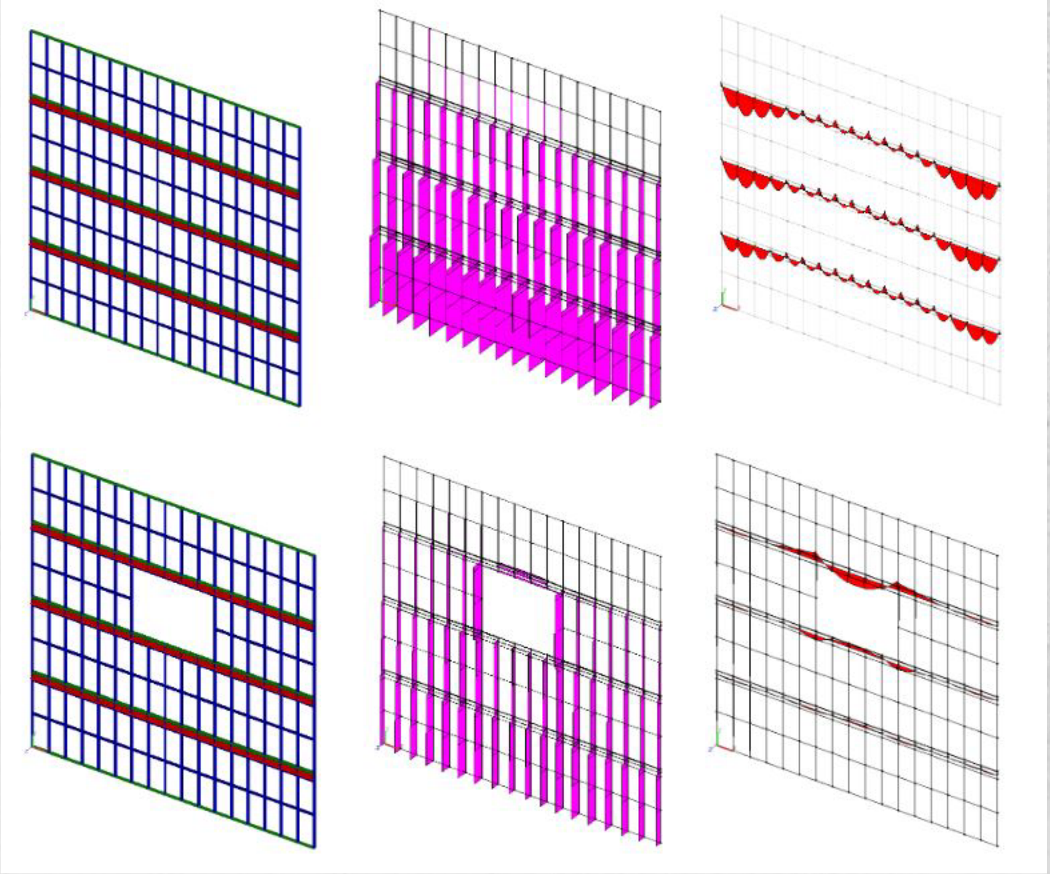
PROJECT 20: SYSTEMS AND METHODS FOR ROBUSTNESS OF MID-RISE LIGHT GAUGE STEEL (LGS) BUILDINGS – PHASE 1 SCOPING STUDY

FINAL REPORT



Australian Government
Department of Industry, Science and Resources
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CONTENTS

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EXECUTIVE SUMMARY

This Scoping Study aims to provide insight into structural robustness as it relates to mid-rise light gauge steel (LGS) structures. This is achieved by first providing a detailed overview of approaches towards achieving structural robustness and then recommending several approaches most relevant to LGS structures, namely Alternative Load Path Analysis (i.e., performance-based approach) and prescriptive approaches, including tying force methods.

This investigation into robustness methods is supported by a review of current structural systems used in LGS structures. Connection details were found to be a particularly important aspect of LGS design in the context of robustness, the strength and ductility of connections must be sufficient to engage last resort collapse mechanisms (i.e., catenary action, membrane action, etc.). The connections between stud and joist were found to be particularly important in the literature as well as a preliminary analysis conducted in this report. Existing research has provided insight into advantageous connection designs, including those that utilize bolted connections as opposed to screw or rivet connections and those that use combinations of clip angle and flange strips in the web and flanges. Additionally, care must be taken toward the design of ledger track and header members, where existing research indicates the need to design such members with structural robustness considered.

Finally, this report provides a case study into a preliminary alternative load path analysis on a simplified representative LGS mid-rise building using Abaqus, a commercial finite element (FE) package. This case study provides insight into the efficient creation of such models through the use of Python scripting. This scripting allows for significantly reduced model creation time, while also promoting easier modification of structural configurations, material properties and connection stiffnesses. Analysis thus far has provided a point of comparison concerning the internal tie-force provisions within both local and international codes, as well as providing insight into the contribution of catenary action and other robustness mechanisms in arresting collapse.

Future work will focus upon experimentally investigating novel connection details under robustness contexts, improving FE modelling by more accurately accounting for connection stiffnesses, among other things, and using this finite element modelling to parametrically study various LGS systems.

PROJECT OVERVIEW

1. Introduction

Light gauge steel (LGS) is used extensively within both non-loadbearing and loadbearing mid-rise structural contexts internationally. This widespread use is attributable to a variety of material advantages, including its very high strength to weight ratios and lightweight characteristics. The use of LGS in load-bearing mid-rise applications within Australia has been comparatively less widespread. In promoting the more extensive use of light gauge steel in these contexts, an understanding of the structural robustness of such structures is integral.

This Scoping Report aims to provide insight into this area of light gauge steel use, first by providing a detailed overview in which structural robustness is ensured within the building industry locally and internationally and providing recommendations as to the most appropriate methods relevant to light gauge steel.

This review of design methods is then supported by an investigation into the structural systems commonly associated with light gauge steel, in particular connection details. This review aims to additionally provide insight into the current body of research concerning alternative light gauge steel connection details.

Lastly, this report will detail a preliminary alternative load path analysis on a simplified representative light gauge steel mid-rise structure. Alternative load path analysis represents one of the centrepieces to designing robust structures, both locally and internationally. This investigation provides insight into some of the challenges in modelling light gauge steel structures and progress made in these areas, particularly in the development of a platform for model creation utilising Python scripting.

PROJECT FINDINGS AND OUTCOMES

2. Review Design Methods For Robustness

2.1 Background

Structural robustness refers to a structure's ability to avoid disproportionate collapse, a collapse state characterised by damage disproportionate to initial local damage. Additionally, distinct to disproportionate collapse is progressive collapse, a collapse configuration associated with a progressive spread of damage.

A well-known example of such disproportionate collapse is represented by the Ronan Point apartment collapse in 1968 (Figure 1 [1]). This disaster involved the disproportionate collapse of a significant portion of the corner of an apartment block following an accidental gas stove explosion toward the top of the building. This incident prompted further consideration around progressive collapse, disproportionate collapse and structural robustness.

Figure 1 Ronan Point 1967 [1]

2.2 Key Approaches Addressing Structural Robustness

Broadly, there are several different approaches to improving the structural robustness of a given structure, that is, improve a structure's ability to resist disproportionate collapse. These include both event-dependent approaches, wherein the design for structural robustness is predicated upon a known adverse event (eg. Fire, Blast, Impact, etc.) or event-independent approaches. Additionally, design approaches concerning structural robustness may take into consideration a structure's specific structural configuration (structure-specific), or alternatively be structure-independent. Robustness approaches include:

- Notional Element Removal
- Key Element Design
- Prescriptive Approaches
- Risk-Based Methods
- Segmentation.[2]

These approaches are summarised in subsequent sections.

2.2.1 Notional Element Removal

Structural robustness can be addressed by selectively removing elements within a structure and assessing the structure's ability to redistribute load to alternative load paths. Commonly, this approach involves analysing a structure's ability to withstand the loss of a load-bearing column or wall segment, avoiding damage disproportionate to this initial element removal. This concept is represented in the simplified diagram, Figure 2, wherein the central column is removed, the adjacent spans become double-spanning and load from floors above (represented by the orange arrow) must be redistributed to the surrounding structure. In providing structural robustness, it is thus integral that the surrounding structure (elements and associated connections) possess sufficient strength and ductility to resist this new loading configuration.

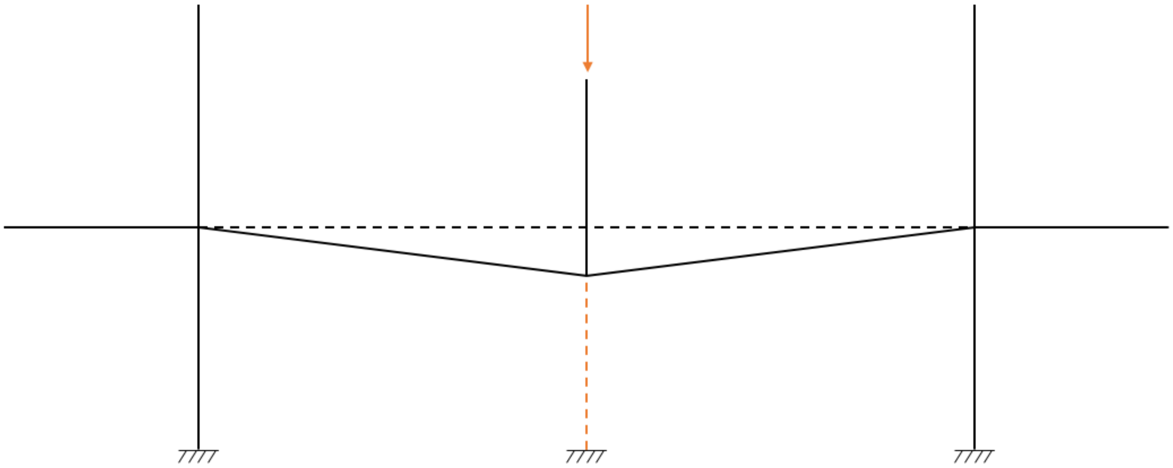


Figure 2 Simplified Column Removal

As described by Cormie et al. [3], this analysis can be performed to varying levels of complexity, from most simple to most complex:

- Linear Static Procedures: Analysis is performed based on linear-elastic material properties, geometric linearity and loads applied statically. The dynamic effects associated with the speed of removing an element are accounted for via a dynamic load factor (DLF), this factor is applied to loads post-element removal. This factor is bounded between 1.0 (no dynamic effects) to 2.0 (instantaneous removal). Typically, linear procedures (both static and dynamic) act as precursors to non-linear procedures as it is uncommon to design structures to behave completely elastically to such extreme events.

- Non-linear Static Procedures: Analysis is performed based on non-linear properties, these include some combination of material non-linearity and geometric non-linearity. Loads however are still applied statically, there is no time-history evolution associated with structural response. Again, a DLF is applied to these loads to account for the dynamic effects of an element removal event.

- Energy Balance Procedures: As described by Cormie et al. [3], energy balance procedures involve balancing the energy released by removing a column with the internal energy of the structure, this internal energy being the summation of relevant elastic strain energy, plastic strain energy and energy associated with damping.

- Linear Dynamic Procedures: Analysis is performed based on linear-elastic material properties and geometric linearity, as was the case in the static variant detailed above. However, within dynamic procedures the requirements for DLFs do not exist, instead the dynamic effects associated with element removal are accounted for directly within dynamic analysis. The time-history of both load application and structural response are directly analysed in dynamic procedures.

- Non-linear Dynamic Procedures: Non-linear dynamic procedures are the most complex variant of alternative load path approaches, with analysis based on non-linear material properties, geometric non-linearity and time-history of the load and response of the structure accounted for directly.

Typically, when assessing the robustness of a structure through notional element removal, simple procedures will act as the precursor to more complex analysis, providing initial insight into structural response, over-stressed regions and the requirements of a more detailed analysis.

The application of notional element removal will be explored in more detail in later sections of this report.

2.2.2 Prescriptive Approaches

Prescriptive approaches cover a variety of code-mandated provisions that aim to ensure that a structure is designed to possess a minimum level of structural robustness. These provisions are independent of structural layout but may be specific to structural material type (i.e.. concrete, timber, hot-rolled steel, light gauge steel, etc.). The most common example of the prescriptive robustness approach is in tie-force provisions, these provisions mandate a minimum level of tying force between the connection of structural elements in both the horizontal and vertical directions. A typical representation of the different types of tie forces mandated is shown in Figure 3 from the UFC Guidelines [4].

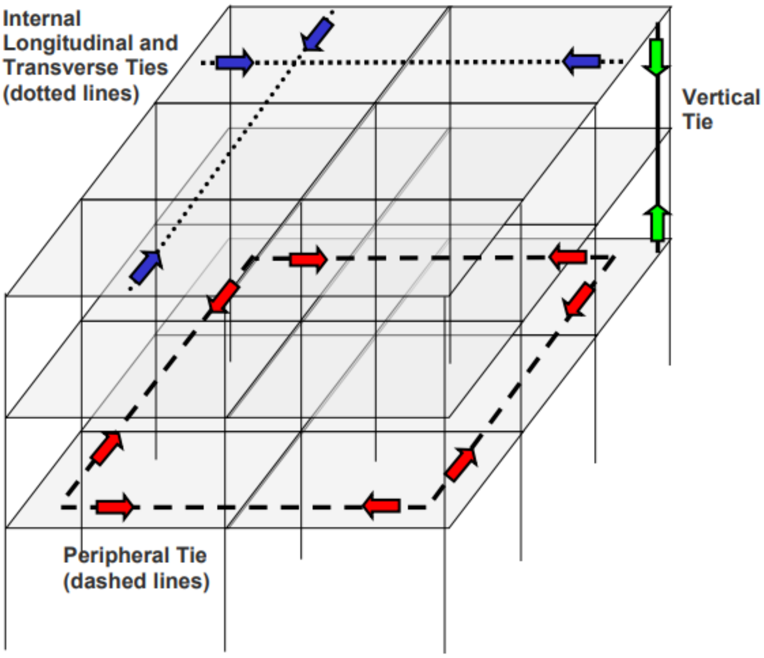


Figure 3 Tie Force Provisions in a Framed Structure [4]

Tie-force provisions assume that some level of localised damage has occurred, however that when a structure conforms to these provisions, the structure will have a minimum level of resistance against the propagation of further damage. The implicit intent behind these prescriptions is that a structure's connections will possess enough strength and ductility to mobilise last resort collapse mechanisms like catenary action and membrane action [3, 5, 6]. Research into the validity of this assumption however has raised doubt as to whether this is the case under current code-mandated provisions in hot-rolled steel structures [7], concrete structures [8] and cold-formed steel structures [9].

2.2.3 Risk-Based Methods

Risk-based methods involve a systematic assessment of the likelihood and consequence of an extreme event, balanced by the cost of mitigating against such an event. This economic approach towards robustness may be used to justify the use of a more involved structural robustness assessment (as opposed to being a standalone method). For example, this method may be used in assessing the departure from prescriptive approaches to also undertaking notional element removal methods where relevant design codes do not provide this guidance. For this reason, it is only used conjunction with other methods.

2.2.4 Key Element Design

Key Element Design involves designing specific elements to be resistant to localised element failure. Criteria around the selection of such elements is typically provided within the relevant design codes and guidelines, selection is usually directed towards elements that carry a substantial proportion of the total structural load. The degree to which elements are strengthened is predicated upon an assessment of identified hazards and their associated impact. For this reason, key element design is typically an event-specific approach supported by a systematic risk assessment (as is the case in Australia under the National Construction Code (NCC) [10]) or the forces in which key elements are to be designed for is specifically mandated within the relevant design codes (as in the case in the United Kingdom under the Eurocodes [11]). Key Element Design is typically viewed as a last resort to designing for structural robustness if other methods of robustness assessment are unsuitable [3].

2.2.5 Segmentation

Segmentation involves preventing localised damage from propagating to other structural elements by selectively isolating structural elements [2]. This approach is typically reserved for bridge structures and its application in building structures is to be determined, it will not be explored further in this report.

2.3 Structural Robustness Within Australia’s Legislative Framework

Structural robustness, as do other structural requirements fall under a legislative hierarchy in Australia (Figure 4), this hierarchy consists of a Building Act specific to each state [12], a set of Building Regulations specific to each state [13], the National Construction Code [10] and reference documents [14].

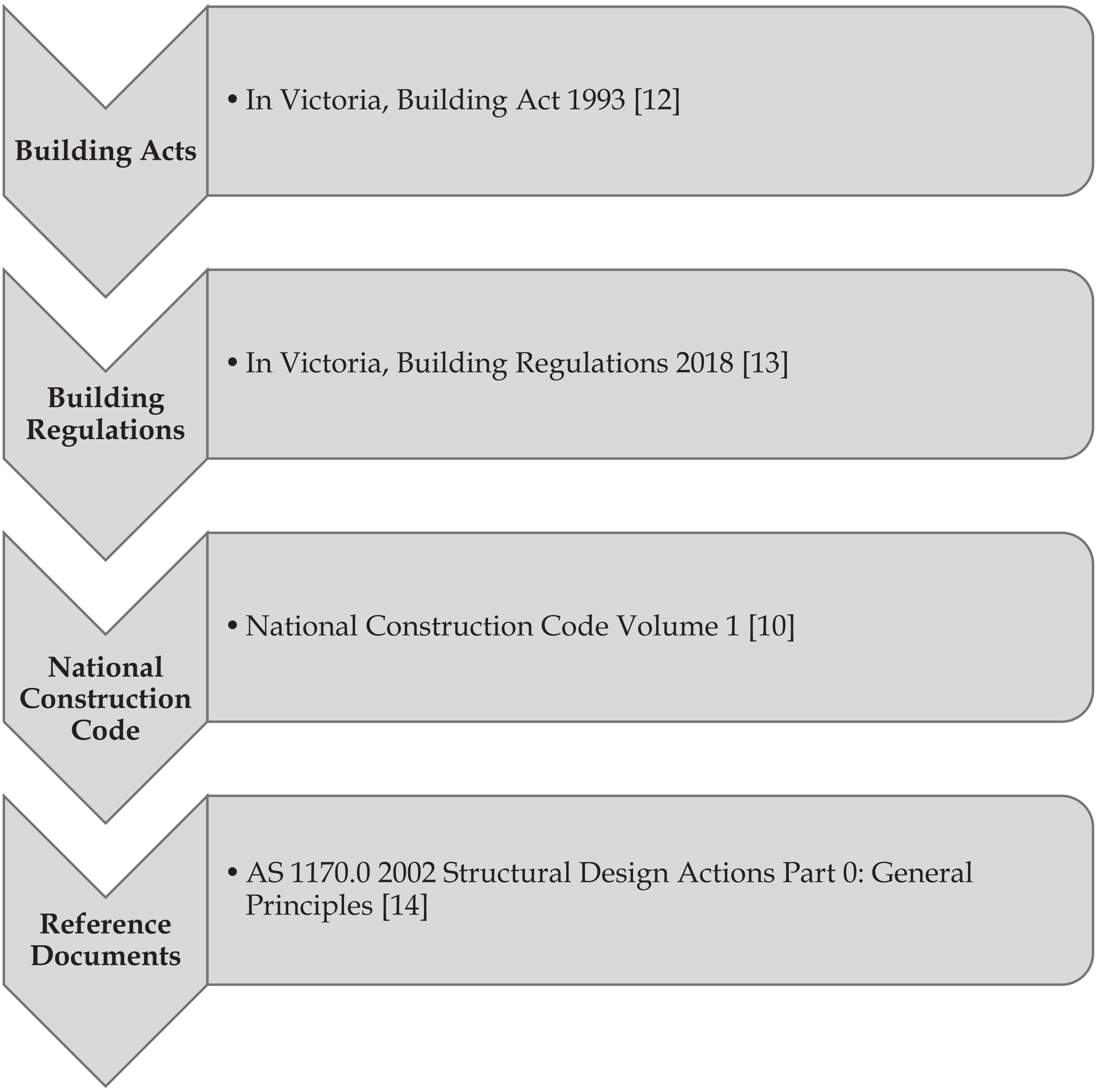


Figure 4 Australia's Legislative Hierarchy

Considerations for structural robustness are explicitly provided for within both the National Construction Code [10] and the reference documents, specifically AS 1170.0 General Principles [14]. Within these documents, several of the approaches outlined in Section 2.2 are detailed and mandated in ensuring the design of robust structures in Australia.

The National Construction Code [10] makes reference to ‘disproportionate collapse’ and by extension, structural robustness in clause BP1.1 (a) (iii) [10], wherein, it states that:

> *A building or structure, during construction and use, with appropriate degrees of reliability, must –*
>
> *[…]*
>
> *(iii) be designed to sustain local damage, with the structural system as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage; […].*

The National Construction Code [10] then outlines the various approaches to satisfying clause BP1.1 (a) (iii) [10] in clause BV2 (a) & (b). These approaches can be summarised as:

- A notional element removal approach, described within the code as,

  (a) assessment of the structure such that upon the notional removal in isolation of –

  (i) any supporting column: or

  (ii) any beam supporting one or more columns; or

  (iii) any segment of a load bearing wall of length equal to the height of the wall,

  the building remains stable and resulting collapse does not extend further than the immediate adjacent storeys; and

- A key element design approach of elements carrying more than 25% of total structural load, that is supported by a systematic risk assessment, described within the code as,

  (b) assessment demonstrating that if a supporting component is relied upon to carry more than 25% of the total structure a systematic risk assessment of the building is undertaken and critical high risk components are identified and designed to cope with the identified hazard or protective measures chosen to minimise the risk.

The requirements of the National Construction Code [10] are supplemented by the reference document relevant to structural robustness, AS 1170.0 General Principles Section 6 [14]. This supplement takes a prescriptive approach, as opposed to the notional element removal and key element design approaches described above. This approach can be summarised as:

- A prescriptive approach mandating the minimum level of lateral resistance the structure shall possess as a whole, described with the Standard as:

  The structure shall have a minimum lateral resistance equivalent to the following percentage (G + φ Q) for each level, applied simultaneously at each level for a given direction:

  (a) For a structure over 15 m tall: 1%

  (b) For all other structures: 1.5%

- A prescriptive approach mandating a minimum level of tying force between connections, described with the Standard as:

  All parts of the structure shall be interconnected. Connections shall be capable of transmitting 5 percent of the value of (G + φ Q) for the connection under consideration.

- Additional commentary is provided around general provisions for the design of floor and roof diaphragms, within the Standard such elements are provisioned as being able to:

  (a) To resist required horizontal forces; and

  (b) To have ties or struts (where used) to distribute the required wall anchorage forces.

- Lastly, wall elements are also provided with a similar prescriptive force as those of the connections:

  Walls shall be connected to the structure to provide horizontal resistance to face loads. The connection between the wall and structure shall be capable of resisting 5% of G.

Further guidance concerning robustness requirements in the context of the NCC and Australian Standards can be found within a non-mandatory Structural Robustness handbook provided by the Australian Building Codes Board [15], although specific guidance around the application of these principles is limited.

The application of the approaches is non-trivial and requires substantial engineering judgement, several useful guidelines have been developed by both the Australian Building Codes Board [15] and by industry bodies, namely WoodSolutions [16]. The application of the NCC and the Australian Standards to Australian building design has been outlined by Hewson [16], wherein a similar approach to that of the European robustness guidelines has been undertaken where Building Class influences the approaches employed. This rational approach is summarised in Table 1 where the final column are recommendations from Hewson [16].

Table 1 Building Class influences Robustness Approach (Adapted from [14] and [16])

Consequence of Failure	Description	Importance Level	Comment	Robustness Approach from Hewson [16]
Low	**Low** consequence for loss of human life, *or* **small or moderate** economic, social, or environmental consequences	1	Minor structures (failure nor likely to endanger human life)	Not necessary to explicitly consider robustness unless there is an abnormal risk of an extreme event.
Ordinary	**Medium** consequence for loss of human life, *or* **considerable** economic, social or environmental consequences	2	Normal structures and structures not falling into other levels	A minimum set of horizontal ties Notional Element Removal Key Element Design
High	**High** consequence for loss of human life, *or* **very great** economic, social or environmental consequences	3	Major structures (affecting crowds)	A minimum set of horizontal and vertical ties Notional Element Removal Key Element Design
High		4	Post-disaster structures (post disaster functions or dangerous activities)	A minimum set of horizontal and vertical ties Notional Element Removal Key Element Design
Exceptional	Circumstances where reliability must be set on a case-by-case basis	5	Exceptional structures	Outside of the scope of Hewson recommendations [16].

2.4 International Approaches to Structural Robustness

Detailed guidance, as compared to that of the Australian framework, is provided by many of the international design codes relevant to structural robustness. Approaches adopted generally range from prescriptive approaches to key element design and notional element removal. A summary of the most relevant international Standards is presented below.

2.4.1 ASCE 7-16: Minimum Design Loads and Associated Criteria for Buildings and Other Structures

ASCE 7-16 [17] provides structural guidance pertaining to loading and design actions for use in general engineering applications, this Standard is analogous to the AS 1170.0 Standard in Australia. Structural robustness is covered briefly in Sections 2.5 and 2.6 and associated commentary, wherein load cases relevant to key element design and alternative load path analysis are provided. Further guidance surrounding the application of such checks however is not provided.

2.4.2 UFC 4-023-03: Design of Buildings to Resist Progressive Collapse

UFC 4-023-03 [4] provides guidance pertaining to the structural robustness of US defence buildings by the Department of Defence, this standard is particularly relevant to this report as these buildings are often constructed using cold-formed steel. This Standard makes recommendations pertaining to robustness approaches based on a risk category table (Figure 5), which categorises risk by the nature of occupancy (number of inhabitants, facility type, etc.) and then recommends a corresponding robustness approach (Figure 6). Additional details around the description of these risk categories can be found in UFC 3-301-01 Table 2-2 [18]. This Standard employs a variety of different approaches including prescriptive approaches, alternative load path analysis and key element design, guidance pertaining to the application of these approaches is comprehensive.

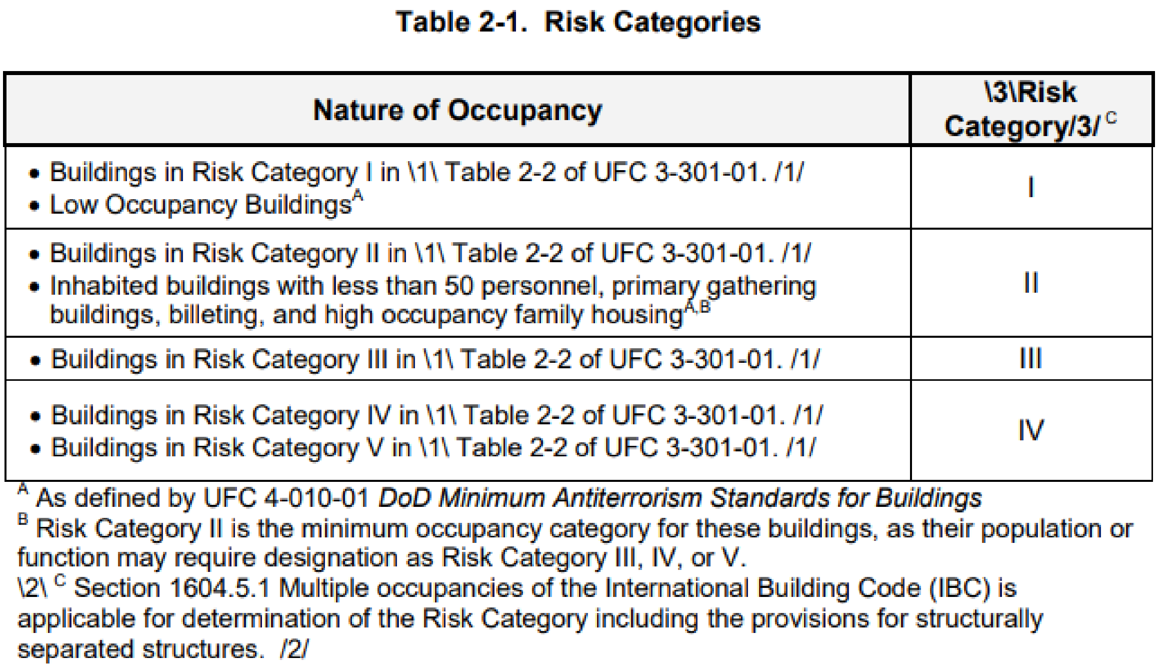


Table 2-1. Risk Categories

Nature of Occupancy	\3\Risk Category/3/ [C]
• Buildings in Risk Category I in \1\ Table 2-2 of UFC 3-301-01. /1/ • Low Occupancy Buildings[A]	I
• Buildings in Risk Category II in \1\ Table 2-2 of UFC 3-301-01. /1/ • Inhabited buildings with less than 50 personnel, primary gathering buildings, billeting, and high occupancy family housing[A,B]	II
• Buildings in Risk Category III in \1\ Table 2-2 of UFC 3-301-01. /1/	III
• Buildings in Risk Category IV in \1\ Table 2-2 of UFC 3-301-01. /1/ • Buildings in Risk Category V in \1\ Table 2-2 of UFC 3-301-01. /1/	IV

[A] As defined by UFC 4-010-01 *DoD Minimum Antiterrorism Standards for Buildings*
[B] Risk Category II is the minimum occupancy category for these buildings, as their population or function may require designation as Risk Category III, IV, or V.
\2\ [C] Section 1604.5.1 Multiple occupancies of the International Building Code (IBC) is applicable for determination of the Risk Category including the provisions for structurally separated structures. /2/

Figure 5 Risk Categories from UFC 4-023-03 [4]

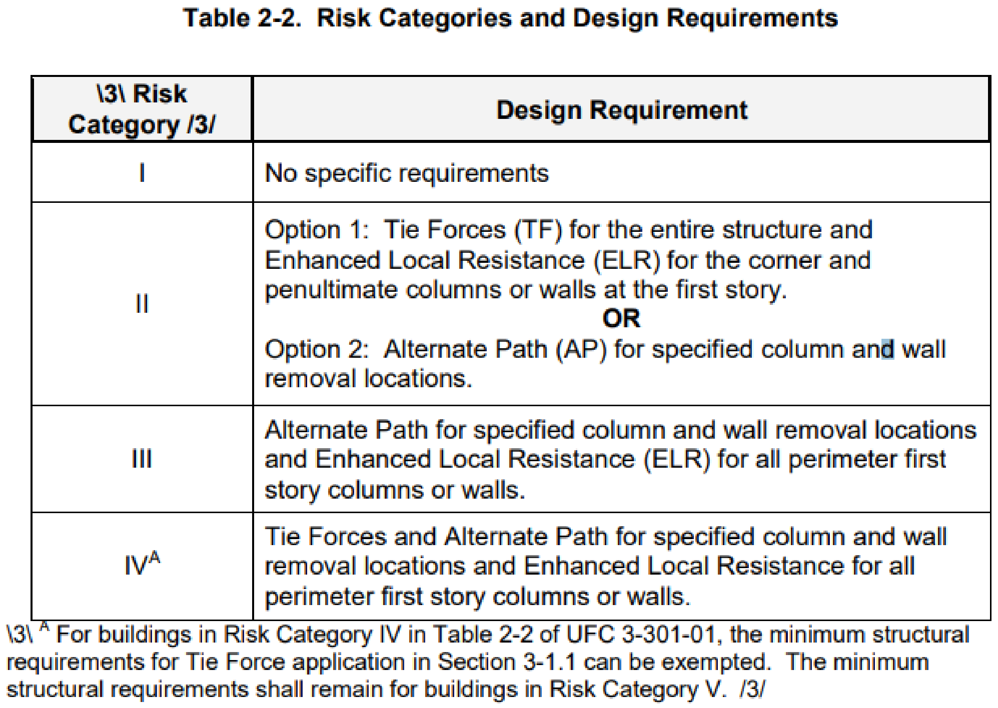


Table 2-2. Risk Categories and Design Requirements

\3\ Risk Category /3/	Design Requirement
I	No specific requirements
II	Option 1: Tie Forces (TF) for the entire structure and Enhanced Local Resistance (ELR) for the corner and penultimate columns or walls at the first story. **OR** Option 2: Alternate Path (AP) for specified column and wall removal locations.
III	Alternate Path for specified column and wall removal locations and Enhanced Local Resistance (ELR) for all perimeter first story columns or walls.
IV[A]	Tie Forces and Alternate Path for specified column and wall removal locations and Enhanced Local Resistance for all perimeter first story columns or walls.

\3\ [A] For buildings in Risk Category IV in Table 2-2 of UFC 3-301-01, the minimum structural requirements for Tie Force application in Section 3-1.1 can be exempted. The minimum structural requirements shall remain for buildings in Risk Category V. /3/

Figure 6 Risk Categories and Design Requirements [4]

2.4.3 GSA Alternative Path Analysis & Design Guidelines for Progressive Collapse Resistance

The General Service Administration (GSA), an independent agency that provides support to the operation of federal agencies in the US, provides structural robustness guidance through the Alternative Path Analysis & Design Guidelines for Progressive Collapse Resistance [19]. This guidance is specifically related to the design of US federal buildings, however, again provides specific guidance for the design of robust cold-formed steel structures. In previous revisions, the GSA included prescriptive approaches, key element design approaches and alternative load path analysis approaches to ensure structural robustness. However, the current GSA guidelines only recommend alternative load path analysis approaches to this end. The design guidance is largely based upon recommendations from both the UFC 4-023-03 [19] and ASCE Standards, as such guidance around application of these approaches is comprehensive.

2.4.4 EN 1991-1-7: Eurocode 1 – Accidental Actions

EN-1991-1-7 [11] provides structural guidance pertaining to the design for accidental loads for use in general engineering applications, making specific recommendations concerning structural robustness based on design context (whether an identified event is being designed for or design is event-independent) (Figure 7) and building class for event-independent design (Figure 8).

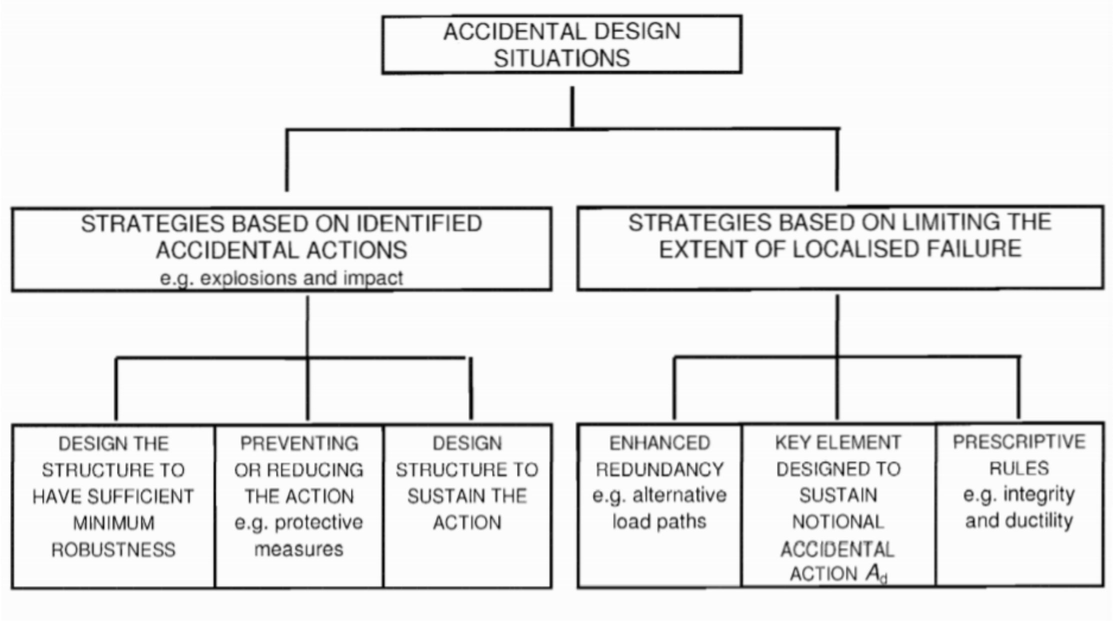


Figure 7 Identified Events and Event-Independent Approaches [11]

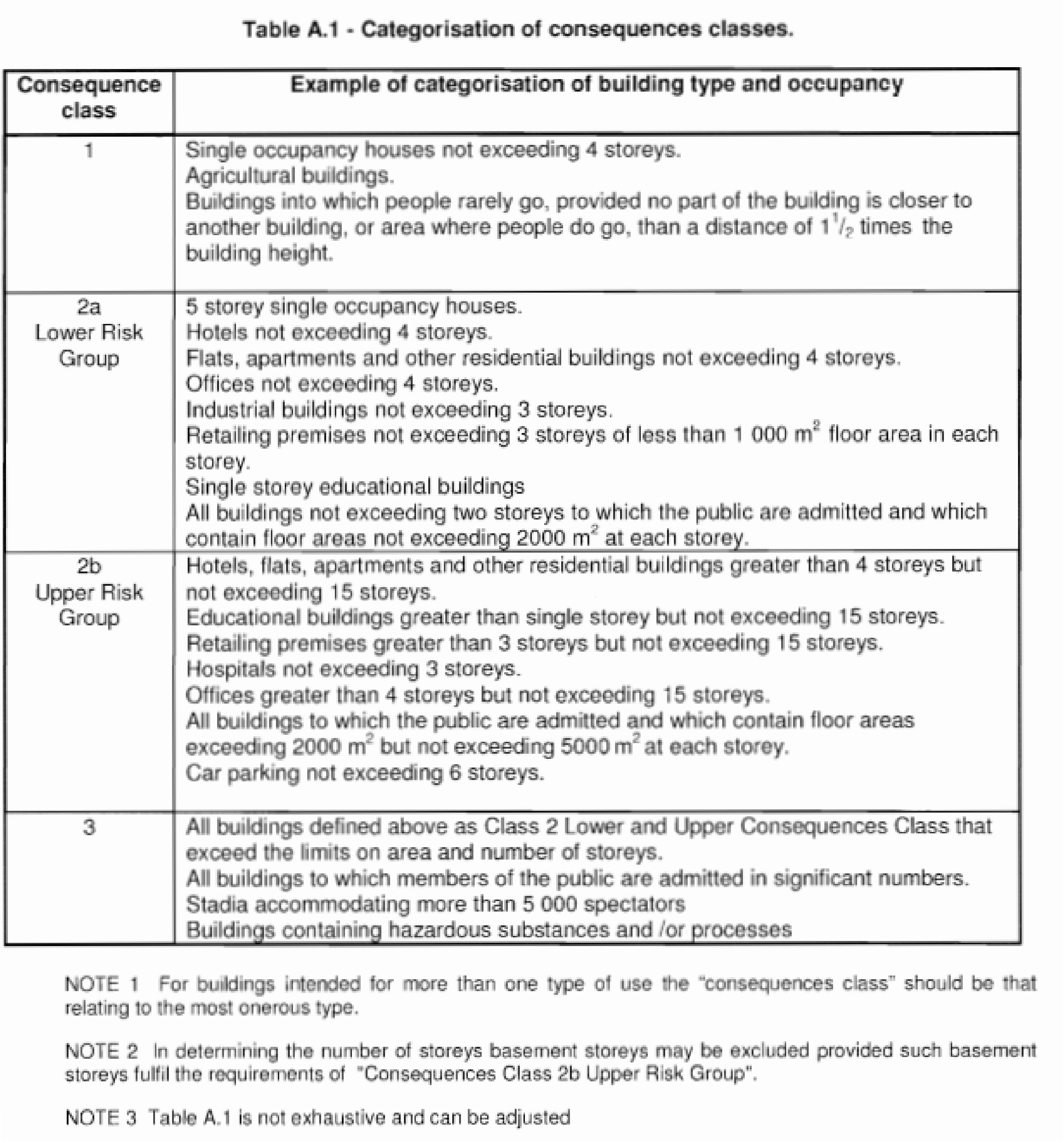


Table A.1 - Categorisation of consequences classes.

Consequence class	Example of categorisation of building type and occupancy
1	Single occupancy houses not exceeding 4 storeys. Agricultural buildings. Buildings into which people rarely go, provided no part of the building is closer to another building, or area where people do go, than a distance of 1 $^{1}/_{2}$ times the building height.
2a Lower Risk Group	5 storey single occupancy houses. Hotels not exceeding 4 storeys. Flats, apartments and other residential buildings not exceeding 4 storeys. Offices not exceeding 4 storeys. Industrial buildings not exceeding 3 storeys. Retailing premises not exceeding 3 storeys of less than 1 000 m^2 floor area in each storey. Single storey educational buildings All buildings not exceeding two storeys to which the public are admitted and which contain floor areas not exceeding 2000 m^2 at each storey.
2b Upper Risk Group	Hotels, flats, apartments and other residential buildings greater than 4 storeys but not exceeding 15 storeys. Educational buildings greater than single storey but not exceeding 15 storeys. Retailing premises greater than 3 storeys but not exceeding 15 storeys. Hospitals not exceeding 3 storeys. Offices greater than 4 storeys but not exceeding 15 storeys. All buildings to which the public are admitted and which contain floor areas exceeding 2000 m^2 but not exceeding 5000 m^2 at each storey. Car parking not exceeding 6 storeys.
3	All buildings defined above as Class 2 Lower and Upper Consequences Class that exceed the limits on area and number of storeys. All buildings to which members of the public are admitted in significant numbers. Stadia accommodating more than 5 000 spectators Buildings containing hazardous substances and /or processes

NOTE 1 For buildings intended for more than one type of use the "consequences class" should be that relating to the most onerous type.

NOTE 2 In determining the number of storeys basement storeys may be excluded provided such basement storeys fulfil the requirements of "Consequences Class 2b Upper Risk Group".

NOTE 3 Table A.1 is not exhaustive and can be adjusted

Figure 8 Building Class Categories for Unspecified Event Robustness [11]

Additionally, EN-1991-1-7 [11] provides specific guidance pertaining to the definition of structural robustness and progressive collapse, localized damage is limited to 15% of floor area or 100 m^2, whichever is lesser. This definition of localised damage is often used outside of this Standard.

2.4.5 International Perspectives on Structural Robustness

Outside of Standards and Guidelines pertaining to robustness internationally, Bita et al. [20] conducted an industry survey of structural robustness in practice. Although this study does not directly investigate LGS structures specifically, it provides insight toward how robustness is addressed generally and in structural forms conceptually similar to LGS structures (timber). In this online survey, a diverse set of 171 individuals from various nationalities (Australia, New Zealand, USA, Canada and Europe) across different professions within the building industry (primarily practising engineers) were surveyed on structural robustness practices. Participants worked with a variety of different building material types (timber, concrete, steel) and building uses (primarily residential, commercial and public buildings). A breakdown of participant's region and primary project material was provided in Figure 9.

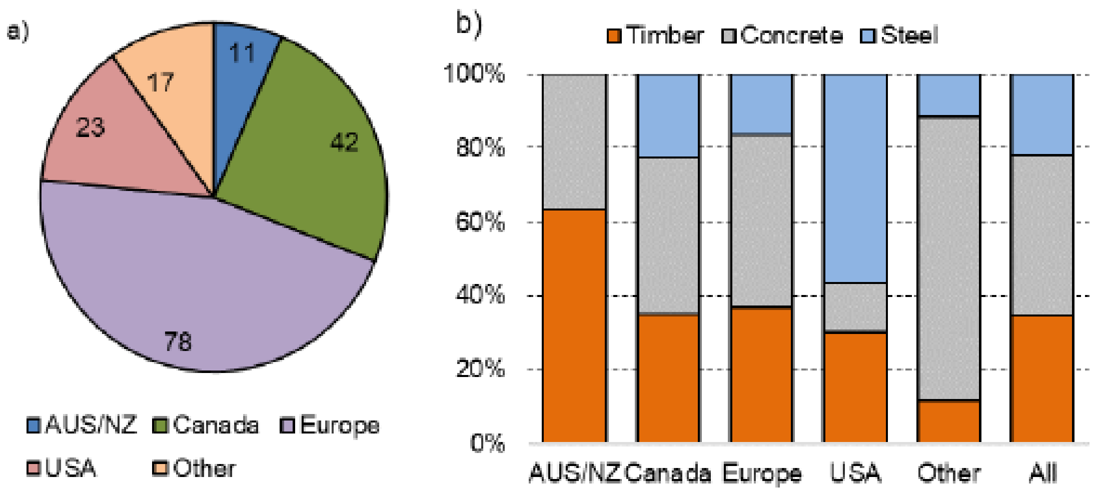


Figure 9 Participant Region and Primary Project Material [20]

Several interesting insights came from this study, firstly, participants considered robustness primarily because it was either best practice for the project or best practice regardless, as opposed to satisfying code requirements. This is represented in Figure 10.

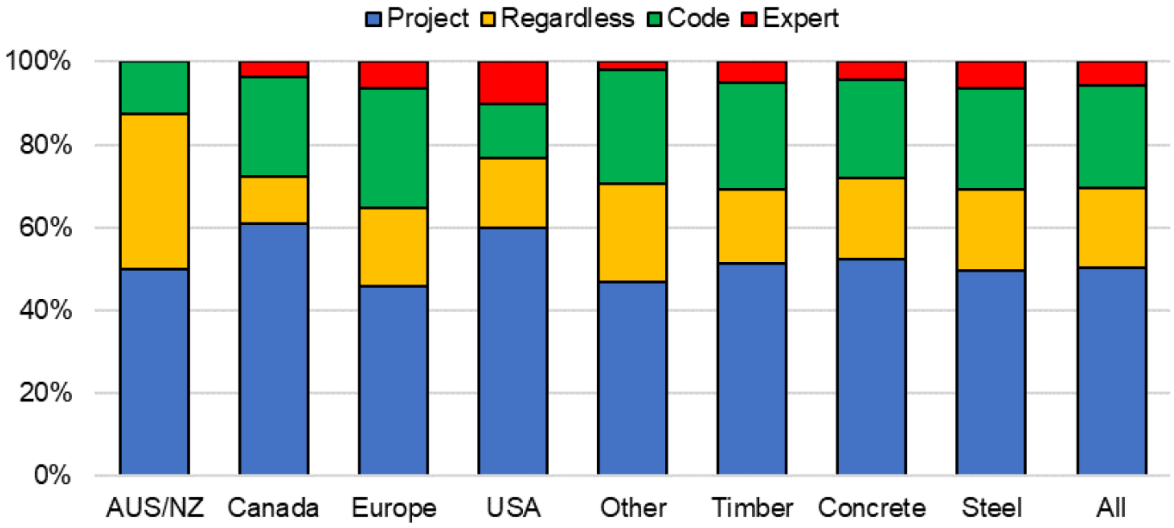


Figure 10 Breakdown of motivations towards considering structural robustness [20]

The level of satisfaction in Australia’s robustness provisions were comparatively low, approximately 70% of Australian participants were not satisfied with the state of current Standards compared to an approximate 40% of participants not satisfied overall from all regions.

When addressing robustness, the dominant approaches applied by participants were linear alternative load path analysis (linear ALPA), tie force approaches, key element design approaches and applying engineer judgement (Figure 11).

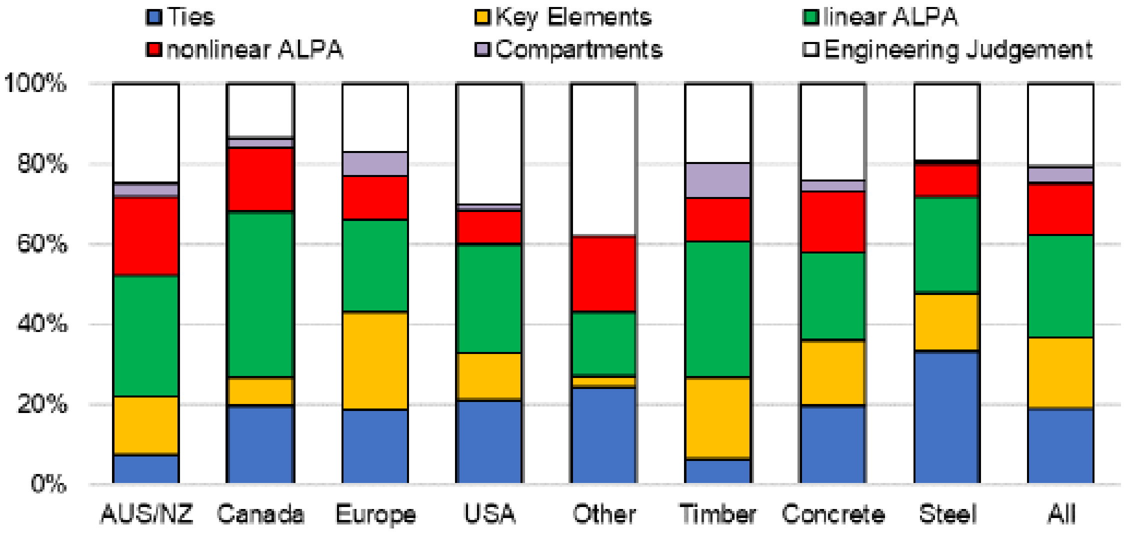


Figure 11 Robustness approaches utilized by participants [20]

Lastly, the most common events underlying consideration for structural robustness were accidents (eg. terrorism, fire, impact, explosion, etc.) and natural catastrophises (extreme earthquakes, wind, etc.), this is illustrated in Figure 12.

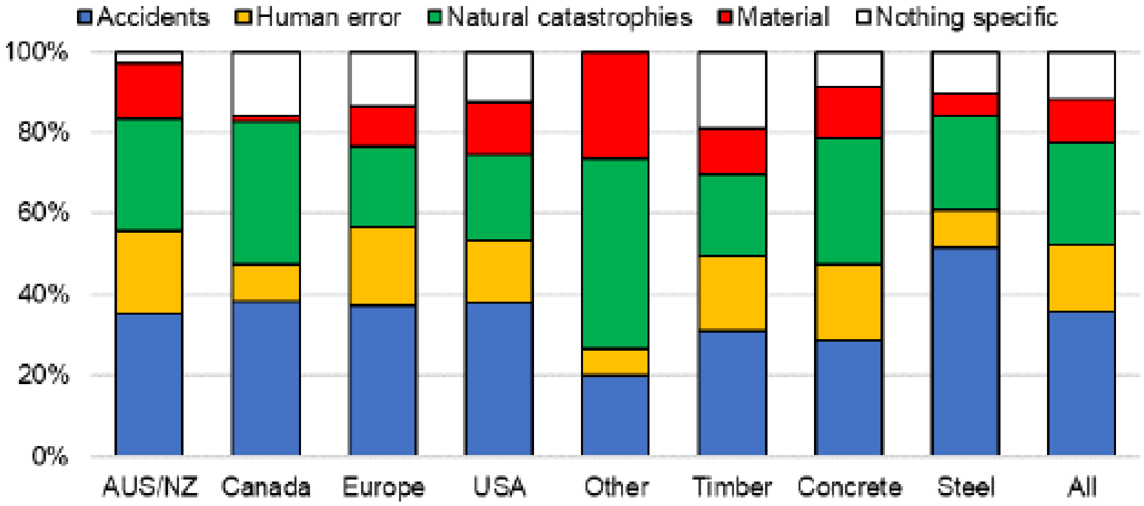


Figure 12 Events underlying consideration for structural robustness [20]

2.5 Cold-Formed Steel Design in The Context of Structural Robustness

As demonstrated by the selection of approaches within international Standards that directly and comprehensively assess cold-formed steel design [4, 19], prescriptive approaches, key element design approaches and notional element removal approaches are preferred.

Notional element removal represents the preferred method for addressing robustness. It is the only method that is represented in both the NCC [10] and all international Standards discussed within this section [4, 11, 17, 19]. Furthermore, alternative load path analysis, in its various forms, is used extensively within industry practice. Lawson et al. [6] argues that CFS structures generally possess a high degree of redundancy and connectivity, lending well to this approach.

With respect to key element design, Lawson et al. [6] argues that this robustness approach can be problematic, at least within Europe, where it is difficult for LGS structures to resist a 34 kPa pressure that is mandated within the Eurocode [11] for this approach. Given the significant number of load bearing elements spreading load in LGS structures, key element design is less preferable.

This method may however be appropriate if some elements are designed to carry a larger load share in the context of extreme events, however this consideration requires further investigation.

With respect to prescriptive approaches, the sufficiency of these approaches to mobilise the last resort collapse mechanisms that implicitly underly this approach (catenary action, membrane action, etc.) is questioned within research literature [7-9]. Despite these concerns, the appropriate application of prescriptive approaches is both time efficient and relatively simple. This approach is also recommended locally under AS 1170.0 [14] and internationally under various Standards [4, 11]. For this reason, its use should be considered within the context of building type and class (as recommended by Hewson [16]) and the sufficiency of these approaches should be investigated further.

Finally, it is important to balance the robustness approaches employed, with the building type and class being considered, as is the case in many of the international Standards and as outlined by Hewson [16] in an Australian context.

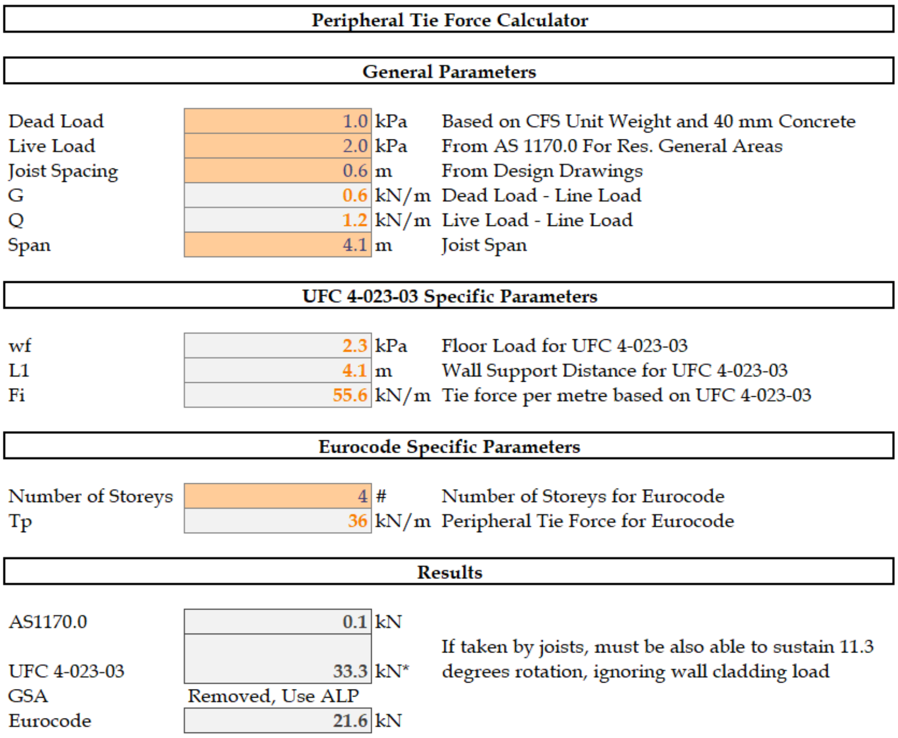
2.6 A Comparison Between Structural Robustness Design Methods

In understanding the differences in employing various robustness approaches, a comparison has been made between the tie-forces mandated within the various local and international Standards and those that result from a preliminary notional element removal analysis.

This comparison in made on a typical CFS structural configuration as outlined in Section 4.1, key details include:

- 4100 mm Joist Spans spaced at 600 mm
- 1 kPa Live Load and 2 kPa Dead Load, Load Factor of G + 0.4Q
- 4 Storeys with Floor-to-Floor Heights of 3100 mm

Tie force provisions for both peripheral (Figure 13) and internal (Figure 14) horizontal ties are made based on AS 1170.0 [14], UFC-023-03 [4] and the EuroCode [11]. These provisions are then compared with the internal tying force demands that are produced by the multi-level modelling in Section 4.3.



Peripheral Tie Force Calculator

General Parameters

Dead Load	1.0	kPa	Based on CFS Unit Weight and 40 mm Concrete
Live Load	2.0	kPa	From AS 1170.0 For Res. General Areas
Joist Spacing	0.6	m	From Design Drawings
G	0.6	kN/m	Dead Load - Line Load
Q	1.2	kN/m	Live Load - Line Load
Span	4.1	m	Joist Span

UFC 4-023-03 Specific Parameters

wf	2.3	kPa	Floor Load for UFC 4-023-03
L1	4.1	m	Wall Support Distance for UFC 4-023-03
Fi	55.6	kN/m	Tie force per metre based on UFC 4-023-03

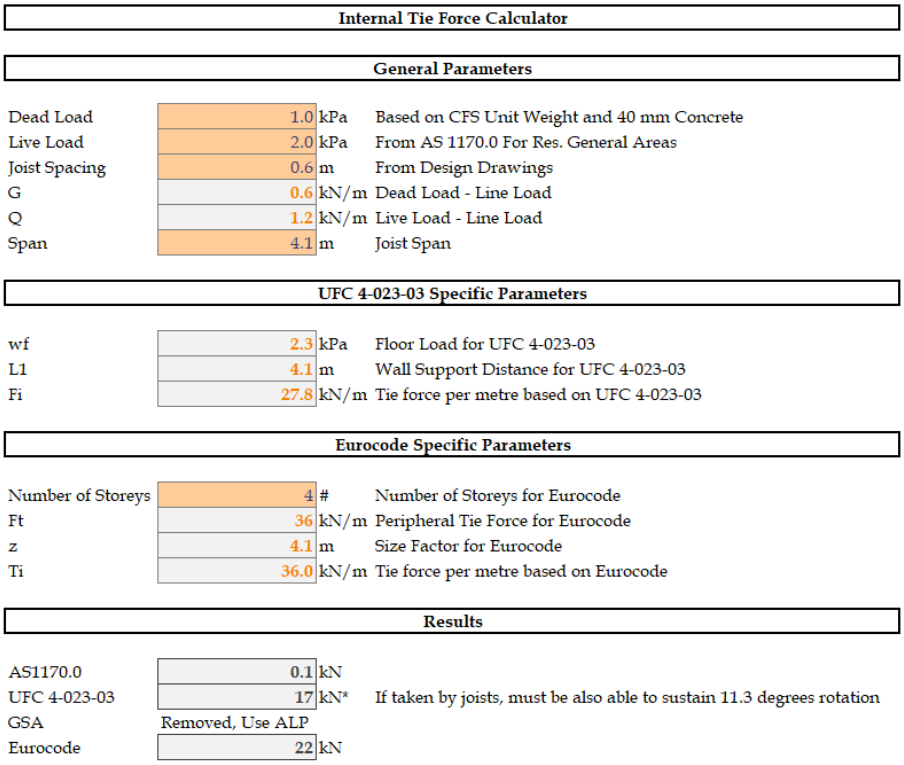
Eurocode Specific Parameters

Number of Storeys	4	#	Number of Storeys for Eurocode
Tp	36	kN/m	Peripheral Tie Force for Eurocode

Results

AS1170.0	0.1	kN	
UFC 4-023-03	33.3	kN*	If taken by joists, must be also able to sustain 11.3 degrees rotation, ignoring wall cladding load
GSA	Removed, Use ALP		
Eurocode	21.6	kN	

Figure 13 Peripheral Tie Force Calculations



Internal Tie Force Calculator

General Parameters

Dead Load	1.0	kPa	Based on CFS Unit Weight and 40 mm Concrete
Live Load	2.0	kPa	From AS 1170.0 For Res. General Areas
Joist Spacing	0.6	m	From Design Drawings
G	0.6	kN/m	Dead Load - Line Load
Q	1.2	kN/m	Live Load - Line Load
Span	4.1	m	Joist Span

UFC 4-023-03 Specific Parameters

wf	2.3	kPa	Floor Load for UFC 4-023-03
L1	4.1	m	Wall Support Distance for UFC 4-023-03
Fi	27.8	kN/m	Tie force per metre based on UFC 4-023-03

Eurocode Specific Parameters

Number of Storeys	4	#	Number of Storeys for Eurocode
Ft	36	kN/m	Peripheral Tie Force for Eurocode
z	4.1	m	Size Factor for Eurocode
Ti	36.0	kN/m	Tie force per metre based on Eurocode

Results

AS1170.0	0.1	kN	
UFC 4-023-03	17	kN*	If taken by joists, must be also able to sustain 11.3 degrees rotation
GSA	Removed, Use ALP		
Eurocode	22	kN	

Figure 14 Internal Tie Force Calculations

These code-mandated tie-force provisions for internal ties (ranging from 0.1 kN to 22 kN), can be compared to that of the internal tying force demands on joists within Section 4.3, producing 112 kN of tying force demand in critical regions.

This discrepancy is currently under investigation, the potential candidates for resolution may lie within the connection stiffness assumptions currently within the modelling in Section 4.3. However, as discussed within previous sections, the adequacy of tie-force provisions supplying a structure with enough strength and ductility to resist the demands imposed by last resort collapse mechanisms like catenary action remains in question [7-9].

A summary of these comparisons in made is in Table 2 and Table 3.

Table 2 Summary of Peripheral Horizontal Tie Force Provisions

Peripheral Horizontal Tie Force Results	
AS 1170.0 [14]	0.11 kN
UFC-023-03 [4]	33.3 kN
EuroCode [11]	21.6 kN

Table 3 Summary of Internal Horizontal Tie Force Provisions

Internal Horizontal Tie Force Results	
AS 1170.0 [14]	0.11 kN
UFC-023-03 [4]	16.7 kN
EuroCode [11]	21.6 kN
Alternative Load Path Analysis (Section 4)	112 kN

2.7 Summary

This section of the report provides an outline of the general approaches used when addressing structural robustness and those most relevant to LGS structures, namely alternative load path analysis and prescriptive approaches. A summary of some of the key recommendations associated with these methods, as applied to a representative LGS structure is presented in Section 2.6. In general, tie-forces mandated across various international Standards for this representative structure lie approximately between 20 – 30 kN whereas alternative load path analysis suggests this demand may be on the order of 112 kN (further refinement of this modelling is needed however, as discussed in Section 4).

3. Review Structural Systems

3.1 Introduction

Cold-formed steel (CFS) has become a popular material for constructing single and multi-storey residential and commercial buildings owing to its lightweight and lower cost. The design guidelines to comply the robustness requirement for steel, concrete and timber buildings have been specified by various design codes/bodies as discussed earlier in Chapter 2. However, considering that the CFS is still an emerging material and design guidelines to comply the robustness requirements of CFS buildings are not explicitly specified in existing design codes, there has been some research undertaken to explore this topic. This section presents an extensive review of the structural systems to comply the robustness requirement of the mid-rise CFS buildings. Both experimental and numerical work performed to investigate the progressive collapse of the CFS structures are reviewed with particular attention is given to the investigations performed on the connections systems to improve the robustness of the mid-rise CFS buildings.

3.2 Current Practice

Considering the lack of specific design guidelines for the design of robustness of CFS structures, the design guidelines for the robustness of structural steel buildings have been used in current practice instead. According to the practice, of the few methods specified by the existing codes, tying and alternative load path (ALP) approaches are the two most widely adopted design methods to meet the robustness of the CFS buildings. However, it is also a common practice in many construction projects to have CFS and structural steel framing or concrete on the same project, as illustrated in Figure 15, where

- structural steel or concrete is used for the ground floor podium to accommodate the large open areas for amenity space and
- CFS is used for the unit floors above.

Therefore, the design guidelines specified for the robustness of structural steel or concrete are also used in the robustness design of such CFS buildings.

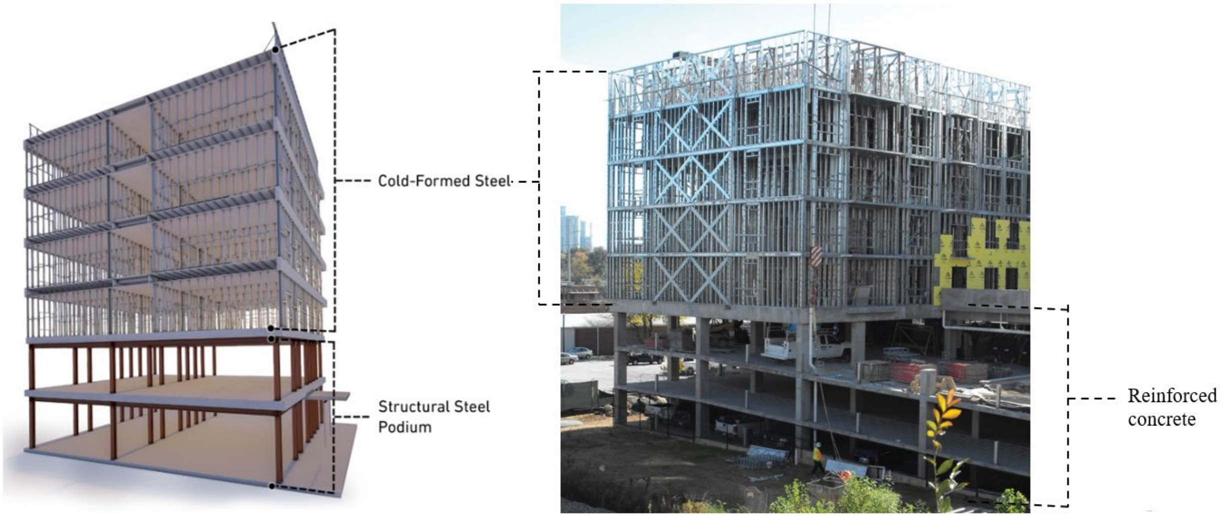


Figure 15 Construction of CFS buildings in conjunction with structural steel framing or reinforced concrete (image source: Google).

The previous chapter presents an example of design calculation of robustness of CFS structures using the tie method. In this section, the investigation of the progressive collapse of a three-story CFS structure carried out by Bae et al. [9] is presented as an example of complying the robustness design of CFS buildings in practice using the ALP approach. Bae et al. [9] considered a three-story prototype army barrack made of CFS for their study. The plan dimension of the building was taken as 50.3 m × 17.1 m with 2.9 m floor to floor height. The structure was designed to be able to carry a combination of dead load of 0.96 kPa (roof), 2.16 kPa (floor), 0.479 kN load of walls (exterior, interior, windows); a combination of the live load of 1.44 kPa (roof snow load), 1.92 kPa (rooms) and 3.83 kPa (corridor). The building was designed to be able to sustain a wind load of 140 km/h and designed for the seismic load for seismic zone 4. All joist and stud members with 1.4 mm thick had a yield strength of 345 MPa whereas members with 1.1 mm thickness and lighter had a yield strength of 228 MPa. The 3D view of the CFS structures is shown in Figure 16.

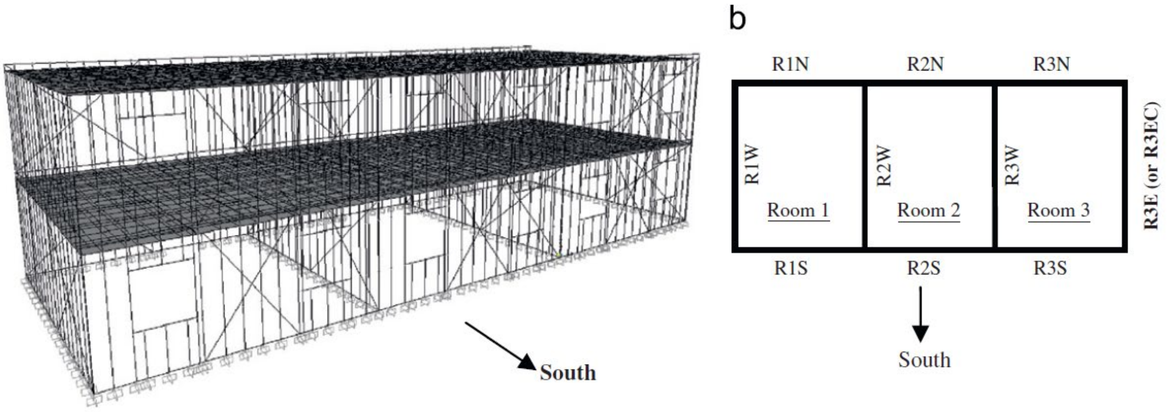


Figure 16 3D view of the CFS structures analyzed by Bae et al. [9]

Five different case scenarios were considered to study the robustness of the CFS structure. Case 1 and 2 were to study the robustness criteria according to the guidelines given by General Service Administration (GSA) [19] and Cases 3 and 4 were according to the guidelines specified by the Department of Defence (DoD) [4]. According to the GSA, the progressive collapse of structures should be checked for the removal of the components at the ground floor only whereas according to the DoD guidelines, the ALP method should be applied for each floor. In addition, according to GSA, the amplified load was applied to all bays whereas, for DoD, the amplified load was applied only to the bay where the CFS components were removed. Figure 17 (a, b) represents Cases 1 and 2 whereas Figure 17 (c, d) represents Cases 3 and 4. In Case scenario 5, the possibility of progressive collapse was investigated using the ALP approach by removing the members damaged in Cases 1-4. For Cases 1 and 3, the exterior wall on the ground floor at the middle was removed whereas for Cases 2 and 4, the corner walls of the structures in each direction of the south-east corner were removed.

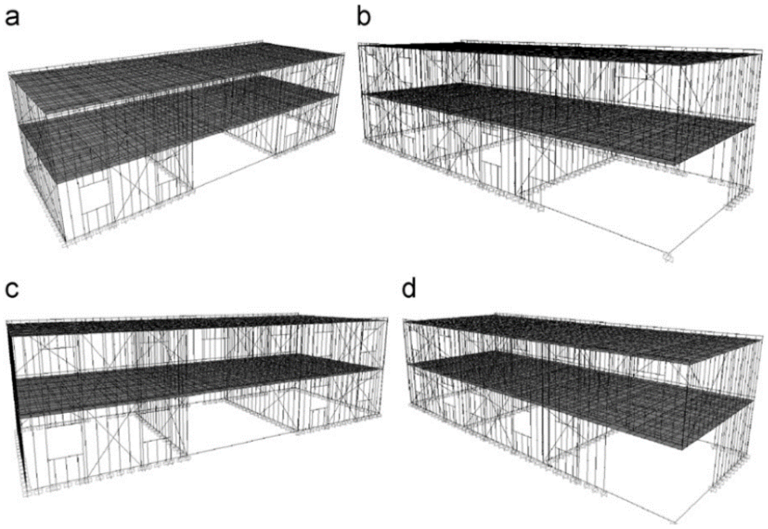
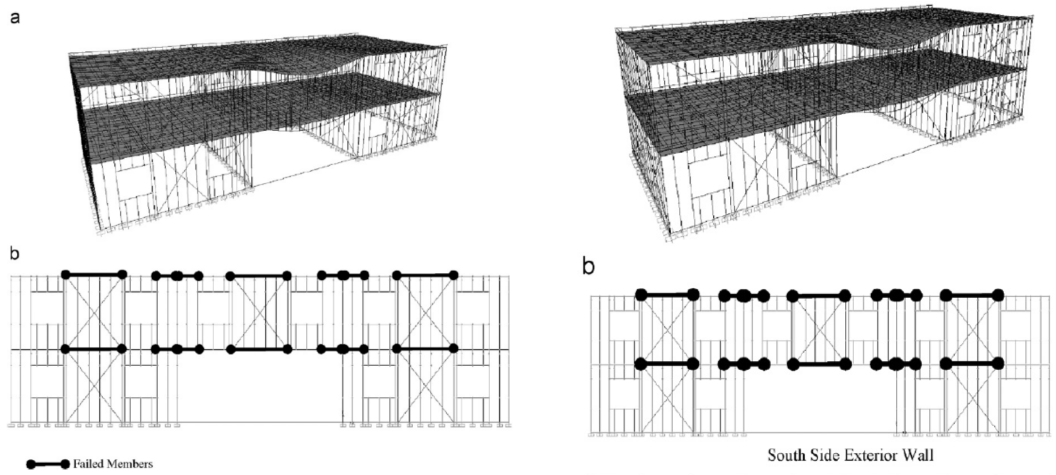


Figure 17 Case scenarios considered by Bae et al. [9].

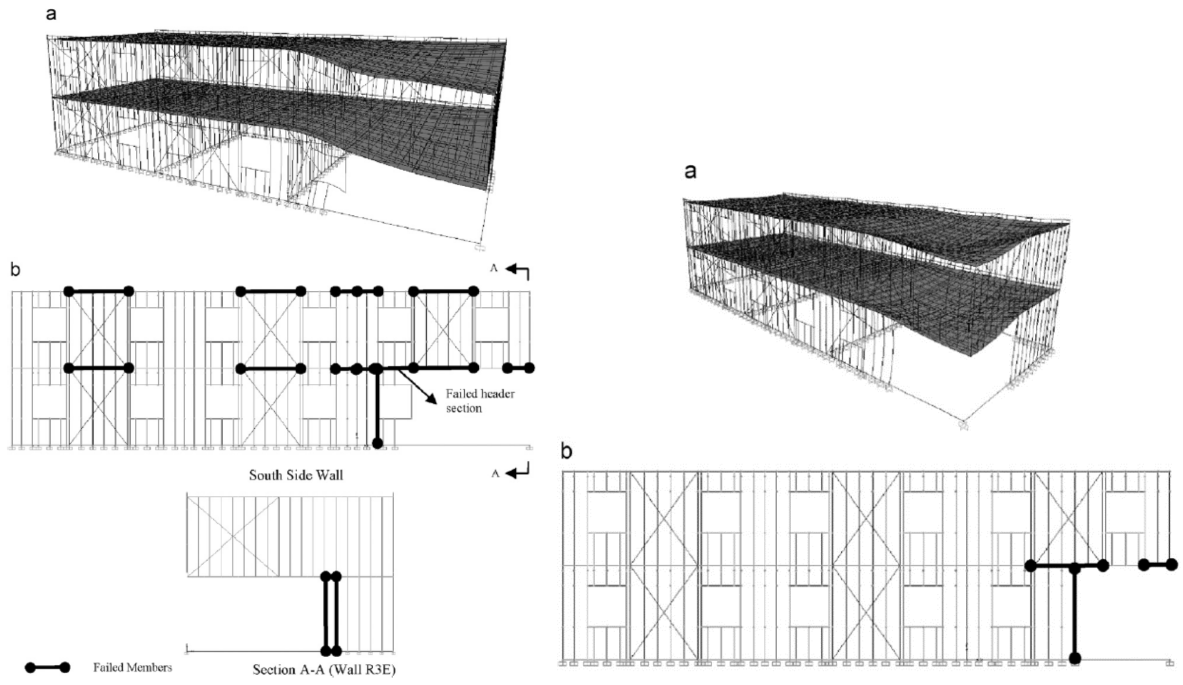
FE analysis showed that Cases 1 and 3 had identical failure modes where the track sections of CFS structures were failed due to the removal of stud columns, as illustrated in Figure 18. This is because the track sections of CFS structures are not designed to resist a moment. Due to the failure of the track sections at the middle of the structures, the wall section on the immediate upper floor will be assumed to be failed. It was recommended by the authors to design the track sections as structural members likewise header sections.



(a) (b)

Figure 18 Failure of CFS structures for (a) Case 1 and (b) Case 3 [9].

The failure modes due to Cases 2 and 4 are shown in Figure 19. For the removal of corner walls, the large deformations directly above the removed walls were found to be significantly higher. In addition, some of the track sections and stud columns were found to be damaged due to the removal of corner walls. According to GSA, these damaged members should be redesigned to enable them to resist the structure from progressive collapse. For Case 4, due to the lesser length of the removed wall, the failure of the members is lesser compared to Case 2, as seen in Figure 19 (b).



(a) (b)

Figure 19 Failure of CFS structures for (a) Case 2 and (b) Case 4 [9].

Based on observed results from FE models, the damage caused to members among all four cases (1-4), Case 2 was found to be severe. Therefore, the initial damage of Case 2 was chosen as the initial damage for initiating the analysis of Case 5. Iterative FE modelling of the CFS structure with simultaneous removal of the damaged members stepwise was performed in Case 5. Figure 20 and Figure 21 show the progressive collapse analysis of the CFS structure for Case 5, where the number in the members represents the sequence of collapse.

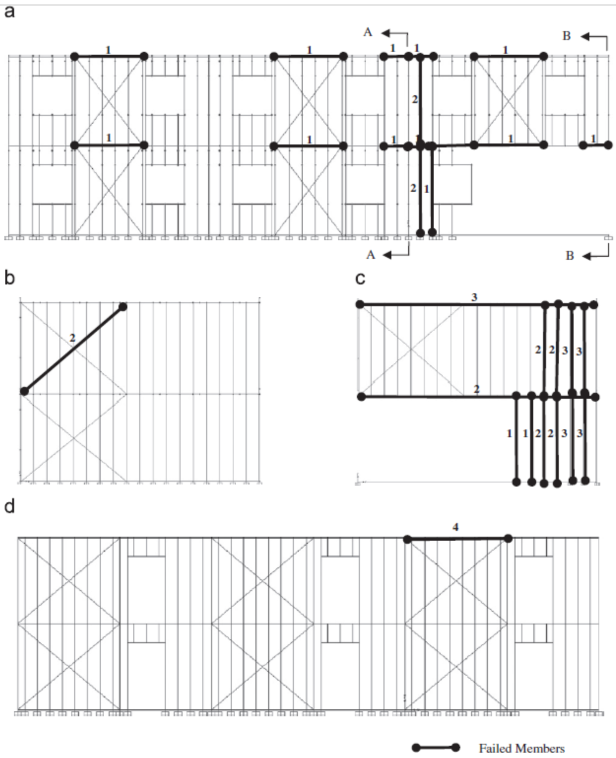


Figure 20 Sequence of the failure of members for Case 5 [9].

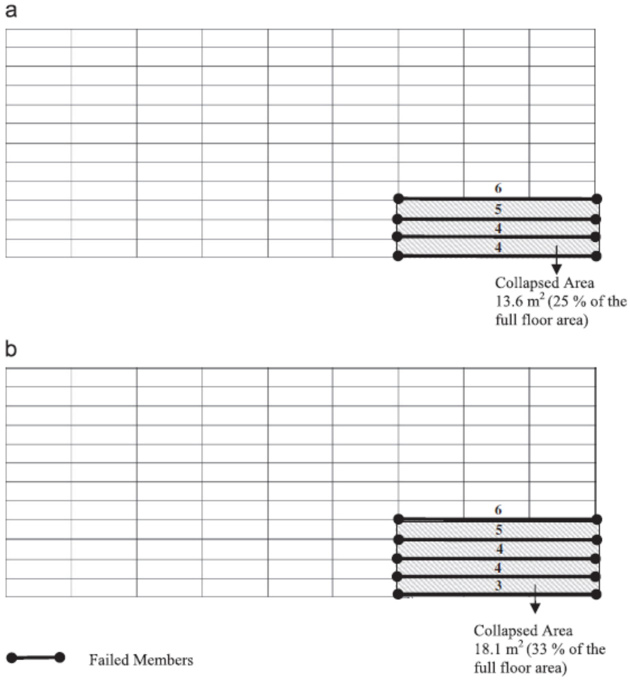


Figure 21 Sequence of the failure of the joists for Case 5: (a) second-floor joist beam plan and (b) third-floor joist beam plan [9].

The collapsed area of the CFS structure at the second and third floors was found to be higher than the recommendation given by guidelines (less than 15%). It was concluded that although the structure did not progressively collapse, some parts of the CFS structure were found to be susceptible to progressive collapse. Furthermore, to investigate the possibility of catenary action upon removal of stud columns, a 2D FE model was also developed to calculate the axial force and compare it against the capacity of the horizontal screw connection. A load that caused a similar moment-curvature as obtained in the 3D model was distributed over the 2D members in the modeling. The analysis results showed the catenary action developed was about 100.9 kN, remarkably larger than the capacity of the screw connection, calculated as 16 kN, this can cause the failure of the stud-track connections prematurely. Therefore, it was recommended to redesign the connection to resist the large tensile force generated due to the removal of the components of CFS structures. However, this study ignored the influences of concrete slabs and insulations on the FE modeling thus providing conservative results.

3.3 Connections

To enable the structure to develop an alternative path or sufficient tie forces in order to resist progressive collapse in the scenario of a notional element removal, connection detailing of CFS members, particularly in beam-to-column connection, is integral. In the scenario of notional element removal, CFS beams are subjected to large deformations. In order to resist progressive collapse by mobilizing alternative load paths, a large tensile force is generated at the connections. Therefore, it is crucial that connections are adequately designed to accommodate this large tensile force.

Chung and Lau [21] carried out an experimental program to study the performance bolted moment connections. Gusset plates made of either hot-rolled or cold-formed steel were proposed to connect the CFS beam to the column to form moment connections, as shown in Figure 22. Three different configurations of gusset plates (i.e. triangular, rectangular and haunched) made of both hot-rolled and cold-formed steel were examined to study the influences of geometry and materials. All the primary members were made of two-lipped C sections with 150 mm depth, 64 mm flange width and 1.6 mm thick. The secondary members were made of a single lipped C section with 100 mm section depth. The moment connections for the webs to the gusset plates were made through 16 mm diameter grade 8.8 bolts. At the intersection, two 12 mm diameter bolts with a grade of 4.6 were used.

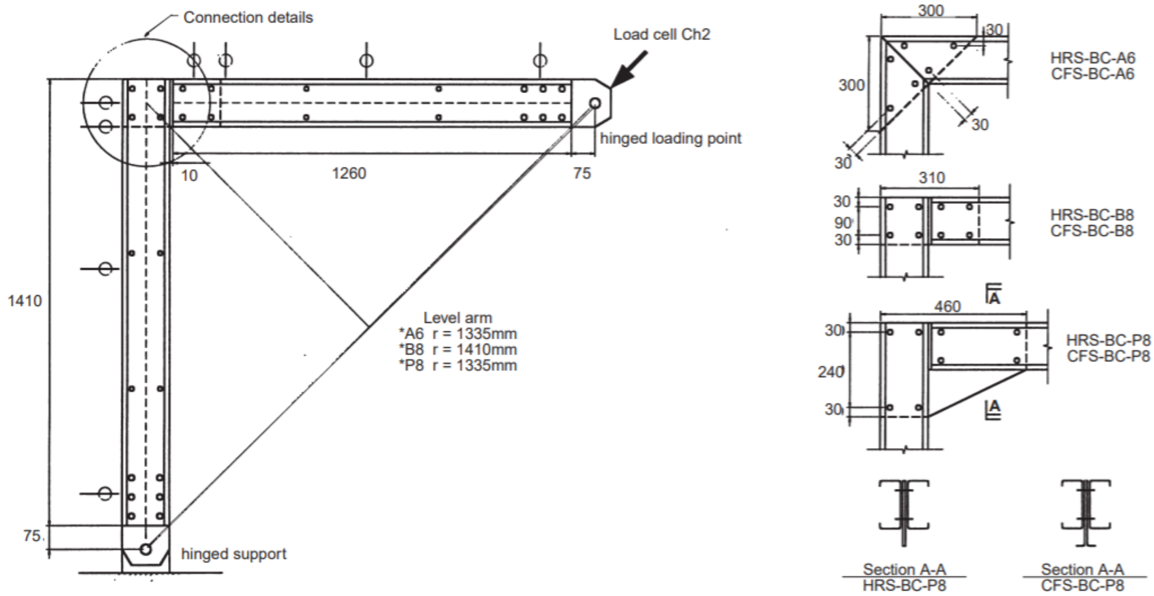
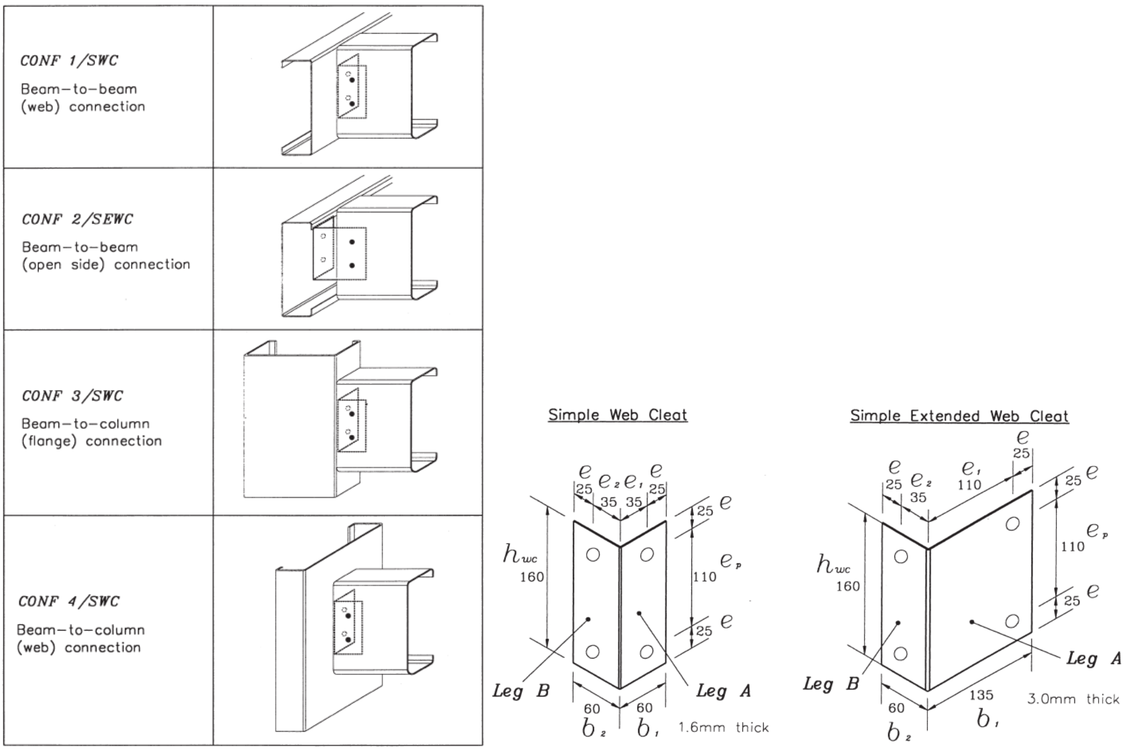


Figure 22 Details of the beam-column connection of CFS members [21].

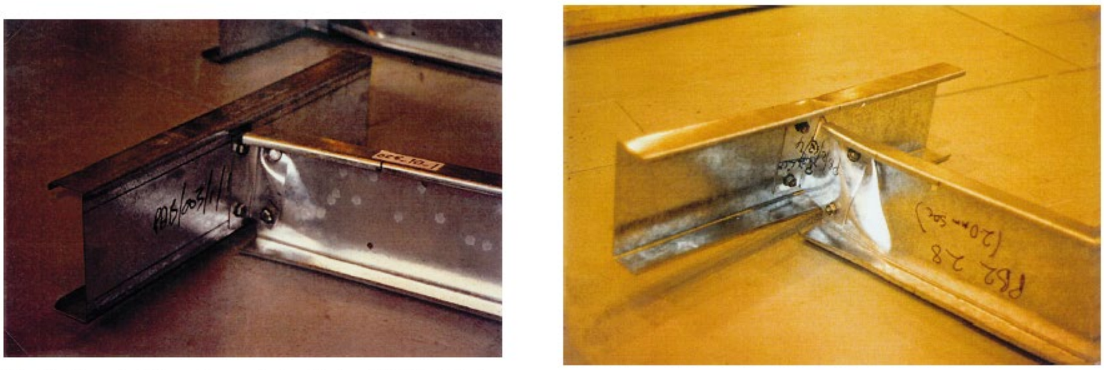
Test results showed that triangular and rectangular gusset plates had lower moment resistance compared to the haunched gusset plate. In general, the moment resistance of gusset plates of hot-rolled steel was found to be always higher than cold-formed steel. Furthermore, the common cause of failure of gusset plates of CFS steel was twisting and lateral bending. For beam-column connection with haunched gusset plate, the moment resistance was over 80% for both hot-rolled and cold-formed steel. Chung and Lau [21] also performed tests on seven full systems of the CFS portal frame incorporating rectangular and haunched gusset plates to connect CFS beams to the column. It was found that haunched gusset plates were very effective, attained moment resistance up to 84% and 77% of the moment capacity of the connected members with hot-rolled steel and cold-formed steel gusset plates, respectively. On the contrary, the rectangular gusset plates were found to be the least efficient in moment connections which attained only a moment resistance of 45% of the moment capacity of the connected members.

Chung and Lawson [22] proposed four different configurations of web clip angle connectors made of CFS strips, as shown in Figure 23, as shear-resisting connections for CFS members and carried out tests to examine their structural performance. Single web clips (SWC) and single extended web clips (SEWC) with a thickness of 1.6 mm and 3.2 mm, respectively were used. Among the four configurations, two were proposed for the beam to beam connections whereas the other two were for the beam to column connection. In addition, three different fasteners (M12 bolts, 4.75 mm diameter blind rivet and 5.5 mm diameter self-drilling screw with hexagonal head) were used to connect the web clips to the CFS members. The CFS members were made of lipped C sections with a dimension of 200 × 65 × 13 × 1.6 mm thick Z28 steel and 200 × 65 × 13 × 2 mm thick Z28 steel with a yield strength of 321 and 309 MPa, respectively. The dimension of the single clip angles was 60 × 60 × 160 × 1.6 mm thick with a yield strength of 305 MPa while the dimension of the single extended web clips was 135 × 60 ×160 × 3.0 mm thick having a yield strength of 362 MPa. Test results showed that the common failure of the connections was due to the shear buckling of the web clip or at the web of the beam (see Figure 24a) and lateral-torsional buckling of the single extended clips (see Figure 24b). The failure of the fasteners was observed as the shear failure and pull-out failure of screws and rivets. Test results showed that for the beam to beam connection, the shear resistance of configuration 1 (SWC) was higher than configuration 2 (SEWC) for all three fasteners. This was because the SEWC was failed due to lateral-torsional buckling. For the beam to column connection, the shear resistance of configuration 4 (SEWC) was higher than configuration 3 (SWC) for bolt and rivet connections. With regards to the fasteners, among the three, blind rivets provided the least shear-resisting connections about 25% of those connections with bolts.



(a) configurations (b) clip angles

Figure 23 Details of the configurations of connections [22].



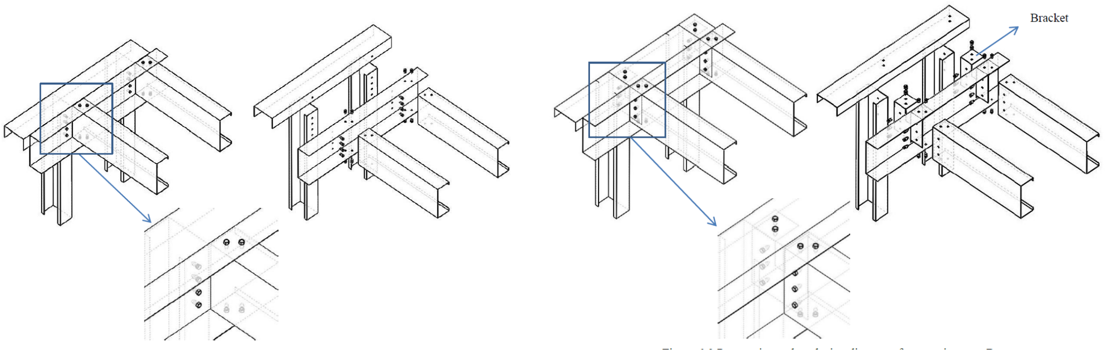
(a) configuration 1 (b) configuration 2

Figure 24 Failure modes of web clip angles reinforced CFS connections [22].

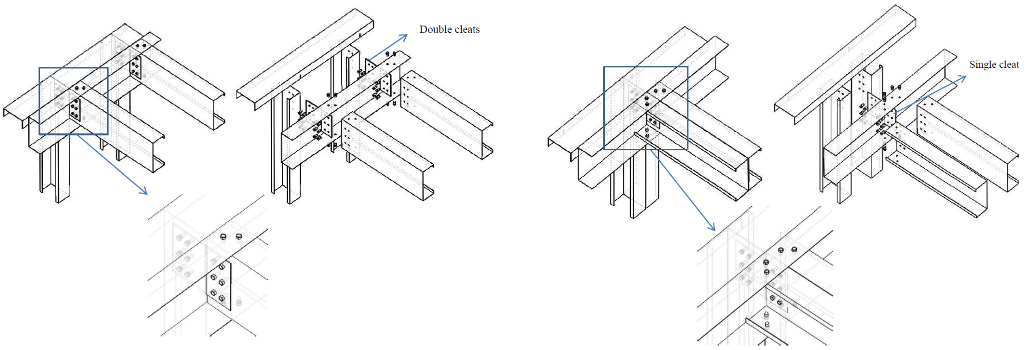
Yu et al. [23] also carried out tests to examine the performance of CFS connections reinforced with web clear angles subjected to shear force. The dimension of the CFS stud column was 508 mm long, 1.37 mm or 2.99 mm thick. One clip was attached to each side of the column using self-drilling screws whereas the anchored leg of the clip angle was fixed to a loading plate by button head socket cap screws. The thicknesses of clip angles varied from 0.84 to 2.46 mm. Test results showed that local and distortional buckling was the common failure of the web clip angles which was dependent on the plate slenderness ratio, as illustrated in Figure 25. It was found that thick clip angles with small aspect ratios lesser than 0.8 showed a local buckling governed failure whereas for thin clip angles with aspect ratios greater than 0.8, distortional local buckling governed failure was observed. In addition, increasing the thickness of the clip angle was found to increase the shear resistance of the connections.

Figure 25 Failure modes of web clip angle reinforced CFS connections [23].

Zhang [24] investigated the effects of various CFS connections including purlin-to-sheeting, stud-to-track and joist-to-post connections on their structural performance. Four different configurations of CFS connections, as illustrated in Figure 26 were chosen to investigate their effects on the performance of CFS members under shear, bending and tension. In connection type A, load from joists is transferred to track through screw joints in the flange. Four screws are used to attach the track to the post. In connection type B, a bracket with a thickness of 3 mm was incorporated into the webs of track and post using screws. In connection type C, there were two rectangular clips whereas, in connection type D, there was only one clip. Test results showed that the tension resistance of connection type B was about 6 times the tension resistance of connection type A. Similarly, the shear resistance of connection type B is about 54% higher than that of connection type A. On the other hand, the tension resistance of connection types C and D was found to be very similar, although, the displacement of connection type D is higher than any other configuration. The tension, shear and bending resistance of connection types C and D were much higher than connection type B. The tension resistance of connections type C and D was almost double than that of connection type B. However, the shear and bending resistance of connection type D were significantly higher than connection type C, estimated as almost 3 times the shear and bending resistance of connection type C.



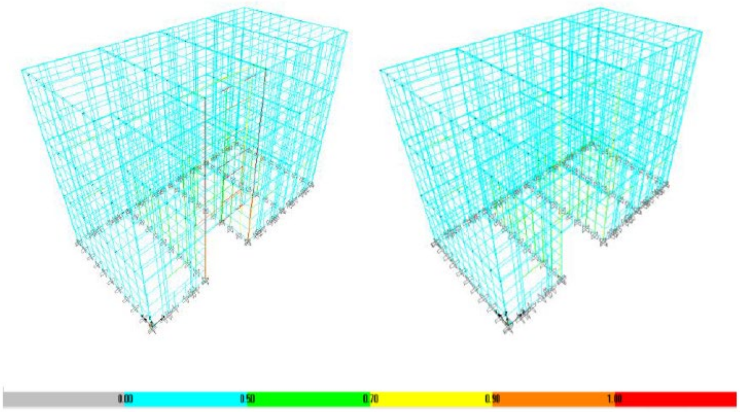
connection type A connection type B



connection type C connection type D

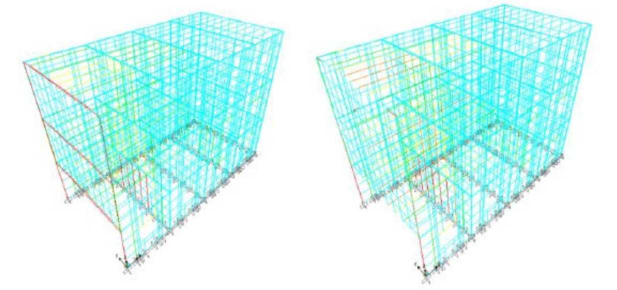
Figure 26 Details of the connections of CFS members [24].

Zhang [24] further investigated the influences of connections on the progressive collapse resistance of CFS panels. A three-floor four-bay residence hall composed of a 50 mm thick concrete slab with a reinforcement ratio of 0.15%, considered in FE modelling using SAP2000v14.1. A dead load of 0.9 kPa (roof), 1.16 kPa (floor), 0.235 kPa of insulator, 1 kPa (windows, interior walls); a combination of the live load of 1 kPa (roof), 2 kPa (floor) and 1.44 kPa (snow) was considered in the modelling. Two different case scenarios were analysed: the removal of the stud column at the middle of the structure (Case 1) and the removal of the corner wall (Case 2). All four configurations of connections proposed were used to study the effects of connections on the progressive collapse of CFS structure. However, for Case 2, the progressive collapse was found to be more severe for connection type C, which was the stiffer connection among all four configurations of the connections. It was concluded that the structure with a stiffer connection is more susceptible to progressive collapse regarding the scenario of the removal of the external wall. The collapsed area was calculated as 27.5% for the first floor and 25% for the second and third floors.

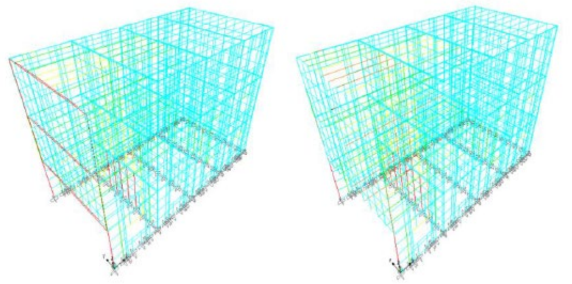


a) Initial result b) Final result

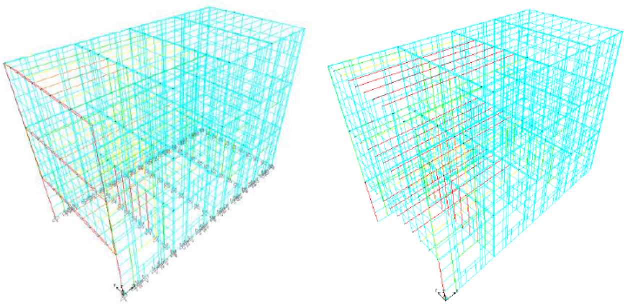
Figure 27 Progressive collapse of CFS members for Case 1 [24].



a) Initial result with connection A b) Final result with connection A



c) Initial result with connection B d) Final result with connection B



e) Initial result with connection C f) Final result with connection C

Figure 28 Progressive collapse of CFS members with a different configuration of connections for Case 2 [24].

Zhang [24] utilized the FE results (shear, tension and rotation properties) of proposed connection types to study the influence of resistance of connections in the progressive collapse of CFS building using a 2D frame structure. The influences of the concrete slab were ignored and the horizontal movement of the structures was restrained. The FE results showed that plastic hinge was developed for connection types B and D, thus, developed a catenary action. An alternative load path appeared in the FE modeling based on the observation of the higher axial tensile force acting on the joists followed by the removal of CFS elements. At the initial loading, the middle post of the CFS structure carried a higher load than other members. Among all the configuration of connections, connection type D had the greater load carrying capacity, however, considering the amount of steel consumption, connection type C was proposed as the most efficient connection type for connecting joist members to posts.

Natesan and Madhavan [25] examined the effectiveness of CFS clip angle bolted connections to subjected to shear force. The test parameters include the aspect ratio and thickness of the clip angles. The thickness of the clip angles varied from 1.5, 2 and 2.5 mm whereas the clips' aspect ratio varied from 0.18 to 0.963. The loaded beam was 1000 mm long and the two columns were 400 mm long. The clip angles were connected to the CFS members using M12 bolts, as illustrated in Figure 29. Tets results showed that the shear resistance of CFS clip angle bolted connections increased with the increase of the thickness of the clip angles but the shear resistance decreased as the aspect ratio increased. Similarly, the stiffness of the clip angles reduced as the aspect ratio increased. Test results showed that the common failure modes of thick clip angles with small aspect ratios lesser than 0.8 were due to the local buckling whereas, for thin clip angles with aspect ratios greater than 0.8, the failure was due to the distortional local buckling, as can be seen in Figure 30. Similar test results were observed by Yu et al. [23]. It was recommended to increase the number of bolts in the clip angle leg connected to the column web for a higher depth of clip angle to reduce the bearing deformation of the bolt hole observed in the test study.

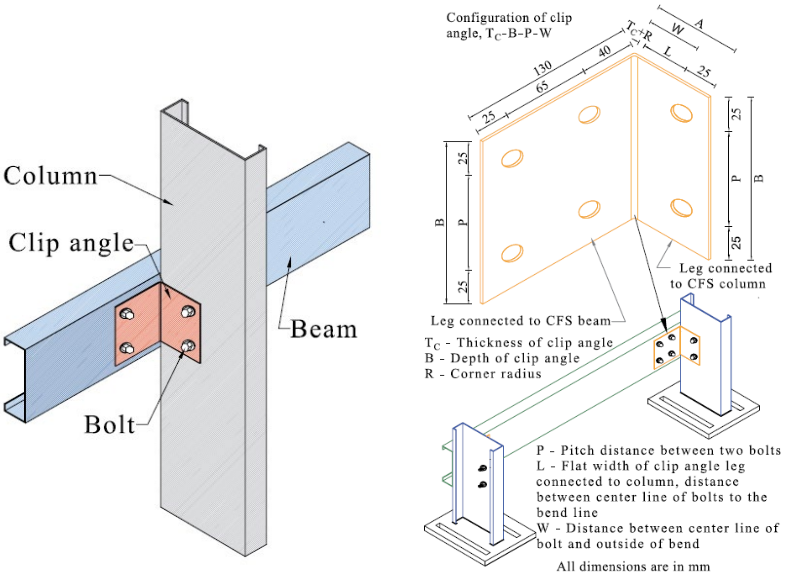


Figure 29 Schematic view of the clip angle reinforced beam to column connection tested by Natesan and Madhavan [25].

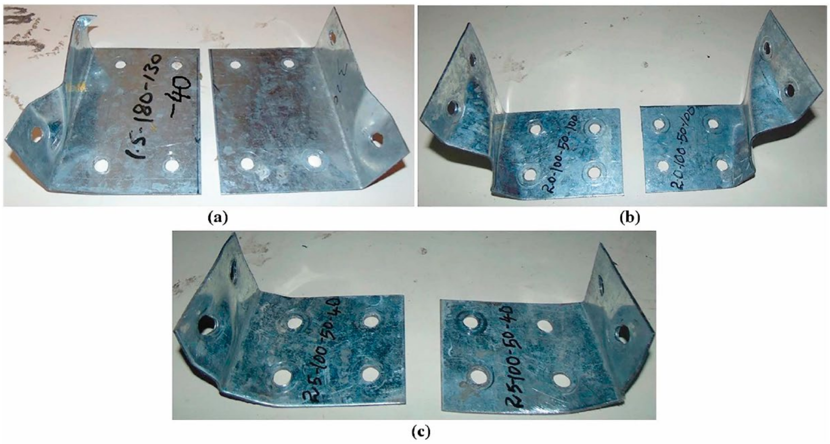


Figure 30 Failure modes of the clip angles observed in the test study [25].

Natesan and Madhavan [26] further extended their previous studies [25] to investigate the effectiveness of beam-to-beam connection using a combination of clip angle and flange strip. The flange strip was connected using self-drilling screws flange whereas the clip angle was connected to the web using bolts, as shown in Figure 31. Test parameters include different aspect ratios and the two different thicknesses of the clip angle of 0.8 and 1.5. In addition, the number of holes in the loaded leg also differed between 2 and 3 (Figure 32). The loaded channel was 780 mm long and the two anchored channels were 390 mm long. The clip angles were connected using M6 bolts. The flange strips were connected only at the top flange at both ends of the support using self-drilling screws. The three common failure of the clip angles was local buckling, distortional buckling and bearing on the bolt hole (see Figure 33). Test results found that the end rotation was influenced by the number of bolts in the clip angle and the thickness of the clip angle. The clip angle with three bolts exhibited higher stiffness. In addition, the ultimate load was found to be increased up to 70% for the inclusion of the flange strip compared to the only clip angle connection tested in the previous study [25] by the authors. The optimum aspect ratio of the clip angles for the clip thickness of 0.8 and 1.5 mm was 1.0 and 1.5, respectively.

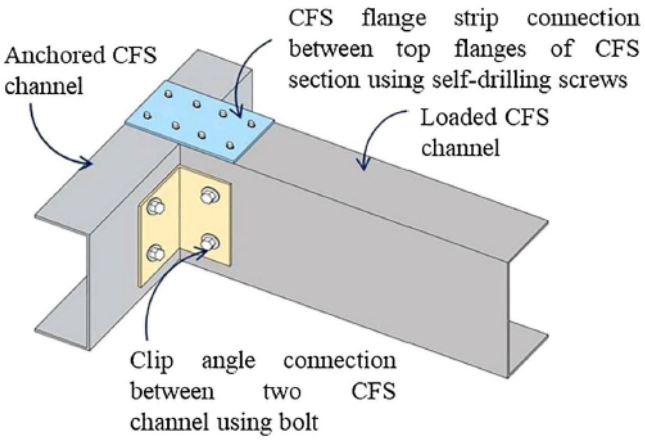


Figure 31 Schematic view of the beam-to-beam connection tested by Natesan and Madhavan [26].

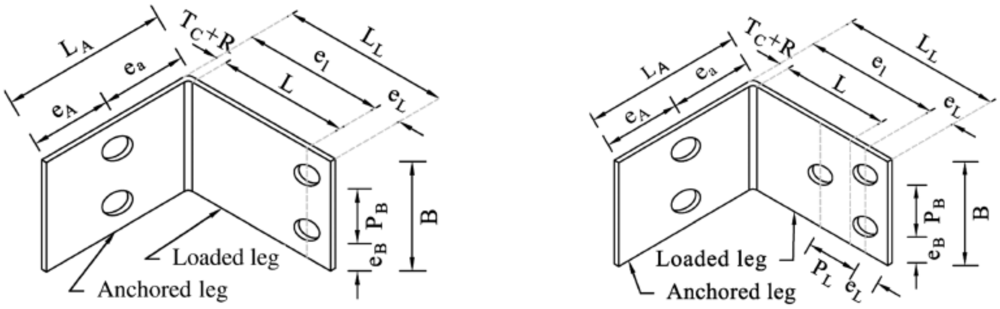
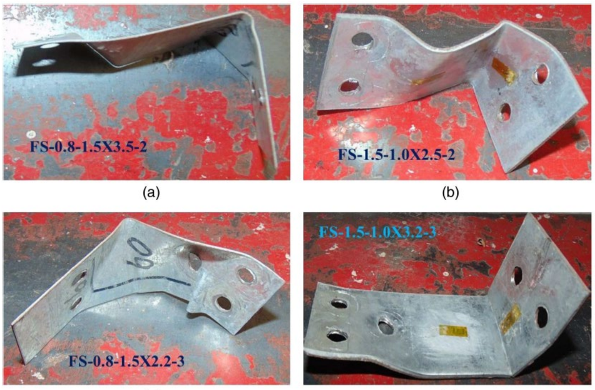


Figure 32 Schematic view of clip angle with a different number of bolts tested by Natesan and Madhavan [26].



(a) (b)

(c) (d)

Figure 33 Local buckling of the clip angle observed in the test study [26].

3.4 Summary

This chapter presents an overview of the structural system to comply the robustness requirements of the CFS structures. An example of designing the robustness of the CFS structures using the ALP method is presented based on the study carried out by Bae et al. [9]. A literature review on various CFS connections proposed to improve the structural systems to comply the robustness requirements of CFS structures is presented. Some of the key findings are highlighted as follows:

1. To resist the progressive collapse of CFS structures, the connection detailing of CFS members particularly the beam-to-column connection plays a key role to develop an alternative path or tie force by resisting the large tensile force generated due to the removal of the components of the CFS structure. Therefore, an effective connection design can improve the progressive collapse of CFS structures.

2. A combination of clip angle and flange strip in the webs and flanges connected using bolts and self-drilling screws, respectively is an effective connection mechanism for beam-to-column and beam-to-beam connection. The ultimate load of such a connection was found to be as high as 70% compared to the ultimate load of only clip angle.

3. Test results show that the shear resistance of CFS clip angle bolted connections is influenced by the thickness and the aspect ratio of the clip angles. Increasing the thickness of the clip angles increases the shear resistance but the shear resistance decreases as the aspect ratio increases. However, further study should be undertaken to investigate the failure of clip angles when the thickness of the clip is larger than the CFS members.

4. It is found that the end rotation of the clip angle is influenced by the number of bolts in the clip angle and the thickness of the clip angle.

5. The common failure modes of thick clip angles with small aspect ratios lesser than 0.8 are governed by the local buckling whereas, for thin clip angles with aspect ratios greater than

0.8, the failure is governed by the distortional local buckling. The optimum aspect ratio of the clip angles for the clip thickness of 0.8 and 1.5 mm was 1.0 and 1.5, respectively.

6. It is found that the stiffness and load-carrying capacity of the bolted connection is much better than the self-drilling screw and rivet connections.

7. In the notional element removal scenario, removal of stud column at the middle of the structure results in the failure of the track sections of CFS structures which may cause the immediate upper floor to collapse as the track sections of CFS structures are not designed to resist a moment. It is therefore recommended to design the track sections as structural members likewise header sections to improve the progressive collapse of the CFS structures.

4. Structural robustness modelling

4.1 Background

4.1.1 STRUCTURAL CONFIGURATION

With floorplans provided by BlueScope for a typical CFS 6-storey structure, a large joist span on the ground floor was chosen and used to conduct a simplified robustness analysis (Figure 34).

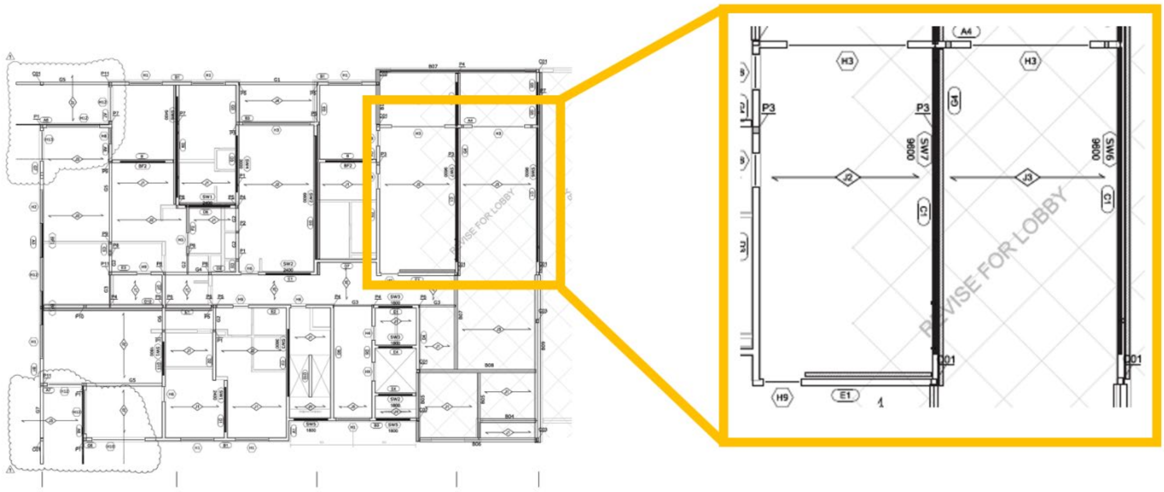


Figure 34 Selected Portion of the Ground Floor Plan provided by BlueScope

This enlarged region (Right - Figure 34) was the region selected, the joist span within this region was measured at 4100 mm, the length of the central track section being 7800 mm. A typical wall detail (Figure 35) was also provided from a different drawing set, this wall detail was assumed as consistent with the central wall in this selected region (more detailed future investigations will account for any apparent differences between these sets). The floor-to-floor height has been taken as 3100 mm.

From these details, the following structural sections have been used within modelling:

- 203 CFS lipped channel sections for joists spaced at 600 mm.
- 203 CFS plain channel sections for the ledger track.
- 152 CFS lipped channel sections for studs spaced at 600 mm. It is however noted that this section may also vary to 102 mm lipped channel sections also.
- In this modelling, the lightweight gypsum-steel composite deck has been ignored, instead lateral restraint has been applied to joist members and the associated deck loads have been applied.
- Additionally, bracing and cross members have also been ignored at this stage of modelling.

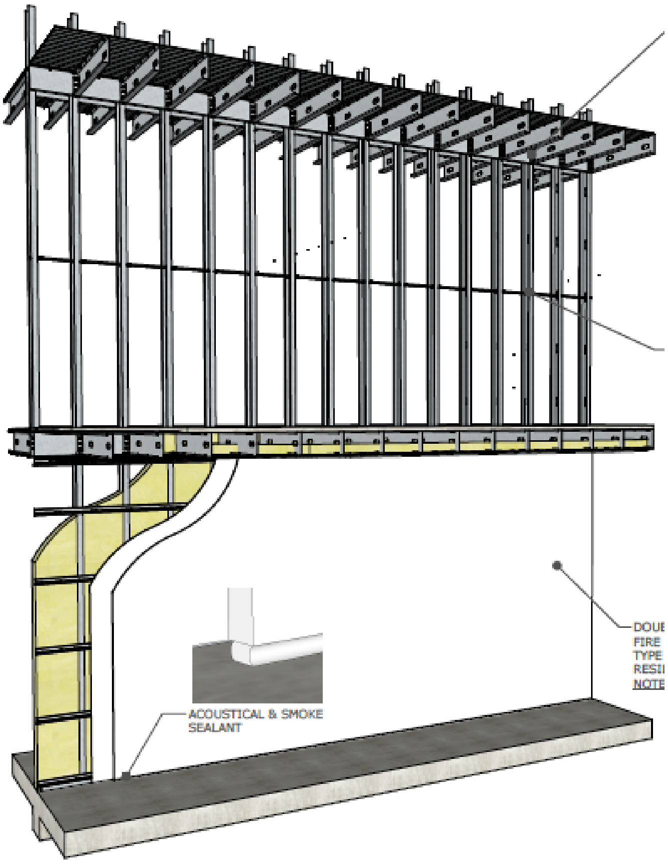


Figure 35 Typical Wall Detail provided by BlueScope

4.1.2 Loading

Floor loading within all models presented in later sections assumes the following loads:

- Dead Load: 1 kPa (Unfactored)
- Live Load: 2 kPa (Unfactored)

Ledger loading (from floors above) however, is taken from a separate report prepared by the SteelHub provided by BlueScope (Figure 36) for simplified sub-models (eg. Section 4.2 where multiple levels have not been modelled) but ignored in multi-level modelling (where loading from levels above is assumed to be a product of floor loads above, wall dead loads ignored due to their small contribution in this simplified modelling).

Within all proceeding models, a non-linear static analysis approach has been taken. With the removal of the wall section (the central wall within the enlarged portion of Figure 34), a dynamic load factor of 2.0 is applied to all loads post-wall removal per UFC 4-023-03 guidelines [4]. This factor is used to account for dynamic effects in a static analysis as discussed in Section 2.2.1, the value of this allowance is not provided within the Australian Codes and Standards.

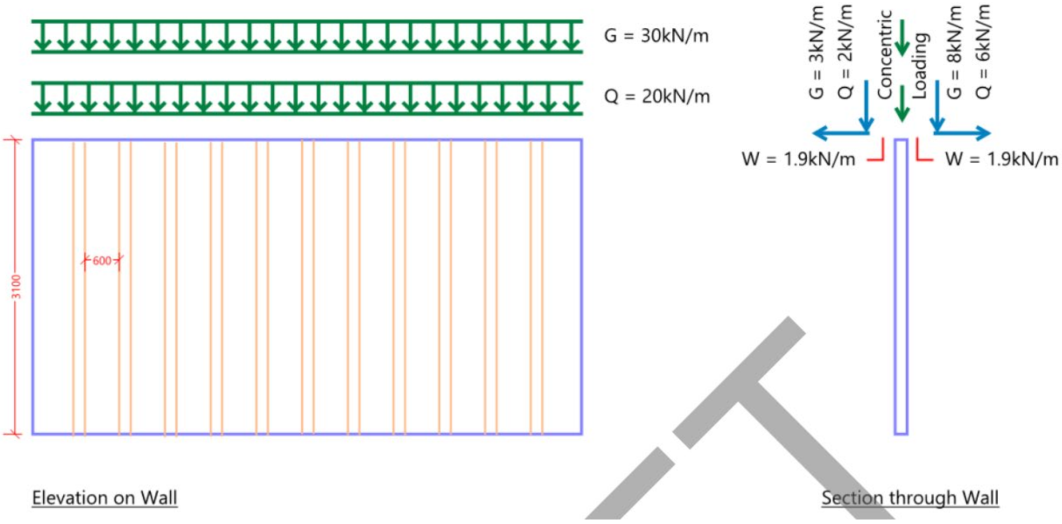


Figure 36 Loading from SteelHub report provided by BlueScope

For this analysis a load factor of 1.0 was applied to dead loads and 0.4 to live loads per Australian Standards [27].

4.1.3 Material Properties

A linear-plastic material model was formulated based on the G450 Datasheet provided by BlueScope, this model includes an Elastic Modulus of 200 GPa and yield stress of 560 MPa. Damage initiation and evolution have not been defined within this simplified modelling. A stress-strain representation of this material is shown in Figure 37.

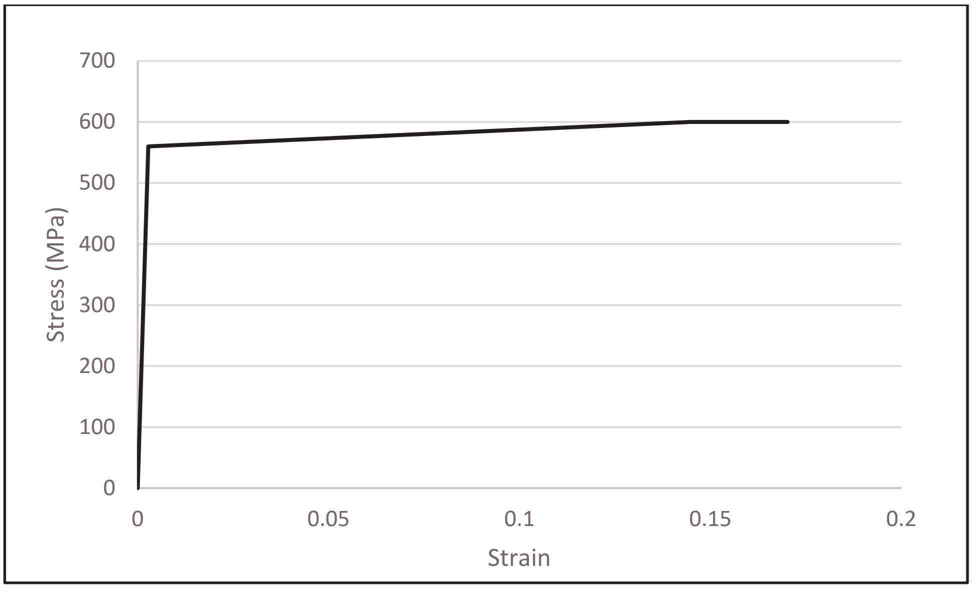


Figure 37 Idealised G450 Stress-Strain Plot

4.1.4 Section Properties

For all proceeding models, beam elements have been used to model all structural members. The specific cross sections corresponding to each member have been inputted to provide appropriate section moduli and areas. Note, that beam render idealisations have been shown in all modelling figures to demonstrate that members are orientated appropriately and to aid in visualisation of structural response.

4.2 Catenary-Only Sub-Model

As described within previous sections, catenary action is a fundamental last resort collapse mechanism employed by LGS structures under large deflections (5-15% of span length [24]). As a wall section is removed and a structure redistributes load, joists undergo significant deformations and transition between joists acting in beam action (wherein load is resisted by the section's corresponding internal moment couple) to catenary action (wherein load is resisted by the section's corresponding tensile axial resistance).

Although it is unlikely to be the only mechanism that resists collapse, its contribution is important and represents a suitable initial inquiry to structure response. For this reason, a simplified sub-model was constructed that represented a single storey of the structural configuration described in Section 4.1.1 and corresponding loading, material and section properties from Sections 4.1.2 to 4.1.4.

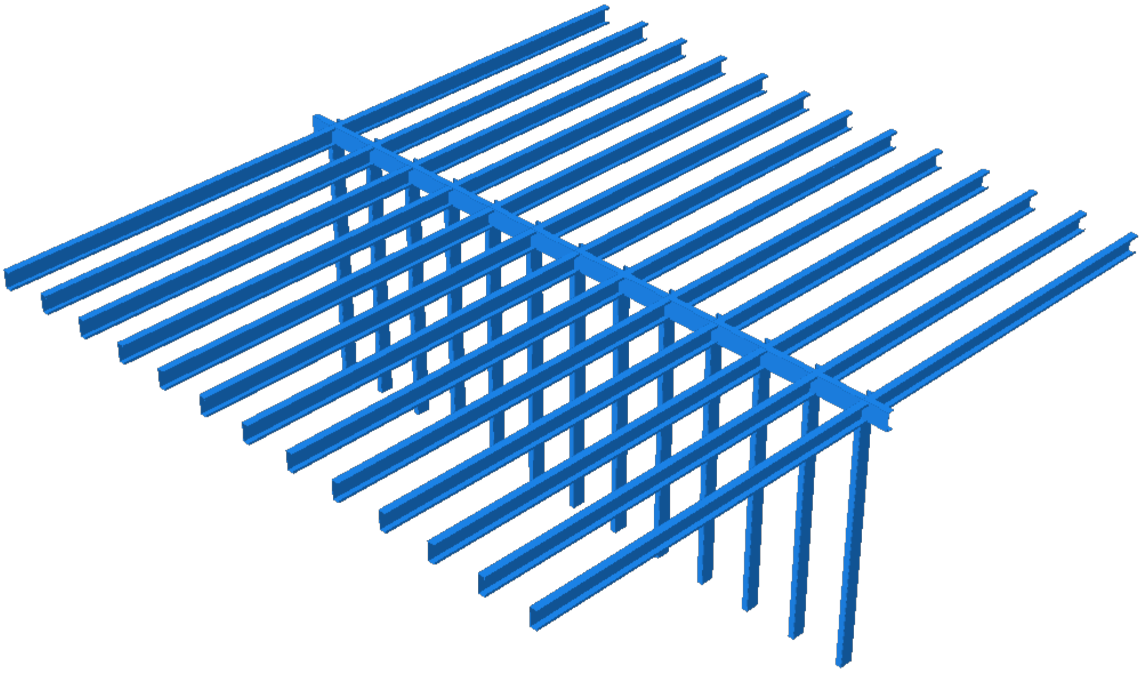


Figure 38 Simplified Sub-Model

In understanding structural response of this sub-model utilizing catenary action in a column removal event, a multi-step approach is required:

Stage 1A: No Load Applied

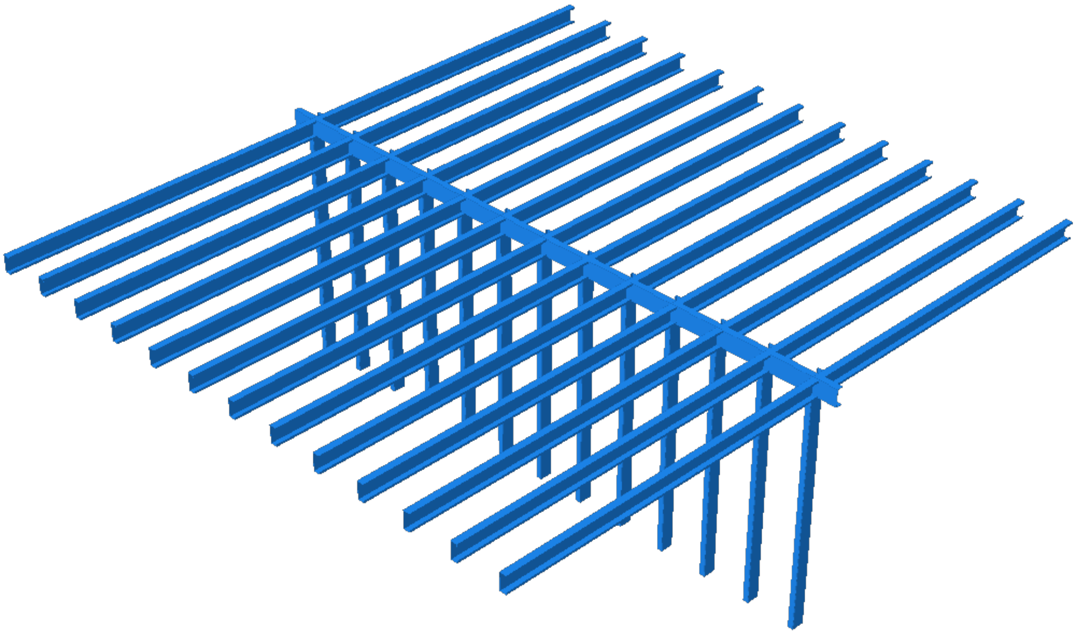


Figure 39 Stage 1A - No Load Applied with Wall Present

Stage 1B: G + 0.4Q Fully Applied, Wall Present

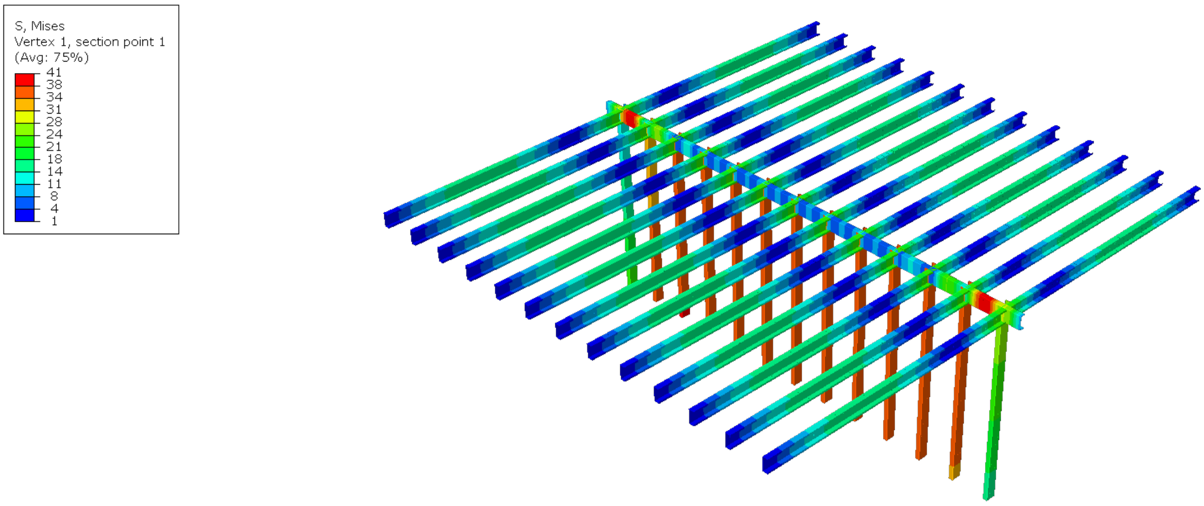


Figure 40 Stage 1B - Initial Loads with Wall Present

Stage 2A: G + 0.4Q Fully Applied, Wall Removed

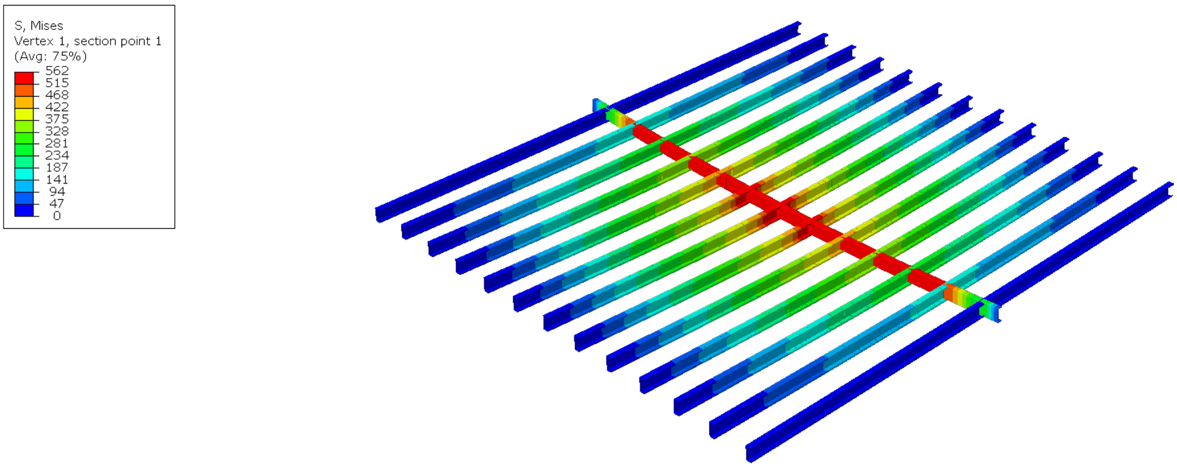


Figure 41 Stage 2A - Initial Loads with Wall Removed

Stage 2B: 2(G + 0.4Q) Fully Applied, Wall Removed, Final Deformed State

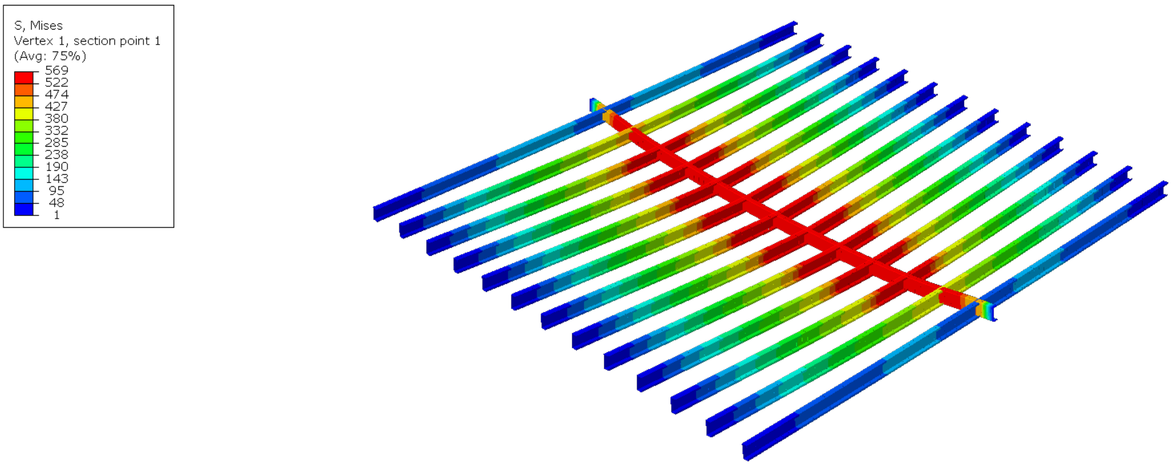


Figure 42 Stage 2B - Loads applied with Dynamic Load Factors and Wall Removed

In constructing this sub-model, it is important to account for the deformation inward of the surrounding structure. This deformation will relieve some of the axial demands imposed on the joist-ledger connections as a result of catenary action. An account for this deformation is made in the application of axial springs with a specified stiffness at the nodes (joist ends and ledger ends) that adjoin to the surrounding structure. In this modelling an arbitrary value of 10 kN/mm has been used for this axial spring stiffness, further research will elucidate an appropriate stiffness value, as well as the need for stiffnesses in other translational and rotational directions.

Under these assumptions, the catenary action end forces, that is, the axial reactions at the end of the joist and ledger sections that allow for the development of such forces, is reported in Figure 43. This modelling suggests that the tension demand imposed at the joist-ledger connections is on the order of 177 kN per joist, while the ledger ends possess 57 kN of tension under catenary action as a result of this wall removal.

Given these significant joist loads, the sub-modelling simplification appears too simplistic at this stage to produce realistic results. Full-frame multi-level modelling has thus been adopted and will be discussed in proceeding sections.

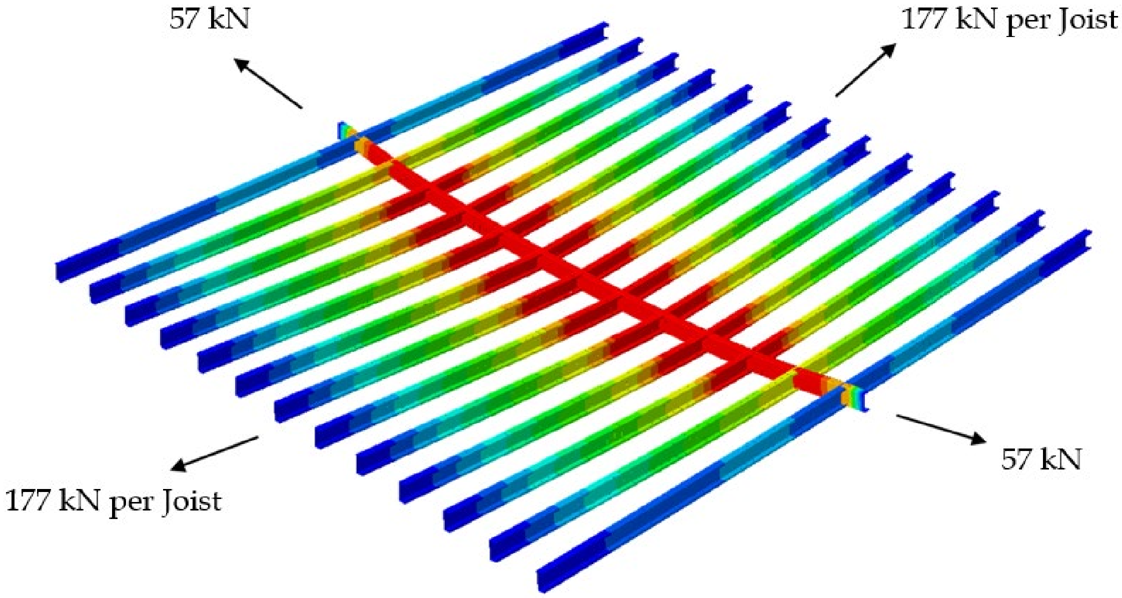


Figure 43 Axial Forces as a result of Catenary Demands

4.3 Preliminary Multi-Level Model with Walls in Both Directions

In producing a more realistic analysis, it became apparent from the sub-modelling approach that the contribution of the surrounding structure to redistribute load must be more fully considered. The primary impediment to this modelling however is the significant number of elements and associated connections present within such modelling. For example, a 4-storey, 4-bay simplified structure is associated with 728 elements (studs, joists, ledgers) and 1,568 connections between these elements. The associated time in manually creating such a model and then modifying based on different structural configurations and properties is significant.

To overcome this issue, Python scripting has been employed and a platform created that automates a significant portion of this model creation process. This script interfaces directly with the structural modelling software and means that both structural configuration and properties can be modified efficiently within a short script edit, as opposed to manually changing the configuration or properties of hundreds of elements or thousands of connections.

As was the case the in simplified sub-modelling, structural configuration is described in Section 4.1.1 and corresponding loading, material and section properties from Sections 4.1.2 to 4.1.4. This modelling fully models the walls, joists and ledgers of a simplified bay structure. Walls are running in both directions with studs spaced at 600 mm.

The same multi-step approach is used, this is presented in the following pages:

Stage 1A: No Load Applied

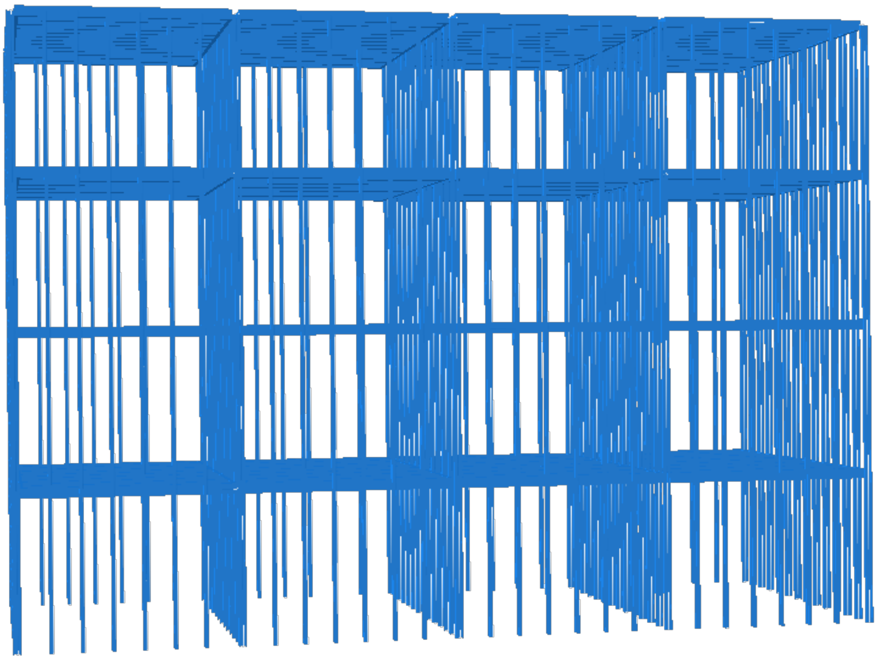


Figure 44 Stage 1A - No Load Applied with Wall Present

Stage 1B: G + 0.4Q Fully Applied, Wall Present

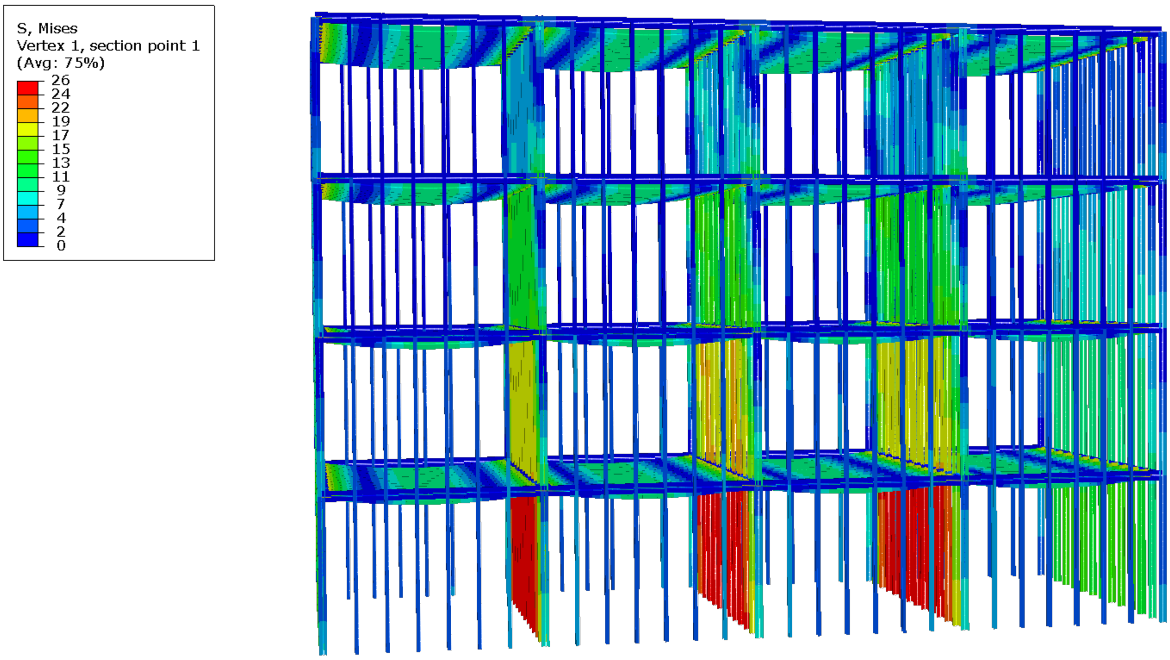


Figure 45 Stage 1B - Initial Loads with Wall Present (Deformation Factor = 10)

Stage 2A: G + 0.4Q Fully Applied, Wall Removed

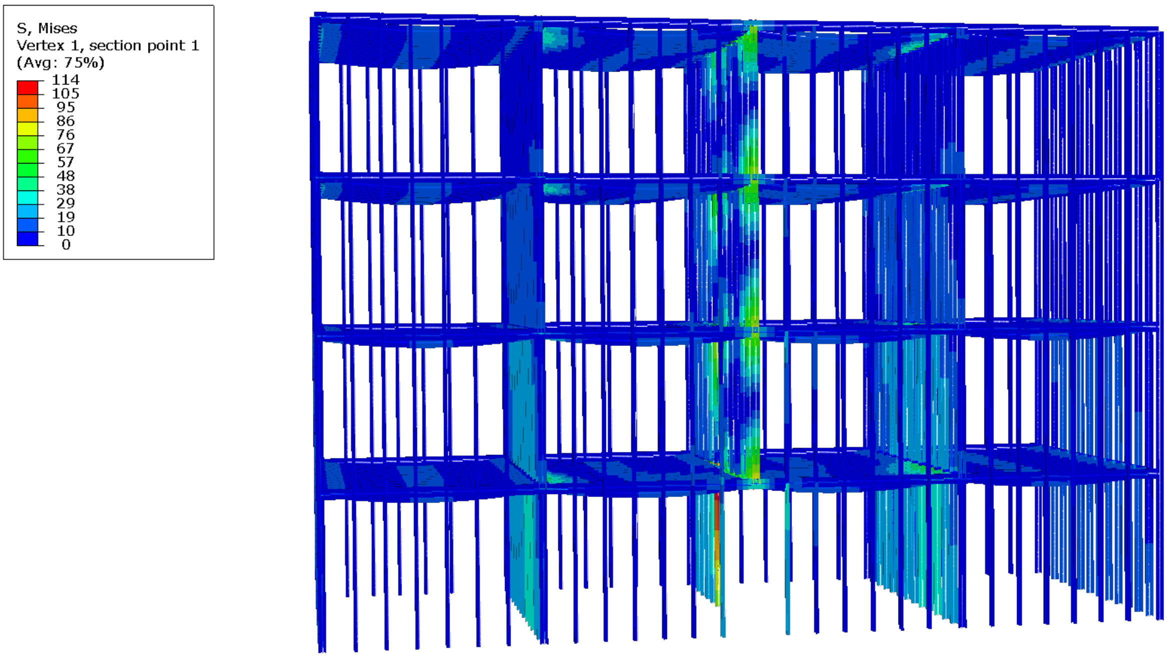


Figure 46 Stage 2A - Initial Loads with Wall Removed (Deformation Factor = 10)

Stage 2B: 2(G + 0.4Q) Fully Applied, Wall Removed, Final Deformed State

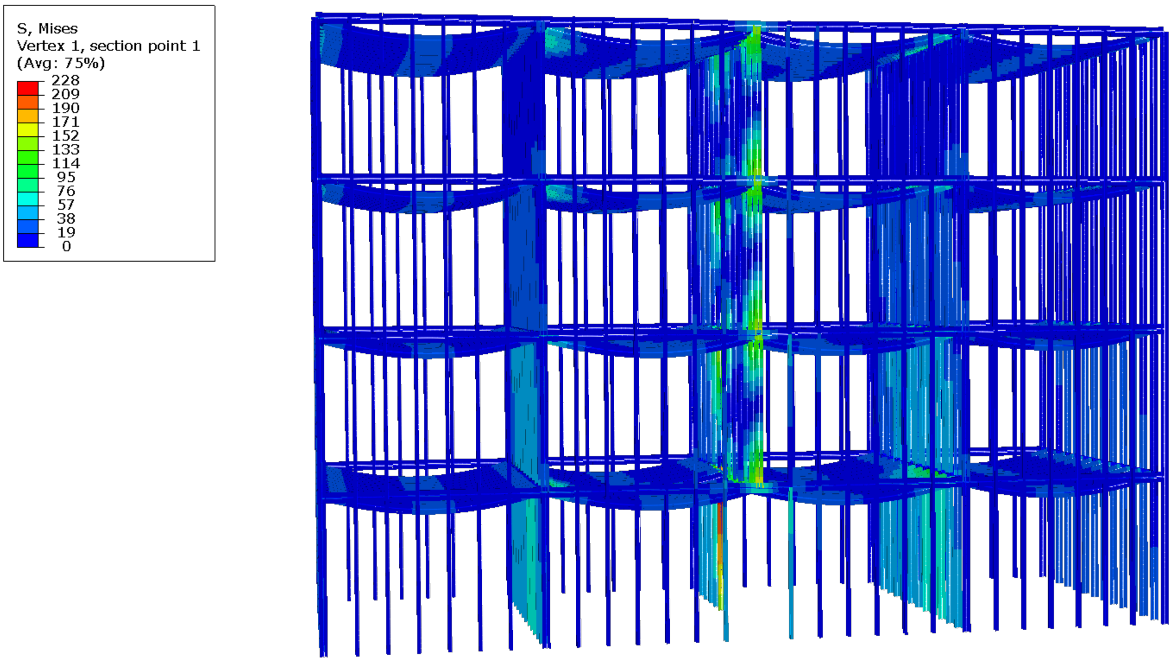


Figure 47 Stage 2B - Loads applied with Dynamic Load Factors and Wall Removed (Deformation Factor = 10)

Tension demands, particularly those at the joist-ledger connections, is shown in Figure 48. Areas experiencing axial tension are coloured, whereas all other areas have been greyed out. These tension values are presented within Figure 48 in stress units (MPa), the catenary force demands can be derived from these values by multiplying by section area.

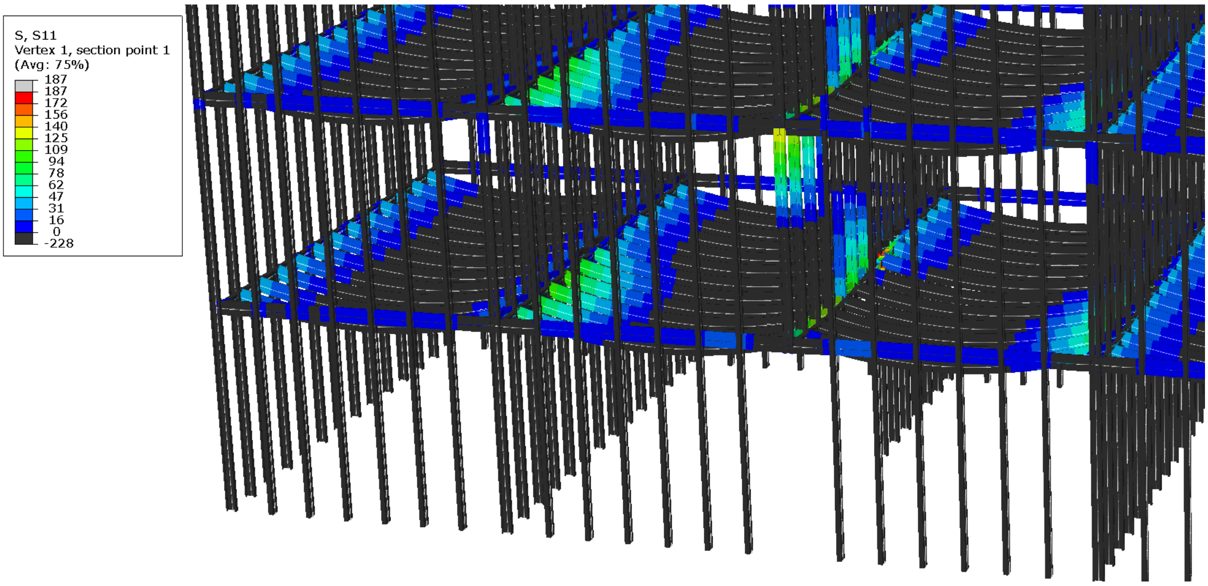


Figure 48 Tensile Stress surrounding Wall Removal (Deformation Factor = 10)

This modelling suggests that the tension demand imposed at the joist-ledger connections is on the order of 112 kN per joist (within critical regions), while the ledger ends possess 217 kN of tension under catenary action as a result of this wall removal.

The most significant shortcoming of this modelling currently is the tie restraint applied between the connection of all elements, currently all elements are tied in both translation and rotation. A translational and rotational stiffness should be assigned appropriately between all elements, this represents an important area of further investigation.

Other important future inclusions to this modelling include:

- An update to the structural layout to more accurately represent a typical CFS structure (inclusion of openings, hallway regions, etc.)
- The inclusion of both the plasterboard on wall elements through shell elements and the lightweight concrete-steel composite deck.
- The inclusion of cross-bracing within wall elements and other bracing systems in typical CFS structures.

These changes will produce a more accurate representation of the connection demands within typical CFS structures and provide an appropriate comparison to the other methods detailed earlier in the report, particularly prescriptive tie-force approaches. The Python scripting platform produced within this phase of the project however should improve the efficiency in enacting these changes substantially.

4.4 Summary

An initial investigation into catenary demands via a simplified sub-model provided insight into the demands imposed on the structure if catenary action were the only arrest-collapsing mechanism employed, however the importance of other mechanisms became apparent following this investigation. This prompted a more detailed modelling approach, wherein a 4-storey, 4-bay simplified structure was modelled based on critical areas within provided LGS floor plans and other relevant data. The creation of this model was aided significantly by the use of Python scripting.

This scripting allowed for the efficient creation and modification of such structures, offering various significant benefits outlined in previous sections.

This modelling provided additional insight into the robustness response of LGS structures, providing an estimate of internal tie forces of 112 kN. There were a number of next steps identified in improving this modelling further, most importantly, in incorporating accurate translation and rotational stiffnesses to each of the element connections (as opposed to the unrealistic tie-restraints currently used).

FUTURE RESEARCH PLANS

5. Conclusions

This report has provided a detailed overview of structural robustness methods broadly and made recommendations as to those most relevant to LGS structures, namely alternative load path analysis approaches and prescriptive tie-force approaches. An investigation has been undertaken into LGS structural systems, the importance of connection detailing in the context of structural robustness, the current state of such connections and potential connection alternatives identified within the existing literature. Finally, a preliminary alternative load path analysis has been performed on a simplified 4-storey 4-bay LGS structure. This analysis has provided several insights into this robustness approach, namely, the importance of an efficient platform (Python scripting) for model creation given the complexity of such structures, a point of comparison concerning tying force demands as compared to those within local and international codes and the need for improvements within this modelling.

6. Next Steps

Several next steps have been identified throughout this report that includes improving CFS members and connections as well as the most important pertaining to improving the alternative load path analysis performed thus far on a simplified representative CFS structure. Some of the findings/limitations that should be addressed in the future study are listed as:

- An effective connection system can play a key role to develop an alternative path or tie force by resisting the large tensile force generated due to the removal of the components of the CFS structure. Therefore, in the future phase of the project, experimental tests of novel connection systems should be carried out for the robustness of LGS structural systems.

- Detailed FE modelling should be performed to optimize the most cost-effective connection systems by investigating the thickness of the connectors (i.e. clip angles), number and diameter of bolts, etc.

- Updating the structural layout to more closely match a realistic representative CFS mid-rise structure.

- Inclusion of the effects of plasterboard on wall stud behaviour in the context of robustness.

- Inclusion of the effects of the light-weight concrete-steel composite deck behaviour in the context of robustness.

- Inclusion of effective wall bracing and other bracing elements found in typical CFS mid-rise structures.

- Replacing current tie restraints between structural elements with appropriate translational and rotational spring stiffnesses.

- Investigate the effects of a strong back system wherein robustness loads are resisted primarily by a number of key, locally-strengthened members.

- Investigate the effects of double-wall discontinuous construction methods that are preferred in some contexts for acoustic performance.

In summary, the primary objectives of further study will be upon experimentally investigating novel connection details that address LGS robustness, developing a detailed FE model that can accurately assess complete LGS robustness responses and to use this FE model to undertake parametric studies that aim to compare the robustness characteristics of various LGS systems.

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