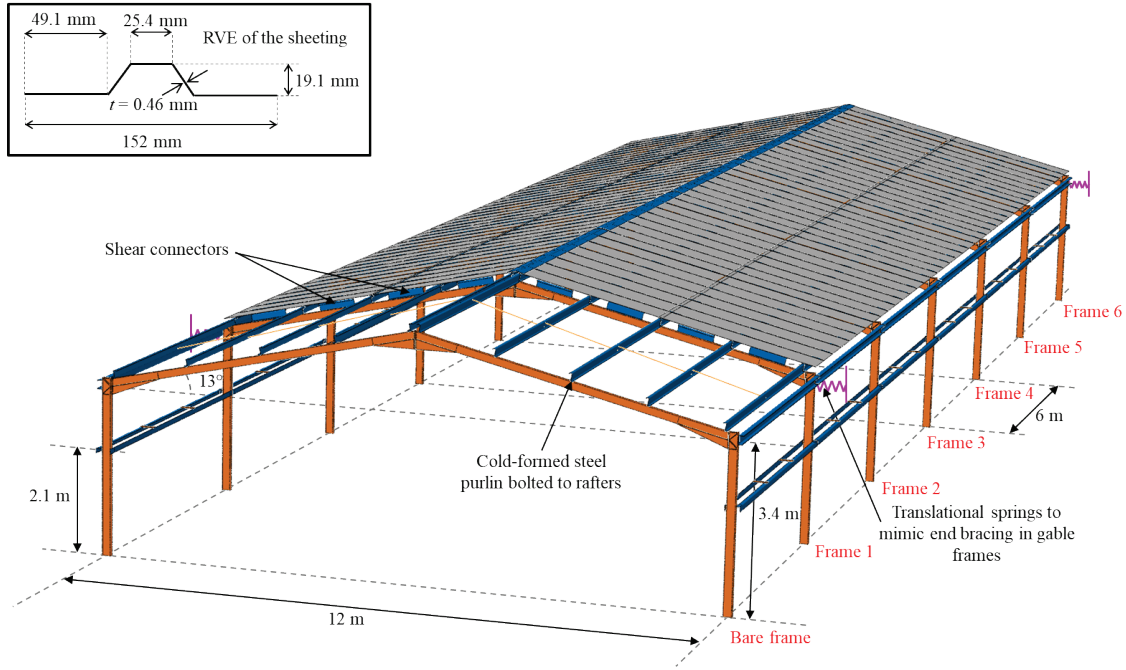


Figure 4.13: Details of the testing arrangement for the frames presented in Davies and Bryan (1982) and Roberts, Davies & Wang (2021)

The frame and cross-section geometries, connection details, material properties, boundary conditions and loading were taken as those provided in Davies and Bryan (1982) and Roberts, Davies & Wang (2021). In the absence of test data, over-strength factors of 1.25 and 1.2 (CEN, 2022) were applied to the nominal yield strength ($f_y = 235 \text{ N/mm}^2$) and ultimate strength ($f_u = 360 \text{ N/mm}^2$) values of the hot-rolled steel, respectively, and the boundary conditions of the column bases were considered as nominally pinned (as discussed in Section 4.2.1). The roof bracing in the end roof bays was modelled as CHS members bolted to the rafters with the same cross-sectional area as recorded in Roberts, Davies & Wang (2021). The behaviour of the bracing members in the gable frames was mimicked through the use of translational springs. A spring was applied at the top of the columns at each end frame with a stiffness $k = 20.7 \text{ kN/mm}$, as determined from the combined axial stiffness of the individual bracing members. The roof was clad with 0.46 mm Everclad L5 cold-formed steel sheeting, modelled using the approach outlined in Section 3.4.1. The translucent sheeting panels were modelled using the same approach as the steel sheeting panels, but with a reduced stiffness. The material used for the translucent panels was not reported, so 10% of the steel sheeting stiffness was used, following the assumption made by Roberts, Davies & Wang (2021). The frame was designed as a stressed skin structure and thus additional shear connectors were utilised to directly connect the sheeting to the rafters. The connections were modelled using MPC ties for the welds, and fastener

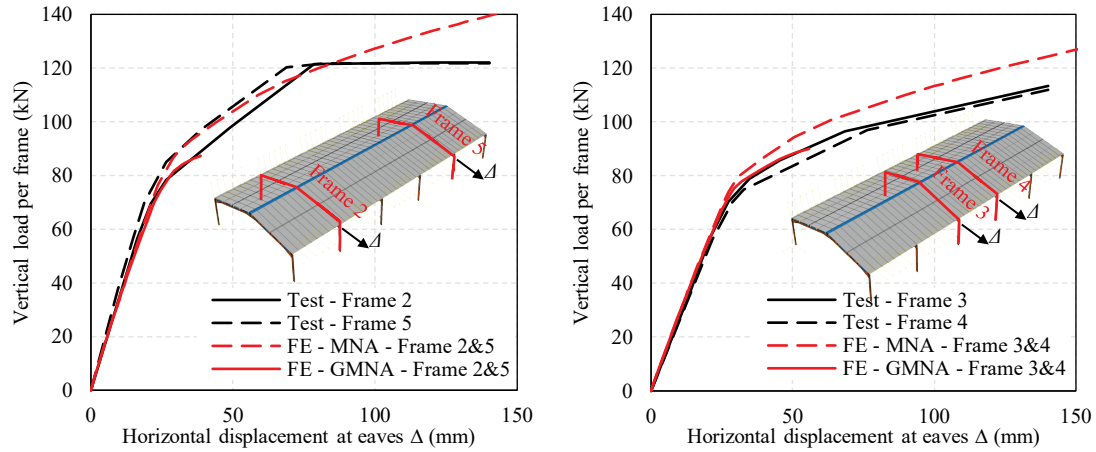
elements for the bolts and screws, as outlined in Section 3.3, with the screw and bolt properties as detailed in Roberts, Davies & Wang (2021). The screws connecting the single-skin sheeting to the purlin were modelled using the modelling approach for cladding secondary connectors (*FASTENER, CONN3D2, RADIAL-THRUST (ABAQUS, 2021)). The radial-thrust connectors were given a bilinear response with the stiffness and ultimate capacity defined using the tested values for 6.1 mm screws presented in Bryan (1972). In all cases, the horizontal deflections were measured at the bottom of the eaves and the vertical load per frame was calculated as the sum of the column base reaction forces.

Table 4.1: Comparison of horizontal displacements at the bottom of the eaves (results taken from frame 3 for the clad frame cases)

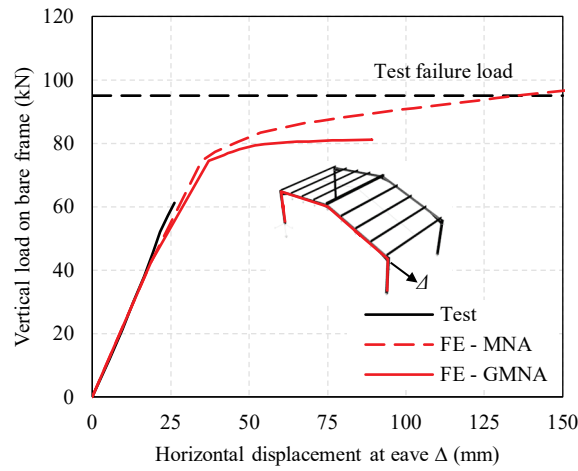
Frame	Load case	Horizontal displacement Δ (mm)	
		Test	FE
Clad frame	(1) Design vertical load of 0.48 kN/mm ² uniformly applied on sheeting	16.3	14.17
		14.3	
	(2) Horizontal load of 5 kN (only frame 3 loaded)	0.9	0.68
		1.1	
	(3) Horizontal loads of 5 kN (all internal frames (2-5) loaded)	3.3	1.67
		3.1	
Bare frame	(1) Design vertical load of 42.5 kN applied as point loads	20.1	18.66
		18.2	
	(2) Horizontal load of 5 kN	34.0	21.53
		31.7	

Table 4.1 compares the horizontal displacements observed during the tests at the design load levels with those extracted from the FE models. It should be noted that the level of loading considered remains within the elastic range of the frame behaviour. The two test values for each load case correspond to the tests with and without lateral support of the columns by side rails positioned at 2.1 m from the column base, respectively; the effect of the side rails was found to have a negligible difference on the horizontal deflections of the frames in the FE analysis, as also concluded by Davies and Bryan (1982). The results in Table 4.1 show that the FE models are able to accurately predict the eaves deflections of both the clad frame and the bare frame under vertical load. While the FE results of the horizontal load cases deviate slightly from the tests, the ratio of the stiffnesses between load case 2 and 3 of the clad frame is captured. Additionally, these errors are deemed acceptable for experimental data on a large structure, and therefore, the FE results are considered to agree well with the tests in the elastic range.

In addition to the comparisons in the elastic range, the FE model has also been validated against the tested frames loaded to collapse under vertical load. The corresponding load-displacement curves are plotted in Figure 4.14 for both the clad and bare frames. Excellent agreement is achieved using a materially nonlinear, but geometrically linear analysis (MNA) with the predicted behaviour closely following that observed in the test. Firstly, plastic hinges form at the eaves of frames 3 and 4, followed by hinges of the eaves of frames 2 and 5, and secondly, hinges form at the apex of frames 3 and 4 in the rafters, followed by equivalent hinges in frames 2 and 5. In Roberts, Davies and Wang (2021), it was noted that second order effects have negligible effects and the buckling of the members in the studied frame and were thus ignored. Comparing the MNA results with those from Roberts, Davies and Wang (2021), close agreement is observed. However, significant effects are observed herein when using a nonlinear geometry analysis (GMNA), with the model failing due to premature buckling. This suggests that there may have been more lateral and torsional restraint in the structure than identified in Davies and Bryan (1982) and Roberts, Davies & Wang (2021). Nonetheless, under serviceability load levels, which are the focus of the present study, there is consistently good agreement between the experimental and numerical results.



(a) Clad frame – Load case (1)



(b) Bare frame – Load case (1)

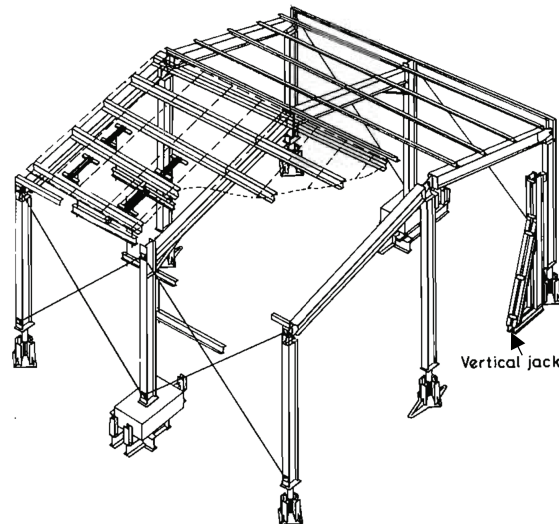
Figure 4.14: Comparison of load-displacement curves for load case 1 of clad and bare frames, respectively, obtained from the FE model and the tests reported in Davies and Bryan (1982) and Roberts, Davies & Wang (2021)

4.3.2 Two bay portal frame

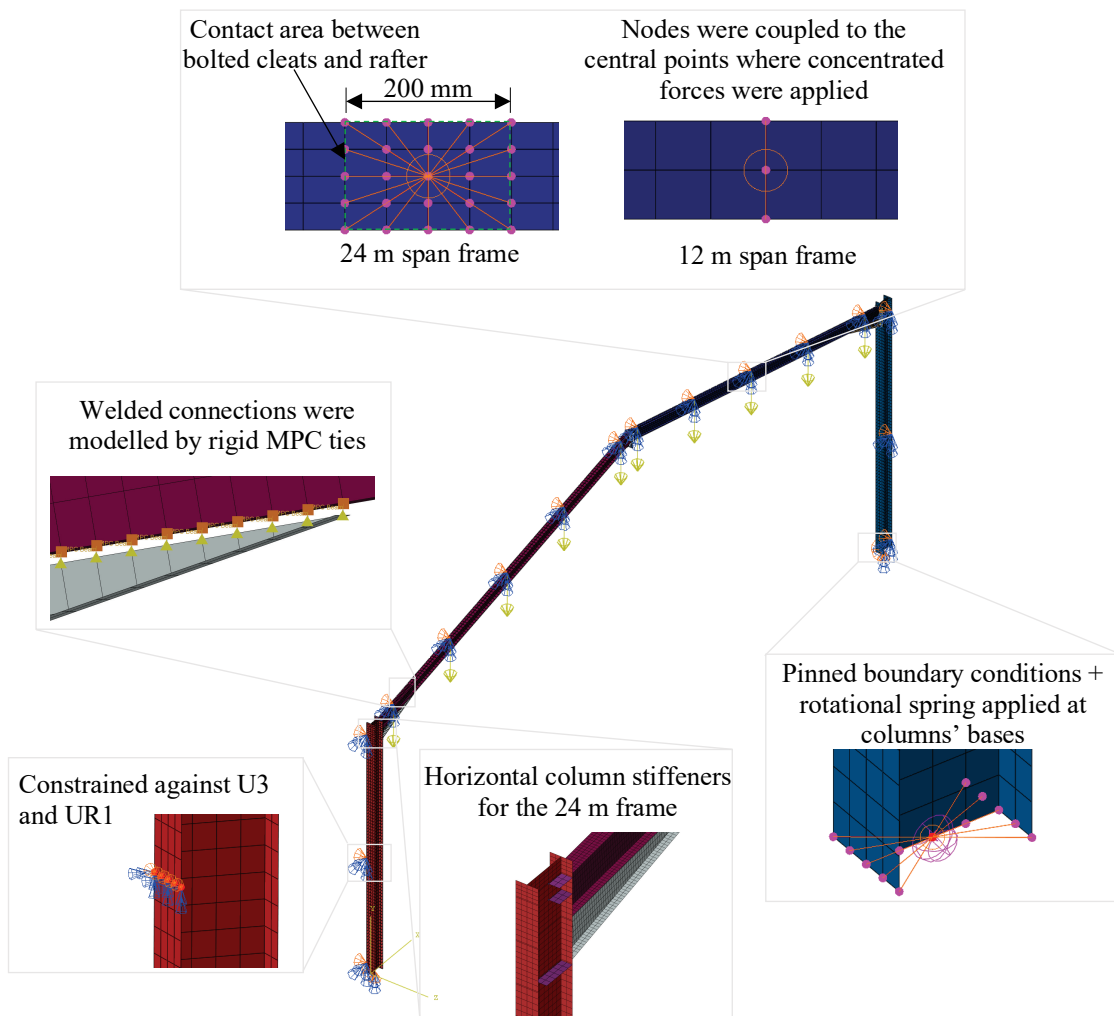
In this section, the FE model is validated against a further two tests: a 12 m span frame and a 24 m span frame, as reported in Davies et al. (1990) and Junid (1992), respectively. The test setup for both was as shown in Figure 4.15a. The tests consisted of a two bay frame and a roof cladding system under uniform vertical loading. The gable frames, purlins, side rails and rafter stays ensured sufficient lateral stability; however, the test programme was designed to eliminate the effects of stressed skin actions and thus the gable frames and the connections were designed to ensure the motion of the intermediate frame was followed. The frames underwent two cycles of loading and unloading, and then the load was increased until the frames failed. The applied load was recorded as the sum of the reaction forces at the base of

the columns and the vertical deflection was recorded at the apex. Note that the deflection caused by the self-weight was not recorded because only the effect of the loading system was considered (Davies et al., 1990; Junid, 1992).

The FE models were developed based on the cross-section geometries, material properties, boundary conditions, connection details and loading arrangement provided in Davies et al. (1990) and Junid (1992). Only the intermediate frame was simulated, with adequate constraints and boundary conditions applied to mimic those achieved in the test from the gable frames and roof cladding, as shown in Figure 4.15b. The boundary conditions at the column bases were simulated using the nominally pinned condition as described in Section 4.2.1. Coupon tensile tests and measurements of the geometry were performed at different locations in the 24 m span frame (Junid, 1992), while for the 12 m span frame, no material properties were reported in (Davies et al., 1990). Since the two tests were part of the same wider experimental project, the average yield strength obtained from the tensile coupon tests of the 24 m span frame specimen was also assumed for the 12 m span frame model. The stress-strain behaviour was modelled using the approach described in Section 3.2.1. Connections were modelled as described in Section 3.3. As mentioned above, the tests were designed such that stressed skin action is eliminated and, thus, the cladding system was not modelled in the FE simulations. The loads were applied directly to the rafters in the form of concentrated forces. In the 12 m span frame, the load was applied to the welded position of the purlin cleats. In the 24 m span frame, purlin cleats were bolted to the rafters and, thus, the load was applied uniformly to the contact area between the purlin cleats and the rafters. Both loading strategies were achieved by applying concentrated forces to the central point of a region connected by coupling constraints, as shown in Figure 4.15b. The frames were constrained against movement along the global Z-direction (U3) and rotation about the global X-direction (UR1) at the locations of the purlin cleats and side rails. The thickness of the internal flange of the column was increased to incorporate the additional plate applied in the test at the height of the eaves bolts.



(a) Test setup



(b) FE representation of the tests

Figure 4.15: Details of the testing arrangement for the frames reported in Davies et al. (1990) and Junid (1992)

Imperfections were implemented to capture the ultimate failure mode. Initial geometric imperfections were incorporated through scaling the first eigenmode from a linear buckling analysis. In the absence of imperfection measurement data, the magnitude of the imperfection was taken as 1/1000 of the major buckling shape length equal to the length of 2400 mm in the 12 m span frame model and 2630 mm in the 24 m span frame model. As a result, imperfection magnitudes of 2.40 mm and 2.63 mm were included in the 12 m span frame and the 24 m span frame, respectively. The Riks method was used as the analysis method in Abaqus/Standard to analyse the buckling behaviour observed in the tests. During the analysis of the 24 m span frame, it was observed that the Riks method could not obtain the full load-displacement characteristics due to an unstable static solution. Therefore, the alternative restart analysis method was used by terminating the static procedure and switching to the dynamic implicit procedure.

Figures 4.16 and 4.17 compare the behaviour of the applied load against the apex vertical displacements obtained from the FE models with those recorded in the tests for the 12 m span frame and the 24 m span frame, respectively. It can be observed that the inclusion of the geometric imperfection results in a decrease in the load bearing capacity of the whole frame following the occurrence of buckling. This sensitivity to the imperfection is more pronounced in the 24 m span frame. In the comparison of the 24 m span frame, due to a convergence problem encountered in the Riks method, a response was also obtained by restarting the analysis with the dynamic implicit method; the FE model was run until the static solution became unstable (i.e. the red curve), and then the analysis method was switched to dynamic implicit (i.e. the orange curve). Based on the results, it can be concluded that the FE model can accurately and reliably predict the structural behaviour of the tested frames under vertical loads.

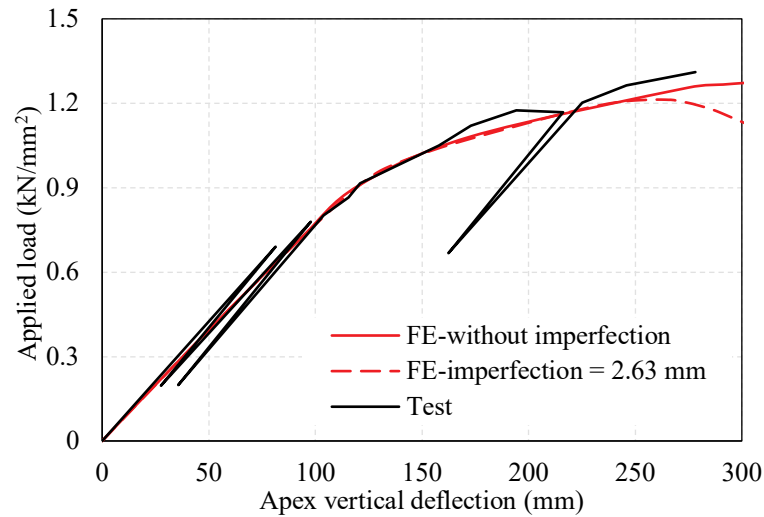


Figure 4.16: Comparison of load-displacement curve for the 12 m span frame obtained from FE model and the test reported in Davies et al. (1990)

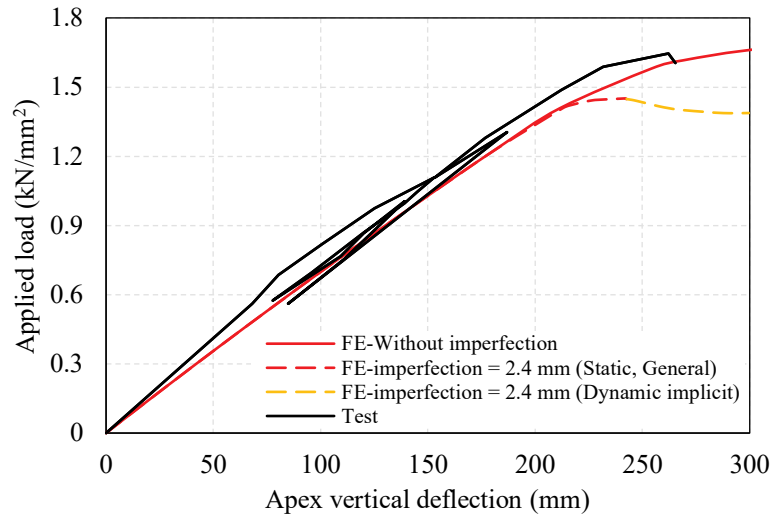
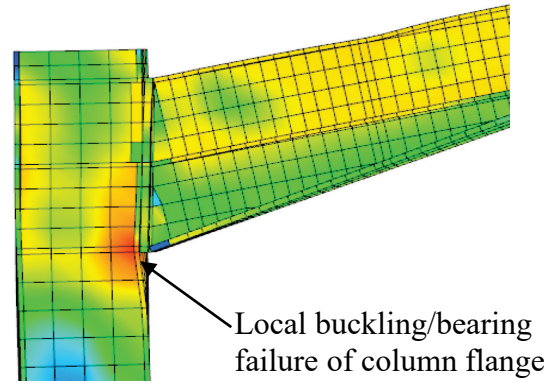
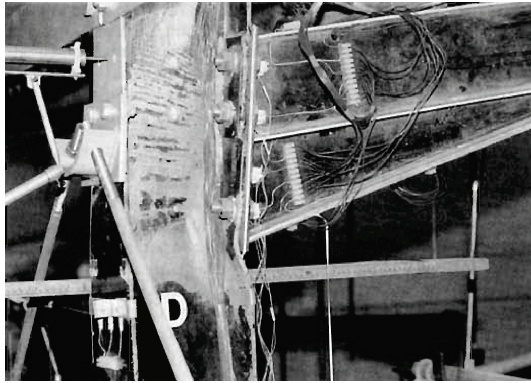


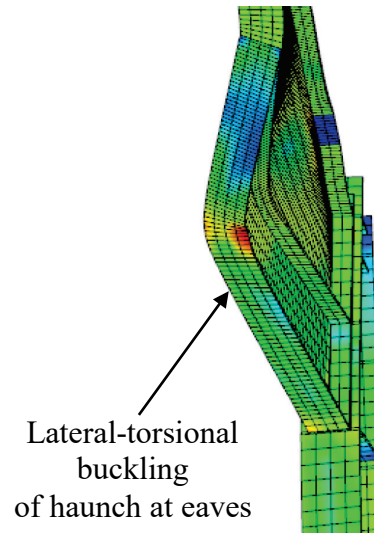
Figure 4.17: Comparison of load-displacement curve for the 24 m span frame obtained from FE model and the test reported in Junid (1992)

Figures. 4.18 and 4.19 illustrate a comparison of the failure modes observed in the tests (Davies et al., 1990; Junid, 1992) and the FE models for the 12 m span frame and the 24 m span frame, respectively. The 12 m span frame buckled locally in the column flange adjacent to the bottom flange of the eaves haunch, while the 24 m span frame failed due to lateral-torsional buckling of the eaves haunch. Both failure modes are in good agreement and accurately predicted by the FE models.



(a) Failure mode reported in Davies et al. (1990) (b) Failure mode observed in FE model

Figure 4.18: Failure mode of the 12 m span frame



(a) Failure mode reported in Junid (1992) (b) Failure mode observed in FE model

Figure 4.19: Failure mode of the 24 m span frame

4.4 Concluding remarks

In this chapter, the validated key components of portal frames, e.g. the primary steelwork, connections and cladding are assembled and utilised to develop an FE model of a full portal frame. The components of the full frame FE model were described including element type, mesh size, material choice and boundary conditions. The connections and cladding system employed in the full frame FE model were also detailed. The assembled full frame finite element model was then validated against three experimental frame tests for both clad and bare frame configurations. Consequently, combined with the component validation, the full frame FE model is considered able to accurately capture true structural behaviour and will be used in Chapter 5 for the parametric assessment of the behaviour of single-span portal frames.

Chapter 5

Full portal frame parametric study

5.1 Introduction

In this chapter, the validated full frame FE model is utilised for a numerical parametric study to investigate the effect of scale and the associated stressed skin action on the overall behaviour of portal frames. Additionally, the local behaviour of the connecting elements including the cladding primary and secondary screws is evaluated.

5.2 General

To investigate the effect of scale, various portal frame dimensions, including spans ranging from 20 to 30 m, eaves heights ranging from 8 to 20 m and structure lengths ranging from 30 to 78 m, were analysed under various loading conditions, including gravity load, wind loads and thermal actions. The frame dimensions were chosen based on standard current practice in the United Kingdom (Davies, Roberts & Wang, 2021). The main focus was on understanding how the stressed skin action, as influenced by these different parameters, affects the overall behaviour of the frame including the distribution of loads between frames and the deflection of the frames themselves. Additionally, the more local effect of stressed skin action on the connectors was examined. Critical connector locations were identified with the magnitude of the critical connector forces determined.

5.3 Frame cases modelled

5.3.1 General

As illustrated in Figure 5.1, three spans: 20 m, 25 m and 30 m, four eaves heights: 8 m, 12 m, 16 m and 20 m, and three structure lengths: 30 m, 54 m and 78 m were investigated in this study. The frame spacing considered in all cases was 6 m. The combination of these dimensions

resulted in 36 portal frames. Additionally, a 40 m span was added to the study to represent an extreme case in single-span portal frame sizes.

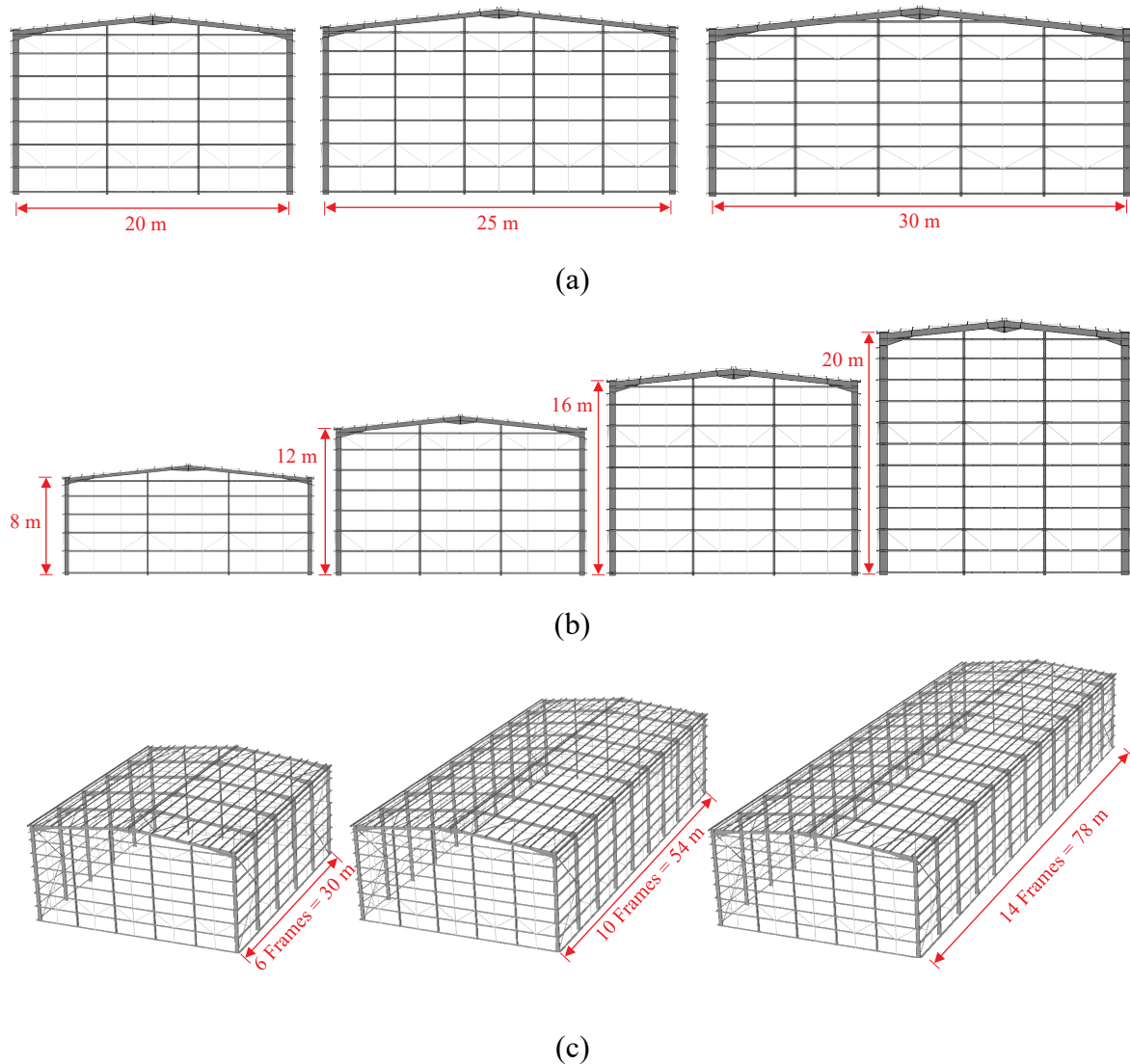


Figure 5.1: Illustration of the variation of (a) span, (b) eaves height and (c) structure length considered in the study

Following typical practice, the frame components were designed independently without consideration for the interaction between the three components. The primary frame members and joints at the eaves and the apex were sized based on the gravity load case. The member sizing and connection design was based on a 0.6 kN/m^2 imposed load, a 0.1 kN/m^2 service load and the use of S355 steel grade. The eaves and apex joints consisted of a 20 mm, S355 steel grade end plate with M24 bolts at 80 mm cross centres. A summary of the member sizes and connection details is shown in Table 5.1. The stiffening effect of the cladding was not

incorporated into the design and therefore there were no shear connectors between the cladding and the primary frame rafters.

The sizes of the purlins and side rails were determined from the METSEC load tables (METSEC, 2017) based on the unfactored vertical and wind load, respectively. In the sleeved connections, the sleeve length was defined in the manufacturer literature (METSEC, 2017) by the size of the sleeved purlin/side rail section detailed in Table 5.1. The sleeve length at the penultimate frames was longer than that at the other internal frames. For the “172Z” purlin and side rail sections, the sleeve length was 914 mm at the penultimate frames and 714 mm for the other internal frames and for the “202Z” side rail section, the sleeve length was 1074 mm at the penultimate frames and 834 mm for the other internal frames.

Table 5.1: Summary of member sizes and connection details used in the parametric study

Span (m)	Eaves height (m)	Slope (°)	Rafter UB section	Column UB section	Gable post UB section	No. of bolt rows in tension in eaves and apex joints	Purlin Z-section	Side rail Z-section
20	8	6	254×146×31	356×171×51	305×102×25	2 rows	172Z15	172Z15
	12	6	356×171×45	457×191×67	406×140×39	3 rows		172Z15
	16	6	457×152×52	533×210×92	356×171×51	4 rows		202Z14
	20	6	533×210×82	686×254×125	457×191×67	4 rows		202Z14
25	8	6	406×140×39	457×191×67	205×102×28	3 rows	172Z15	172Z15
	12	6	356×171×45	457×191×74	406×140×39	3 rows		172Z15
	16	6	457×191×67	533×210×92	356×171×51	4 rows		202Z14
	20	6	533×210×82	686×254×125	457×191×74	4 rows		202Z14
30	8	6	356×171×51	533×210×82	254×146×31	3 rows	172Z16	172Z15
	12	6	457×152×52	533×210×82	356×171×45	4 rows		172Z15
	16	6	457×191×67	610×229×101	457×191×67	4 rows		202Z14
	20	6	533×210×82	686×254×125	533×210×82	4 rows		202Z14
40	8	10	457×191×74	610×229×113	254×146×31	4 rows	172Z18	172Z15
	12	10	457×191×74	686×254×125	406×140×39	4 rows		172Z15
	16	10	533×210×82	686×254×125	406×178×54	5 rows		202Z14
	20	10	533×210×92	762×267×134	457×191×74	6 rows		202Z14

All frames were examined under three loading conditions: gravity load, wind sway load and thermal actions. Each frame case was labelled with a unique identifier according to the loading conditions and frame geometry. The labelling system begins with the loading condition, with Vr, WS and Thr, representing gravity load, wind sway load and thermal action, respectively,

and is followed by the span, then the eaves height, and ends with the number of frames. For example, WS_S25_H12_F06 corresponds to a frame case under wind sway loading with a 25 m span, a 12 m eaves height and 6 frames giving a total length of 30 m.

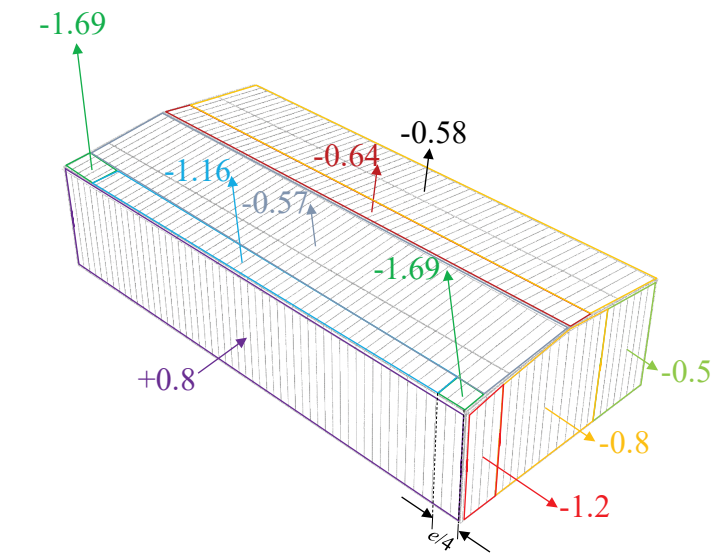
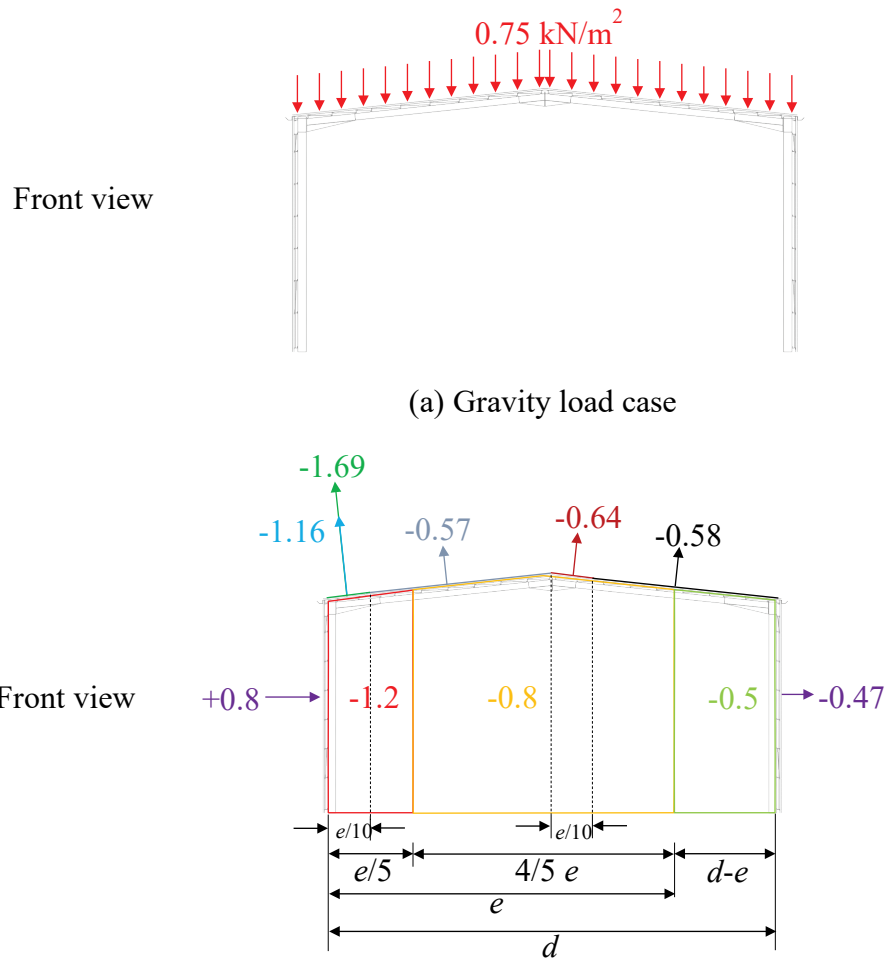
5.3.2 Loading combinations

The load combinations considered in this study were taken as the serviceability limit state (SLS) and at 1.5 SLS as defined in BS EN 1990 (CEN, 2002). The load combinations considered herein are summarised in Table 5.2, where G_k is the portal frame self-weight, Q_k is the imposed and service load combined, F_{wk} is the wind force and T_k is the thermal action.

Table 5.2: Load combinations considered in the parametric study

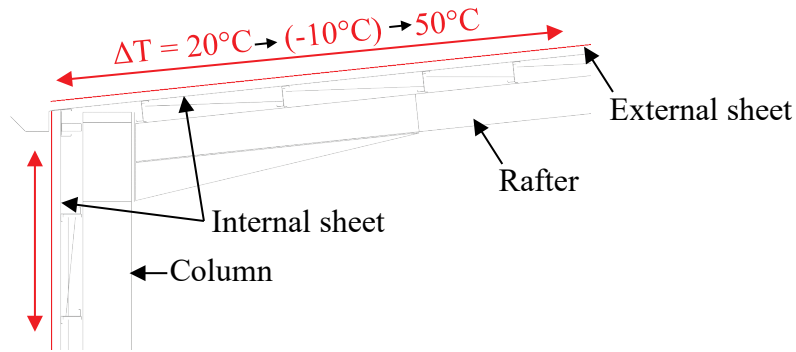
Persistent load type	SLS load combination
Gravity imposed loads	$G_k + Q_k$
Wind sway loads	$G_k + F_{wk}$ (pressure)
Thermal actions	$G_k + T_k$

The imposed load Q_k was taken as 0.75 kN/m^2 (CEN, 2005b) and the wind load F_{wk} was taken as $1.0 \text{ kN/m}^2 + 0.01 \times h_e$, where h_e is the eaves height of the frame. The external pressure coefficients considered for the wind load case are illustrated in Figure 5.2, where $e = \min(b, 2h)$, where b is the crosswind dimension and h is the total height of the portal frame (CEN, 2005a). The internal pressure coefficient was taken as -0.32 for the wind load case for buildings without a dominant face (CEN, 2005a), as was the assumed case in the present study. The thermal action was applied to the external sheet of the cladding only, while the internal sheet was kept at room temperature. The installation temperature of the cladding was assumed to be 20°C , with the minimum temperature taken as -10°C and the maximum temperature taken as $+50^\circ\text{C}$. The self-weight can be applied using *Dload, grav (ABAQUS, 2021); however, due to numerical convergence issues, the equivalent self-weight of the sheeting, the purlins and the rafters was applied on the roof as a pressure.



(b) Wind sway load case – external pressure coefficients

Figure 5.2: Load combinations



(d) Thermal actions

Figure 5.2 (continued): Load combinations

5.3.3 Key aspects under investigation

The key aspects under investigation herein are the load distribution between frames and the local behaviour of the cladding primary and secondary screw connectors, as summarised in Table 5.3. The load distribution between the frames is obtained by considering the reaction forces (RF (ABAQUS, 2021)) at the column bases of each frame. The applied load per frame was compared to sum of the reaction force at the column bases. The calculation of the applied load was based on the conventional design assumption that all internal frames evenly share the load, while the gable frame bears half of this load. The local behaviour of the cladding screw connectors is defined by extracting the relative displacements (CU_Local (ABAQUS, 2021)) and forces (CTF_Local (ABAQUS, 2021)) of the radial-thrust (ABAQUS, 2021) connectors used to represent these screws.

Table 5.3: Summary of key aspects under investigation

Outputs	Location	Variables (ABAQUS, 2021)	Objectives
Reaction force	Column bases	RF	To investigate the loading path and distribution
Relative displacement and force	Flashing seam screws	CTF & CU	To identify critical screw location and magnitude
	Longitudinal and end lap seam screws		
	Internal sheet connectors		
	External sheet connectors		

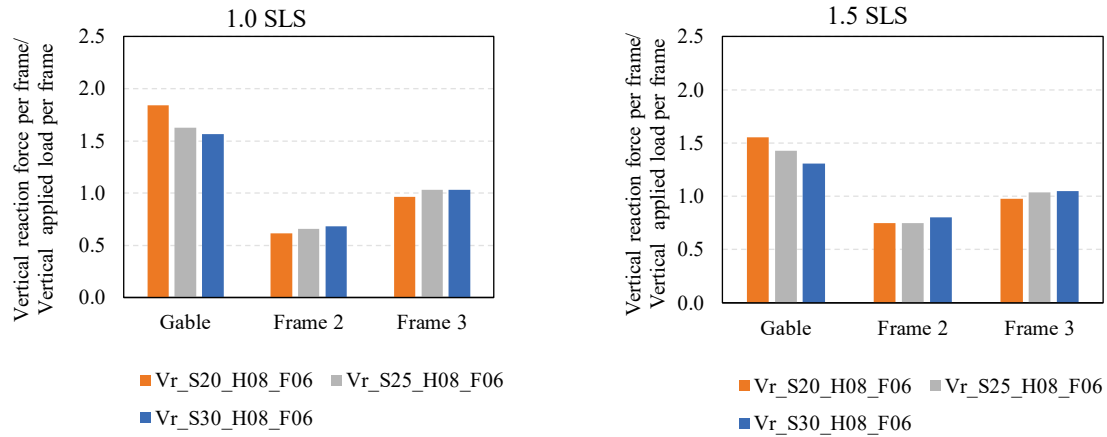
5.4 Influence of scale

5.4.1 Effect of span

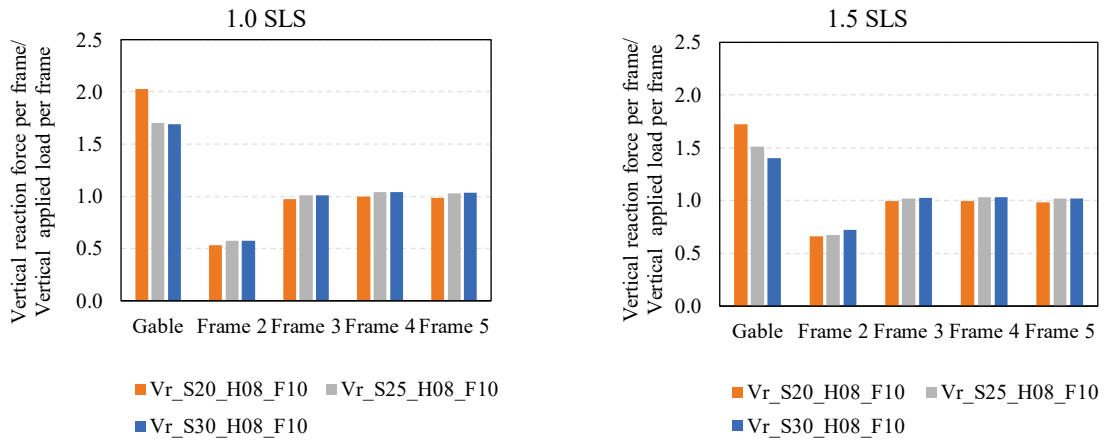
Gravity load

The influence of the portal frame span under gravity load on the overall behaviour of the frame and the behaviour of the cladding screws is discussed in this section. The effect of span on the load distribution between the frames is shown in Figure 5.3. In all cases, it was observed that, apart from the gable and penultimate frames, all frames resisted the expected applied load. Frame 2 (i.e. the penultimate frame) carried 50-80% of the expected applied load while the gable frame carried between 130-210%. The transfer of load between the gable and the penultimate frame was due to the gable frame high relative stiffness which created a shear field in the end bay cladding system causing more load to be attracted to the gable frame, as shown in Figure 5.4. This behaviour was also seen by Roberts, Davies & Wang (2021) in portal frames clad with single-skin sheeting under gravity loading conditions.

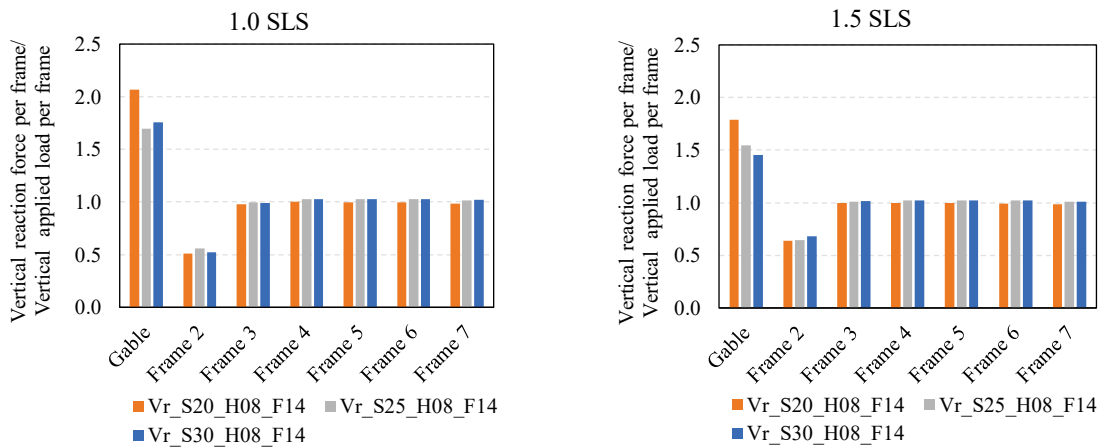
The ratio of the reaction force carried by the gable frame in comparison to the expected applied load decreased from 150-210% at the serviceability limit state (SLS) to 130-180% at 1.5 SLS. Meanwhile, the ratio carried by the penultimate frame increased from 50-80% at the serviceability limit state (SLS) to 60-80% at 1.5 SLS. Generally, the ratio of the load transferred to the gable frame in the 20 m span frames was 10-30% higher than that in the frames with greater spans.



(a) 6 frame models



(b) 10 frame models



(c) 14 frame models

Figure 5.3: Influence of portal frame span on gravity load distribution between frames

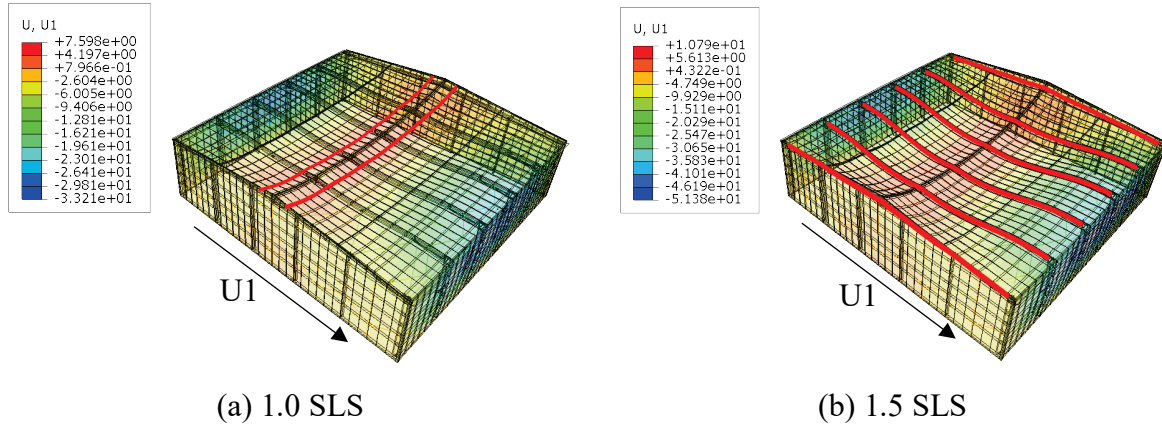
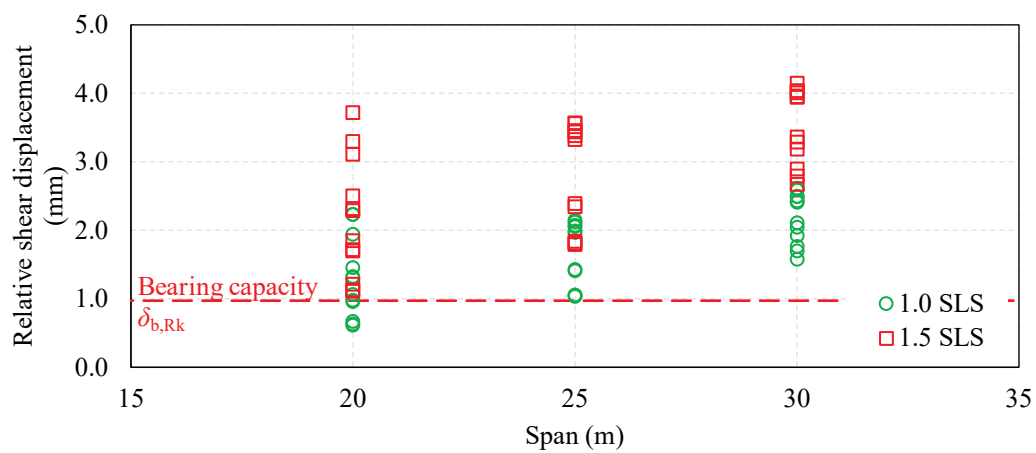
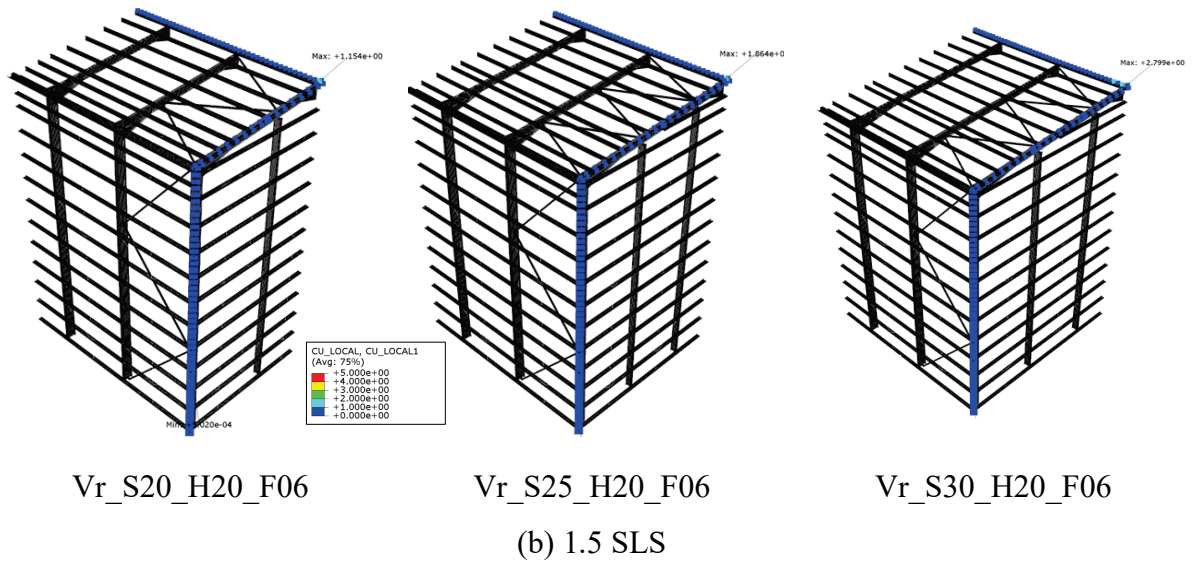
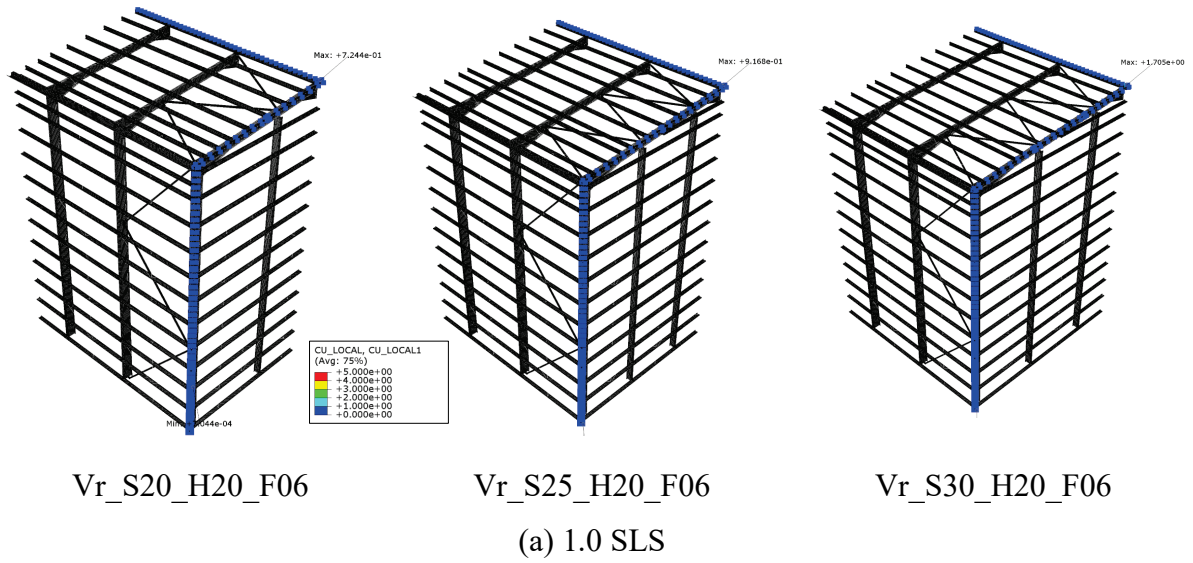


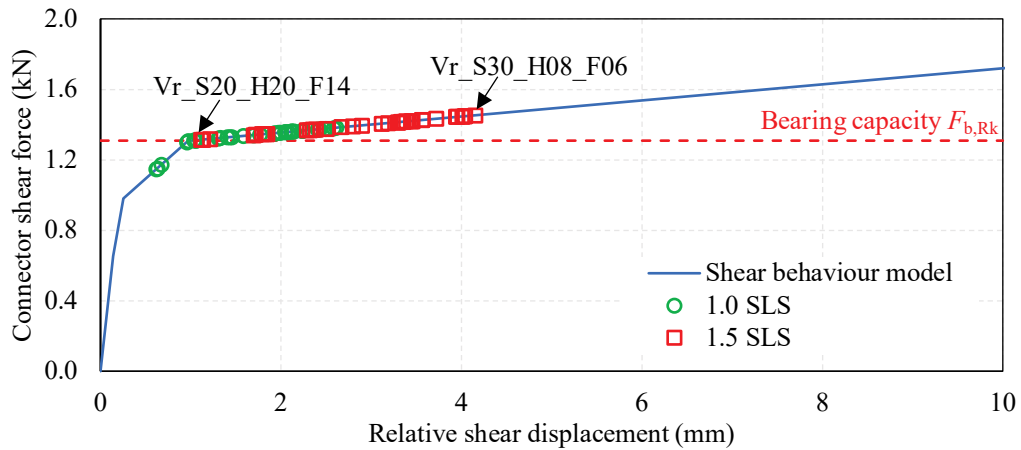
Figure 5.4: Deflected shape of a portal frame under gravity load

The effect of span on the flashing seam screws is shown in Figure 5.5. In Figures 5.5a and b (and all similar figures herein), the displacements are illustrated with increasing magnitude in mm shown from blue to red. In all cases the demand on the flashing seam screws at the end bay was higher than that at the internal bays. Additionally, the flashing seam screw at the apex hat near the gable frame experienced the highest demand. The behaviour of the flashing seam screws was dominated by shear and no significant axial displacement was observed, as can be seen in Figure 5.6. The shear and axial behaviour models shown in Figure 5.6 correspond to the cladding secondary connections detailed in Section 3.3.6. In almost all cases, at least one flashing seam screw reached its bearing capacity at the serviceability limit state. The effect of the span on the shear displacement of the critical flashing seam screw is shown in Figure 5.5c. It can clearly be observed that the maximum shear displacements increase with increasing span. At the serviceability limit state, a 25% increase in span (i.e. 20 m to 25 m) resulted in up to a 70% increase in the shear displacement of the critical flashing seam screw, while a 50% increase in span (i.e. 20 m to 30 m) resulted in up to a 170% increase in the shear displacement of the critical flashing seam screw.

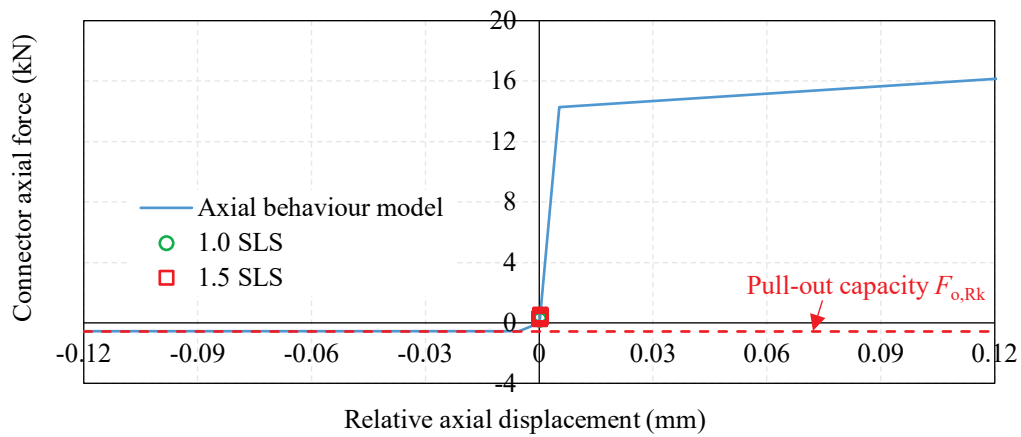


(c) Critical flashing seam screw shear displacement

Figure 5.5: Influence of portal frame span on the relative shear displacement of flashing seam screws under gravity load



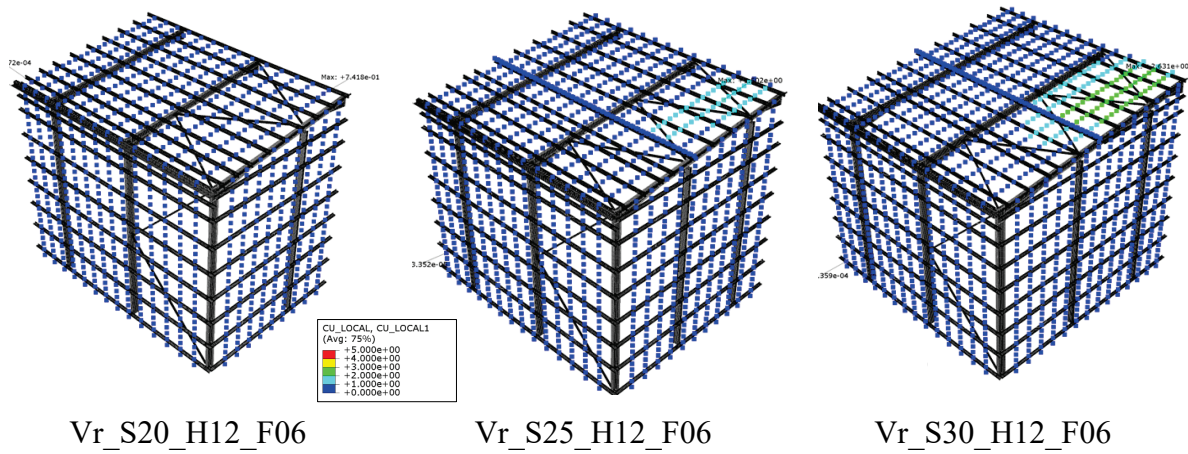
(a) Shear



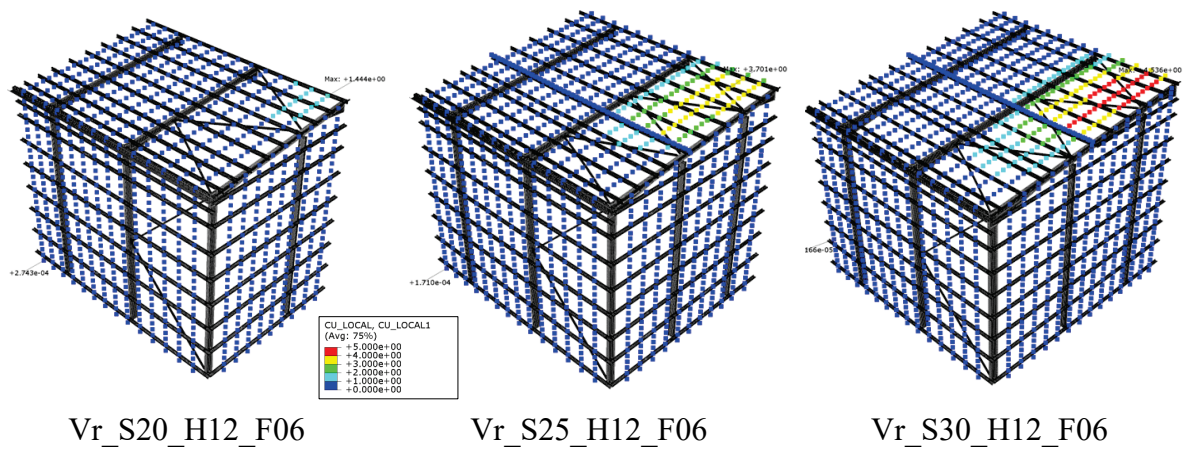
(b) Axial

Figure 5.6: Relative load-displacement behaviour of the critical flashing seam screw under gravity load

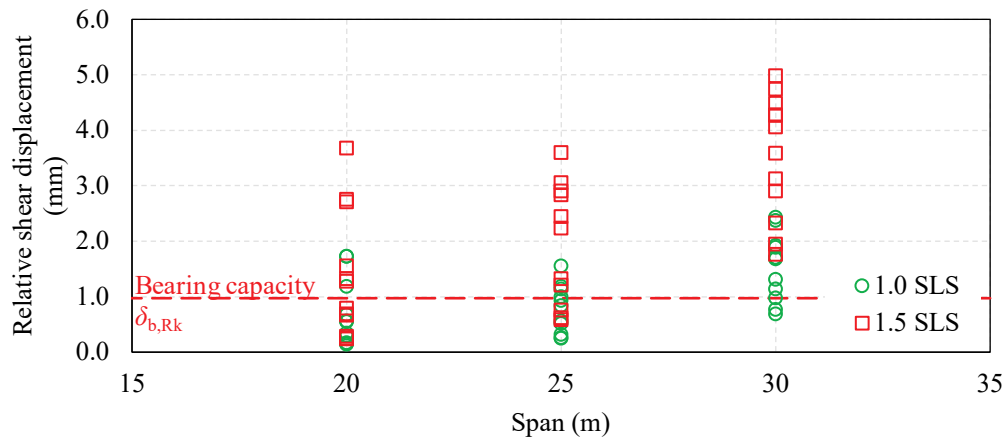
The effect of span on the longitudinal and end lap seam screws is shown in Figure 5.7. The same conclusions can be made as for the flashing screws. In all cases the demand on the longitudinal seam screws at the end bay was higher than that at the internal bays and the longitudinal seam screws between the panels near the apex hat experienced the highest demand. The load attracted to the gable frame resulted in an increased demand on the end bay screws. As shown in Figure 5.8, there were generally uniform displacements in the internal frames, resulting in minimal demand on the internal bay screws. However, relative displacements were observed between the gable and the penultimate frames, imposing a higher demand on the end bay screws.



(a) 1.0 SLS



(b) 1.5 SLS



(c) Critical longitudinal seam screw shear displacement

Figure 5.7: Influence of portal frame span on the relative shear displacement of longitudinal and end lap seam screws under gravity load

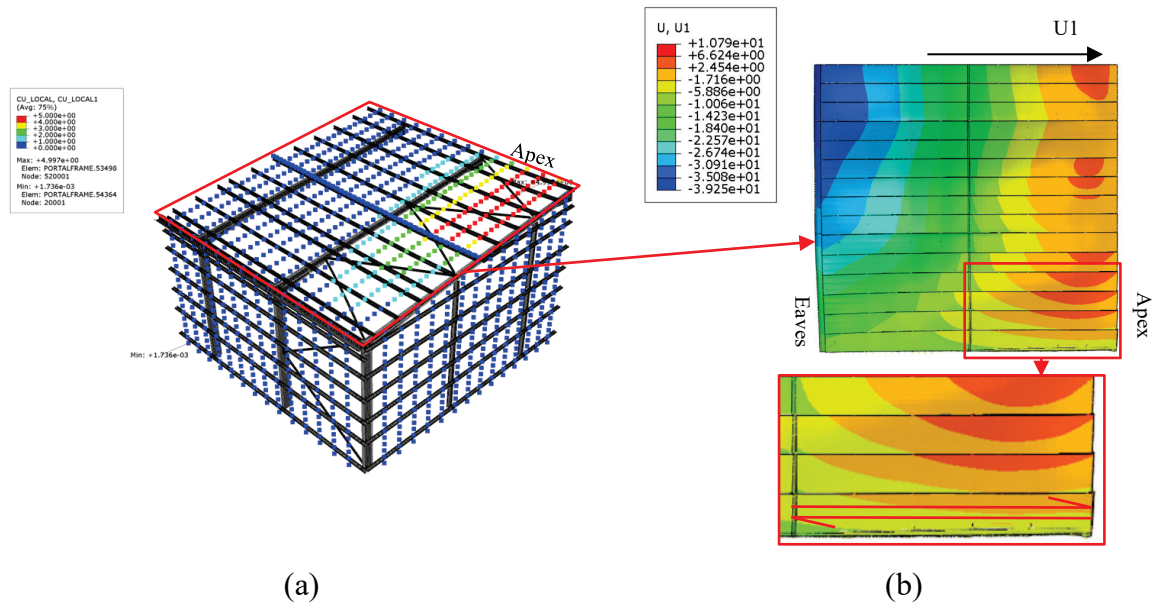
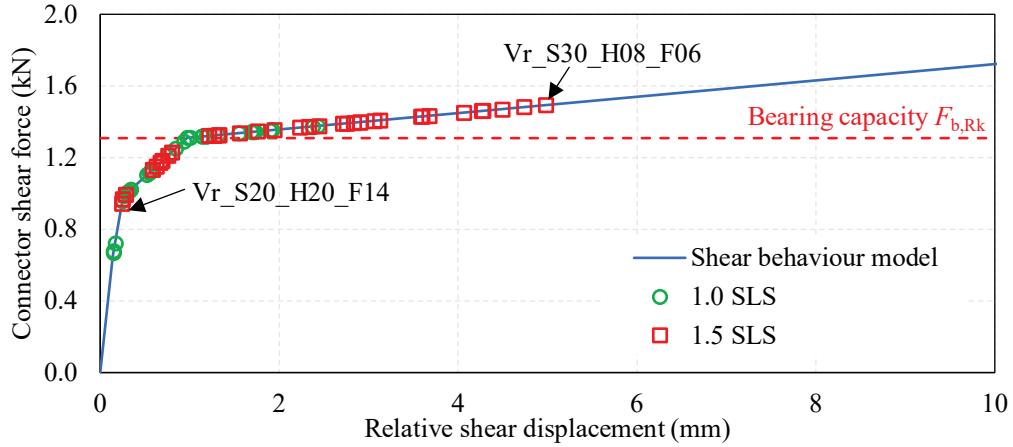


Figure 5.8: Illustration of (a) Relative shear displacement of longitudinal and end lap seam screws and (b) horizontal displacement of roof cladding panels under gravity load

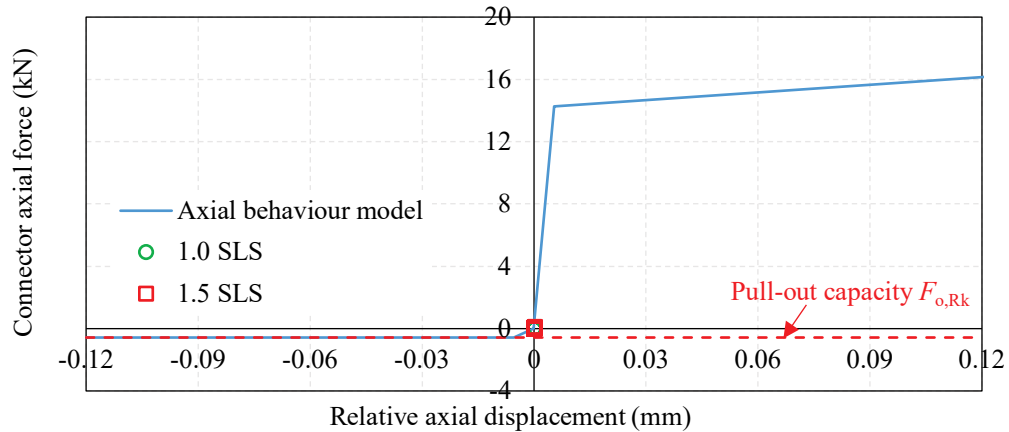
Again, the behaviour of the longitudinal seam screws was dominated by shear and no significant axial displacement was observed - see Figure 5.9. The behaviour models shown in Figure 5.9 correspond to the cladding secondary connections detailed in Section 3.3.6. In about half of the cases modelled, at least one longitudinal seam screw reached its bearing capacity at the serviceability limit state. The effect of the span on the shear displacement of the critical longitudinal seam screw is shown in Figure 5.7c. A 25% increase in span resulted in up to a 170% increase in the shear displacement of the critical longitudinal seam screw while a 50% increase in span resulted in up to a 700% increase in the shear displacement of the critical longitudinal seam screw, both at 1.5 SLS.

The effect of span on the internal sheet connectors is shown in Figure 5.10. These are the connectors that represent the interface between the cladding primary screws and the cladding internal sheet. In all cases, the demand on the internal sheet connectors along the purlin near the apex was the highest, propagating into the end bay in the larger frame spans. The behaviour of the internal sheet connectors was dominated by shear with displacements up to 6 mm, see Figure 5.9a. Unlike the previous connectors discussed, significant axial displacements were also observed, with failure in pull-out occurring in 43% of all cases at the serviceability limit state, see Figure 5.9b. The behaviour models shown in Figure 5.9 correspond to the cladding primary connections detailed in Section 3.3.5 and illustrated in Figure 3.6c. Additionally, in almost all cases modelled, at least one internal sheet connector reached its bearing capacity at

the serviceability limit state. The effect of the span on the shear displacement of the critical internal sheet connectors is shown in Figure 5.10c. At the serviceability limit state, a 25% increase in span resulted in up to a 140% increase in the shear displacement of the critical internal sheet connector at 1.5 SLS while a 50% increase in span resulted in up to a 500% increase in the shear displacement of the critical internal sheet connector.

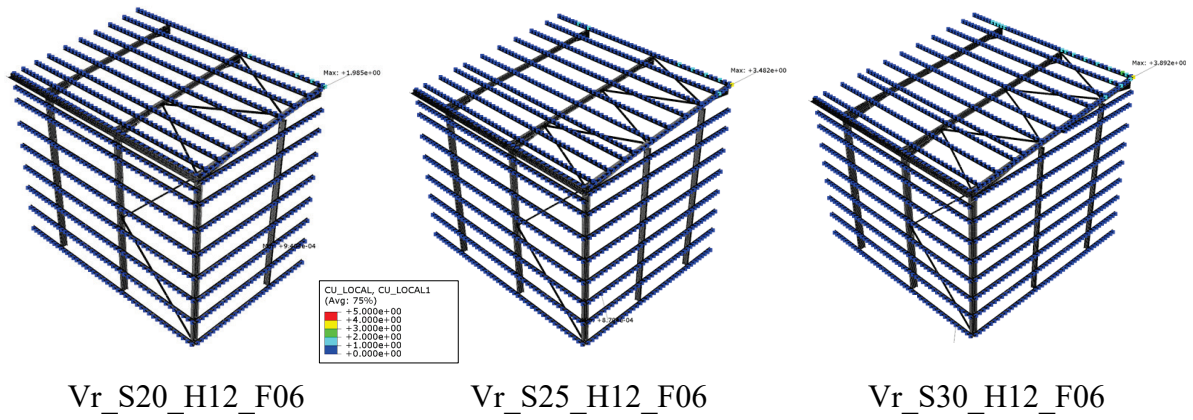


(a) Shear

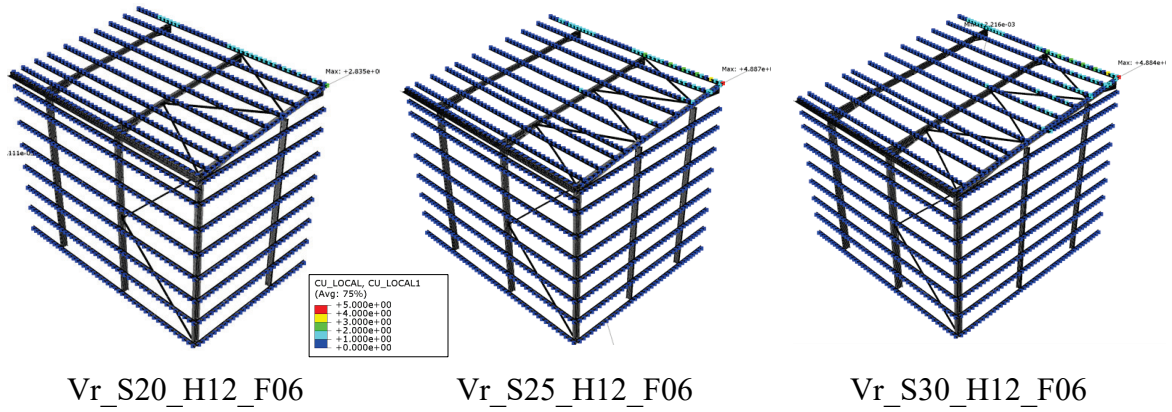


(b) Axial

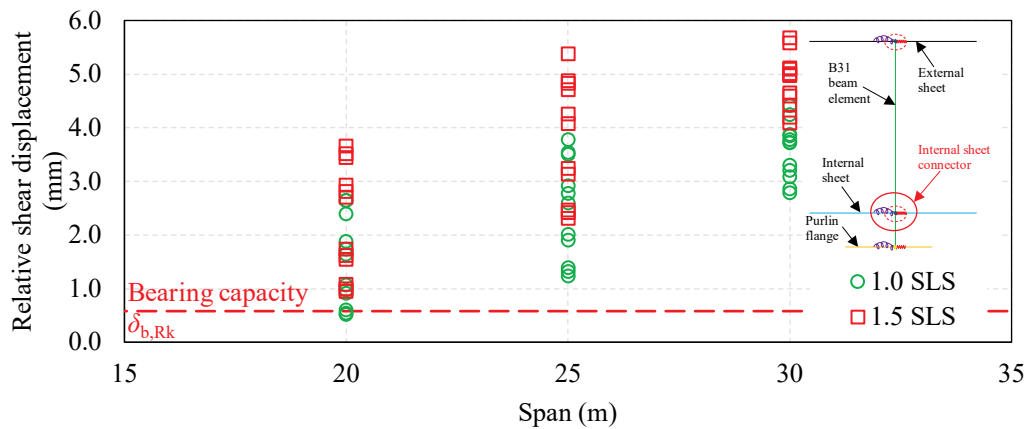
Figure 5.9: Relative load-displacement behaviour of the critical longitudinal/end lap seam screw under gravity load



(a) 1.0 SLS

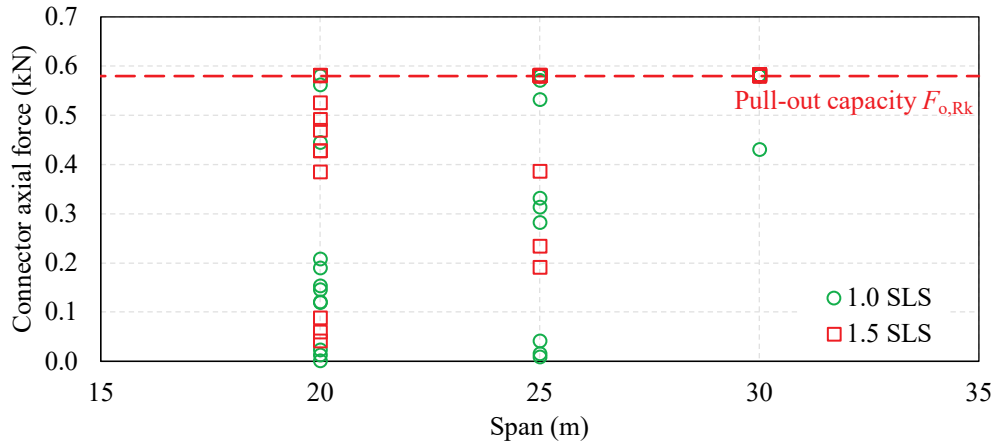


(b) 1.5 SLS



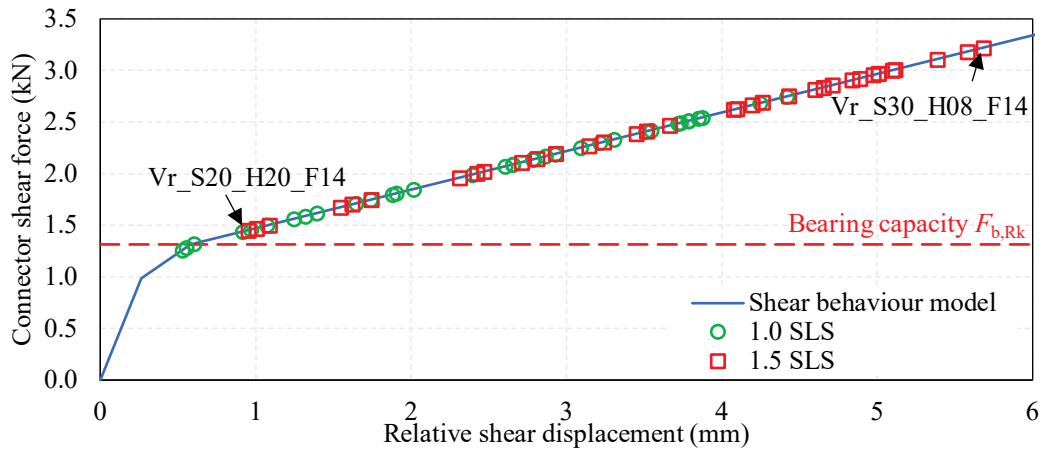
(c) Critical internal sheet connector shear displacement

Figure 5.10: Influence of portal frame span on the relative shear displacement of internal sheet connectors under gravity load

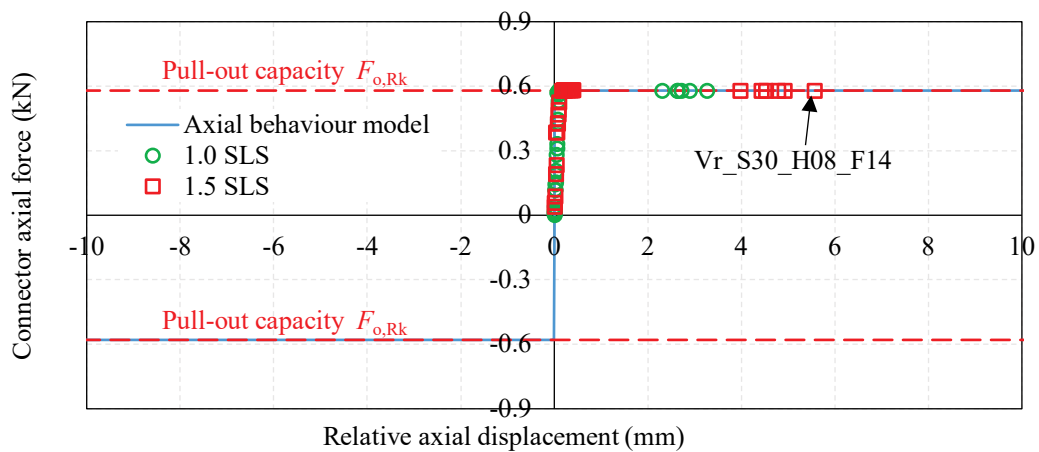


(d) Critical internal sheet connector axial force

Figure 5.10 (continued): Influence of portal frame span on the relative shear displacement of internal sheet connectors under gravity load



(a) Shear

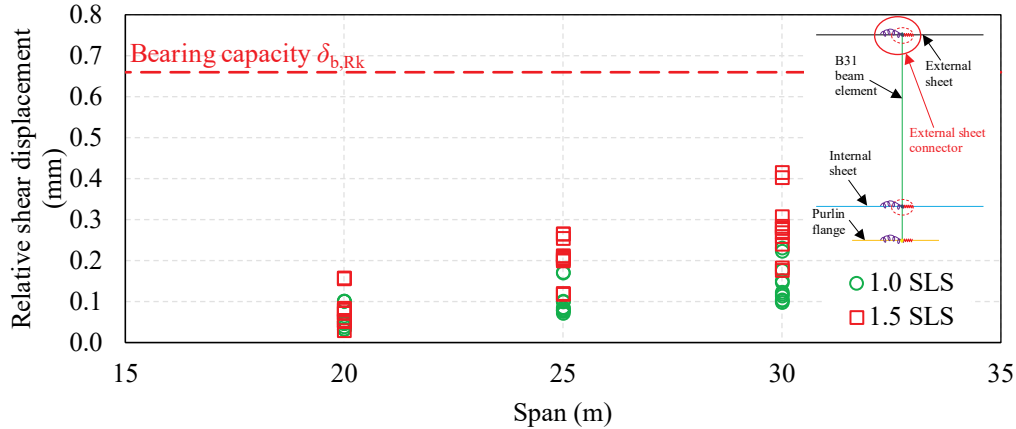


(b) Axial

Figure 5.11: Relative load-displacement behaviour of the critical internal sheet connector under gravity load

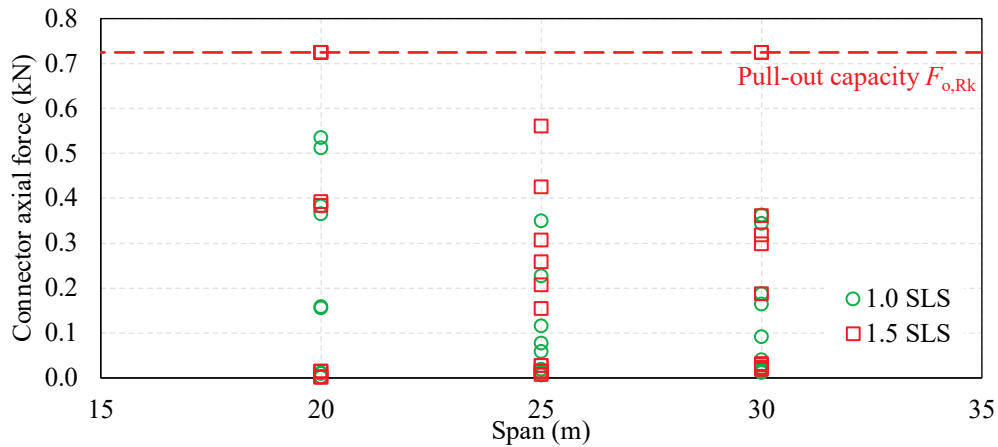
The effect of span on the external sheet connectors is shown in Figure 5.12. These are the connectors that represent the interface between the cladding primary screws and the cladding external sheet. The displacements in the external sheet connectors were significantly lower than those in the internal sheet connector. The behaviour of the external sheet connectors was dominated by shear with displacements up to 0.4 mm with no significant axial displacements, see Figure 5.13a. The effect of the span on the shear displacement of the critical external sheet connectors is shown in Figure 5.12a. The shear displacements of the critical external sheet connectors, although insignificant, increased with the increasing span. The maximum increase reached 750% observed at 1.5 SLS when the span was extended by 50% (i.e. from 20 m to 30 m).

In conclusion, under gravity load, the maximum shear displacements in screws increase with increasing the span. The cladding screws at the end bay near the apex experienced the highest demand, with the longitudinal seam screws being the most critical.



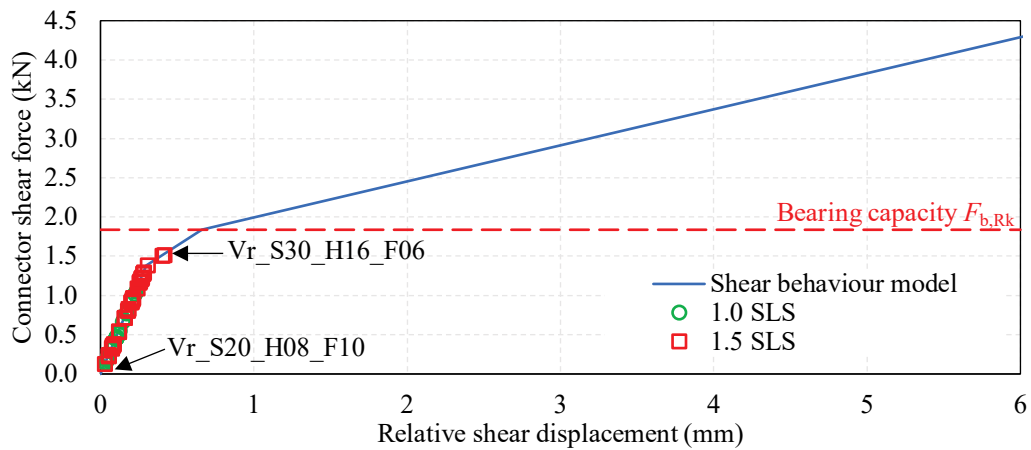
(a) Critical external sheet connector shear displacement

Figure 5.12: Influence of portal frame span on the relative shear displacement of external sheet connectors under gravity load

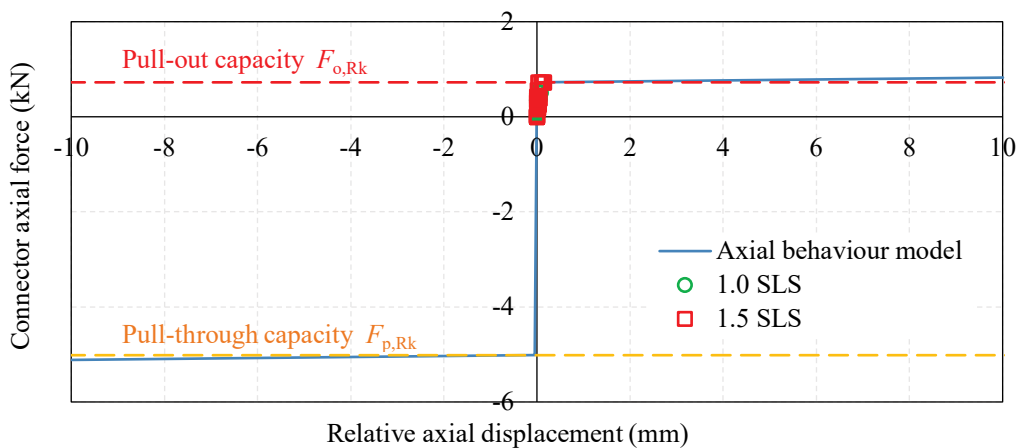


(b) Critical external sheet connector axial force

Figure 5.12 (continued): Influence of portal frame span on the relative shear displacement of external sheet connectors under gravity load



(a) Shear



(b) Axial

Figure 5.13: Relative load-displacement behaviour of the critical external sheet connectors under gravity load

Wind load

The influence of the span on the overall behaviour of the frame and the behaviour of the cladding screws under wind sway load is discussed in this section. Due to numerical convergence problems, some frame models subjected to wind sway loads stopped before reaching 1.5 SLS. It can be clearly seen in Figure 5.14 that the ratio of the reaction force carried by the gable frame to the expected applied wind load has significantly increased compared to the frames under gravity load, reaching up to 280%. Additionally, it was observed that all internal frames carried between 50-75% of the expected applied load, with the penultimate frame carrying either the same or a slightly higher ratio of load compared to the other internal frames. However, the change in span alone did not independently influence the load distribution between the frames.

The transfer of load from the internal frames to the gable frames was attributed to the gable frame high relative stiffness. Similar behaviour was documented by Roberts, Davies & Wang (2021) for portal frames clad with single-skin sheeting under wind sway loading conditions. However, the gable frames in Roberts, Davies & Wang (2021) models carried significantly higher load levels than the gable frames herein. The difference can be attributed to the cladding systems utilised in the two frame models. The frame in the current study utilised roof, wall and gable cladding using sandwich panel systems whereas the frame models in the Roberts, Davies & Wang (2021) study utilised single-skin cladding.

The location of the critical flashing, longitudinal and end lap seam screws as well as the internal sheet connectors due to wind sway loads was at the gable wall cladding, as can be seen in Figures 5.15, 5.16 and 5.19, respectively. The seam screws in the end bay and in the gable wall experienced a higher demand, attributed to the substantial load levels resisted by the gable frames. It can be seen in Figure 5.17 that the cladding panels in the gable wall underwent considerable shear displacements, resulting in an increased demand on the screws.

There was a slight increase in the shear displacement of the critical seam screws with the increasing span. It was observed that the longitudinal seam screws experienced a significantly higher displacements of up to 20 mm under wind sway load (see Figure 5.18) compared to a maximum of 5 mm under gravity loads (see Figure 5.9). For all spans considered, a number of internal sheet and external sheet connectors failed due to pull-out capacity being exceeded, as shown in Figures 5.19d, 5.20b and 5.21b.

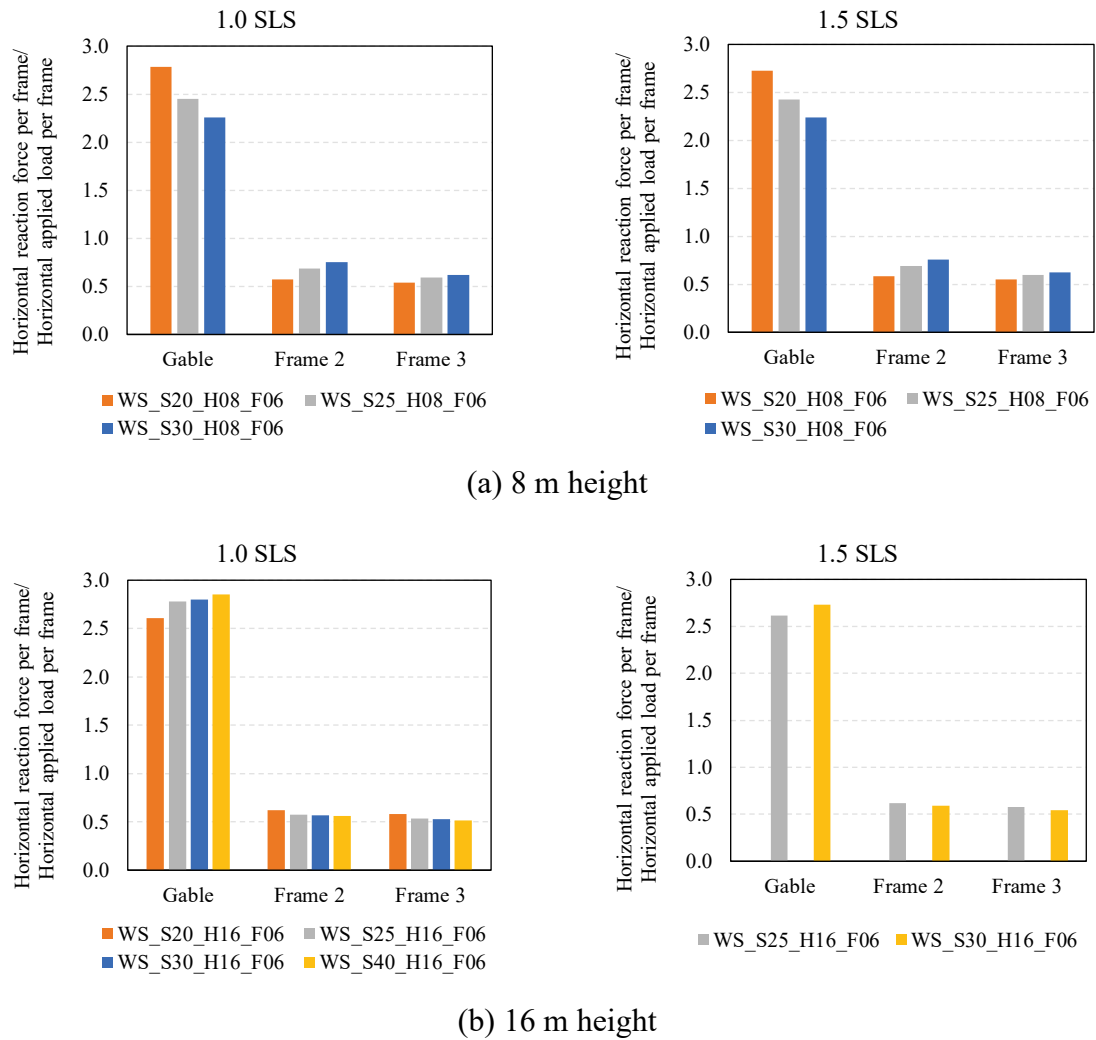
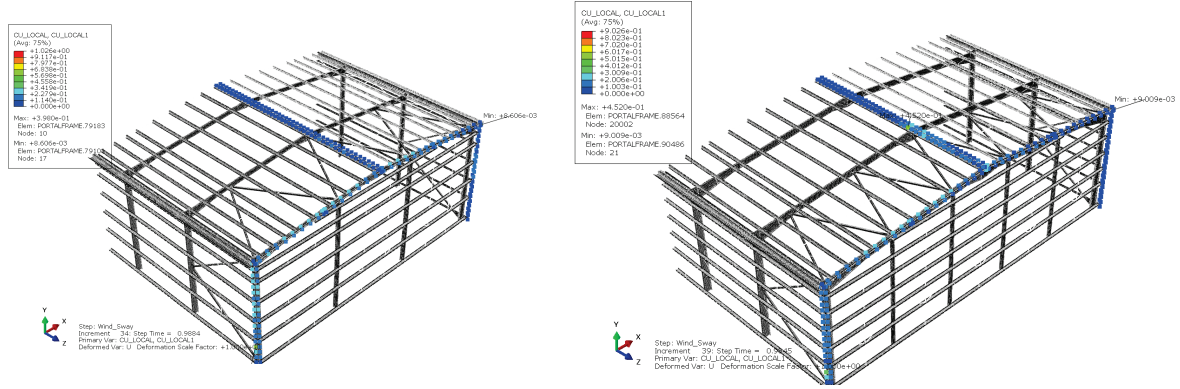
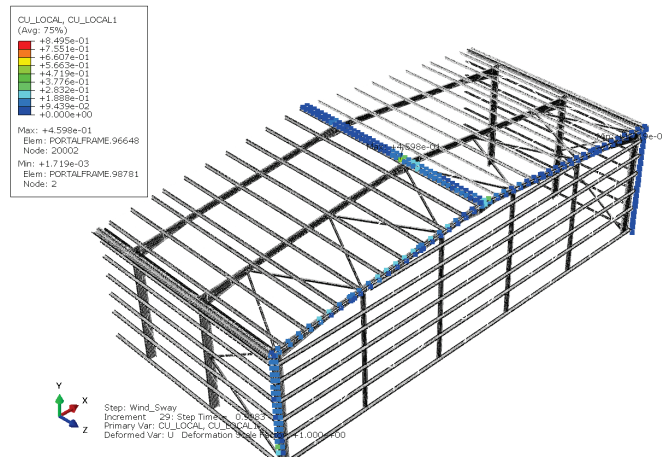


Figure 5.14: Influence of portal frame span on wind load distribution between frames



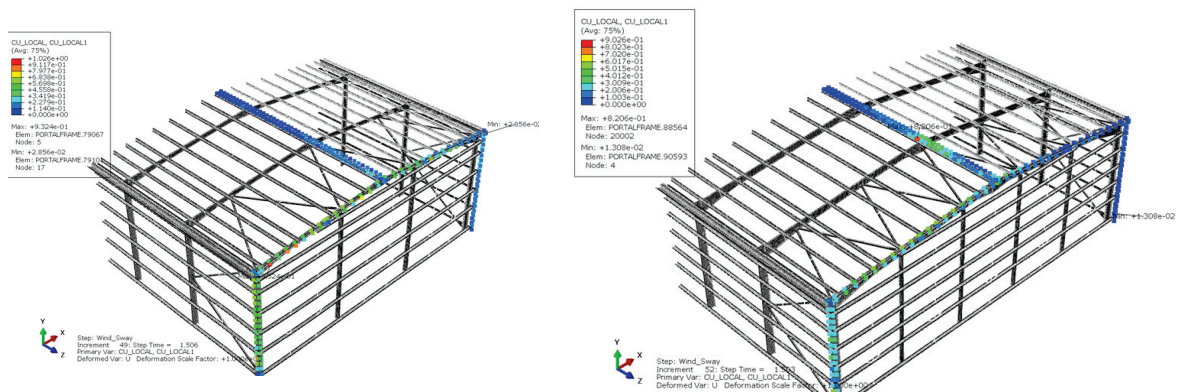
WS_S20_H08_F06

WS_S25_H08_F06



WS_S30_H08_F06

(a) 1.0 SLS

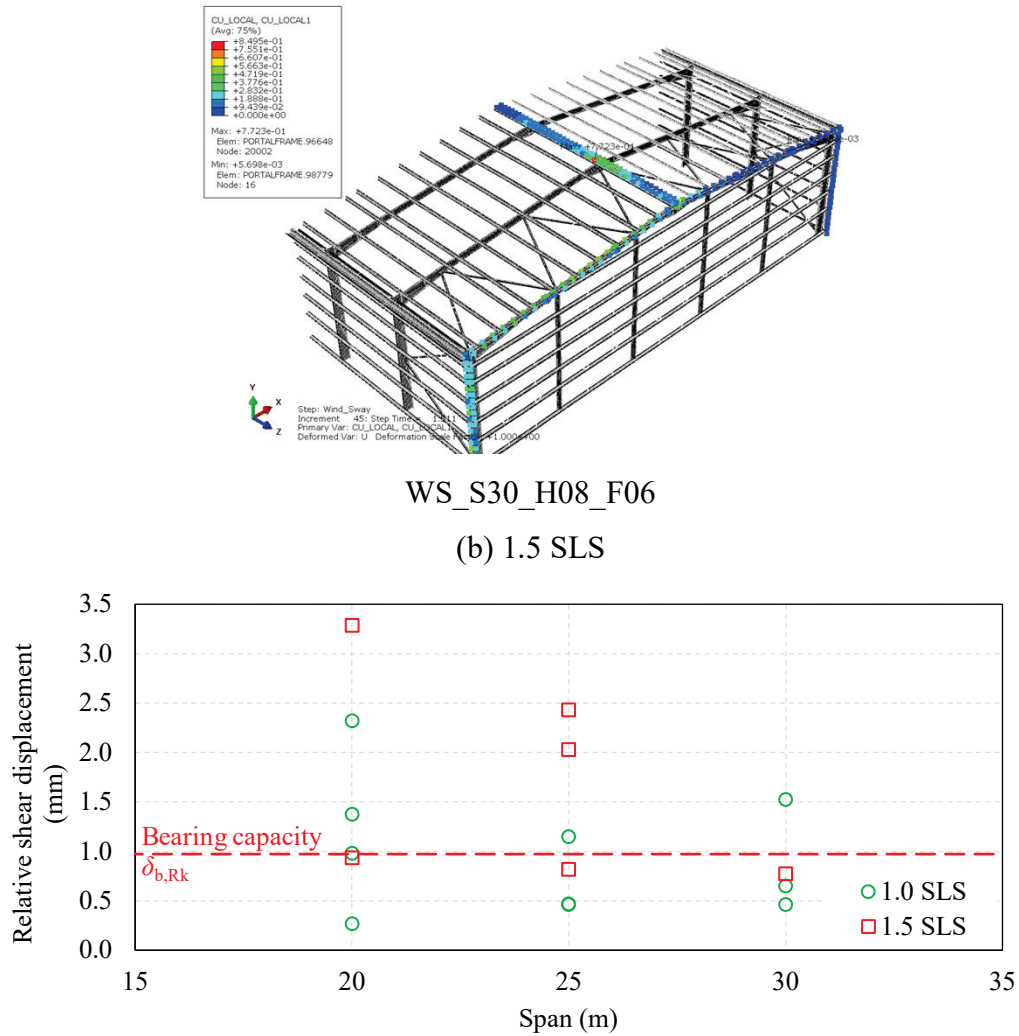


WS_S20_H08_F06

WS_S25_H08_F06

(b) 1.5 SLS

Figure 5.15: Influence of portal frame span on the relative shear displacement of flashing seam screws under wind sway load



(c) Critical flashing seam screw shear displacement

Figure 5.15 (continued): Influence of portal frame span on the relative shear displacement of flashing seam screws under wind sway load

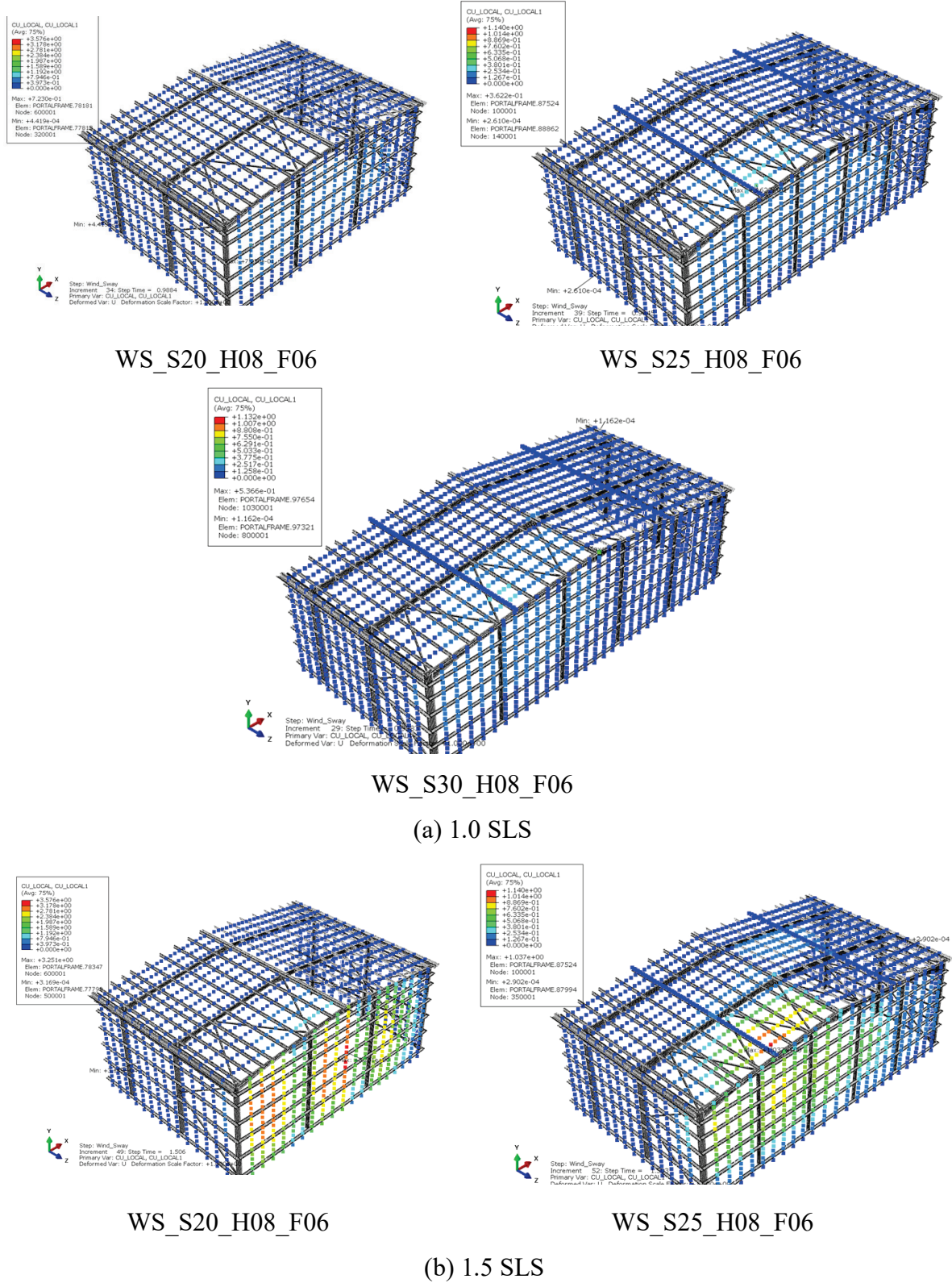


Figure 5.16: Influence of portal frame span on the relative shear displacement of longitudinal and end lap seam screws under wind sway load

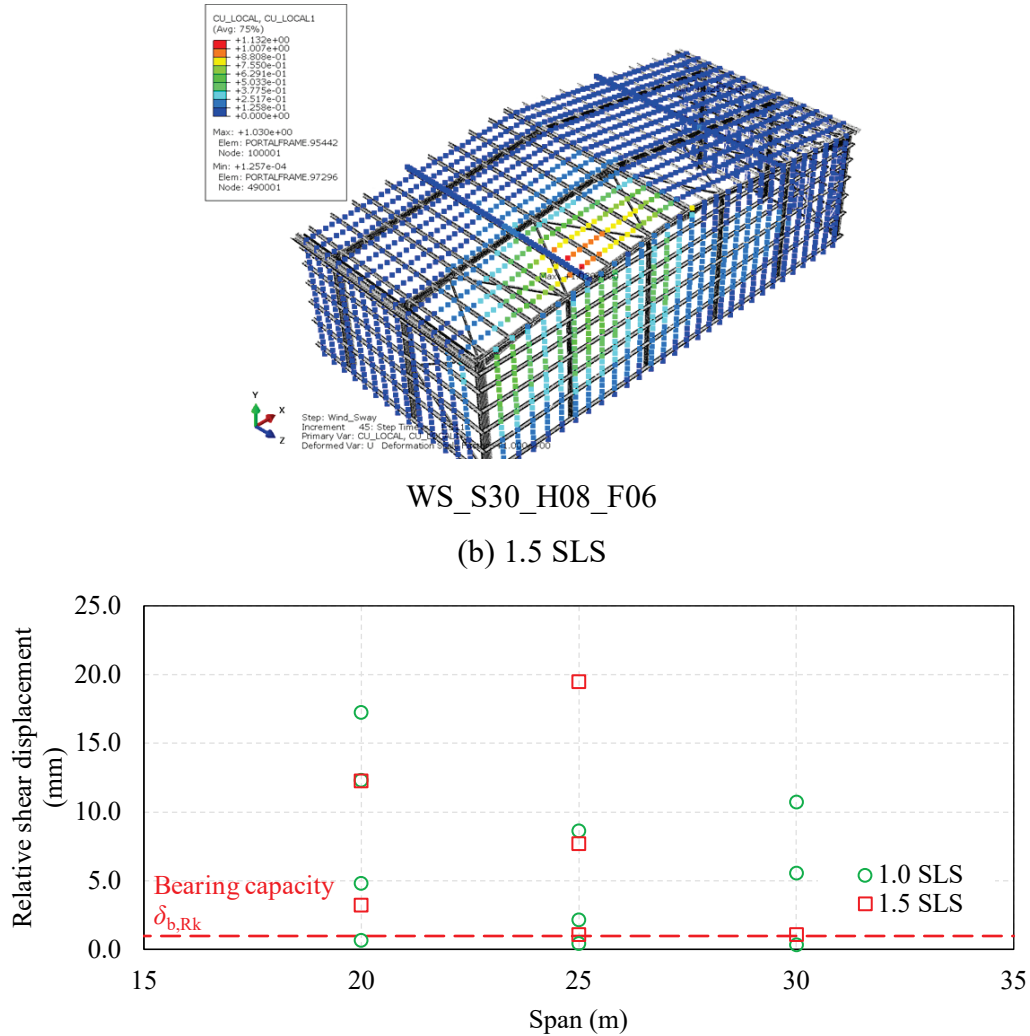


Figure 5.16 (continued): Influence of portal frame span on the relative shear displacement of longitudinal and end lap seam screws under wind sway load

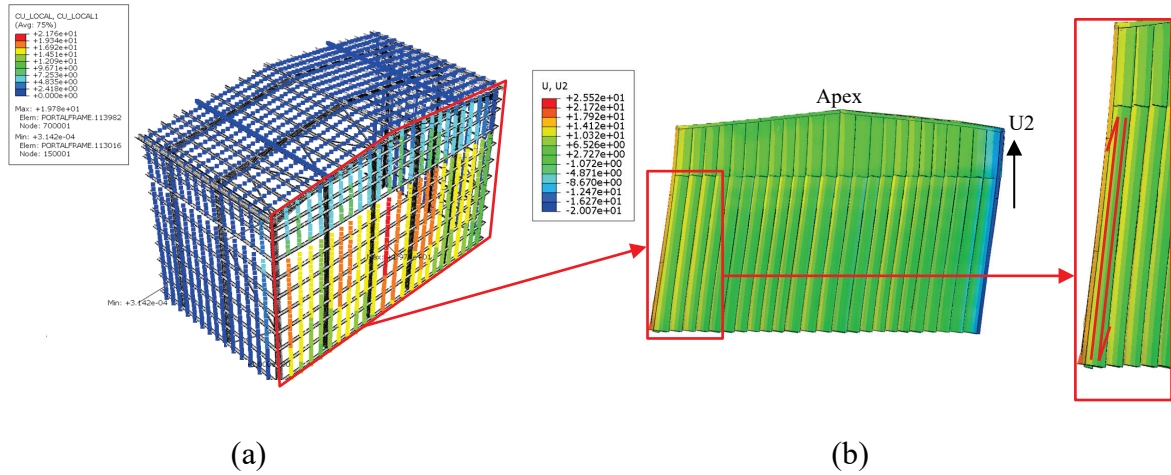
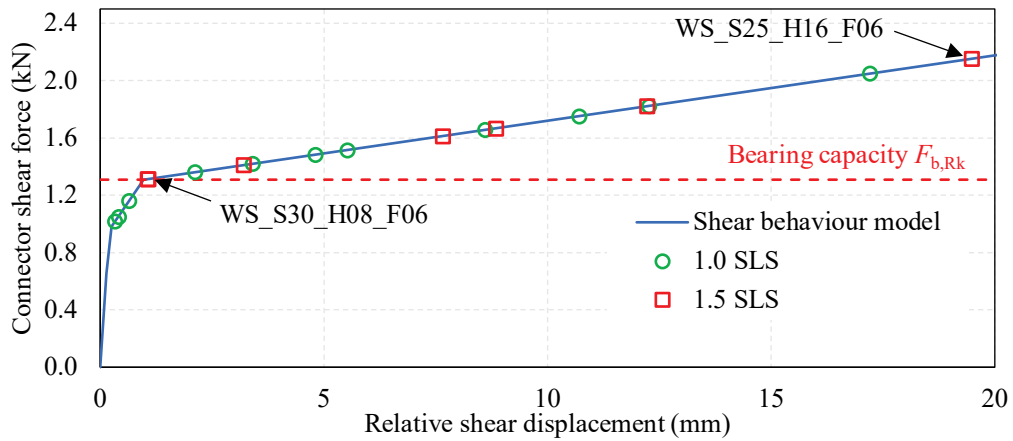
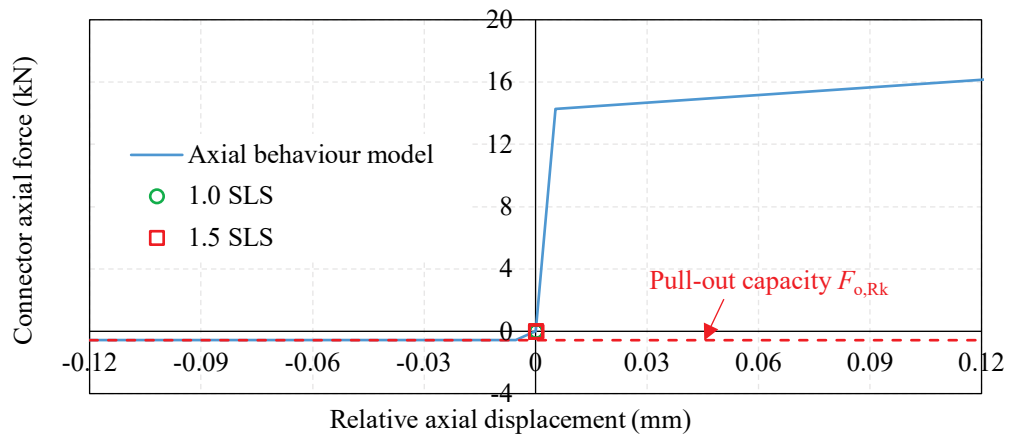


Figure 5.17: Illustration of (a) Relative shear displacement of longitudinal and end lap seam screws and (b) vertical displacement of gable cladding panels under wind sway load



(a) Shear



(b) Axial

Figure 5.18: Relative load-displacement behaviour of the critical longitudinal/end lap seam screw under wind sway load

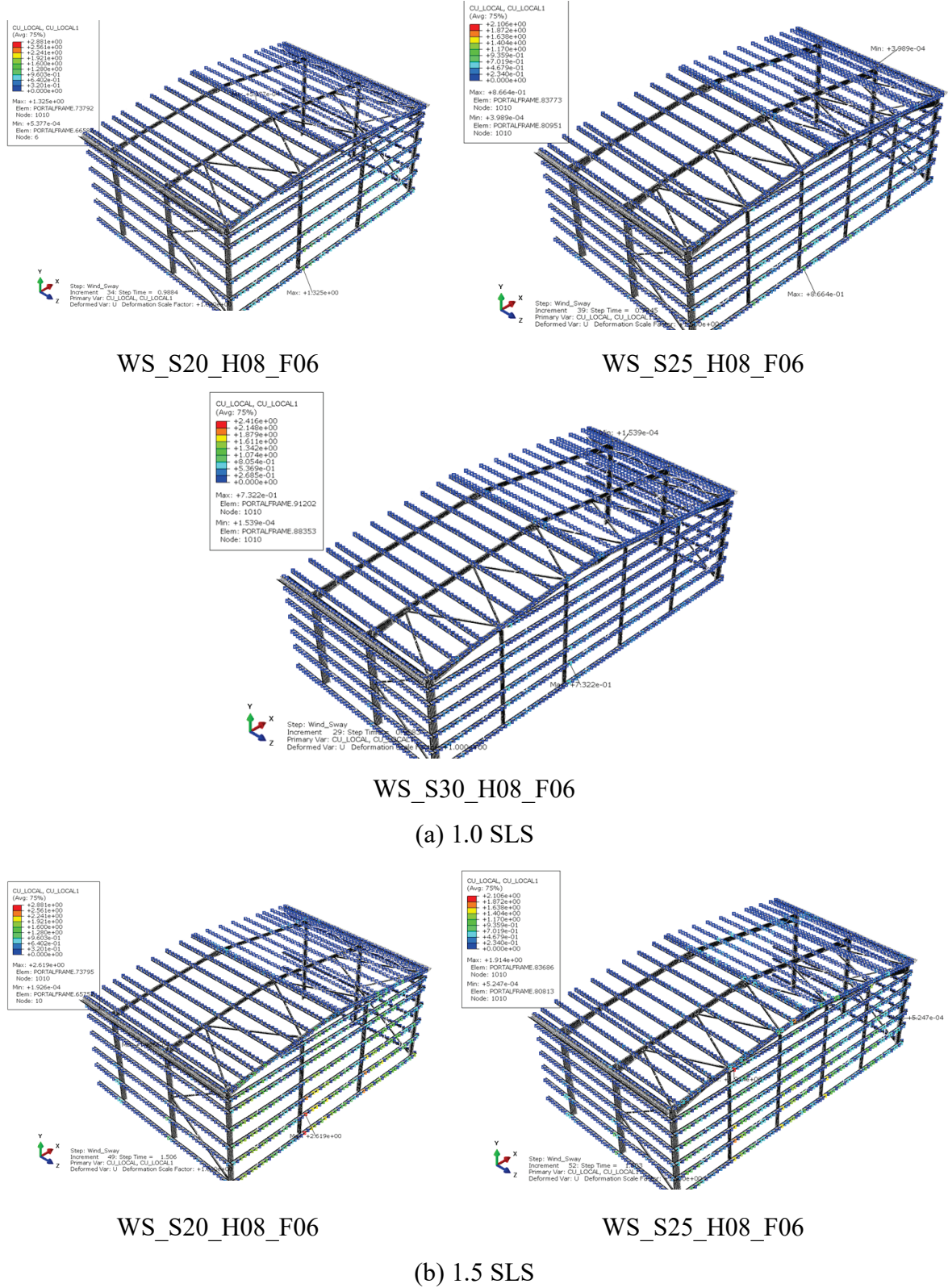
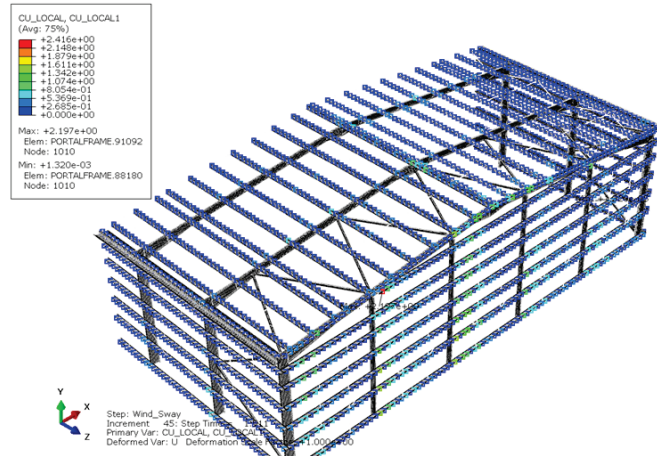
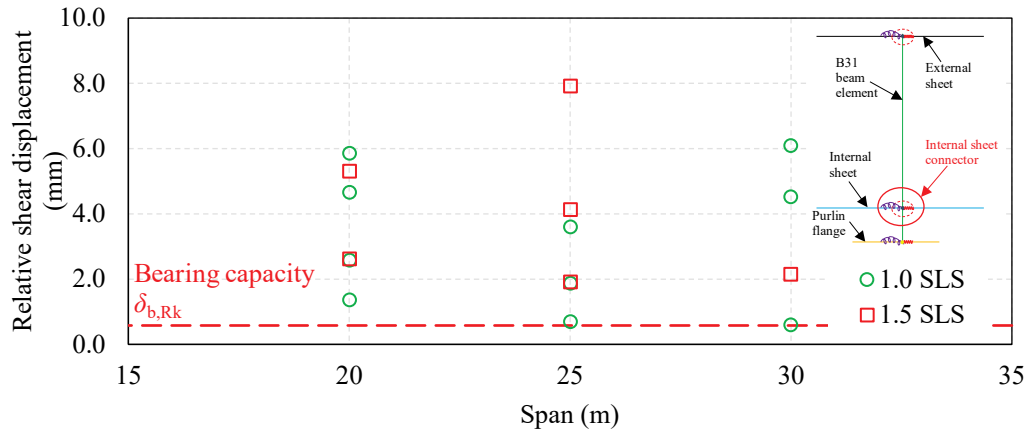


Figure 5.19: Influence of portal frame span on the relative shear displacement of internal sheet connectors under wind sway load

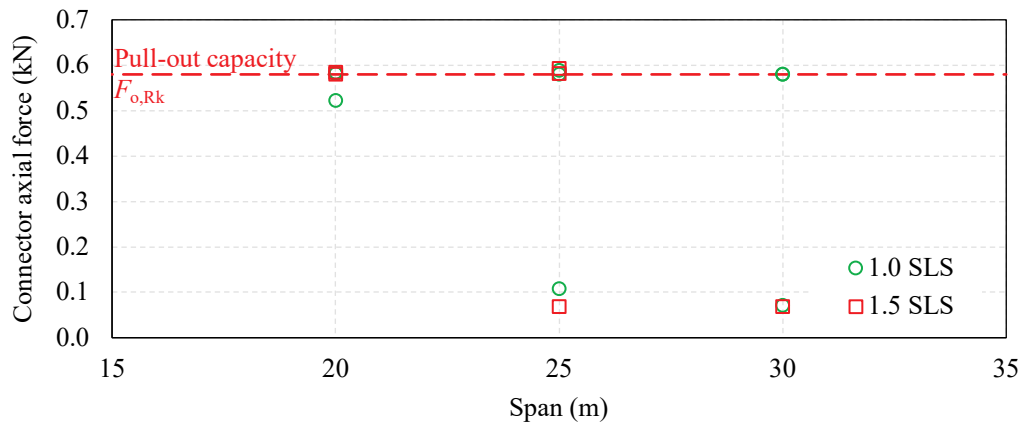


WS_S30_H08_F06

(b) 1.5 SLS

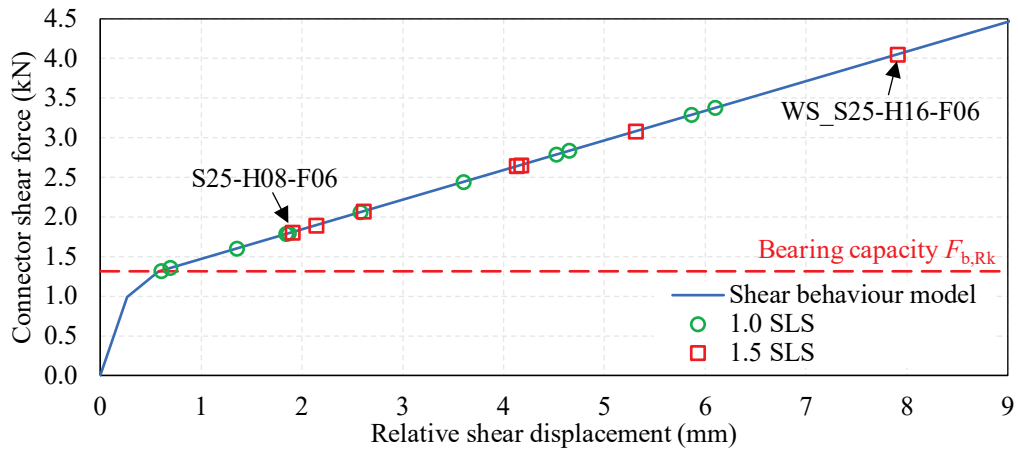


(c) Critical internal sheet connector shear displacement

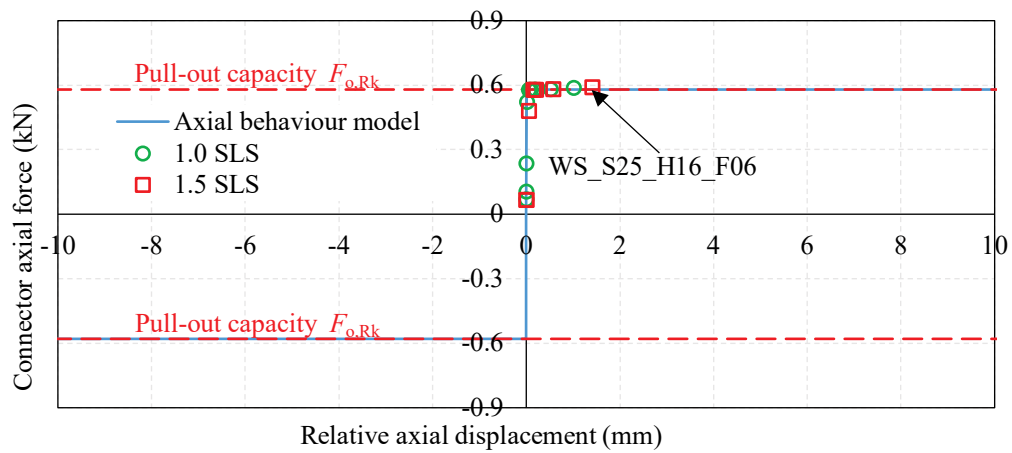


(d) Critical internal sheet connector axial force

Figure 5.19 (continued): Influence of portal frame span on the relative shear displacement of internal sheet connectors under wind sway load

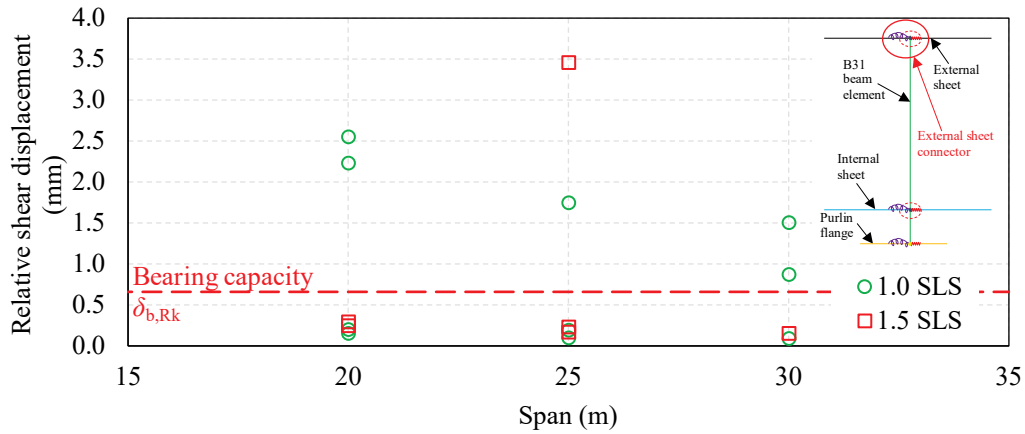


(a) Shear

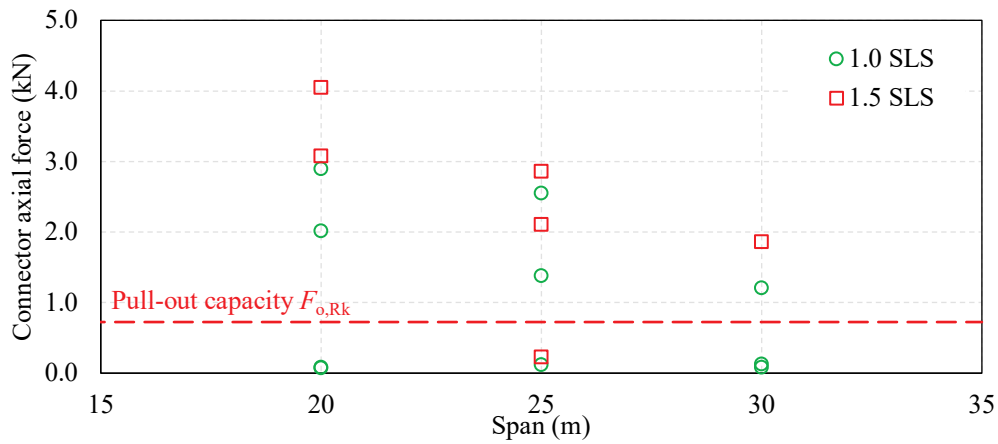


(b) Axial

Figure 5.20: Relative load-displacement behaviour of the critical internal sheet connector under wind sway load



(a) Critical external sheet connector shear displacement



(b) Critical external sheet connector axial force

Figure 5.21: Influence of portal frame span on the relative shear displacement of external sheet connectors under wind sway load

Thermal actions

The influence of the span on the overall behaviour of the frame and the behaviour of the cladding screws under thermal action is discussed in this section. The external sheet temperature was first lowered from +20°C to -10°C then raised again to +50°C, causing connector displacements in opposite directions. Therefore, the connector displacements shown here at +50°C are cumulative, reflecting the actual displacements that would be observed in the structure.

Although the location of the critical screws was consistent with other load cases, being at the end bays and near the apex, it was also affected by the dimensions of the frame, see Figures 5.22, 5.24 and 5.26. For example, it can be noted in Figure 5.22 that the flashing seam screws were most critical at the gable flashings (i.e. the corner flashing and the barge cap) in the smaller frame and at the apex hat at the larger frames.

On the contrary to the other load cases, the most critical longitudinal and end lap seam screw was the end lap seam screw at the gable frame, see Figure Figure 5.24. It can also be seen that the critical screw behaviour of the longitudinal and end lap seam screw in the larger frame propagated into the second bay compared to the end bay in other frame cases. The increase in span from 20 m to 30 had a slight effect on the maximum connector shear displacements. However, the largest frame span (i.e. 40 m) has resulted in a significantly higher connector shear displacements as shown in Figures 5.23, 5.25 and 5.27.

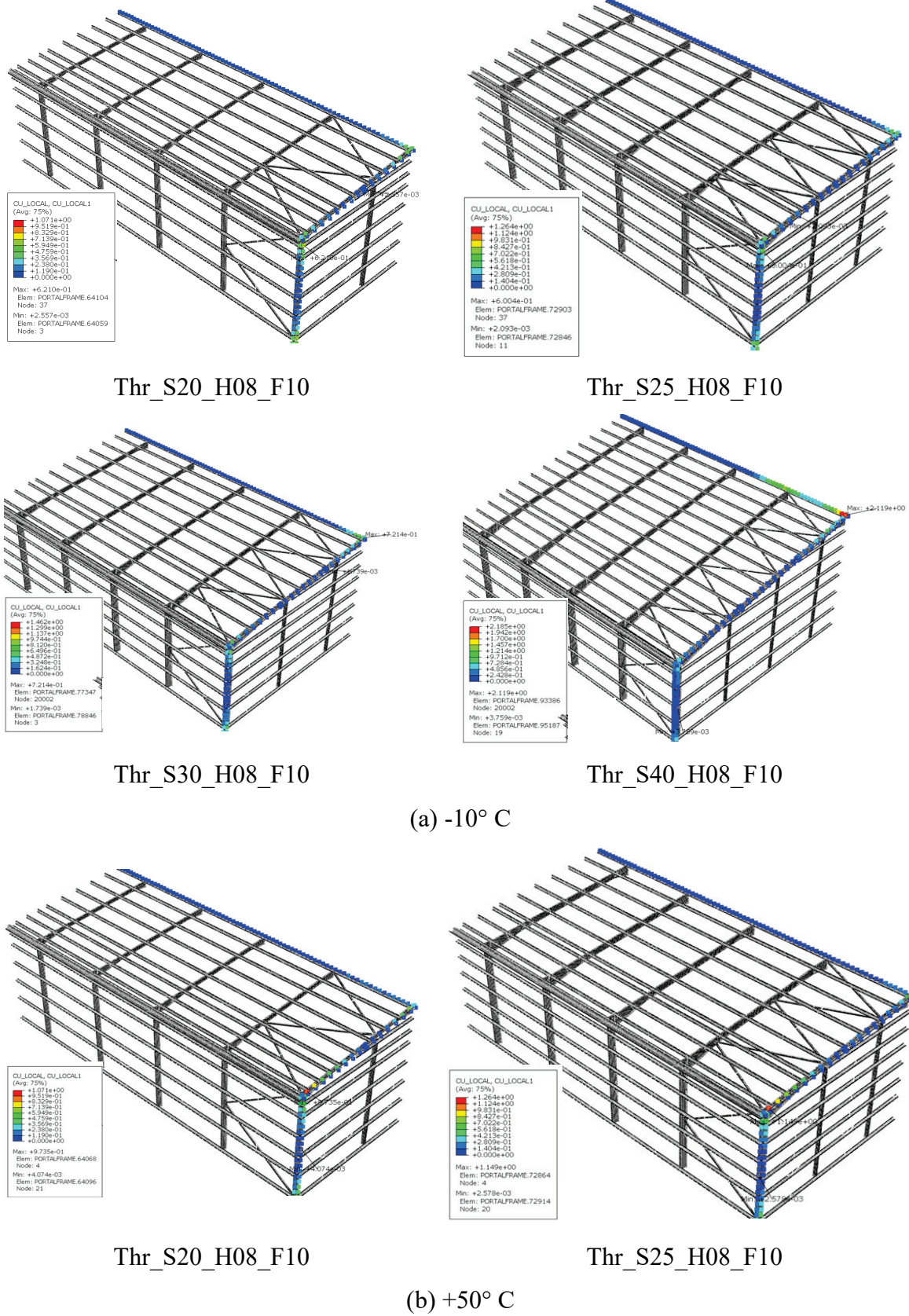
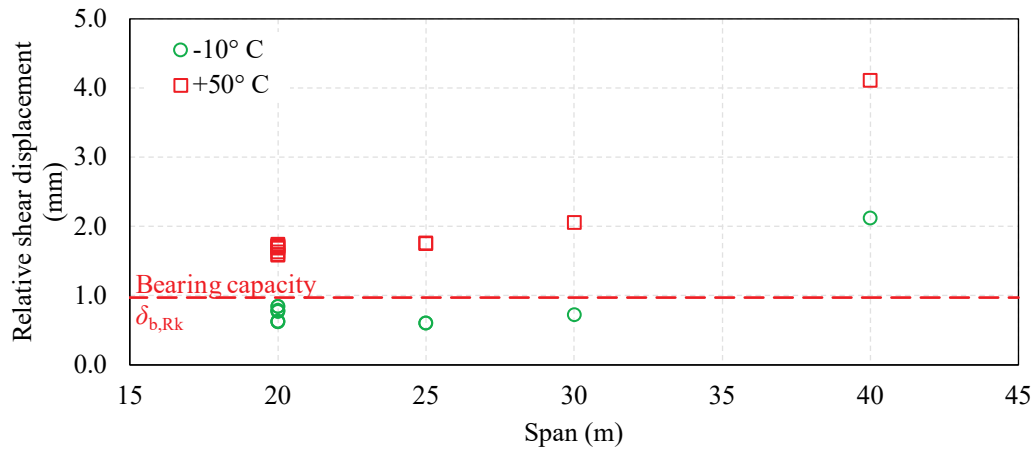
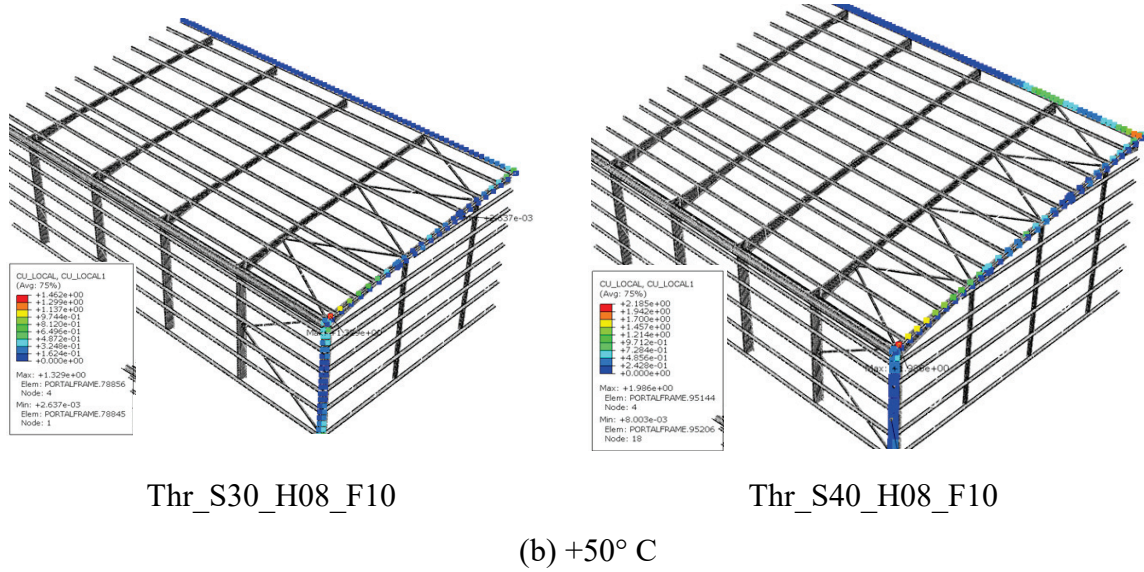


Figure 5.22: Influence of portal frame span on the relative shear displacement of flashing seam screws under thermal action



(c) Critical flashing seam screw shear displacement

Figure 5.22(continued): Influence of portal frame span on the relative shear displacement of flashing seam screws under thermal action

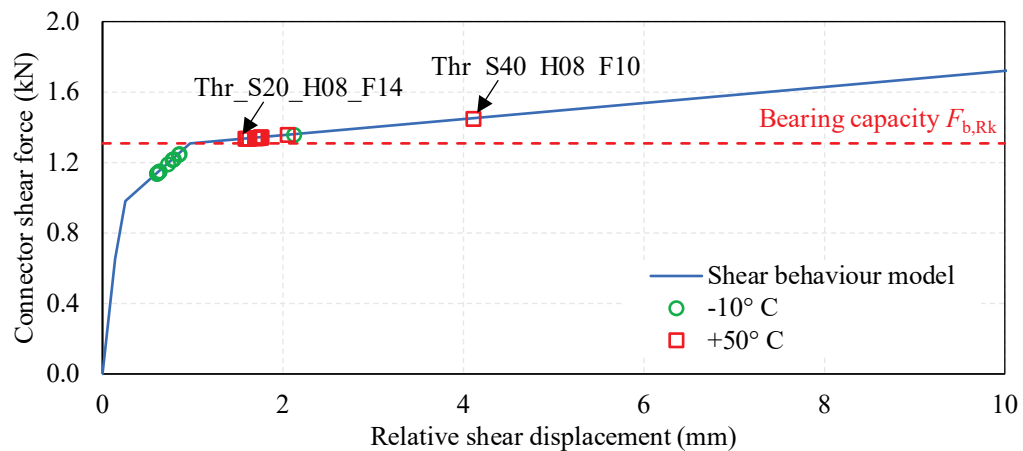


Figure 5.23: Relative shear load-displacement behaviour of the critical flashing seam screw under thermal action

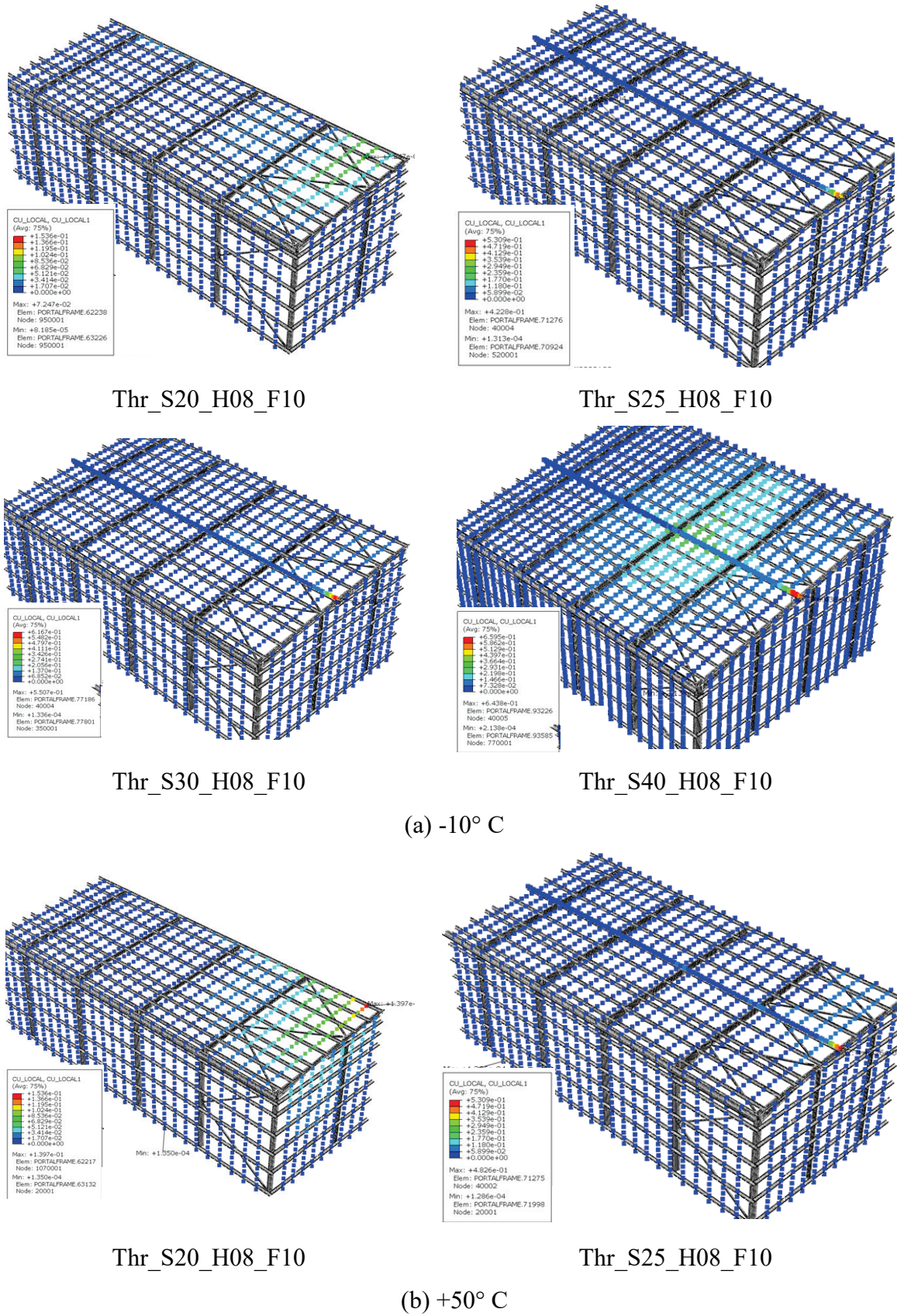
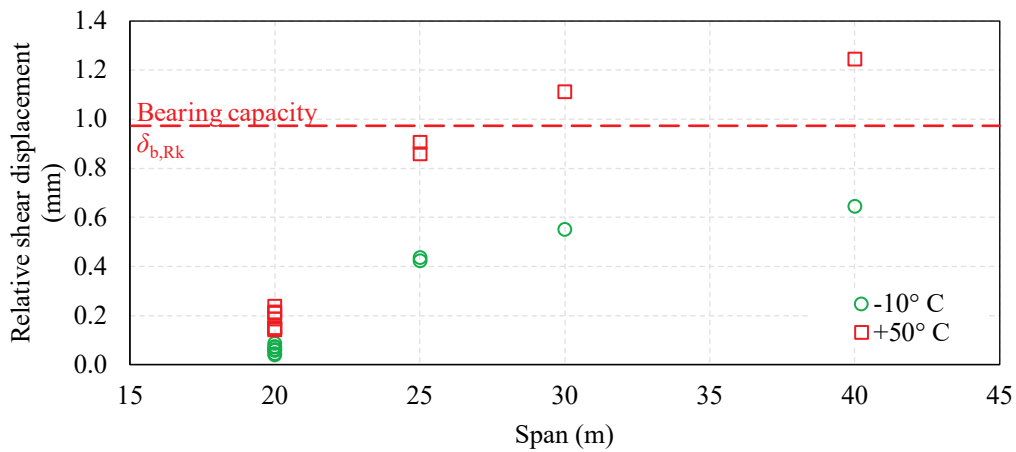
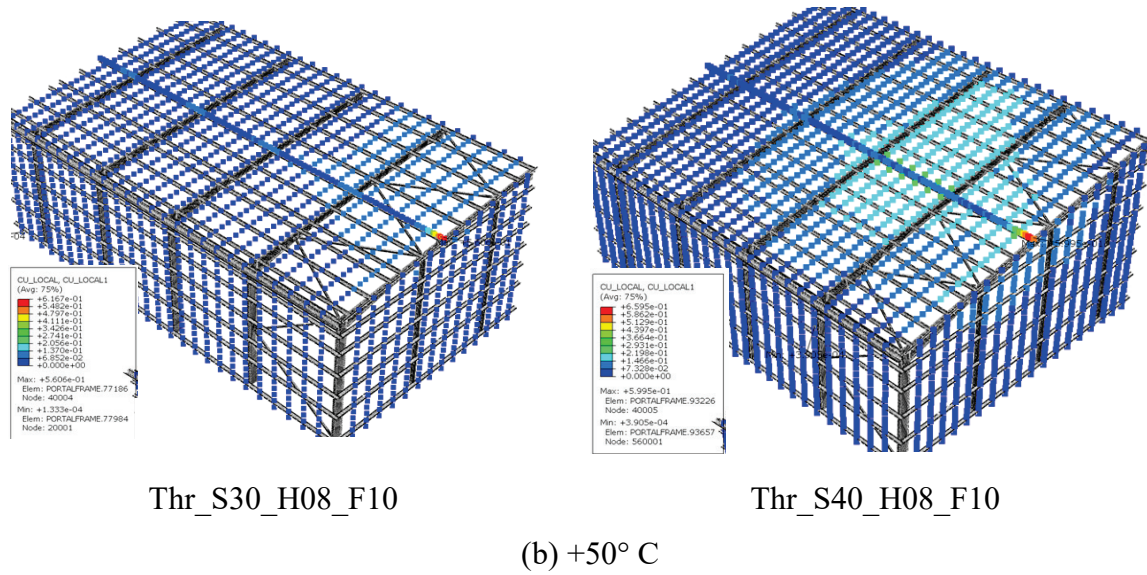


Figure 5.24: Influence of portal frame span on the relative shear displacement of longitudinal and end lap seam screws under thermal action



(c) Critical longitudinal and end lap seam screw shear displacement

Figure 5.24 (continued): Influence of portal frame span on the relative shear displacement of longitudinal and end lap seam screws under thermal action

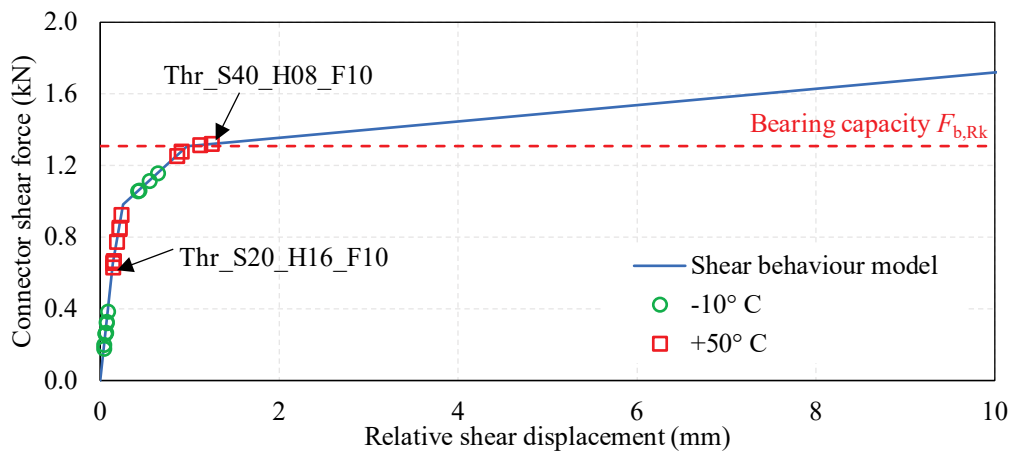


Figure 5.25: Relative load-displacement behaviour of the critical longitudinal/end lap seam screw under thermal action

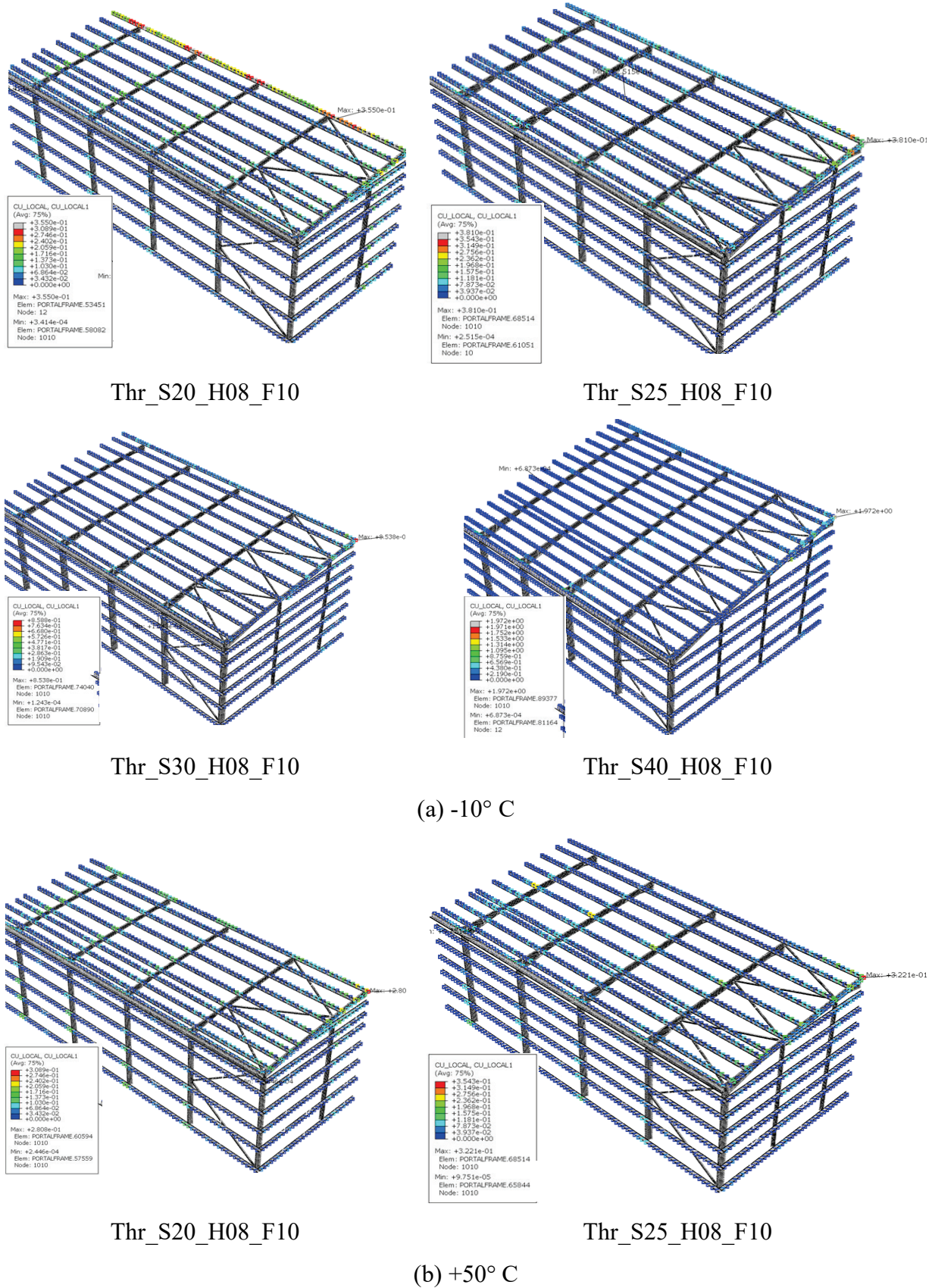
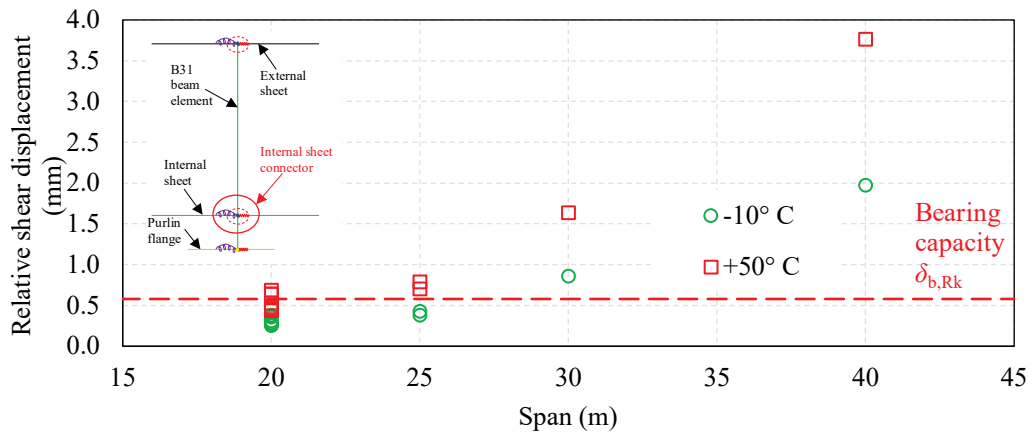
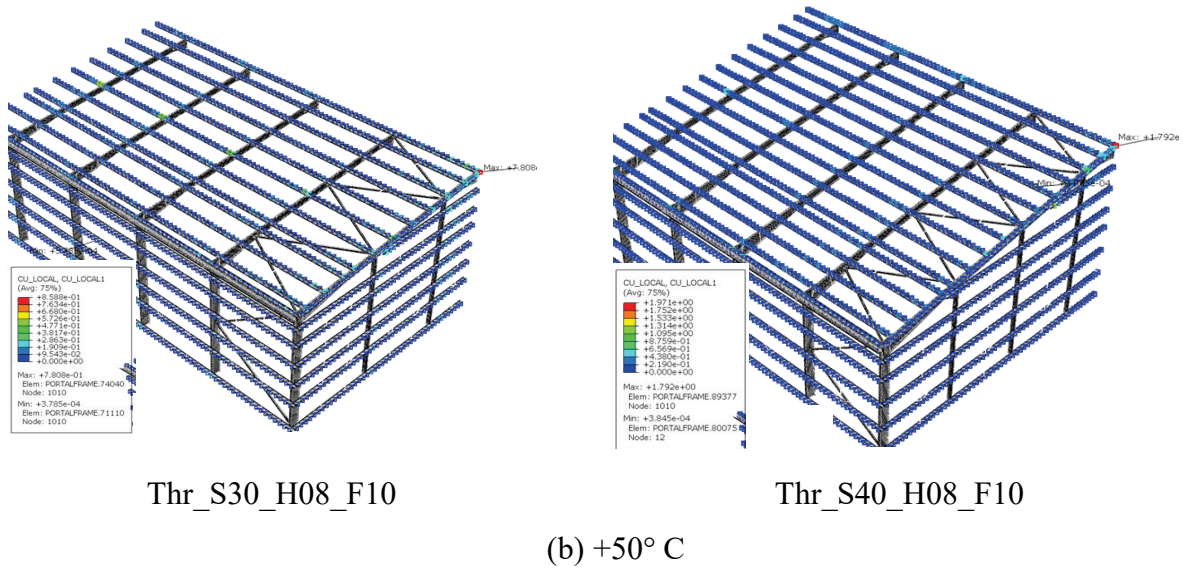


Figure 5.26: Influence of portal frame span on the relative shear displacement of internal sheet connectors under thermal action



(c) Critical internal sheet connector shear displacement

Figure 5.26 (continued): Influence of portal frame span on the relative shear displacement of internal sheet connectors under thermal action

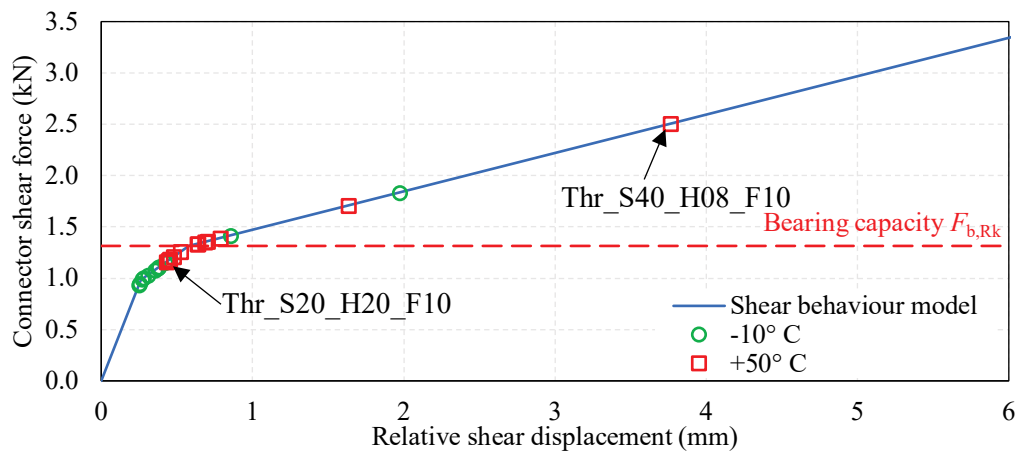


Figure 5.27: Relative load-displacement behaviour of the critical internal sheet connector under thermal action

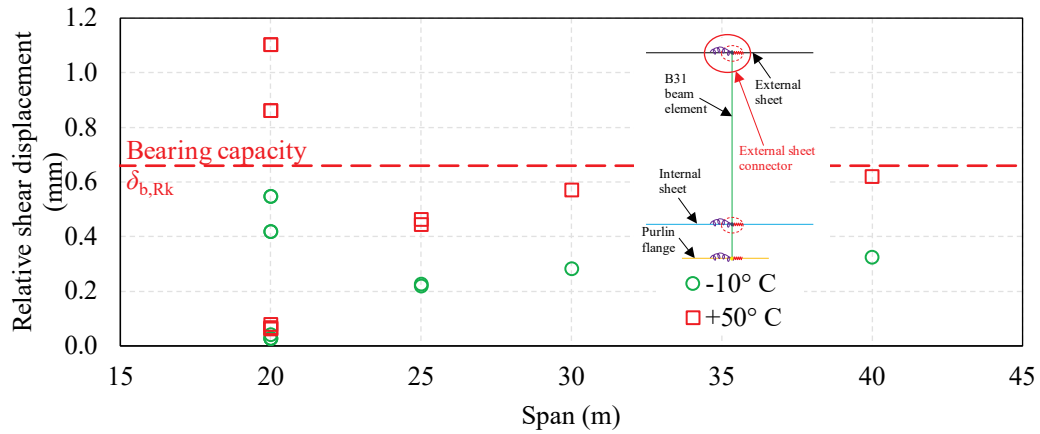
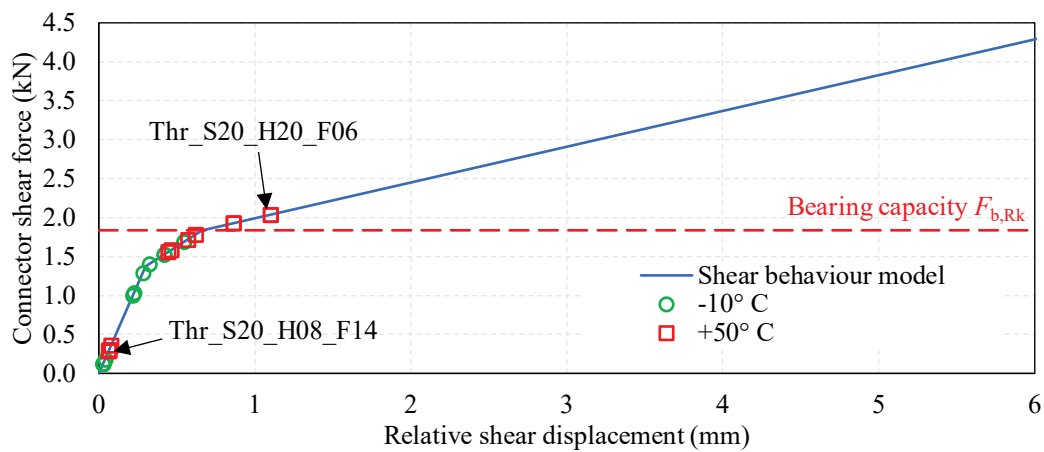


Figure 5.28: Influence of portal frame span on the relative shear displacement of external sheet connectors under thermal action



(b) Axial

Figure 5.29: Relative load-displacement behaviour of the critical external sheet connector under thermal action

5.4.2 Effect of height

Gravity load

The influence of portal frame height under gravity load on the overall behaviour of the frame and the behaviour of the cladding screws is discussed in this section. The effect of height on the load distribution between frames is shown in Figure 5.30 considering the 25 m span frames. A slight increase in the ratio of the reaction force carried by the penultimate frame compared to the expected applied load was observed with increasing height.

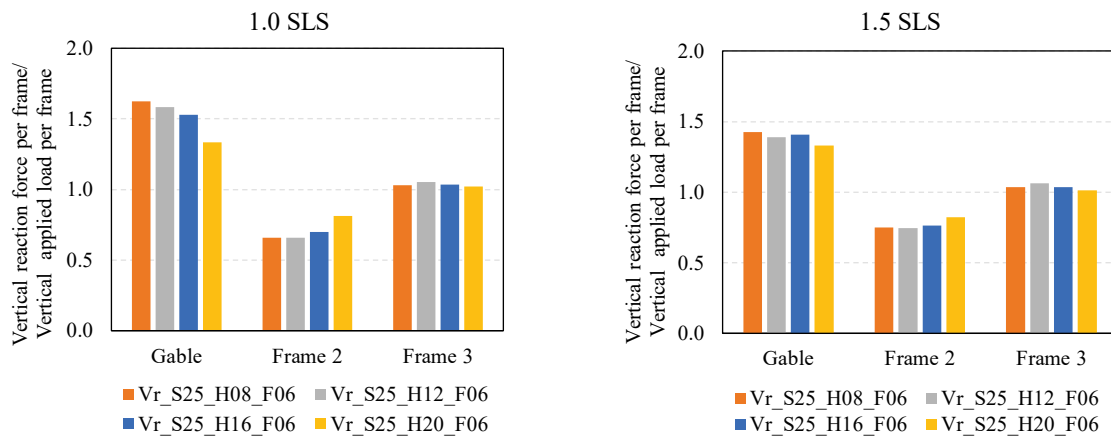
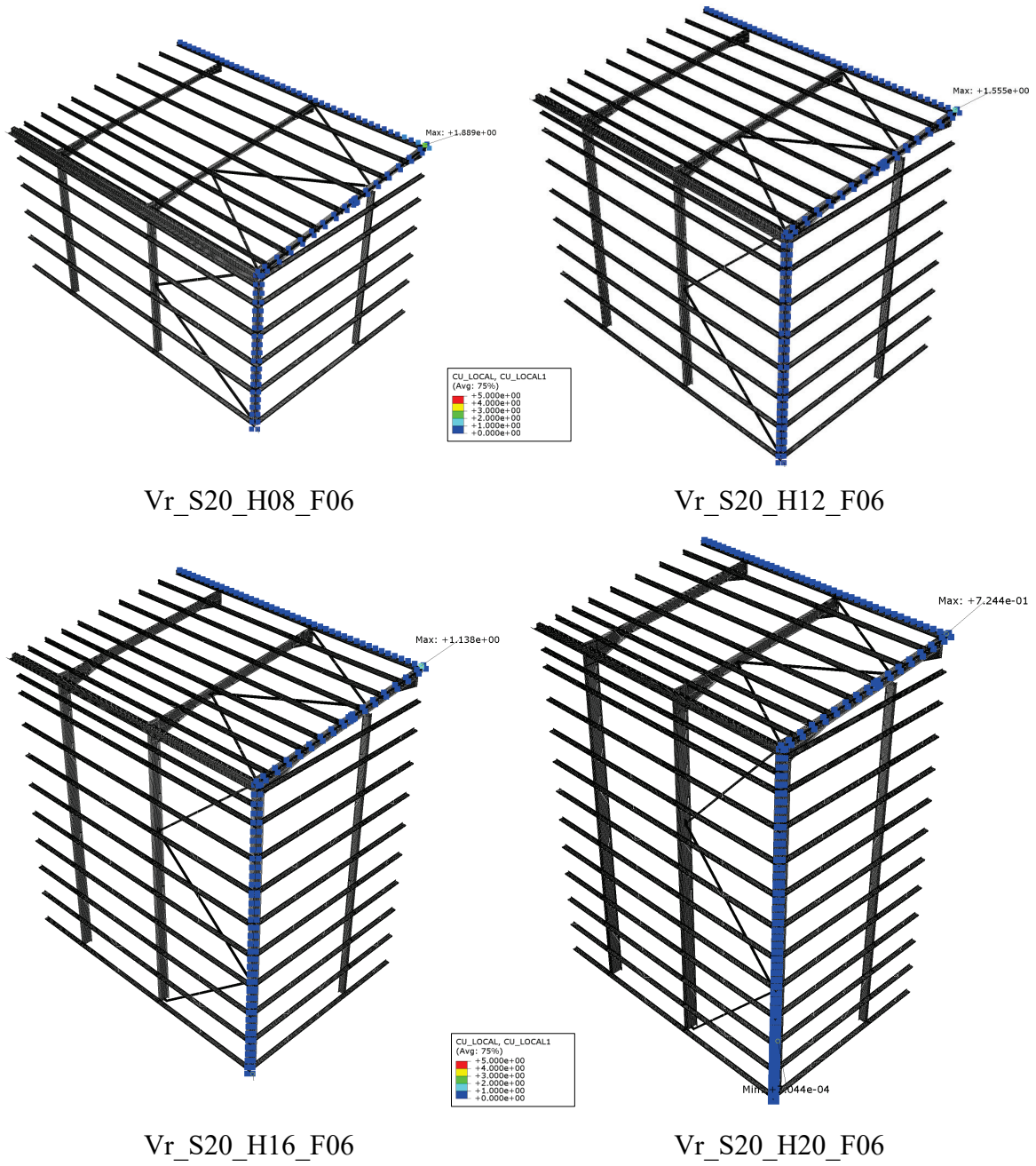


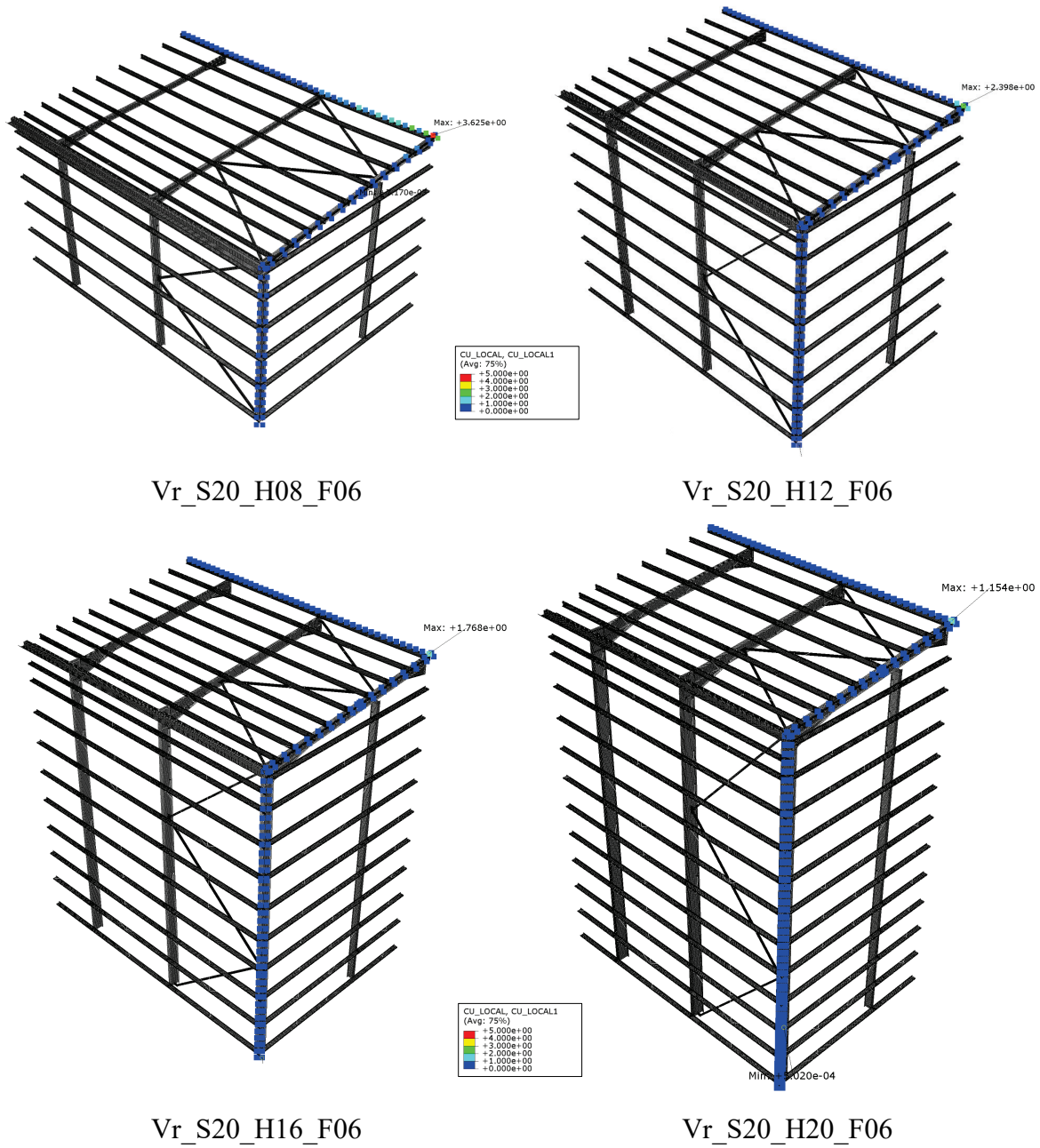
Figure 5.30: Influence of portal frame height on gravity load distribution between frames

The effect of height on the flashing seam screws is shown in Figure 5.31, with the magnitudes of shear displacement of the critical flashing seam screw shown in Figure 5.31c. It can clearly be observed that the maximum shear displacements decrease with increasing height. A 50% increase in height (8 m to 12 m) resulted in up to a 40% decrease in the shear displacement of the critical flashing seam screw while a 150% increase in height (8 m to 20 m) resulted in up to a 70% decrease in the shear displacement of the critical flashing seam screw, both observed at the serviceability limit state.



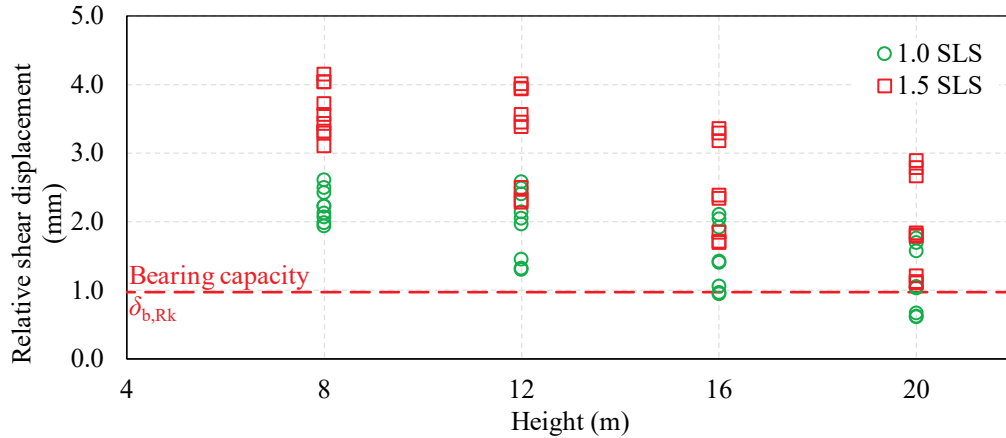
(a) 1.0 SLS

Figure 5.31: Influence of portal frame height on the relative shear displacement of flashing seam screws under gravity load



(b) 1.5 SLS

Figure 5.31 (continued): Influence of portal frame height on the relative shear displacement of flashing seam screws under gravity load



(c) Critical flashing seam screw shear displacement

Figure 5.31 (continued): Influence of portal frame height on the relative shear displacement of flashing seam screws under gravity load

The effect of height on the longitudinal and end lap seam screws is shown in Figure 5.32. The effect of the height on the shear displacement of the critical longitudinal seam screw is shown in Figure 5.32c. The same conclusions can be made as for the flashing seam screws, with a decrease in maximum shear displacement with increasing height. The maximum effect of height was observed at the serviceability limit state (SLS) with 50% increase in height resulting in up to a 70% decrease in the shear displacement of the critical longitudinal seam screw and a 150% increase in height resulting in up to a 90% decrease in the shear displacement of the critical longitudinal seam screw.

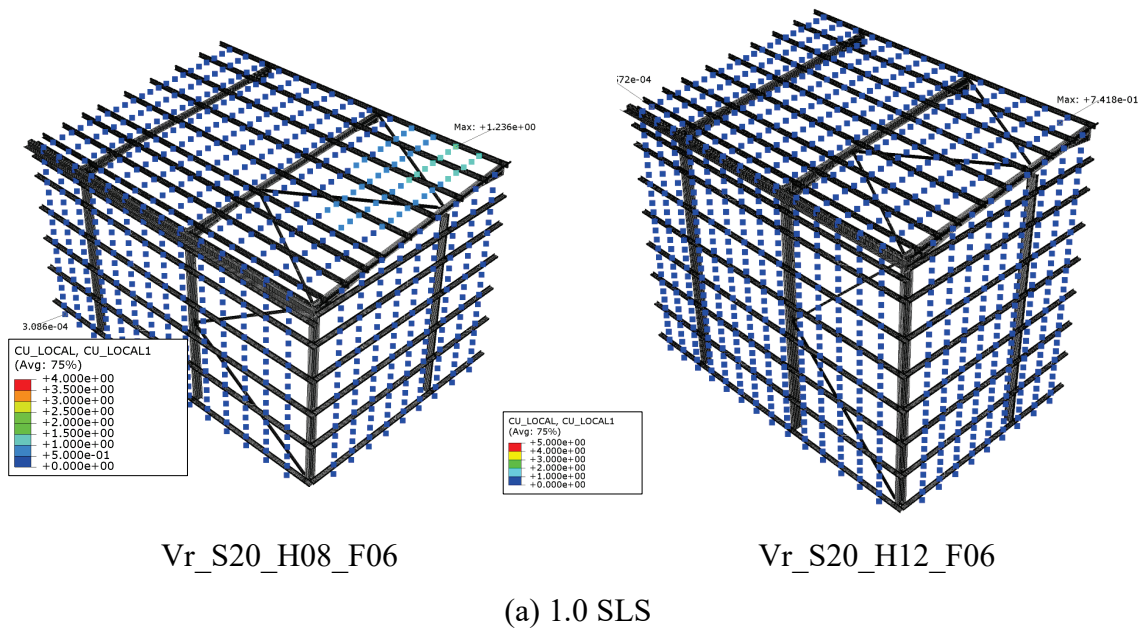


Figure 5.32: Influence of portal frame height on the relative shear displacement of longitudinal and end lap seam screws under gravity load

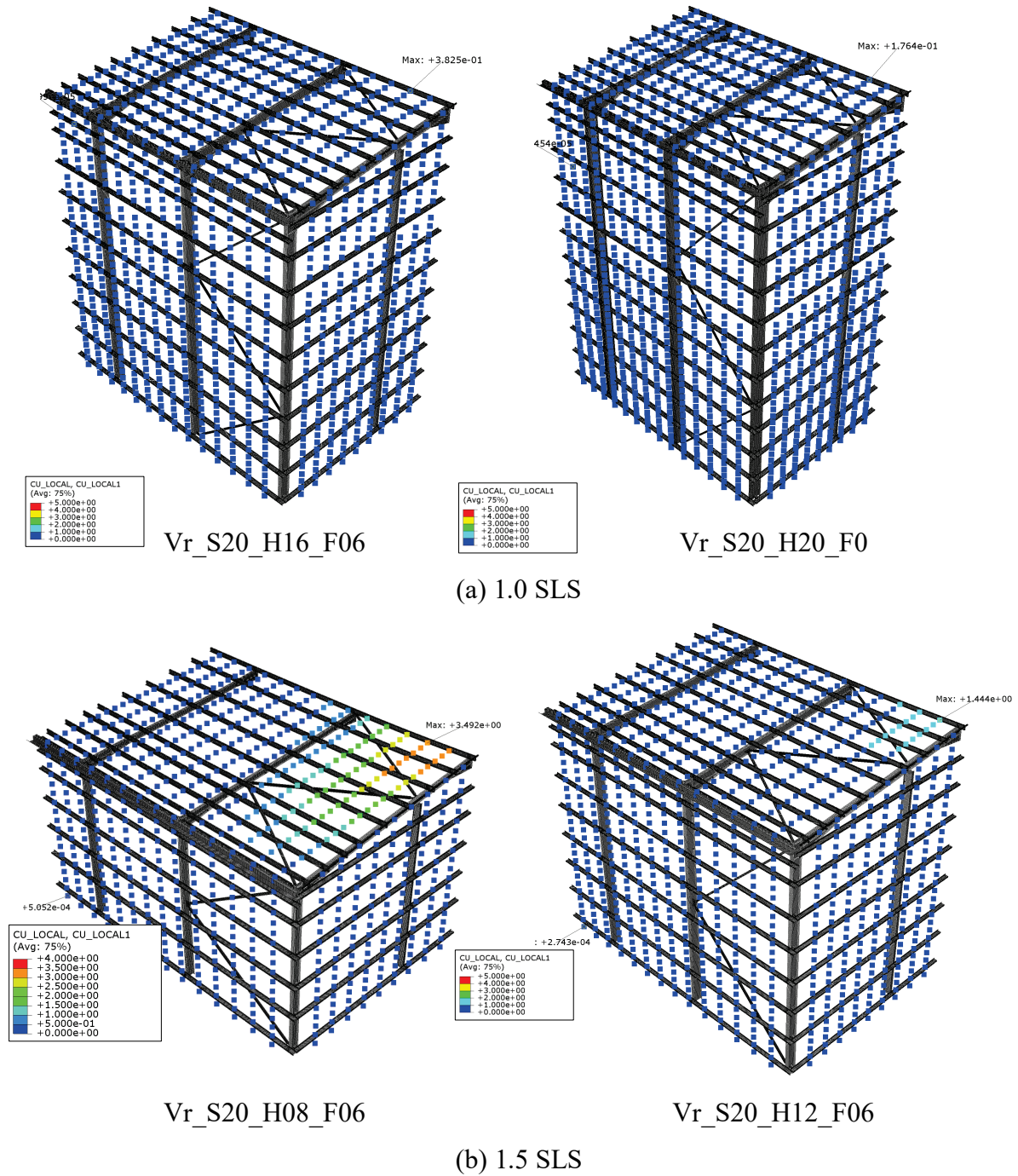
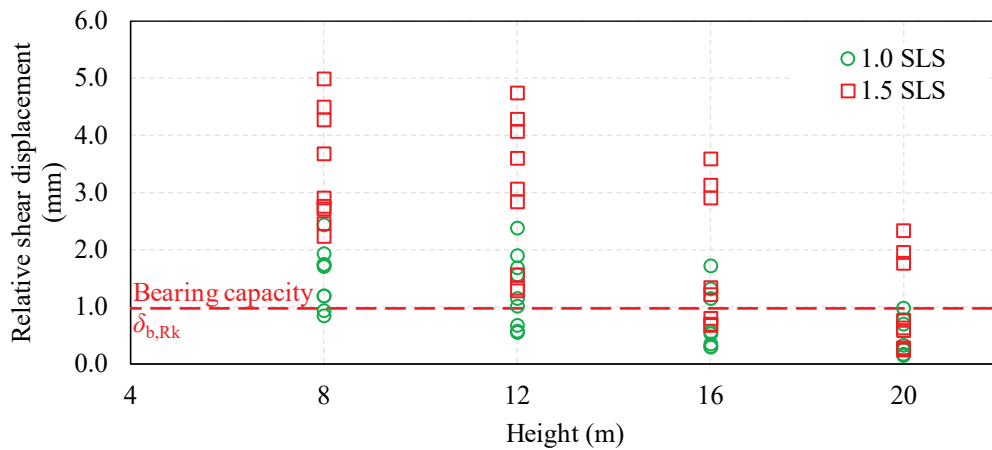
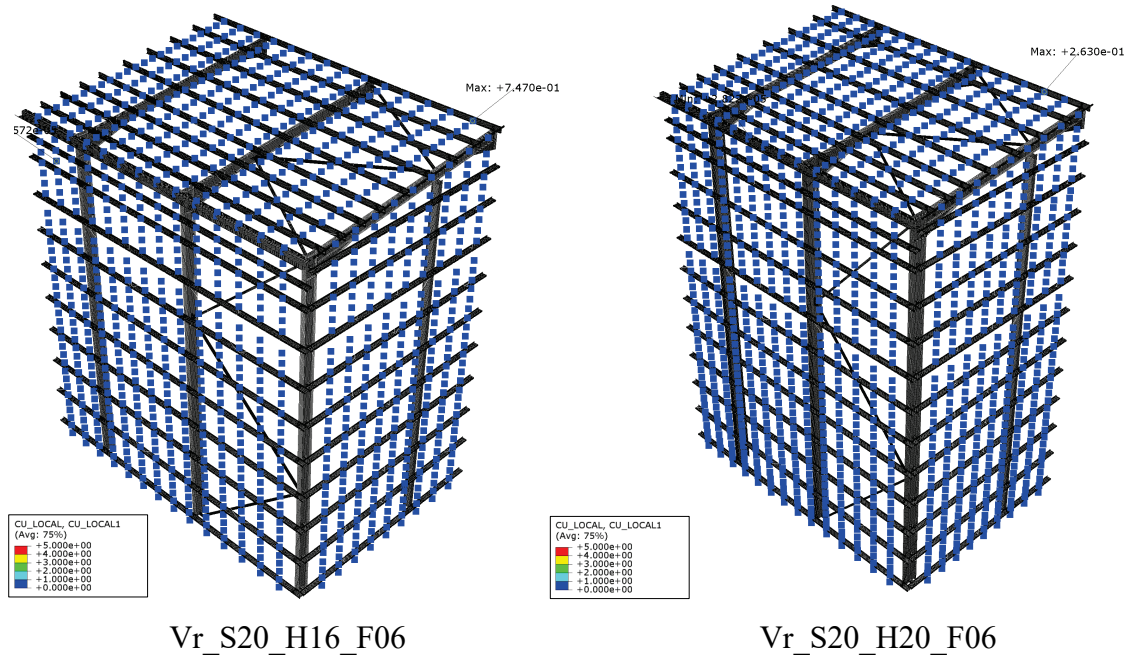


Figure 5.32 (continued): Influence of portal frame height on the relative shear displacement of longitudinal and end lap seam screws under gravity load



(c) Critical longitudinal seam screw shear displacement

Figure 5.32 (continued): Influence of portal frame height on the relative shear displacement of longitudinal and end lap seam screws under gravity load

The effect of height on the internal sheet connectors is shown in Figure 5.33, while the shear displacement of the critical internal sheet connectors is shown in Figure 5.33c. At the serviceability limit state, a 50% increase in height resulted in up to a 35% decrease in the shear displacement of the critical internal sheet connector while a 150% increase in height resulted in up to a 80% decrease in the shear displacement of the critical internal sheet connector.

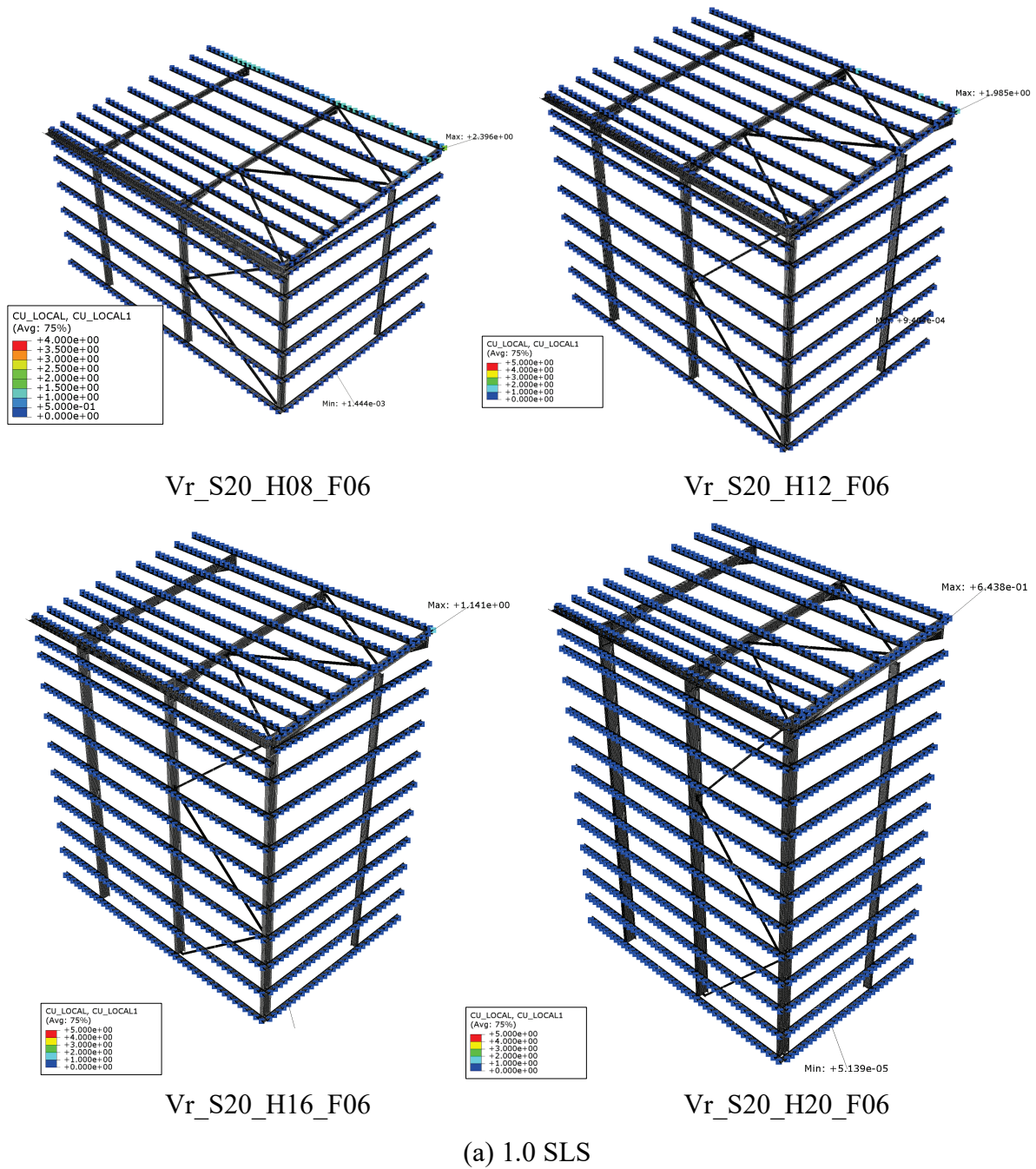
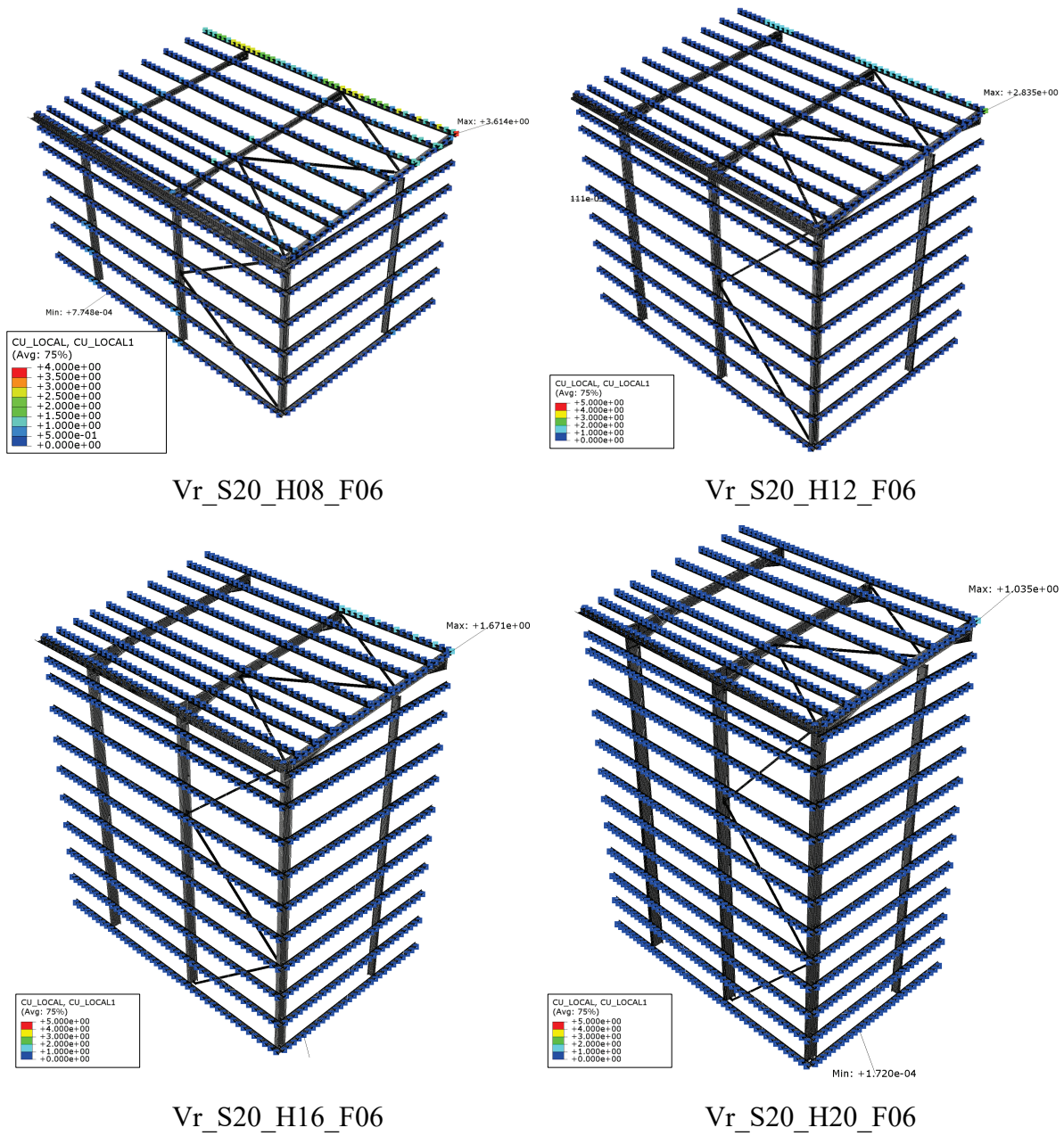
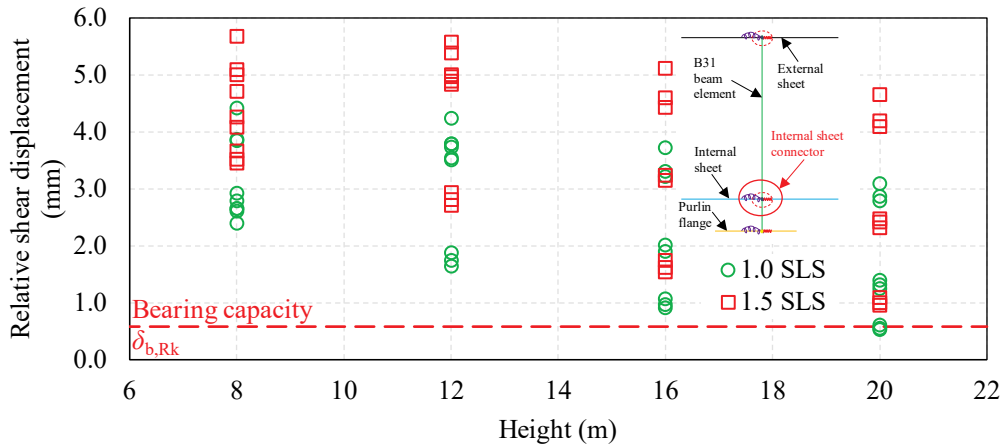


Figure 5.33: Influence of portal frame height on the relative shear displacement of internal sheet connectors under gravity load



(b) 1.5 SLS

Figure 5.33 (continued): Influence of portal frame height on the relative shear displacement of internal sheet connectors under gravity load



(c) Critical internal sheet connector shear displacement

Figure 5.33 (continued): Influence of portal frame height on the relative shear displacement of internal sheet connectors under gravity load

The effect of the height on the shear displacement of the critical external sheet connectors is shown in Figure 5.34. While the shear displacement of the critical external sheet connector remained relatively small, it increased with increasing height with a maximum increase reaching 450% observed at 1.5 SLS when the height was increased by 100% (i.e. from 8 m to 16 m).

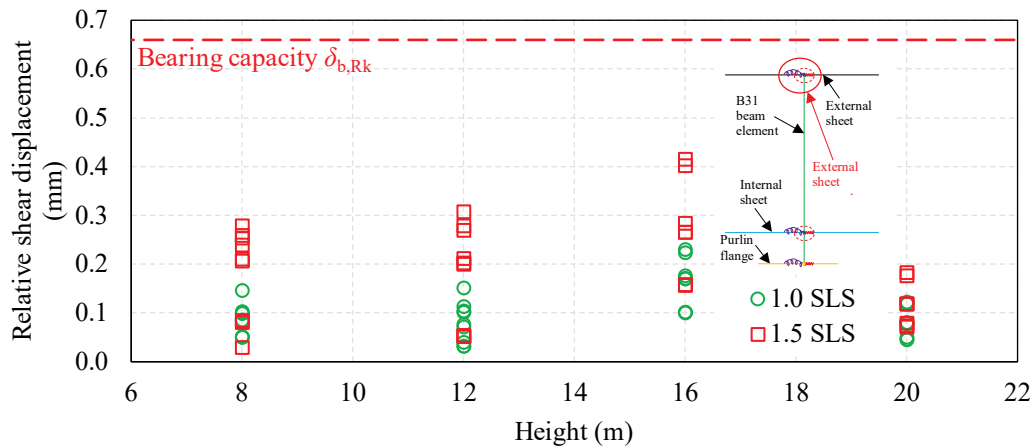


Figure 5.34: Influence of portal frame height on the relative shear displacement of external sheet connectors under gravity load

In conclusion, under gravity load, the maximum shear displacements in screws decrease with increasing the height. The cladding screws at the end bay near the apex experienced the highest demand with the internal sheet connectors being the most affected by the increasing span at displacements reaching almost 6 mm at 1.5 SLS.

Wind load

The influence of the portal frame height under wind sway load on the overall behaviour of the frame and the behaviour of the cladding screws is discussed in this section. As noted before, the proportion of the reaction force carried by the gable was significantly higher compared to the frames subjected to gravity load with no clear isolated effect of the height on the load distribution between the frames.

On the contrary to the effect of span, the maximum shear displacements in the critical screws has significantly increased with the increasing height as shown in Figures 5.36, 5.37, 5.38 and 5.39. At a 160% increase in height (8 m to 20 m), the flashing seam screws and the internal sheet connectors maximum shear displacements increased by 750% and 650%, respectively. The longitudinal seam screws were the most affected with an increase of 3000% when increasing the height of a 30 m span from 8 m to 20 m.

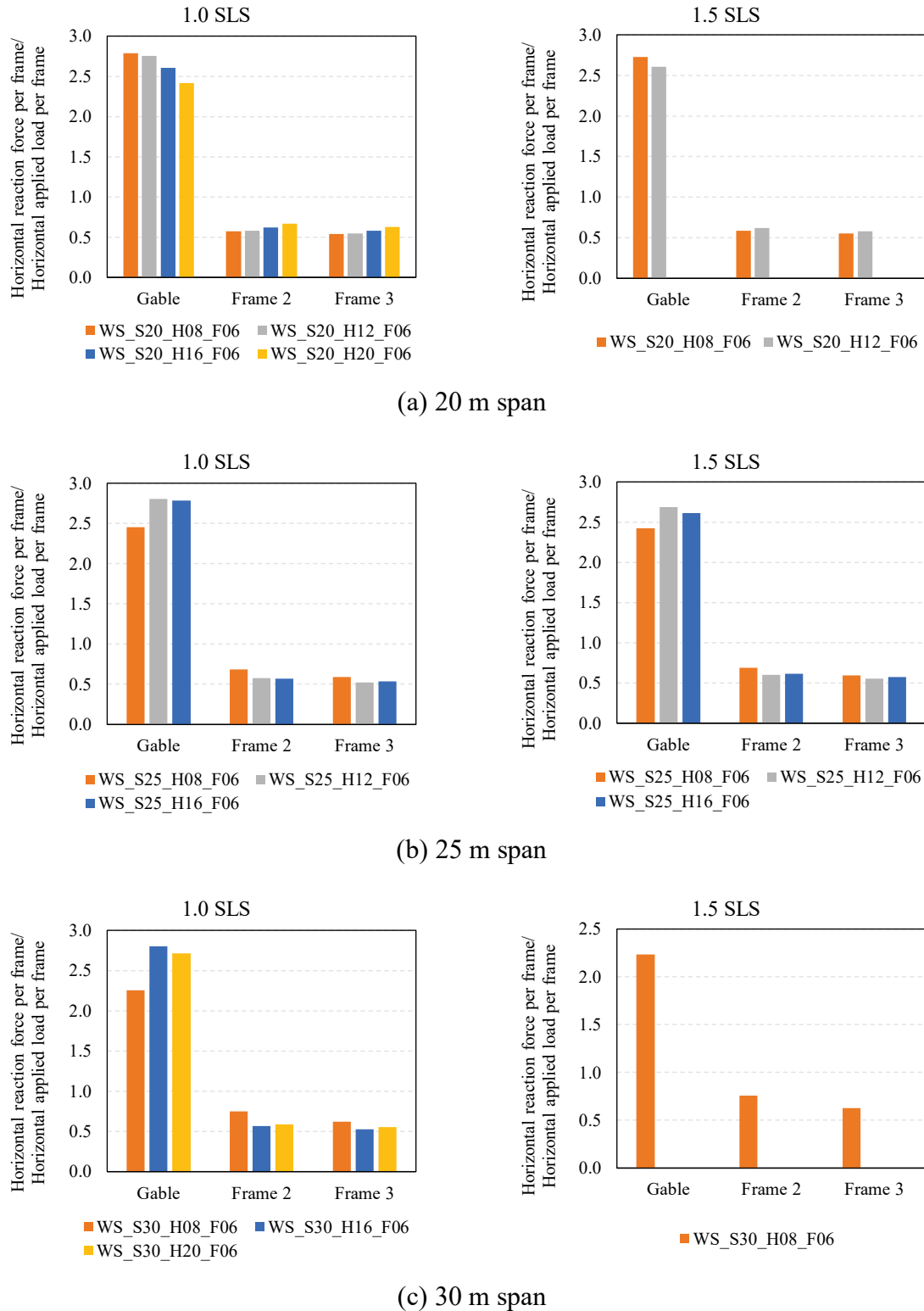


Figure 5.35: Influence of portal frame height on wind load distribution between frames

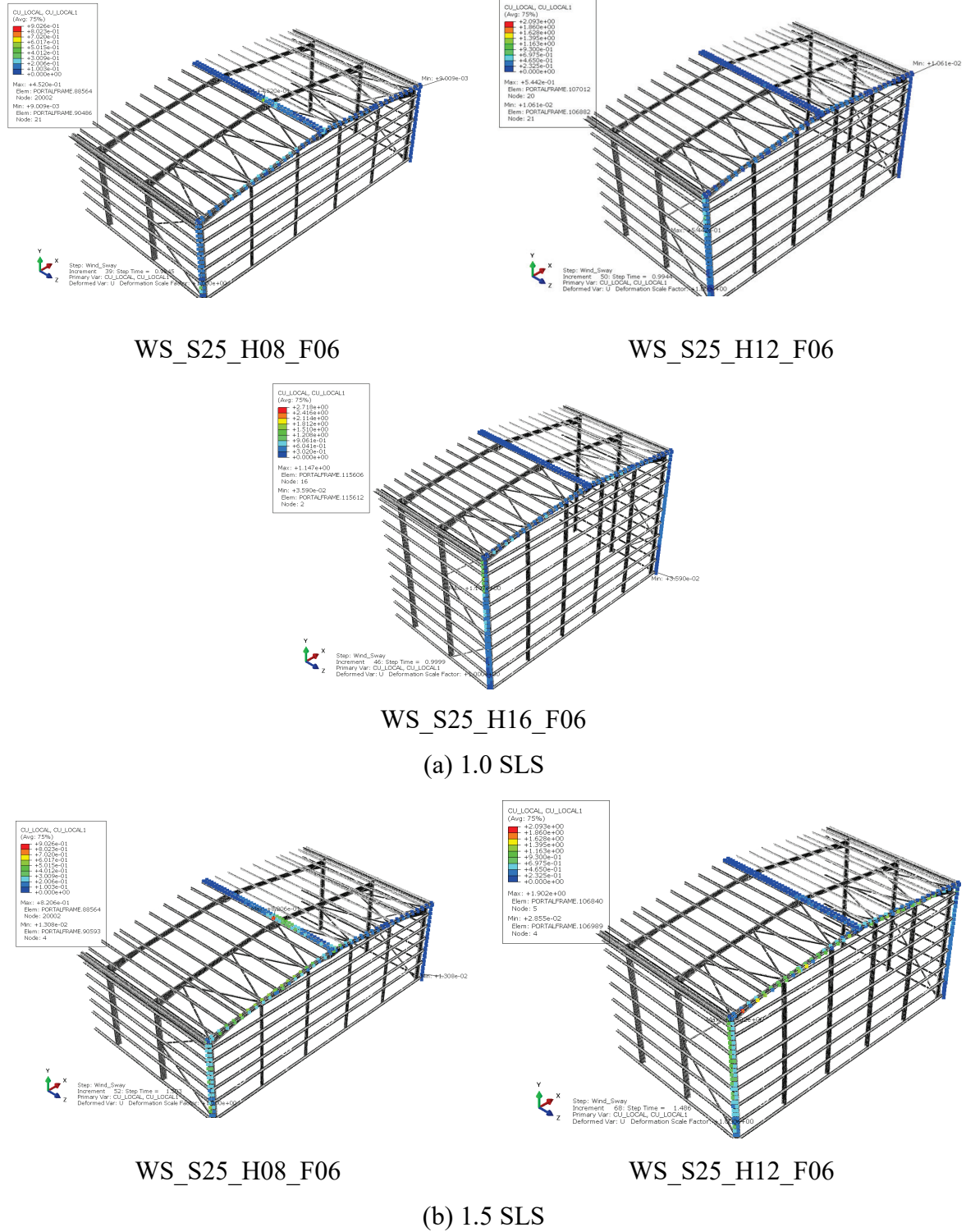
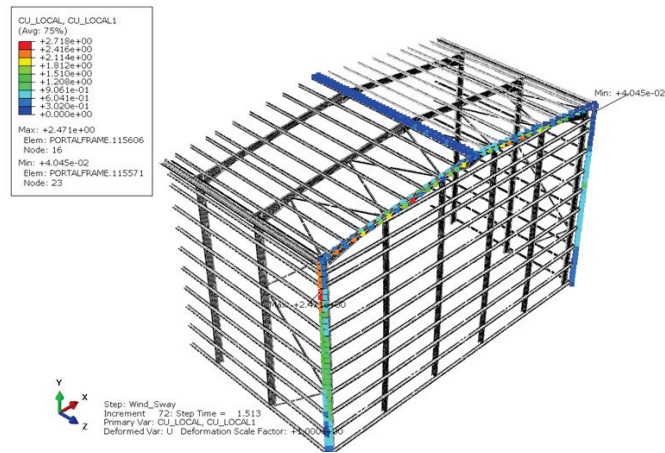
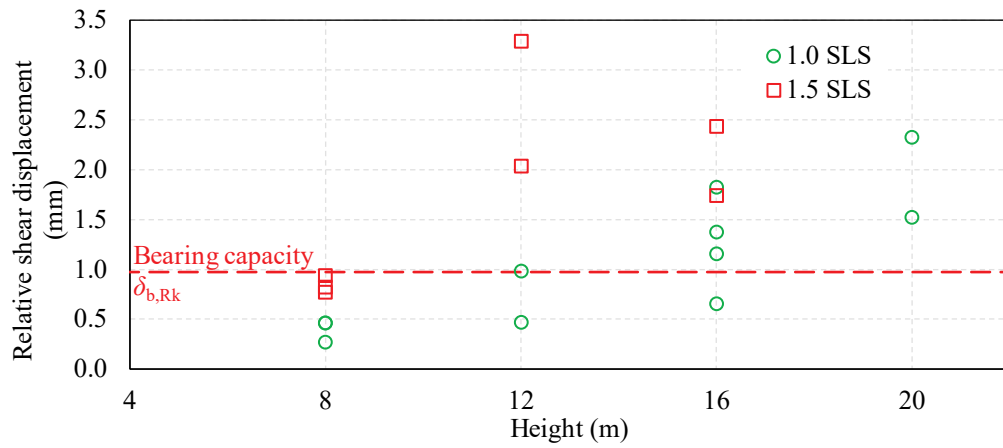


Figure 5.36: Influence of portal frame height on the relative shear displacement of flashing seam screws under wind sway load



WS_S25_H16_F06

(b) 1.5 SLS



(c) Critical flashing seam screw shear displacement

Figure 5.36 (continued): Influence of portal frame height on the relative shear displacement of flashing seam screws under wind sway load

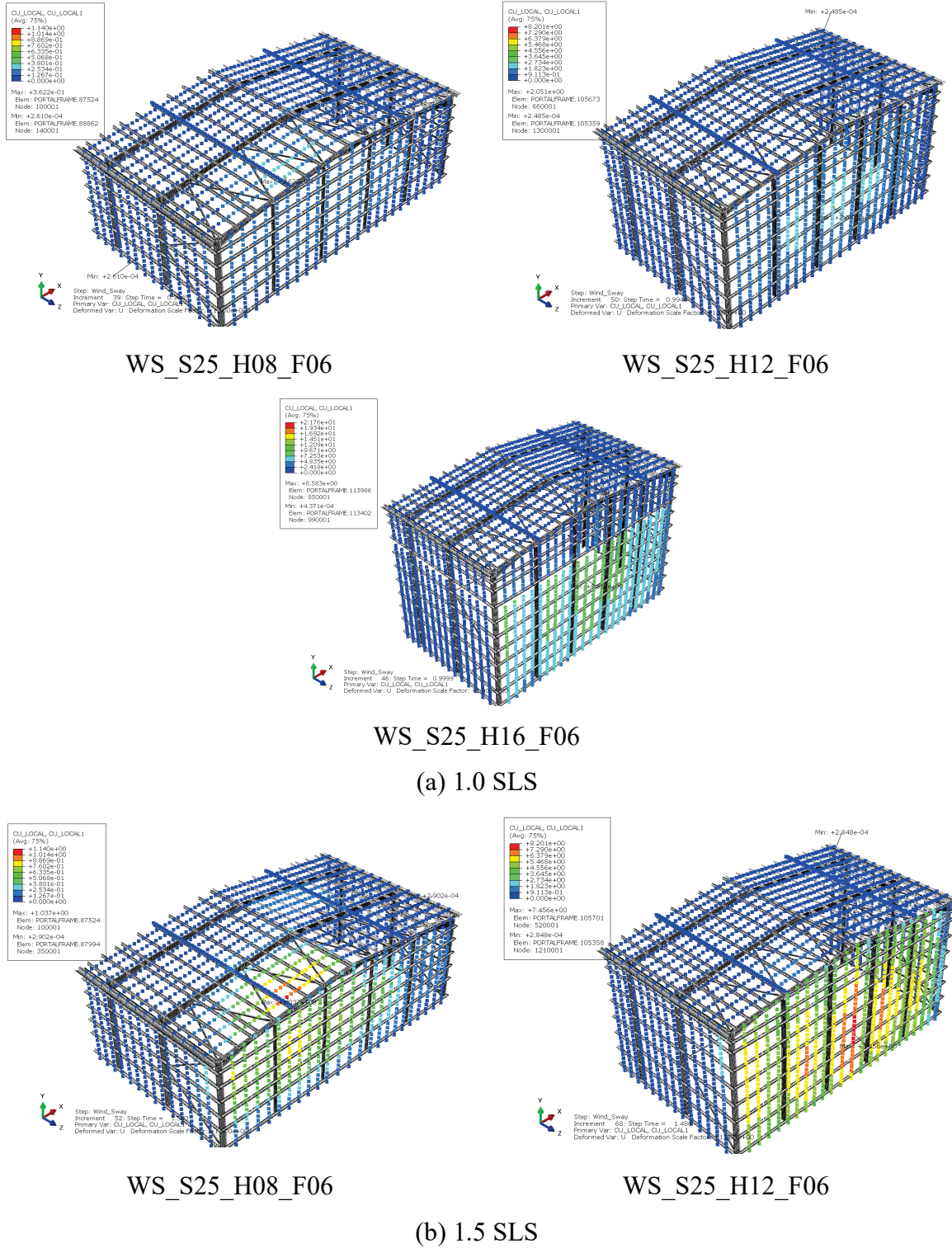
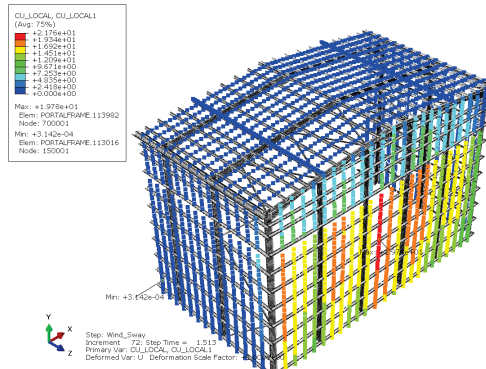
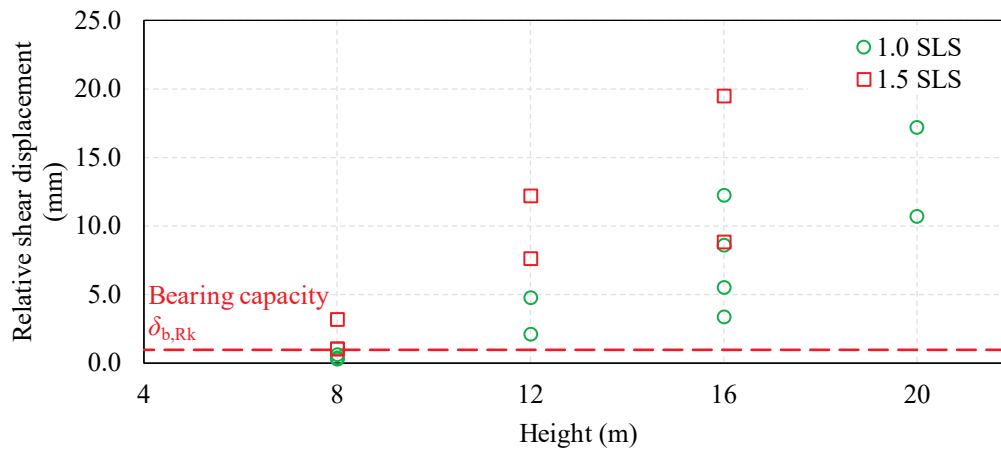


Figure 5.37: Influence of portal frame height on the relative shear displacement of longitudinal and end lap seam screws under wind sway load



WS_S25_H16_F06

(b) 1.5 SLS



(c) Critical longitudinal seam screw shear displacement

Figure 5.37 (continued): Influence of portal frame height on the relative shear displacement of longitudinal and end lap seam screws under wind sway load

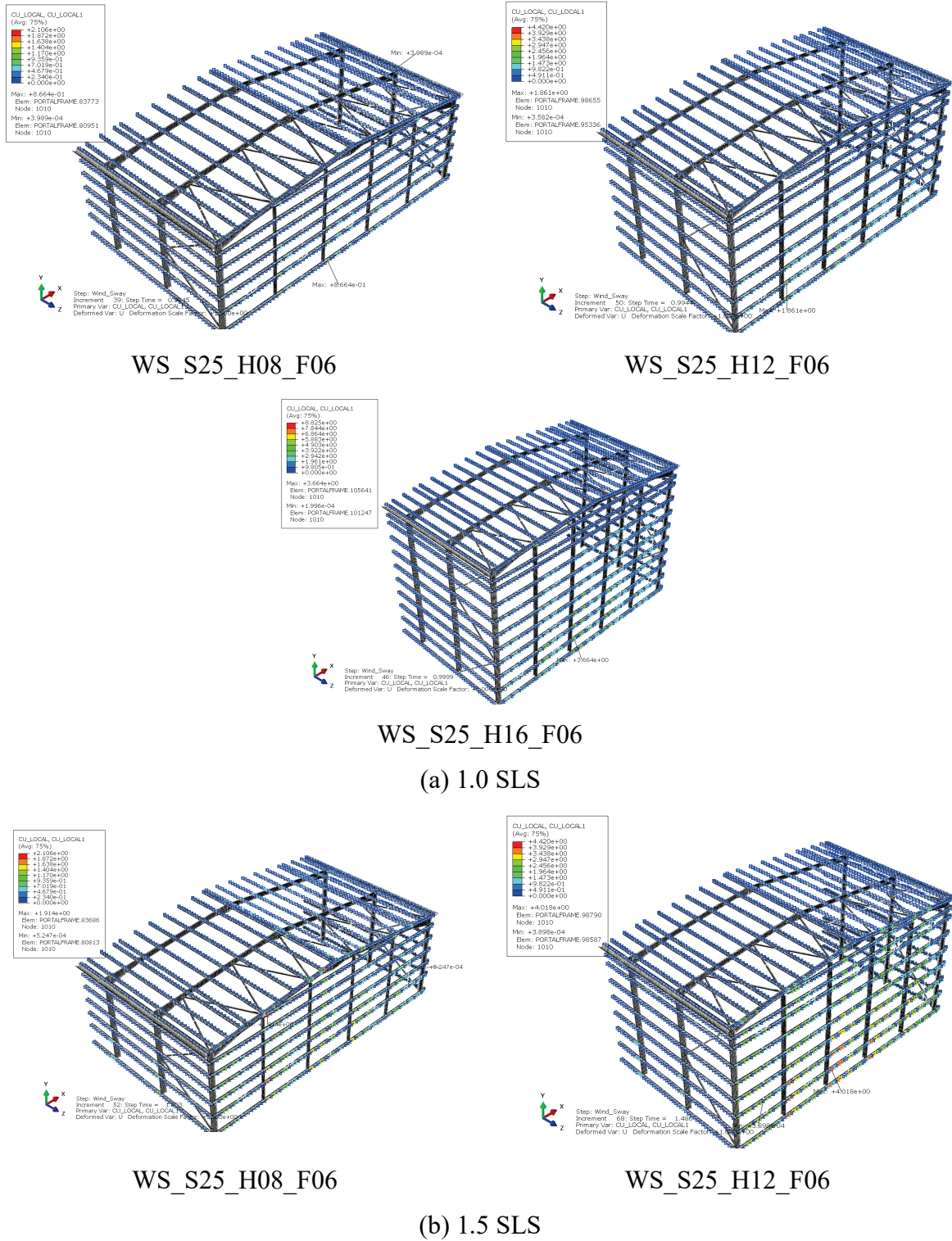


Figure 5.38: Influence of portal frame height on the relative shear displacement of internal sheet connectors under wind sway load

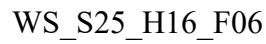


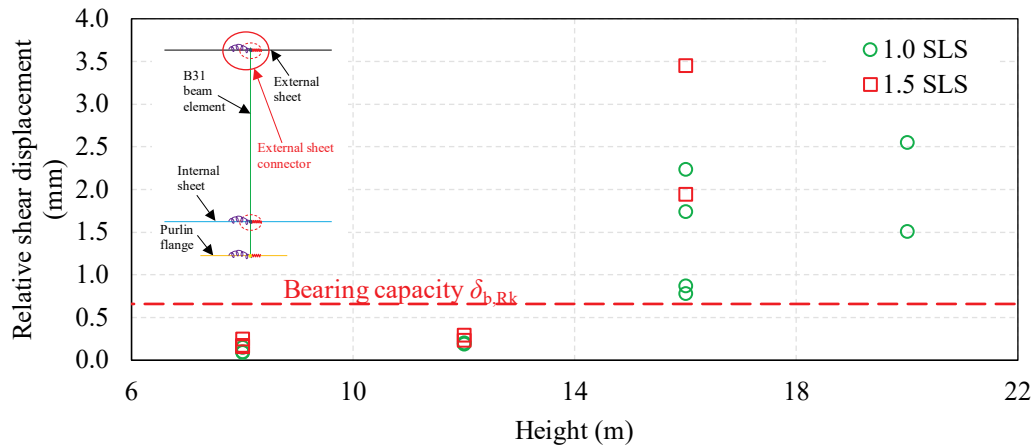
Figure 10 is a scatter plot showing the relative shear displacement (mm) versus height (m) for two service limit states (SLS): 1.0 SLS (green circles) and 1.5 SLS (red squares). The y-axis ranges from 0.0 to 10.0 mm, and the x-axis ranges from 4 to 20 m. A dashed red line at approximately 0.5 mm is labeled 'Bearing capacity' and $\delta_{b,Rk}$. An inset diagram shows a cross-section of the wall with labels: 'B31 beam element', 'External sheet', 'Internal sheet', and 'Purkin flange'. A red circle highlights the 'Internal sheet connector'.

Height (m)	1.0 SLS (mm)	1.5 SLS (mm)
8	0.5, 1.0, 1.5	2.0, 2.5, 2.8
12	1.8, 2.5	4.1, 5.3
16	1.8, 3.6, 4.1, 6.1	4.1, 7.9
20	4.5, 5.9	-

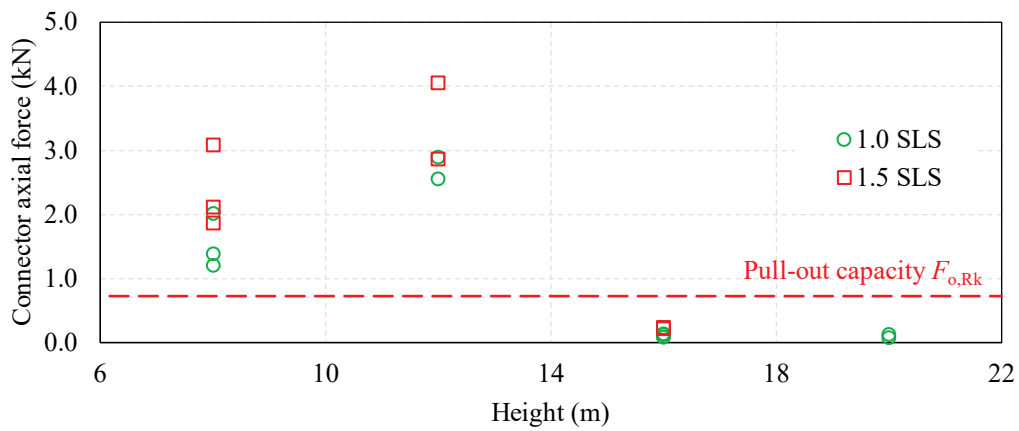
Figure 10 is a scatter plot showing the Connector axial force (kN) on the Y-axis (ranging from 0.0 to 0.7) versus Height (m) on the X-axis (ranging from 4 to 20). The plot compares the axial force for two service limit states (SLS): 1.0 SLS (represented by green circles) and 1.5 SLS (represented by red squares). A horizontal dashed red line indicates the Pull-out capacity $F_{o,Rk}$ at approximately 0.58 kN.

Height (m)	1.0 SLS (kN)	1.5 SLS (kN)
8	0.11	0.07
8	0.53	0.58
12	-	0.58
16	0.24	0.49
16	0.59	0.59
20	0.58	-

Figure 5.38 (continued): Influence of portal frame height on the relative shear displacement of internal sheet connectors under wind sway load



(a) Critical external sheet connector shear displacement

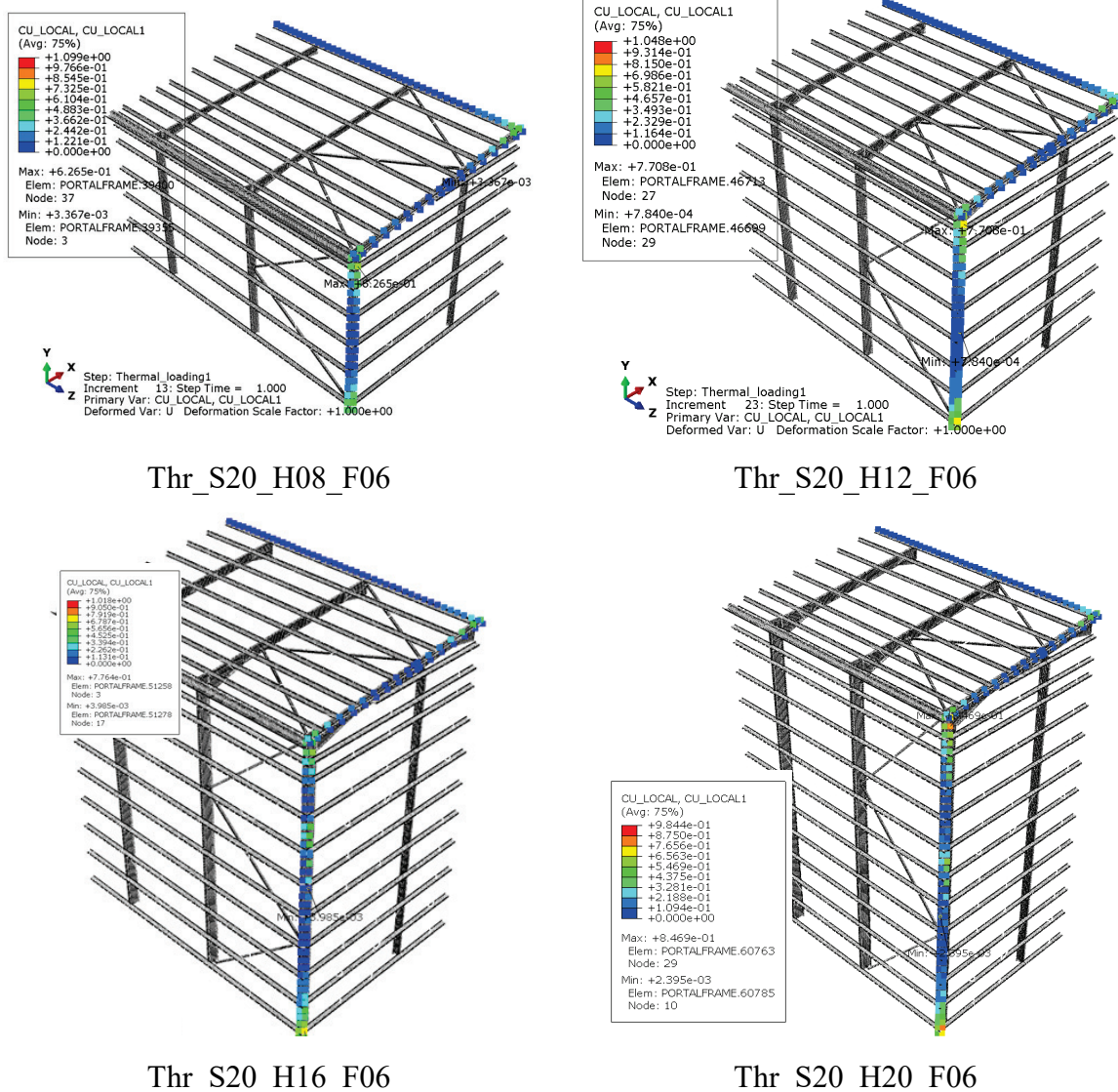


(b) Critical external sheet connector axial force

Figure 5.39: Influence of portal frame height on the relative shear displacement of external sheet connectors under wind sway load

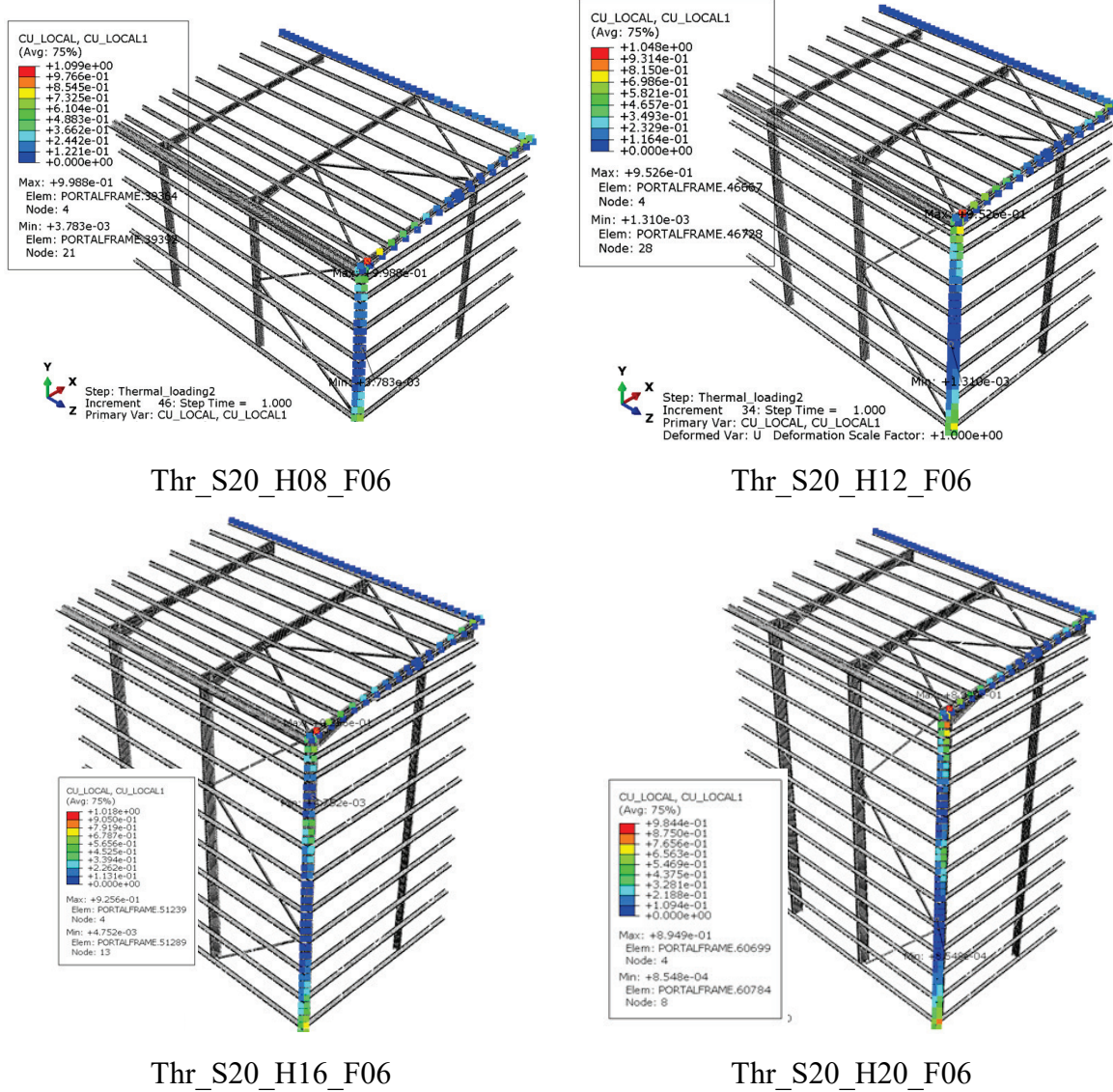
Thermal actions

The influence of the height on the overall behaviour of the frame and the behaviour of the cladding screws under thermal action is discussed in this section. As previously mentioned, the location of the critical screws due to thermal action was affected by the dimensions of the frame. With the increasing height, the critical connector behaviour propagated towards the wall and gable cladding as shown in Figures 5.40, 5.41 and 5.42. Whereas the increase in height did not have a significant effect on the maximum shear displacement of the majority of critical screws, it had a substantial effect on the shear displacement of the external sheet connectors with an increase of up to 2000% when increasing the height from 8 m to 20 m, see Figure 5.43.

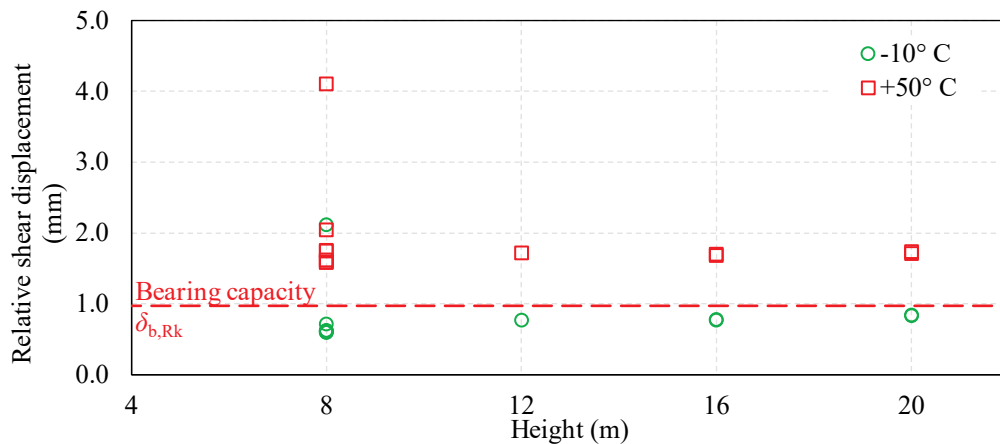


(a) -10° C

Figure 5.40: Influence of portal frame height on the relative shear displacement of flashing seam screws under thermal action

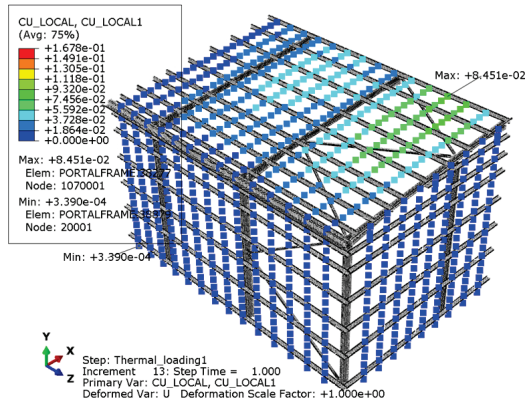


(b) +50° C

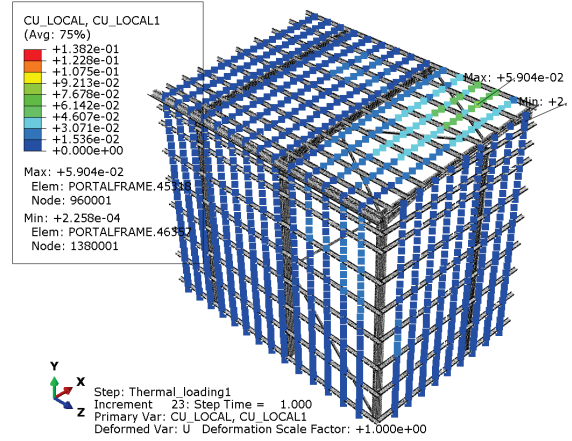


(c) Critical flashing seam screw shear displacement

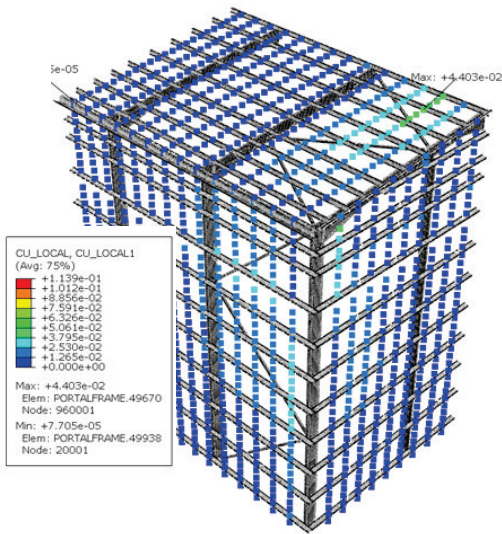
Figure 5.40 (continued): Influence of portal frame height on the relative shear displacement of flashing seam screws under thermal action



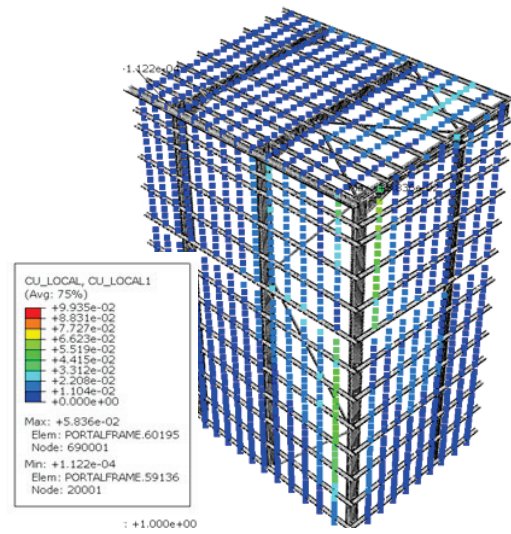
Thr_S20_H08_F06



Thr_S20_H12_F06

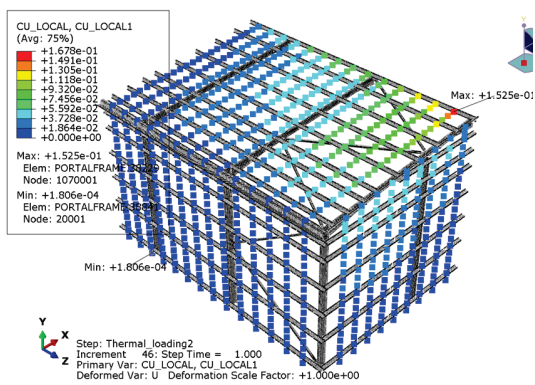


Thr_S20_H16_F06

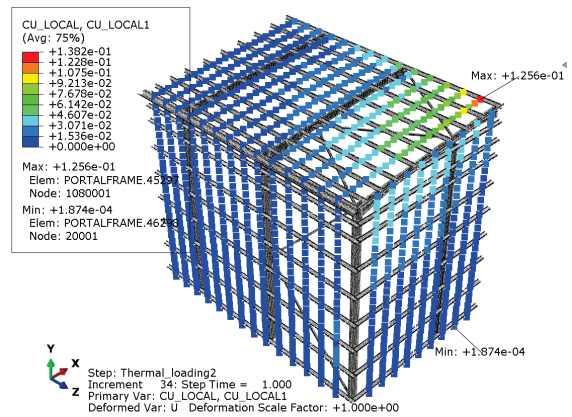


Thr_S20_H20_F06

(a) -10° C



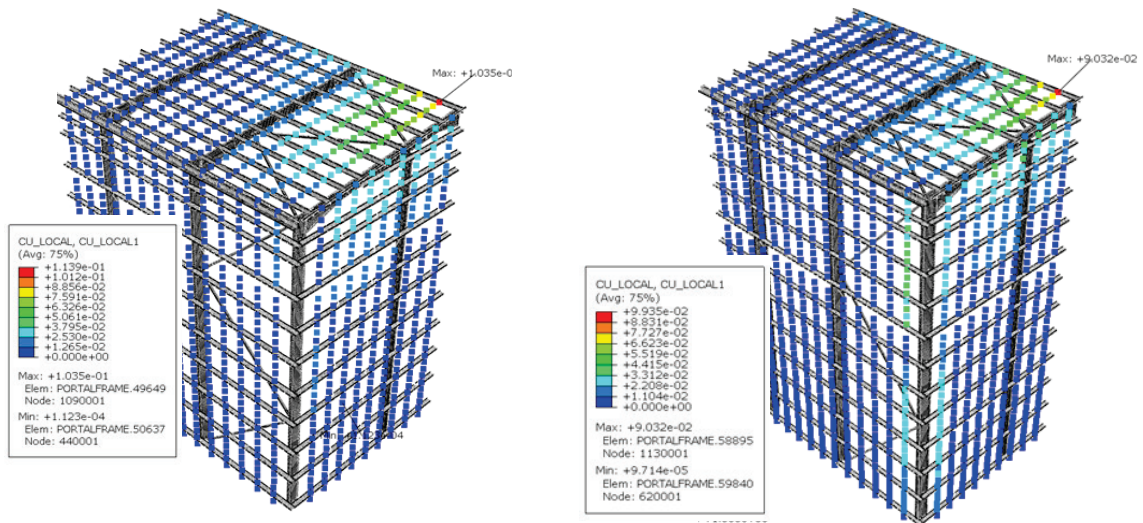
Thr_S20_H08_F06



Thr_S20_H12_F06

(b) +50° C

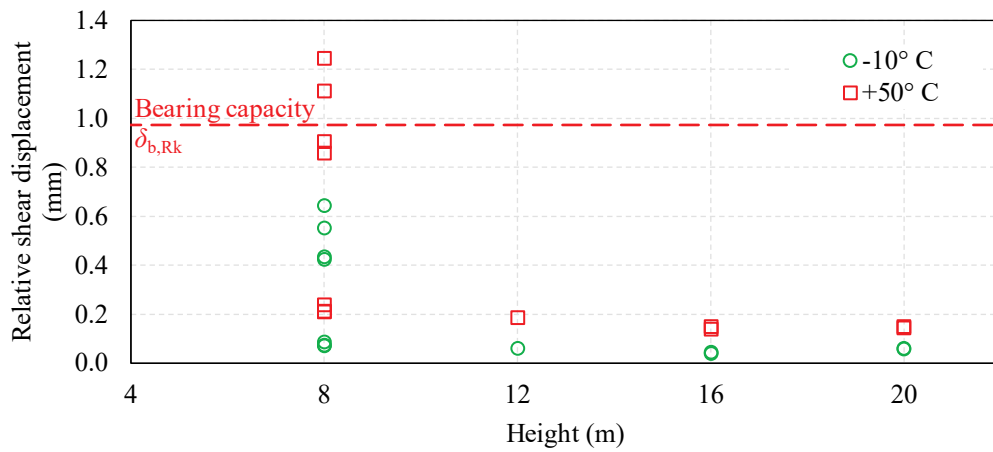
Figure 5.41: Influence of portal frame height on the relative shear displacement of longitudinal and end lap seam screws under thermal action



Thr_S20_H16_F06

Thr_S20_H20_F06

(b) +50° C



(c) Critical longitudinal and end lap seam screw shear displacement

Figure 5.41 (continued): Influence of portal frame height on the relative shear displacement of longitudinal and end lap seam screws under thermal action

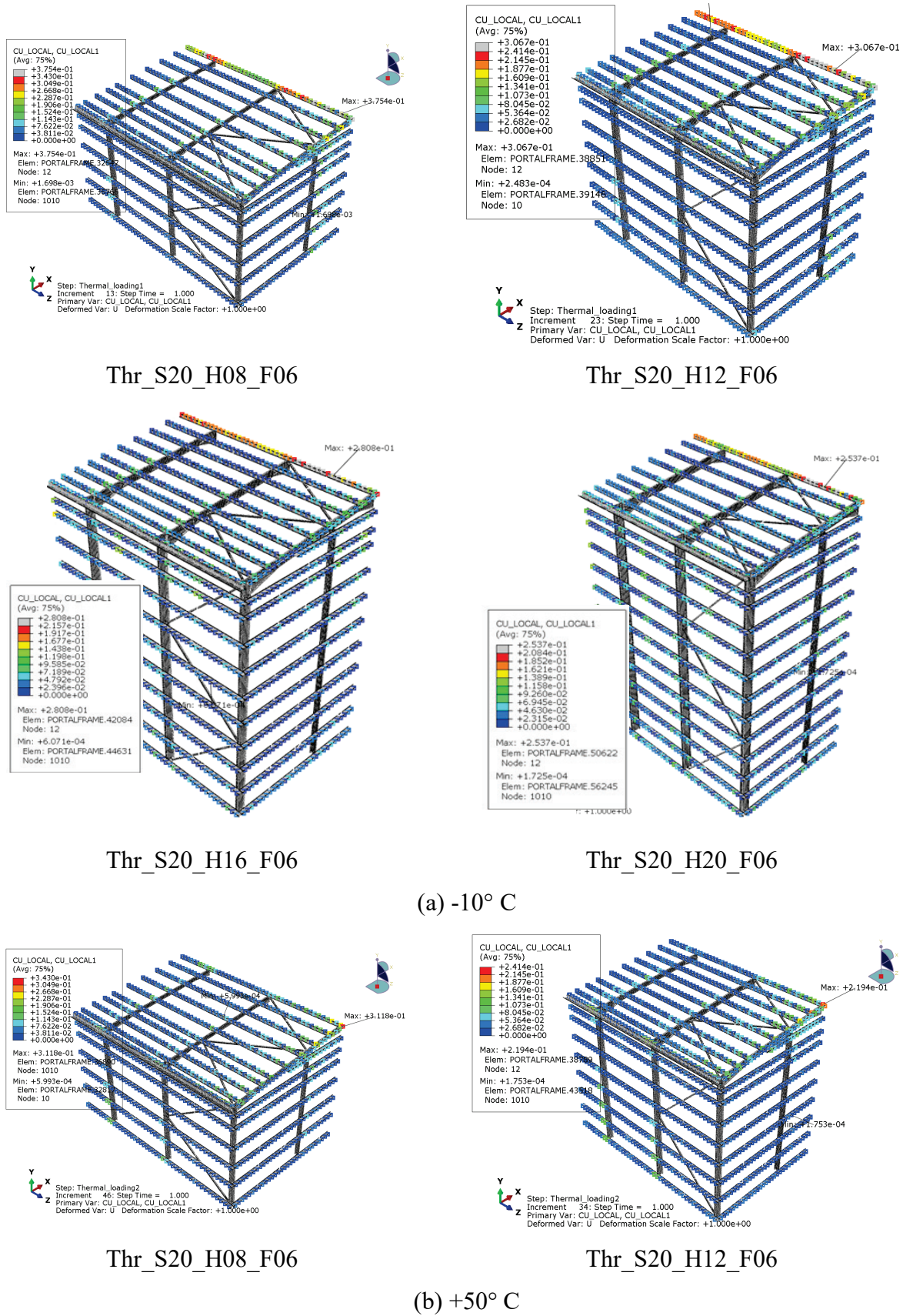


Figure 5.42: Influence of portal frame height on the relative shear displacement of internal sheet connectors under thermal action

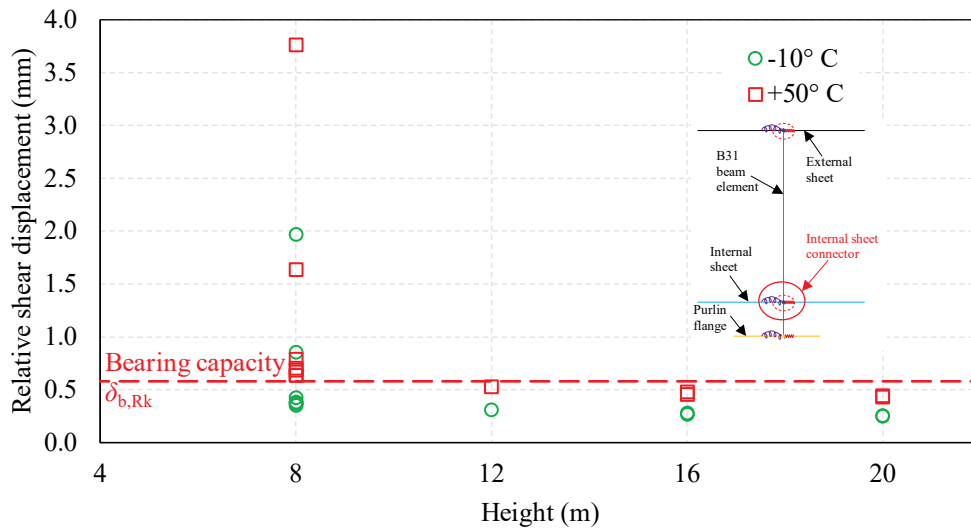
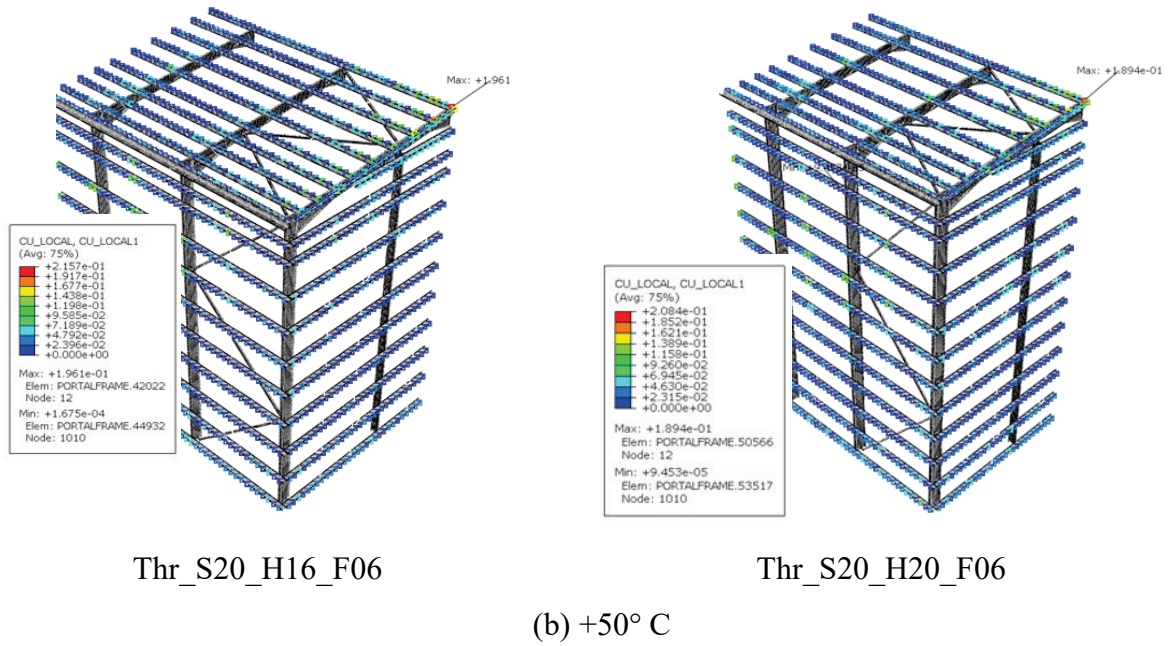


Figure 5.42 (continued): Influence of portal frame height on the relative shear displacement of internal sheet connectors under thermal action

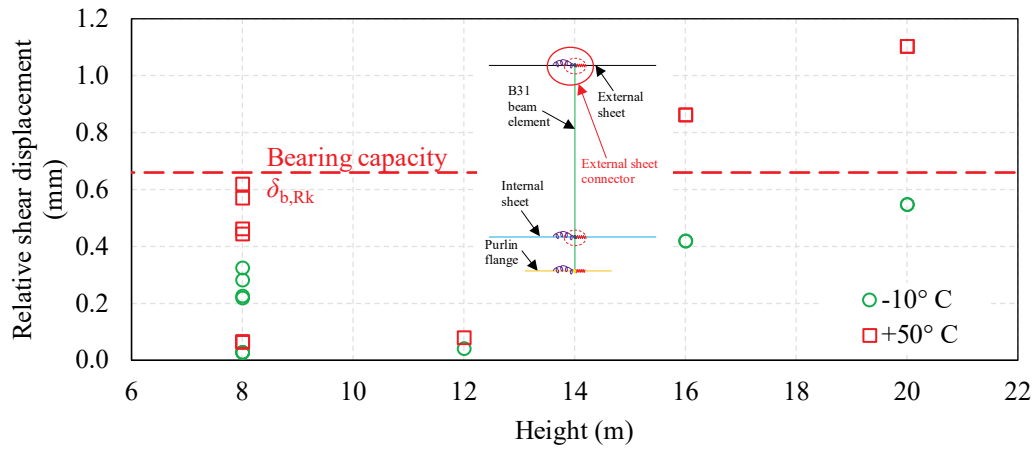


Figure 5.43: Influence of portal frame height on the relative shear displacement of external sheet connectors under thermal action

5.4.3 Effect of length

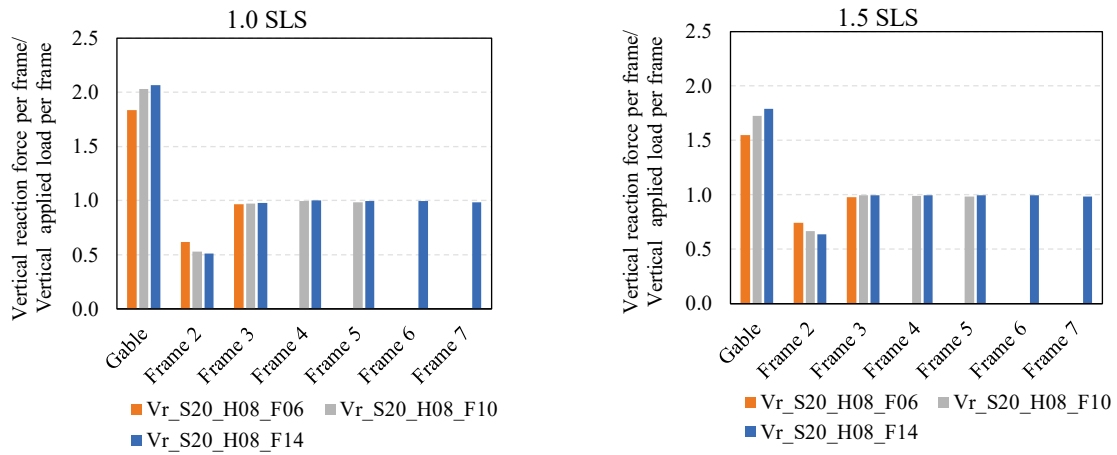
Gravity load

The influence of the structure length under gravity load on the overall behaviour of the frame and the behaviour of the cladding screws is discussed in this section. The effect of length on the load distribution between frames is shown in Figure 5.44. A 10-20% increase in the ratio of the reaction force carried by the gable frame compared to the expected applied load was observed with the 160% increase in length (i.e. 30 m to 78 m).

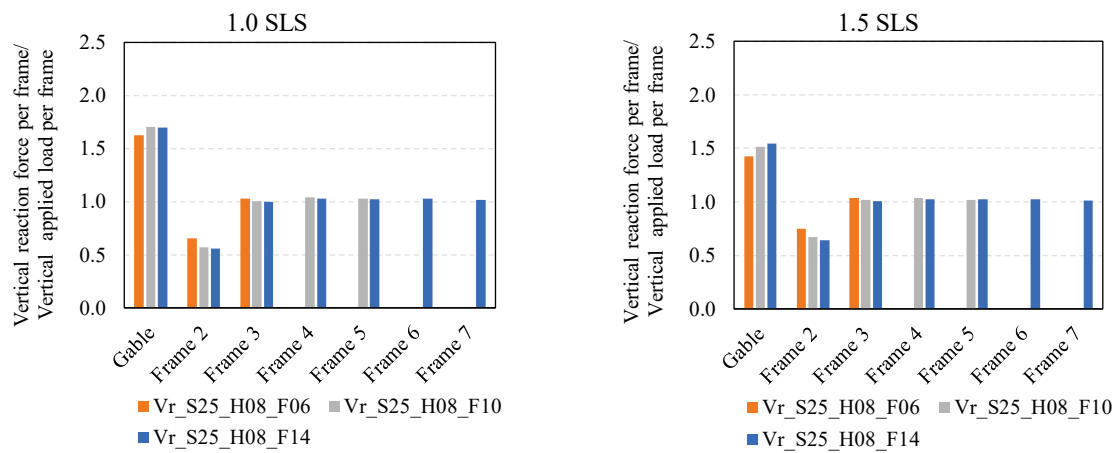
The effect of length on the flashing seam screws is shown in Figure 5.45. It can be clearly observed that change in structure length had no significant effect on the shear displacement of the critical flashing seam screw, as shown in Figure 5.45c. The maximum change observed was a 10% reduction in the shear displacement of the critical flashing seam screw associated with a 160% increase in structure length.

In Figure 5.46, the effect of length on the longitudinal and end lap seam screws is shown. Again, the change in structure length only had a marginal effect on the shear displacement of the critical longitudinal seam screw, as shown in Figure 5.46c. The maximum change observed was a 35% reduction in the shear displacement of the critical longitudinal seam screw associated with a 160% increase in structure length.

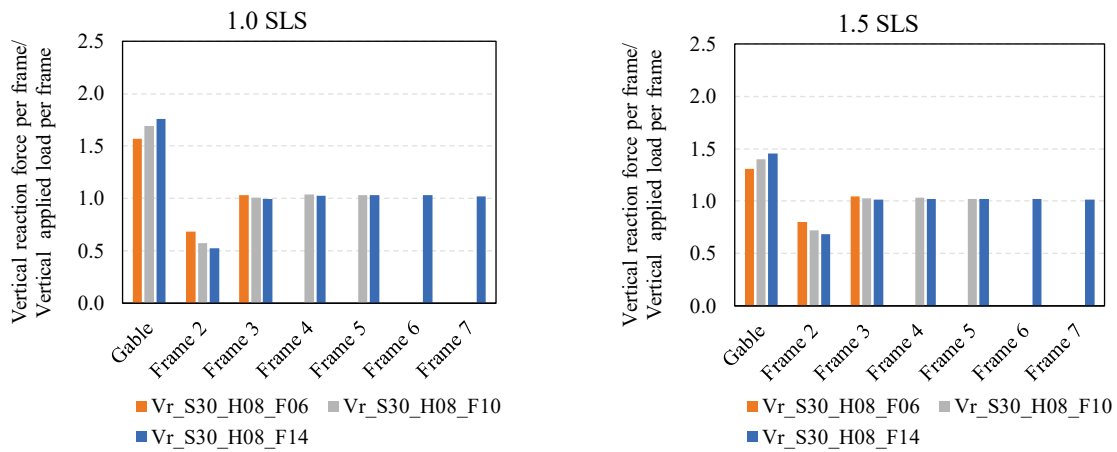
The effect of length on the internal sheet connectors is shown in Figure 5.47. The change in structure length had no significant effect on the shear displacement of the critical the internal sheet connectors as shown in Figure 5.47c. The maximum change observed was a 15% increase in the shear displacement of the critical the internal sheet connector associated with a 160% increase in structure length. The effect of length on the external sheet connectors is shown in Figure 5.48. The change in structure length had a minor effect on the shear displacement of the critical the external sheet connectors as shown in Figure 5.48c. The maximum change observed was a 65% reduction in the shear displacement of the critical the external sheet connector associated with a 160% increase in structure length. Therefore, the effect of length was not examined under wind sway load and thermal action cases.



(a) 20 m span, 8 m height models



(b) 25 m span, 8 m height models



(c) 30 m span, 8 m height models

Figure 5.44: Influence of portal frame structure length on gravity load distribution between frames under gravity load

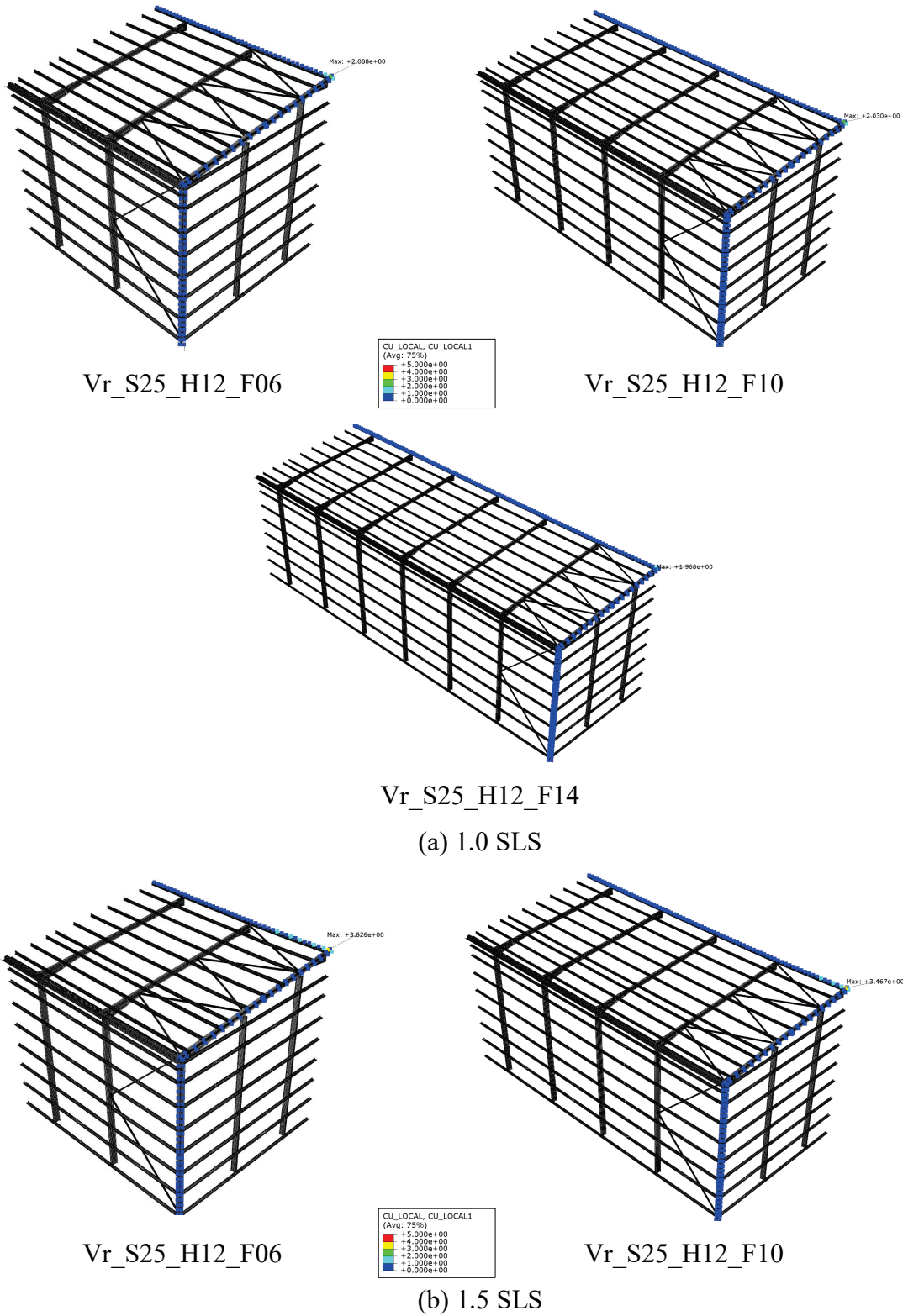
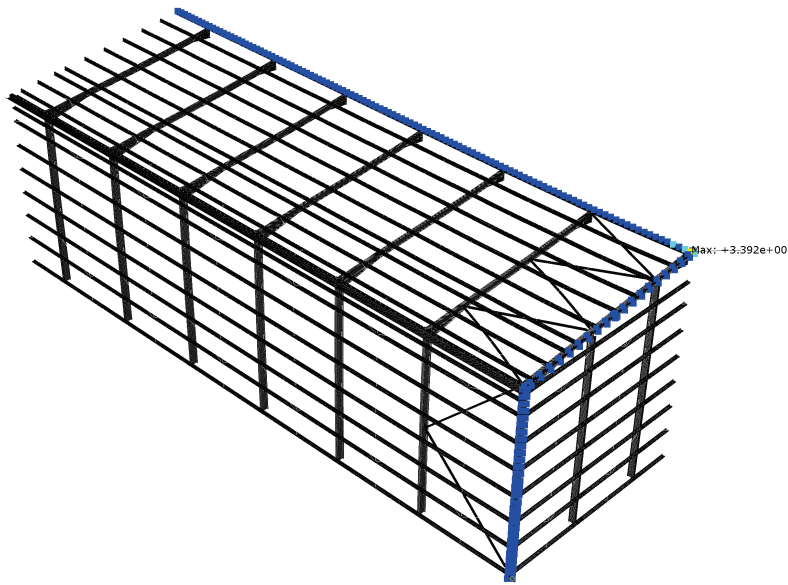
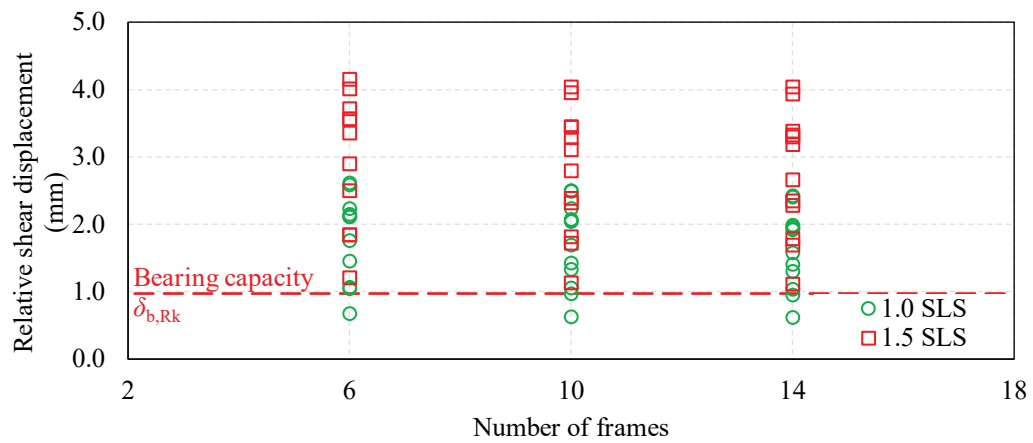


Figure 5.45: Influence of portal frame structure length on the relative shear displacement of flashing seam screws under gravity load



Vr_S25_H12_F14

(b) 1.5 SLS



(c) Critical flashing seam screw shear displacement

Figure 5.45(continued): Influence of portal frame structure length on the relative shear displacement of flashing seam screws under gravity load

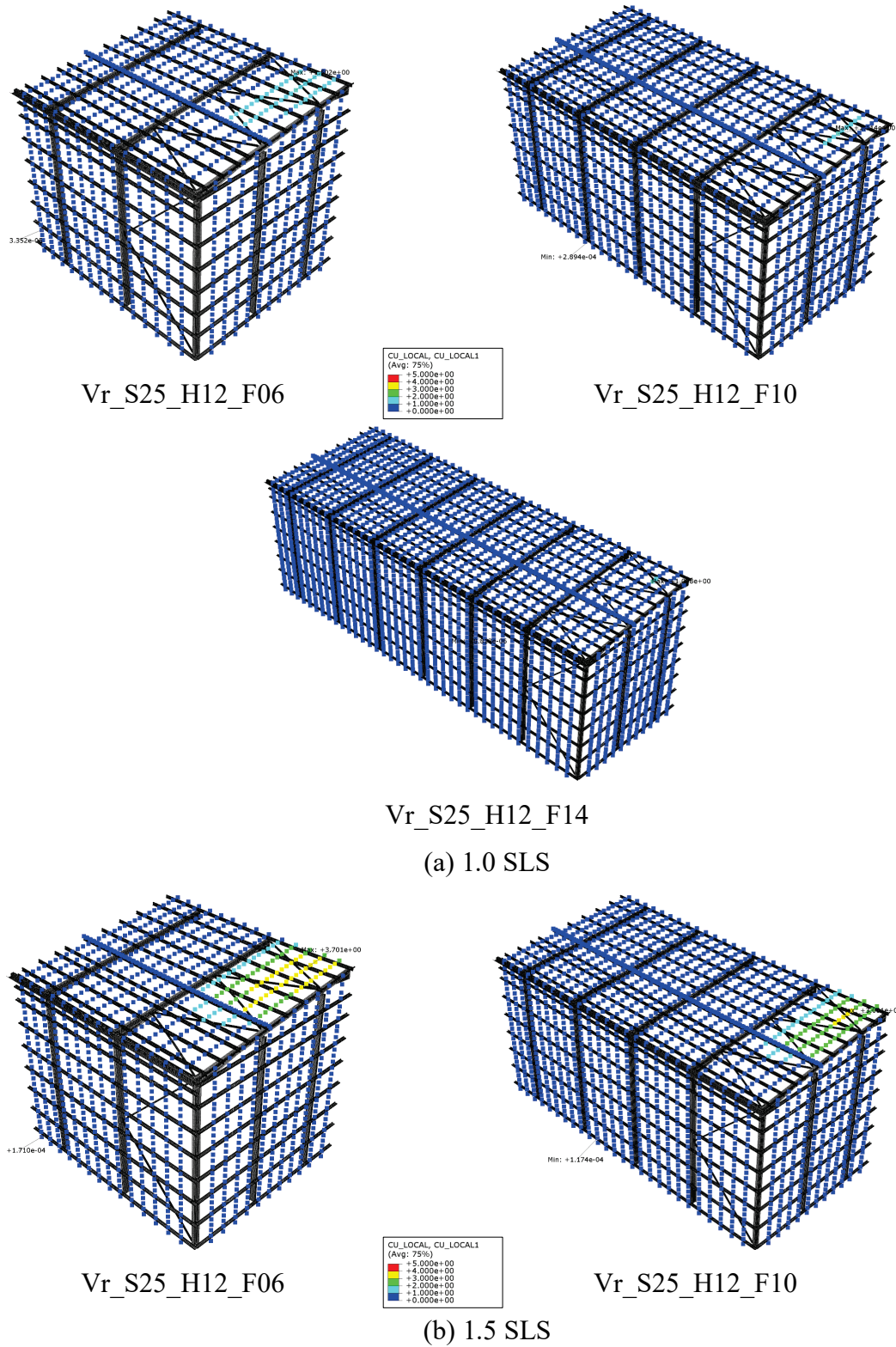
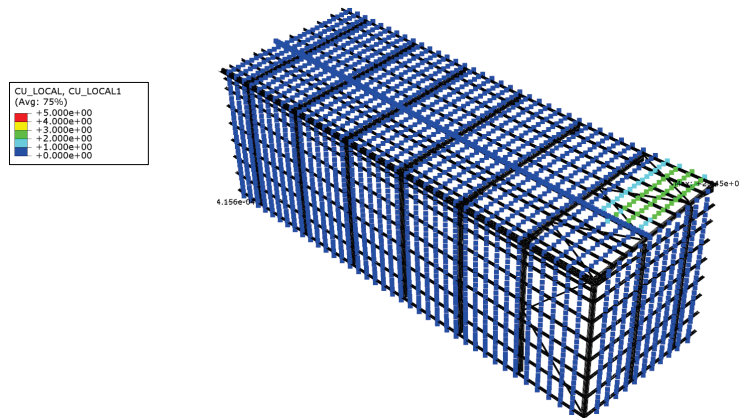
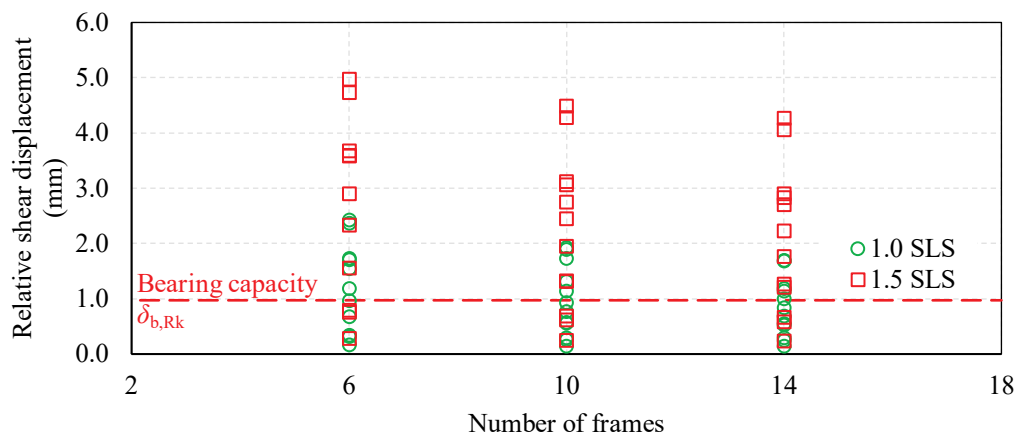


Figure 5.46: Influence of portal frame structure length on the relative shear displacement of longitudinal and end lap seam screws under gravity load



(b) 1.5 SLS



(c) Critical longitudinal and end lap seam screw shear displacement

Figure 5.46 (continued): Influence of portal frame structure length on the relative shear displacement of longitudinal and end lap seam screws under gravity load

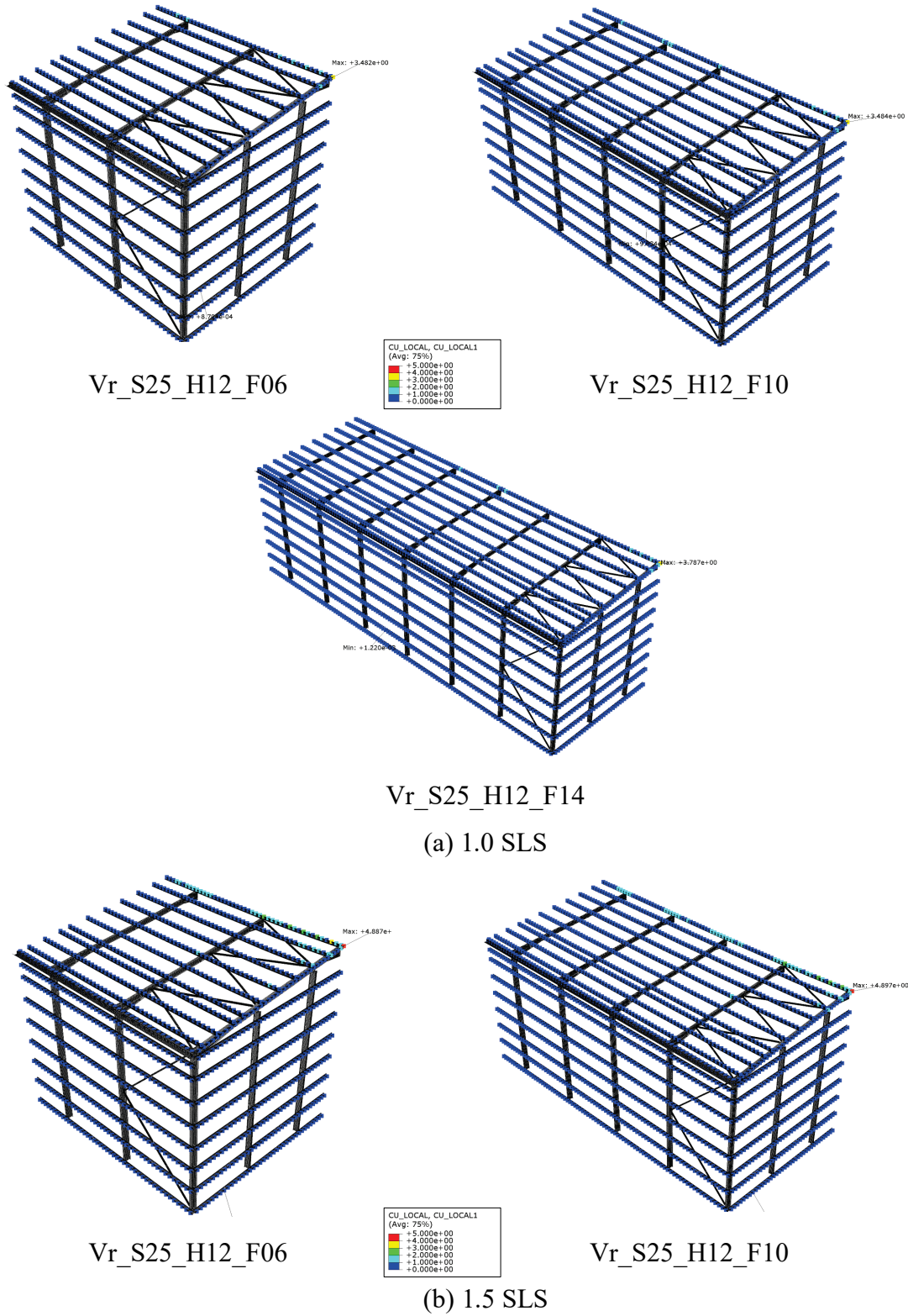


Figure 5.47: Influence of portal frame structure length on the relative shear displacement of internal sheet connectors under gravity load

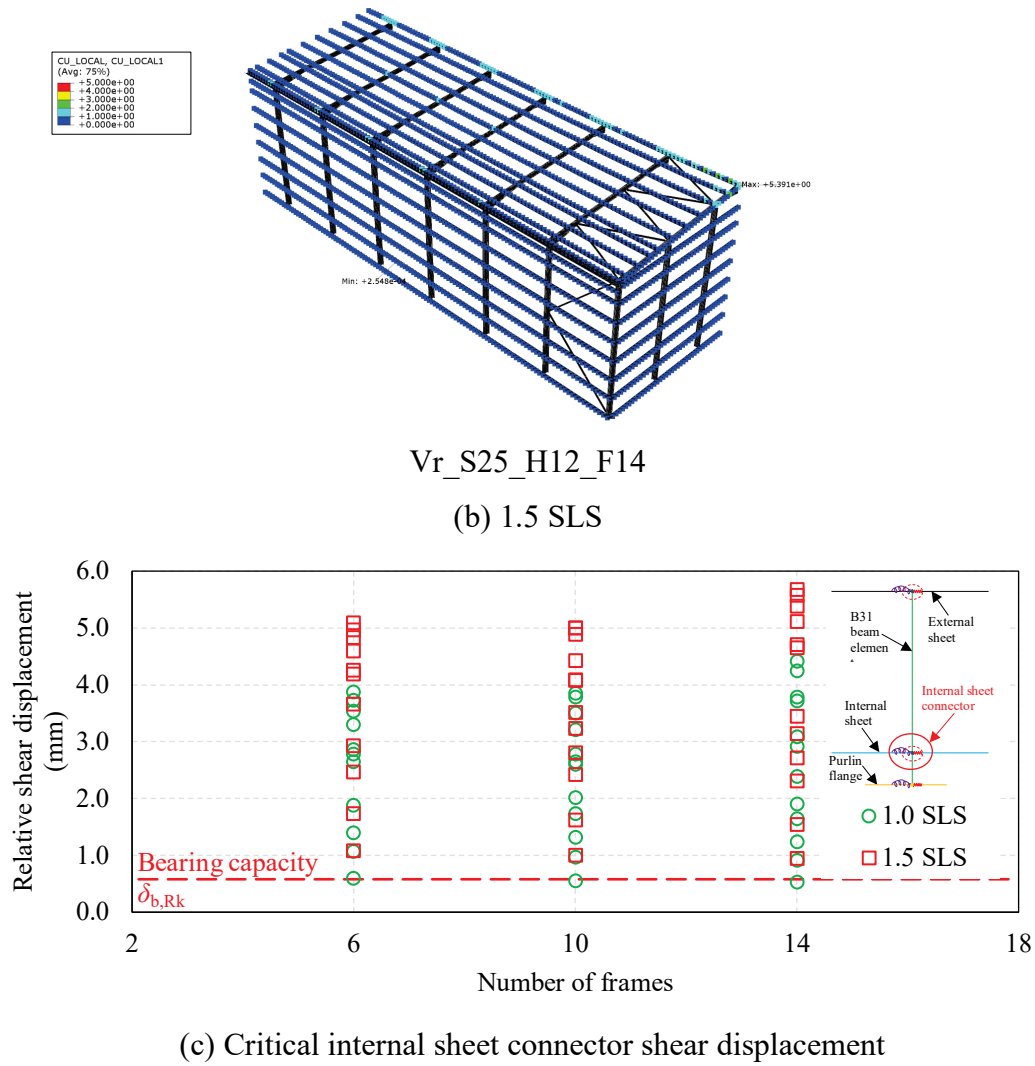


Figure 5.47 (continued): Influence of portal frame structure length on the relative shear displacement of internal sheet connectors under gravity load

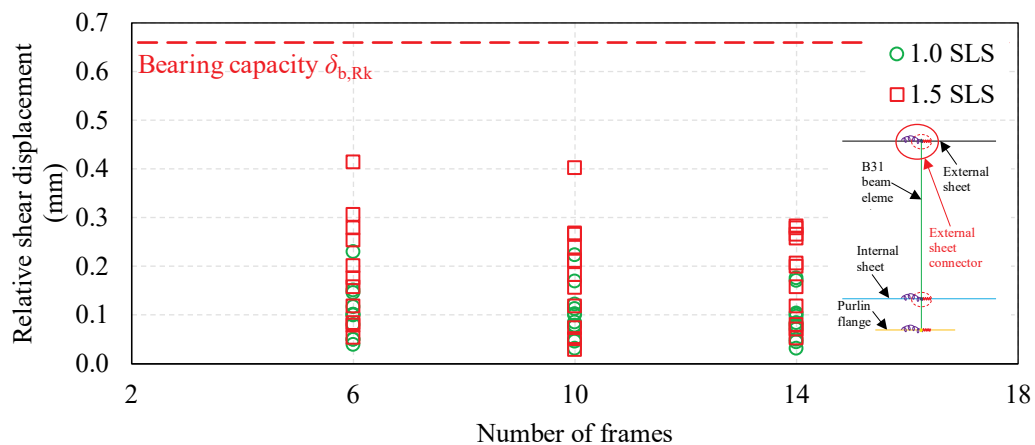
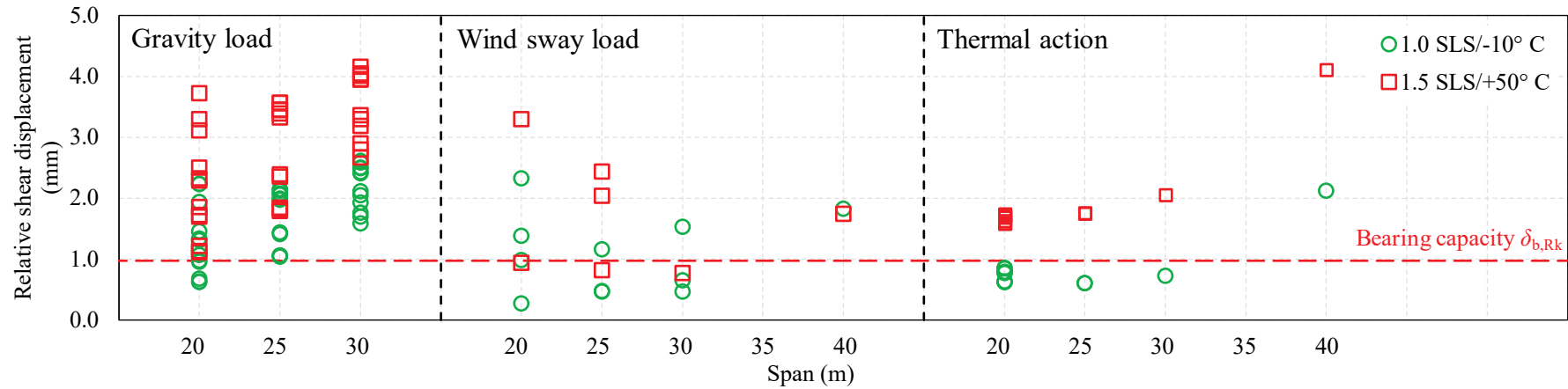


Figure 5.48: Influence of portal frame structure length on the relative shear displacement of external sheet connectors under gravity load

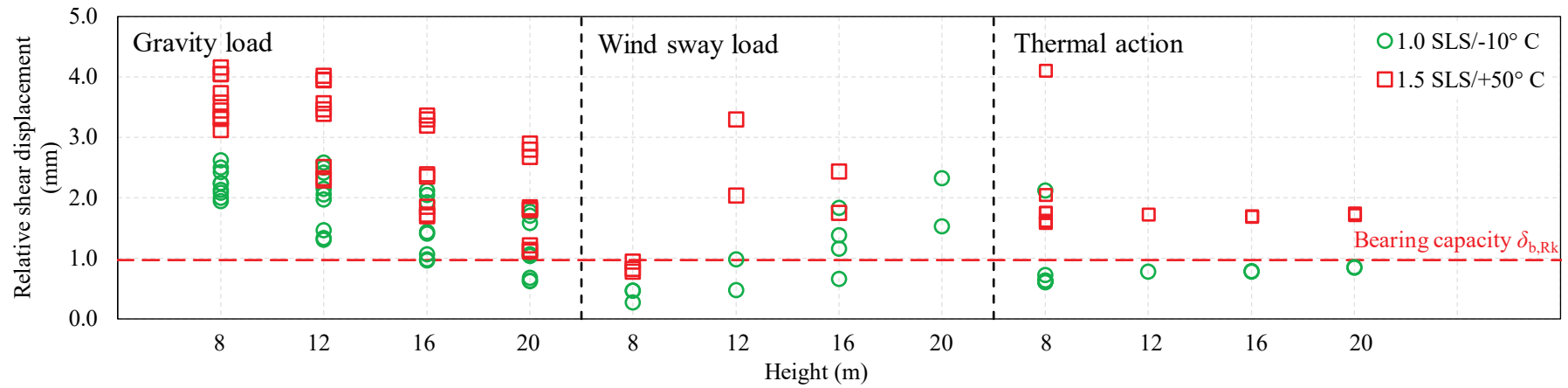
5.4.4 Discussion and summary

It was noted that under gravity loading conditions, the middle frames, except for the penultimate frame, effectively supported the expected load. On the other hand, the gable frames resisted approximately 130-180% of the expected load, which was transferred from the penultimate frames that carried 50-80% of the expected applied load. However, under wind sway loading conditions, a higher proportion of load was transferred to the gable frame, reaching up to 280%, with all internal frames, including the penultimate frame, carrying just above half of the expected load. In both loading cases, the gable frames attracted more load due to its high relative stiffness compared to the internal frames, therefore causing more demand on the screws in the end bays and in the gable wall. This overall behaviour aligns with the findings of Roberts, Davies & Wang (2021). However, Roberts, Davies & Wang (2021) noted that under horizontal load, the gable frames carried a larger proportion of the load compared to the frames under wind sway load in this investigation.

A summary for the influence of scale on cladding connectors is shown in Figures 5.49, 5.50, 5.51 and 5.52. Notably, even at serviceability load level (SLS), a significant number of screws are considered to have reached its bearing capacity. It should be noted though that this does not constitute a ‘failure’, with a corresponding drop in capacity, but to high levels of bearing deformation into the surrounding sheet material. In practice, this would be observed as elongated holes (slots) around the screw location. As the span increased, a greater number of connectors reached their bearing capacity, both under gravity loads and thermal actions. However, an increase in height exhibited a more critical influence in wind sway load scenarios. Generally, wind sway load cases were considerably more detrimental to the cladding connectors compared to other load cases and therefore can be considered the most critical load case. The substantial shear displacements observed in screws under wind sway loading cases raises a cause for concern. It is crucial to consider the adverse effect of stressed skin action under wind sway load case in the design of steel portal frames. Further investigation is essential to propose mitigation strategies to prevent possible damage to the structure.

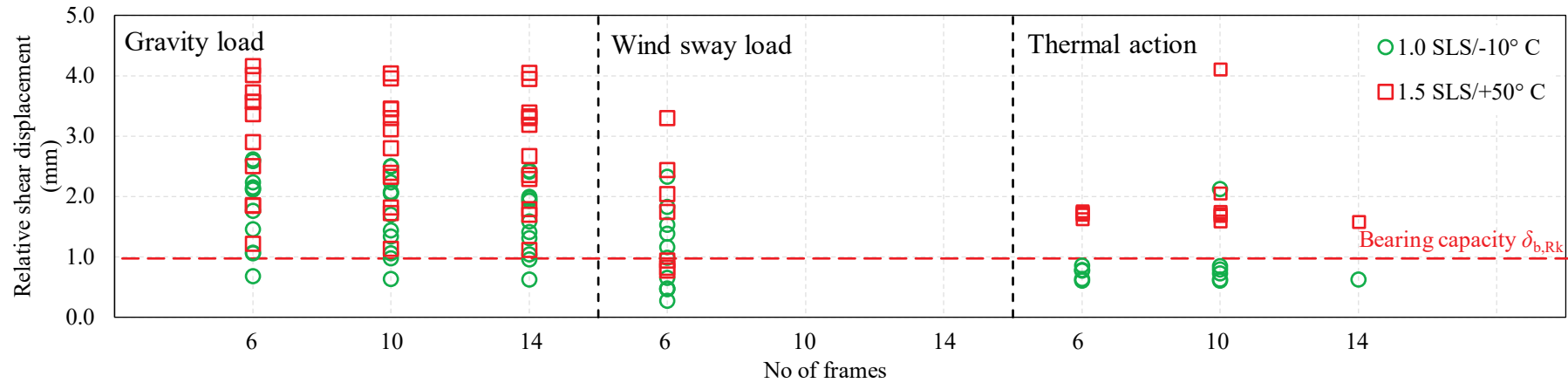


(a) Effect of span



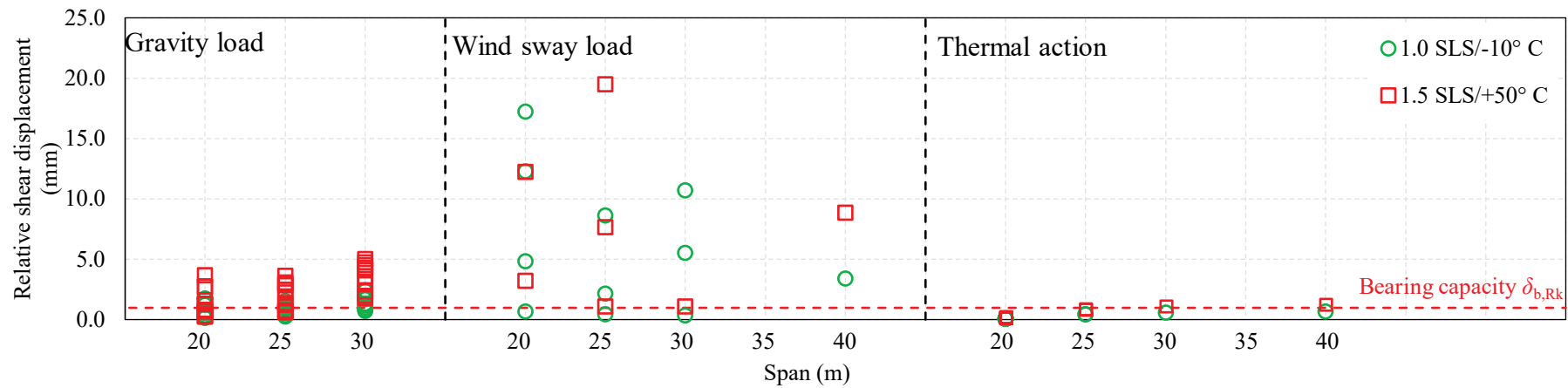
(b) Effect of height

Figure 5.49: Influence of scale on flashing seam screws



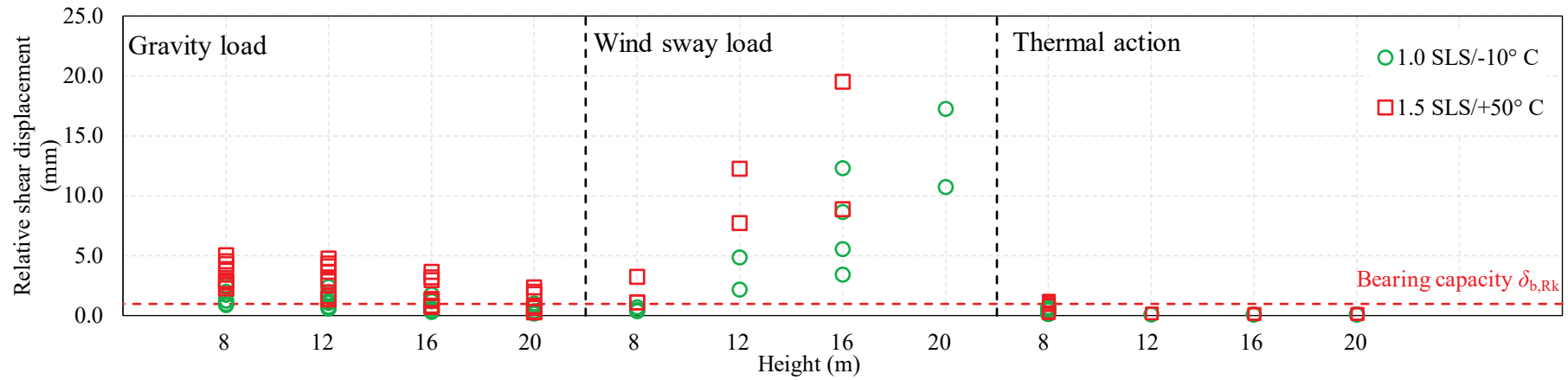
(c) Effect of length

Figure 5.49 (continued): Influence of scale on flashing seam screws

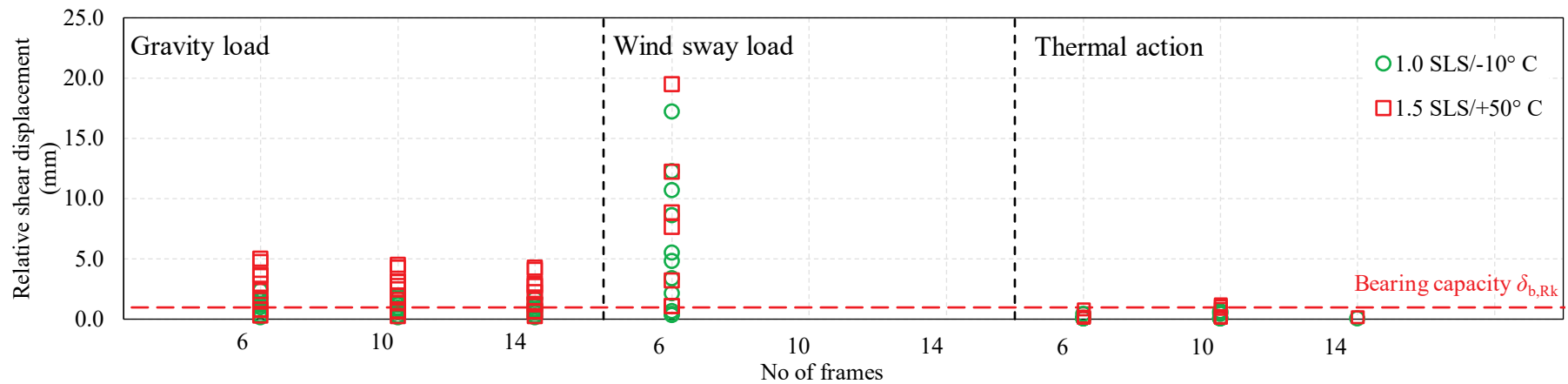


(a) Effect of span

Figure 5.50: Influence of scale on longitudinal and end lap seam screws

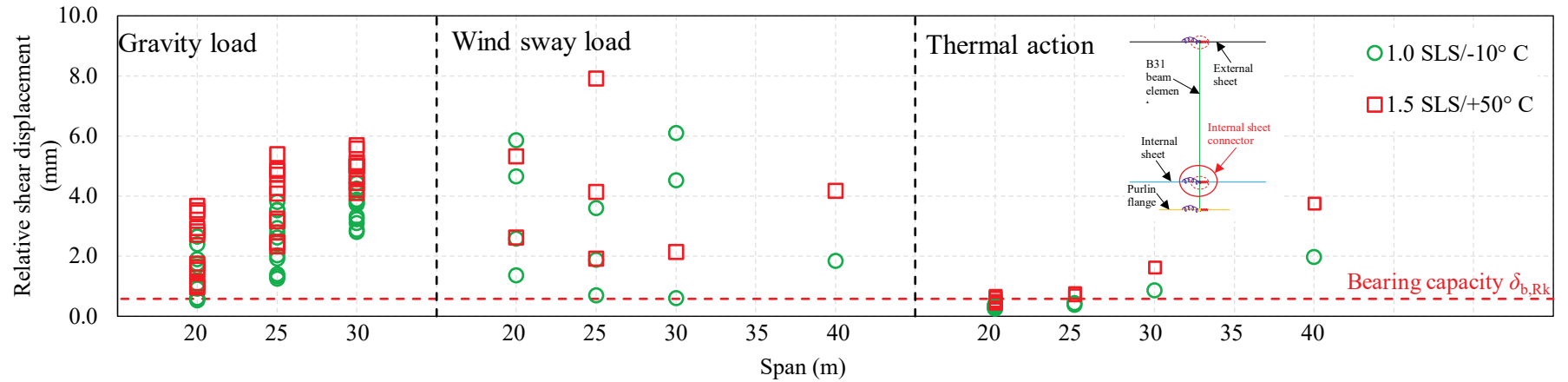


(b) Effect of height

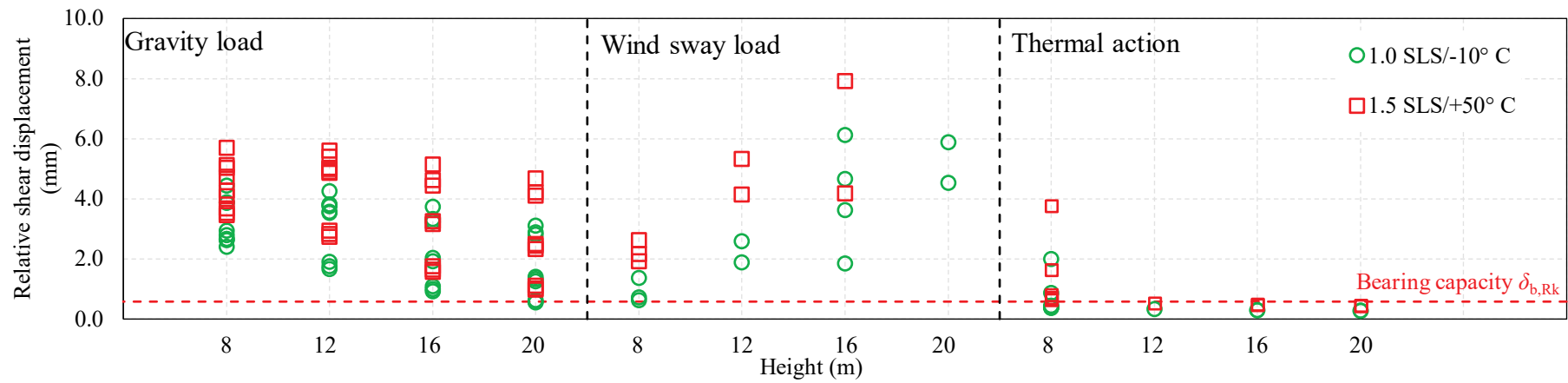


(c) Effect of length

Figure 5.50 (continued): Influence of scale on longitudinal and end lap seam screws

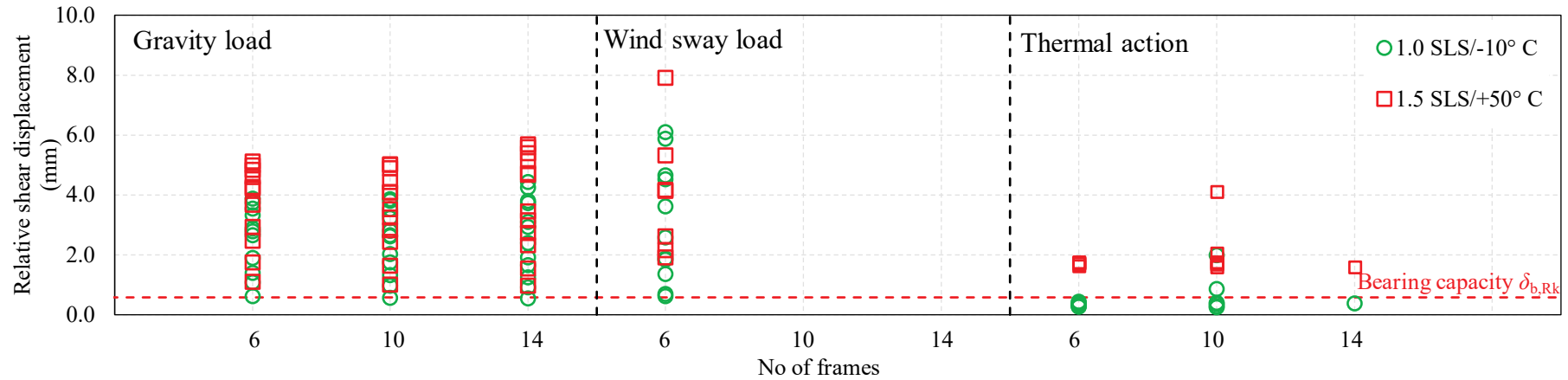


(a) Effect of span



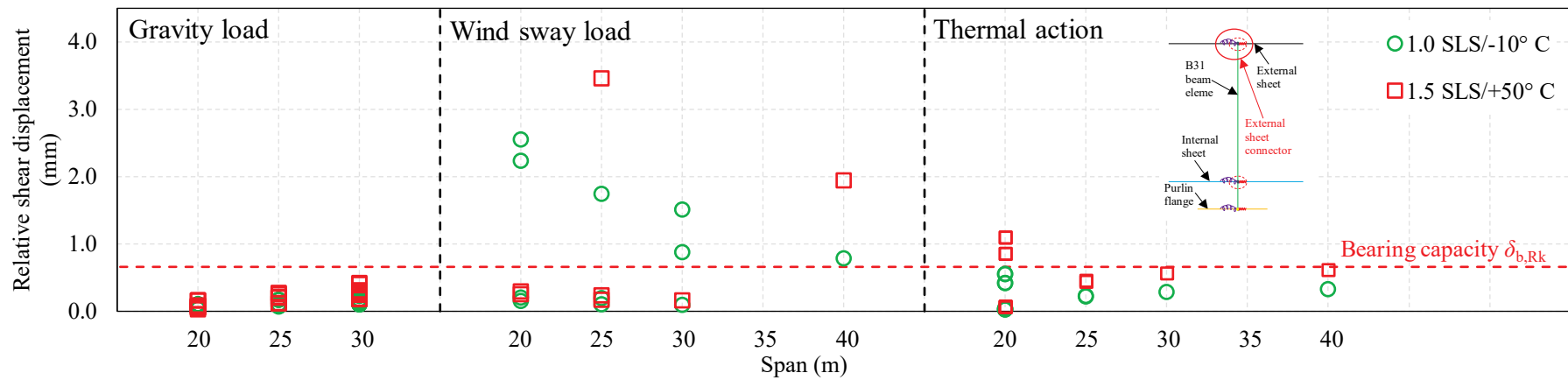
(b) Effect of height

Figure 5.51: Influence of scale on the internal sheet connectors



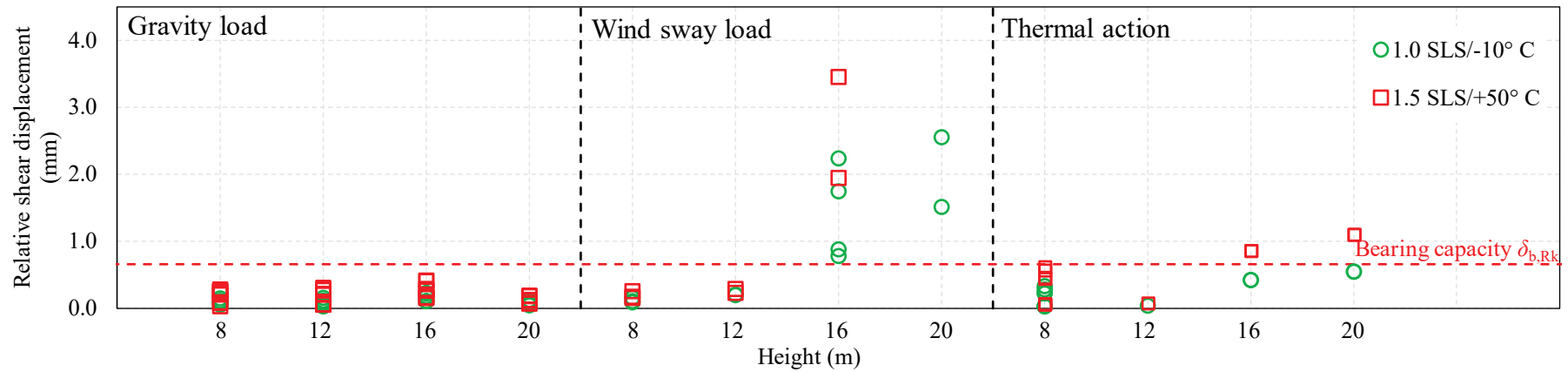
(c) Effect of length

Figure 5.51 (continued): Influence of scale on the internal sheet connectors

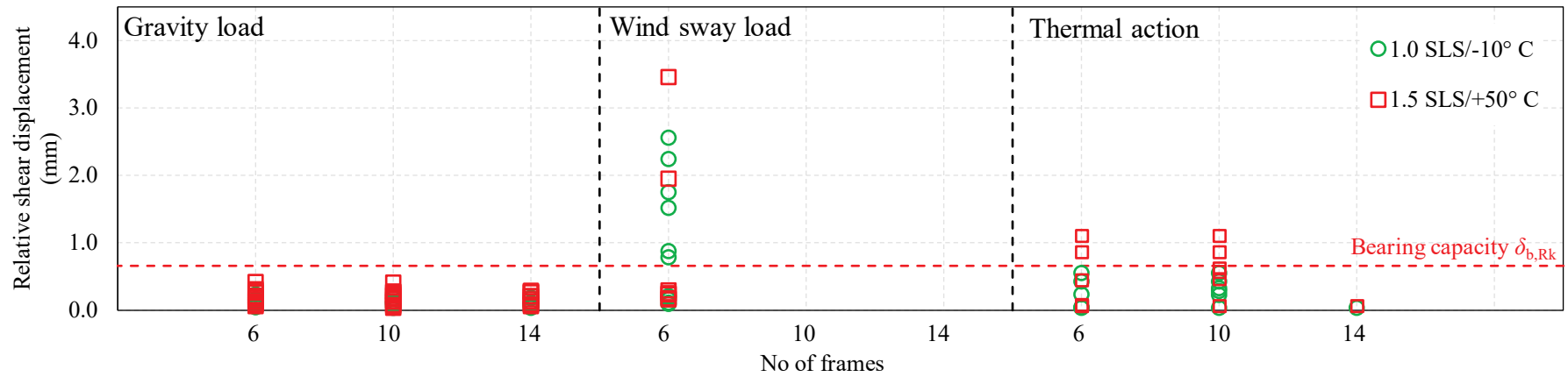


(a) Effect of span

Figure 5.52: Influence of scale on the external sheet connectors



(b) Effect of height



(c) Effect of length

Figure 5.52 (continued): Influence of scale on the external sheet connectors

5.5 Concluding remarks

In conclusion, a parametric study was carried to investigate the effect of scale on the overall behaviour of portal frames with emphasis on the local effect on the cladding screw connectors. Various frame dimensions were considered including spans ranging from 20 to 40 m, eaves heights ranging from 8 to 20 m and structure lengths ranging from 30 to 78 m. The frame models were analysed under three loading conditions: gravity load, wind load and thermal action.

It was found that the wind sway load is the most critical load case with regards to its effect on the shear displacement of the cladding screw connectors. As for the effect of scale, an increase in span increased the shear displacements of the cladding screw connectors under gravity load and thermal action. Eaves height was found to be critical in wind load cases only as the increase in height increased the shear displacements of the cladding screw connectors significantly. There was no significant effect associated with the increase in structure length in this study.

Chapter 6

Conclusions and further research

6.1 Conclusions

Portal frames are a common form of structural system that enables versatile functionality within expansive areas. These frames have evolved significantly over the past decades towards longer, lighter and taller structures utilising a range of two-skin cladding systems. The impact of scale on the behaviour of steel portal frames gained notable attention in a SCOSS alert in 2018 (SCOSS, 2018). This alert highlighted several structural concerns associated with the increasing size of these structures, including larger deflections resulting from increased spans. The interaction between the flexible frames and stiffer cladding systems was identified as a crucial consideration, questioning the applicability of current design recommendations for contemporary portal frames.

This research investigated the effect of scale on the behaviour of steel portal frames with an emphasis on the effect of stressed skin action. Powerful parametric finite element models (FE) were established to evaluate the impact of different frame dimensions on the overall structural performance of steel portal frames. Particular consideration was given to assess the effect of scale on the connecting fasteners at the cladding level.

In Chapter 3, the fundamental aspects of the finite element modelling employed to simulate the components of steel portal frames were presented and validated. These included models used to capture the stress-strain behaviour of hot-rolled steel and cold-formed steel. Generic models describing the load-displacement response of bolted and screwed connections were proposed and validated against experimental results from the literature. The modelling approach used to simulate the cladding system as orthotropic plates was explained and validated against test results for both single-skin sheeting and two-skin sandwich cladding systems. The validated modelling approaches were used to assemble a full frame finite element model.

In Chapter 4, the fundamental aspects and modelling approaches employed in the full frame FE model were presented. These included the material choice, element type, mesh size, boundary conditions and the type of cladding system employed. The arrangement of the portal frame connections, including details such as bolt size, bolt grade and bolt spacings, were also detailed. The assembled full frame FE model was then validated against physical tests for both clad and bare frame configurations. Therefore, combined with the component validation, the full frame FE model was considered able to accurately capture true structural behaviour for use in the parametric assessment of the behaviour of single-span portal frames.

In Chapter 5, a parametric study was conducted to investigate the effect of different frame dimensions on the overall behaviour of portal frames and their connecting elements. The investigation included spans ranging from 20 m to 30 m, eaves heights ranging from 8 m to 20 m and structure lengths ranging from 30 m to 78 m. The frames were subjected to multiple loading conditions including gravity load, wind load and thermal actions. The load distribution between frames was examined and the critical connectors were identified, determining their location and magnitude.

It was found that more load was attracted to the gable frames due to its high relative stiffness compared to the internal frames leading to an increased demand on screws in the end bays and in the gable wall. Consequently, the screws located in the end bays were consistently found to be the most critical. It was found that the wind sway load was the most critical load case, where the shear displacements of the cladding screw connectors were highest. Regarding the effect of scale, the cladding screw connector shear displacements under gravity loads and thermal actions were notably amplified by an increase in span. Height, on the other hand, was found to cause a significant increase in the shear displacements of the cladding screw connectors in wind load scenarios. However, structure length was observed to have minimal effects within the scope of this study. This was attributed to the uniform frame displacements among the internal frames regardless of the structure length, resulting in a minimal demand on the connecting screws in the internal bays while the high relative frame displacements in the end bays resulted in localised high screw displacements.

Overall, it has been revealed that considerable bearing deformations can arise around the connecting screws in steel portal frame cladding systems at serviceability load levels, with greater prominence under wind sway conditions. Such deformations would manifest themselves as slots in the steel sheeting, which could lead to leaking, other practical problems

and possibly more severe consequences if supporting elements were to become detached. This can be explored further in future work with the developed model, but what is clear that further consideration is needed.

6.2 Suggestions for further work

The investigation into scale included the analysis of the overall behaviour of the frames and their connecting fasteners. However, it is important to note that scale can also increase the potential for fatigue failure in the purlin cleats due to higher stress range. To assess the susceptibility of these cleats to fatigue failure at serviceability limit state, a thorough examination of their stress history is necessary. This could be achieved with the developed model.

As discussed in Chapter 5, it was established that the wind sway case emerged as the most critical load case. Further investigation is needed to comprehensively assess the influence of scale under wind sway and wind uplift load conditions on cladding screw connectors.

The parametric study included a single load case for each analysis, reflecting idealised conditions. However, in real-world scenarios, structures experience a combination of loads concurrently. To comprehensively understand the behaviour of portal frames, further exploration involving various loading combinations is necessary.

While the study primarily focused on the influence of the global frame dimensions, it is also important to consider the impact of cladding screw spacing and the dimensions of the cladding e.g. thickness of panels. Analysis with these varying parameters would provide insights into both the overall and local behaviour of the frame.

As discussed in Chapter 5, the connectors near the frame apex were critical in multiple instances. Given that different arrangements of apex flashings are already employed in current practice, these configurations are worth investigation within the FE model framework.

Finally, this study exclusively examined single-span portal frames. Multiple span portal frames, characterised by increased flexibility, are expected to exert even greater demands on connecting elements. Exploring this aspect could provide further insights into the behaviour of these structural systems.

References

- ABAQUS. (2021). ABAQUS online documentation, version 2021. Dassault Systèmes. 2021
- Arrayago, I., Real, E. & Gardner, L. (2015). Description of stress-strain curves for stainless steel alloys. *Materials & Design*, **87**; 540-552.
- Bates, W., Bryan, E. R. & El-Dakhakhni, W. M. (1965). Full-scale tests on a portal frame shed. *The Structural Engineer*, **43**; 199-208.
- Brown, D., Iles, D., Brettle, M. & Malik, A. (2013). *Joints in Steel Construction: Moment-Resisting Joints to Eurocode 3*, SCI Publication No. P398, UK, The Steel Construction Institute and The British Constructional Steelwork Association Limited.
- Bryan, E. R. (1964). The stiffening effect of sheeting in buildings. *IABSE congress report: Seventh congress*, Rio de Janeiro, 1964, **7**; 285-290.
- Bryan, E. R. (1971a). Research into the structural behaviour of a sheeted building. *Proceedings of the Institution of Civil Engineers*, **48**; 65-84.
- Bryan, E. R. (1971b). Tests at Salford. *Building With Steel*, **8**; 43-45.
- Bryan, E. R. (1972). *The stressed skin design of steel buildings*, London, Wiley.
- Bryan, E. R. (1976). Stressed skin design and construction: a state of the art report. *The Structural Engineer*, **54**; 374-351.
- Bryan, E. R. & El-Dakhakhni, W. M. (1968). Shear of corrugated decks: calculated and observed behaviour. *Proceedings of the Institution of Civil Engineers*, **41**; 523-540.
- Bryan, E. R. & El Dakhakhni, W. M. (1964). Behaviour of sheeted portal frame sheds: theory and experiments. *Proceedings of the Institution of Civil Engineers*, **29**; 743-778.
- Bryan, E. R. & Mohsin, M. E. (1972). The design and testing of a steel building taking account of the sheeting. *IABSE congress report: Ninth congress*, Amsterdam, 1972, **9**; 305-314.

CEN (1992). EN 20273. Fasteners - Clearance holes for bolts and screws. Brussels: European Committee for Standardization (CEN); 1992.

CEN (2002). EN 1990. Eurocode - Basis of structural design. Brussels: European Committee for Standardization (CEN); 2002.

CEN (2003). EN 1993-1-3. Eurocode 3: Design of Steel Structures - Part 1-3 : General rules - Supplementary rules for cold-formed members and sheeting. Brussels: European Committee for Standardization (CEN); 2003.

CEN (2005a). EN 1991-1-4. Eurocode 1: Actions on structures - Part 1-4: General actions - Wind actions. Brussels: European Committee for Standardization (CEN); 2005a.

CEN (2005b). EN 1993-1-1. Eurocode 3: Design of Steel Structures - Part 1-1: General rules and rules for buildings. Brussels: European Committee for Standardization (CEN); 2005b.

CEN (2005c). EN 1993-1-8. Eurocode 3: Design of Steel Structures - Part 1-8 : Design of joints. Brussels: European Committee for Standardization (CEN); 2005c.

CEN (2018). EN 1090-2. Execution of Steel Structures and Aluminium Structures - Part 2: Technical requirements for steel structures. Brussels: European Committee for Standardization (CEN); 2018.

CEN (2021). prEN 1993-1-14. prEurocode 3: Design of Steel Structures - Part 1-14: Design assisted by finite element analysis, Draft. Brussels: European Committee for Standardization (CEN); 2021.

CEN (2022). prEN 1993-1-1. prEurocode 3: Design of Steel Structures - Part 1-1: General rules and rules for buildings, Final draft. Brussels: European Committee for Standardization (CEN); 2022.

Chung, K. F. & Ip, K. H. (2000). Finite element modeling of bolted connections between cold-formed steel strips and hot rolled steel plates under static shear loading. *Engineering Structures*, **22**; 1271-1284.

Coelho, A. M. G., Bijlaard, F. S. K., Gresnigt, N. & da Silva, L. S. (2004). Experimental assessment of the behaviour of bolted T-stub connections made up of welded plates. *Journal of Constructional Steel Research*, **60**; 269-311.

- Davies, J. M. (1973). The plastic collapse of framed structures clad with corrugated steel sheeting. *Proceedings of the Institution of Civil Engineers*, **55**; 23-42.
- Davies, J. M. (2006). Light gauge steel cassette wall construction - theory and practice. *Journal of Constructional Steel Research*, **62**; 1077-1086.
- Davies, J. M. & Bryan, E. R. (1982). *Manual of stressed skin diaphragm design*, London, Granada Publishing.
- Davies, J. M., Engel, P., Liu, T. T. C. & Morris, L. J. (1990). Realistic modelling of steel portal frame behaviour. *The Structural Engineer*, **68**.
- Davies, J. M. & Fisher, J. (1987). End failures in stressed skin diaphragms. *Proceedings of the Institution of Civil Engineers Part 2-Research and Theory*, **83**; 275-293.
- Davies, J. M., Roberts, M. J. & Wang, Y. C. (2021). Stressed skin theory and structure cladding interaction: Safety concerns with Big Sheds. *Thin-Walled Structures*, **169**; 108415.
- Davison, B. & Owens, G. W. (2012). *Steel designers' manual*, Wiley Online Library.
- Draganić, H., Dokšanović, T. & Markulak, D. (2014). Investigation of bearing failure in steel single bolt lap connections. *Journal of Constructional Steel Research*, **98**; 59-72.
- Duoc, T. P., James, B. P. L., Tiku, T. T., Wrzesien, A. M., Wei, S. & Lawson, R. M. (2015). Optimal design of cold-formed steel portal frames for stressed-skin action using genetic algorithm. *Engineering Structures*, **93**; 36-49.
- ECCS. (1995). European recommendations for the application of metal sheeting acting as a diaphragm - stressed skin design. In *European convention for constructional steelwork, No. 88 (ECCS, TC7)*, 1995
- Gahlen, P., Fröbel, S., Karbach, A., Gabriel, D. & Stommel, M. (2021). Experimental multi-scale approach to determine the local mechanical properties of foam base material in polyisocyanurate metal panels. *Polymer Testing*, **93**; 106965.
- Gardner, L. & Yun, X. (2018). Description of stress-strain curves for cold-formed steels. *Construction and Building Materials*, **189**; 527-538.

- Godfrey, D. A. & Bryan, E. R. (1959). The calculated and observed effects of dead loads and dynamic crane on the framework of a workshops building. *Proceedings of the Institution of Civil Engineers*, **13**; 197-214.
- Gryniewicz, M., Roberts, M. J. & Davies, J. M. (2021). Testing and analysis of a full-scale steel-framed building including the consideration of structure-cladding interaction. *Journal of Constructional Steel Research*, **181**; 106611.
- Hayward, A. & Weare, F. (2002). *Steel detailers' manual*, UK, Blackwell Science Ltd.
- Henderson, J. R. (2015). *Design of steel portal frame buildings to Eurocode 3*, SCI Publication No. P399, UK, The Steel Construction Institute.
- Henriques, J., Jaspart, J. P. & da Silva, L. S. (2014). Ductility requirements for the design of bolted shear connections in bearing. *Advanced Steel Construction*, **10**; 33-52.
- Heywood, M. D. (2006). *Best practice for the specification and installation of metal cladding and secondary steelwork*, SCI Publication No. P346, UK, The Steel Construction Institute.
- Hill, H. N. (1944). Determination of stress-strain relations from offset yield strength values, Technical note No. 927. *National Advisory Committee for Aeronautics*.
- Hui, C., Gardner, L. & Nethercot, D. A. (2016). Moment redistribution in cold-formed steel continuous beams. *Thin-Walled Structures*, **98**; 465-477.
- Jaspart, J. P. & Weynand, K. (2017). *Design of joints in steel structures - UK edition*, ECCS – European Convention for Constructional Steelwork
- Junid, S. M. B. S. (1992). *Lateral-torsional buckling of haunched members in portal frames: An assessment of BS 5950: Part 1*. PhD thesis, University of Salford.
- Käpplein, S. & Misiek, T. (2011). EASIE - Connections of sandwich panels. *Karlsruher Institut für Technologie (KIT), Versuchsanstalt für Stahl, Holz und Steine*.
- King, C. (2001). *In-plane Stability of Portal Frames to BS 5950-1:2000*, SCI Publication No. P292, UK, The Steel Construction Institute.
- Kingspan (2015). *Kingspan KS1000RW Product Data Sheet*. UK, Kingspan®.

- Kingspan (2020a). *QuadCore® Trapezoidal Roof KS1000 RW Installation Guide*. UK, Kingspan®.
- Kingspan (2020b). *Trapezoidal Roof Panel (KS1000 RW) Data Sheet*. UK, Kingspan®.
- Kingspan. (2022). *QuadCore® KS1000RW Technical Information* [Online]. Kingspan®. Available: <https://www.kingspan.com/gb/en/products/insulated-panels/roof-panels/quadcore-ks1000rw-roof-panel/?s=t> [Accessed 04/08/2022].
- Koschmidder, D. & Brown, D. G. (2012). *Elastic design of single-span steel portal frame buildings to Eurocode 3, SCI Publication No. P397*, UK, The Steel Construction Institute.
- Kucukler, M., Gardner, L. & Macorini, L. (2015). Lateral–torsional buckling assessment of steel beams through a stiffness reduction method. *Journal of Constructional Steel Research*, **109**; 87-100.
- Kunkel, C. & Lange, J. (2015). Experimental analysis on the bearing capacity of sandwich panel joints. In *20th International Conference on Composite Materials*, 2015
- Kyvelou, P., Hui, C., Gardner, L. & Nethercot, D. A. (2018). Moment redistribution in cold-formed steel sleeved and overlapped two-span purlin systems. *Advances in Structural Engineering*, **21**; 2534-2552.
- Kyvelou, P., Kyprianou, C., Gardner, L. & Nethercot, D. A. (2019). Challenges and solutions associated with the simulation and design of cold-formed steel structural systems. *Thin-Walled Structures*, **141**; 526-539.
- Lendvai, A. & Joo, A. L. (2018). Experimental full-scale tests on steel portal frames for development of diaphragm action - Part II Effect of structural components on shear flexibility. *Thin-Walled Structures*, **132**; 740-758.
- Lendvai, A., Joo, A. L. & Dunai, L. (2018). Experimental full-scale tests on steel portal frames for development of diaphragm action - Part I experimental results. *Thin-Walled Structures*, **132**; 729-739.
- MatWeb. (2023). *Overview of materials for Thermoset Polyurethane Foam, Unreinforced* [Online]. MatWeb. Available:

<https://www.matweb.com/search/DataSheet.aspx?MatGUID=91d44cae736e4b36bcba94720654eeae&ckck=1> [Accessed 10.03.2023].

MCRMA (2000). *Technical paper No. 12 Fasteners for metal roof and wall cladding: design, detailing and installation guide*. UK, The Metal Cladding and Roofing Manufacturers Association (MCRMA).

MCRMA (2021). *Guidance document 33: Fasteners for metal roof and wall cladding: Design, detailing and installation guide*. UK, The Metal Cladding and Roofing Manufacturers Association (MCRMA).

MCRMA (2022a). *Guidance document 39: Sustainability and durability of metal roofing and cladding systems - Section 1: Introduction and overview*. UK, The Metal Cladding and Roofing Manufacturers Association (MCRMA).

MCRMA (2022b). *Guidance document 39: Sustainability and durability of metal roofing and cladding systems - Section 7: Products*. UK, The Metal Cladding and Roofing Manufacturers Association (MCRMA).

MCRMA (2022c). *Guidance document 39: Sustainability and durability of metal roofing and cladding systems - Section 8: Components*. UK, The Metal Cladding and Roofing Manufacturers Association (MCRMA).

Meng, X. & Gardner, L. (2020). Behavior and design of normal-and high-strength steel SHS and RHS columns. *Journal of Structural Engineering*, **146**; 04020227.

METSEC (2017). *Purlins technical manual: Z and C sections for building shell and mezzanine floor applications*. UK, voestalpine Metsec plc.

Mirambell, E. & Real, E. (2000). On the calculation of deflections in structural stainless steel beams: an experimental and numerical investigation. *Journal of Constructional Steel Research*, **54**; 109-133.

Moreno, E. N., Tarbé, C. & Brown, D. (2014). *Joints in steel construction: simple Joints to eurocode 3, SCI Publication No. P358*, UK, The Steel Construction Institute and The British Constructional Steelwork Association Limited.

- Nagy, Z., Bács, B., Kelemen, A., Sánduly, A., Nagy, Ö. & Lőrincz, B. (2023). Rafter-purlin connection stiffness impact on the stress skin effect of corrugated sheet claddings. *Thin-Walled Structures*, **185**; 110615.
- NSC (January 2019). 'Eastern gateway', *New Steel Construction (NSC)*, **27(1)**; 22-24. Available: https://www.newsteelconstruction.com/wp/wp-content/uploads/digi/NSC_Jan19digi/pubData/source/NSC_Jan19digi.pdf#page=22 [Accessed 29/07/2023].
- NSC (September 2020). 'Steel's the ticket for new rail depot', *New Steel Construction (NSC)*, **28(8)**; 22-23. Available: https://www.newsteelconstruction.com/wp/wp-content/uploads/digi/2020pdf/NSC_Sept2020digi.pdf#page=22 [Accessed 10/07/2023].
- NSC (January 2021). 'HS2 precast manufacturing facilities', *New Steel Construction (NSC)*, **29(1)**; 16-17. Available: <https://www.newsteelconstruction.com/wp/wp-content/uploads/2021/01/NSCJan2021digi.pdf#page=16> [Accessed 10/07/2023].
- Quach, W. M., Teng, J. G. & Chung, K. F. (2008). Three-stage full-range stress-strain model for stainless steels. *Journal of Structural Engineering-Asce*, **134**; 1518-1527.
- Ramberg, W. & Osgood, W. R. (1943). Description of stress-strain curves by three parameters, Technical note No. 902. *National Advisory Committee for Aeronautics*.
- Rasmussen, K. J. R. (2003). Full-range stress-strain curves for stainless steel alloys. *Journal of Constructional Steel Research*, **59**; 47-61.
- Rinchen & Rasmussen, K. J. R. (2019). Numerical modelling of cold-formed steel single C-section portal frames. *Journal of Constructional Steel Research*, **158**; 143-155.
- Roberts, M. J. & Davies, J. M. (2023). Structure cladding interaction in sandwich panel roofs. *Structures*, **57**; 105064.
- Roberts, M. J., Davies, J. M. & Wang, Y. C. (2021). Numerical analysis of a clad portal frame structure tested to destruction. *Structures*, **33**; 3779-3797.
- Salter, P. R., Malik, A. S. & King, C. M. (2004). *Design of Single-Span Steel Portal Frames to BS 5950-1:2000*, SCI Publication P252, UK, The Steel Construction Institute.

- SCOSS (2018). SCOSS Alert 2018 - Effects of scale, Standing Committee on Structural Safety. *The Institution of Structural Engineers*.
- Stainsby, R. & Pope, R. (2003). *Commentary on the fourth edition of the national structural steelwork specification for building construction*, The British Constructional Steelwork Association Ltd.
- TATA Steel (2013). *Colorcoat Technical Paper - Structural performance of built-up roof and wall cladding systems*. UK, TATA Steel.
- TATA Steel (2016). *Colorcoat Technical Paper - Creating an airtight building envelope*. UK, TATA Steel.
- Vacev, T., Zorić, A., Milić, M., Paunović, S. & Nešović, I. (2020). Stressed Skin Design Versus Braced Frame Design Through Efficient Numerical Modelling. *International Journal of Steel Structures*, **20**; 1209-1229.
- Vella, N., Gardner, L. & Buhagiar, S. (2020). Experimental analysis of cold-formed steel-to-timber connections with inclined screws. *Structures*, **24**; 890-904.
- Wang, Z. Y., Zhang, J. F. & Jiang, J. Q. (2016). A multi-spring component modelling approach for T-stub connections. *Advances in Structural Engineering*, **19**; 883-896.
- Wrzesien, A. M., Lim, J. B. P., Xu, Y., MacLeod, I. A. & Lawson, R. M. (2015). Effect of stressed skin action on the behaviour of cold-formed steel portal frames. *Engineering Structures*, **105**; 123-136.
- Xia, Y., Friswell, M. & Flores, E. S. (2012). Equivalent models of corrugated panels. *International Journal of Solids and Structures*, **49**; 1453-1462.
- Ye, J., Mojtabaei, S. M., Hajirasouliha, I. & Pilakoutas, K. (2020). Efficient design of cold-formed steel bolted-moment connections for earthquake resistant frames. *Thin-Walled Structures*, **150**.
- Yuan, H. X., Wang, Y. Q., Gardner, L. & Shi, Y. J. (2014). Local–overall interactive buckling of welded stainless steel box section compression members. *Engineering Structures*, **67**; 62-76.

-
- Yun, X. & Gardner, L. (2017). Stress-strain curves for hot-rolled steels. *Journal of Constructional Steel Research*, **133**; 36-46.
- Yun, X. & Gardner, L. (2018). The continuous strength method for the design of cold-formed steel non slender tubular cross-sections. *Engineering Structures*, **175**; 549-564.
- Yun, X., Gardner, L. & Boissonnade, N. (2018). Ultimate capacity of I-sections under combined loading - Part 1: Experiments and FE model validation. *Journal of Constructional Steel Research*, **147**; 408-421.
- Yun, X., Wang, Z. & Gardner, L. (2021). Full-range stress–strain curves for aluminum alloys. *Journal of Structural Engineering*, **147**; 04021060.
- Zhang, X. & Rasmussen, K. J. R. (2014). Tests of cold-formed steel portal frames with slender sections. *Steel Construction*, **7**; 199-203.
- Zhang, X., Rasmussen, K. J. R. & Zhang, H. (2015). Structural modeling of cold-formed steel portal frames. *Structures*, 2015, **4**; 58-68. Elsevier.