STRENGTH AND BEHAVIOUR OF COLD-ROLLED ALUMINIUM PORTAL FRAMES

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ABSTRACT

This doctoral thesis presents a comprehensive study on cold-rolled aluminium portal frames composed of back-to-back lipped channel sections. The research included an extensive experimental program on full-scale portal frames and components, a numerical simulation part on finite element modelling, and finally a proposal development of the design for the cold-rolled aluminium frame systems. The primary aim is to explore the structural behaviour and ultimate strength through experiments and numerical analyses, leading to the development of proposed provisions for the design of cold-rolled aluminium portal frames using Advanced Analysis. The study also evaluates the applicability of the current standards to the design of cold-rolled aluminium portal frames.

Towards observing the structural behaviour and determining the ultimate strength, a series of seven full-scale tests on two-bay single-span cold-rolled aluminium portal frame systems, having a size of 14 m long by 6.7 m high, were carried out. Various frame configurations were tested for both gravity loading and combined horizontal and gravity loading conditions. Separate tests were performed on the column base connection to quantify the flexural stiffnesses of the base connections about the column-major and -minor axes so that the semi-rigidity of the base connection used in the full-scale frame test was evaluated, and the effects of different base brackets on the column base stiffness were also examined. Other laboratory experiments, including coupon and point fastener connection tests, were also conducted to obtain the necessary information on material properties, connection characteristics. Further, initial geometric imperfections of both members and systems were thoroughly measured for use in further numerical investigations.

Nonlinear finite element simulations using shell elements and advanced analysis of the full-scale frames and the base connections were developed and calibrated against experimental results. All sources of major nonlinear actions, notably geometric, material, connector and contact nonlinearities, were included in the numerical finite element models. In details, the individual bolts used for the connections were simulated by deformable point-based fasteners. The force-deformation characteristics of the deformable fasteners, which were obtained from the point fastener connection tests, were incorporated and successfully implemented in the Advanced Analysis. Parametric studies were subsequently carried out on the basis of the calibrated modelling technique to determine the effect of the column base stiffness, as well as various configurations of the lateral bracing for columns, on the frame ultimate strength for both gravity load and a combination of horizontal load and gravity load. A larger span finite element model was also created to study the suitability of the frame design for larger spans.

The strengths of cold-rolled aluminium portal frames were determined by the conventional methods available in the current international aluminium design standards/specifications and by the Direct Design Method using Advanced Analysis. A
comparison of predicted strengths from the various design approaches was then performed. To account for inherent uncertainties in the strength of the cold-rolled aluminium portal frames, a system reliability analysis was conducted to derive system resistance factors. It was highlighted that the Direct Design Method using Advanced Analysis as proposed in this study is the robust and realistic method for the design of cold-rolled aluminium portal frames and is likely the future design method for all types of structures including those comprised of cold-rolled aluminium sections.

**Keywords:** Cold-rolled aluminium sections; Portal frames; Strength and behaviour; Advanced Analysis; Direct Design Method.
STATEMENT OF ORIGINALITY

This is to certify that to the best of my knowledge, the content of this thesis is my own work. This thesis has not been submitted for any degree or other purposes.

I certify that the intellectual content of this thesis is the product of my own work and that all the assistance received in preparing this thesis and sources have been acknowledged.

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ABBREVIATIONS

CFS : Cold-Formed Steel
COV: Coefficient of variation
CRA : Cold-Rolled Aluminium
CSM : Continuous Strength Method
DDM : Direct Design Method
DoF : Degree of Freedom
DSM : Direct Strength Method
EBA : Elastic buckling analysis
ELM : Effective length method
ENG : Energy method
ETM : Effective Thickness Method
EWM : Effective Width Method
FBD : Free Body Diagram
FE : Finite Element
FOSM : First Order Second Moment
GMNIA : Geometric and Material Nonlinear Analysis with Imperfection
GNA : Geometric Nonlinear Analysis
KBC : Knee Brace-to-Column brackets
KBR : Knee Brace-to-Rafter brackets
LA : Linear Analysis
LBA : Linear Buckling Analysis
LRFD : Load and Resistance Factor Design
MB : Moment Base
MPC : Multi-Point Constraint
M12 : 12 mm diameter bolt
M16 : 16 mm diameter bolt
M24 : 24 mm diameter bolt
NHF : Notional Horizontal Forces
PDF : Probability Density Functions
PVC : Polyvinyl Chloride
RHS : Rectangular Hollow Section
SFSM : Spline Finite Strip Method
WAM : Weighted Average Method
3D : Three-Dimensional
NOTATIONS

The main notations of this thesis are listed below. All symbols used in the chapters will again be explained right after they first appear in the text. In most cases, only a single meaning was assigned to each notation, but if this is not the case, the definition will be clearly explained in the text.

\[ A \quad : \quad \text{Area of the cross-section} \]
\[ A_c \quad : \quad \text{Area of the column cross-section} \]
\[ A_{eff} \quad : \quad \text{Effective area} \]
\[ A_g \quad : \quad \text{Gross area of the cross-section} \]
\[ B \quad : \quad \text{Bimoment} \]
\[ C_b \quad : \quad \text{Coefficient that accounts for moment gradient along a beam’s length} \]
\[ E \quad : \quad \text{Elastic modulus} \]
\[ F \quad : \quad \text{Compression force in the knee brace} \]
\[ F_L \quad : \quad \text{Limit state design stress in AS/NZS 1664} \]
\[ F_{Le} \quad : \quad \text{Limit state stress of a member for global buckling in AS/NZS 1664} \]
\[ F_{Ll} \quad : \quad \text{Limit state stress of a member for local buckling in AS/NZS 1664} \]
\[ F_{bl} \quad : \quad \text{Local buckling stress corresponding to the flexural compressive strength in AA2015} \]
\[ F_{ce} \quad : \quad \text{Stress corresponding to the uniform compressive strength in AA2015} \]
\[ F_{cl} \quad : \quad \text{Local buckling stress in AA2015} \]
\[ F_{cy} \quad : \quad \text{Compressive yield strength} \]
\[ F_e \quad : \quad \text{Elastic critical stress for global buckling} \]
\[ F_m \quad : \quad \text{Mean ratio of actual to nominal dimensions of the cross-section} \]
\[ F_{tu} \quad : \quad \text{Tensile ultimate strength} \]
\[ F_{ty} \quad : \quad \text{Tensile yield strength} \]
\[ G \quad : \quad \text{Dead load} \]
\[ G \quad : \quad \text{Shear modulus} \]
\[ I_{cw} \quad : \quad \text{Warping constant of the column cross-section about the x-axis} \]
\[ I_{cx} \quad : \quad \text{Second moment of area of the column cross-section about the x-axis} \]
\[ I_{cy} \quad : \quad \text{Second moment of area of the column cross-section about the y-axis} \]
\[ I_{sx} \quad : \quad \text{Second moment of area of the sleeve stiffener cross-section about the x-axis} \]
\[ I_{sx} \]: Second moment of area of the sleeve stiffener cross-section about the x-axis
\[ I_{sy} \]: Second moment of area of the sleeve stiffener cross-section about the y-axis
\[ I_w \]: Warping constant
\[ I_x \]: Second moment of area about the x-axis
\[ I_y \]: Second moment of area about the y-axis
\[ J \]: Torsion constant
\[ L \]: Member length
\[ L_e \]: Effective length
\[ L_{ex} \]: Effective length for buckling about the x-axis
\[ L_{ey} \]: Effective length for buckling about the y-axis
\[ L_{ez} \]: Effective length for twisting
\[ L_k \]: Length of the knee brace
\[ M_b \]: Moment capacity for limit state of rupture in AA2015
\[ M_{bd} \]: Member moment capacity for distortional buckling
\[ M_{bd-i} \]: Member moment capacity for distortional-global interaction buckling
\[ M_{be} \]: Lateral-torsional buckling strength of a beam
\[ M_{bl} \]: Local buckling strength of a beam
\[ M_{bn} \]: Moment capacity for limit state of yielding in AA2015
\[ M_{bx} \]: Nominal member moment capacity about the x-axis
\[ M_{by} \]: Nominal member moment capacity about the y-axis
\[ M_m \]: Mean ratio of actual mechanical properties to the corresponding specified values
\[ M_o \]: Elastic lateral-torsional buckling moment
\[ M_x \]: Major axis bending moment
\[ M_y \]: Minor axis bending moment
\[ M_{yc} \]: Yield moment on the compression side
\[ M_x^* \]: Design bending moment about the x-axis of the gross-section
\[ M_y^* \]: Design bending moment about the y-axis of the gross-section
\[ N \]: Axial compression
\[ N_c \]: Nominal capacity of member in compression
\[ N_{cd} \]: Member capacity of a member in compression for distortional buckling
\( N_{ce} \) : Member capacity of a member in compression for global buckling

\( N_{cl} \) : Member capacity of a member in compression for local buckling

\( N_{cl-i} \) : Capacity of a member in compression for local-global interaction buckling

\( N_{oc} \) : Elastic buckling load for a compression member

\( N_{od} \) : Distortional buckling strength

\( N_{ol} \) : Local buckling strength

\( N_y \) : Yield capacity of a member in compression

\( N^* \) : Design axial compression

\( P_f \) : Probability of failure

\( P_m \) : Mean ratio of the experimental strengths to the predicted values

\( P_u \) : Ultimate strength of the frame system

\( P_x \) : Flexural buckling load for buckling about the x-axis

\( P_y \) : Flexural buckling load for buckling about the y-axis

\( P_z \) : Torsional buckling load for torsional buckling

\( Q \) : Load effect of a structure

\( Q \) : Live load

\( R_m \) : Mean strength

\( R_n \) : Nominal resistance of a structure

\( S_c \) : Section modulus on the compression side of the neutral axis in AA2015

\( S_t \) : Section modulus on the tension side of the neutral axis in AA2015

\( V_F \) : Coefficient of variation of the cross-section dimensions

\( V_M \) : Coefficient of variation of the material properties

\( V_P \) : Coefficient of variation of the accuracy of the strength prediction

\( V_Q \) : Coefficient of variation of the load effects

\( V_R \) : Coefficient of variation of the resistance

\( W_{eff} \) : Effective section modulus in bending

\( W_{el} \) : Elastic section modulus of gross-section

\( W_{pl} \) : Plastic modulus of gross-section

\( W_u \) : Wind load

\( Z \) : Plastic section modulus
\( Z_c \) : Section modulus for the extreme compression fibre for bending in AS/NZS 1664

\( Z_f \) : Full section modulus of the extreme fibre at first yield

\( b \) : Plate width

\( b_e \) : Effective width

\( d_s \) : Distance from the shear centre to the web-flange junction of the column section

\( f_{oc} \) : Elastic buckling stress

\( f_{od} \) : Elastic distortional buckling stress

\( f_{ol} \) : Elastic local buckling stress

\( f_{ox} \) : Elastic buckling stress for buckling about the x-axis

\( f_{oy} \) : Elastic buckling stress for buckling about the y-axis

\( f_{oz} \) : Elastic buckling stress for torsional buckling

\( f_y \) : Yield stress

\( f_v \) : Elastic shear buckling stress

\( k \) : Buckling coefficient

\( k_c \) : Coefficient for compression member in AS/NZS 1664

\( k_{ex} \) : Effective length factor for buckling about the x-axis

\( k_{ey} \) : Effective length factor for buckling about the y-axis

\( k_{ez} \) : Effective length factor for twisting

\( k_v \) : Shear buckling coefficient

\( k_\phi \) : Plate torsional stiffness

\( n \) : Ramberg-Osgood parameter

\( p \) : Plastic strain

\( r_x \) : Radius of gyration of the cross-section about the x-axis

\( r_y \) : Radius of gyration of the cross-section about the y-axis

\( r_{ye} \) : Effective radius of gyration of a beam about the y-axis (the axis parallel to the web)

\( t \) : Plate thickness

\( \Delta_{Ax} \) : Horizontal displacement of the apex

\( \Delta_{Ay} \) : Vertical displacement of the apex

\( \Delta_{Az} \) : Out-of-plane displacement of the apex

\( \Delta_{Eax} \) : Horizontal displacement of the north eaves
\( \Delta E_{nz} \) : Out-of-plane displacement of the north eaves
\( \Delta E_{esx} \) : Horizontal displacement of the south eaves
\( \Delta E_{esz} \) : Out-of-plane displacement of the south eaves
\( \beta \) : Reliability index
\( \gamma_{M1} \) : Partial factor in Eurocode 9
\( \varepsilon \) : Engineering strain
\( \varepsilon_f \) : Total elongation after fracture
\( \varepsilon_l \) : Longitudinal strain
\( \varepsilon_{p,\text{true}} \) : Plastic true strain
\( \varepsilon_{\text{true}} \) : True strain
\( \varepsilon_u \) : Uniform elongation corresponding to the ultimate tensile strength
\( \varepsilon_{0.01} \) : Uniform elongation corresponding to 0.01\% proportionality stress
\( \varepsilon_{0.2} \) : Uniform elongation corresponding to 0.2\% proof stress
\( \lambda \) : Slenderness
\( \lambda_d \) : Distortional buckling slenderness
\( \lambda_{d-i} \) : Distortional-global buckling slenderness
\( \lambda_l \) : Local buckling slenderness
\( \sigma \) : Engineering stress
\( \sigma_{cr} \) : Elastic critical stress for local buckling of a plate element
\( \sigma_p \) : The proof stress
\( \sigma_{\text{true}} \) : True stress
\( \sigma_u \) : Ultimate tensile strength
\( \sigma_{0.01} \) : 0.01\% proportionality stress
\( \sigma_{0.2} \) : 0.2\% proof stress
\( \tau_{cr} \) : Shear elastic buckling stress
\( \nu \) : Poisson’s ratio
\( \chi \) : Reduction factor for member capacity of a compression member in Eurocode 9
\( \chi_{LT} \) : Reduction factor for lateral-torsional buckling strength of a beam in Eurocode 9
\( \omega \) : Sectorial coordinate
\( \phi \) : Initial out-of-plumb angle
\( \phi_b \) : Capacity reduction factor for a member in bending

\( \phi_c \) : Capacity reduction factor for a member in compression

\( \phi_{cc} \) : Capacity factor for compression member in AS/NZS 1664

\( \phi_s \) : System resistance factor
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CHAPTER 1. INTRODUCTION

1.1. Background

The construction industry has been developing rapidly in recent years, requiring the need for more advanced structures to fulfil the quality, utility, and economy of construction. For this reason, both structural geometries and material mechanical properties are important aspects that need to be considered. At the structural system level, among various types of structures, portal frames, as shown in Figure 1.1, are structural frameworks that are widely used to provide large open spaces for industrial, farming, and residential purposes. In the past, the majority of portal frames were constructed from hot-rolled steel sections while the light-material structures such as cold-formed steel or aluminium sections were mostly limited to secondary members (i.e. purlins, girts, cladding, etc). Until recently, with the increasing technology and availability of researches and design guidelines, the use of cold-formed steel sections as the primary structural members in the portal frames of light industrial buildings has seen intensive growth. This impetus has been mainly motivated by the fact that cold-formed members can be used efficiently as a result of the high strength-to-weight ratio, ease of fabrication and transportation, and simple erection and installation. Therefore, the use of light material structures with possible ability to enhance structural capacities is a new trend in the construction industry.

Figure 1.1. Portal Frame [1]

In parallel with the development of steel structures, aluminium ones have become more popular with new advanced solutions to compete with steel. Aluminium material is the second most widely specified metal in buildings after steel and has been commonly used in all construction sectors, from commercial buildings to domestic dwellings [2]. The success of aluminium alloys when used in structural members is due to their physical and mechanical properties, in which the notable features can be mentioned as follows [2-4]: (1) Aluminium alloys represent a wide family of materials and cover a whole range of various strengths offered
by the most commonly used mild steels, which can be considered as the basic reason of their success as constructional material, (2) Aluminium alloy materials possess high corrosion resistance, which does not require any protective coatings, ideally applicable in harsh environments, (3) The lightness of aluminium material produces advantages in weight reduction of the whole structures, (4) Aluminium is 100% recyclable without losing any natural characteristics while only 5% of the energy used to make the original product is required, and (5) Aluminium structural members are suitable for large choice for the connection systems (such as bolted and riveted). Hence, aluminium alloy materials seem to be economical considering the whole lifecycle of the structures, and lead to more competitive in applications in construction industry.

Traditionally, aluminium alloy structural members are mainly manufactured using the extrusion process. Therefore in the literature, previous research studies on aluminium structural members were well understood and focused on the strength and behaviour of extruded sections. As a result, the available design guidelines on aluminium structures are mainly suitable for extruded sections, members, systems, and components. Meanwhile, roll-forming techniques have advanced rapidly over the recent decades, indicating that such reliable and proven approach for metal shaping is ideal for modern applications. Recently, BlueScope Permalite [5] has demonstrated that it is possible to roll-form aluminium coil into lip-stiffened C- and Z-sections, as seen in Figure 1.2, to AS/NZS1734 tolerances [6]. These commercially available cold-rolled aluminium sections thus potentially open the door for commercialising cold-rolled aluminium structural sections world wide since structural strength can be enhanced by the cold forming process and cold-rolling is substantially faster and far less energy demanding than extrusion.

As cold-rolled aluminium sections are relatively new structural products in the Australian market, the available researches on these types of cold-rolled aluminium members and systems are limited. Therefore, more researches are required to enhance the knowledge base for cold-
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rolled aluminium structures. For this reason, the Australian Research Council Linkage Research Project – LP140100863 “Cold-rolled Aluminium Structural Members and Systems” collaborated between the BlueScope Permalite and Sydney University was successfully granted to perform research on cold-rolled aluminium sections. The objectives of this project are to (i) quantify the strength enhancements achievable by the cold-rolling process, (ii) devise guidelines for determining the strength of cold-rolled aluminium sections considering the prevalent buckling modes for these sections and the effect of gradual yielding, and (iii) especially develop design methodologies for particular applications of cold-rolled aluminium sections, viz. purlins and portal frames. As the third part of this research project, the portal frame systems and design guidelines for cold-rolled aluminium sections will be developed in this study. The ultimate findings will benefit the end consumers as a result of substantially lower costs of corrosion-resistant and eco-friendly buildings. No prior research has been presented for constructing a structural system like portal frames from cold-rolled aluminium sections. Hence, developing such portal frames systems requires significant innovation, particularly in devising semi-rigid joints which are capable of transferring moments and minimising deflections under service loads.

1.2. Present problem

The portal frame systems made of cold-rolled aluminium (CRA) profiles have not been previously attempted for design and construction by any producers, and thus, new research is required to develop an integrated portal frame system with the main columns and rafters made from the CRA double-channel sections. The secondary members (purlins and girts) are used to connect the rafters and columns at discrete points, which provide intermittent lateral and torsional restraints to the rafters and columns. The bolted connections are generally used to connect the members together through brackets. These connections exhibit semi-rigid behaviour in nature so that the strength of the frame system depends largely on the stiffness of the connections. Consequently, the full rigidity usually assumed in the design practice may not be applicable.

Cold-rolled aluminium sections are generally prone to local, distortional, and global instabilities or a combination of these modes due to the thinness of the cross-sections so that the effect of these instabilities and possible interaction between them needs to be intensively considered in the design of portal frame systems composed of CRA sections. In addition, due to the cold-working process, the structural strength of cold-rolled aluminium sections is significantly enhanced while the design guidelines in the current standards/specifications are premised on research of extruded sections, which do not allow the increase in strength produced by cold-forming to be accounted for. Therefore, the cold-rolled aluminium portal frames may not be designed in the same manner as those comprised of extruded sections.

The portal frames made of cold-formed sections may be designed in a variety of layouts. They can be formed by single-channel sections for the primary structural sections, which are
generally utilised for small to medium spans, or by back-to-back double-channel sections for medium to large spans. The knee braces and rafter ties can be provided in the frame system at the eaves and apex to transfer large bending moment between the rafters and columns or between rafters as well as to reduce the deflections of the system. The columns of the frames can be braced or unbraced, resulting in different strengths and behaviours among the frames. As a part of the design process, the final responsibility of the structural engineering designers is crucial for the choice of which layout of the frame system will be used, which braces should be included, and how to analyse and design the portal frame systems in each case. However, to help structural engineers to safely and efficiently design a portal frame system, the design guidelines and knowledge base are required, including the design recommendations, the quantity of the effects of individual components to the system, and the possibility of optimising the system, etc.

The current design procedure of a portal frame system is based on the member-based approach where a structural system is treated as individual members, connections. The design has been customarily based on the elastic analysis while the system effects are implicitly reflected in the design only through the use of effective length factors. This method, however, is limited in its ability to provide a realistic assessment of the ultimate strength of redundant structural systems since the complex interactions between members of a large structural system may not be accurately predicted by the simple effective length factor concept and the inelastic load redistribution after yielding cannot be captured \[8-10\]. Therefore, the design methodology for portal frame systems is gradually shifting towards the system-based design \[11\]. This new approach is a design-by-advanced analysis, in which both the material and geometric nonlinearities are included in the analysis, allowing the ultimate limit state strength and stability of a real structural system to be most accurately captured. However, design guidelines for the cold-formed section structures in general and for CRA portal frame systems, in particular, are still in the nascent stage, requiring more intensive researches on this topic.

In light of the above facts, a comprehensive study on the strength and behaviour of cold-rolled aluminium portal frames and associated components is necessary through the experimental investigations and advanced numerical finite element analysis. This would enhance the knowledge base for aluminium structural systems and potentially lead to the further development of the advanced design methodologies.

1.3. Research scope and objectives

The primary objective of this study is to provide a comprehensive study on portal frames composed of cold-rolled aluminium (CRA) alloy back-to-back channel sections. On the basis of this study, appropriate design recommendations are recommended to give a guideline for engineers to safely and efficiently design a CRA portal frame system. In particular, this research has been carried out with the following objectives:
1. To develop a portal frame system with the main structural members made from CRA back-to-back channel sections and the bolted connections between members. The layouts and geometries of the frame systems were preliminarily designed on the basis of the similar design concept of cold-formed steel portal frames. The potential modifications to improve the performance of these frame systems were also considered in this stage.

2. To carry out a series of full-scale tests on the CRA portal frames, which have been preliminarily designed in the previous steps, to determine the ultimate strengths and gain great insights into the behaviour of the frames, to identify the main parameters affecting the ultimate system strengths, and to examine the possibility of optimal design of the CRA portal frame systems.

3. To conduct component tests to investigate the behaviour and obtain the necessary information for the validation of numerical simulations. The component experimental program included the coupon tests to determine the actual material properties, fastener tests to obtain characteristics of the bolted connections, and the base connection tests to determine their flexural stiffnesses and strengths and the effect of various connection brackets on the bending stiffness of the connections.

4. To develop numerical nonlinear finite element modelling and advanced analyses of the testing frames and connections, accounting for geometric and material nonlinearities and the nonlinear fastener characteristics, for predicting their strengths and behaviours and to carry out parametric studies to extend data.

5. To validate the strengths of CRA portal frames based on the current standards/specifications and propose the newly developed Direct Design Method (DDM) using Advanced Analysis so that the current standards would be evaluated and recommendations would be provided.

6. To perform system reliability analysis and derive system resistance factors for the CRA portal frames.

1.4. Research methodology

In order to achieve the objectives as mentioned above, the following approaches have been adopted to carry out this study:

1. **Literature review**: The relevant existing knowledge on portal frame systems composed of cold-formed sections in the literature is reviewed to gain an understanding of the structural behaviour of the structural systems and associated components. In particular, the structural characteristics and material mechanical properties of aluminium alloys used as a structural material are briefly reviewed as they are important information for the structural behaviour and analysis. Since the cold-rolled sections are susceptible to different instabilities due to the thinness of the section, the prevalent buckling modes for
thin-walled sections are also highlighted. Particular attention is paid to the structural behaviour of cold-formed steel portal frames and associated connections, which serve as the knowledge base for the research on CRA portal frames in this study. Different types of analyses and design methodologies available for portal frame systems are also reviewed and potential possibilities for improving the existing systems and advanced design approach are explored.

2. Experimental investigation: An experimental program consisting of seven full-scale tests on portal frames composed of CRA back-to-back lipped channel sections and sixteen tests on base connections are carried out to determine the benchmark performance of the frames and the base connections. The layouts of the frames are designed on the basis of similar design of cold-formed steel portal frames constructed in industry practice. The frames are subjected to downward loading arrangement, simulating gravity, and combined downward and horizontal loading, simulating combined gravity and wind loading. Various modifications to the portal frames are recommended and tested to evaluate the contribution of associated components to the overall performance of the structural systems. The flexural stiffness of the base connections is measured for both bending about the major and minor axes by the isolated connection tests, which are not only used to verify the results in the full-frame tests but also provide a solid basis for further evaluations. A series of coupon tests and fastener tests are performed to establish material properties and nonlinear behaviour of the bolted connections, which are subsequently used in the numerical finite element simulations and relevant design calculations.

3. Numerical investigation: Detailed full three-dimensional shell element models of CRA portal frames are developed and nonlinear finite element analysis is carried out in the widely-used finite element program ABAQUS [12]. Both actual material and geometric nonlinearities are incorporated into the models. The individual behaviour of the bolted connections is idealised by using the deformable mesh-independent point-based fastener, in which the nonlinear force-slip deformation assigned to the fastener is validated with the component test results conducted in this study. Second-order inelastic analyses are completed to predict the strength and behaviour of the frame systems. Such the approach of analysis is usually known as “Advanced Analysis”. The numerical models are validated with experimental ultimate loads and deformations and parametric studies can be conducted by using the validated modelling technique.

4. Design: The strength of CRA portal frames are determined on the basis of the conventional methods used in the current standards for aluminium structures and the newly developed Direct Design Method (DDM) using Advanced Analysis. The design procedures are highlighted in detail in this study. In conventional methods, the design action effects are obtained from the second-order elastic analysis using the non-
commercial computer program MASTAN2 [13], while the buckling load for members in compression and buckling moment for members in bending are determined by different methods. The design procedures are compared to the experimental ultimate strengths so that the applicability of the current standards/specifications to the design of cold-rolled aluminium portal frames can be evaluated and the potential use of DDM can be explored. A system reliability analysis is also performed to derive appropriate system resistance factors for the CRA portal frames.

The research methodology adopted in the research can be summarised in a flow chart shown in Figure 1.3.

![Figure 1.3. Research methodology](#)

### 1.5. Thesis outline

This thesis consists of six chapters, which are structured as follows:

Chapter 1 presents the problem statement on CRA portal frames, as well as the objectives and research methodology of the thesis.

Chapter 2 contains a comprehensive review of the relevant literature on the behaviour, analysis, and design methodology for thin-walled sections in general and CRA portal frames in particular. The chapter begins with a brief review of aluminium alloys, providing an overview of the potential use of aluminium alloys as a structural material, manufacturing processes, and mechanical and physical properties followed by the highlight of instabilities that are commonly encountered in thin-walled sections. A review on portal frame systems composed of cold-formed sections including the main components of the system, connections, fasteners, fastening methods, prevalent geometric imperfections, and existing experimental studies is described
The chapter concludes with an overview and discussions on different types of analyses, available design methods and the potential use of Advanced Analysis for the design of cold-rolled aluminium portal frames.

Chapter 3 describes a comprehensive experimental investigation performed on the CRA portal frames and associated components. This includes full-scale tests on CRA portal frames, coupon and fastener tests and tests on the base connections. The test objectives, design, preparation, procedures, results, and discussions are included within each series of tests.

Chapter 4 details the development of numerical nonlinear finite element models for CRA portal frames. This contains the idealisation and definition of material properties, fastener properties, contact, boundary conditions, and loading. The technique used to incorporate initial geometric imperfections is also presented. The analyses of numerical models of the frames along with the technique to overcome the convergence deficiencies are detailed in the next part. The numerical models are then validated against the results of full-scale tests which enable their use as analysis tools for further studies, such as parametric studies.

Chapter 5 demonstrates the design procedure to determine the strength of CRA portal frames based on prevailing design methods available in current standards/specifications of aluminium structures and then evaluates the applicability of these standards to design CRA portal frames. The utilisation of DDM as a potential alternative design methodology is highlighted, followed by a system reliability analysis performed for the derivation of system resistance factors for the whole frame systems comprised of CRA sections.

Chapter 6 summarises the research and provides recommendations for future research work.

The Appendices contain drawings, additional information or detailed/specific experimental results, programming codes, and additional information for the design that, nevertheless, form an important part of this research.
CHAPTER 2. LITERATURE REVIEW

2.1. INTRODUCTION

This chapter presents a comprehensive review from previous studies relevant to the research program of this study. It starts with a brief review on aluminium alloy material including manufacturing process and material mechanical properties, followed by the discussions on instabilities of aluminium alloy thin-walled sections and members subjected to different actions. An overview of portal frame systems and their connections are provided in the next part, in which the experiments on cold-formed steel portal frames and associated connections are of particular focus as there is no consistent investigation of such frame systems comprising aluminium profiles. Subsequently, this is followed by the discussions on imperfections prevalent in members and frames and incorporating these imperfections into numerical models. Different types of analyses are described in the next sections. The chapter concludes with a review and discussion on the current design methods and the potential use of Advanced Analysis for the design of frame structures.

2.2. STRUCTURAL ALUMINIUM ALLOYS MEMBERS

Aluminium alloys have been widely and successfully used in the aeronautical industry. These materials are now also used successfully in other branches of transportation, the auto industry and the shipping industry. From the structural point of view, aluminium material and its alloys can be considered as a “new” material for structural components in the construction industry. In fact, the first building structures made of aluminium alloy appeared in Europe in the early 1950s of the last century, when concrete, masonry and steel were the main materials for fabricating civil engineering structures [2-4].

Aluminium material with numerous advanced characteristics such as low self-weight, high corrosion resistance, and functionality of the structural shape has been used to replace other materials in some applications. However, in its pure form, aluminium has a relatively low strength which is insufficient to adapt the structural requirements. Therefore, aluminium alloys have been developed to overcome this challenge and meet users’ expectations.

2.2.1. Aluminium Alloys

Aluminium alloys, known as “light alloy”, have been developed by adding alloy elements to the base aluminium metal, to enhance their material properties [3, 14]. It has been indicated in practice that only several elements have proven to be fully effective as alloying elements for aluminium material suitable for structural applications. They include magnesium (Mg), silicon (Si), manganese (Mn), copper (Cu), and zinc (Zn), which can be used either individually or in combinations, resulting in different families of aluminium alloys with different chemical compositions and material mechanical properties.
Aluminium alloys can be classified according to either their fabrication process (casting or wrought alloys) or the heat treatment (heat-treatable and non-heat-treatable alloys) or on the basis of their chemical compositions. Aluminium alloys can be grouped into different families with similar mechanical and technological behaviour [3]. However, the nomenclature for aluminium alloys may be different in each country. In Australia, a list of wrought aluminium and aluminium alloys and the temper designation system for aluminium alloys is given in Australian Standard AS 2848.1-1998 [15]. Based on chemical compositions, aluminium alloys can be grouped into eight groups, as shown in Table 2.1.

Table 2.1. Aluminium and aluminium alloy groups [15]

<table>
<thead>
<tr>
<th>Designation</th>
<th>The major alloying element(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1XXX</td>
<td>Aluminium, 99.00% min</td>
</tr>
<tr>
<td>2XXX</td>
<td>Aluminium alloy - Copper</td>
</tr>
<tr>
<td>3XXX</td>
<td>Aluminium alloy - Manganese</td>
</tr>
<tr>
<td>4XXX</td>
<td>Aluminium alloy - Silicon</td>
</tr>
<tr>
<td>5XXX</td>
<td>Aluminium alloy - Magnesium</td>
</tr>
<tr>
<td>6XXX</td>
<td>Aluminium alloy - Magnesium and silicon</td>
</tr>
<tr>
<td>7XXX</td>
<td>Aluminium alloy - Zinc</td>
</tr>
<tr>
<td>8XXX</td>
<td>Aluminium alloy - Other alloying element(s)</td>
</tr>
</tbody>
</table>

The alloy designation system consists of four digits as follows: (i) the first digit indicates the alloy group ranging from 1 to 8 (see Table 2.1); (ii) the second digit specifies the modification of an alloy. If this number equals 0, it is an original alloy. If the number is different from 0, it is a modification of an alloy; (iii) the last two digits have no particular meaning as they are only used to distinguish different aluminium alloys in a group (for group 2XXX to 8XXX) or to identify the digits to the right of the decimal point of the aluminium percentage for group 1XXX.

A suffix letter ‘A’ is added to indicate a national variation of an alloy such as 1080A, 2014A or 5154A.

For tempering process, the basic temper designations are defined in [15] as follows:

(i) F temper stands for “fabricated” which applies to products that require some tempers from shaping process without any control over the amount of strain-hardening or thermal treatment.

(ii) O temper stands for “annealed” which applies to fully annealed product

(iii) Strain-hardened tempers (H) apply to the products with their strengths increased by strain-hardening, with or without supplementary thermal treatments to produce partial softening. More details about this temper are available in clause 3.4, AS 2848.1-1998 [15].
(iv) Thermally treated tempers (T) apply to the products thermally treated with or without supplementary strain-hardening to produce stable tempers. More details about this temper are available in clause 3.5, AS2848.1-1998 [15].

For the structural members, the 5000 and 6000 groups are mainly used due to their high strength and formability. While the aluminium alloys 6000 series with the main alloying elements of magnesium and silicon are heat-treated to increase their strength, the aluminium-magnesium alloys (5000 series) can be strengthened by strain-hardening using the cold-working process. Two methods are commonly used to manufacture aluminium alloys structural applications in buildings namely extrusion and cold-rolling processes. The extrusion process has been developed over decades and is the major manufacturing method, whilst the roll-forming process has been recently attempted by BlueScope Permalite [5] to cold-roll aluminium coil into C- and Z sections which are now available in Australian market. Thus, this fact potentially opens the door for commercialising cold-rolled aluminium structural sections worldwide. These manufacturing processes are summarised in the following sections.

2.2.2. The manufacturing process of aluminium members

2.2.2.1. Extrusion process

Although several methods are available to fabricate aluminium alloy structural sections for structural applications, the extrusion process is still the major manufacturing method partly due to the relatively low temperature for aluminium alloy extrusion (approximately 500°C). Among various versions of the extrusion process, the direct extrusion is the most common method to produce aluminium sections. The schematic illustration of the direct extrusion process is shown in Figure 2.1.

In the first step of the extrusion process, the appropriately sized extrusion billets are cut from the long cylindrical logs. The billet is heated up to the extrusion temperature in an induction furnace and then inserted in the press container of the extrusion press (Figure 2.1). The exit of the container is blocked by a drawplate with a hole, namely called “a die”. The shape of the hole defines the shape of the extruded profile to be produced. After the billet is placed, the hydraulic ram at the back of the billet is actuated, causing the material at the front end to extrude through the die. To develop the strength of the extrusions, solution treatment (quenching) or air-quenching is needed for different alloy tempers, followed by artificial or natural ageing as the second stage of heat treatment. The extruded profiles are likely to distort as they come off the press and the heat-treatment operations even make this worse. The profiles are subsequently corrected by applying a high-tension force to the profiles on the straightening table.

The ability to be manufactured using extrusion is one of the main advantages of aluminium alloys as opposed to steel. The extrusion process allows the manufacture of aluminium sections of any shape that may not be achieved by hot-rolling. As a result, the
The geometrical properties of the cross-section can be improved by manufacturing aluminium shapes which simultaneously provide the minimum weight and the highest structural efficiency of stiffened shapes directly produced without using built-up sections, thus avoiding welding or bolting. In addition, the connection details are improved as the connecting system between different components can be simplified. These advantages make extruded profiles remain the most common form of aluminium alloy structural members in buildings and constructions.

![Diagram of extrusion process](image)

**Figure 2.1. Direct extrusion process [16]**

### 2.2.2. Cold-rolled process

Cold-rolling is an alternative to conventional methods in manufacturing structural metal profiles with its unique potential. The roll-forming techniques have advanced rapidly over the last 20 years where the complex steel profiles with edge and intermediate stiffeners for structural applications are now produced by roll-forming systems. Recently, BlueScope Permalite [5] has demonstrated that it is possible to roll-form aluminium coil into lip-stiffened C- and Z-sections to AS/NZS1734 [6] tolerances. The cold-rolled aluminium structural profiles are now available in Australian market. The schematic illustration of the cold-rolling process is presented in **Figure 2.2**.

**Figure 2.2** shows the roll-forming process of a continuous bending operation where long metal strips, typically coiled aluminium, are passed through a series of rollers mounted on consecutive stands at room temperature. Each set of rollers only performs an incremental part of the bend until the desired cross-section profile is produced. Hence, the more complex the cross-sectional shape is, the greater number of roller pairs is generally required. Since the process is consistent and easy to repeat, the roll forming provides a great method to precisely produce high volumes of aluminium structural sections and allows a wide variety of cross-section profiles to be formed.
The cold-rolling method provides a number of advantages as opposed to other manufacturing ones. The first advantage is that the structural strength of the profile is enhanced significantly due to uninterrupted grain orientation in work formed metals. The cold-rolling utilises material more efficiently and is substantially faster and far less energy demanding, leading to reduction of not only plant costs but also carbon footprint as compared to other competing processes. Further, the roll forming process allows labour cost and time for secondary operations to be reduced or eliminated as the operations such as punching and notching can be performed during the roll-forming. The cold-rolling method provides better dimensional control and a high degree of flexibility. A single set of roll-forming tools will make almost any lengths of the same cross-sections, so that multiple sets of tools for varying length parts are not required. Roll-forming also results in a superior surface finish that requires little to no secondary finishing. This is because the surface finish mirrors the smooth condition and dimensions of the dies. Therefore, the use of the cold-rolling process to manufacture aluminium structural profiles not only helps maximise economic benefits for manufacturing companies but also contributes to the sustainable development of the world as cold rolling is more environmentally friendly than competing methods. This fact also motivates and is the main driver for the choice of cold-rolled aluminium alloys sections and systems in this study.

![Figure 2.2. Roll-forming process](image)

2.2.3. Mechanical and physical properties

2.2.3.1. Physical properties

A comprehensive comparison of the main physical properties and the structural behaviours between aluminium alloys, steel, and stainless steel was conducted by Mazzolani[3]. Based on the study, it can be summarised as follows:
- The density of aluminium is approximately one-third of that of steel. It varies from 2600 to 2800 kg/m³ depending on alloys.

- Young’s modulus of aluminium is also approximately one-third that of steel. In different alloys, it varies from 68500 to 74500 MPa.

- The thermal expansion coefficient of aluminium is twice that of steel.

### 2.2.3.2. Mechanical properties

Aluminium alloys feature distinctly different mechanical properties to those of normal carbon steel. A typical comparison of stress-strain relations between ordinary carbon steel and aluminium alloys 5052-H38 is shown in Figure 2.3. The elastic modulus is about a third of steel’s one, and thus greater attention for checking deflections of aluminium structures at service load, particularly relevant to large-span portal frames, is needed. Another noteworthy point is that the aluminium material does not exhibit a distinct yield point with a yield plateau, instead possesses gradual yielding (see Figure 2.3). This type of stress-strain curves may be described as “roundhouse” type [18]. The proportional relationship between stress and strain, and the sharpness of the curve roundedness mainly depend on alloy types and the fabrication process. In the absence of a well-defined yield stress, the 0.2% proof stress is generally used as an equivalent yield strength in structural design [19]. Further, the associated gradual loss of stiffness causes a reduction of the stability strength of aluminium sections and members. Hence, there are fundamental differences between design provisions for aluminium and steel structures. In particular, the ultimate strengths of compressed plate elements, columns, and/or unbraced beams subjected to flexural-torsional buckling are influenced by gradual yielding.

![Figure 2.3. Stress-strain curves of aluminium and carbon steel](image-url)

Although the stress-strain curves obtained from the coupon tests are always preferred, it may not be possible to perform coupon tests for every practical design. In such cases, a common approach of expressing the nonlinear behaviour of aluminium alloys is proposed by using the Ramberg-Osgood expression [20]:

\[ \varepsilon = \frac{\sigma}{E} + \frac{\sigma}{K} + \frac{\sigma}{E} \left( \frac{\sigma}{K} \right)^{n} \]
\[ \varepsilon = \frac{\sigma}{E} + p \left( \frac{\sigma}{\sigma_p} \right)^n \]  

(2.1)

in which \( \varepsilon \) is the strain, \( \sigma \) is the stress, \( \sigma_p \) is the proof stress corresponding to the plastic strain \( p \), and \( n \) is the Ramberg-Osgood parameter, which determines the sharpness of the knee of the stress-strain curve. In the design of aluminium structures, it has been practically common to use the 0.2% proof stress (\( \sigma_{0.2} \)) as the equivalent yield stress \([19]\), so the stress-strain relations take the form as follows:

\[ \varepsilon = \frac{\sigma}{E} + 0.002 \left( \frac{\sigma}{\sigma_{0.2}} \right)^n \]  

(2.2)

The Ramberg-Osgood parameter is determined by using 0.01% and 0.2% proof stresses and given by:

\[ n = \frac{\ln(20)}{\ln(\sigma_{0.2}/\sigma_{0.01})} \]  

(2.3)

Apart from the above characteristics, a notable feature of the cold-rolled aluminium sections is the non-uniform stress distribution resulting from cold-working of corners during the manufacturing process, which is characterised by higher yield stress and ultimate tensile strength in the corners (bends) as compared with the flat portions of the cross-sections. This was confirmed by Huynh et al. \([21]\) for cold-rolled 5052-H36 aluminium alloy sections. Therefore, the corner strength enhancement should be considered in the practice design.

### 2.2.3.3. Residual stresses of cold-rolled aluminium sections

Residual stresses are self-equilibrium stresses, which exist in the structural member without the application of any services or other external loads. Residual stresses need to be carefully treated in the design of structural members as they generally have a significant effect on the brittle fracture, fatigue, stress corrosion, buckling and post-buckling strength of the members \([22]\). The residual stresses in a cold-formed structural section are mainly caused by the bending effect during the roll-forming process. In general, residual stresses are idealised as a superposition of flexural (bending) residual stress and membrane residual stress \([23]\), as defined in Figure 2.4.
To obtain the actual residual stress distribution in a structural member, experimental measurements need to be conducted [22]. A number of measurement techniques can be utilised to measure residual stresses such as X-ray diffraction, magnetic and electrical method, and/or the conventional saw cutting method [24]. Among all the methods the sectioning method is a reliable technique and has been commonly used. This technique was also utilised by Huynh et al. [21] to measure and evaluate the residual stresses in cold-rolled 5052-H36 aluminium alloy sections, as shown in Figure 2.5.

![AC25025 specimen for the sectioning method with strain gauges attached on both sides: (a) Before cutting; (b) After cutting [25]](image)

Huynh et al. [21] reported that the longitudinal membrane residual stresses are relatively small with the magnitude of approximately 3% of the yield stresses, whilst the longitudinal bending residual stresses are considerable with the maximum measured stress up to 30% of the relevant yield stress.

### 2.3. INSTABILITIES IN COLD-ROLLED ALUMINIUM MEMBERS

Cold-rolled aluminium sections can be used efficiently as structural components. However, this efficiency comes along with a complication in the design problems due to the thinness of the section. When the slender elements of a cold-formed member are subjected to compressive stresses, the element deforms in-plane under the compressive stress but also out-of-plane due to the low bending rigidity of the element. The out-of-plane deformation is
strongly associated with the elastic buckling stress of the element. As the cross-section of a member is an assembly of thin elements, several different buckling modes are potentially associated with the instabilities of the member [26].

The member buckling modes depend upon the nature of the actions applied to the members, including compression, bending, shear, and localised loading [27]. For each of these actions, the buckling modes may be classified into three types: (1) local buckling (short half-wavelength); (2) distortional buckling (intermediate half-wavelength); and (3) global buckling such as flexural, torsional, or flexural-torsional (long half-wavelength).

Numerical solution of buckling analysis of thin-walled sections using the semi-analytical finite strip method (FSM) [28] is a very efficient approach for investigating the buckling behaviour of cold-formed members. The finite strip buckling analysis does not distinguish between local, distortional or global modes. As a result, it is required the analysis to be repeated over a range of lengths corresponding to buckle half-wavelengths, especially for local and distortional buckling modes [29]. The buckling modes calculated by the analysis can be plotted using computer graphics, and it is, consequently, a useful method for demonstrating the different buckling modes of thin-walled members [30]. The curve describing the buckling stress versus the half-wavelength of a buckle, commonly called a “signature curve”, was first studied by Hancock [31] for I-sections in flexure in which distortional modes were demonstrated. On the basis of the FSM, a number of computer programs have been developed to perform a finite strip buckling analysis of thin-walled sections and plot the corresponding signature curves, such as CUFSM [32], THINWALL [29], or THINWALL-2 [33, 34]. Figure 2.6 illustrates a typical signature curve for channel sections in bending, where different buckling modes are identified. These buckling modes are summarised in the following sections.

Figure 2.6. Buckling modes of cold-formed steel plain C-sections and SupaCee sections [35]
2.3.1. Local buckling

Local buckling mode involves only in plate flexure without transverse deformation of the line or lines of intersection of adjoining plates, as shown in Figure 2.7. The mode has a strong post-buckling reserve and occurs at short half-wavelengths [30, 36].

![Local buckling modes of C and Z sections](image)

**Figure 2.7. Local buckling modes of C and Z sections**

When the thickness of a plate in a section is small, the cross-section of the member may undergo local plate buckling prior to the member yields or any other buckling modes. While local buckling does not cause immediate failure, it will radically reduce the stiffness of the member against further actions and hasten overall failure [37, 38]. The elastic critical local stress for a plate element in compression, bending, or shear is given by [30]:

\[
\sigma_{cr} = \frac{k \pi^2 E}{12(1-\nu^2)} \left( \frac{t}{b} \right)^2
\]

(2.4)

where \( k \) is the plate local buckling coefficient, depending upon the support conditions, \( b/t \) is the plate slenderness which is the plate width (\( b \)) divided by the plate thickness (\( t \)), and \( E \) and \( \nu \) are respectively the elastic modulus and Poisson’s ratio.

Depending on the restraint conditions along the longitudinal boundaries and the loading patterns, the plate local buckling coefficient (\( k \)) has a different value with a correspondingly different buckling half-wavelength. A summary of the plate local buckling coefficient with the corresponding half-wavelengths of the local buckles is shown in Table 2.2. The open thin-walled sections, such as C-, U, and I-sections, normally consist of stiffened and unstiffened elements. A stiffened element is defined as a flat element, in which both edges parallel to the direction of loading are supported. An unstiffened element is a flat element, in which only one longitudinal edge is supported.

Since local buckling does not normally result in failure of the section, the plate can carry a further load above the buckling point, known as post-buckling behaviour. An example of the post-buckling behaviour of a stiffened element is presented in Figure 2.8. The plate subjected to uniform compressive strain deforms after buckling (Figure 2.8a) and the longitudinal membrane stress is redistributed from uniform compression to that shown in Figure 2.8b, which
moves away from the centre of the plate and towards the stiffened edges. The stress redistribution continues until the stresses at the edges reach the yield stress.

![Figure 2.8. Post-buckling behaviour of a plate: (a) The plate deformations; (b) The redistribution of stresses](image)

The theoretical analysis of post-buckling and failure of plates is extremely difficult and generally requires a computer analysis to achieve an accurate solution [30]. To avoid this complexity in design, von Karman [39] suggested the effective width formulae in which the plate stress was idealised as shown in Figure 2.9. He argued that the non-uniform stress distribution across the width of the buckled stiffened plate could be replaced by two widths ($b_e/2$) on each side of the plate where each width is subjected to a uniform stress equal to the stress acting at the edge of the plate, $\sigma_{max}$.

![Figure 2.9. Actual stress and effective stress distributions](image)

Von Karman also suggested that the plate fails when the elastic critical stress of the plate reaches the yield stress ($f_y$) of the material, which can be found from Equation (2.4) as follows:

$$f_y = \frac{k \pi^2 E}{12 (1 - \nu^2)} \left( \frac{t}{b_e} \right)^2$$

(2.5)

Dividing Equation (2.4) by Equation (2.5) produces

$$\frac{b_e}{b} = \sqrt{\frac{\sigma_{cr}}{f_y}}$$

(2.6)

Compression elements in cold-formed sections contain geometric imperfections and residual stresses resulted in the cold-forming process. In such case, the effective width approach of von Karman results generally in significantly high values for the ultimate buckling resistance.
Winter [40, 41] proposed and verified experimentally a modification of the effective width approach of von Karman as found in Equation (2.7) instead of Equation (2.6).

\[
\frac{b_e}{b} = \alpha \sqrt{\frac{\sigma_{xx}}{f_y}} \left(1 - \beta \sqrt{\frac{\sigma_{xx}}{f_y}}\right)
\]  

(2.7)

Winter [40] proposed \( \alpha = 1.0 \) and \( \beta = 0.25 \) for stiffened compression elements. Based on a long period of accumulated experience, Winter [42] later indicated a more realistic value of \( \beta \) of 0.22 to determine the effective width of stiffened compression elements. For unstiffened compression elements, they are \( \alpha = 1.19 \) and \( \beta = 0.298 \), respectively.

Table 2.2. Plate buckling coefficients [30]

<table>
<thead>
<tr>
<th>Case</th>
<th>Boundary Conditions</th>
<th>Loading</th>
<th>Buckling Coefficient (k)</th>
<th>Half-wavelength</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>S.S  S.S  S.S</td>
<td>Uniform Compression</td>
<td>4.0</td>
<td>( b )</td>
</tr>
<tr>
<td>2</td>
<td>S.S  Built-in S.S</td>
<td>Uniform Compression</td>
<td>4.97</td>
<td>( 0.66b )</td>
</tr>
<tr>
<td>3</td>
<td>S.S  S.S  Free</td>
<td>Uniform Compression</td>
<td>0.425 ( b )</td>
<td>( L = \infty )</td>
</tr>
<tr>
<td></td>
<td>Free S.S S.S</td>
<td></td>
<td>0.675 ( b )</td>
<td>( L = 2b )</td>
</tr>
<tr>
<td>4</td>
<td>S.S  Built-in S.S</td>
<td>Uniform Compression</td>
<td>1.247</td>
<td>( 1.636b )</td>
</tr>
<tr>
<td>5</td>
<td>S.S  S.S  S.S</td>
<td>Pure Cending</td>
<td>23.9</td>
<td>( 0.7b )</td>
</tr>
<tr>
<td>6</td>
<td>S.S  S.S  S.S</td>
<td>Bending + Compression</td>
<td>7.81</td>
<td>( b )</td>
</tr>
<tr>
<td>7</td>
<td>S.S  Free S.S</td>
<td>Bending + Compression</td>
<td>0.57</td>
<td>( L = \infty )</td>
</tr>
<tr>
<td>8</td>
<td>S.S  S.S  S.S</td>
<td>Pure Shear</td>
<td>5.35 9.35</td>
<td>( L = \infty ) ( L = b )</td>
</tr>
</tbody>
</table>

L = Plate length, \( b \) = Plate width
2.3.2. Distortional buckling

Since the thin plates of a cold-formed section are likely prone to local buckling, edge and intermediate stiffeners are incorporated into the cross-section in order to increase the effectiveness of the compression plate elements. As a subsequence, the local buckling behaviour was improved by the presence of stiffeners. However, this improvement may lead to the occurrence of a new relevant buckling phenomenon commonly known as distortional buckling.

Distortional buckling, also known as “flange distortional buckling” or “stiffener buckling”, is a mode characterised by rotation of the flange and lip about the flange-web junction in opposite directions, as shown in Figure 2.10, in members with edge stiffened elements [26, 30, 43]. In members with intermediately stiffened elements, distortional buckling is characterised by displacement of the intermediate stiffener normal to the plane of the element [43]. The half-wavelength of distortional buckling is generally intermediate between the half-wavelengths of local and global buckling modes and approximately 3-6 times that of the half-wavelength of local buckling [44].

![Figure 2.10. Distortional buckling modes of C and Z sections](image)

Research on distortional buckling started much later compared to local buckling in the literature. In the 1950s and 1960s, researchers recognised the phenomena, now known as distortional buckling, when studying on the cold-formed steel columns and beam. They considered it as too complicated to predict analytically [43]. Sharp (1966) [45] presented an early theoretical treatment of “overall” buckling – i.e., distortional mode. By simplifying the rotational restraint at the web-flange junction, the distortional buckling stress of a lipped channel was approximately calculated. This was verified with the experiments carried out by Dwight [46].

In 1977, Desmond developed an analytical method for predicting the distortional buckling, which later was the basis for the AISI Specification on the edge stiffened elements. However, distortional buckling was considered as another local mode and was not treated as explicitly different from local plate buckling. During this research period, distortional buckling was restricted as local buckling phenomena. Several researchers such as Thomasson [47] and Mulligan [48] provided straps/braces across the flanges to ensure that distortional buckling did not occur and therefore making the local mode dominant.
In the 1980s researchers began to focus on the distortional buckling. It was most evident by the pioneering work of Hancock [49] at the University of Sydney. A series of experiments on cold-formed steel storage racks conducted by Hancock et al. [49-51] indicated that the distortional buckling occurred separately and its post-buckling capacity was lower than that of the local buckling. The reason for this lower post-buckling strength reserve was explained by Sridharan [52]. He demonstrated that membrane stresses at the lips increased quickly after distortional buckling. As a result, the collapse of the member is hastened due to the yielding of one of main sources of member stiffness.

From the middle 1980s until now, a wide interest on the topic has been propelled all over the world not only for cold-formed steel but also for other materials such as stainless steel and cold-rolled aluminium. Notable studies on distortional buckling phenomena of cold-formed steel can be listed as research works at Sydney University [53-57], Cornell University [58, 59], Johns Hopkins University [60], Monash University [61-63] and The Technical University of Lisbon [64-68]. The main focus of the research is to develop conceptual and analytical methods for obtaining a more accurate prediction of the distortional buckling behaviour and evaluate its post-buckling capacity. The phenomena were also investigated experimentally and numerically for cold-formed stainless steel sections [69, 70] and recently for cold-rolled aluminium structures [71-74].

2.3.3. Global buckling

Member or global buckling phenomenon includes flexural, torsional, or flexural-torsional buckling of columns and/or lateral-torsional buckling of beams. The global buckling mode is characterised by the fact that the whole cross-section, without distortion, bends laterally, rotates, or bends and rotate simultaneously [26]. The mode involves in large deformation whilst the material remains in the elastic region or, in other words, the failure mode is not related to the material yielding but to elastic instability because of material stiffness and geometry. Long natural half-wavelengths are associated with this type of buckling and they are determined by the member’s unbraced length.

2.3.3.1. Global buckling of columns

2.3.3.1.1. Euler buckling (flexural buckling)

In 1759, Euler recognised that apart from material crushing and yielding, there was another criterion influencing the column strength, in which a larger change in geometry is caused by a small change in the loading. This was called the buckling phenomenon, commonly known as Euler buckling, and was the earliest recognised buckling mode. The differential equation for determining critical load is given by [75]:

\[ \frac{d^2y}{dx^2} + \frac{4EI}{k^4} y = 0 \]
where \( y \) is the lateral deflection of the column, \( x \) is the coordinate along the longitudinal axis, and \( EI \) is the flexural rigidity. The Euler buckling load is obtained by solving Equation (2.8), and the solutions for different boundary conditions were presented by Timoshenko and Gere [75]. The general solution can be written as:

\[
P_{cr} = \frac{\pi^2 EI}{L_e^2}
\]

where \( L_e \) is the effective length depended on the specific boundary conditions.

### 2.3.3.1.2. Torsional buckling

Torsional buckling refers to as the case in which a column will buckle either by twisting or by a combination of bending and twisting. Such torsional buckling modes occur when the torsional rigidity of the section is very low, as seen in a column of the thin-walled open section [75].

A column of the generally unsymmetrical cross-section is considered, as shown in Figure 2.11 with \( x-x \) and \( y-y \) being the principal axes.

![Figure 2.11. Unsymmetrical open cross-section [44]](image)

For this column under the action of an axial load applied through the centroid of the cross-section, the global buckling load \( P \) is given by the following expression [76]:

\[
r_z^2 (P - P_x)(P - P_y)(P - P_z) - P^2 y_0^2 (P - P_x) - P^2 x_0^2 (P - P_y) = 0
\]

where \( P_x \) and \( P_y \) are flexural (Euler) buckling loads about the \( x-x \) and \( y-y \) axes, respectively, and \( P_z \) is the torsional buckling load, as given below, for simply support columns:
\[ P_x = \frac{\pi^2 EI_x}{L^2} \]  
(2.11)

\[ P_y = \frac{\pi^2 EI_y}{L^2} \]  
(2.12)

\[ P_c = \frac{1}{r_2^2} \left( GJ + \frac{\pi^2 EI_w}{L^2} \right) \]  
(2.13)

where

\[ r_2 = \sqrt{\frac{I_x + I_y}{A} + x_0^2 + y_0^2} \]  
(2.14)

\( I_x \) and \( I_y \) are second moments of area about the \( x-x \) and \( y-y \) axes, \( G \) is the modulus of rigidity, \( J \) is the torsion constant, \( I_w \) is the warping constant, \( A \) is the area of the cross-section, and \( x_0 \) and \( y_0 \) are the coordinates of the shear centre.

For columns with different end conditions other than simple supports, Equations (2.11) ÷(2.13) are still valid by replacing the length \( (L) \) by respective effective lengths \( L_{ex}, L_{ey}, L_{ec} \).

For double symmetric sections such as back-to-back channel sections intendedly used in this study, the shear centre and centroid coincide, and \( x_0 \) and \( y_0 \) (coordinates of shear centre) are equal to zero. In such cases, Equation (2.10) is satisfied if the buckling load \( P_{cr} \) is equal to any of the individual flexural or torsional buckling loads. Hence, the buckling behaviour is either purely torsional or purely flexural.

### 2.3.3.2. Global buckling of beams

Lateral–torsional buckling in a beam is basically equivalent to flexural-torsional buckling in a column. In the common case, a beam, which is bent in the plane of the greatest flexural rigidity, may buckle laterally with a degree of twisting at a certain critical value of the load. This buckling mode can be critical for thin-walled open sections as they are generally weak in the lateral direction and in torsion. This global buckling mode of a beam was well studied by many researchers such as Timoshenko [75], and Trahair [76]. The elastic buckling moment can be obtained by either solving the differential equilibrium equation or by the energy approach for a beam under various boundary and loading conditions. For a very basic case, the elastic buckling of simply supported beams with doubly-symmetric cross-sections under uniform moments is given by [76]:

\[ M_{cr} = \sqrt{\left( \frac{\pi^2 EI_y}{L^2} \right) \left( GJ + \frac{\pi^2 EI_w}{L^2} \right)} \]  
(2.15)

where \( E \) and \( G \) are the Young’s and shear stress moduli respectively, \( I_y \) is the second moment of area about the minor axis, \( J \) is the torsion constant, \( I_w \) is the warping constant, and \( L \) is the member length. Similar to the determination of the critical flexural-torsional buckling load for
columns, different end conditions of beams can be accounted for by replacing the length $L$ by the respective effective lengths. In addition, bending moment distribution other than uniform is also taken into account in design codes by using a factor $C_b$ - bending coefficient.

Research studies on the global buckling of beams indicated that only limited post-buckling capacity may be expected from this type of buckling. Therefore, the elastic buckling load is a reasonable prediction of the ultimate strength for beams.

### 2.3.3.3. Shear buckling

Shear buckling in the webs of short and deep beams needs to be carefully paid attention as shear buckling may also be either local or global. Local shear buckling can occur in the slender webs of members subjected to bending or in the wide flanges of sheeting and decking profiles subjected to diaphragm actions. Global shear buckling generally only arises when relatively lightweight sheeting or lining profiles are subjected to diaphragm actions [77].

Analysis of the shear buckling stress of flat rectangular plates has been thoroughly investigated by many researchers [75, 78-80]. For a thin plate of length $a$, width $b$, and thickness $t$ with simply supported conditions along all four edges subjected to uniformly distributed shear stresses along the edges, the shear buckling stress is given by [75]:

$$
\tau_{cr} = k_v \frac{\pi^2 E}{12(1-\nu^2)} \left( \frac{t}{b} \right)^2
$$

(2.16)

where $E$ is the elastic modulus, $\nu$ is the Poisson’s ratio, and $k_v$ is the shear buckling coefficient depending on the boundary conditions and the aspect ratio of the plate $a/b$. For simply supported rectangular plates, the value of $k_v$ as a function of the aspect ratio of the plates ($a/b$) can be determined by using the following approximate formulae:

$$
k_v = \begin{cases} 
4.00 + \frac{5.34}{(a/b)^2} & \text{for } \frac{a}{b} \leq 1 \\
5.34 + \frac{4}{(a/b)^2} & \text{for } \frac{a}{b} > 1
\end{cases}
$$

(2.17)

(2.18)

The conventional approach has been to investigate shear plate buckling in the web only whilst the effect of the whole section including the flanges has been ignored. However, recent researches [81-87] have proven that the flanges can have a significant effect on the improvement of the shear buckling capacity of thin-walled sections. Solutions have been provided to determine the shear buckling load of complete thin-walled sections under pure shear parallel with the web by using a Spline Finite Strip Analysis (SFSM) [88] and a signature curve was developed for thin-walled sections in a similar manner to that for bending and compression.
2.4. PORTAL FRAMES

Portal frames are structural systems that were first developed in the 1960s, and are now commonly used in the construction of single-storey buildings [89]. They are mostly used to provide a larger open space for industrial, farming, and residential applications such as residential houses, barns, shelter, sheds, auto workshops, garages and/or warehouses. During the history of development, various forms of portal frames have been designed and constructed. However, the single-span portal frame tends to be the most popular structure due to its structural efficiency and simplicity in design and construction.

In the past, most of the portal frames were made from hot-rolled steel sections, which were motivated by the availability of exhaustive resources of design guidelines on hot-rolled steel sections. With the recent advances in design technologies and an adequate understanding of the behaviour of the cold-formed steel (CFS) sections, the CFS portal frames have been developed as a viable alternative to the traditional ones from hot-rolled steel sections for spans up to 12 m or more [90-92]. The advantages of using cold-formed sections instead of conventional hot-rolled one for a portal frame system are not only due to its lightweight and the use of high strength steel but also due to the facts that are summarised as follows [90]: (1) frame components can be pre-fabricated at the factories according to design specifications and delivered to the site ready for installation so that the quality is better controlled; (2) the transportation costs can be reduced significantly with the efficient stacking of cold-formed sections; (3) Since both primary and secondary cold-formed members can be purchased from the same manufacturer, the acquisition costs are reduced; (4) the sections are pre-galvanised during the cold-forming process, and thus do not require additional coatings to prevent rusting; (5) the frames can be constructed by both skilled artisan and unskilled labourers without the need for an on-site crane; (6) smaller foundations are required as the structure is lighter.

Along with the development of steel structures, aluminium structures have become more popular in providing new solutions to compete with steel. Until recently, Bluescope Permalite [5] has successfully produced aluminium structural C- and Z-sections by roll-forming aluminium coils, potentially leading to strength enhancement as compared with extrusion process and it has proven as a more cost-effective method of production. This allows the company to be able to market a complete structural system using CRA products with great resistance ability against galvanic corrosion and 100% recyclability.

On the basis of the success of CFS sections as discussed above, it is believed that the use of such portal frames composed of CRA sections is promising. The performances of CRA sections are even more competitive than those made from CFS ones, especially in the applications where the prerequisites including lightness, corrosion resistance and functionality are important. This end-user application has not been previously attempted by any producer of aluminium profiles. Hence, in order to develop an appropriate CRA portal frame system, a review on the composition, available studies, and issues related to the analysis and design of
CFS portal frames need to be performed carefully. The choice of CFS portal frame as the subject to review is mainly motivated by the availability of exhaustive resources of references and the similarity in structural behaviour.

2.4.1. Components of portal frames

A typical single-span portal frame system consisted of a series of portal frames erected at regular bays, accompanied by a description of its main components is illustrated in Figure 2.12. Each portal frame serving as the main load-carrying structures of the system comprises columns and rafters connected at the apex, eaves, and base connections through the respective brackets. Structural bolts are generally used to fasten members at these connections.

The secondary members, supporting the cladding, consists of purlins for the roof and girts for the wall systems. These secondary members are connected to the rafters and columns by using cleats whilst corrugated roof sheeting and wall cladding are attached to the purlins and girts, respectively. In terms of structural analysis, loads from the cladding are transferred to the secondary members, and ultimately to the primary frames in the form of reactive force at discrete connection points between secondary members and columns/rafters. In addition to transferring loads to the frames, purlins and girts also play an important role in restraining the rafters and columns against out-of-plane buckling [93].

![Figure 2.12. Main structural components of single span portal frames](image)

It is common in the practice that knee braces and apex braces (i.e. rafter ties) are provided to improve the stiffness of the primary frame, and the ultimate strength of the whole system is therefore increased and the deflections are reduced relatively.

Fly bracings are commonly used to provide restraints to the compression flange of columns and rafters at the critical regions with significantly large compression forces and
bending moments such as at the eaves and apex regions. In the longitudinal directions of the system, K-bracing or cross bracings are also provided at either the end bays or within the central portion of the frame system [93].

2.4.2. Portal frame connections

Connections are important components of every structure not only from the structural behaviour point of view but also in relation to the cost of production. It is estimated that approximately 40% of the total costs of steel structures are directly or indirectly influenced by the connections including the base connections [95, 96]. It is also agreed that the strength of the connections may often dictate the strength of a cold-formed member or system [97]. Therefore, well-designed joints are essential to provide the satisfactory performances of a structure.

In aluminium structures, welding may be preferred for general engineering purposes due to the simplification of fabrication and assembly. However, where site assembly is required, joints with mechanical fasteners such as bolts or rivets may be more competitive. Furthermore, such joints provide useful system damping which is virtually absent in continuously welded structures and there is no softening of materials due to the effect of heat [98, 99]. Therefore, bolted connections are intended to use as the fastening method for the cold-rolled aluminium portal frames in this study. The choice of bolted connections is also motivated by the success of bolted connections used in cold-formed steel portal frames in several previous studies [90-92, 100-104]

Connections used in portal frame systems in this study include joints in the primary frames and secondary joints such as connections between purlins and purlin cleats. These connections using bolts as fasteners are discussed in the following sections.

2.4.2.1. Bolts, methods for the tightening of bolts and failure modes of bolted connections

Bolted connections are more frequently used than any other types of connections due to the ease of operation without the requirement of special equipment. Bolts with associated nuts and washers are threaded fasteners which are assembled in pre-drilled holes through the material elements to be joined. Figure 2.13 shows the main parts of the bolt assembly. The grip is the distance between the bolt head and the back of the nut or washer, which is the total thickness of all parts being joined. Bolts threaded close to the head are often used in the thin members such as cold-formed steel or cold-rolled aluminium as the sum of the plate thicknesses are normally small.

Currently, normal and high-strength carbon steel bolts are the most commonly used fasteners in the metal structures due to the availability of design criteria and lower initial costs as compared with other competing bolts such as aluminium and stainless steel bolts. However, stainless steel bolts have become increasingly attractive among researchers and engineers because of their high level of durability, high ductility [105-107], superior fatigue behaviour [108], and excellent corrosion resistant [106, 107, 109]. Especially in aluminium structures,
stainless steel bolts exhibit a more pronounced advantage than that of their steel counterpart. The use of steel bolts is only allowed when adequate superficial protection is provided to prevent corrosion by contact (electromechanical) between aluminium and steel, whilst contact corrosion does not occur between aluminium and stainless steel [3].

![Figure 2.13. Bolt assembly components [110]](image)

In terms of load transferring, there are two basic load transfer mechanisms in bolted connections as shown in Figure 2.14. In bearing-type joints, loads are transferred between components by bearing on the bolts. The bearing strength of the connection relies on the shear strength of the bolt and the bearing resistance of the connected plates. Meanwhile, in friction type connections, also referred to as slip-resistant connections, loads are initially transferred by friction resistance between the bolted components and subsequently by bearing on the plates once the friction is overcome [110-112].

![Figure 2.14. Load transmission in bolted connections: (a) Bearing-type connection; (b) Slip-resistant connection [110]](image)

In the slip-critical joints or slip-resistant joints, the frictional resistance between the bolted plates is developed by a clamping force created when the bolts are pre-tensioned. It is generally suggested that bolts in friction connections should be tightened such that the resulting pretension is at least 70% of the minimum specified tensile strength of the bolts [111, 112].
There are three common methods of tightening bolts used to induce the desired level of pretension in the bolts, and thus achieve the required clamping force to develop the frictional resistance between bolted components. They include the turn-of-nut method, torque method, and the combined method \cite{113, 114}.

In the turn-of-nut method, the bolts are first tightened to the snug-tight condition which can be attained by a few impacts of an impact wrench or the full effort of a person using a standard spud wrench or Podger spanner, as guided in \cite{112}. Both the bolts and nuts are then marked, and the bolt is held in place while the nut is further rotated by a prescribed rotation (see Figure 2.15). The specified rotation varies depending on the diameter and length of the bolt. The drawback of the method is the difficulty to measure the pretension in the bolt. However, this is the simple but widely used method for bolt tightening. In the torque method, a wrench is calibrated or adjusted to shut off when the desired torque is reached. However, the calibrated wrench method indicates a significant scatter in installed pretension \cite{112}. Hence, this method is not permitted by some guidelines \cite{110}. In the combination method, the bolts are tightened in two stages. The nut is initially tightened using the torque method to approximately 65% of the yield load, and then the turn-of-nut method is utilised to induce full pretension.
Mechanically bolted joints are conveniently classified on the basis of the type of forces which the bolted are subjected to. These classes are (1) shear, (2) tension, and (3) combined tension and shear. The basic types of failure for bolted connections subjected to shear and tension can be predicted as shown in Figure 2.16 and Figure 2.17 respectively.
Apex and eaves joints along with base joints are the main connections of the primary frames. While the apex joint is used to connect two rafters at the ridge, the eaves joints connect the rafters to the columns. In the portal frames using cold-formed sections, these connections are usually constructed using thin connection brackets bolted to the columns and rafters. In case of longer span frames, knee braces and rafter tie can be used at the eaves and apex connection respectively for stiffening and stabilising the frame. These connections need to be carefully designed as the performance of the portal frame is directly dependent on the performance of the connections used in the portal frame [116].

It is well-known that the conventional hot-rolled steel portal frames depend on the rigidity of the joints at the knees and apex to transfer the applied loads through bending actions of the members to the foundations. The joints are often assumed to be 100% rigidity [117], and thus the plastic design method can be utilised as in standard design practice. On the other hand, most cold-formed section portal frames are designed elastically. The requirement for rigid joints,
which are often expensive to fabricate or difficult to construct, may not be therefore as important [118]. Over the past decades, researchers have undertaken tests on cold-formed steel portal frames and joint arrangements, which can be used for the eaves and apex connections. The research studies on the joints mainly focused on quantifying the rotational stiffness of the joints and they can be considered to be semi-rigid [119-122]. The semi-rigidity of the joints is due to the bolt-hole elongation (the bearing effect), caused when the joints resist applied bending moments [90, 119] and the deformation is induced by the local or distortional buckling of the thin-walled sections [119]. Lim and Nethercot [120] indicated that the stiffness of the bolted connections depends on several factors including (1) thickness of the cold-formed plate; (2) diameter of the bolt hole; (3) material properties of the cold-formed plate; (4) diameter of the bolt; (5) the material of the bolt; (6) whether the bolt-shank is plain or threaded. The rotational stiffness of the joints can be improved by either the length of bolt-group or the number of bolts used for the bolt-group [123].

Among the joints in portal frames, the eaves joint is usually the most highly stressed portion of the portal frame under any loading as the maximum bending moment occurs at the eaves joint (for frames without the knee braces) [124, 125]. It was also found in the portal frames without knee and apex braces that the bending moment at the apex joint is generally 50-60% of the knee bending moment [117].

Over the past decades, several researchers conducted tests on different arrangements for the eaves and apex joints of cold-formed steel portal frames. In the following paragraphs, the reported eaves and apex joints for portal frames with both single C-section and double C-section joined back-to-back are summarised.

The tests on single C-section joints have been reported by Baigent and Hancock [100], Mills and LaBoube [117], Dundu and Kemp [125], and Rinchen and Rasmussen [126], as follows:

Baigent and Hancock [100] used 12 mm thick plates to form brackets for the eaves and the apex joints in their experimental program on pitched roof portal frames. The eave connection is shown in Figure 2.18a, in which the single C section rafters and columns were connected together in between the bracket and a cover plate using bolts having a diameter of 20 mm. The joints could be considered as rigid connections due to the high-tensile grip bolts.

Mills and LaBoube [117] conducted tests on various eaves joints connecting single C25015 section columns and rafters. The joints included an end-plate bolted to the column and welded to the rafter (Figure 2.18c), a mitred joint (Figure 2.18d), and a screw joint (Figure 2.18e). Mills and LaBoube demonstrated that self-drilling screws can be used as an alternative to conventional bolting and that the screw connection process can provide a rigid joint with no possible slip between the connected components, unlike bolted joints where oversize holes are necessary for construction and slip is an unavoidable phenomenon. It is also suggested that screws should be positioned as close as possible to the flanges of the C-section member used.
in such eaves joints to prevent local buckling of the webs. Two types of apex joints, welded ridge joint and self-drilling screw ridge joint, were also studied by Mills and Laboube. It was reported in the test of the welded apex joints that the failure occurred in the channel member adjacent to the joint whilst the weld remained intact. On the other hand, the screw apex joints in which a lipped channel was used as the gusset plate and screwed back-to-back to the rafters, failed by tilting and shearing of screws. The ultimate moments obtained from tests for the four screw joint (Figure 2.19a) and eight screw joint (Figure 2.19b) were 3.84 kNm and 6.97 kNm, respectively.

Dundu and Kemp [125] performed research on bolted eaves joints formed by single channels connected back-to-back, which has a similar arrangement as the screw joint presented by Mills and LaBoube [117]. Different failure modes, including local buckling of the compression zone of the flange and the web of the channels, lateral-torsional buckling of the channels between points of lateral support, and bolts in bearing, were observed and evaluated. Based on test results, it was indicated that the failure is governed by the local buckling of the compression flange and web.

Recently, experimental and numerical studies on eaves and apex joints used for the single C30024 section portal frames were performed by Rinchen and Rasmussen [126]. Three configurations of the eaves joint were tested, as shown in Figure 2.18f. The joint was formed by bolting two C30024 sections to a 3mm thick specified eaves bracket (Figure 2.18g) using twelve M16 8.8/TF bolts induced full pretension to present a friction-type connection. Zed stiffeners (Figure 2.18h) were also used to enhance the rigidity of the connections. These arrangements were identical in all configurations while modifications were made by using M8 bolts in place of screws that were used to fasten the lip of the bracket to the lips of the channels or adding a stiffener plate to the eaves connection. From the tests, the failure modes were observed and moment-rotation characteristics of the connections were obtained. It was demonstrated that the failure modes of the eaves joints are dominated by bending of the brackets, followed by the fracture of screws or tearing of the channel lips, and the ultimate moment capacities vary from 25.7 kNm to 30 kNm. A similar arrangement for the apex joints was also studied. The joint was formed by bolting two ends of C30024 section rafters to a specified apex bracket using twelve M16 8.8/TF bolts. The lower lips of the channels and the lip of the bracket were connected using two 5.5 mm diameter screws, forming it the first configuration of the apex joint. Minor modifications were introduced in the two other configurations by replacing screws by M8 bolts along with the use of downward lipped bracket or provision of additional screws to the first configuration. These configurations are shown in Figure 2.19c. The tests demonstrated that the apex connections failed by the inelastic local buckling of the compression flang-web junction of one of the channel sections, and the fracture of screws was not the precursor to the attainment of ultimate loads of the connections [126].
Figure 2.18. Common eaves connections in cold-formed steel single C-section portal frames
There are also a number of research studies on the eaves and apex connection for CFS portal frames composed of back-to-back sections published and are reviewed as follows:

Kirk [103] reported an experimental study on the Swagebeam portal frame building. The Swagebeam sections were bolted back-to-back for columns and rafters, whilst the eaves and ridge brackets with the inclusion of swageing, which interlock with the section, were used for the eaves and apex joints. The Swagebeam eave bracket, apex bracket, and details of the eaves joint are shown in Figure 2.20a, Figure 2.20b, and Figure 2.20c, respectively. The joints were formed through back-to-back brackets bolted between the webs of the swagebeam sections. The use of such joints provides advances in design as it not only produces rigid joints with no slip under load, but also reduces the number of bolts required by 50% [103]. It was also evident from the tests that the joints were strong enough to ensure that failure occurred in the members and not in the brackets.

Makelainen [127] demonstrated a test program on the eaves and apex connections of the thin-gauge portal frame with cold-formed back-to-back sigma sections. In the eaves joints tests (Figure 2.20d), different bracket configurations were considered, as presented in Figure 2.20e. These included a single plate of thicknesses of 8 mm, 10 mm, and 12 mm, four layers cold-formed steel plates of 2.5 mm each, and four cold-formed plates with two outers of these
outwardly lipped. The depth of the sigma sections used for the eaves joint tests were 300 mm, whilst the thicknesses of 2.5 mm and 3.0 mm were examined. In the ridge joint tests, sigma sections with the heights of 250 mm and 400 mm and thicknesses of 2.5 mm and 3.0 mm were adopted, whereas connection bracket used a compact steel plate with a thickness of 12 mm. An equal angle tie bar of $50 \times 50 \times 2.5$ mm was used in the tests to provide additional stiffness to the connections. The moment-rotation characteristics of the studied connections were obtained from the experiments. On the basis of the moment-rotation relationships, analytical models of the semi-rigid connection behaviour were developed for the connections, which were then used in design calculations of the portal frame.

Chung and Lau [121] reported an experimental investigation on bolted eaves connections with three types of gusset plate (bracket). In all tests, the connections were formed by connecting the webs of the back-to-back lipped channel sections with the gusset plate using bolts. The aim of the test series was to examine the effects of the geometry and material of the gusset plates together with associated bolt arrangements on the performance of the eaves joints, so that gusset plates of three different shapes, i.e. triangle, rectangular, and haunched, fabricated from both the 6 mm thick hot-rolled steel plates and 2.5 mm thick cold-formed steel strips were tested. The tests demonstrated that the connections using gusset plates of hot-rolled steel are stiffer and provide higher moment resistance as compared with those using cold-formed steel gusset plates. It was also evident that the connections with haunched gusset plates give much higher moment resistance as compared with the other connection configurations and are able to form stiff and strong moment connections with a minimum rotational stiffness of 750 kNm/rad. In general, the moment capacities of the eaves connection were found to vary in a range from 22% to 84% of the capacity of the connected members, depending on the shapes and the materials of the gusset plates.
(a) Swagebeam eaves brackets [103]

(b) Swagebeam ridge brackets [103]

(c) Swagebeam eaves joints [103]

(d) Eave joints for back-to-back sigma section [127]

(e) Eave joints with different brackets configuration after Makelainen [127] (sketched by Wrzesien and Lim [118])

(f) Triangle bracket [121]

(g) Rectangular bracket [121]  
(h) Haunched bracket [121]

Figure 2.20. Common eaves connections for double channel portal frames

Similar to the joint arrangement tested by Chung and Lau [121], Lim and Nethercot [90, 120, 123, 128, 129] presented tests on the bolted moment-connections formed by bolting the back-to-back brackets to the webs of the CFS channel sections. Such arrangement was applied for the eaves and apex connections of cold-formed back-to-back sections portal frames, as illustrated in Figure 2.21. Unlike Chung and Lau, the joints tested by Lim and Nethercot isolated the failure of the brackets from that of the sections. With the concept that the moment capacity of the brackets used for the connections is designed to be greater than that of back-to-back channel sections, or in other words, the brackets themselves would not fail. Research was
focused on the strength and stiffness of the cold-formed sections in the immediate vicinity of the joint, as influenced by the bolt-group size.

![Image](image_url)

(a) Eaves joint arrangement  
(b) Apex joint arrangement

Figure 2.21. The bolted moment-connections applied for the eaves and apex joints of cold-formed portal frames [90]

Dubina et al. [119, 130] reported an experimental program carried out to evaluate the performance of apex and eaves joints of pitched roof cold-formed portal frames under monotonic and cyclic loading. Three different configurations of the joints, namely (KSG and RSG), (KIS and RIS), and (KIP and RIP), were tested. Of which, the first group of specimens (KSG and RSG) represented the joints where space gussets were used as brackets. In this group, the brackets were bolted only through the webs of the back-to-back sections, whilst in other groups, welded I-sections only (KIS and RIS) and welded I-sections with plate bisector (KIP and RIP) represented two different types of the connecting bracket. Bolts were provided either on the webs only or both on the webs and flanges. The details of these joints are shown in Figure 2.22. It was reported that the force distribution is unequal due to the flexibility of the connected members with a bigger force distributed to the outer bolt rows compared to the most inner one and that the flanges bolts would increase the moment capacity and the rotational stiffness of the bolt group. In such cases, the joints failed by stresses concentrating in the vicinity of the first row of bolts on the flange, and there was no initial elongation of the bolt holes. Buckling initially occurred in the flange and then extended into the web.
Figure 2.22. Bolted connection after Dubina et al. [130]: (a) Apex joints; (b) Eaves joints; (c) bolt configuration in the cross-sections

Rhodes and Burns [131] conducted extensive component tests on stanchion base, apex and eave connections with knee braces for cold-formed portal frames with the aims of assisting the development of the frame system, validating the design, checking and modifying connection details, and obtaining experimental stiffnesses for incorporation into the design software. The arrangement of the eaves joint with knee brace tested by Rhodes and Burns is shown in Figure 2.23. The sections were composed of double lipped channel sections bolted back-to-back for
the columns and rafters. At the eaves, the joint was formed by bolting a pair of angle brackets to the webs of the columns and the flanges of the rafters, and another pair of angle stiffeners were provided for the rafters to avoid the failure of the rafter flanges under concentrated load. The knee braces were formed from back-to-back channel sections bolted to the column and rafter by using various types of brackets. In the first type of the studied bracket used for the connections between brace and rafter and brace and column, thick plates were fitted between the channel sections of all members (the column, rafter, and knee brace). Although this connection method provided good results, it had some undesirable features, which involved in complicated erection issues. The gusset plates were then replaced by welded brackets in combination with angle sections to form an angle connection type. By using the angles with the depth covering the full web in rafters and columns, it was reported that the angle connection can produce a capacity equal to the thick plate connections. The knee-braced joint arrangement has been proved to be more advantageous over other joint arrangements reported in the previous literature as it allowed close to rigid joints to be fabricated inexpensively and significantly improved the overall sway of the frame through the use of the knee braces [118].

Zhang et al. [132] carried out experiments to investigate the structural performance of cold-formed steel pitched portal frames subjected to local and distortional buckling (cross-section instability) prior to global failure. For this purpose, tests on eaves and apex joints were also conducted to investigate the behaviour of these joints and to ensure that the bending capacity of the joints was larger than those developed at the joints in the full-scale frame tests. The details of the eaves and apex joints tested by Zhang are shown in Figure 2.24. Back-to-back channel sections were utilised for the members while eaves and apex brackets were formed from 6 mm G350 mild steel. This arrangement is similar to that of Lim and Nethercot [123], but in the joints of Lim and Nethercot, bolts were provided on the webs only, whilst in the connection presented by Zhang et al., the brackets were bolted to both the webs and flanges of
the adjoining C-section members. It was reported that by providing bolts to the flanges of the sections, the capacities of the joints increased significantly.

![Diagram of joint configuration](image)

Figure 2.24. Configuration of the joints tested by Zhang et al. [132]: (a) Eaves joint; (b) Apex joint

Experimental and numerical studies on apex connections of portal frames composed of back-to-back double sigma profile rafters were completed by Ozturk and Pul [133] with the aim of evaluating the effect of changes in gap between rafter ends on the performance of the apex joints with and without stiffener.

Recently, Blum and Rasmussen [101] have performed tests on a large span multiple bay haunched portal frame system, which comprised cold-formed doubly-symmetric back-to-back C-sections with knee braces connected in between the columns and rafters. The tests demonstrated that knee braces play an important role in the performance of the frame system. Tests on the apex connections, used for these frames, were also reported in [134]. The stiffness of the apex joints was quantified for different rafter channel thicknesses and depths along with the corresponding apex brackets, which used all 2.4 mm lipped L-brackets bolted back-to-back through the webs but varied in size for the respective sections. It was shown that all joints failed
by buckling of the apex brackets and occurred well below yield capacity of the bracket and rafters and that there was no visible bolt-hole elongation found in the tests.

2.4.2.3. Base connection

The portal frame system has been proved to be sensitive to the stiffness and the strength of the connections including base connections [101, 134]. The column bases are typically assumed to be either ideally pinned or fully rigid in the design practice. However, the column base connections are naturally semi-rigid as they tend to exhibit stiffness and moment capacity [135]. The overly simplified assumptions of pinned or fully rigid connections are not ideal. In general, the base joints in cold-formed section portal frames are formed by connecting the columns to the foundation through the base brackets using bolted connections. Depending on the material and geometry of the base brackets, the column sections, and the bolt configurations, the rotational stiffness developed at the base connections can vary vastly.

The effect of the base fixity on the behaviour of the Swagebeam section portal frames was studied by Robertson [136]. It was found that the base fixity reduced significantly lateral displacements and modified the bending moment distributions as compared with those predicted from the assumption of pinned bases. The base restraint provided a beneficial effect on the stability and effective lengths of the columns. It was suggested that rotational stiffness should be included in the design practice and typical stiffness values in the range 500÷3500 kNm/ rad can be utilised.

Chung and Lau [121] performed tests on the column base connections consisted of cold-formed back-to-back channel sections bolted to the triangulated tee sections fabricated from 6 mm and 10 mm hot-rolled steel grade 43 to evaluate the effect of bolt arrangement on the performance of the base connections. It was determined that a base connection with four bolts resulted in better results compared to the joints with two and three bolts since the stiffness and moment capacity were improved significantly. It was shown that the rotational stiffness of the four-bolt connection is 175 kNm/ rad.

The π-shaped and L-shaped mild steel brackets with the thickness of 3 mm, 4.3 mm, and 5.6 mm were used as the base brackets for the column base connections as tested by Kwon et al. [137, 138]. The closed section columns were inserted into the brackets and fixed using self-drilling screws, whilst the base was firmly anchored to the massive concrete block by two chemical anchor bolts. The tests indicated that the base connections exhibited the semi-rigid behaviour, and the performance of the joints can be improved remarkably by increasing the thickness of the brackets. In addition, the π-shaped connection provided higher initial flexural stiffness and rotational ductility compared with the L-shaped connection.

Dundu [95] investigated the behaviour of the angle cleat base connections of the CFS single C-section portal frame system. The tests were carried out on four different configurations, as shown in Figure 2.25, with the angle cleats made from CFS and hot-rolled
steel and different bolt arrangements. Dundu reported that premature deformation of the angle cleats was experienced in all tests with cold-formed angle cleats, whilst failure occurred by local buckling of the channel section with the hot-rolled angle cleat connections in this study.

Figure 2.25. Angle cleat base connection: (a) Angle cleat connected in the flanges only (elevation); (b) Angle cleat connected to the flanges and web (Plan view) [95]

Another arrangement of the base connection for single C-section portal frames has been recently tested by Rinchen and Rasmussen [126]. In the tests, the base joints (see Figure 2.26) were formed by bolting the flanges of the C30024 section column to a 6.0 mm thick base strap. The base strap was clamped to the base plate through a 10 mm thick steel clamp plate using three M20 8.8/TF bolts. The base tests were conducted for the columns bent about both the major and minor axes, and thus the flexural stiffnesses were obtained. The average initial stiffness was 1256 kNm/rad for major axis bending and 119 kNm/rad for minor axis bending, respectively.
An intensive testing program on the column base connections used for haunched double-channel back-to-back section portal frame was performed by Blum and Rasmussen [134] to quantify the flexural stiffnesses of the base connections about the column major and minor axes. Various base brackets were tested to evaluate their effects on the base stiffness, including 5 mm, 6 mm, and 8 mm L-shaped and a 5 mm U-shaped brackets. The U-plate was fabricated by welding a mild steel plate in between 2 L-plates to form a complete section. The brackets were bolted to the flanges of the back-to-back section column using four bolts each, as illustrated in Figure 2.27, whilst the brackets were bolted to the base plate, which was also used in the tests presented by Rinchen and Rasmussen [126]. The flexural stiffness for major axis bending ranged from 4.87 kNm/degree to 7.69 kNm/degree depending on the thickness and the shape of the bracket, whilst the stiffness for bending about the minor axis was small. It was also noted that the U-shaped bracket provided a stiffer connection for both major and minor axes compared to L-shaped bracket with the same thickness. Besides, the full-frame tests [101] demonstrated...
that this arrangement of the base connection was sufficient to carry loads as there was no failure occurred at the base connection.

(a) U bracket connected to the base plate  
(b) The column bolted to the bracket

Figure 2.27. The base connection use for double C back-to-back portal frames [134]

2.4.2.4. Secondary connections

The secondary connections in the frame system may include the connections between purlins/girts and purlin cleats or the connections between the claddings and secondary members (purlins/girts).

In the connections between purlins/girts and cleats, the purlin and girt cleats are typically subjected not only to axial loads but also to bending moments. However, they are generally used without analysis or design [139]. A standard purlin cleat supplied by Bluescope Permalite is shown in Figure 2.28a. The sizes of the cleat depend on the size of utilised purlin, for example, if an AC15025 purlin is used, the width of 210 mm (A+B+C) and the length of 252 mm (D+E+F) can be adopted for the cleat [140]. It is also suggested that the standard bolts used for the secondary connections are usually Grade 316 Stainless Steel bolts M12 or M16. The washers at both the heads and the nuts are also essential. The standard punched holes in purlins are 18 mm high by 22 mm long (Figure 2.28b) and the standard hole diameter in the cleats is 18 mm. As a result, these hole sizes are too big for M12 bolts without washers. Further, rigid plastic grommet washers under the bolt heads or the washers are recommended to reduce thermal movement, noise and any risk of crevice corrosion in highly corrosive environments. The use of PVC isolation tapes to the full width of the flange should be used to separate dissimilar materials [140].
The claddings are fastened to the outside of purlins and girts using either screws or hooks. They indirectly improve the buckling capacity of the main members by providing elastic restraints to the purlins and the girts.

### 2.4.3. Past experimental studies on portal frames composed of thin-walled sections

Tests on cold-formed section portal frame systems have been recorded since the 1980s. These include the portal frames consisted of single sections and back-to-back sections with failure modes unique to each test.

The first tests reported in the literature on thin-walled section portal frames were those by Hancock [141], which aimed to investigate the three-dimensional collapse behaviour of plane frames with a variety of lateral and torsional restraints. Four tests on the pitched roof portal frames with the frame geometry and cross-section shown in Figure 2.29, were completed. The I-cross-section was fabricated from 3.2 mm thick plates using penetration weld. Bolted end plate connections were used for both eaves and apex joints. High tensile bolts were used to connect the end-plates at each joint, resulting in the rigid joints. Vertical loads were applied to the frame at four points along the frames (see Figure 2.29a) by suspending lead blocks on platforms supported by the hangers. A wide range of lateral and torsional restraints attained by using a designed restraint system was considered and their effect on the strength and behaviour of the frame system was evaluated. Various general observations were made about the out-of-plane response of all frame tests. In all the lightly restrained tests, a stable elastic post-buckling response was achieved, whilst in the medium restrained tests, inelastic buckling caused by yielding occurred. In the heavily restrained tests, significant yielding resulting from an in-plane bending moment was observed in the frames before an inelastic lateral-torsional failure occurred.
In 1982 Baigent and Hancock [100] performed an experimental study of seven pitched roof portal frames composed of single lipped channel sections with the pinned bases, as shown in Figure 2.30. The column and rafter channels were bolted together through their webs at the eaves and ridge using respective brackets formed from holt-rolled steel plates with a thickness of 12 mm. Due to high-tensile grip bolts, the joints could be considered as rigid connections.

In the test program, two lateral restraint configurations were considered with three different load sets for each. While the first four tests were performed with external restraints only, the other three tests were completed with both external restraints and internal restraints. External restraints were provided at sixteen locations, as shown in Figure 2.30, to simulate the effect of purlins and girts. Meanwhile, internal restraints were provided near the eaves and apex regions of the frame to simulate the effect of fly bracing and involved prevention of lateral...
movement of the internal flange of the frame. The three load conditions used in the study included gravity load, transverse wind load and longitudinal wind load. The loads were applied to the frames as point loads at the restraint points.

It was shown that the frames with both external and internal restraints experienced higher strength compared to the frames with external restraints only. This fact proved that the fly bracings contributed significantly to the strength enhancement of the frame system, especially in the frames subjected to wind loads. The tests also demonstrated that the collapsed loads in all frames were significantly higher than the respective yield loads, indicating the ability of the cold-formed portal frames to carry loads in the post-buckling and post-yielding range. At critical cross-sections of the frames, strain gauges were placed around the section to determine the stress distributions around the cross-section during experiments. The reported data showed that the most highly stressed points occurred at the flange-web junction and the section immediately below the eaves joints would yield first.

Kirk [103] conducted a series of tests on CFS portal frames composed of back-to-back Swagebeam-sections. The frames had an eaves height of 3 m and the roof slope of 15°. The columns and rafters were bolted together at the eaves and ridge through brackets. The frames were tested in pair with 2.5 m wide bay spacing, one frame with pinned column bases and the other with fixed column bases. Six pairs of frames were completed with spans of 9 m and 12 m. Cross bracings were also provided to prevent the frames from failing in the out-of-plane direction. Vertical loads were applied to the frames at the purlin positions through a system of steel rods and spreader beams using two hydraulic jacks per frame. The test arrangement is shown in Figure 2.31.

Tests on a 12 m span of the portal frames demonstrated that the ultimate strength of the frame with fixed bases was 63.3 kN, which is higher by 16% compared to that of the pinned base frame. The failure mode of the frames was the buckling of the compression flanges, which initiated at the eaves and subsequently at the apex region. It was also reported that the deflections of the fixed base frames were smaller than those of the pinned base frames.
Wilkinson and Hancock [142] tested three pinned base portal frames constructed from CFS rectangular hollow sections (RHS) of 150 mm x 50 mm x 4 mm using grade 350 and grade 450 steel. The frames with a span of 7 m, an eaves height of 3 m, and a total height of 4 m were tested under gravity and wind loads. The general layout and test setup of the frames are shown in Figure 2.32.

At the eaves, the columns and the rafters were welded together using full penetration butt weld, whereas a moment-resisting bolted end plate connection was used at the apex of the frames, which was formed by bolting the 10 mm thick gusset plates butt welded to each end of the RHS rafter using eight M16 high strength structural bolts. In the experiments, a collar tie made of a pair of channels were used to transfer vertical loads from a hydraulic jack to the midpoint of the rafters, whilst the horizontal load was applied to one of the columns at 2 m above the base using a load cradle and a roller system. Lateral restraints were also provided to prevent lateral buckling of the frames. The test results indicated that a plastic collapse mechanism was formed in all frames near the loading point on the rafter and below the eaves joint. It was also pointed out that such frames can be designed plastically as the connections are sufficiently ductile to undergo required plastic rotations.
A numerical and experimental investigation on the effect of the bolted joints, which were proved to be semi-rigid, on the behaviour of CFS portal frames was completed by Lim and Nethercot [90]. In the study, the columns and rafters of the portal frame were made of back-to-back channel sections. The eaves and apex joints were formed by bolting the webs of the back-to-back sections to the 3 mm thick steel brackets, as shown in Figure 2.21. Two pinned base portal frames using different sizes of brackets and bolt-group with a span of 12 m, a height of 3 m, and a pitch of $10^0$ were utilised for both numerical and experimental studies. The general arrangement of the frame test is shown in Figure 2.33, where the cross marks indicate the locations of lateral restraints provided to prevent the lateral buckling failure of the back-to-back sections.
The reported study showed that the bolted joints were semi-rigid mainly due to the bolt-hole elongation developed when the joints resisted the applied bending moments. It was also demonstrated that the rotational stiffness and the connection-length of the joints can significantly affect the bending moments and deflections of the frames. Of the two tested frames, the portal frame with the shorter bolt-group length failed at a load approximately 20% lower than that of the other with the longer bolt-group length due to the development of bi-moment [143].

Dubina et al. [104] carried out two full-scale tests on the portal frames composed of CFS back-to-back channel sections using bolted joints to assess the performance of the frames under horizontal loading, with particular emphasis on earthquake loading. The frames were designed for the experimental purpose with a span of 12 m, an eaves height of 4 m and a pitch of 10°. The test setup consisted of two parallel portal frames with a space of 1.5 m, connected by purlins installed on the top of the rafters. Cross-bracings were installed between the two frames to provide out-of-plane stability. The schematic representation of the test setup is shown in Figure 2.34. In the first test, only lateral loading was applied, whilst in the second frame test, a total of 31.2 kN gravity loading was applied on each frame, followed by increasing lateral load until failure. It was reported that the studied cold-formed portal frames with bolted connections were characterised by a rapid degradation of strength after the first local buckling presented in the members, which was attributed to the similar rapid drop in moment resistance of cold-formed cross-sections. Therefore, it is possible to estimate the ultimate strength of the frame system at the attainment of the moment capacity in the most critical section using an elastic analysis. It was also recommended that axial forces needed to be considered as they may influence the moment resistance of cold-formed members.

![Figure 2.34. Experimental setup for full-scale portal frame tests of Dubina et al. [104]](image-url)

Zhang and Rasmussen [144], Zhang et al. [132], and Zhang [145] conducted a series of four tests on portal frames composed of back-to-back channel sections to investigate the effect of cross-sectional instability on the behaviour of the overall frame system. All frames had the
same nominal geometry as shown in Figure 2.35. Each frame has a span of 8 m, an eaves height of 4 m, and a pitch of 14°. The eaves and apex joints were formed by bolting both the webs and flanges of the connecting members to the 6 mm G350 mild steel brackets, resulting in extremely stiff connections. At the column bases, pinned bearings were used to create the pinned-base conditions. The lateral restraints were spaced at 1.35 m in both columns and rafters to prevent twisting and lateral movements. Thereby the columns and rafters failed by bending about the major axis. Two load configurations were investigated in the study where the first three frames were tested with vertical load, and the last frame was tested with the combined vertical and horizontal loads.

![Figure 2.35. Portal frame tests of Zhang et al. [132]](image)

It was shown in all the tests that local buckling occurred in one column near the eaves joint when the applied load reached 50% of the ultimate load, followed by the local buckling of the other column. The local buckling then extended to the full length of the column. When the load was close to the ultimate strength of the frame, distortional buckling occurred and the frames ultimately failed due to the formation of a spatial plastic mechanism on the flanges and webs of one column at the location immediately below the eaves joints. The test results also indicated that the occurrence of local and distortional buckling reduced significantly the horizontal stiffness and ultimate load of the frames.

Blum and Rasmussen [101] performed an experimental program consisting of a series of nine long-span CFS portal frames with double channel sections connected back-to-back. The test setup included three portal frames connected in parallel with purlins between rafters to create a free-standing two-bay system with a bay spacing of 3.6 m, as shown in Figure 2.36. Several frame configurations including variations in the knee connections, sleeve stiffeners in the columns and rafters, braced and unbraced columns by girts, and loading combinations were studied. In general, it was determined from the study that the capacity of the portal frame system
is sensitive to the stiffness and the strength of the connections, and the inclusion of sleeve stiffeners has minimal impact on the ultimate strength of the frame. It was also reported that the failure of the frame with braced columns was due to the localised deformations of the apex bracket and the rafters at the location of the knee connections while the unbraced frames failed by lateral-torsional buckling of the columns initiated by lateral movement of the knee brace-to-column connection.

![Figure 2.36. Experimental setup of the cold-formed steel double-channel portal frames tested by Blum and Rasmussen [113]](image)

Recently, Rinchen and Rasmussen [102] presented a series of full-scale tests on long-span CFS single C-section portal frames with bolted joints considered as semi-rigid connections. The test configuration was similar to that of Blum and Rasmussen [101], which consisted of two-bay portal frames with a centerline span of 13.6 m and an apex height of 6.8 m. While the two end frames formed by double-channel sections with knee bracing at the eaves regions were reused from the tests of Blum and Rasmussen [101], the central frame (i.e. test frame) composed of single C30024 section columns and rafters connected at the base, eaves, and apex connections by using connection brackets, which were specifically designed and fabricated for the single C-section portal frames. The geometry of the test frame is presented in Figure 2.37. Six tests including variations in the used connection brackets, the restraint condition of the columns, and the load configurations were completed. The tests on CFS portal frames demonstrated that torsion of the columns, followed by flexural-torsional buckling, is the dominant structural behaviour as a result of the eccentricity between the shear centre and the centroid of the cross-section. The flexural-torsional deformations of the columns were exhibited
even with the presence of girts serving as lateral restraints for the columns, although a stiffness enhancement and a much smaller twisting were observed. It was also reported that the ultimate failure deformations were concentrated at the ridge and eaves regions and were governed by the fastener types used for the connections between the column lips and the brackets. By comparing the ultimate capacities of the frames determined by using the direct strength method (DSM) available in the current design standard [146] and that obtained from the tests, it was indicated that the bi-moment has a significant influence on the capacity of the cold-formed steel portal frames.

![Diagram of cold-formed steel single-C section portal frame geometry tested by Rinchen and Rasmussen [102]](image)

Figure 2.37. Cold-formed steel single-C section portal frame geometry tested by Rinchen and Rasmussen [102]

Apart from the research studies on structural behaviour and strength of the portal frame systems and their connections, the effect of stressed-skin actions from roof sheeting and wall cladding on the performance of the portal frame system was also studied by many researchers, such as Mahendran and More [147], Phan et al. [148], and Wrzesien et al. [122, 149]. It was demonstrated that stressed-skin action can create a significant reduction in bending moments and deflections in the frames, especially when the frames are subjected to lateral load. However, it is commonly found in practice that portal frames are analysed without considering the effect of the stressed-skin diaphragm action.

2.4.4. Geometric imperfections in a portal frame system

All actual structural members in reality are imperfect. The imperfections can be classified into initial geometric imperfections, material imperfections, and mechanical imperfections [150]. While the material imperfections refer to the residual stress or the yield strength distribution across the section, the mechanical type pertains to the supporting conditions and loading introduction. Meanwhile, geometric imperfections refer to the deviation of a member or a structure from its perfect geometry.
In practice, the members and the entire structure of a portal frame system are not perfectly straight due to geometric imperfections caused by unavoidable disturbances during production, transportation, and assembly processes, whilst the structural analysis is often performed on a perfect structural concept. The structure is idealised to be perfectly straight or perfectly flat or curved surfaces. As a result, there may be considerably different in the performance (strength, deflection,…) between the real structure and the idealised model. Large discrepancies existed between the theoretical and experimental results caused by the presence of small imperfection in the structure were pointed out by Koiter [151] in 1945.

Since the initial geometric imperfections, which are the major factors contributing to nonlinear behaviour of a structure, have also a detrimental effect on the strength and stability of the whole system. They need to be considered in the structure analysis. In the portal frames, the geometric imperfections can be typically categorised in sectional imperfections, member imperfections and frame imperfections and will be discussed in the following sections.

**2.4.4.1. Sectional and member imperfections**

The geometric imperfections of a member refer to the deviation from the member perfect geometry [23]. This can be broadly classified into three categories: (1) sectional imperfections which are the deviations in cross-sectional shapes; (2) member imperfections, which are characterised by the cross-section displacement and/or rotation as a rigid body without distortion; and (3) localised imperfections characterised by dents and regular undulations in the plates. The sectional imperfections can be further classified as distortional and local shapes. Similarly, member imperfections can be further classified as Bow (bending in minor axis), Camber (bending in major axis), and Twist (twisting about the longitudinal axis of the member). The representation of sectional and member imperfections for a member composed of back-to-back channel sections is shown in Figure 2.38.

**Figure 2.38.** Representation of sectional and member imperfections: (a) Distortional; (b) Local; (c) Bow; (d) Camber; (e) Twist

**2.4.4.2. Frame imperfections**

Frame imperfections consist of the deviation of the overall frame from its perfect geometry. Although various types of deviation can be classified as the frame imperfections, it
is typically associated with a sway shaped imperfection, which is defined by an out-of-plumb angle.

Out-of-plumb has a profound effect on the load capacity of the frame system as it may increase the second-order effects on the members and trigger the sway buckling failure. The Eurocode 9 – Design of aluminium structures [152] includes the provision that imposes the frame imperfections through an initial out-of-plumb angle for global analysis of frames as follows:

\[ \phi = \phi_0 \alpha_h \alpha_m \]  

(2.19)

where:

\[ \phi_0 = \frac{1}{200} \]  

(2.20)

\[ \alpha_h = \frac{2}{\sqrt{h}}, \quad \text{but} \quad \frac{2}{3} \leq \alpha_h \leq 1.0 \]  

(2.21)

\[ \alpha_m = \sqrt{0.5(1+1/m)} \]  

(2.22)

in which, \( h \) is the height of the column in metre and \( m \) is the number of column in a row including only those columns which carry a vertical load not less than 50% of the average value of the column in the vertical plane considered.

2.4.4.3. Incorporating imperfections into numerical models

Integrating imperfections in numerical models for a frame system is more complex than for a single member as not only the magnitude but also the configuration and orientation of the imperfections need to be considered to have the direct influence on the response of the frame system in the ultimate state [153]. There are several approaches, which can be used to incorporate geometric imperfections into the finite element analysis of the portal frames. Of which, the most well-known methods are listed hereby, including the scaling of Eigen buckling modes, the direct and explicit modelling of measured initial geometric imperfections, and application of notational horizontal forces.

2.4.4.3.1. Scale Eigen buckling modes

One of the most widely used methods of introducing geometric imperfections of a structure into a numerical model is to superimpose the eigenmodes. The method concept is that the most critical imperfect geometry is the closest to the final collapsed configuration. The method is performed by carrying out a classic elastic buckling analysis on the perfect geometry of the structure and then scaling the shape of the first few buckling modes. These modes are frequently assumed to provide the most critical imperfections and will be added into the perfect structure. The scaling factors are generally computed from the measurements. Once the scaled
buckling modes are superimposed onto the perfect geometry, the non-linear analysis can be performed to predict the behaviour of the structure.

The selection of buckling modes are important as different shapes of imperfections have a different effect on the member buckling strength [150]. The selection of buckling modes varied among researchers in the literature. Either the first buckling mode [154] or a combination of buckling modes [150, 154-158] or a combination of buckling modes obtained from modified structures to avoid coupled modes [159, 160] can be used to model the initial imperfections.

The incorporation of initial geometric imperfections by using buckling modes is convenient. However, it has been found that this method is not grounded in physical reality as collapse mechanisms do not typically resemble buckling mode shapes. Also, the use of buckling mode shapes for imperfections is not able to capture the localised deviations commonly presented in practice [161].

2.4.4.3.2. Notional horizontal force

An alternative method of integrating imperfections in numerical models is the notional horizontal force (NHF) method, as proposed by Liew [162]. In this method, the frame out-of-plumbness is modelled by introducing an additional horizontal force at the top of each column of the undeformed frame, where the notional horizontal forces are assumed to act in any one direction at a time [163]. Figure 2.39 illustrates the application of the notional horizontal force method to model frame imperfection to a simple portal frame. Similarly, out-of-straightness can also be simulated by applying a lateral distributed force along the member or a concentrated force at the mid-height of the member.

Figure 2.39. Imperfection model using notional horizontal force method [163]

The NHF method is incorporated in many steel and aluminium design standards. The European code BS EN: 1999-2007 [152] recommends that the effects of initial sway imperfection and out-of-straightness of members can be replaced by systems of equivalent horizontal forces introduced for each column. For initial sway imperfection, an equivalent horizontal force of $\phi N_{Ed}$ is applied at the top of each column, whilst a uniform distributed force of $8N_{ed}e_{0,d}/L^2$ on each column can be used for out-of-straightness of the columns, where $\phi$ is the initial out-of-plumb angle, as given in Equation (2.19), $N_{Ed}$ is the design value of the compression force in the column, and $e_{0,d}$ is the design value of the maximum amplitude of an imperfection.
2.4.4.3.3. Explicit modelling of initial geometric imperfection

Another common approach of incorporating imperfections is by direct perturbation of geometry based on the measured data of imperfections. This method can be used to model either frame global imperfections or local sectional imperfections which may not enable to be integrated by other methods [164]. In this approach, the actual measurements of imperfection are incorporated into the numerical models by directly modifying the mesh geometry. The coordinates of the mesh nodes are altered from their perfect geometry to the desired imperfect geometry.

Although this method is numerically harder to implement, it provides an accurate representation of the imperfect model and is able to define any shape of geometric imperfection including a combination of pure buckling modes. This method has been used by various researchers [150, 155, 165-171].

2.5. METHODS OF ANALYSIS

Structural analysis is an integral part of any structural engineering projects, which is the prediction of the performance of the proposed structure. In general, a frame system consists of members connected at the joints, and the overall frame behaviour is therefore often more complicated as compared with the behaviour of an individual member as a result of interactions between the members. It is even more complex in thin-walled frames due to the specific material, the geometrical layout, the joint details, and the loading conditions. Depending on the arrangement of the structure, material types, and loading arrangement, the behaviour of a frame structure may fall into one of the categories represented by the typical load-deformation responses as shown in Figure 2.40.

![Figure 2.40. Structure behaviour [172]](image)

Central to the study of the frame behaviour are the various types of structural analysis that have evolved over the years. Analysis methods of the frame structures are typically identified on the basis of the assumptions regarding the material behaviour (elastic or inelastic) and geometrical linearity (linear or non-linear). The available types of structural analysis corresponding to the structural behaviour depicted in Figure 2.40 are presented in Figure 2.41.
Of these methods, the first-order elastic analysis is the most basic type of analysis, whilst second-order inelastic analysis is the most advanced type of analysis due to the capability to encapsulate all other types of analysis where both material and geometric non-linearity are taken into account. It should, however, be emphasised that the more complex the analysis is, the greater the computational power is required, and more sensitive the results will be created due to incorrect assumptions. Therefore, structural engineers need to assess the relative advantages of each analysis method and make a decision of which type of analysis to employ for a specific application.

Figure 2.41. Structural analysis [172]

2.5.1. First-order elastic analysis

The first-order elastic analysis, also referred to as Linear Analysis (LA), is a common analysis method for many structures. This is based on linear elastic constitutive relationships and ignores any geometrical nonlinearities and associated instability problems [172]. The LA is characterised by the linear proportion of the structural response to the applied load, and the principle of superposition can be therefore used to simplify the analysis. For a linear static analysis, the force-deformation relations are defined by an equilibrium equation given by [173]:

\[ Ku_i = F \]  \hspace{1cm} (2.23)

where \( K \) is the stiffness matrix, \( u_i \) is the deformation, and \( F \) is the applied load.

Stiffness relations are formulated on the basis of the undeformed configuration of the structure. Hence, this type of analysis is limited to small displacement. For a relatively large loading, the structure is likely to undergo large deformation (geometric nonlinearity) and stresses in the material are increased beyond the elastic limit. In such case, the prediction of the force-deformation relationships based on the LA becomes inaccurate. Further, a first-order elastic analysis will underestimate the forces and the deformations of a structure with the presence of instabilities, which are the characteristics of the frames comprised of cold-formed sections.
2.5.2. Elastic buckling analysis

It is well-known that the most significant nonlinear influence in the elastic behaviour of frames is the influence of axial forces on the flexural stiffness of members. Particularly, the compressive forces decrease the flexural stiffness. When a set of compressive member axial forces is increased to the extent that the bending stiffness of the frame as a whole reduces to zero, the frame becomes unstable \([174]\). Elastic buckling analysis, also called Linear Buckling Analysis (LBA) or Eigenvalue buckling analysis, is generally used to estimate the critical buckling load of an ideal linear elastic structure that corresponds to a state of bifurcation of equilibrium of the structure according to \([173]\):

\[
\mathbf{u}_b^T \left[ \mathbf{K}_0 - \lambda \mathbf{K}_G \right] \mathbf{u}_b = 0
\]  

(2.24)

where \(\mathbf{K}_0\) is the linear elastic stiffness matrix and \(\mathbf{K}_G\) is the geometric stiffness matrix which accounts for the effect of axial force on the flexural stiffness of the member, \(\lambda\) is the buckling load factor and \(\mathbf{u}_b\) is the buckling modes.

At the bifurcation, the stiffness of the structure vanishes and thus critical conditions for the complete structure occur when:

\[
\left| \mathbf{K}_0 - \lambda \mathbf{K}_G \right| = 0
\]  

(2.25)

Equation (2.25) represents a standard eigenvalue problem where the lowest eigenvalue defines the critical load factor \(\lambda\) and the corresponding eigenvector defines the buckled mode. By using this type of analysis, the relative displacement representing the buckling mode shapes can be obtained, but the actual displacement cannot be obtained. In addition, the buckling loads predicted by the LBA are often different from the buckling load predicted by the Advanced analysis as pre-buckling deformation, initial imperfection and material non-linearity cannot be incorporated in this analysis method.

Nevertheless, the buckling shapes find their use in representing geometric imperfections, especially in thin-walled sections. Besides, the load factors obtained from LBA can be used to amplify the results of an elastic first-order analysis to account for the effect of instability resulting in the amplified first-order elastic analysis, so that \([173]\):

\[
\mathbf{u}_{\text{ia}} = \frac{1}{1-1/\lambda} \mathbf{u}_i
\]  

(2.26)

For thin-walled members, a more efficient method for investigating the buckling behaviour of the members is the finite strip buckling analysis \([28]\). Several computer programs such as CUFSM \([32]\) and THINWALL \([29]\) have been developed to perform a finite strip buckling analysis of thin-walled sections under compression and bending to obtain the local, distortional, and global buckling stresses. The recently developed software THINWALL-2 \([33, 34]\) has included options for different boundary conditions, no-uniform stress, localised and shear loading cases. In addition to these, another software called GBTUL 2.06 has been
developed by Bebiano et al. [175] for the buckling and vibration analysis of thin-walled members based on the Generalized Beam Theory.

2.5.3. Second-order elastic analysis

While the first-order elastic analysis is the most basic type of structural analysis, which is limited to small displacement as the analysis based on the geometry of the undeformed structure, the second-order elastic analysis, often known as Geometric Nonlinear Analysis (GNA), in contrast, is a relatively more complicated approach, accounting for the influence of elastic instability and changes in the effective stiffness of the members. The equilibrium equations of the GNA are formulated with respect to the deformed geometry of the structure, whilst the material is assumed to remain linear elastic. Since the influence of deformations on the state of stress of the structure is considered, the principle of superposition does not apply to this type of analysis.

Allowance for the effects of instability can be made by using a second-order elastic analysis, in accordance with [173]:

\[
[K_0 - K_e]u_2 = F
\]  \hspace{1cm} (2.27)

The effects of geometrical imperfections may be required in some codes, such as in [152], and can be approximately accounted by using equivalent geometrical imperfections \( u_i \). The shape of these is usually taken to be the same as that of the lowest elastic buckling mode \( u_b \) [173], so that:

\[
u_i = \alpha_i u_b
\]  \hspace{1cm} (2.28)

in which \( \alpha_i \) is the scaling factor generally provided in the standards. It is also often convenient to replace the equivalent imperfections by equivalent loads, as follows:

\[
F_i = K_0 u_i
\]  \hspace{1cm} (2.29)

These induce first-order deformations equal to the imperfections, whence:

\[
[K_0 - K_e]u_2 = F_i + F
\]  \hspace{1cm} (2.30)

In the second-order elastic analysis, the force-displacement relations must be determined iteratively because the deformed configuration of the structure is not known in advance and is constantly changing with the applied loads. In effect, the nonlinear responses are approximated by a series of linear analyses. In each iteration, the equilibrium of external and internal forces must be satisfied and the stiffness matrix is updated from the solution of the previous iteration.

2.5.4. Plastic analysis

The analysis of statically indeterminate structures near the ultimate load is complicated by the decisive influence of the material non-linearities. Many frames have very small axial forces and instability effects, in which case the plastic analysis is applicable. According to this
a sufficient number of plastic hinges must form to transform the frame into a collapse mechanism [172, 173].

This analysis approach is generally limited to the frames with the members comprised of compact sections and the material is assumed to be an elastic-perfectly-plastic behaviour, which means that, beyond the elastic limit, the stress remains constant while the strain increases. It is also implicitly assumed that there is no buckling failure occurred before the plastic collapse is reached [176]. In the plastic analysis, the maximum moment that the member can carry is the plastic moment capacity of the cross-section and the remaining moment is distributed to the elastic sections of the members. The plastic analysis can be divided into the rigid-plastic analysis, first-order elastic-plastic analysis, and second-order elastic-plastic analysis, corresponding to the assumption of no elastic deformation, small elastic deformation, and large elastic deformation before the progressive formation of plastic hinges at member ends in structures.

In the routine design of cold-formed section frames, the plastic analysis may have little relevance as the cold-formed members are normally slender.

### 2.5.5. Advance analysis

The use of a non-linear, inelastic analysis to directly determine the overall strength and the stability of a framing system is widely referred to in the literature as “Advanced Analysis” [177]. In the advanced analysis, which is also called the Geometric and Material Nonlinear Analysis with Imperfections (GMNIA), both the geometry and the material nonlinearities are included. While the geometric nonlinearity includes second-order effects associated with $P-\delta$ and $P-\Delta$ effects and geometric imperfections, the material nonlinearity includes gradual yielding associated with the influence of residual stresses and flexural [178].

Over the decades, great attention has been devoted to the research studies on advanced analysis [153, 177-188], which have focused on modelling requirements, serviceability considerations, and design procedures when the strength of structures is explicitly assessed in the analysis. The motivation for the application of advanced analysis is due to its simplification of the structural strength assessment and this analysis can provide the engineer with greater design flexibility [177].

To predict the nonlinear response of structures by the advance analysis, it is necessary to formulate an exact mathematical representation of the nonlinear behaviour of the structures, including the formulation of equilibrium equations in a deformed configuration of the structure. However, the mathematical formulation of full nonlinearity is often difficult and once formulated, it is hard to solve. To circumvent this issue, the nonlinear problem can be approximated as a series of linear problems in small time steps using incremental load or displacement approach, whereas ensuring static equilibrium is maintained within each time step [189, 190]. Iterations are generally required to achieve the static equilibrium within the time
step through the development of various nonlinear algorithms with each providing the same solution.

The successful implementation of Advanced Analysis in the prediction of structural behaviour is mainly motivated by the concurrent development of computer programs that are available both commercially and freely. One of the most widely used commercial software for solving nonlinear structural problems is ABAQUS [12], which can satisfy most of the academic and industrial demands. The other software namely MASTAN2 [13] is a noncommercial software, in which the warping functions has been included in the formulation and thus making it attractive for the increasing use in research.

2.6. DESIGN METHODOLOGIES FOR COLD-FORMED PORTAL FRAMES

2.6.1. Conventional design methods

The current design method for cold-formed frame systems is member-based or component-based, in which a frame system is treated as a set of individual components, i.e, beams, columns, connections, and so on, and they must comply with the design safety checks. The system effects are incorporated into the design procedures only implicitly through the use of effective length factors. Hence, the design process of a frame system entails two steps: structural analysis followed by design safety checks. In the first step, the second-order analysis or the first-order analysis followed by moment amplification to take into account second-order effects is carried out to determine the internal actions such as axial forces, shear forces, and bending moments in the frame. Beam elements are usually used to model the frame. In the second step, design safety checks are completed to verify that each member and connection have adequate capacities based on the current Standards/Specifications. The process is repeated until all requirements (the strength and serviceability) are fulfilled.

The current Standards/Specifications for the design of aluminium and other cold-formed metal structures include several design methods: Effective Width Method, Effective Thickness Method, Weighted Average Method, Direct Strength Method, and Continuous Strength Method. The basic concepts of these methods are described in the following subsections.

2.6.1.1. Effective width method

The Effective Width Method (EWM), which is the very first method used for designing thin-walled members in their post-buckling phase, has been developed on the basis of the research from von Karman [39] on the prediction of the plate strength in compression. In this method, the most severely buckled portions of an element are assumed to be ineffective in resisting load and the applied compression is carried by other portions that are situated adjacent to the supported edges. For this purpose, the non-uniform compressive stress distribution in the element (both the stiffened and unstiffened elements) is replaced by a uniform stress distribution, which is applied to the effective portions of the buckled plate element. Once the
effective width of all plates forming a cross-section is determined, the cross-sectional properties are computed using the effective width of the section.

The EWM has been incorporated in the standards/specifications for the design of cold-formed steel members, such as the North American Specification [191] and the Australia/New Zealand Standard [146]. Although the EWM is world-wide applied for formal use in design, it has some limitations as follows: (1) The method ignores the inter-element equilibrium and compatibility in determining the elastic buckling behaviour, (2) incorporation of competing buckling modes, such as distortional buckling can be awkward, (3) cumbersome iterations are required to determine even basic member strength, and (4) determining the effective section becomes increasingly more complicated as attempts to optimise the sections are made [192].

2.6.1.2. Effective thickness method

Another approach in the design of aluminium sections is the Effective Thickness Method (ETM), which is used to account for the effect of local and distortional buckling in slender members. In these members, the true section is replaced by an effective section, which is obtained by using a local buckling coefficient to factor down the thickness of any slender element that is wholly or partly in compression [98, 193]. It has already been underlined that the reduced thickness concept has no physical meaning, and if unwisely used can lead to confusion and subsequent error. However, for better or for worse this approach has been used in the current European Code for structural aluminium [194].

2.6.1.3. Weighted average method

The Weighted average method (WAM) proposed by Jombock and Clark [195] is another method to predict the ultimate strength of aluminium plates. After a plate is buckled, the in-plane stress distribution becomes non-uniform, with high values of stress at the edges with respect to the interior of the plate. Due to this redistribution, the plate is able to provide an ultimate strength generally higher than the buckling stress. The average stress at maximum load is often referred to as the “crippling stress” [196]. The weighted average stress of all elements is calculated to obtain the ultimate strength for the section as a whole. This method has been adopted in the AS/NZS 1664.1:1997 [197], the American Aluminium Design Manual (AA2015) [198] to account for the effect of the local buckling.

2.6.1.4. Direct strength method

The Direct Strength Method (DSM) [199] is the design approach primarily developed for cold-formed steel members and has been formally adopted in the North American Specification AISI S100-16 [191] and the Australia/New Zealand standard AS/NZS 4600:2018 [146] as an alternative to the conventional effective width method. The method uses elastic buckling solutions for the entire member cross-section to provide the direct strength rather than for elements in isolation [200].
The DSM allows direct determination of the capacity of thin-walled members of complex section shape, which does not require to calculate cumbersome effective sections, especially with the sections including intermediate stiffeners. Apart from this, with the use of numerical solutions for buckling analysis, the method can account for the compatibility and equilibrium between plate elements, which are not considered in the EWM. The elastic buckling analysis is often performed using computer software such as CUFSM [32], THINWALL [29], or THINWALL-2 [33, 34] to obtain elastic buckling stresses. Accurate member elastic stability is the fundamental idea behind the Direct Strength Method [192]. With the buckling loads obtained by buckling analysis, the nominal strength of the member under different buckling modes can be determined. The nominal strength of the member is the minimum of the nominal strength due to local buckling, distortional buckling and global buckling [146, 191].

Although the DSM has several advantages over the EWM, as discussed above, it also has limitations which have been reported in the literature [26, 199]. The DSM predicts the member strength based on the member's elastic buckling loads. If a member section contains very slender elements, it would result in a very low local buckling load. In this case, the strength predicted by the DSM would be over-conservative. However, members with very lasher elements are inefficient and prone to serviceability issues.

The success of the DSM in the design of cold-formed steel members has motivated the extensive development of the DSM for both extruded and cold-rolled aluminium members. Majority of proposed DSM equations of aluminium extrusion have been developed for aluminium 6000 series alloy with different cross-sectional shapes. Zhu and Young proposed the DSM equations for square, rectangular, and circular hollow columns [201, 202] and square hollow beams [203], whilst Chang et al. [204] proposed the DSM equations for irregular-shaped aluminium columns subjected to distortional buckling. Recently, the DSM for cold-rolled aluminium alloy 5052-H36 columns and beams subjected to different buckling modes has been intensively developed at the University of Sydney [25, 205] and the DSM shear design rules for cold-rolled aluminium lipped channel beams has been recently proposed by Rouholanmin et al. [206].

2.6.1.5. Continuous strength method

It is well-known that aluminium alloys and stainless steel exhibit the rounded stress-strain curved with the significant hardening, whilst the current Standards/Specifications for aluminium alloys and stainless steel have been based on an idealised plastic behaviour of the material properties. Consequently, the predicted strengths based on the design codes are lower than the actual strengths. To obtain new economic design rules for stainless steel members, the Continuous Strength Method (CSM) has been developed by Gardner [207] and Afshan and Gardner [208]. The method employs more precise material modelling by providing consistency with the observed material stress-strain response and allowing for strain hardening. Apart from the accurate material modelling, another key feature of the CSM is to replace the concept of
cross-section classification with a continuous non-dimensional numerical measure of the deformation capacity of the cross-section. In this method, there is no need to calculate the effective section properties and the effect of local buckling is incorporated by empirically reducing the peak load when deriving the section slenderness versus deformation capacity curve. At the global system level, the CSM allows for a global plastic design with reference to the rotational demands of the plastic hinges. The CSM approach has been incorporated in the AISC design guideline 27 for structural stainless steel [209] and it has also been developed for aluminium 6000 series alloy columns and beams with different shaped-cross-sections by Su et al. [210, 211].

2.6.2. Direct design method using advanced analysis

Although the traditional two-stage design process has been world-wide used in the routine design of the framing system, it involves a number of limitations. In this design method, a structural system is treated as a set of independent components and the interaction between the structural system and its members is only reflected indirectly by the use of effective factors. However, this approach does not provide an accurate indication of the factor against failure as it does not consider the interaction of strength and stability between the member and the structural system in a direct manner. In addition, in the member-based design approach, the elastic analysis is used to determine forces and moments on each member of a structural system while the capacities of each member treated as an isolated member are based on inelastic analysis and checked member-by-member following a lengthy process. There is no verification of the compatibility between the isolated member and the member as part of a frame. Therefore, there is no explicit guarantee that all members will sustain their design loads under the geometric configuration imposed by the framework [212]. Furthermore, enormous equation and conditions needing to be considered during the design process make the traditional two-step design process less attractive.

The limitations of the member-based design approach and the current advance in computer technology motivate the need to develop a practical system-based design method that accounts for compatibility between the separate members and the structure as a whole by using “Advanced Analysis”. This design approach is marked as the direct analysis and design method. In the Advanced Analysis, both the geometric and material nonlinear effects are taken into account in the analysis and thus Advanced Analysis is most capable of predicting the true behaviour of real structures [213]. One of the main advantages of the direct design method using advanced analysis is that separate member capacity checks encompassed by the specification equations are not required since the stability of separate members, and the stability of the structure as a whole, can be treated rigorously for the determination of the maximum strength of the structures.

The Direct Design Method (DDM) is now included in the provisions for steel structures, such as in the Australian steel standard AS4100 [214], the American steel specification
AISC360-16 [191], and the Australian/New Zealand standard for CFS structures AS/NZS 4600 [146], thus providing further incentive for using the DDM for CRA portal frames.

2.6.2.1. Modelling

The fundamental idea of the DDM is to capture the realistic behaviour of the structures by using the advanced analysis. To achieve actual behaviour of a cold-formed portal frame system, the numerical model of the structural system must include all nonlinearities and address the influence of finite joint size, joint flexibility, local instabilities, load height within the cross-section, and the effects of secondary structural elements on the frames [114]. Also, in the portal frame system composed of numerous components, individual components may come into contact with others during deformations, thus demanding full knowledge in contact mechanics, meshing techniques, numerical analysis and convergence algorithms. Extensive research studies over the decades have indicated that full three-dimensional modelling using shell elements is one of the promising options in capturing the strength and behaviour of a cold-formed portal frame system. Guidance on how to create finite element models for thin-walled structures has been provided in the literature [215, 216].

Indeed, in recent years, several studies to determine appropriate modelling techniques for cold-formed portal frame systems using shell element models that consider the effects of semi-rigid joints were conducted by Lim and Nethercot [217], Zhang et al. [159], Cardoso [218], Hannah and Rasmussen [134] with double C-sections connected back-to-back, and Rinchen and Rasmussen with single C sections [160].

2.6.2.2. System reliability analysis

Structural reliability analysis can be considered as a rational evaluation criterion, which provides a good basis for decisions in the design of a structure. Various sources of uncertainty are inherent in structural design, such as the variabilities associated with material properties and geometric imperfections or unpredictability of loads. As a consequence, the parameters of the loading and structural resistances are random variables, and thus absolute safety cannot be achieved. Therefore, structures must be designed to perform their function with an acceptable low probability of “failure” [219]. The term “failure” may have different meanings, so it must be clearly defined to perform a structural reliability analysis. For this purpose, a limit state function (i.e. performance function) representing the boundary between desired and undesired performance of a structure is used to define failure in the context of structural reliability analyses.

A traditional notion of the safety margin is associated with the ultimate limit states [219], where $R$ represents the resistance and $Q$ represents the load effect of a structure, a limit state function can be defined for this mode of failure as:
If $R$ and $Q$ are continuous random variables and have probability density functions (PDFs), the quantity $(R - Q)$ is also a random variable (Figure 2.42). The structure fails when the load exceeds the resistance, so the probability of failure, $P_f$, is equal to the probability that the function $g(R,Q) < 0$ will occur. Hence, the probability of failure can be expressed in terms of the limit state function as given in Equation (2.32) and represented by the shaded area in Figure 2.42.

$$P_f = P(R - Q < 0) = P(g(R - Q) < 0) = \int F_R(q_i) f_Q(q_i) dq_i$$  \hspace{1cm} (2.32)

where $F_R$ represents the cumulative distribution function of the variable $R$, $f_Q$ represents the probability density function of the variable $Q$, and $q_i$ is a specific value of $Q$.

![Figure 2.42. Probability functions of load, resistance, and safety margin [219]](image)

To determine the probability of failure, Equation (2.32) needs to be solved. In general, it is difficult to evaluate the integral, and thus the integration requires special numerical techniques, which may have inadequate accuracy. Hence, it is common in practice that the concept of a reliability index ($\beta$) is utilised to quantify structural reliability, firstly introduced by Hasofer and Lind [220].

For the convenience in analysis, random variables of resistance and load are converted to their standard forms as given in Equations (2.33) and (2.34), respectively, in which, $Z_R$ and $Z_Q$ are called reduced variables.

$$Z_R = \frac{R - \mu_R}{\sigma_R}$$  \hspace{1cm} (2.33)

$$Z_Q = \frac{Q - \mu_Q}{\sigma_Q}$$  \hspace{1cm} (2.34)

The limit state function (Equation (2.31)) can then be expressed in terms of the reduced variables, as follows:
The boundary separates the safe and failure domain in the space of reduced variables is given by the condition $g(Z_R, Z_Q) = 0$, which is a straight line as presented in Figure 2.43. According to Hasofer and Lind [220], the reliability index ($\beta$) is the shortest distance from the origin of reduced variables to the line $g(Z_R, Z_Q) = 0$, as illustrated in Figure 2.43.

Figure 2.43. Reliability index defined as the shortest distance in the space of reduced variables [219]

By using the geometry from Figure 2.43, the reliability index ($\beta$) can be determined as:

$$\beta = \frac{\mu_R - \mu_Q}{\sqrt{\sigma_R^2 + \sigma_Q^2}}$$

(2.36)

where $\beta$ is the inverse of the coefficient of variation of the limit state function when R and Q are uncorrelated. For normally distributed random variables R and Q, the reliability index is related to the probability of failure by:

$$\beta = -\Phi^{-1}(P_f) \quad \text{or} \quad P_f = \Phi(-\beta)$$

(2.37)

Since $\beta$ depends only on the first two moments, the mean and variance, it is called a “second-moment” measure of structural safety. If all random variables are normally distributed and uncorrelated, $\beta$ and $P_f$ are exactly related by Equation (2.37), otherwise, Equation (2.37) provides only an approximate means of relating $\beta$ to a probability of failure.

The reliability index ($\beta$) is a relative measure of the safety of a structure. When $\beta$ is higher, the probability of failure is smaller, i.e., the structure is safer or the “reliability” is increased. It has generally been preferred to use $\beta$ instead of the probability of failure in developing the new Load and Resistance Factor Design (LRFD) specifications [221].

In reliability-based design, the Advanced Analysis is performed in combination with a probabilistic assessment of structural variables to determine the strength of the structural system. On the basis of the current LRFD philosophy, in the Direct Design Method using
Advanced Analysis, a global system resistance factor can be applied to the nominal frame system strength so that the familiar LRFD format for the whole frame system as follows [10]:

$$\phi_s R_n \geq \sum \gamma_i Q_{ni}$$  \hspace{1cm} (2.38)

where \( \sum \gamma_i Q_{ni} \) represent the total load effect on the structure, \( R_n \) is the nominal system strength predicted by Advanced Analysis, and \( \phi_s \) is the system resistance factor determined by reliability assessment.

The system resistance factor \( (\phi_s) \) and the reliability index \( (\beta) \) are interrelated, so the practice is to fix one of them and derive the other. A more general approach to determine the relations between the resistance factor and reliability index involves establishing the limit state functions and determining the unknowns following the established procedures based on the first-order second-moment (FOSM) reliability analysis method. The system reliability analysis performed to establish relations between the reliability index \( (\beta) \) and the system resistance factor \( (\phi_s) \) for the CRA portal frames is described in Section 5.6.

2.7. CURRENT DESIGN STANDARDS/SPECIFICATIONS FOR ALUMINIUM STRUCTURES

There are currently a number of global aluminium alloy structural design standards/specifications, which provide design guidelines for a range of extruded aluminium structural components and applications. For cold-rolled aluminium structures, no proper design guidelines are available apart from several provisions for cold-formed structural sheeting provided in European code-BS EN 1999-1-4:2007 [222]. However, the examination of the typical specifications, including the Australian/New Zealand standard – AS/NZS 1664.1:1997 [197], the American Specification – Aluminium design manual AA2015 [198], and the European Code – BS EN 1999: 2007 [194] should not be underestimated.


For a specific portal frame structure, the whole system is treated as individual members and the design checks are performed for each member or critical members. The columns, which are usually the critical members of the frame, is subjected to the combined axial compression
and bending. Hence, as per Clause 4.1.1 of AS/NZS 1664.1:1997 [197], the combined axial compression and bending must satisfy the following interaction equations:

\[
\frac{f_a}{F_a} + \frac{C_{mx}f_{bs}}{F_{bx}(1 - f_a/F_{ex})} + \frac{C_{my}f_{by}}{F_{by}(1 - f_a/F_{ey})} \leq 1
\]  

(2.39)

and

\[
\frac{f_a}{F_{ax}} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1
\]  

(2.40)

where

- \(x\): subscript for major principal axis bending
- \(y\): subscript for minor principal axis bending
- \(f_a\): average compressive stress on cross-section produced by the factored compressive load
- \(f_b\): maximum compressive bending stress on cross-section produced by the factored transverse loads and/or end moments
- \(F_a\): factored limit state stress for member considered as axially loaded column
- \(F_{ax}\): factored limit state stress for member considered as a short column
- \(F_b\): factored limit state stress for member considered as a beam
- \(C_m = 0.6 - 0.4(M_1/M_2)\) for members whose ends are prevented from swaying; or = 0.85 for members whose ends are not prevented from swaying
- \(M_1/M_2\): ratio of end moments where \(M_2\) is the larger of the two end moments and \(M_1/M_2\) is positive when the member is bent in reverse curvature, negative when bent in single curvature
- \(F_e\): factored elastic buckling stress

When \(f/a/F_a \leq 0.15\), the following equation shall be permitted to be used in lieu of (2.39) & (2.40)

\[
\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1
\]  

(2.41)

More details about the calculations of terms such as limit state stresses and design checks for other types of members can be found in relevant Clauses in AS/NZS 1664.1:1997 [197]

2.7.2. American Specification – Aluminium design manual AA2015

In 1967, the Aluminium Association published the first edition of the “Specification for Aluminium Structures” with the purpose to provide designers guidelines to calculate the strength of aluminium components in structural applications. After decades of development,
the specification has been updated based on researches conducted at universities and producer’s laboratories, most notably at the Alcoa Research Laboratories and its successor, the Alcoa Technical Center in Alcoa Center. The latest version of the specification currently is the 11th one released in 2020 [224], making it the most modern global design provision for aluminium structures. However, in this study, the relevant calculations are based on the 10th edition (2015) [198], which is not much different from the latest edition.

Similar to AS/NZS 1664.1:1997, the AA2015 [198] treats the frame system as individual beams, columns, connections, and other components. For a critical column subjected to flexural and axial force, the member must satisfy the interaction equation check as follows:

$\frac{P_P + \frac{M_{cx}}{P} + \frac{M_{cy}}{P}}{P} \leq 1$ \hspace{1cm} (2.42)

where

$x$: subscript for major principal axis bending

$y$: subscript for minor principal axis bending

$P_r$: required axial strength

$P_c$: available axial strength

$M_r$: required flexural strength

$M_c$: available flexural strength

The details of determining member strengths and the design check for other types of components can be found in AA2015 [198].


The European Code BS EN 1999:2007 [194] has been prepared by the Technical Committee CEN/TC250 – Structural Eurocodes, which provides guidelines for the design of buildings, civil engineering, and structural works in aluminium. The standard is intended to be used in conjunction with other relevant Eurocodes such as BS EN 1990 – Basic of structural design [225] and BS EN 1991 – Actions on structures [226] and includes five parts, of which the first part (EN 1999-1-1) [152] provides basic design rules for structures made of wrought aluminium alloys and limited guidance for cast alloys while the other four parts give design rules for specific aluminium components or limit states. For example, the design requirements for cold-formed trapezoidal aluminium sheeting are included in BS EN 1999-1-4:2007 [222].

Although BS EN 1999-1-4:2007 [222] does not apply to CRA C- or Z profiles, some of its recommendations are still used in conjunction with BS EN 1999-1-1:2007 [152] to give the interaction equation checks for CRA members in combined compression and bending. The details of these are described in Section 5.3.2.3.
2.8. CONCLUSIONS

The literature review in this chapter provides insights into the current state of knowledge on aluminium alloys considered as structural materials and portal frame systems comprising cold-formed sections. Particularly, the manufacturing methods of aluminium alloy structural members, material mechanical properties, various instabilities that are common in thin-walled sections, components and connections are reviewed. Portal frames composed of cold-formed sections, imperfections, types and structural analysis and design, and the future trends in design are also highlighted.

Reviewing the literature reveals that aluminium alloys, especially the 5000 group alloys, possess several advanced characteristics, which make them attractive to be used as structural materials, providing new solutions to compete with steel. Furthermore, the application of the roll-forming process in manufacture aluminium alloy structural members helps enhance the structural strength as well as contributes to sustainable development, and promoting more intensive use of CRA profiles in structural applications.

Portal frames are one of the most common structural applications in Australia where large open spaces are required. Especially, the use of CFS portal frames has dramatically increased in demand over the last decade and substantial research studies have been devoted to providing guidelines for construction and design of CFS portal frames. The past studies on the CFS portal frames and associated connections have been mostly performed on the frames consisting of double symmetric sections (mainly are back-to-back channel sections), whilst research on CFS single section portal frames is relatively less. It was also indicated that bolted connections are preferable to form the joints of portal frames, which are generally semi-rigid and the stiffness of the connections highly depends on the geometry of the relevant brackets, member thickness, and the bolt configuration.

The behaviour of thin-walled members is characterised by instabilities due to the slenderness of the section, which is highly sensitive to imperfections. Also, the strength of a structure may be greatly reduced by imperfections. Hence, they need to be incorporated in the analysis and design.

The literature review also provides an overview of the prevailing design methods, including the Effective Width Method, Effective Thickness Methods, and Direct Strength Method, which are based on the safety checks of the individual members and connections. Although serving as the contemporary mean for the design of aluminium and steel structures, these two steps design methods contain obvious shortcomings. As a result, the Direct Design Method (DDM) using Advanced Analysis can be seen as an attractive alternative design approach that has the potential to radically change the design paradigm. The DDM using Advanced Analysis is a novel design method where the nominal ultimate strength of the whole system can be determined directly while the inelasticity of material and nonlinearity of
geometric are included. This is carried out in combination with a reliability analysis accounting for the structural variables to determine the design strength of the structure using a whole system resistance factor.
CHAPTER 3. EXPERIMENTAL INVESTIGATION OF FULL-SCALE COLD-ROLLED ALUMINIUM PORTAL FRAMES

3.1. INTRODUCTION

This chapter presents an intensive experimental program carried out in the J.W. Roderick Laboratory for Materials and Structures at the University of Sydney on full-scale portal frames composed of cold-rolled aluminium double-channel sections and their components.

Since there does not appear to be any consistent investigation of the portal frame system made of cold-rolled aluminium profiles, it is essential to perform a comprehensive experimental program on this structural application to achieve a number of objectives as follows: (1) to understand the structural behaviour of the cold-rolled aluminium back-to-back channel section portal frames; (2) to quantify the ultimate strength and the performance of various portal frame configurations comprised of cold-rolled aluminium sections; (3) to validate the structural behaviour simulated by the numerical finite element models; and (4) to enhance the knowledge base for aluminium structures in general and aluminium portal frame in particular. In this context, seven full-scale tests on cold-rolled aluminium (CRA) back-to-back channel portal frames subjected to gravity load (five tests) and the combined horizontal and gravity loads (two tests) were carried out. The system consisted of three portal frames connected in parallel with purlins spanning across to create a free-standing structure with a size of 14 m x 7.32 m (length x width) and 6.7 m (height). Various configurations, as detailed in the following sections, were examined.

Apart from the full-scale tests, component tests on coupons, point fastener connections, and column base connections were conducted to obtain the necessary information on material properties and connection characteristics. Further, imperfection measurements were also thoroughly performed for the incorporation of the geometric imperfections into the finite element models in further investigations.

The first part of this Chapter demonstrates the details of the full-scale tests including experimental preparation, testing procedures, and experimental results followed by the mechanical property tests on coupons with detailed preparation, testing, results, and discussions for each series of test are provided. The description of experiments on point fastener connections used in the full-scale tests is presented in the next part. Lastly, tests performed on various configurations of the base connection and data processing are described.

3.2. FULL-SCALE COLD-ROLLED ALUMINIUM PORTAL FRAME TESTS

3.2.1. Design of test frames

Portal frames have been widely used to provide large open spaces for industrial, farming, and residential purposes. Traditionally, portal frames are composed of hot-rolled steel sections, or can be constructed from cold-formed steel members as an alternative in recent years. This
end-user application has not been previously attempted by any producers of aluminium profiles. Hence, a proposed design is required to develop a portal frame system with the primary columns and beams made from aluminium cold-rolled double C-sections before the experimental program is performed. The design was come up with by the reference to the commercially available designs of cold-form steel portal frames in practice. The design was also accommodated to the limitations of spacing and available resource in the Structures Laboratory of the School of Civil Engineering and motivated partly by the excellent results from the previous cold-formed steel portal frames tests [101, 102]. Drawings of the proposed frame system, including the frame layout, components, brackets, and so on, are detailed in Appendix A.1.

3.2.2. Geometry and general details of test frames

The experimental configuration consists of a single-span CRA portal frame system including three frames connected in parallel with a series of purlins spanning between the rafters (and girts between the columns in the modified configurations) to create a free-standing two-bay structure. The frame configuration for Frame Test 1 is shown in Figure 3.1, which can be considered as the typical test configuration throughout the experimental program. The general layout of the test frames and the relative positions of components are shown in Figure 3.2.

Figure 3.1. Full-scale cold-rolled aluminium double channels portal frames (Frame Test 1)
The front view of the typical experimental frame is shown in Figure 3.3. Each frame had a centerline span of 14m and a total height of approximately 6.7m from the ground to the centre of the apex joint. The distances from the base plate to the centre of eaves joints and from the base plate to the intersection of columns with knee bracings were 5.4m and 4.46m, respectively. The frames had a 10° pitch, whilst an angle of 50° between the columns and the knee braces
was designed. As the elastic modulus \( E \) of aluminium is about a third compared to that of steel, the rafter tie was utilised to connect rafters in order to minimise the deflection of the test frames. The length of the rafter tie was 5.6 m horizontal direction as shown in Figure 3.3.

Figure 3.4 shows the side elevation of the frames. The stability of the whole system was improved by using a cross-bracing system of tension-steel rods, which diagonally braced the top and the bottom of the columns in both sides of one bay. This bracing system was also used to prevent the whole test frames from an unexpected catastrophic collapse during the test.

During the experiments, the testing load was applied only to the central frame through a load spreading system as fully described in Section 3.2.7, whilst two end frames served as supports to the far end of the purlins, which provided lateral restraints to the central frame. The purlin spacings in the apex and knee regions and in the intermediate regions were 730 mm and 1750 mm, respectively. Girts were also utilised in the tests 4, 5, and 7 to provide lateral restraints for the columns of the test frames with the girt spacing of approximately 1030 mm as shown in Figure 3.5.
Figure 3.5. Full-scale frame test with girts (Frame Test 4)

The sections fabricated from cold-rolled 5052 - H36 aluminium alloy were sourced from the proprietary company BlueScope PERMALITE, Australia throughout the experimental program. The columns were built up from 2xAC25025 channel sections with a length of 5232 mm, whilst the rafters were composed of the same sections with a greater length of 7182 mm. The channel section for purlin was AC15025 with 3469 mm length, whereas those for knee braces and rafter ties were 2xAC15025 channel sections of 1210 mm length and 2xAC20025 channel sections of 3840 mm length, respectively.

As standardised in the AS/NZS1664.1:1997 [197] for structural aluminium alloy 5052 - H36, the yield stress, the ultimate tensile strength, and the elastic modulus can be taken as 179 MPa, 255 MPa, and 69.3 GPa, respectively. However, as the cold-forming process causes changes in the mechanical properties [227], the measured actual material properties were conducted and are fully described in Section 3.3.

3.2.3. Frame test series

Seven full-scale systems consisting of various configurations under different load conditions were performed on the CRA double-channel portal frames to gain great insights into
the behaviour of frames and their components, to identify the main parameters which affect the ultimate system strength, and to examine the possibility of optimising the cold-rolled aluminium portal frame system. The summaries of these tests are presented in Table 3.1.

Table 3.1. The experimental program of cold-rolled aluminium portal frames

<table>
<thead>
<tr>
<th>Test series</th>
<th>Loading</th>
<th>Distinct configuration</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frame Test 1</td>
<td>Gravity Load</td>
<td>Original frame system</td>
</tr>
<tr>
<td>Frame Test 2</td>
<td>Gravity Load</td>
<td>Original frame system</td>
</tr>
<tr>
<td>Frame Test 3</td>
<td>Gravity Load + Wind Load</td>
<td>Original frame system</td>
</tr>
<tr>
<td>Frame Test 4</td>
<td>Gravity Load</td>
<td>Original frame system and additional girts between columns</td>
</tr>
<tr>
<td>Frame Test 5</td>
<td>Gravity Load + Wind Load</td>
<td>Original frame system and additional girts between columns</td>
</tr>
<tr>
<td>Frame Test 6</td>
<td>Gravity Load</td>
<td>Frame system with modified apex region</td>
</tr>
<tr>
<td>Frame Test 7</td>
<td>Gravity Load</td>
<td>Original frame system and additional girts and sleeve stiffeners</td>
</tr>
</tbody>
</table>

The first three tests on the portal frames based on the proposed design (Section 3.2.1) identified as “Original frame system” in Table 3.1, in which the columns of the frames were unbraced laterally (out-of-plane) (Figure 3.1). Frame test 1 was carried out as a pilot experiment to evaluate the feasibility of the test set-up and the loading effect. Accordingly, the load was applied vertically to produce the effect of Gravity Load. Frame Test 2 was a repetition of Frame Test 1 to confirm the accuracy of the test results of the original frame system under Gravity Load. In Frame Test 3, the effect of Wind Load was investigated by applying an additional horizontal loading to the north eaves of the experimental frame system as detailed in Section 3.2.7.

The experimental set-up and test loading configuration in Tests 2 and 3 are similar to those in Tests 4 and 5, respectively, except that the lateral or torsional movements (i.e. out-of-plane displacements) of the columns were restrained by using an additional series of girts serving as lateral braces along the webs of the column channels as shown in Figure 3.5.

For examining the possibility of simplifying the apex connection, a modified design of the apex connection was come up with and adopted in Frame Test 6 as shown in Figure 3.6. Unlike the set-up for the apex region in the other tests, the rafter tie for bracing the rafters in the apex region was removed, and two L-shaped brackets were attached to the connection at the ridge to strengthen the apex joint as seen in Figure 3.7.
In the final experimental frame (Frame Test 7), due to the possibility that the occurrence of the maximum bending moment can be found in the column segment at the knee connection, additional sleeve stiffeners were attached to the upper column portion at the knee region to enhance the ultimate strength of the portal frame system which might be governed by the failure of the columns. The system of girts was still retained to improve the stability of the columns during the test. With this attempt, the strength of the frame system can be maximised.
3.2.4. The sequence of installation of the test frames

A consistent method for installation of the test frame systems needs to be adopted because the frame erection has been proved as one of the most crucial activities in the process of testing cold-rolled aluminium portal frames. As the behaviour and the ultimate strength of the portal frames were proved to be significantly affected by the system imperfections, a well-prepared plan and intensive care of the installation process was performed in order to achieve the uniformity and to minimise the initiation of global imperfections on the frames. The outline of this process is informed as follows:

Step 1: All column and rafter members were composed of two single lipped C-sections AC25025 bolted back-to-back through the webs with 12 mm aluminium spacers in between.

Step 2: The measurements of sectional and member imperfections of the columns and rafters were carried out prior to the assembly of a full test frame. The details of the imperfection measurements are fully described in Section 3.2.6.2.

Step 3: The base connections were fixed to the steel I-beam embedded in the concrete floor (i.e. the strong floor) by aligning with the previously marked centreline on the surface of the steel beam. The L-shaped base brackets were connected to the top base plates using M24 high-strength steel bolts and using the base cleat washers (Part No: BW25-SS/ Dwg No: S14 in Appendix A.1). The details of the base connection are demonstrated in Section 3.2.5.3.

Step 4: Back-to-back columns were connected with the base connections using M16 stainless steel bolts. Before one end of the columns were positioned between the L-shaped base brackets, eaves brackets were attached to the other end with bolts in snug-tight condition. Long spirit levels were used to ensure the correct vertical alignment of the columns in two orthogonal directions during positioning. M16 stainless steel bolts were inserted and snug-tightened after the columns were securely placed to the desired positions.

Step 5: For setting up the upper part of the frames, the rafter members, the apex brackets, and the purlin cleats were assembled at the floor level. The rafter tie for bracing the rafters was also prearranged at this stage. After the desired pre-tension in the bolted connections were induced by the turn-of-nut method, the whole assemblage of the upper portions of the frames was gradually lifted with intensive care by the overhead crane to minimise the possibility of introducing any serious secondary imperfections during craning, especially unexpected twisting in the rafters. Once the assemblage was in place, the ends of the rafters were fit in eaves brackets previously attached to the top ends of the columns. The bolted connections were also used to connect the rafters and columns through the eaves brackets. The knee braces for bracing the knee regions of the frame were connected in the last step of the single-frame building process.

Step 6: The two end frames were supported by lateral bracings using designed brackets, which were connected to supporting frames to keep these frames standing for the next progress (see Figure 3.8).
Step 7: The central frame after being assembled was aligned to the position by adjusting the overhead crane. While the central frame was still held by the crane, the purlins at the knees and the apex regions were installed to provide stability to the central frame and maintain the alignment.

Step 8: After removing the crane from supporting the central frame, the remaining purlins were installed and bolted to purlin brackets on the rafters using M12 stainless steel bolts/nuts/washers. The method of bolting can be seen in Section 3.2.5.5. In this step, girts were also installed in the Frame Tests with the braced columns.

Step 9: Cross-bracings were arranged to provide the lateral brace for the whole system.

Step 10: All transducers were attached at the designated positions by using transducer frames and supporting brackets that were previously fabricated to measure all displacements during the tests.

Step 11: Installation of the load spreading system followed by connecting to the hydraulic jacks.

![Figure 3.8. Lateral bracings for two end frames: (a) End frame A; (b) End frame C](image)

### 3.2.5. Frame connections

As connections play an extremely crucial role in the whole system, they need to be designed at a high level of care. In the testing program, bolted joints were utilised as they have been proved to be adequate to resist moments in previous studies [90, 121]. The primary members including the columns and the rafters were connected together at the eaves and the apex and were braced by rafter tie between two rafters and knee braces at the knee regions using brackets specifically designed for the proposed portal frames. The frames were also fixed into the strong floor by the base joints. While the base brackets were made of 2205 stainless steel plates of 8 mm thickness, the other connection brackets were formed from 5083 – H321 aluminium plates with a nominal thickness of 6mm. Fasteners used in these connections were
the 316 stainless steel bolts with the nominal diameter of 16mm, and the bolt hole sizes of 18mm. Details of the connections are described in the following sub-sections.

3.2.5.1. Apex connection

The apex joint was formed by bolting the ridge-ends of the rafters to the apex bracket as shown in Figure 3.9. The bracket included two single aluminium lipped L-shaped brackets with a thickness of 6 mm as shown in Figure 3.9c. The double apex brackets were placed in the middle between the two single rafter channels, and ten M16 stainless steel bolts were used to connect them together through the webs. The detailed configuration and dimensions of the apex joint are provided in relevant drawings in Appendix A.1.

![Figure 3.9. Apex connection: (a) Original configuration; (b) Modified configuration (Frame Test 6); (c) Apex bracket in back-to-back form](image)

To reduce deflection as well as to enhance the strength for rafters, a rafter tie system was introduced and provided for all Frame Tests at the apex region, except for Frame Test 6. The system included a rafter tie that composed of cold-rolled aluminium double lipped channel AC20025 sections connected to the rafters through the rafter tie brackets as shown in Figure 3.10. The configuration and dimensions of the rafter tie and the rafter tie bracket are detailed in the respective drawings provided in Appendix A.1. For Frame Test 6, as introduced in Section 3.2.3, the rafter tie for bracing the rafters was not provided, instead, two L-shaped brackets were attached to the apex joint as presented in Figure 3.9b.
Figure 3.10. Rafter tie connection

3.2.5.2. Knee connection

Figure 3.11 shows the knee connections including eaves joints and knee bracing connections known as haunched connections. The eaves joints were formed by bolting the top ends of the columns and the eaves-ends of the rafters to the knee brackets (Figure 3.12a) using seven M16 stainless steel bolts. The knee brace consisted of two single AC15025 channel sections bolted back-to-back were connected to the columns using knee brace-to-column brackets (KBC) (Figure 3.12b) and to the rafters using knee brace-to-rafter brackets (KBR) (Figure 3.12c). The KBC and KBR brackets were extended to cover the full web depths of the column and rafter channels, respectively. The detailed dimensions including the fastener locations of the brackets and the knee brace can be found in the relevant drawings in Appendix A.1.
Figure 3.11. The knee connection

Figure 3.12. Aluminium brackets used in the knee connection: (a) Eaves brackets; (b) Knee brace-to-column brackets (KBC); (c) Knee brace-to-rafter brackets (KBR)
3.2.5.3. **Base connection**

The base connection was formed by bolting the flanges of the back-to-back columns to the L-shaped base brackets of 8 mm thickness using four M16 stainless steel bolts each as detailed in Figure 3.13. For the central frames, as shown in Figure 3.13a, four M24 8.8/TF bolts were used to connect the bases of the L-shaped cleats to an upper base plate of 25 mm thickness (also referred to as “the base plate” in the following Sections), which was supported on four cylindrical load cells. The load cells were arranged on the lower base plate firmly clamped to the universal beam on the strong floor and had been calibrated to measure reactions using the strain measurement during the tests (Appendix A.2). For the end frames, the load cells were not provided, but instead, spacer plates were welded to the upper and lower base plates to form a thick base plate, as shown in Figure 3.13b, so that the end frames had the same height as the central test frames. The detailed dimensions of the base connections and load cell arrangement can be found in the relevant drawings in Appendix A.1.

![Figure 3.13. Base connection: (a) For central frames; (b) For end frames](image)

3.2.5.4. **Purlin connection**

Twenty-four channel purlins from AC15025 channel section with a length of 3469 mm were used to connect the test frames to end supporting frames as shown in Figure 3.1 and Figure 3.2. Two ends of each purlin were fastened to available purlin cleats previously attached to the webs of the rafter members by using two M12 stainless steel bolts each. The leg of the purlin cleats in contact with the rafter webs was rigidly bolted to prevent any slips when the vertical testing loads were applied. The arrangement of the purlin connection is illustrated in Figure 3.14.
3.2.5.5. The method of bolting for the frame connections

As demonstrated in previous sections, bolted connections were used for the frame connections. In such connections, the method of bolt tightening affects the rigidity of the joints. Therefore, to induce the desired level of the pretension in the bolts to achieve the required clamping force, the turn-of-nut method was employed to tighten all the M16 bolts. The details of the turn-of-nut method are provided in Section 2.4.2.1.

Two M12 bolts were used in each connection of the purlin to respective purlin cleat. Because the applied loads were required to transfer only to the test frame, the method used to connect purlins therefore plays an important role in achieving this desired requirement. Particularly, at the cleats, all purlins were designed to displace without being rotationally restrained which otherwise would transfer reactions to the end frames. In order to satisfy this
requirement, only one bolt was snug-tightened to minimise shear in the purlins, thereby ensuring that the applied loads were only transferred to the test frame. At the central frame, the top bolt was snug-tightened, whilst the bottom bolt was hand-tightened. In contrast, at the end frames, the bottom bolt was snug-tightened, whereas the top bolt was hand-tightened. The finger-tightened bolts (loose bolts) in the purlin connection are depicted in Figure 3.14. Figure 3.14c shows the expected deformation of the purlin during the downward deflection of the test frame.

3.2.6. Measured frame and section geometry

The cold-rolled aluminium members are not perfectly straight and usually associated with geometric imperfections introduced during the process of manufacturing and erecting. Since the initial geometric imperfections influence the stability response of the frames and then affect the load-carrying capacity of the whole frame system, they must be considered when performing any analysis of the frame systems. Therefore, geometric imperfections of all columns and rafters were measured using a laser scanner method as reported in Pham et al. [168] to ascertain the sectional imperfections and the overall geometry of the member before being assembled into frames. The thickness of the channel sections, the aluminium brackets (apex, eaves, KBC, KBR, rafter tie brackets), and stainless steel base brackets were also carefully measured. Furthermore, the global imperfections (i.e. frame imperfections) of each frame were subsequently measured before the load spreading system was installed. The details of the imperfection measurements are described in the following sections.

3.2.6.1. Measured sectional dimensions

The actual sectional dimensions of the column and rafter members, including the thickness, the section depth, the flange width and the lip length were measured before conducting the frame testing. The details of the measurements for all frame tests are provided in Appendix A.3 and the mean values of the measurements are given in Table 3.2. Further, the thickness of other components including the knee brace, the rafter tie and all other brackets were also validated against the nominal thickness.

<table>
<thead>
<tr>
<th>Structural components</th>
<th>Designation</th>
<th>t (mm)</th>
<th>D (mm)</th>
<th>B (mm)</th>
<th>L (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Columns and Rafters</td>
<td>AC25025</td>
<td>2.44</td>
<td>257.10</td>
<td>77.75</td>
<td>23.30</td>
</tr>
<tr>
<td>Knee Braces</td>
<td>AC15025</td>
<td>2.44</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Rafter Ties</td>
<td>AC20025</td>
<td>2.44</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Purlins</td>
<td>AC15025</td>
<td>2.44</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Aluminium Brackets</td>
<td></td>
<td>5.82</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
3.2.6.2. *Sectional and member imperfections*

The sectional and member geometric imperfections may have significant effects on the strength and behaviour of the thin-walled structures. It is, therefore, crucial to accurately quantify the initial sectional and member imperfections prior to assembling and testing. In order to estimate the deviation of the primary members from their perfect (i.e. nominal) dimensions, geometric imperfections were measured on all columns and rafters used for the seven frame tests.

3.2.6.2.1. *Imperfection measuring rig and recording methods*

Several methods were proposed to measure sectional and member geometric imperfections in the past studies, such as the methods using a dial gauge [48, 228], a telescope [228], transducers [229], laser scanners [168, 230-236], or the photogrammetry method [237]. In this study, the laser scanner method was adopted to measure the sectional and member imperfections due to its popularity and availability in the laboratory.

A test-rig for measuring sectional and member imperfections was custom-built in the Materials and Structures Laboratory at Sydney University as shown in Figure 3.15. The rig includes a supporting frame, laser devices and data-recorder. The supporting frame consists of two high precision bars mounted on a rigid frame, serving as a rail system, and a trolley travelling along the rail system with a desired speed controlled by a motor. The laser scanners were attached to the trolley to measure the distances from the scanners to the surfaces of specimens. The imperfections were measured at 22 points around the cross-sections as shown in Figure 3.16. The measured points were offset 10 mm from the edges. The speed was set constantly at 10mm/second and a data sampling of 10 samples/second was used, resulting in reading at 1mm intervals.

![Imperfection measuring rig set-up](image)

*Figure 3.15. The imperfection measuring rig set-up: (a) Measuring test-rig; (b) Typical measurements*
3.2.6.2.2. Sketching cross-sections at both ends

The actual imperfections at an arbitrary section could be obtained by adding the relative imperfections to the true imperfections at the ends. While the relative imperfections were obtained by normalising the readings at two ends of the specimen, the true imperfections at the ends were determined by superimposing the nominal (i.e. perfect) cross-section profile onto the actual (i.e. imperfect) cross-section profile. For such purpose, the cross-section profiles at the two ends of each column or rafter specimen were sketched on papers using a fine-tip pencil. The two end sections are labelled as A and B, in which, the laser scanners run from Section A to section B. Typical sketch profiles of the end sections for the south column of Frame Test 1 are shown in Figure 3.17. The differences between the nominal profile and the sketched profiles are the true imperfections at the end cross-sections. The typical true imperfections at the two ends for the south column of Frame Test 1 are shown in Figure 3.18, where the nominal cross-section (the red line) is superimposed onto the imperfect cross-section (the blue line). The imperfection magnitudes corresponding to the 22 measuring points at the two ends, A and B, are designated as $a_i$ and $b_i$, respectively. The positive values in the directions presented in Figure 3.18 indicate by arrows in Figure 3.16. The complete set of imperfections measured for all frame tests is provided in Appendix A.3.
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Figure 3.17. Sketch profiles of the end sections for the south column of Frame Test 1: (a) End section A; (b) End section B

Figure 3.18. Imperfections at the two ends for the south column of Frame Test 1: (a) End section A; (b) End section B
3.2.6.2.3. Data processing method

The raw data of an imperfection line contains a number “N” of discrete readings evenly distributed over the length “L” of the specimen. In order to introduce this data into numerical finite element models, each data line was decomposed into a Fourier series as given in Equations (3.1) and (3.2).

\[ f(z) = \sum_{n=1}^{N} K_n \sin \left( \frac{n\pi z}{L} \right) \quad \text{where} \quad 0 < z < L \]  

(3.1)

\[ K_n = \frac{2}{L} \int_{0}^{L} f(z) \sin \left( \frac{n\pi z}{L} \right) dz \quad (n = 1, 2, 3, \ldots, \infty) \]  

(3.2)

By discretisation, Equation (3.2) can be expressed as Equation (3.3) using the N discrete data points of an imperfection line.

\[ K_n = \frac{2}{N-1} \sum_{i=0}^{N} f(z) \sin \left( \frac{n\pi z}{N-1} \right) \quad (n = 1, 2, 3, \ldots, \infty) \]  

(3.3)

A finite number of terms in Equation (3.3) can be retained to encompass short half wavelengths which are more influential to local buckling behaviour [238, 239]. For this purpose, 35 terms with coefficients of \( K_1 \sim K_{35} \) were chosen for all imperfection lines of all specimens. A typical Fourier representation of the relative imperfection taken from line 1 of the south rater of Frame Test 1 (SR-1) is illustrated in Figure 3.19 along with the corresponding reading data. As seen in Figure 3.19, the imperfections expressed by Fourier series agree well with the measured data, and thus the Fourier expression can be used for numerical finite element models.

![Figure 3.19. Fourier representation and measured imperfection for line 1 of the SR-1](image)

The non-perfect cross-section at the two ends was transformed into linear equations as follows:

\[ g(z) = a_i + \frac{(b_i - a_i)}{L} z \quad \text{where} \quad 0 < z < L \]  

(3.4)

where
\( L \) is the length of the specimen

\( z \) is the coordinate along the length

\( a_i \) is the imperfection of line \( i \) \((i = 1, 2, \ldots, 22)\) at the end section A

\( b_i \) is the imperfection of line \( i \) \((i = 1, 2, \ldots, 22)\) at the end section B

The true imperfections of the measured line \( i \) along the length of a specimen can be expressed by the combinations of the Fourier series \( f(z) \) and the linear equation \( g(z) \), as given in Equation (3.5)

\[
F(z) = \sum_{n=1}^{\infty} K_n \sin \left( \frac{n\pi z}{L} \right) + a_i + \frac{b_i - a_i}{L} z \quad \text{where} \quad 0 < z < L
\] (3.5)

The Fourier representations for each measured line for all members and the corresponding imperfections at the ends are provided in Appendix A.3.

### 3.2.6.2.4, Statistic of representative parameters

The measured imperfection data can be introduced directly by using the interpolation algorithm as described in Section 4.6.5. However, for the parametric studies where there is no available imperfection data, the assumed imperfections need to be adopted. The sectional and member imperfections are assumed on the basis of the representative parameters, which can be defined by the statistical analysis of the measured imperfections.

The measured imperfections on each member can be categorised in the sectional and member imperfections as discussed in Section 2.4.4.1. The member imperfections include the bow \((G_1)\), camber \((G_2)\) and twist \((G_3)\), whereas the sectional imperfections consist of the local imperfection \((d_1)\) and distortional imperfection \((d_2)\).

The member imperfections \((G_1, G_2, G_3)\) and the sectional imperfections \((d_1, d_2)\) are different along the length of the member. Based on the measured data, the maximum magnitudes of these imperfections can be obtained. The imperfection components are given in Appendix A.3 and are summarised in Table 3.3.

<table>
<thead>
<tr>
<th>Components</th>
<th>( G_1/L )</th>
<th>( G_2/L )</th>
<th>( G_3 )</th>
<th>( d_1 )</th>
<th>( d_2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Degree/m</td>
<td>mm</td>
<td>mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Column AC25025</td>
<td>1/5810</td>
<td>1/6677</td>
<td>0.097</td>
<td>0.620</td>
<td>1.734</td>
</tr>
<tr>
<td>Rafter AC25025</td>
<td>1/6041</td>
<td>1/4375</td>
<td>0.134</td>
<td>0.730</td>
<td>2.192</td>
</tr>
</tbody>
</table>
3.2.6.3. Frame imperfections

During the frame installation, the columns and rafters are also prone to global imperfection as a result of the lack-of-fit assembly. Since the frame imperfections have a detrimental effect on the strength and behaviour of the system, the measurement of global imperfections needs to be carried out to quantify the magnitudes of the frame imperfections prior to conducting the tests. For this purpose, the measurements including the out-of-plumb imperfections for both south and north columns ($\delta_1, \delta_2$), the vertical deflection of the apex, and the diagonal distance from the eaves to the base at the far end ($d$) of the column, as shown in Figure 3.20, were implemented after the completion of the frame erections but before the set-up of the load spreading system.

Figure 3.20. Global imperfection measurements

The results of the measurements for all seven frame tests are provided in Table 3.4.

Table 3.4. Global imperfections of the test frames

<table>
<thead>
<tr>
<th>Test Series</th>
<th>Out-of-plumbness of the south column, $\delta_1$</th>
<th>Out-of-plumbness of the north column, $\delta_2$</th>
<th>Height of the underside of apex bracket, h</th>
<th>Diagonal distance from base plate to eaves brackets, d</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frame Test 1</td>
<td>8.50</td>
<td>7.00</td>
<td>6450</td>
<td>14880</td>
</tr>
<tr>
<td>Frame Test 2</td>
<td>23.50</td>
<td>12.50</td>
<td>6370</td>
<td>14850</td>
</tr>
<tr>
<td>Frame Test 3</td>
<td>-7.25</td>
<td>28.25</td>
<td>6400</td>
<td>14900</td>
</tr>
<tr>
<td>Frame Test 4</td>
<td>22.50</td>
<td>7.25</td>
<td>6390</td>
<td>14910</td>
</tr>
<tr>
<td>Frame Test 5</td>
<td>3.75</td>
<td>-7.50</td>
<td>6486</td>
<td>14900</td>
</tr>
<tr>
<td>Frame Test 6</td>
<td>1.63</td>
<td>13.00</td>
<td>6444</td>
<td>14860</td>
</tr>
<tr>
<td>Frame Test 7</td>
<td>10.50</td>
<td>-14.75</td>
<td>6513</td>
<td>14850</td>
</tr>
</tbody>
</table>

Note: Dimensions in mm; positive value of $\delta_1$ and $\delta_2$ indicate outward global imperfections.
3.2.6.4. Other geometric imperfections

In addition to the aforementioned sectional, member and frame imperfections, there are other types of geometric imperfections. The dents, known as a sort of localised geometric imperfections, in the members can be taken as an example. The magnitudes of such imperfections cannot be quantified because of the aberrant nature of the impacts that occur during transportation, storage, and installation. For another example, unexpected slight misalignments were observed at the joints due to the difference between dimensional values of nominally designed components and the actual experimental components. The misfits in the joints were eliminated by stretching the components manually to allow them to fit each other. In such cases, additional geometric imperfections were likely to be introduced. However, it was unable to quantify these imperfections when the frame assembly/installation was completed. Therefore, the quantification of such geometric imperfections can be ignored in this study.

3.2.7. Loading method

In the experimental program, two loading conditions were investigated including a pure downward loading, representing the effects of gravity loads, and a combination of downward and horizontal loadings, simulating the combined gravity and wind loads. For such purposes, a load spreading system was specially designed to transfer vertical loading as shown in Figure 3.21. The loading system consisted of a series of hollow steel beams (HSB) arranged in three levels and steel straps with different lengths acting as vertical hangers. These steel bars and loading HSB were pin-connected together using threaded rods Φ16. The top straps were bolted to the loading brackets, which were attached to the loaded-ends of the purlins. These purlins were then bolted to the purlin-to-rafter L-shaped cleats (i.e. purlin cleats) on either side of the loaded central frame. The details of this connection are shown in Figure 3.22.

The vertical load was applied using a vertical hydraulic actuator, which was connected to the centre of the lowest loading beam as shown in Figure 3.23. The verticality of the applied loads was maintained during testing using a special trolley to allow the vertical jack to move horizontally in the plane of the frame. This trolley was also specially designed to prevent the jack from moving in the vertical and out-of-plane directions. The in-plane horizontal movement of the vertical loading actuator was controlled automatically by engaging an additional horizontal hydraulic jack that was connected to the base of the vertical jack (see Figure 3.24). The information of the horizontal movement of the test frame was collected by a potentiometer installed at the apex as shown in Figure 3.25.
Figure 3.21. Vertical load spreading system

Figure 3.22. Details of load spreading system
Figure 3.23. Load spreading system on the frame

Figure 3.24. The actuator system
In Frame Tests 3 and 5, the horizontal load of 7.5 kN representing wind loads was applied at the north eaves. The arrangement for the horizontal load set-up is shown in Figure 3.26. As can be seen in Figure 3.26, the horizontal load was simulated by hanging a concrete block of 758 kg with a weighing scale (9 kg) at the end of a steel cable. The other end of the steel cable passed over a pulley system and connected to a custom steel bracket (the black bracket as seen in Figure 3.26) rigidly bolted to the north eaves brackets. The use of the steel bracket not only made it easier to connect the steel cable but also stiffened the eaves brackets. The horizontal load was applied just after the installation of the vertical load system but before connecting the vertical load system to the vertical hydraulic actuator, and was kept constant during the test.
3.2.8. Instrumentation

A number of linear variable differential transducers with 25, 50, 100, and 200 mm stroke lengths were installed at various locations on the test frame to monitor the deformations of the frame through the duration of the tests. The locations of the transducers on the central frame, which were categorized into seven groups, is shown in Figure 3.27. In addition, two transducers having a stroke length of 100 mm were also attached at the apex of each end frame to measure its vertical and out-of-plane displacements during the tests. The details of the transducers corresponding to their respective functions are summarised in Table 3.5. The typical configuration of the transducers at the base, column, eaves, rafter, apex, end frame and knee brace are detailed in Figure 3.28 to Figure 3.34.
At the base of each column, four transducers were bolted to a custom-made bracket, which was clamped directly to the strong floor using zig clamps, to measure the base rotation in the major axis (in-plane rotation). This rotation is necessary for determining the semi-rigidity of the column base connections. The locations of these transducers are provided in Figure 3.28a, whilst Figure 3.28b shows a typical arrangement of the transducers at the base of the south column. In addition to the use of transducers, an inclinometer was also installed on each column at 180 mm above the base plate to measure the rotation of the base connections (see Figure 3.28b). The reading of the inclinometer was not only used to validate the reading of the transducers but also provided the rotation of the column base in the minor axis.

Figure 3.27. Location of transducers on the test frame

Figure 3.28. Transducers at the base of the south column: (a) Locations of the transducers; (b) Typical transducer setup at the base of the south column
<table>
<thead>
<tr>
<th>No</th>
<th>Location</th>
<th>No. of Transducers</th>
<th>Measurement Type</th>
<th>Functions</th>
<th>Stroke length (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Base</td>
<td>4</td>
<td>Global</td>
<td>Measures base rotation</td>
<td>50</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>Local</td>
<td>Monitors bolt slip</td>
<td>25</td>
</tr>
<tr>
<td>2</td>
<td>Centre of Columns</td>
<td>4</td>
<td>Global</td>
<td>Measures horizontal deflections and twist rotation</td>
<td>100</td>
</tr>
<tr>
<td>3</td>
<td>125 mm below the bottom of the Knee connection</td>
<td>4</td>
<td>Global</td>
<td>Measures horizontal deflections and twist rotation</td>
<td>100 / 200</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6</td>
<td>Local</td>
<td>Measures local deflection of flange and web and distortional displacement of flanges of the column</td>
<td>50</td>
</tr>
<tr>
<td>4</td>
<td>Eaves bracket</td>
<td>3</td>
<td>Global</td>
<td>Measures horizontal deflections in two orthogonal directions</td>
<td>100 / 200</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>Local</td>
<td>Monitors bolt slip</td>
<td>25</td>
</tr>
<tr>
<td>5</td>
<td>Apex</td>
<td>3</td>
<td>Global</td>
<td>Measures frame deflection in three orthogonal directions</td>
<td>200/500</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>Local</td>
<td>Monitors bolt slip</td>
<td>25</td>
</tr>
<tr>
<td>6</td>
<td>Rafter</td>
<td>3</td>
<td>Global</td>
<td>Measures horizontal deflections and twist rotation</td>
<td>200</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6</td>
<td>Local</td>
<td>Measures local deflection of flange and web and distortional displacement of flanges of the column</td>
<td>50</td>
</tr>
<tr>
<td>7</td>
<td>Knee Brace</td>
<td>3</td>
<td>Global</td>
<td>Measures out-of-plane horizontal deflection</td>
<td>100</td>
</tr>
<tr>
<td>8</td>
<td>Apex of each end frame</td>
<td>2</td>
<td>Global</td>
<td>Monitors out-of-plane horizontal deflection and in-plane vertical deflection</td>
<td>100</td>
</tr>
</tbody>
</table>

Before setting the instrumentation, six transducer frames were designed and fabricated from aluminium angle and square hollow sections with different features. On each column in the test frame (central frame), two transducer frames were used to attach transducers. While the first transducer frame was located at 2.15 m above the base plate of the column to accommodate four transducers for measuring the global displacements of the column (horizontal, lateral displacement and twist rotation) (Figure 3.29), the other transducer frame was attached to the
column at 125 mm below the bottom of the knee connection (Figure 3.30) to attach six local transducers for measuring the local and distortional displacements of the column section and accommodating other four global transducers for recording global deformation of the column about its major and minor axes and twist. The last two transducer frames were placed on the rafters adjacent to the rafter tie connections as shown in Figure 3.31a. Six transducers were connected directly to the transducer frame to measure the local deformation of the rafter section. In addition, four other transducers were attached to a stable steel pole (Figure 3.31b) and were connected to two corners of the transducer frame through four steel strings and a pulley system. The global deformations of the rafter including vertical and lateral displacements along with twisting were determined on the basis of the data recorded by those four transducers.

Figure 3.29. Transducers at 2.15 m above the base plate of the south column: (a) Locations of the transducers; (b) Typical transducer setup at 2.15 m above the base plate of the south column

Figure 3.30. Transducers at 125 mm below the knee connection of the south column: (a) Locations of the transducers; (b) Typical transducer setup
Figure 3.31. Transducers at the north rafter: (a) Local transducers; (b) Potentiometer used to measure global displacements of the north rafter

Figure 3.32. Transducers at the south eaves: (a) Locations of the transducers; (b) Typical setup of the transducers

To support the instrumentation at the apex region, an inverted pole was hung from the overhead crane which remained in a fixed position during the tests (Figure 3.33a). At the apex, all displacements in three directions were monitored during the tests. While a wire-type transducer with the measurement range of 500 mm was used to measure the vertical displacement, the out-of-plane displacement was recorded by a 200 mm transducer. These transducers were supported by brackets bolted to the inverted support as shown in Figure 3.33b. In addition, a potentiometer was also installed to measure the horizontal displacement of the apex. The reading from the potentiometer also was used to control the horizontal hydraulic actuator as discussed in Section 3.2.7.
Five additional transducers were used to monitor the relative displacements of the rafter/column sections and relevant brackets at the apex, eaves, and the base connections as shown in Figure 3.35. At the apex, a transducer was attached directly to a web of the south or north rafter using strong adhesive tape while its stroke (needle tip) bore on a bracket glued to the apex bracket. Similar setups were adopted at the junction of the rafter webs and eaves brackets for both the south and the north eaves connections and at the junction of the outside flanges of columns and base brackets. The readings of these transducers allow the bolt slips at the respective bolted connections to be evaluated.
Figure 3.35. Transducers measuring relative displacement of connected components: (a) At the apex connection; (b) At the south eaves; (c) At the north eaves; (d) At the south base; (e) At the north base
In Frame Test 7, sleeve stiffeners were placed in the columns to improve the performance of the frame system. Strain gauges were placed on the sleeve stiffeners and columns at the knee brace-column connection (also known as the knee connection), 300 mm above the knee connection, and 300 mm and 500 mm below the knee connection, as shown in Figure 3.36a. At each location, eight strain gauges were attached on the flanges of the column and stiffeners approximately 20 mm away from the web-flange junction (on the outer surface of the column and the inner surface of the stiffeners). The strain gauge setup for the north column is shown in Figure 3.36b. The longitudinal strains obtained from the strain gauge readings allow the relative moment carried by the stiffeners as compared with the columns to be determined so that the contribution of the sleeve stiffeners can be quantified.

![Figure 3.36. Strain gauge arrangement for Frame Test 7: (a) Strain gauge locations; (b) Strain gauges in the north column](image)

Further, four calibrated load cells were installed beneath the base plate of each column in the test frame to measure the column base reactions. The arrangement and calibration of these load cells can be found in Section 3.2.5.3 and Section A.2.

### 3.2.9. Testing of frames

After the installation of the load spreading system, the vertical hydraulic jack was connected to the lowest beams of the vertical loading system. Before conducting the tests, all instrumentation was carefully checked to ensure that they worked properly and the recording
data system was set up to log data at a rate of one data point per second during the tests. The experiments were conducted by applying an incremental vertical load using a 250 kN capacity hydraulic actuator with a stroke length of 260 mm. The loading jack was operated using the displacement control mode with an incremental movement rate of 1 mm/minute. The slow loading rate chosen is crucial to ensure the loading was predominantly static, and the incremental displacement rate was maintained in the duration of testing.

One of the key objectives in the full-scale aluminium portal frame tests is to obtain the full pre- and post-ultimate load-displacement responses. However, prior to reaching the ultimate loads, the members of the frames are likely to undergo large deformations, which may be out of the measurement ranges of the transducers. In order to capture the full-displacements of the members until the failure of the whole frame systems, the positions of the transducers needed to be adjusted wherever required during the tests by momentarily pausing the tests.

3.2.10. Results of full-scale portal frame tests

3.2.10.1. The general behaviour of the frames observed during the tests

During the tests, the behaviours of tested frames were carefully observed whilst recording the data. The experiments in this research study were grouped into four categories: (1) unbraced columns subjected to gravity only (Frame Tests 1, 2, and 6), (2) unbraced columns with a combination of gravity and wind loads (Frame Test 3), (3) braced columns with applied gravity load only (Frame Tests 4 and 7), and (4) braced columns with applied wind and gravity loads (Frame Test 5). Frames were tested with various configurations.

For the frames subjected to vertical load only, as the applied load was gradually increased, the frames started to deform along with the vertical deflection of the apex, the rafters pushed the eaves away from each other, and both the columns also bent outwards. In the frames with unbraced columns (Frame Tests 1, 2, and 6), the columns also twisted gradually in opposite directions as the load increased with major deformation being observed near the column-knee connections. Generally during these tests, the appearance of local buckling at the web of the column sections was not observed, but rather the flexural-torsional instability was shown clearly. When the testing load reached approximately 5 kN, the data of the out-of-plane displacement and twisting deformation of columns began to be recognised by instrumentation devices, although these deformations were not visible. The magnitude of deformation in the frames progressively increased with the increasing applied load. The typical deformation of a frame subjected to the downward load alone and the global deformation of an unbraced column are shown in Figure 3.37. The ultimate strengths of the frames were reached when the formation of a plastic mechanism occurred in the compression lips (distortional mode) or in the compression flanges (local mode) of the columns. This phenomenon was triggered by the flexural-torsional buckling of the column members. Such failures were often recognised by a sudden loud and violent shaking of the whole frame.
In the Frame Tests loaded by vertical load only with braced column (Frame Tests 4 and 7), the flexural-torsional buckling deformations of the columns was relatively small attributed to the partial lateral restraints provided by the girts spanning between the columns of the central frame and the two end frames. The in-plane deformation was developed along with the increase of the vertical applied load. Eventually, localised deformations occurred in the columns at the knee connection or/and the rafters at the rafter tie connection caused by large displacements at these regions. While the failure of the Frame Test 4 occurred at the north column and rafter, the failure of the frame test 7 occurred at the south column and rafter. In Frame Test 7, the appearance of distortional buckling half-waves was observed in the rafters over the rafter tie connections prior to reaching of the ultimate load (see Figure 3.38). In Frame Test 4, distortional buckling of the north rafter occurred after the frame attained the first peak-load (i.e. the ultimate load).

In Frame Tests 3 and 5, the central frame was subjected to both horizontal and vertical loads simulating combined gravity and wind loading. Before attaching the vertical hydraulic jack to the load spreading system, the 7.5 kN concrete block representing the horizontal load, was slowly lowered by a manual forklift. Due to the fact that the horizontal load was applied in the north side of the frames, the whole central frame swayed northwards, as can be seen in Figure 3.39. When the deformation of the frame was stable under the effect of wind load, the vertical load was applied with the same procedure in the frame tests subjected to vertical load only. The apex deflected vertically when the vertical testing loads were applied, whilst both the south and north eaves continued to move northwards with greater displacement tracked in the north eaves. Meanwhile, both the columns deformed outwards, with the south column bent southwards and the north column bent northwards. It was observed clearly at the frame failure that the south column overall deflected northwards compared to its undeformed state. For the cases of unbraced columns, the test frame taking account of a combination of horizontal and vertical loads (Frame Test 3) and the test frames with applied vertical load alone (Frame Tests 1, 2, and 6) experienced similar flexural-torsional failure modes at the columns. For the cases of braced columns, the frame failed under a combination of simulated gravity and wind loads (Frame Test 5) and those with the effect of gravity load only (Frame Tests 4) had the similar failure modes mainly caused by the localised deformation of the column at the knee brace connection.

More details about the behaviour of the frame tests are presented in the following sections through a series of load-deformation curves and detailed discussions. To facilitate the load-displacement graphs, a sign convention as presented in Figure 3.40 was adopted for the frame deformation. It is noted that the arrow direction indicates a positive value.
Figure 3.37. Deformation of Frame Test 1: (a) Typical deformation of frame; (b) Flexural-torsional buckling of the south column

Figure 3.38. Distortional buckling of the rafters in frame test 7
Figure 3.39. Deformation of Frame Test 3 with the application of the horizontal load: (a) Typical deformation of the central frame; (b) The south column; (c) The north column

Figure 3.40. Sign convention adopted for frame deformation
3.2.10.2. The ultimate load capacity and failure modes of frames

The ultimate vertical loads of all frame tests along with the corresponding vertical displacements of the apex and in-plane horizontal displacements of the eaves at collapse are given in Table 3.6. It is noted that the values of the ultimate load capacities in Table 3.6 include the weight of the load spreading system and self-weights of the rafters.

Table 3.6. Ultimate load capacity and corresponding deformation of frames

<table>
<thead>
<tr>
<th>Test series</th>
<th>$P_u$ (kN)</th>
<th>$\Delta_{Ay}$ (mm)</th>
<th>$\Delta_{Eas}$ (mm)</th>
<th>$\Delta_{Eas}$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frame Test 1</td>
<td>33.25</td>
<td>88.33</td>
<td>-9.69</td>
<td>13.32</td>
</tr>
<tr>
<td>Frame Test 2</td>
<td>32.16</td>
<td>89.87</td>
<td>-10.49</td>
<td>8.89</td>
</tr>
<tr>
<td>Frame Test 3</td>
<td>18.84</td>
<td>76.93</td>
<td>33.20</td>
<td>113.70</td>
</tr>
<tr>
<td>Frame Test 4</td>
<td>39.20</td>
<td>113.01</td>
<td>-11.34</td>
<td>16.65</td>
</tr>
<tr>
<td>Frame Test 5</td>
<td>27.59 (29.17*)</td>
<td>89.51</td>
<td>75.34</td>
<td>96.72</td>
</tr>
<tr>
<td>Frame Test 6</td>
<td>27.42</td>
<td>137.99</td>
<td>-21.23</td>
<td>19.17</td>
</tr>
<tr>
<td>Frame Test 7</td>
<td>60.69</td>
<td>165.70</td>
<td>-8.67</td>
<td>30.93</td>
</tr>
</tbody>
</table>

Note: $P_u$ is the ultimate load ((*) is the second peak-load). $\Delta_{Ay}$ is the vertical deflection of the apex, and $\Delta_{Eas}$, $\Delta_{Eas}$ are the horizontal displacements of the south eaves and north eaves, respectively.

As per the aforementioned facts, all Frame Tests 1, 2, 3, and 6 with unbraced columns failed at lateral-torsional failure modes, although the obtained ultimate loads ($P_u$) in these frame tests are different. Such failure modes were initiated by lateral movements of the knee braces, which subsequently triggered the columns to displace out-of-plane and twist while being deflected in-plane due to the bending moments. Since the columns were subjected to the combination of axial load and bending moment, the failure mode of the column was observed as the flexural-torsional buckling, which subsequently caused a formation of the plastic mechanism on the compression lips or flanges of the columns right after buckling. In contrast, the lateral-torsional buckling was minimal in Frame Tests 4, 5, and 7 with braced columns. These frames failed due to the formation of a spatial plastic mechanism in the flanges and webs of the columns at the knee connection and the rafters at the rafter tie connections. In the testing process, the vertical loadings were continued to be applied even after obtaining the ultimate vertical loads so that the post-peak behaviour could be observed and captured until the destruction of the test frames except that Frame Test 3 was ended after reaching the ultimate vertical load. The reason for such termination in Frame Test 3 was the insufficient remaining
movable range of the concrete block used to induce the horizontal load. A summary of the collapse in each frame test is given in Table 3.7.

Table 3.7. Failure modes of test frames

<table>
<thead>
<tr>
<th>Test series</th>
<th>Failure modes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frame Test 1</td>
<td>The columns underwent the lateral-torsional buckling that initiated the overall collapse of frames. The frame progressively deformed with an increasingly incremental vertical loading up to the first peak-load when the distortional and local buckling in the compression flanges and lips in the south column (see Figure 3.41a) occurred, following by a sudden drop in the carrying load. The vertical loading was kept being applied to investigate the post-ultimate response of the frame and to obtain the second peak-load at different location, which was slightly lower than the first ultimate strength. At the second failure, the similar distortional and local buckling in the compression flanges and the compression lips were clearly observed at the same height of the opposite column (see Figure 3.41b)</td>
</tr>
<tr>
<td>Frame Test 2</td>
<td>Failure was initiated by the lateral-torsional buckling of the columns. When the ultimate load (first peak-load) was reached, a local buckle of the compression flanges of the north column occurred due to the large twisting of the column (Figure 3.42b). The frame continued to sustain load (of lower magnitude) in the new equilibrium position. The vertical hydraulic jack continued to pull down until a local buckle of the compression flanges of the south column occurred (Figure 3.42a), at that moment the second peak-load attained.</td>
</tr>
<tr>
<td>Frame Test 3</td>
<td>The failure was initiated by the lateral-torsional buckling in the north column. With the further increase in the vertical testing load, the north column experienced more twisting, triggering the local and distortional buckling observed, as shown in Figure 3.43.</td>
</tr>
<tr>
<td>Frame Test 4</td>
<td>The frame failed due to a formation of the spatial plastic mechanism in the compression flanges and compression web portion of the north column at the knee connection when the applied load reached the ultimate load (see Figure 3.44a). The distortional buckling occurred in the north rafter flange at the rafter tie connection region, after the first peak-load. The post-peak failure of the testing frame is attributed to a spatial plastic collapse in the</td>
</tr>
<tr>
<td>Test series</td>
<td>Failure modes</td>
</tr>
<tr>
<td>-------------</td>
<td>---------------</td>
</tr>
<tr>
<td>Frame Test 5</td>
<td>The failure was a result of the localised deformation of the north column at the knee connection captured at the first peak-load as shown in Figure 3.45a, and the distortional buckling of the north rafter at the rafter tie connection (Figure 3.45b), followed by a formation of the spatial mechanism in the top flanges and webs of the north rafter adjacent to the rafter tie connection (Figure 3.45c).</td>
</tr>
<tr>
<td>Frame Test 6</td>
<td>Failure was initiated by the flexural-torsional buckling of the columns. The frame progressively deformed with increasing vertical load until a distortional buckle of the compression flanges and local buckle of the lips in the north column (the first peak-load was attained) occurred, as shown in Figure 3.46b. The frame continued to sustain vertical loads at much lower magnitude until a distortional buckle of the compression flanges and local buckle of the lips in the south column (Figure 3.46a) occurred, at this moment the second peak-load was reached and then the load dropped significantly.</td>
</tr>
<tr>
<td>Frame Test 7</td>
<td>The failure was initiated by the distortional buckling of the rafters which resulted in a change in the stiffness of the frame. The frame ultimately failed by a formation of the spatial plastic mechanism in the flanges and webs of the south column adjacent to the bottom end of sleeve stiffeners and also in the south rafter at the rafter tie connection, as shown in Figure 3.47a and Figure 3.47b, respectively. The second collapse due to the localised deformation of the north rafter at the knee brace-to-rafter connection (see Figure 3.47c) was observed once the second peak-load was reached.</td>
</tr>
</tbody>
</table>
Figure 3.41. Failures of Frame Test 1: (a) south column; (b) north column

Figure 3.42. Failures of Frame Test 2: (a) south column; (b) north column
Figure 3.43. The failure mode of the north column (Frame test 3): (a) View 1; (b) View 2

Figure 3.44. Failure modes of Frame Test 4: (a) North column (first peak load); (b) North rafter (second peak load)
Figure 3.45. Failure modes of Frame Test 5: (a) Failure of the north column (first peak load); (b) Distortional buckling of the north rafter; (c) failure of the north rafter (second peak load)

Figure 3.46. Failure modes of Frame Test 6: (a) South column; (b) North column
As shown in Table 3.6, the ultimate vertical load of Frame Test 1 ($P_u = 33.25$ kN) and Frame Test 2 ($P_u = 32.16$ kN) were closely identical with approximately 3.3% error. Additionally, the observed deflections and the corresponding failure modes were very similar. This fact proved the accuracy of the test rig and the test design. It is worth noting that the post-peak behaviours of Frame Tests 1 and 2 were also nearly identical. After completing the tests, the frames were unloaded and the vertical hydraulic jack was disconnected from the load spreading system, followed by the removal of the instrumentations and the disassembling of the frames. The columns reverted to their pre-buckling shapes with the remaining little distortional and local buckle in the failed columns. This fact shows that the columns failed in the elastic buckling. Additionally, there were no damages or buckles as observed in the rafters.

For Frame Test 3, due to the presence of an additional horizontal load applied on the north eaves brackets, the north column was subjected to a greater load compared to the south column and the whole frame swayed northwards (Figure 3.39). Despite having the similar behaviour as observed in Frame Tests 1 and 2, Frame Test 3 ($P_u = 18.84$ kN) showed an approximately 42% decrease in ultimate vertical load compared to the first peak-loads of Frame Test 1 ($P_u = 33.25$ kN) and Frame Test 2 ($P_u = 32.16$ kN).
Due to the effect of the horizontal load, Frame Test 5 ($P_u = 27.59$ kN) had a lower ultimate vertical load compared to Frame Test 4 ($P_u = 39.20$ kN), although both experiments had the same configuration. It is interesting to observe that the second peak load ($P_{u2} = 29.17$ kN) in Frame Test 5 was even higher than its first peak load ($P_u = 27.59$ kN), which is in contrast to other experiments. Additionally, the columns in Frame Tests 4 and 5 were provided with partial lateral restraints by the girts spanning between the test frame and the end frames. As a result, lateral-torsional buckling in the columns was minimised considerably. Therefore, the performance of these frames were improved remarkably compared to the frames with unbraced columns, commonly producing lower ultimate strengths due to the member buckling. The ultimate strength of Frame Test 4 was enhanced by approximately 20% compared to the average load capacity of Frame Tests 1 and 2 under vertical load only. Meanwhile, with the presence of girts along the column height, Frame Test 5 showed a 46% increase in the ultimate vertical load compared to the capacity of Frame Test 3 although both of the frame tests were additionally subjected to a constant horizontal load of 7.5 kN. This indicates the important role of the lateral bracing system for the columns in the performance of the frame system, especially when the frames are subjected to wind loads.

For Frame Test 6, the rafter tie used to brace the rafters and apex region was removed, whilst the apex connection was strengthened by two L-shaped brackets, as shown in Figure 3.7. The test results demonstrated that the frame without the rafter tie performed less efficiently than the original configuration. Specifically, by comparison to Frame Tests 1 and 2, Frame Test 6 presented an approximately 17.8% decrease in the ultimate capacity, whereas the observed deformation increased significantly. Therefore, it is recommended that the rafter tie is necessary to improve the performance of the cold-rolled aluminium portal frame systems.

For Frame Test 7, sleeve stiffeners with a length of 2.4 m were inserted into the internal open-sections of the columns at the knee connections, and the presence of girts improved the member stability of the columns. This is likely to maximise the ultimate strength of the portal frame system. It was obtained that the ultimate strength of Frame Test 7 ($P_u = 60.69$ kN) increases by approximately 85% compared to the average of Frame Tests 1 and 2 and by 54.5% compared to the capacity of Frame Test 4. Therefore, sleeve stiffeners contributed significantly to the enhancement of the ultimate strength of the frame system. The contribution of stiffeners is discussed in Section 3.2.10.8.

### 3.2.10.3. Apex deformation

The total vertical loads versus the apex vertical deflection curves for all Frame Tests are graphically reproduced in Figure 3.48. The total vertical loads include the load applied from the vertical jack, the weight of the load spreading system, and the self-weight of the frame at roof level - which included the weight of the two rafters, half of the purlins on two sides of the central frame, the apex bracket, the rafter tie system, the purlin cleats, the knee brace-to-rafter brackets, half of the knee braces, and bolts. While the weight of the load spreading system was
4.80 kN (Table A.12), the self-weight was determined as 2.43 kN (Table A.11), making them a total of approximately 7.23 kN added to the vertical load reading by the vertical hydraulic actuator. The deflection due to the weight of the load spreading system and the self-weight could be determined by assuming that the frames remained elastic during loading. The dashed line at the beginning of each plot represents the linear load-displacement relation resulting from the sum of the weight of the load spreading system and the self-weight at roof level.

It can be seen in Figure 3.48 that the load-displacement responses in the most Frame Tests were almost in the form of linear elasticity until the obtainment of the first peak-load in the experiments except for Frame Test 7 where the frame began to depict non-linearity at a vertical load of approximately 50 kN. The vertical rigidity of the frames with the same configuration was nominally identical which is evident by the almost identical slopes in the load-deflection curves of these frames, except Frame Test 3 where it exhibited a more flexible system. In Frame Test 6, due to the removal of the rafter tie used to strengthen the rafters and apex joint, the vertical stiffness of the frame system was remarkably reduced. In contrast, Frame Test 7 had a higher vertical stiffness due to the contribution of the sleeve stiffeners placed in the columns. Once the vertical load of Frame Test 7 increased over 50 kN, a non-linear response was experienced due to the occurrence of the distortional buckling in the rafters and the development of slippage deformation in the bolted connections when the ultimate friction was reached. The evidence for the development of bolt slips in Frame Test 7 is provided in Section 3.2.10.7.

![Figure 3.48. Applied load versus vertical displacement of the apex](image.png)
For the Frame Tests 3 and 5, a horizontal load of 7.5 kN was applied to the north eaves after the installation of the vertical load spreading system but before applying the vertical testing load. This resulted in a certain amount of the apex vertical displacement before the frame was loaded vertically. To account for this amount of displacement, initial offsets were produced in the graphs for Frame Test 3 and Frame Test 5 as observed in Figure 3.48. It can be noted that the horizontal load was introduced after the frames were subjected to the weight of the load spreading system and the self-weight. However, for the convenience of representation and comparison among Frame Tests, it was implicitly assumed that all vertical loads were applied to the frames after applying 7.5 kN horizontal load. The same technique is adopted for graphs in the following section.

In the majority of the Frame Tests, except for Frame Test 3, the loading was kept being applied after the first vertical peak-load to capture the post-peak behaviour of the frame systems. After a sudden drop at the first ultimate load, it can be observed that the capability of carrying loads was recovered as per a new equilibrium path in the second segment of the load-displacement response curves until the second peak-loads occurred. It is interesting to observe that, the second peak-load in Frame Test 5 is greater than the first peak-load.

In addition to the apex vertical displacement of the central frames, the apex in-plane horizontal and out-of-plane lateral displacements of the central frames and the apex out-of-plane displacement of the two end frames during the tests are also provided in Appendix A.5.

3.2.10.4. Eaves deformation

The eaves deformation was also monitored during testing. The vertical load – horizontal displacement relationship curves of the south eaves and north eaves are shown in Figure 3.49 and Figure 3.50, respectively.
As seen in Figure 3.49 and Figure 3.50, initial offsets observed in the graphs for Frame Test 3 and Frame Test 5 account for the horizontal displacement of the eaves due to the action of the horizontal load at the north eaves. As seen as a flexible system, the south and north eaves of Frame Test 3 displaced differently to the north direction under the effect of the horizontal load before applying the vertical testing load. Especially, the horizontal displacement of the south eaves was captured about 64 mm compared to that of the north eaves of 93 mm. However, in Frame Test 5, when only the horizontal was applied, the horizontal movements of the south
eaves and the north eaves almost had the same displacement values of 70.09 mm and 70.06 mm, respectively. This can be explained due to the presence of girts, which eliminated most of the out-of-plane deformation of the columns, resulting in a better performance of the system.

When the vertical testing load was applied to the frames, it is also observed that the relationship curves between the vertical load and the horizontal displacement of the eaves were almost linear. In the Frame Tests, due to the fact that when the vertical load increased gradually, the rafters deflected downwards and pushed the eaves outwards. In other words, the eaves moved horizontally away to each other. However, in Frame Tests 3 and 5, with the effect of the horizontal load, both the eaves kept moving in the same direction (northwards) when the incremental vertical testing load increased although the amount of displacement of the south eaves was much less than that of the north eaves. It is also noticed that the reversal of displacement can be observed in the graphs for all frames when the vertical load reached the peaks (the first and second peak loads). This fact may be possibly caused by the stress relief and redistribution. The reversal phenomenon could occur in one of the eaves or both eaves depending on each experiment.

Apart from the in-plane displacement, the lateral (out-of-plane) displacement of the eaves was also recorded. The lateral displacements of the south eaves and the north eaves are plotted in Figure 3.51 and Figure 3.52, respectively. Before applying the vertical loading, both the eaves of Frame Test 3 and Frame Test 5 displaced towards the east due to the presence of the horizontal load. After that, under the incremental increase in the vertical testing load until the obtainment of the ultimate load, both the eaves of Frame Test 5 and the south eaves of Frame Test 3 gradually moved westwards while the north eaves of Frame Test 3 continued moving to the east. However, the south eaves of Frame Test 5 reversed after reaching the first peak load.

Among the experiments, Frame Test 1 experienced the least lateral displacement, and the out-of-plane displacements of both the eaves were insignificant during the test. In contrast, a considerable amount of displacement was observed in Frame Test 7, in which both the south and the north eaves displaced towards the west by about 6.9 mm and 4.2 mm at the ultimate loads, respectively. The similar behaviour was also observed in Frame Test 4, in which both eaves moved in a westward direction with lower values compared to those of Frame Test 7.

In Frame Tests 2 and 6, while the north eaves displaced slightly to the west, there was almost no out-of-plane displacement in the south eaves until the attainment of ultimate vertical load occurred. However, after reaching the second peak, the lateral displacements of the south eaves in both Frame Tests were disturbed due to a frame shaking, but they stabilised afterwards.
3.2.10.5. Deformation of columns

3.2.10.5.1. Global deformation

For the representation convenience in this section, the test frames can be categorised into two groups: Frames Tests with unbraced columns (incl. Frame Tests 1, 2, 3 and 6) and Frame Tests with brace columns (incl. Frame Tests 4, 5 and 7). In the frames with unbraced columns, apart from the in-plane horizontal displacement, the flexural-torsional buckling (global

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**Figure 3.51.** Vertical load versus lateral (out-of-plane) displacement of south eaves

**Figure 3.52.** Vertical load versus lateral (out-of-plane) displacement of north eaves
buckling) of the columns was the distinctive behaviour. In the frames with braced columns, the lateral-torsional buckling of columns was eliminated by the additional girts. Therefore, the in-plane deformation is the main response of the columns.

The global deformation of the columns was monitored at three different locations in each column: at 0.18 m and 2.15 m above the base plate and 125 mm below the knee brace as shown in Section 3.2.8. Transducers were configured to measure the deflections and twist rotations of the columns. The twist rotations of the columns at 125 mm below the knee connection and at 2.15 m above the base plate are presented in Figure 3.53 and Figure 3.54, respectively. The twist rotations of the columns at 180 mm above the base plate are provided in Appendix A.5. The sign convention adopted for the deformation is shown in Figure 3.40.

The dashed line at the beginning of each curve represents the linear load-twist rotation relation, resulting from the self-weight of the frame at roof level and the weight of the load spreading system. In addition, initial slight offsets in the curves for Frame Tests 3 and 5 were taken into account for the twist rotations generated by the horizontal loading. As seen in Figure 3.53 and Figure 3.54, the general behaviours of the columns at the two considered locations were similar. In the case of braced column frames (Frame Tests 4, 5, and 7), with the presence of girts along the height of the columns, the column twisting was partially restrained, and the twist rotations of columns were very close to zero in the initial stage of loading. When a certain level of loading was reached, the twist deformation started to be observed, however, was insignificant. In contrast, in the frames with unbraced columns, twist rotations were observed clearly before the ultimate loads were reached. At the first peak-loads of Frame Tests 1, 2 and 6, the first collapsed columns exhibited larger twist rotations compared to those in the opposite columns. In the case of Frame Test 3 with a horizontal load of 7.5 kN applied to the north eaves, the twist rotation of the north column was larger than that of the south column due to a frame sway.
Figure 3.53. Applied load versus twist rotation of columns at 125 mm below the knee brace: (a) South column; (b) North column
Besides twisting, the columns also deflected both in horizontal (in-plane) and lateral (out-of-plane) directions. The horizontal displacements of the columns at 125 mm below the knee
connection and 2.15 m above the base plate are presented in Figure 3.55 and Figure 3.56, respectively. The lateral displacements at the respective locations are shown in Figure 3.57 and Figure 3.58. In addition, the horizontal displacements of the columns at 180 mm above the base plate are provided in Appendix A.5.

In terms of in-plane deformation, the columns of the frames subjected to the gravity load alone bent outwards. The horizontal displacements of the columns at 125 mm below the knee brace were generally higher than those at the mid-span (i.e. 2.15 m above the base plate) because the major bending moment developed at the knee connection was the highest in the column. In Frame Tests 3 and 5, due to the action of the horizontal load before the vertical loading, both the south and north columns displaced to the north direction although the magnitude of the displacement of the south columns was less than that of the north column. As the vertical load progressively increased, the two columns began to move away from each other. Specifically, the north column kept moving northwards, whilst the south column reversed southwards, as shown in Figure 3.55 and Figure 3.56. At the first failure, the south column suddenly moved northwards over its state before the vertical testing load was applied.

![Graph](image_url)

**Figure 3.55.** Applied load versus horizontal displacement of columns at 125 mm below the knee brace: (a) South column; (b) North column
Figure 3.56. Applied load versus horizontal displacement of columns at 2.15 m above the base plate: (a) South column; (b) North column

Figure 3.57. Applied load versus lateral displacement of columns at 125 mm below the knee brace: (a) South column; (b) North column
Figure 3.58. Applied load versus lateral displacement of columns at 2.15 m above the base plate: (a) South column; (b) North column

Similar to the observed twist deformation, the lateral displacements of the columns in Frame Tests with unbraced columns were relatively small as compared to that with the unbrace columns. As observed in Figure 3.57 and Figure 3.58, for Frame Tests 1 and 2, both columns displaced towards the west. For Frame Test 6, the south column moved westwards, whilst the north column displaced to the east direction. In the case of Frame Test 3, both columns displaced towards the east under the effect of the initial horizontal load, but they reversed westwards after the vertical testing load was applied.

3.2.10.5.2. Local deformation

Apart from the global deflections and twist rotation, the local deformations of the column sections were also monitored at 125 mm below the knee connection by using 12 transducers as presented in Section 3.2.8. Measurements from the transducers allowed the local rotation/distortional displacements of the flanges and the local displacements of the webs to be determined. The local displacements and rotations were measured with respect to the web-flange junctions.

To facilitate the representation of the local behaviour of the columns, the local deformation shown in Figure 3.59 is considered as the positive displacements and rotations. The local deformations of flanges and webs of columns at 125 mm below the knee connection are shown in Figure 3.60 to Figure 3.65.

It can be seen in the graphs that until the ultimate load was reached, the local deformation of the columns at 125 mm below the knee connection was relatively small. A maximum rotation of $2.5^\circ$ was observed at the outside flanges of the north column in Frame Tests 4 and 5 and a maximum local displacement of 3.0 mm was observed at the web 1 (east web) of the north
column in Frame Test 5. After the first peak-load, the local deformations in the columns at the knee connection spiked. This behaviour caused by the formation of a spatial plastic mechanism in the flanges and webs of the columns, especially in the Frame Tests 4 and 5. In Frame Test 7, the local deformations in the both columns were minimal resulting from the fact that the columns were stiffened by sleeve stiffeners at the region of maximum bending moment.

An interesting phenomenon depicted by the south column was that the reversal of local web displacement and local rotation of inside flanges upon reaching a certain level of loading signified the reversal of the direction of the local deformation.

![Sign convention adopted for the local deformation of the columns: (a) Local deformation of the webs; (b) Distortional deformation of flanges](image)

**Figure 3.59.** Sign convention adopted for the local deformation of the columns: (a) Local deformation of the webs; (b) Distortional deformation of flanges

![Applied load versus rotation of outside flanges of the south column](image)

**Figure 3.60.** Applied load versus rotation of outside flanges of the south column
Figure 3.61. Applied load versus rotation of inside flanges of the south column

Figure 3.62. Applied load versus local displacement of the webs of the south column
Figure 3.63. Applied load versus rotation of outside flanges of the north column

Figure 3.64. Applied load versus rotation of inside flanges of the north column
3.2.10.6. Deformation of rafters

3.2.10.6.1. Global deformation

The rafters in all frame tests were partially restrained against the lateral displacement and twist rotation especially at the initial stage of loading. As a result, vertical displacement is the major deformation of the rafters during the tests. In the tests, the deformation of the rafters was measured at the locations adjacent to the rafter tie connections. The relationship of applied vertical load versus the vertical displacement at both the south and north rafters are plotted in Figure 3.66. As seen in the graphs, the rafters gradually deflected downwards under the progressive action of the vertical load. In Frame Tests 3 and 5, due to the application of a horizontal load of 7.5 kN to the north eaves, the rafters deflected vertically in opposite directions. While the south rafter moved downwards by about 21.5 mm, the north rafter displaced upwards slightly. However, both the columns significantly deflected downwards when the vertical testing load was applied.
Figure 3.66. Applied load versus vertical displacement of rafters: (a) South rafter; (b) North rafter

Figure 3.67. Applied load versus lateral (out-of-plane) displacement of rafters: (a) South rafter; (b) North rafter
Figure 3.68. Applied load versus twist rotation of rafters: (a) South rafter; (b) North rafter

Apart from the rafter vertical displacement, the out-of-plane displacements and twist rotations of the rafters were also monitored during the tests. The corresponding graphs are shown in Figure 3.67 and Figure 3.68, respectively. As seen in the graphs, at the first peak-load, the lateral displacement and twist rotation of the rafters for the most Frame Tests are small. Specifically, the rafter twist rotation of less than 2° was observed in all Frame Tests while the rafter lateral displacements of the most Frame Tests were generally less than 5 mm, except for Frame Test 7. For Frame Test 7, the south and north rafters displaced to the east by about 12 mm and 17 mm, respectively.

3.2.10.6.2. Local deformation

The local deformation of the rafter sections at the rafter tie connection was also monitored. While the local deformation of the flanges and webs of the south rafter is shown in Figure 3.70 to Figure 3.72, that of the north rafter is presented in Figure 3.73 to Figure 3.75 (The positive local displacements and rotations are shown in Figure 3.69). Except for Frame Test 7, the rotation of the flanges of the south rafter at ultimate load was generally negligible. The maximum rotation in the south rafter was observed at the top flange 1 (east side) of Frame Test 7 with a magnitude of around 3.25°. Similarly, the local displacement of the webs of the south rafter was minimal with the largest magnitude of 1.75 mm displacement as experienced by the web 1 (east web) of Frame Test 7. In the north rafter, the local deformation for all Frame Tests was also minimal with the increase of the applied load until approaching the ultimate load. However, after reaching the second peak load, significant local displacements of the webs and rotation of the flanges were observed in Frame Test 4 and Frame Test 5 because the second failures of these frames occurred in the north rafter at the rafter tie connection.
Figure 3.69. Sign convention adopted for the local deformation of the rafters: (a) local displacement of the webs; (b) Distortional deformation of the flanges

Additional graphs explaining the local behaviour of the rafters can be found in Appendix A.5.

Figure 3.70. Applied load versus rotation of top flanges of the south rafter
Figure 3.71. Applied load versus rotation of bottom flanges of the south rafter

Figure 3.72. Applied load versus local displacement of the webs of the south rafter
Figure 3.73. Applied load versus rotation of top flanges of the north rafter

Figure 3.74. Applied load versus rotation of bottom flanges of the north rafter
Bolted connections were used to connect components together to build the completed frame systems. The turn-of-nut method was adopted to install the bolts, resulting in slip-resistant connections. Hence, the bolted connections were expected to resist slippage to a certain level of loading. However, at the eaves, apex, and base connection where the high level of loading was transferred, bolt slip might occur. Therefore, the slip was carefully monitored by using five transducers attached near the bolts located close to the junction of brackets and members at the eaves, apex, and base connections in all experiments. The details of the transducer arrangement for measuring bolt slip can be found in Section 3.2.8. The use of these transducers allowed the relative displacement of two flat sheets (i.e. one sheet belongs to the rafters/columns and the other belongs to the corresponding brackets) to be measured, indicating the amount of bolt slip. The relative displacements of the relative pair of sheets are plotted in Figure 3.76 to Figure 3.78.
Figure 3.76. Relative displacements of the web of the rafter and apex bracket

Figure 3.77. Relative displacements of the web of the rafters and eaves brackets
It can be seen in the above graphs that the slips are generally small except at the north eaves connection in Frame Test 3. The relative displacement of the web of the north rafter and the eaves bracket in Experiment 3 was recorded to be 1.5 mm at peak load. The slip at the apex connection observed in all experiments was insignificant with the maximum slip of 0.5 mm as observed in Frame Test 7. The bolt slip on the north base connection seems to be larger than that on the south base connection. The maximum relative displacement of the outside flanges of the north column and base bracket was about 1.15 mm as recognised in Frame Tests 5 and 7.

After removal of the load and disassembling of the frame, further diagnosis of the occurrence of bolt slip and bearing of bolt holes was performed. It was indicated that there is no sign of any elongation presented in all bolt holes except for the bottom bolt hole of the north rafter at the north eaves connection in Frame Test 3. The deformation of this bolt hole is shown in Figure 3.79a. The bolt hole elongations at other bolt holes were negligible indicating that the deformation was confined to the bolt hole clearance. In some bolt holes, the friction marks were observed around the holes, possibly introduced as a result of the process of inducing pretension in the bolts.
3.2.10.8. Sleeve stiffener contribution

In Frame Test 7, sleeve stiffeners with a length of 2.4 m were placed in the columns from the top of the column. The stiffeners were connected to the main columns through eleven bolts in the web at several locations along the length of the stiffeners and two bolts in the outside flanges at the eaves connection. The details of the bolt arrangement used to connect the sleeve stiffeners to the columns can be found in the relevant drawings in Appendix A.1. The test results demonstrated that the use of sleeve stiffeners in conjunction with the girt system is very efficient to improve the performance of the frame system.

To quantify the contribution of the sleeve stiffeners to the load capacity of the frame system, strain gauges were attached to the stiffeners and the columns as detailed in Section 3.2.8 to monitor the behaviour of the stiffeners and columns during testing. The data recorded by strain gauges allow the major bending moment carried by the columns and each sleeve stiffener at the location of strain gauges to be determined. In this particular test, the strain gauges were attached at the knee connection, 300 mm above the knee connections, and 300 mm and 500 mm below the knee connections.

Each strain gauge of a column measured the longitudinal strain at its position, which was induced by four stress resultants experienced by the column due to the applied load. These stress resultants in the column included the axial compression ($N$), major axis bending moment ($M_x$), minor axis bending moment ($M_y$), and the Bimoment ($B$). Longitudinal strain ($\varepsilon_i$) were measured at four positions around the cross-section on each of the flanges of the column, which allows the constitutive relations to be utilised to compute the stress resultants as follows:
where $x_c$ and $y_c$ are the distances from the strain gauges to the centroid of the column, $\omega$ is the value of sectorial coordinate at the location of the strain gauges, $A_c$ is the area of the column cross-section, $I_{cx}$ and $I_{cy}$ are the second moments of areas, $I_{cw}$ is the warping constant, and $E$ is the elastic modulus.

The stress resultants were calculated for the four different locations on the column height corresponding to the locations of relative strain gauge group. It was assumed that the action between the constituent channel sections is fully composite.

For the sleeve stiffeners, it was assumed that each stiffener acts as a separate section, and thus it was analysed individually. At one location of a sleeve stiffener on the column height, the strain was measured at two positions around the cross-section (Section 3.2.8). Thus, to solve the constitutive relations, only two stress resultants can be considered. Since the stiffener was bolted to the column through the web only except two bolts used for the outside flange at the eaves connection, its contribution to supporting axial compression force was relatively small, therefore the axial compression $N$ can be omitted. In contrast, the major axis bending moment $M_x$ was the largest internal action contributing to stress in the column, so it must be included. If only $M_x$ and $B$ are considered on the stiffener, it results in linearly dependent equations, and thus cannot be solved. Therefore, it was assumed that only $M_x$ and $M_y$ are considered to cause the longitudinal strains on the stiffeners, and thus, these stress resultants can be determined as follows:

\[
\begin{bmatrix}
\varepsilon_{1} \\
\varepsilon_{2} \\
\varepsilon_{3} \\
\varepsilon_{4}
\end{bmatrix} = \frac{1}{E} \begin{bmatrix}
\frac{1}{A_c} & y_c & x_c & \omega \\
\frac{1}{A_c} & -y_c & x_c & -\omega \\
\frac{1}{A_c} & y_c & -x_c & \omega \\
\frac{1}{A_c} & -y_c & -x_c & -\omega
\end{bmatrix} \begin{bmatrix}
N \\
M_x \\
M_y \\
B
\end{bmatrix}_{\text{column}}
\]  \hspace{1cm} (3.6)

where $x_s$ and $y_s$ are the distances from the strain gauges to the centroid of the stiffener, and $I_{sx}$ and $I_{sy}$ are the corresponding second moments of areas, and $E$ is the elastic modulus.

The stress resultants were determined for both the east and west stiffeners for different locations along the stiffener height. The major axis bending moments in the two sleeve stiffeners along with that in the corresponding column are shown in Figure 3.80 to Figure 3.83.
for different locations. It is noted that the stress resultants in the column were divided by two to determine the stress resultant per channel for direct comparison with the individual stiffeners.

Figure 3.80. Applied load versus major axis bending moment in columns and sleeve stiffeners for Frame Test 7 at 300 mm above the knee connection: (a) South column; (b) North column

Figure 3.81. Applied load versus major axis bending moment in columns and sleeve stiffeners for Frame Test 7 at the knee connection: (a) South column; (b) North column
Figure 3.82. Applied load versus major axis bending moment in columns and sleeve stiffeners for Frame Test 7 at 300 mm below the knee connection: (a) South column; (b) North column

Figure 3.83. Applied load versus major axis bending moment in columns and sleeve stiffeners for Frame Test 7 at 500 mm below the knee connection: (a) South column; (b) North column

It was determined that the bending moments carried by the sleeve stiffeners roughly equal to the bending moment in the main column. The largest difference was recognised at 500 mm below the knee connection, however, this difference was only approximately 10%. The use of the sleeve stiffeners therefore increased the ultimate load capacity of the frame system by effectively reducing the moment carried by the columns. It was determined that the sleeve stiffeners resulted in an estimated 54.5% increase in the frame ultimate vertical load as compared to the identical frame system without sleeve stiffeners.
3.2.10.9. Base reactions and Semi-rigid behaviour of the base connection

3.2.10.9.1. Base reactions

In all experiments, the upper base plates of the central frames, onto which the base brackets were bolted under each column, were supported on four load cells. The load cell had been calibrated before conducting the full-frame tests. The calibration allowed the reaction of each load cell to be determined based on the readings of the four strain gauges that were attached to the shank of each of the load cells. Further details about the arrangement and calibration of the load cells can be found in Section 3.2.5.3 and Section A.2. The reaction under each column was calculated by the algebraic sum of the reactions from the four load cells, whereas the column base moments were computed by considering the coupled contributions from the reaction generated in each load cell beneath the footing. The applied vertical load versus the column base reaction and major bending moment relations for all Frame Tests are shown in Figure 3.84 to Figure 3.90.

In Frame Tests 3 and 5, the frames were subjected to a constant horizontal load of 7.5 kN before applying the vertical load. However, in order to make it easier to compare with other experiments where the frames were subjected to gravity load only as well as to facilitate the determination of the column base stiffness in the next section, the effects of the horizontal load were not included in the graphs for Frame Tests 3 and 5. It is also noted that although the major bending moments developed at the base of respective columns were in opposite directions when the frames are subjected to vertical load, the same positive sign was used to represent the base moments of the columns for the convenience of comparison. The dash lines in the graphs represent the initial reactions generated at the base due to the load spreading system and the self-weight at roof level.

In majority of experiments, the applied vertical load was slightly greater than the combined reaction of the south and the north column bases. This indicates that the outer supporting frames also resisted the vertical load applied to the central frame. However, the contribution of these outer frames to carrying the applied is negligible. It is also seen that the major bending moments developed in the south and north bases of the Frame Tests 1 and 4 are almost identical, but in other Frame Tests, the moment in the north column is higher as compared to that in the south column.
Figure 3.84. Column base reactions for Frame Test 1: (a) Compression; (b) Major moment

Figure 3.85. Column base reactions for Frame Test 2: (a) Compression; (b) Major moment
Figure 3.86. Column base reactions for Frame Test 3: (a) Compression; (b) Major moment

Figure 3.87. Column base reactions for Frame Test 4: (a) Compression; (b) Major moment
Figure 3.88. Column base reactions for Frame Test 5: (a) Compression; (b) Major moment

Figure 3.89. Column base reactions for Frame Test 6: (a) Compression; (b) Major moment
The base connections were formed by bolting the column flanges to the vertical legs of the base brackets. The bolts were tightened by using the turn-of-nut method so that bolted connections at the bases can be considered as slip-resistant type. As a characteristic of the slip-resistant bolted configuration, the column base connections are able to resist moment, however, can also rotate under the effect of loading. Therefore, the column base connection is semi-rigid.

In order to determine the stiffness of the base connection, the moment-rotation relations need to be derived. While the bending moments at the column bases were calculated and provided in the preceding section, the base rotations were recorded during the experiments by using an inclinometer placed on each column base. The inclinometer was located at 180 mm above the base plate. The location is just right outside the influence of the connection bracket, and thus allows the results to be measured more accurately. The base rotation can also be determined from the readings of transducers attached at the base regions. The details of the arrangement of inclinometers and transducers can be found in Section 3.2.8. The in-plane major axis bending moment versus the corresponding rotation graphs due to the action of the vertical testing loads are provided in Figure 3.91 and Figure 3.92. In the graphs for Frame Tests 3 and 5, only the effect of the vertical load was considered although these frames were tested with both horizontal and vertical loads. The dash lines at the initial part of the curves represent the relations generated due to the weight of the load spreading system and the self-weight at roof level.
Figure 3.91. Base moment versus rotation for vertical load only: (a) Frame Test 1; (b) Frame Test 2; (c) Frame Test 3; (d) Frame Test 4
To quantify the semi-rigidity of the base connection, the flexural stiffness of the column bases for each Frame Test were estimated by using linear regression on the initial portion of the respective moment-rotation curves as seen in Figure 3.91 and Figure 3.92. The column base stiffness for bending about the major axis is given in Table 3.8 for both the south and north columns of the test frames.

As seen in Table 3.8, there is considerable dispersion in the base stiffness obtained from the full-scale frame tests although all frames had an identical base connection geometry and used the same method to connect the connection components. The reasons for the variability are not obvious. On average, the column bases had a stiffness of 765.96 kNm/rad with a coefficient of variation of 0.25.
### Table 3.8. Column base stiffness measured from the full-scale frame tests

<table>
<thead>
<tr>
<th>Test series</th>
<th>Column</th>
<th>Base Stiffness (kNm/rad)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frame Test 1</td>
<td>South</td>
<td>1065.30</td>
</tr>
<tr>
<td></td>
<td>North</td>
<td>984.80</td>
</tr>
<tr>
<td>Frame Test 2</td>
<td>South</td>
<td>687.32</td>
</tr>
<tr>
<td></td>
<td>North</td>
<td>601.26</td>
</tr>
<tr>
<td>Frame Test 3</td>
<td>South</td>
<td>898.46</td>
</tr>
<tr>
<td></td>
<td>North</td>
<td>323.21</td>
</tr>
<tr>
<td>Frame Test 4</td>
<td>South</td>
<td>704.34</td>
</tr>
<tr>
<td></td>
<td>North</td>
<td>649.05</td>
</tr>
<tr>
<td>Frame Test 5</td>
<td>South</td>
<td>797.56</td>
</tr>
<tr>
<td></td>
<td>North</td>
<td>814.80</td>
</tr>
<tr>
<td>Frame Test 6</td>
<td>South</td>
<td>681.65</td>
</tr>
<tr>
<td></td>
<td>North</td>
<td>792.06</td>
</tr>
<tr>
<td>Frame Test 7</td>
<td>South</td>
<td>979.7</td>
</tr>
<tr>
<td></td>
<td>North</td>
<td>743.99</td>
</tr>
<tr>
<td>Mean</td>
<td></td>
<td>765.96</td>
</tr>
<tr>
<td>COV</td>
<td></td>
<td>0.25</td>
</tr>
</tbody>
</table>
3.3. MATERIAL MECHANICAL PROPERTIES

The mechanical properties of the frame components are essential for the design of the appropriate sections and the accurate prediction of structural behaviour in numerical models. Hence, a comprehensive series of coupon tests were performed to determine the material properties and the post-yield material behaviour of the frame components. The test program including 63 coupon tests was classified into tensile flat coupon tests, tensile corner coupon tests and compressive flat coupon tests. The details of the coupon test program are described in the following sections.

3.3.1. Mechanical Properties of the Flat Material in Tension

3.3.1.1. Preparation

A total of 13 sets of tensile flat coupon tests with each set consisting of 4 specimens were carried out to obtain the material properties of the CRA AC25025 sections, which were used to form the primary frame members (i.e. columns and rafters) and the aluminium brackets. The details of the tensile flat coupon test series are given in Table 3.9. Due to the effect of the cold-rolling process, the mechanical properties of cold-rolled aluminium sections may vary in different directions relative to the direction of rolling, i.e. parallel, perpendicular and diagonal to the rolling direction, and thus it is necessary to conduct the tensile flat coupon tests for the cold-rolled aluminium columns and rafters at different locations around the cross-section in longitudinal, transverse, and 45° diagonal directions. In this study, the longitudinal coupons were cut from the web and the two flanges of the AC25025 section, whilst the coupon specimens for other directions were cut from the central portion of the web of the columns and rafters. The aluminium brackets at the apex, eaves, and knee brace connections are also the crucial components in the frame, and thus the material properties of these components also need to be determined. For this reason, tensile flat coupon tests with the coupon specimens taken from the flat portion of the apex, knee and knee brace brackets were also conducted.

All coupon specimens to the Australian Standard AS1391:2007 [240] were machined using a high-precision water jet cutting machine with dimensions as shown in Figure 3.93. After machining the samples, the original gauge length along with its mid-point was marked, using a scribe, on each coupon. The actual relevant dimensions of each coupon, subsequently used to compute the tensile stress, were also measured before testing. To ensure the accuracy of the measurements, a micrometer was used for measuring the thickness, whilst the width was measured by a digital vernier calliper, and each measurement was completed at three locations equidistant from each other within the gauge length. The details of the measurements are provided in Appendix B.1.

All coupons were instrumented with the calibrated extensometer of 25 mm gauge length to measure the extension of the coupons during testing. Two strain gauges were also attached to the mid-length of the selected coupon specimens for the accurate determination of the initial
elastic modulus. After the surfaces of the specimens at the central portion were cleaned with acetone, the strain gauges with a gauge length of 5mm were glued to the two surfaces of each coupon at the marked mid-length.

Table 3.9. Tensile flat coupon test series

<table>
<thead>
<tr>
<th>Set No</th>
<th>Coupon Source</th>
<th>Coupon ID</th>
<th>Nominal thickness (mm)</th>
<th>No of tests (Nos)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Web</td>
<td>AC25025_CL_LW_(1-4)</td>
<td>2.5</td>
<td>4</td>
</tr>
<tr>
<td>2</td>
<td>Longitudinal</td>
<td>AC25025_CL_LTF_(1-4)</td>
<td>2.5</td>
<td>4</td>
</tr>
<tr>
<td>3</td>
<td>Outside flange</td>
<td>AC25025_CL_LBF_(1-4)</td>
<td>2.5</td>
<td>4</td>
</tr>
<tr>
<td>4</td>
<td>Transverse</td>
<td>AC25025_CL_TRW_(1-4)</td>
<td>2.5</td>
<td>4</td>
</tr>
<tr>
<td>5</td>
<td>Diagonal</td>
<td>AC25025_CL_DW_(1-4)</td>
<td>2.5</td>
<td>4</td>
</tr>
<tr>
<td>6</td>
<td>Web</td>
<td>AC25025_RT_LW_(1-4)</td>
<td>2.5</td>
<td>4</td>
</tr>
<tr>
<td>7</td>
<td>Longitudinal</td>
<td>AC25025_RT_LTF_(1-4)</td>
<td>2.5</td>
<td>4</td>
</tr>
<tr>
<td>8</td>
<td>Outside flange</td>
<td>AC25025_RT_LBF_(1-4)</td>
<td>2.5</td>
<td>4</td>
</tr>
<tr>
<td>9</td>
<td>Transverse</td>
<td>AC25025_RT_TRW_(1-4)</td>
<td>2.5</td>
<td>4</td>
</tr>
<tr>
<td>10</td>
<td>Diagonal</td>
<td>AC25025_RT_DW_(1-4)</td>
<td>2.5</td>
<td>4</td>
</tr>
<tr>
<td>11</td>
<td>Apex bracket</td>
<td>ApexB_(1-4)</td>
<td>6.0</td>
<td>4</td>
</tr>
<tr>
<td>12</td>
<td>Knee bracket</td>
<td>KneeB_(1-4)</td>
<td>6.0</td>
<td>4</td>
</tr>
<tr>
<td>13</td>
<td>Knee brace</td>
<td>KneeBraceB_(1-4)</td>
<td>6.0</td>
<td>4</td>
</tr>
</tbody>
</table>

Figure 3.9.3. Dimensions of tensile flat coupon specimen

3.3.1.2. Testing of coupons

The coupon tests were performed using a 50 kN capacity MTS Model 43 testing machine in the J.W. Roderick Laboratory at the University of Sydney, as shown in Figure 3.94. The
coupon specimen was set up by clamping the upper end first and checking the vertical alignment using a plastic Pocket Level, followed by clamping the lower end. Once the coupon was clamped, the calibrated MTS extensometer of 25 mm gauge length was mounted at the central portion of the coupon. The readings were reset to zero before applying load.

The experiments commenced by applying the incremental tensile load with displacement controlled at the rate of 0.4 mm/min corresponding to a strain rate of about $2.5 \times 10^{-4}$ s$^{-1}$. A data acquisition system was used to record data (load, extension, and strain) at regular intervals during the tests. The tests were regularly paused for at least 100s, as recommended by Huang and Young [241], to allow stress relaxation and thus obtain static material properties. After the fracture, the specimens were taken out of the rig and recorded data were processed to produce the relevant material properties.

![Figure 3.94. Tensile coupon tests arrangement: (a) AC25025 Coupon; (b) Bracket coupon](image)

**3.3.1.3. Results of coupon tests**

The stress-strain relation of each coupon test was determined based on the recorded data and the measured dimensions of the coupon. The typical stress-strain curves obtained from the tensile flat coupon tests are plotted in Figure 3.95. Figure 3.95a shows the typical stress-strain relations for aluminium brackets at the apex, eaves, and knee brace connections along with the mean of all coupon tests for brackets, whilst typical stress-strain relations of the AC25025 section in three directions and the relevant average are presented in Figure 3.95a, Figure 3.95b, and Figure 3.95c, respectively. The stress-strain relationship curves for other specimens can be seen in Appendix B.1. As evidently seen in Figure 3.95, the serrated phenomenon occurred in all coupon tests after reaching 0.2% proof stress. This phenomenon can be explained by the precipitation of Mg atoms in the macroscopic unstable plastic flow [21, 242]. Based on the
stress-strain curves, the mechanical properties were determined by taking the mean of all relevant coupons tests and are presented in Table 3.10.

![Typical stress-strain curves](image)

**Figure 3.95. Typical stress-strain curves:** (a) Aluminium brackets; (b) AC25025 in Longitudinal direction; (c) AC25025 in Transverse direction; (d) AC25025 in Diagonal direction

**Table 3.10. Mechanical properties of AC25025 section and connection brackets**

<table>
<thead>
<tr>
<th>Coupon Source</th>
<th>Test rate</th>
<th>$E$</th>
<th>$\sigma_{0.01}$</th>
<th>$\sigma_{0.2}$</th>
<th>$\sigma_u$</th>
<th>$n$</th>
<th>$\varepsilon_u$</th>
<th>$\varepsilon_f$</th>
</tr>
</thead>
<tbody>
<tr>
<td>AC25025_Longitudinal</td>
<td>0.4</td>
<td>70.25</td>
<td>167.87</td>
<td>210.41</td>
<td>261.39</td>
<td>13.55</td>
<td>6.79</td>
<td>7.93</td>
</tr>
<tr>
<td>AC25025_Transverse</td>
<td>0.4</td>
<td>71.39</td>
<td>157.28</td>
<td>201.10</td>
<td>258.95</td>
<td>12.27</td>
<td>7.95</td>
<td>13.26</td>
</tr>
<tr>
<td>AC25025_Diagonal</td>
<td>0.4</td>
<td>71.02</td>
<td>150.55</td>
<td>203.73</td>
<td>257.18</td>
<td>10.01</td>
<td>6.87</td>
<td>8.35</td>
</tr>
<tr>
<td>Aluminium Brackets</td>
<td>0.4</td>
<td>72.22</td>
<td>180.55</td>
<td>221.57</td>
<td>348.59</td>
<td>14.86</td>
<td>11.63</td>
<td>10.31</td>
</tr>
</tbody>
</table>
The columns of Table 3.10 show: (1) The speed of tests, (2) the mean elastic Young’s modulus \( E \) of the flat portions of the AC25025 section in the respective directions and the aluminium brackets, (3) the stress \( \sigma_{0.01} \) corresponding to 0.01% strain; (4) 0.2% proof stress \( \sigma_{0.2} \), which is the equivalent to yield stress; (5) the ultimate tensile strength \( \sigma_u \), (6) the Ramberg-Osgood index “n”, describing the roundedness of the stress-strain curve in the transition from the elastic to the inelastic, (7) the strain corresponding to the ultimate tensile strength \( \varepsilon_u \), and (8) the total strain after fracture \( \varepsilon_f \). The results of the individual coupon tests are provided in Appendix B.1.

The test results show that the variations among different specimens within each coupon group are small, as clearly seen in Appendix B.1. For example, the coefficient of variations of elastic modulus, yield stress, and ultimate tensile strength calculated for the AC25025 section in the longitudinal direction are 0.039, 0.02, and 0.015 respectively. Therefore, the mean values can be used for subsequent design calculations and numerical finite element investigations. It is also revealed in Table 3.10 that there is the existence of a certain degree of anisotropy in mechanical properties in the AC25025 section as exhibited by the different magnitudes of elastic modulus, yield stress, ultimate strength and ductility in coupon specimens from different directions.

3.3.2. Mechanical Properties of the corner Material in tension

3.3.2.1. Preparation and testing

To assess the effects of cold-rolling on the material properties of the AC25025 section, longitudinal tensile tests were performed on small coupon specimens taken from the corners of the section as shown in Figure 3.96. The coupons were cut within the boundary of the internal radius \( r \) of the corner, as illustrated in Figure 3.96a, using a high-performance wire-cut electric discharge machine (Figure 3.97a). Among 8 specimens prepared, 4 samples were cut from the web-flange corners (Figure 3.96b), whilst the others were extracted from the flange-lip junctions (Figure 3.96c).

![Figure 3.96. Samples for tensile corner coupon tests](attachment:image.png)
In all corner coupon tests, both the extensometer and strain gauges were used to measure the extension of the specimens. The corner coupon tests were conducted in a specially designed test rig with pinned ends as shown in Figure 3.97b. The coupon specimens were tested in the 50 kN capacity MTS Model 43 testing machine with the same testing procedure as implemented for the tensile flat coupon tests.

![Figure 3.97](image)

Figure 3.97. Corner coupon tests: (a) Machining the sample in the wire-cut machine; (b) coupon test setup in the MTS Model 43 testing machine

3.3.2.2. Results

Figure 3.98 shows the stress-strain relationship curves obtained from the tensile corner coupon tests and the calculated material properties are provided in Table 3.11 along with the longitudinal mechanical properties of the flat portion of the section.

![Figure 3.98](image)

Figure 3.98. Tensile stress-strain curves of the corner coupon tests: (a) plots for all specimens; (b) Comparison of the flat and corner properties
Table 3.11. Mechanical properties of the corner portion of the AC25025 section

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Test rate mm/min</th>
<th>$E$ (GPa)</th>
<th>$\sigma_{0.01}$ (MPa)</th>
<th>$\sigma_{0.2}$ (MPa)</th>
<th>$\sigma_u$ (MPa)</th>
<th>$\varepsilon_u$ (%)</th>
<th>$\varepsilon_f$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AC25025_WF_LC_1</td>
<td>0.4</td>
<td>71.90</td>
<td>179.27</td>
<td>229.37</td>
<td>275.84</td>
<td>12.16</td>
<td>6.66</td>
</tr>
<tr>
<td>AC25025_WF_LC_2</td>
<td>0.4</td>
<td>71.88</td>
<td>181.51</td>
<td>230.21</td>
<td>275.06</td>
<td>12.60</td>
<td>6.55</td>
</tr>
<tr>
<td>AC25025_WF_LC_3</td>
<td>0.4</td>
<td>70.98</td>
<td>181.89</td>
<td>228.75</td>
<td>274.03</td>
<td>13.07</td>
<td>4.90</td>
</tr>
<tr>
<td>AC25025_WF_LC_4</td>
<td>0.4</td>
<td>72.25</td>
<td>183.37</td>
<td>230.00</td>
<td>273.90</td>
<td>13.22</td>
<td>4.62</td>
</tr>
<tr>
<td>AC25025_FL_LC_1</td>
<td>0.4</td>
<td>72.13</td>
<td>181.41</td>
<td>227.59</td>
<td>275.95</td>
<td>13.21</td>
<td>6.56</td>
</tr>
<tr>
<td>AC25025_FL_LC_2</td>
<td>0.4</td>
<td>71.93</td>
<td>183.03</td>
<td>228.74</td>
<td>276.61</td>
<td>13.44</td>
<td>5.47</td>
</tr>
<tr>
<td>AC25025_FL_LC_3</td>
<td>0.4</td>
<td>71.46</td>
<td>180.97</td>
<td>230.12</td>
<td>276.88</td>
<td>12.47</td>
<td>5.15</td>
</tr>
<tr>
<td>AC25025_FL_LC_4</td>
<td>0.4</td>
<td>71.70</td>
<td>176.88</td>
<td>231.49</td>
<td>274.88</td>
<td>11.14</td>
<td>5.33</td>
</tr>
<tr>
<td>Mean</td>
<td></td>
<td>71.78</td>
<td>181.04</td>
<td>229.53</td>
<td>275.39</td>
<td>12.66</td>
<td>5.65</td>
</tr>
<tr>
<td>Longitudinal Mean</td>
<td></td>
<td>70.25</td>
<td>167.87</td>
<td>210.41</td>
<td>261.39</td>
<td>13.55</td>
<td>6.79</td>
</tr>
</tbody>
</table>

The test results indicated that there is the enhancement of strengths at the corner due to the cold-forming process. The average yield stress and ultimate strength at the corner are 229.53 MPa and 275.39 MPa, which are 9.1% and 5.4% higher than the results of the flat portion in the longitudinal direction, respectively. Likewise, the elastic modulus at the corner is also slightly higher compared to that at the flat portion.

3.3.3. Mechanical Properties of the Flat Material in Compression

3.3.3.1. Preparation and testing

The compressive material properties of the AC25025 section were also tested in the rolling direction (longitudinal direction). A special supporting jig with the height of 90 mm was specially designed and particularly used for the compressive flat coupon tests as shown in Figure 3.99a. The use of such jig was mainly motivated by the successful utilisation in the previous studies [21, 70, 238, 239, 243, 244]. Coupon specimens were cut from the flat portion of the AC25025 section with the nominal dimensions of 25 mm by 93 mm. Three single coupons were glued together to build-up a compressive specimen, and its edges were subsequently machined flat. A total of three compressive specimens were prepared as the tests were repeated three times to ensure the accuracy of the test results.

The actual dimensions of the specimens were measured prior to testing. The details of the measurements are provided in Table 3.12. Two strain gauges with a gauge length of 10 mm
were glued to the two laminated faces of each specimen at mid-length to measure the longitudinal strain of the coupon. The sides of the specimens were lubricated before being placed into the supporting jig to eliminate friction between the specimen surfaces and the jig. The six bolts were mildly tightened to prevent the specimen from buckling about its weak axis, whilst allowing the coupons to expand laterally due to the Poisson’s effect. When placed in the jig, the coupons protruded about 3 mm beyond the top of the jig, which allowed sufficient strain to develop in the coupon tests.

Table 3.12. Compressive coupon test series

<table>
<thead>
<tr>
<th>Coupon source</th>
<th>Coupon ID</th>
<th>Thickness (mm)</th>
<th>Width (mm)</th>
<th>Average Area (mm²)</th>
<th>Length (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Measured</td>
<td>Mean</td>
<td>Measured</td>
<td>Mean</td>
</tr>
<tr>
<td>AC25025-1</td>
<td>7.30</td>
<td>7.29</td>
<td>7.29</td>
<td>7.30</td>
<td>7.29</td>
</tr>
<tr>
<td>AC25025-2</td>
<td>7.32</td>
<td>7.29</td>
<td>7.30</td>
<td>7.28</td>
<td>7.29</td>
</tr>
<tr>
<td>AC25025-2</td>
<td>7.28</td>
<td>7.29</td>
<td>7.29</td>
<td>7.28</td>
<td>7.29</td>
</tr>
</tbody>
</table>

Figure 3.99. Compressive coupon test: (a) Specimens and supporting jig; (b) Test setup

The compressive coupon tests were performed in a 300 kN capacity MTS Sintech 65/G machine. The test rig, as shown in Figure 3.99b, included a fixed loading ram and an adjustable base which was utilised to make the top and bottom surfaces of the specimen flush with the loading platforms. The coupons were tested with a displacement rate of 0.1 mm/min and loading
was regularly paused to allow stress relaxation and thus obtain static material properties. The tests were stopped when strain gauges reached their measuring range at about 1.5% strain and data were processed to produce the material properties.

3.3.3.2. Results

The stress-strain curves for the compressive coupon tests are shown in Figure 3.100a, and the calculated material properties are summarised in Table 3.13. The average compressive stress-strain curve is also plotted in Figure 3.100b along with the average tensile flat coupon stress-strain curve in the longitudinal direction.

It can be seen that the tensile and compressive mechanical properties of the CRA sections are almost identical. This conclusion was also confirmed by Huynh et al. [21]. Therefore, the same material properties are assigned to both compressive and tensile parts of the cold-rolled aluminium sections in numerical modelling and design calculations in latter sections.

![Figure 3.100. Compressive stress-strain curves of the AC25025 section](image)

**Table 3.13. Mechanical properties of the AC25025 section in compression**

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Test rate (mm/min)</th>
<th>$E$ (GPa)</th>
<th>$\sigma_{0.01%}$ (MPa)</th>
<th>$\sigma_{0.2%}$ (MPa)</th>
<th>$\varepsilon_{0.01%}$ (%)</th>
<th>$\varepsilon_{0.2%}$ (%)</th>
<th>$n$</th>
</tr>
</thead>
<tbody>
<tr>
<td>AC25025-1</td>
<td>0.1</td>
<td>69.83</td>
<td>159.42</td>
<td>209.45</td>
<td>0.24</td>
<td>0.50</td>
<td>11.0</td>
</tr>
<tr>
<td>AC25025-2</td>
<td>0.1</td>
<td>69.53</td>
<td>166.65</td>
<td>210.16</td>
<td>0.25</td>
<td>0.50</td>
<td>12.9</td>
</tr>
<tr>
<td>AC25025-3</td>
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<td>69.64</td>
<td>166.88</td>
<td>210.48</td>
<td>0.25</td>
<td>0.50</td>
<td>12.9</td>
</tr>
<tr>
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<td></td>
<td>69.67</td>
<td>164.32</td>
<td>210.03</td>
<td>0.25</td>
<td>0.50</td>
<td>12.3</td>
</tr>
<tr>
<td>Longitudinal Mean</td>
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<td>167.87</td>
<td>210.41</td>
<td>0.25</td>
<td>0.50</td>
<td>13.6</td>
</tr>
</tbody>
</table>
3.4. POINT FASTENER CONNECTION TESTS

In the portal frames in this study, bolt connections were used to assemble individual components into the complete frame system. The bolts in the connections can be idealised in the shell finite element models of the frames by using the deformable mesh-independent point-based fasteners as fully described in Section 4.4. For this purpose, the force-displacement relations of the fastener connections are required for the simulation of the actual deformable behaviour of the fasteners. Therefore, the point fastener connection tests were carried out and are presented in detail in this Section.

There are several types of fasteners connecting various components in the full-scale frame tests. These included M16 slip-resistant bolted connections, which were utilised to form the back-to-back sections and connect the main members to their respective brackets, and M12 snug-tight bolted connections, which were used to connect the purlins to purlin brackets. The M16 slip-resistant bolts were used in the frames to either connect a nominal 2.5 mm thick cold-rolled aluminium plate to a 6 mm thick aluminium bracket plate (single lap joint type) or fasten two thick aluminium bracket plates with two outer cold-rolled aluminium plates (double lap joint type). In such connections, point fasteners can experience either shear force or torque and/or the combination of both depending on the locations of the fasteners in the bolt group.

In this study, five series of point fastener tests were conducted to determine the load-deformation characteristics of each type of fastener connections. These included the single lap M16 slip-resistant bolted connection in shear, double lap M16 slip-resistant bolted connection in shear, single lap and double lap M16 slip-resistant bolted connections under torque, and M12 bolted connection in shear. The test program is summarised in Table 3.14. The details of individual tests are demonstrated in the following section.

<table>
<thead>
<tr>
<th>Test series</th>
<th>Configurations</th>
<th>Number of tests (Nos)</th>
<th>Fastener type used</th>
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</thead>
<tbody>
<tr>
<td>1</td>
<td>Slip-resistance bolted connection in Shear (Single-lap Joint)</td>
<td>3</td>
<td>M16 stainless steel bolts/nuts/washers</td>
</tr>
<tr>
<td>2</td>
<td>Slip-resistance bolted connection in Shear (Double-lap Joint)</td>
<td>3</td>
<td>M16 stainless steel bolts/nuts/washers</td>
</tr>
<tr>
<td>3</td>
<td>Slip-resistance bolted connection under Torque (Single-lap Joint)</td>
<td>3</td>
<td>M16 stainless steel bolts/nuts/washers</td>
</tr>
<tr>
<td>4</td>
<td>Slip-resistance bolted connection under Torque (Double-lap Joint)</td>
<td>3</td>
<td>M16 stainless steel bolts/nuts/washers</td>
</tr>
<tr>
<td>5</td>
<td>M12 bolted connection in Shear (For Purlin Connection)</td>
<td>3</td>
<td>M12 stainless steel bolts/nuts/washers</td>
</tr>
</tbody>
</table>
3.4.1. Slip-resistant bolted connection in shear tests

3.4.1.1. The slip-resistance of M16 bolted connections in shear

Experiments on the single-lap-joint and double-lap-joint specimens in shear were carried out to obtain the load-deformation characteristics of each type of fastener connections. In order to minimise the sheeting curling and tilting of the connection, two M16 stainless steel bolts were used to rigidly fasten the flat plates together in each specimen. The turn-of-nut method was utilised to induce the desired level of pretension in the bolts to achieve the required clamping force. This procedure is similar to that adopted for the M16 slip-resistant bolted connection in the full-scale frame tests. The single-lap-joint specimen consisted of two flat plates with the nominal thicknesses of 2.5 mm and 6 mm, cut from the cold-rolled aluminium AC250 section and the aluminium brackets, respectively. This connection type represents the bolted connection between the flanges of a column and the eaves bracket. Meanwhile, the double-lap-joint specimen, formed by two 6 mm thick aluminium plates and two outer 2.5 mm thick cold-rolled aluminium plates, represents the bolted connection between the back-to-back sections and the back-to-back brackets.

For preparation of the experiments, the flat plates were cut from the AC25025 channels and aluminium brackets, using the water-jet laser cutter. The dimensions of the plates for all types of specimens can be found in the drawings for fastener tests in Appendix A.1. The surfaces of the plates were thoroughly cleaned to remove any dirt prior to fastening together. The bolts were placed concentrically to the bolt holes and tightened with the level of pretension in the bolts induced by the turn-of-nut method. However, maintaining concentricity between the bolts and the bolt holes was not always feasible since both bolts and washers were likely to move during the pretension phase.

The lap-joint shear tests were performed using a 100 kN capacity 810 MTS testing machine in the J.W. Roderick Laboratory at the University of Sydney. The prepared specimens were placed vertically in the test rig and the verticality was checked using a handheld level. Figure 3.101 illustrates a typical single-lap-joint shear test setup. Packing plates were used at the ends of specimens at the grips to minimize the eccentric loading as recommended in the standards [245, 246]. Four displacement transducers were used to capture the deformation of the specimens during the tests. The transducers were positioned at a distance of 40 mm from the bolt centre and away from one end of the plates (Figure 3.101). This setting ensured that only the deformation near the bolted portion was monitored. Aluminium angle brackets with a size of 50 mm x 50 mm x 3 mm and a length of 80 mm were glued to the specimen to provide bearing areas for the transducers. The average results of two transducers placed symmetrically on both flat sides of the plates were used for deformation calculations. As a result, the bending effect of the plates on the measured deformation during loading can be minimized and the use of relative deformation of the upper and lower transducers can eliminate the displacement caused by slippage of the grips at two ends. The specimens were tested by applying monotonic
shear loading using displacement control at a rate of 0.25 mm/minute. The test was stopped when the connector failed.

Figure 3.101. Slip-resistant bolted connection shear test setup

The load-deformation relationship curves of the bolted connections obtained from the tests are illustrated in Figure 3.102, and the corresponding ultimate capacities are provided in Table 3.15. All specimens failed by fracture in tension of the 2.5 mm thick cold-rolled aluminium plates at the position of the upper bolt. The typical failure modes are presented in Appendix B.2.

The relative displacements for an individual bolt are plotted in Figure 3.103, in which the applied shear force was assumed to be equally distributed to the two bolts. It was found that the average slip loads per bolt are 7.68 kN and 14.59 kN for single-lap-joint and double-lap-joint, respectively. These slip loads were obtained at the relative slip between plates of 0.13 mm as defined in the Clause J4 of AS 4100 [214]. The constitutive relations are used to simulate the bolt and bolt-hole deformation in the numerical finite element models of the cold-rolled aluminium portal frames as fully demonstrated in Chapter 4.
Figure 3.102. Load versus displacement of M16 bolted connection in shear: (a) Single lap-joint; (b) Double lap-joint

Table 3.15. Ultimate capacity of M16 bolted connection in shear

<table>
<thead>
<tr>
<th></th>
<th>Single lap-joint</th>
<th></th>
<th>Double lap-joint</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Test specimens</td>
<td>Ultimate load</td>
<td>Test specimens</td>
<td>Ultimate load</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(kN)</td>
<td></td>
<td>(kN)</td>
<td></td>
</tr>
<tr>
<td>SBF-1</td>
<td>43.30</td>
<td>DBF-1</td>
<td>79.32</td>
<td></td>
</tr>
<tr>
<td>SBF-2</td>
<td>43.29</td>
<td>DBF-2</td>
<td>75.56</td>
<td></td>
</tr>
<tr>
<td>SBF-3</td>
<td>42.60</td>
<td>DBF-3</td>
<td>78.61</td>
<td></td>
</tr>
<tr>
<td><strong>Mean</strong></td>
<td><strong>43.07</strong></td>
<td><strong>Mean</strong></td>
<td><strong>77.83</strong></td>
<td></td>
</tr>
<tr>
<td><strong>COV</strong></td>
<td><strong>0.009</strong></td>
<td><strong>COV</strong></td>
<td><strong>0.026</strong></td>
<td></td>
</tr>
</tbody>
</table>

Figure 3.103. Load versus displacement of individual M16 bolts in shear: (a) Single lap-joint; (b) Double lap-joint
3.4.1.2. The M12 snug-tight bolted connection in shear

The shear tests were also conducted on the snug-tight M12 bolted connection to determine the force-displacement relation for incorporation in the finite element models. For accuracy, the experiment was based on the test of three specimens. The typical test setup and the connection configurations are shown in Figure 3.104.

![Figure 3.104. M12 bolted connection test setup](image)

The specimens were prepared by rigidly clamping the two machined plates using G-clamps whilst ensuring the concentric alignment of the bolt holes of the plates. Two M12 stainless steel bolts were placed concentrically in the bolt holes and snug-tightened with a handheld spanner that was also used for the tightening of M12 bolts in the connections between purlins and purlin cleats in the full-scale frame tests.

Similar to the shear tests of the slip-resistant M16 bolted connection, packing plates were also provided at the end of specimens at the grips to minimise the bending in the plates. Two transducers were placed symmetrically on both flat sides of the plates at each measuring position to produce average results. This setting aimed at eliminating the effect of bending of the plates on the measured deformation during loading. The grips at the bottom of the specimen were connected to the crosshead of the MTS machine, which was loaded with a constant displacement of 0.25 mm/minute.

The force-deformation response of the M12 bolted connection is presented in Figure 3.105 along with the average value curve. As observed in Figure 3.105, when the load reached approximately 5 kN, the plates started slipping against each other as the friction induced by the snug-tight condition exceeded. Thereafter, slip ensued until the bolt hole clearance of each plate was fully traversed. The load was then resisted by the ply bearing of the bolts on the plates and continued to increase until reaching a peak load at approximately 20 kN. All specimens
ultimately failed by the 2.5 mm cold-rolled aluminium plate fracture in tension at the location of the lower bolt. The ultimate loads obtained from different specimens are provided in Table 3.16 along with the mean ultimate load.

![Graph showing load versus relative displacement of M12 bolted connection](image)

**Figure 3.105. Load versus relative displacement of M12 bolted connection**

**Table 3.16. The ultimate capacity of M12 bolted connection in shear**

<table>
<thead>
<tr>
<th>Test specimens</th>
<th>Ultimate load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M12 – 1</td>
<td>20.24</td>
</tr>
<tr>
<td>M12 – 2</td>
<td>20.14</td>
</tr>
<tr>
<td>M12 – 3</td>
<td>20.01</td>
</tr>
<tr>
<td><strong>Mean</strong></td>
<td><strong>20.13</strong></td>
</tr>
<tr>
<td><strong>COV</strong></td>
<td><strong>0.006</strong></td>
</tr>
</tbody>
</table>

### 3.4.2. Slip-resistant bolted connection under torque tests

Experiments on slip-resistance of the point fastener subjected to the in-plane torque were also carried out to obtain the moment-slip rotation relations. Two configurations of the M16 slip-resistant bolted connection specimens were tested under torque, including single-lap and double-lap joints, with three specimens for each type.

Similar to the shear tests, the flat plates were cut from the AC25025 channels and the aluminium brackets by the water-jet laser cutter. The dimensions of the plates are provided in the drawings for fastener tests in Appendix A.1. After the surfaces of the plates were cleaned, a bolt was placed concentrically to the bolt hole and tightened manually with the sufficient force to hold the places together. The desired level of pretension in the bolt was then achieved by using the turn-of-nut method.

The tests were also conducted using the same 810 MTS testing machine. A typical test setup for single-lap-joint and double-lap-joint specimens is shown in Figure 3.106. The specimens were subjected to a torque at the bolt location created by a tensile force applied at the pinned connections at its ends as shown in Figure 3.106c. To create pinned conditions at the specimen ends, steel gripping plates were utilised and two M18 bolts with a plain shank
were used to connect the specimen to the gripping plates. In addition, to minimise friction between the gripping plates and the specimen, packing plates were placed in between the gripping plates at the grips. Further, the nuts of the two pin-joints were kept loose during the tests. Two dual Fredericks model 0751-3002 inclinometers, having a linear range of \( \pm 30^0 \), were attached at the mid-length of two plates of the specimen to measure the inclination of the plates. The tests were conducted by applying a displacement controlled loading with the test rate of 0.25 mm/minute.

Based on the initial measurements of the specimens before testing and the rotation of the two connected plates along with the load recorded during the tests, the moment at the fastener point was deduced. The moment-slip rotation curves of the bolted connection are plotted in Figure 3.107. Details of the initial geometric measurement of the specimens and the calculation of the moment at the bolt point are presented in Appendix B.2. It is noted that when using this method for testing, the bolt experienced shear force in addition to the torque. However, the shear force developed at the bolt position was much lower the force required to induce slip in the corresponding connection (Section 3.4.1.1). Therefore, the developed shear force has an insignificant impact on the results. In deriving the moment-slip rotation relationships, it is also implicitly assumed that deformation of the plates remained small and that the bolt did not experience large displacement while undergoing rotation.

![Figure 3.106. Tests on slip-resistant M16 bolted connection subjected to torque: (a) Single lap joint; (b) Double lap joint; (c) Dimensions and positions of instrumentation](image)
Figure 3.107. Moment versus relative rotation of M16 bolted connection: (a) Single lap-joint; (b) Double lap-joint
3.5. BASE CONNECTION TESTS

Column base rotation experiments were conducted to quantify the flexural stiffnesses of the base connections about both major and minor axes of the column. A total of 16 specimens with various base connection brackets, including 3 mm, 6 mm, and 8 mm L-plates and 3 mm U-plate made from stainless steel material, were tested. The purpose of this testing series was not only to evaluate the base stiffness of the base connections used in the full-scale frame tests but also to examine the effect of the brackets on the column base stiffness. Details of an experimental program for the base connection are given in Table 3.17. In Table 3.17, the test specimens were labelled in order to express the name of the component test such as the shape types of brackets, the thicknesses, the test type, and the serial number. Typical test label “BC_Lplate_8_MA-1” is defined as follows: “BC” indicates the base connection test; “Lplate” indicates the L-shape of the bracket; “8” indicates the thickness of the bracket in mm; “MA” indicates the major axis bending test; and “1” indicates the test number 1 in the series.

<table>
<thead>
<tr>
<th>Test series</th>
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<th>Components</th>
<th>Thickness (mm)</th>
<th>No. of Test (Nos)</th>
<th>Test type</th>
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<td>1</td>
<td>BC_Lplate_8_MA-(1-2)</td>
<td>L Plate</td>
<td>8</td>
<td>2</td>
<td>Major axis bending</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Washer Plate</td>
<td>6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>BC_Lplate_8_MI-(1-2)</td>
<td>L Plate</td>
<td>8</td>
<td>2</td>
<td>Minor axis bending</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Washer Plate</td>
<td>6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>BC_Lplate_6_MA-(1-2)</td>
<td>L Plate</td>
<td>6</td>
<td>2</td>
<td>Major axis bending</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Washer Plate</td>
<td>6</td>
<td></td>
<td></td>
</tr>
<tr>
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<td>2</td>
<td>Minor axis bending</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Washer Plate</td>
<td>6</td>
<td></td>
<td></td>
</tr>
<tr>
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<td>BC_Lplate_3_MA-(1-2)</td>
<td>L Plate</td>
<td>3</td>
<td>2</td>
<td>Major axis bending</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Washer Plate</td>
<td>3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>BC_Lplate_3_MI-(1-2)</td>
<td>L Plate</td>
<td>3</td>
<td>2</td>
<td>Minor axis bending</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Washer Plate</td>
<td>3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>BC_Uplate_3_MA-(1-2)</td>
<td>U Plate</td>
<td>3</td>
<td>2</td>
<td>Major axis bending</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Washer Plate</td>
<td>3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>BC_Uplate_3_MI-(1-2)</td>
<td>U Plate</td>
<td>3</td>
<td>2</td>
<td>Minor axis bending</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Washer Plate</td>
<td>3</td>
<td></td>
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</tbody>
</table>

3.5.1. Bending about the column major axis

3.5.1.1. The set-up and testing procedure

A total number of eight major axis bending tests on the column base connections were conducted with four test groups with same bracket-shapes and base metal thickness as shown in Table 3.17 (repeat same configuration twice). Based on the tests, the major flexural
stiffnesses of the base connection used in the full-scale frame tests were evaluated, and the moment-rotation characteristics of various base connections were determined.

In the base connection tests, the back-to-back channel section columns were cut to length size of 1.85 m and connected to the base brackets using M16 stainless steel bolts/nuts/washers. The bolts were tightened using the turn-of-nut method with the same level of pretension in the bolts as implemented in the frame tests. The studied brackets were rigidly fastened to the base plate system using four M24 bolts and respective washer plates. Details of the base plate system are described in Section 3.2.5.3.

A test rig was designed and assembled in the J.W. Roderick Laboratory at the University of Sydney for these major axis bending tests. Full details of the test rig are shown in Figure 3.108 and Figure 3.109. The horizontal load was applied to the top of the cantilevered column at a height of 1.74 m from the base plate by using a 100 kN capacity hydraulic actuator mounted on the supporting column. A 12 mm thick aluminium loading bracket was attached to the column at the loading position between the webs of two back-to-back sections, which served as a spacer plate for the column as well as enabled uniform dispersion of load to the column webs. To avoid any additional restraint due to the connection between the loading bracket and the hydraulic jack, a strut formed by two flat plates and a spherical hinge was used to allow rotation at the connection points.

The displacements and rotations of the base connections were measured by 8 transducers and 4 inclinometers as shown in Figure 3.108. The in-plane displacements of the base were recorded at the heights of 60 mm, 120 mm, 180 mm, 280 mm, and 480 mm from the base plate while four dual Fredericks model 0751-3002 inclinometers were placed on the webs of the column at 180 mm and 280 mm above the base plate. Additional transducers were also placed in various locations to monitor the twist deformation at the top of the column and the lateral displacements at the loading point and 480 mm above the base plate. The deformation of the base plate was also recorded during the duration of the tests by four transducers installed at the corners of the base plate.

The tests commenced with the application of horizontal load through the hydraulic jack with its crosshead controlled to displace horizontally at a low rate of 1 mm/minute. The tests were terminated when the specimen failed.
Figure 3.108. Test setup for major axis bending of the base connection

Figure 3.109. Column base rotation test for bending about the major axis: (a) front view; (b) rear view
3.5.1.2. Test observations

The aim of the experimental program on the base connections with various connection bracket configurations was to evaluate the effect of these brackets on column base stiffness and to determine the capacity of the base connections. The behaviour of the base connections was observed and recorded during the tests. All base connection tests studied in this research demonstrated unique failure modes in each type of brackets.

As the applied load increased, the column started to rotate about the base with deformation largely observed at the base. Full pretension was induced in all bolts by the turn-of-nut method so that the bolted connections, considered herein, are the friction-type connection where loads are initially transferred by friction developed between the clamped plates. In the specimens with 8 mm L-shaped brackets, since the brackets are significantly strong and stiff, there was no observable deformation of the brackets on both tension and compression sides in the initial loading stage. The applied load was initially carried by the friction between the flanges of the column and bracket plates. When the applied load exceeded the friction resistance, the slip between plates occurred. With the gradual increase of the slip, the bottom edges of the column started touching to the inner corner of the bracket in the compression side as shown in Figure 3.110a. Since the gap between the bottom edges of the column and the bracket corner is relatively small, the touching occurred quite early along with the buckling of the column webs. As the load kept increasing, the compression side was subjected to stress concentration and the flanges of the column underwent distortional buckling when the ultimate load was reached. The test kept continuing even after passing the peak load so that post-ultimate behaviour could be obtained. It indicated that stress was redistributed and concentrated at the web-flange junctions, and thus the large localised deformation was observed in this area right at the upper bolt row (Figure 3.110b). When the test was terminated, the column and brackets were disassembled to better observe the deformation of these parts. As seen in Figure 3.110c, although the channels at the base region experienced large deformation, there was no bearing presented in the bolt holes, which could be explained by the small gap between the bottom edges of the column and the bracket corners was insufficient for the development of bearing failure mode. Regarding the brackets, while the bracket in the compression side remarkably bent, the tension side bracket had insignificant deformation. The identical behaviour was presented in the two repeated specimens.
The behaviour of the base connection with 6 mm L-plate bracket was different from that of 8 mm bracket specimens. When the load was gradually applied at the top of the column, the first sign of deformation was observed at the base bracket on the tension side. It gradually bent along the edge of the washer plate due to the pull exerted on the tension side of the connection. On the compression side, the bottom edges of the column did not touch the bracket corner partly because of the use of thinner brackets resulting in a larger gap between the column edges and the bracket corner. However, the column flanges leaning on the bracket leg makes them deform gradually. The specimen failed in tensile fracture of the tensile flanges of the column as shown in Figure 3.111, immediately followed by a sudden drop in loading. This same behaviour was observed in both specimens of 6 mm brackets studied.
A similar response was shown on the base connection tests with 3 mm L-shaped and U-shaped brackets. The first sign of deformation was also observed in the bracket on the tension side of the connection. In these cases, both the washer plate and the brackets bent under the effect of the applied load due to the use of thinner washer plates (see Figure 3.112c and Figure 3.112d). The deformation of brackets and washer plates for both 3mm L-shaped and U-shaped specimens was clearer than that of the base connection with the 6 mm L-shaped brackets. When the deformation of components was large enough, the bottom edges of the column on the compression side began to touch the inner corner of the bracket leading to stress concentration at the edges of the column. Consequently, localised failure occurred in this region after passing the peak load. The failure modes of the 3 mm L-shaped and U-shaped bracket specimens are presented in Figure 3.112a and Figure 3.112b, respectively.

![Figure 3.112a](image1.png) ![Figure 3.112b](image2.png) ![Figure 3.112c](image3.png) ![Figure 3.112d](image4.png)

Figure 3.112. Failure of specimens using brackets with a thickness of 3 mm: (a) specimen BC_Lplate_3_MA-1; (b) specimen BC_Uplate_3_MA-1; (c) 3 mm L-shaped brackets; (d) 3 mm U-shaped bracket
3.5.1.3. Load-deformation responses

The load-deformation responses of the base connections were recorded by instrumentation during the tests. The horizontal load versus horizontal displacement relationship curves at the top of the column are plotted for all base connection configurations with studied base brackets in Figure 3.113 to Figure 3.116. The moment versus rotation relations corresponding to five locations above the base are presented in Figure 3.117 to Figure 3.120 for the respective specimens. In these graphs, the moments were calculated at the points where corresponding in-plane rotations were measured. As seen in these figures, all specimens show a linear response in the initial stage of loading followed by a nonlinear range. However, there is no distinct demarcation between the linear and nonlinear behaviour represented in the graphs.

The variations of the base moments (MB), which were calculated at the base plate of the connection, with different rotations ($\theta_1 \div \theta_5$) at five measured points for all specimens are plotted in Figure 3.121 to Figure 3.124. It can be seen that they also depict comparable moment-rotation graphs. The moment-rotation relations of each specimen are nearly identical in the initial loading stage as the curvature of the column at this stage was small. As larger curvature was developed, its effect became clearer with the difference in the moment-rotation relations between the measured points, especially in the specimens using strong and stiff brackets. Other graphs of minor importance representing other load-deformation responses can be found in Appendix B.3.

![Figure 3.113. Applied load versus horizontal displacement of the column at loading point in major axis bending for 8 mm thick L-plate brackets](image_url)
Figure 3.114. Applied load versus horizontal displacement of the column at loading point in major axis bending for 6 mm thick L-plate brackets

Figure 3.115. Applied load versus horizontal displacement of the column at loading point in major axis bending for 3 mm thick L-plate brackets
Figure 3.116. Applied load versus horizontal displacement of the column at loading point in major axis bending for 3 mm thick U-plate brackets

Figure 3.117. Major axis moment versus rotations above the base: (a) specimen BC_Uplate_3_MA-1; (b) specimen BC_Uplate_3_MA-2
Figure 3.118. Major axis moment versus rotations above the base: (a) specimen BC_Lplate_6_MA-1; (b) specimen BC_Lplate_6_MA-2

Figure 3.119. Major axis moment versus rotations above the base: (a) specimen BC_Lplate_3_MA-1; (b) specimen BC_Lplate_3_MA-2
Figure 3.120. Major axis moment versus rotations above the base: (a) specimen BC_Uplate_3_MA-1; (b) specimen BC_Uplate_3_MA-2

Figure 3.121. Major axis moment (MB) versus rotations above the base: (a) specimen BC_Lplate_8_MA-1; (b) specimen BC_Lplate_8_MA-2
Figure 3.122. Major axis moment (MB) versus rotations above the base: (a) specimen BC_Lplate_6_MA-1; (b) specimen BC_Lplate_6_MA-2

Figure 3.123. Major axis moment (MB) versus rotations above the base: (a) specimen BC_Lplate_3_MA-1; (b) specimen BC_Lplate_3_MA-2
Figure 3.124. Major axis moment (MB) versus rotations above the base: (a) specimen BC_Uplate_3_MA-1; (b) specimen BC_Uplate_3_MA-2

3.5.1.4. Flexural stiffnesses of the base connections in major axis bending

The moment-rotation curves are characterised by an initial linear region followed by a non-linear region. The initial parts of the moment rotation curves can be represented by a linear relation as follows:

\[ M = K_f \theta \]  \hspace{1cm} (3.8)

where

- \( M \) is the applied moment
- \( \theta \) is the corresponding rotations
- \( K_f \) is the elastic flexural stiffness

On the basis of this, the elastic major axis flexural stiffnesses of the base connections were determined from the graphs based on a linear regression analysis by considering moments up to 6 kNm. The major axis flexural stiffness values, \( K_f \), corresponding to five moment-rotation pairs (M1-01, M2-02, M3-03, M4-04, and M5-05) for each specimen are provided in Table 3.18.

As seen in Table 3.18, the values of flexural stiffness corresponding to the five moment-rotation curves are different. Among them, \( K_{f\beta} \) seems to be the best representative of the base stiffness since it was determined on the basis of the reading data at the point located right outside the effect of the connection bracket. Hence, \( K_{f\beta} \) is adopted as the base connection stiffness. The moment-rotation relations for all specimens in major axis bending corresponding to \( K_{f\beta} \) are presented in Figure 3.125.
Table 3.18. Elastic flexural stiffness of base connections in major axis bending

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$K_{f1}$</th>
<th>$K_{f2}$</th>
<th>$K_{f3}$</th>
<th>$K_{f4}$</th>
<th>$K_{f5}$</th>
<th>Bending direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>BC_Lplate_8_MA-1</td>
<td>1223.26</td>
<td>936.38</td>
<td>853.08</td>
<td>809.02</td>
<td>630.37</td>
<td>Major axis bending</td>
</tr>
<tr>
<td>BC_Lplate_8_MA-2</td>
<td>928.42</td>
<td>1050.58</td>
<td>972.31</td>
<td>889.86</td>
<td>671.45</td>
<td></td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>1075.84</strong></td>
<td><strong>993.48</strong></td>
<td><strong>912.69</strong></td>
<td><strong>849.44</strong></td>
<td><strong>650.91</strong></td>
<td></td>
</tr>
<tr>
<td>BC_Lplate_6_MA-1</td>
<td>1029.95</td>
<td>932.78</td>
<td>910.83</td>
<td>851.82</td>
<td>656.67</td>
<td></td>
</tr>
<tr>
<td>BC_Lplate_6_MA-2</td>
<td>877.77</td>
<td>737.74</td>
<td>730.64</td>
<td>727.03</td>
<td>590.72</td>
<td></td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>953.86</strong></td>
<td><strong>835.26</strong></td>
<td><strong>820.73</strong></td>
<td><strong>789.42</strong></td>
<td><strong>623.69</strong></td>
<td></td>
</tr>
<tr>
<td>BC_Lplate_3_MA-1</td>
<td>554.77</td>
<td>513.16</td>
<td>455.90</td>
<td>453.34</td>
<td>356.21</td>
<td></td>
</tr>
<tr>
<td>BC_Lplate_3_MA-2</td>
<td>598.17</td>
<td>536.45</td>
<td>478.67</td>
<td>461.29</td>
<td>358.66</td>
<td></td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>576.47</strong></td>
<td><strong>524.81</strong></td>
<td><strong>467.28</strong></td>
<td><strong>457.32</strong></td>
<td><strong>357.44</strong></td>
<td></td>
</tr>
<tr>
<td>BC_Uplate_3_MA-1</td>
<td>612.84</td>
<td>573.36</td>
<td>509.31</td>
<td>487.40</td>
<td>372.30</td>
<td></td>
</tr>
<tr>
<td>BC_Uplate_3_MA-2</td>
<td>545.24</td>
<td>498.25</td>
<td>499.91</td>
<td>474.38</td>
<td>369.99</td>
<td></td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>579.04</strong></td>
<td><strong>535.80</strong></td>
<td><strong>504.61</strong></td>
<td><strong>480.89</strong></td>
<td><strong>371.15</strong></td>
<td></td>
</tr>
</tbody>
</table>

Figure 3.125. Moment-rotation relations for base connections major axis bending

The test results demonstrated that the base stiffness for major axis bending of the base connection could be improved by increasing the thickness of the L-shaped base brackets. Increasing from the 3 mm thick L-shaped brackets to 6 mm or 8 mm thick L-shaped brackets resulted in massive increases in the column base stiffness of 75.64% and 95.32%, respectively. Likewise, the base stiffness of the 8 mm L-shaped bracket connection is only 11.2% higher than that of the base connection using 6 mm L-shaped brackets. It was also found that the use of a U-shaped bracket is likely to provide a higher stiffness of the base connection as compared to
the L-shaped brackets with the same thickness. As can be seen in Table 3.18, the 3 mm U-shaped bracket has an increase of approximately 8% in the base stiffness as compared to that of the 3 mm L-shaped brackets.

The average initial flexural stiffness obtained from the isolated base connection tests with 8 mm L-shaped brackets is 912.69 kNm/rad, which is 19.16 % higher than the average column base stiffness of 765.96 kNm/rad measured from the full-scale frame tests (Table 3.8). The difference is possibly attributed to the different approaches adopted for the determination of the base moment from which the column base stiffnesses were derived. In the full-scale frame test, the base moment was determined from the reactions of the load cells installed beneath the base plate, whilst in the case of isolated base connection tests, the base moment was directly calculated as the product of horizontal load applied at the top of the column specimen and the vertical distance to the considered section of the base. This is also realised that the variation of the base stiffness among individual values were significant in both full-scale frame tests and component tests with no obvious reasons.

3.5.2. Bending about the column minor axis

3.5.2.1. The set-up and testing procedure

The same test rig as used for the base connection tests about the major axis was utilised to perform the tests on the base connections subjected to bending about the minor axis. For testing arrangements, specimens were rotated 90° as compared to the major axis bending tests. The horizontal load was applied to the top of the column through a loading bracket, which was redesigned for these particular minor axis bending tests. Full details of the test setup along with the arrangement of instruments are illustrated in Figure 3.126.

To monitor the displacements and rotations of the base, transducers and inclinometers were installed at various locations on the column. While the in-plane rotation was recorded by four dual inclinometers placed at 180 mm and 280 mm above the base plate, the in-plane displacements were measured by ten transducers installed on the web near each flange-web junction at the locations of 60 mm, 120 mm, 180 mm, 280 mm, and 480 mm above the base plate. Additional transducers were also used to monitor the displacements and rotations of the column at the loading point and also the deformation of the base plate. A typical minor axis bending test is shown in Figure 3.127.

The tests were performed by applying an incremental horizontal force using the same loading jack operated in the displacement control mode at a constant rate of 1 mm/minute. The specimen was loaded until failure or when the maximum displacement capacity of the loading jack was exhausted. It is noted that the columns in the frame tests did not experience high magnitudes of the horizontal loads as seen in the isolated base connection tests, and thus the loading range was sufficient to cover the loading range of the frame tests even when the ultimate loads were not achieved.
3.5.2.2. Test observations

In the minor axis bending tests, the base connections with L-shaped brackets with the thicknesses of 6 mm and 8 mm were tested until the obtainment of the ultimate load, whilst the specimens composed of 3 mm L-shaped and U-shaped brackets were discontinued when the
maximum displacement capacity of the loading jack was exhausted. The base connection with 6 mm and 8 mm L-shaped brackets exhibited similar behaviour where the failure mode was distortional buckling of the channel in compression side. As the loading progressed, the column began to rotate about the base, and the compressive stresses on the flanges and lips of the columns on the compression side increased. When the compressive stress reached the critical stress, the flanges of the compressive channels failed due to distortional buckling at peak load as shown in Figure 3.128a and Figure 3.128b. As clearly observed, there was no significant deformation developed in the both 6 mm and 8 mm brackets, as opposed to that of the 3 mm L-shaped and U-shaped brackets. In these tests, deformation was mainly observed in the brackets (see Figure 3.128c and Figure 3.128d). After the tests were terminated, the columns and brackets were disassembled to check the deformation of the column. It was shown that no appearance of deformation on the channels or bearing at the bolt holes was observed. The tests were stopped before reaching the peak load, however, it should be noted that the columns in the frame tests never experienced that high magnitude of lateral load due to the extent of large deformation in the base brackets. Hence, this loading range was sufficient to cover the loading range of the full-scale frames.

![Figure 3.128a](image1.png) ![Figure 3.128b](image2.png) ![Figure 3.128c](image3.png) ![Figure 3.128d](image4.png)

**Figure 3.128.** Deformation of specimens in minor axis bending
3.5.2.3. Load-deformation responses

The horizontal load versus horizontal displacement graphs at the top of the column for all specimens are provided in Figure 3.129 to Figure 3.132. These graphs indicate that a consistent response was obtained from each pair of specimens.

![Graph showing load-deformation responses for 8 mm thick L-plate brackets](image1)

**Figure 3.129.** Applied load versus horizontal displacement of the column at loading point in minor axis bending for 8 mm thick L-plate brackets

![Graph showing load-deformation responses for 6 mm thick L-plate brackets](image2)

**Figure 3.130.** Applied load versus horizontal displacement of the column at loading point in minor axis bending for 6 mm thick L-plate brackets
Figure 3.131. Applied load versus horizontal displacement of the column at loading point in minor axis bending for 3 mm thick L-plate brackets

Figure 3.132. Applied load versus horizontal displacement of the column at loading point in minor axis bending for 3 mm thick U-plate brackets

The moment-rotation curves corresponding to five locations above the base plate are provided in Figure 3.133 to Figure 3.136 for the respective connection specimens. While a similar response is shown in the graphs for the base connections with 6 mm and 8 mm brackets, nearly identical behaviour can be seen in the curves of specimens with 3 mm L-shaped and U-shaped brackets. This fact implies that a big change in the thickness of used brackets significantly affects the behaviour of the connection. However, using 3 mm U-shaped brackets instead of 3 mm L-shaped brackets does not noticeably improve the performance of the column base connections in minor axis bending.
Figure 3.133. Minor axis moment versus rotations above the base: (a) specimen BC_Lplate_8_MI-1; (b) specimen BC_Lplate_8_MI-2

Figure 3.134. Minor axis moment versus rotations above the base: (a) specimen BC_Lplate_6_MI-1; (b) specimen BC_Lplate_6_MI-2
Variations of the base moment (MB) with different rotations ($\theta_1 \div \theta_5$) at five measured points are shown in Figure 3.137 to Figure 3.140 for all specimens. As is evident from these graphs, the moment-rotation relations at different points are different.
Figure 3.137. Minor axis moment (MB) versus rotations above the base: (a) specimen BC_Lplate_8_MI-1; (b) specimen BC_Lplate_8_MI-2

Figure 3.138. Minor axis moment (MB) versus rotations above the base: (a) specimen BC_Lplate_6_MI-1; (b) specimen BC_Lplate_6_MI-2
3.5.2.4. Flexural stiffnesses of the base connections in minor axis bending

Similar to the major axis bending test results, the moment-rotation curves obtained from the minor axis bending tests are also characterised by an initial linear region followed by a non-linear region. The minor axis flexural stiffness values, $K_f$, corresponding to five moment-rotation pairs (M1-01, M2-02, M3-03, M4-04, and M5-05) for each specimen are also determined and provided in Table 3.19. As discussed in Section 3.5.1.4, $K_f$ is seen as the best representative of the base stiffness, so $K_f$ will be therefore used to represent the base connection.
stiffness for bending about the minor axis. The moment-rotation relations for all specimens in minor axis bending corresponding to $K_f\beta$ are presented in Figure 3.141.

Table 3.19. Elastic flexural stiffness of base connections in minor axis bending

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$K_f\beta$</th>
<th>$K_f\gamma$</th>
<th>$K_f\beta$</th>
<th>$K_f\gamma$</th>
<th>$K_f\beta$</th>
<th>$K_f\gamma$</th>
<th>Bending direction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>kNm/rad</td>
<td>kNm/rad</td>
<td>kNm/rad</td>
<td>kNm/rad</td>
<td>kNm/rad</td>
<td>kNm/rad</td>
<td></td>
</tr>
<tr>
<td>BC_Lplate_8_MI-1</td>
<td>264.01</td>
<td>313.06</td>
<td>366.33</td>
<td>286.94</td>
<td>213.21</td>
<td></td>
<td>Minor axis bending</td>
</tr>
<tr>
<td>BC_Lplate_8_MI-2</td>
<td>326.71</td>
<td>330.46</td>
<td>375.15</td>
<td>292.12</td>
<td>218.37</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>295.36</strong></td>
<td><strong>321.76</strong></td>
<td><strong>370.74</strong></td>
<td><strong>289.53</strong></td>
<td><strong>215.79</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BC_Lplate_6_MI-1</td>
<td>260.24</td>
<td>307.49</td>
<td>343.71</td>
<td>282.36</td>
<td>206.25</td>
<td></td>
<td>Minor axis bending</td>
</tr>
<tr>
<td>BC_Lplate_6_MI-2</td>
<td>298.61</td>
<td>293.82</td>
<td>339.28</td>
<td>272.20</td>
<td>202.28</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>279.42</strong></td>
<td><strong>300.66</strong></td>
<td><strong>341.49</strong></td>
<td><strong>277.28</strong></td>
<td><strong>204.26</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BC_Lplate_3_MI-1</td>
<td>96.92</td>
<td>101.54</td>
<td>119.62</td>
<td>103.84</td>
<td>80.53</td>
<td></td>
<td>Minor axis bending</td>
</tr>
<tr>
<td>BC_Lplate_3_MI-2</td>
<td>95.25</td>
<td>101.32</td>
<td>116.96</td>
<td>102.03</td>
<td>76.60</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>96.09</strong></td>
<td><strong>101.43</strong></td>
<td><strong>118.29</strong></td>
<td><strong>102.93</strong></td>
<td><strong>78.57</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BC_Uplate_3_MI-1</td>
<td>90.75</td>
<td>105.58</td>
<td>130.94</td>
<td>113.71</td>
<td>84.53</td>
<td></td>
<td>Minor axis bending</td>
</tr>
<tr>
<td>BC_Uplate_3_MI-2</td>
<td>105.40</td>
<td>108.29</td>
<td>135.58</td>
<td>116.57</td>
<td>88.31</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>98.08</strong></td>
<td><strong>106.93</strong></td>
<td><strong>133.26</strong></td>
<td><strong>115.14</strong></td>
<td><strong>86.42</strong></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 3.141. Moment-rotation relations for base connections minor axis bending
As seen from Table 3.18 and Table 3.19 that the minor axis flexural stiffness is generally approximately 40% of the major axis flexural stiffness for 6 mm and 8 mm L-shaped bracket connections and about 25% of the major axis flexural stiffness for 3 mm L-shaped and U-shaped bracket connections. Similar to the bending about the major axis, the minor axis stiffness could be also enhanced by increasing the thickness of the brackets or using the U-shaped brackets instead of the L-shaped brackets. Most notably, the stiffness of the base connection with 8 mm L-shaped brackets is greater than triple that of the base connection using 3 mm L-shaped brackets.

### 3.6. DISCUSSIONS AND CONCLUSIONS

A series of full-scale experiments on cold-rolled aluminium portal frames for various frame configurations and loading conditions along with component tests on the material, point fastener connections, and the base connections have been completed. The test objectives, testing procedures, and results are discussed within each series of tests in this Chapter.

Seven full-scale tests were completed to obtain the behaviour of two-bay single-span CRA portal frames. The prevalent initial geometric imperfections of each portal frame were measured prior to testing. The global and local deformation was measured at various locations on the frame including at the main connections. It was demonstrated that the flexural-torsional buckling of the columns is the dominant structural behaviour in Frame Tests with unbraced columns, whilst Frame Tests with braced columns, in which the girts were provided along the height of the columns, failed by the formation of a spatial plastic mechanism at critical locations. In all tests, there was no sign of failure presented at connections, indicating the adequate design of the connections. It was also revealed that the rafter tie is essential to improve the behaviour of frame systems, whilst sleeve stiffeners have shown an excellent performance in enhancing the load capacity of the frame system. Therefore, it is recommended that the columns should be braced laterally and/or strengthened, where practically possible, to optimise the structural performance.

A total number of 63 tests on coupon specimens were conducted to determine the material properties of the cold-rolled aluminium AC25025 and connection brackets. The material properties derived from the coupon tests indicated that the material used for the connection brackets was slightly stronger and stiffer as compared to those used in the cold-rolled sections. It was also reported that the material properties in the longitudinal direction of the flat portions of the cold-rolled aluminium sections obtained from compressive and tensile coupon tests are almost identical, indicating that the reduction in compressive yield strength is not required. A considerable enhancement at corner due to the cold-working process was revealed from the test results, and thus the favourable effect of corner enhancement should be included in numerical models. Moreover, the material properties of coupons cut from the diagonal and transverse directions of the AC25025 section slightly differed in strength and stiffness from those of
specimens cut from the longitudinal directions, indicating the existence of a certain degree of anisotropy in the cold-rolled aluminium sections.

In the frame tests, the members were connected by using bolted connections, in which the bolts were tightened by the turn-of-nut method. Consequently, the bolted connections were designed as slip-resistance connections. The constitutive relations of the bolted connections are required to idealise the behaviour of individual fasteners in the finite element models of the frames, which are fully described in the following Chapter. Hence, a series of experiments were completed on the point fastener connections in shear and under torque to obtain the fastener load and corresponding deformations.

The isolated column base connection tests were also carried out to quantify the flexural stiffness of the base connections subjected to bending about both major and minor axes. Various connection bracket sizes and shapes were tested, including 3 mm, 6 mm, and 8 mm L-shaped brackets and 3 mm U-shaped bracket. The test results enabled the flexural stiffness and the moment-rotation characteristics of the base connection used in the full-scale frame tests to be evaluated as well as the effect of the connection brackets on column base stiffness to be examined. The tests demonstrated that the average initial flexural stiffness obtained from the isolated base connection tests is relatively higher than the average column base stiffness measured from the full-scale frame tests when using the same brackets. This was possibly attributed to the different approaches adopted for the determination of the base moment from which the column base stiffnesses were derived. Another conclusion was drawn that the change in shape and stiffness of the connection brackets affected the flexural stiffness of the column base connections.
CHAPTER 4. NUMERICAL MODELLING AND ADVANCED ANALYSIS OF COLD-ROLLED ALUMINIUM PORTAL FRAME

4.1. INTRODUCTION

It is well known that the finite element method is a convenient approach for studying the behaviour of structures, allowing a wide range of structural configurations to be investigated economically with time-efficiency. However, to perform a parametric study, reliable and robust numerical models calibrated against laboratory test results of the benchmark structures are required to be developed. In the present context, various aspects for the development of detailed finite element (FE) models are discussed in this Chapter to provide an efficient analysis tool, which is capable of accurately capturing the behaviour and ultimate strength of portal frames made of cold-rolled aluminium (CRA) profiles and suitable to be employed in the design practice by advanced analysis.

The Chapter begins with the description of modelling nonlinear material behaviour based on the engineering stress-strain curves from the coupon tests. Modelling of fasteners to represent the actual individual bolted connection is presented in the next part, in which the nonlinear behaviour of the fasteners was validated using the load-deformation characteristics obtained from the point fastener connection tests. Subsequently, the nonlinear finite element modelling of the base connections is described. The numerical modelling and analysis of frame systems are detailed in the next part, in which the incorporation of contact, boundary conditions, loading, and geometric imperfections is highlighted. The parametric studies are subsequently performed using the calibrated modelling technique to extend the test database and examine the behaviour of the CRA portal frames in greater details.

4.2. MATERIAL MODEL AND PROPERTIES

The accuracy of the material definition is crucial for the numerical finite element simulations. This implies that the relevant measured stress and strain relationship curves should be incorporated properly in the finite element (FE) models. In general, cold-rolled aluminium material exhibits ductile behaviour, and hence it is capable of undergoing large deformation. To simulate such nonlinear material behaviour in the FE models, the classical metal plasticity model, which implements the von-Mises yielding and associated plastic flow theory, was adopted in all analyses using the software package ABAQUS/Standard [12]. In ABAQUS, material properties are modelled ideally with respect of true stress ($\sigma_{\text{true}}$), logarithmic true plastic strain ($\varepsilon_{\text{p, true}}$), elastic modulus ($E$), and Poisson’s ratio ($\nu$). The true stress and logarithmic strain for FE modelling are derived from average engineering stress ($\sigma$) and engineering strain ($\varepsilon$) obtained from the coupon tests as reported in Section 3.3 by the following equations:
\[ \sigma_{\text{true}} = \sigma (1 + \varepsilon) \]  
\[ \varepsilon_{\text{true}} = \ln (1 + \varepsilon) \]  

(4.1)  
(4.2)

The plastic strain (\( \varepsilon_{p,\text{true}} \)) is determined by subtracting the elastic strain (\( \varepsilon_e = \sigma / E \)) from the total strain (\( \varepsilon_{\text{true}} \)), and thus:

\[ \varepsilon_{p,\text{true}} = \ln (1 + \varepsilon) - \frac{\sigma_{\text{true}}}{E} \]  

(4.3)

The material mechanical properties introduced into the FE models vary depending on component types and regions. The tensile properties from the coupon specimens cut from the aluminium brackets were assigned to all brackets used in the assembly of the portal frames. For main members (i.e. rafters, columns, …) using CRA sections, the cross-section of the channel section was divided into flat parts and corner regions as shown in Figure 4.1. The tensile corner properties were assigned to all corner regions in the numerical models, whilst the tensile flat properties in the longitudinal direction were used to assign to all flat parts of the cross-sections. It should be noted that the anisotropy as found in the material properties when conducting the coupon tests in different directions was omitted in the numerical finite element simulations as the differences are negligible, and the stresses in the cross-sections under applied loads were predominantly developed in the longitudinal direction in all frame tests performed in this study. Further, under different load cases, some parts of the cross-sections are in compression, whilst the others are in tension. Therefore, the material properties should be assigned differently to compressive and tensile parts to achieve accurate results. However, as evidently seen in Section 3.3.3, the differences between the longitudinal compressive properties and the longitudinal tensile properties of CRA sections are found insignificant. Consequently, for the purpose of simplicity, only the longitudinal tensile properties of flat parts were assigned to the flat regions in the FE models. Meanwhile, the yield strength and ultimate strength enhancement at the corners due to the cold-forming process are proved to be significant, so these enhancements were taken into accounts in the assignment of material properties into FE models by using the tensile corner properties for corner regions.

Figure 4.1. Material property assignments for cold-rolled aluminium sections
By using Equations (4.1) to (4.3), the true stress-strain curves and the true plastic strain of the aluminium brackets, the longitudinal flat parts and the corner parts of the aluminium channel sections AC25025 were converted from the corresponding measured mean engineering stress-strain curves. These true stress-strain and the corresponding true stress – plastic strain relations up to the ultimate tensile strength were plotted in Figure 4.2 and Figure 4.3, respectively.

In ABAQUS software [12], the plasticity model of material is introduced by two parts. The onset of nonlinear stress-strain behaviour is assumed to be linear and thus Young’s modulus ($E$) and Poisson’s ratio are the required input parameters, whilst in the second part of the input, the material nonlinearity is defined by piecewise linear segments using the true stress – plastic strain relations. For the linear elastic behaviour, the Young’s modulus values provided in Section 3.3 and a Poisson’s ratio of 0.3 was assigned for the respective components of the frames in the FE models. Meanwhile, the true stress – plastic strain relations presented in Figure 4.3 was used for the second part of the nonlinear behaviour.

![Figure 4.2. True stress-strain curves](image-url)
The residual stresses of cold-rolled aluminium material were not explicitly examined in the material modelling in the numerical finite element simulations. For a CRA section, the residual stresses are mainly developed by the influence of the cold-bending during the forming process. Residual stresses are generally categorised as flexural residual stresses (bending residual stresses) and membrane residual stresses. Based on the current research on CRA sections and members reported in [25, 205, 247], it could be stated that membrane residual stresses have an insignificant effect on the strength capacity of CRA alloy members, and are usually ignored in the FE models. Meanwhile, it is implicitly assumed that the effect of flexural residual stresses was automatically introduced in the FE models through the assignments of stress-strain relations obtained from the coupon tests. Because, during the coupon tests, the ends of specimens were clamped tightly by the grips of the testing machine, and thus the coupons were straightened and almost returned to their flat state. As a result, the flexural residual stresses were closely reintroduced again in the coupons during gripping [244].

4.4. FASTENER MODELLING

4.4.1. Modelling of bolted connections

In the experimental investigation of the CRA portal frames, bolted connections were adopted to assemble components into finished complete portal frame structures. The main members (columns, rafters, knee braces and rater ties) with the back-to-back cross-sections were connected to their respective brackets using M16 stainless steel 316 bolts. The central frame (i.e. testing frame) was connected to two parallel end frames with purlins spanning between the rafters of the portal frames to create a two-bay frame system. M12 stainless steel
316 bolts were used for connections between purlins and purlin cleats, which were finally attached to rafters.

Several studies in the past were conducted to determine appropriate modelings for bolt connectors in the FE models. Zhang et al. [159] did not model bolt holes and bolts explicitly. Alternatively, the semi-rigid behaviour of joints was modelled by incorporating rational springs at the intersection of the column and rafter centerlines in a shell element model. A simplified bolt model that composed of two perpendicular linear spring was developed and validated with experimental results by Lim and Nethercot [123, 217] to model the bearing behaviour of a single bolt. However, these spring elements are not suitable if nonlinearities in the bearing behaviour of bolts are taken into account. A more direct approach to model the behaviour of bolts was adopted in research studies [133, 248, 249] where solid elements and surface-to-surface contact interactions were used to model bolt behaviour in ABAQUS. However, by using this method, the model becomes more complex, and thus reduces the computational efficiency, especially in models with a large number of bolts like portal frames, and convergence problem may therefore occur in the presence of bolt rigid body movement and slippage. In lieu of directly modelling bolts and bolt holes, rigid connector elements were applied at the locations of the bolts [134, 250, 251] with the assumption that bolts were sufficiently tight and there was no bolt slip. To represent the effects of bolt hole elongation, Jun Ye et al. [169] employed a “Cartesian” connector which was characterised by a parallel combination of “rigid elasticity” and “plasticity” behaviours using the experimental load-deformation relations of a bolt slip and bearing against a steel plate. Similarly, deformable mesh-independent point-based fasteners were employed by Rinchen and Rasmussen [160] to idealise the connections, in which the behaviour of slip-resistant bolts/screws was modelled by using the combination of “Cartesian” and “Cardan” sections, which not only represents translational behaviour in three local cartesian directions but also provides a rotational connection between two nodes.

On the basis of the above discussion, in this study, bolt connectors were modelled by using deformable mesh-independent point-based fasteners (Figure 4.4), which can be located anywhere between surfaces and can connect multiple layers [12]. It is anticipated that this bolt model can provide accurate results with efficiently computational costs and fewer convergence problems. The fastener’s positioning points were specified by using the attachment points created at the location of bolts. The required input parameters for each point-based fastener need to be defined including size (influence radius), direction vector, mass, orientation, coupling type, and the number of surfaces to be connected. Particularly, for the formulation of fasteners, structural distributing was chosen, which couples the displacements and rotations of the fastening point to the average displacement and rotation of the nearby nodes lying within the area covered by the influence radius.
The deformable behaviour of fasteners can be specified by using connector properties that were appropriately defined for the stipulated connection type. In this study, the combination of “Cartesian” and “Cardan” sections was employed to represent the behaviour of M16 slip-resistant bolts used in the connections between the main members and brackets, and the back-to-back channel connections, whilst the “Cartesian” section was utilised to model the behaviour of M12 bolts used in the purlin connections. The diagrams of the “Cartesian” section and the “Cardan” section are presented in Figure 4.5a and Figure 4.5b, respectively. While the connection type “Cartesian” provides a connection between two nodes, allowing independent translational behaviour in three local cartesian directions, the connection type “Cardan” provides a rotational connection between two nodes parametrized by Cardan angles [12]. The connector behaviour definition includes the effects of elasticity and plasticity. The initial linear elastic part of the bolt force-displacement curve was extracted and assigned to the connector elasticity, and the nonlinear force-displacement relation, which represented the bolt slip and ply bearing behaviour, was employed to define the connector plasticity. Further, “uncoupled” was assumed as the coupling definition in plasticity behaviour. The force-displacement relations of the fastener connections used to simulate connector sections will be described in the following section.
4.4.2. Validation of the point fastener properties

The results of the point fastener connection tests, which include the nonlinear relations between the fastener force and the corresponding deformations of the individual fastener, were reported in Section 3.4. The average force-displacement relationship of the fastener connections in each category of the test is presented by the average value curve. To achieve lower computational costs, multi-linear segments were attempted for further simplified force-displacement relations of the fastener. The experimental and simplified curves for M16 bolted connections in shear and under torque are plotted in Figure 4.6 and Figure 4.7 for a single plate fastener and a double plate fastener, respectively, whereas these curves of M12 bolted connections in shear are shown in Figure 4.8.
Figure 4.7. Constitutive relations of double plate M16 bolted connections: (a) in shear; (b) under torque

In the FE models, each point-based fastener simulated in FE models was assigned to the relevant connector properties. The input data of elasticity and plasticity behaviours of connectors were extracted from respective Figure 4.6 to Figure 4.8. A local coordinate system, as shown in Figure 4.9, was adopted for the connector behaviour definition to define the relevant Degree of freedoms (DoFs) for appropriate assignment of force-displacement data. In this coordinate system, the slippage of two plates as shown in Figure 4.9 tends to be induced by force components 1 and 2 acting in the local x- and y-axes, respectively, and moment component 6 about the z-axis. Therefore, for both single plate and double plate M16 bolted connections, the plasticity behaviour was only assigned to the force components corresponding to the x- and y-axes and moment components about the z-axis, whilst the other DoFs were assumed to remain elastic. This same procedure was also applied for M12 bolted connections.
at purlins. However, only force components were considered, whereas the moment components were assumed to be negligible.

On the basis of these discussions, to examine the performance of the deformable fastener assigned to connector sections in the numerical models, single plate M16 bolted connection tests in shear were simulated and analysed in ABAQUS. The shell element model of the connection was developed by assembling two plates: the top plate of 2.5 mm thickness representing the CRA members, and the bottom plate of 6 mm thickness using for aluminium brackets, as shown in Figure 4.10a. The overall configuration of the tested connection along with detailed dimensions can be found in Figure 3.101. S4R shell elements with the mesh size of 10 x 10 mm were assigned to the plates. Two bolts were modelled using deformable point-based fasteners as discussed earlier. For connector section definitions, both the actual and simplified connection behaviours as shown in Figure 4.6 were considered. In the experiments, the upper portion of the 2.5 mm plate was clamped firmly, whilst the lower part of the 6 mm plate was pulled down by the controlled hydraulic actuator. Therefore, to simulate these boundary conditions, the top end of the upper part was fixed while the bottom end of the lower part was allowed to displace vertically downward but restrained in all other DoFs. A displacement controlled analysis was performed. The numerical results showing the applied forces versus relative displacements of the connection were plotted along with the input behaviour of the fastener in Figure 4.11.

Figure 4.9. The local coordinate system used to define connector sections
Figure 4.10. Finite element model of single plate M16 bolted connections in shear

![Finite element model of single plate M16 bolted connections in shear](image)

Figure 4.11. Point fastener behaviour: (a) Real behaviour; (b) Simplified behaviour

As clearly evident from Figure 4.11, the behaviour of fasteners obtained by numerical simulations is an accurate reflection of the actual behaviour of bolts, which can be adopted by using deformable point-based fasteners instead of the physical modelling of bolts in FE models. In addition, the CPU time of the simulation for connector sections with the actual behaviour of bolts was 1293.8 s, whereas, by using the simplified curve to define connector sections, the total CPU time was 46.3 s (all simulations were performed using a 3.41 GHz Intel Core i7-6700 CPU desktop computer with 16GB RAM). The use of simplified data has a negligible effect on the response of the connection, but can reduce significantly the computational costs. Therefore, the simplified properties represented the behaviour of bolts are adopted in the subsequent FE models to facilitate computational efficiency.
4.5. FINITE ELEMENT MODELLING OF THE BASE CONNECTIONS

4.5.1. Numerical model

In the full-scale frame tests, the columns were connected to the strong floor through the base connections by bolting the flanges of the column to 8 mm thick L-shaped stainless steel brackets using four bolts each. The brackets were firmly bolted to the top base plate using four M24 8.8/TF bolts. Based on data obtained from the tests, the semi-rigidity of the base connection was determined and provided in previous Section 3.2.10.9.2. The results demonstrated that there is substantial dispersion in the base stiffness obtained from the full-scale frame tests although all frames had the same base connection geometry and used the same method to fasten the connection components. Therefore, a series of tests on the isolated column base connections were also completed to evaluate the flexural stiffness of the connection used in the full-scale frame tests for bending about the two principal axes. Besides, the effect of the thickness and the shape of the connection brackets on the column base stiffness was also examined. The details of base connection tests with various base brackets are described in Section 3.5. To simulate the behaviour of the base connections, finite element models were created, including all components likely to affect the connection behaviour.

The FE models for both major axis and minor axis bending tests were generated with the same geometry except for the configuration of loading. The individual parts of the column including base brackets, washer plates, base plate, and loading bracket were created separately using relevant material properties. The dimensions of all components can be found in Appendix A.1. All parts were then assembled to generate a completed model. Figure 4.12 shows complete models for both the major axis and minor axis bending tests.

In the FE model for major axis bending tests, a custom aluminium loading bracket with a thickness of 12 mm was inserted between the two webs of the back-to-back section at the loading position (Figure 4.12a). In the test, the loading bracket also served as a spacer plate for the column as well as enabled uniform dispersion of load to the column webs. In a similar manner, a T-shaped loading bracket was specially designed for the minor axis bending tests. This bracket was attached directly to the webs of the back-to-back cross-section as seen in Figure 4.12b. The beam MPC constraints were created on these loading brackets with the control reference points defined at the point of applied load to distribute the loads to the neighbourhood nodes.
The columns, base connection brackets, washer plates, base plate, and loading brackets were modelled using S4R shell elements with a mesh size of 10 mm x 10 mm. The bolts used in the connections between the flanges of the column and the base brackets, the base brackets and the base plate, the webs of the column and the loading brackets, and in the packing connection to form back-to-back section were modelled by using deformable mesh-independent point-based fasteners. Full details of fastener modelling can be found in Section 4.4.

For contact modelling, contact formulation was also defined between surfaces. While “hard” was assigned to the normal contact, the tangential contact was represented by a slip coefficient of 1.05 [252] for aluminium-aluminium contact or 0.4 [253] for aluminium-stainless steel contact with penalty friction formulation.

Boundary conditions were applied to the bottom face of the base plate. A static nonlinear analysis using “static general” was performed for each configuration of the test with a displacement controlled method.

### 4.5.2. Results

Relationship curves between the moment versus rotation relations obtained from the numerical modelling and respective experimental curves for both bending about the major and minor axes for all types of brackets are plotted in Figure 4.13 to Figure 4.16. Based on these graphs, the flexural stiffness of the respective base connections was then determined and is provided in Table 4.1.

It can be seen that the moment-rotation characteristics predicted by the numerical FE models were in good agreements with those of the base connection tests especially for the initial stage of loading in which the elastic flexural stiffness to be determined. Consequently, the
predictions of the base stiffness, as seen in Table 4.1, agree very well with the experimental results. For the connections in major axis bending, the average test-to-numerical stiffness ratio is 1.004 and the corresponding coefficient of variation is 0.057, whilst the average test-to-numerical stiffness ratio and the corresponding coefficient of variation for the minor axis bending are 0.922 and 0.061, respectively. Hence, this reliable modelling technique for the base connections is adopted for the modelling of the full-scale frames and parametric studies in the following Sections.

Figure 4.13. Moment versus rotation of the base connection with 8 mm L-shaped brackets: (a) Major axis bending; (b) Minor axis bending

Figure 4.14. Moment versus rotation of the base connection with 6 mm L-shaped brackets: (a) Major axis bending; (b) Minor axis bending
Figure 4.15. Moment versus rotation of the base connection with 3 mm L-shaped brackets: (a) Major axis bending; (b) Minor axis bending

Figure 4.16. Moment versus rotation of the base connection with 3 mm U-shaped brackets: (a) Major axis bending; (b) Minor axis bending
Table 4.1: The flexural stiffness of the base connections obtained from numerical modelling

<table>
<thead>
<tr>
<th>Base condition</th>
<th>Flexural stiffness (kNm/rad)</th>
<th>Bending direction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Numerical</td>
<td>Experiment</td>
</tr>
<tr>
<td>8 mm L-shaped Brackets</td>
<td>845.86</td>
<td>912.69</td>
</tr>
<tr>
<td>6 mm L-shaped Brackets</td>
<td>805.29</td>
<td>820.73</td>
</tr>
<tr>
<td>3 mm U-shaped Bracket</td>
<td>529.22</td>
<td>504.61</td>
</tr>
<tr>
<td>3 mm L-shaped Brackets</td>
<td>484.10</td>
<td>467.28</td>
</tr>
<tr>
<td><strong>MEAN</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>COV</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8 mm L-shaped Brackets</td>
<td>414.89</td>
<td>370.74</td>
</tr>
<tr>
<td>6 mm L-shaped Brackets</td>
<td>389.10</td>
<td>341.49</td>
</tr>
<tr>
<td>3 mm U-shaped Bracket</td>
<td>146.26</td>
<td>133.26</td>
</tr>
<tr>
<td>3 mm L-shaped Brackets</td>
<td>117.81</td>
<td>118.29</td>
</tr>
<tr>
<td><strong>MEAN</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>COV</strong></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
4.6. FINITE ELEMENT MODELLING OF COLD-ROLLED ALUMINIUM PORTAL FRAMES

4.6.1. Geometrical modelling of frames

Full three-dimensional (3D) frame models were created in ABAQUS. Basically, the modelled frames have the same geometry and configurations as those in the test frames, as fully demonstrated in Chapter 3. Various components including columns, rafters, knee braces, rafter ties, apex brackets, eaves brackets, purlins, purlin brackets, and base brackets were generated in sequence and assigned to the relevant material properties (Section 4.2). These components were then assembled together to create the complete structure, which consisted of three parallel portal frames connected by purlins to generate a single-span two-bay structure. The overall dimensions of the test frames can be found in Figure 3.2 and Figure 3.3.

Deformable shell extrusion approach in the “Part Module” was used to create the frame components. The decision for using shell elements will be explained in the subsequent section. By using this method, the cross-sectional profile of each component was required to be defined based on the measured dimensions, followed by extruding the cross-section in the longitudinal direction to a specified length. Besides, the use of extrusion approach was also required in the combination with the “cut extrude” feature to form the custom shaped components. The cross-sectional dimensions of the components were randomly measured to verify the nominal dimensions prior to conducting the tests. The measured dimensions include the thickness of all components and the depth of webs, the width of flanges, and the length of lips of channel sections (Figure 4.17), whilst nominal dimensions were adopted for the others as the differences between the measured and the nominal values were negligible. The dimensions of various component sections adopted in FE models are given in Table 4.2.

Table 4.2. Measured dimensions of cross-sections adopted in ABAQUS

<table>
<thead>
<tr>
<th>Structural components</th>
<th>Designation</th>
<th>t (mm)</th>
<th>D (mm)</th>
<th>B (mm)</th>
<th>L (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Columns and Rafters</td>
<td>AC25025</td>
<td>2.44</td>
<td>257</td>
<td>77.75</td>
<td>23.30</td>
</tr>
<tr>
<td>Knee Braces</td>
<td>AC15025</td>
<td>2.44</td>
<td>155</td>
<td>62</td>
<td>23</td>
</tr>
<tr>
<td>Rafter Ties</td>
<td>AC20025</td>
<td>2.44</td>
<td>205</td>
<td>76</td>
<td>25</td>
</tr>
<tr>
<td>Purlins</td>
<td>AC15025</td>
<td>2.44</td>
<td>155</td>
<td>62</td>
<td>23</td>
</tr>
<tr>
<td>Aluminium Brackets</td>
<td>-</td>
<td>5.82</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
When a part is created, it exists in its local coordinate system and independent of other parts in the model [12]. By using the “Assembly Module”, the instances of individual parts were imported and positioned relative to each other in the global coordinate system of the structure by applying translations and rotations to the desired location, thus creating the assembly of the complete structures. A typical full 3D FE model of aluminium portal frames with double channel sections is presented in Figure 4.18.

Figure 4.17. Channel section geometry

Figure 4.18. Finite element model of a test frame
In each frame, the columns, rafters, knee braces, and rafter tie are doubly symmetric sections made up of two channels fastened by M16 bolts through the webs to create the back-to-back profiles. These members were connected to the respective back-to-back brackets also using M16 bolts. The outer frames provided supports to the purlins, which served as discrete lateral restraints on the central frames. M12 bolts were used to connect the purlins to purlin cleats, which were then attached to the rafters. To simulate the behaviour of the bolted connections, the bolts were modelled by using deformable mesh-independent point-based fasteners, as described in Section 4.4. They were located in the FE models to coincide with the physical positions of bolts. Properties of fasteners were defined using the connector properties based on the relevant load-displacement relations as provided in Figure 4.6 to Figure 4.8. Figure 4.19 shows a typical FE model of bolts at the apex region in the FE models.

![Figure 4.19. Deformable point-based fastener in apex connection](image)

### 4.6.2. Contact modelling

In the real tests, individual components may come into contact with others during deformations. Therefore, in the numerical simulation, the implementation of contact properties is required to simulate the interactions between components, which need to be explicitly defined in ABAQUS. The software ABAQUS offers two approaches for modelling contact interactions by using surfaces or contact elements [12]. While the contact between components at the bolted connection points was implicitly introduced by using the deformable fasteners assigned to the nonlinear force-slip deformation behaviour of bolts, the surface-based contact modelling was adopted to simulate the actual contact behaviour between surfaces in the frames. The selection of surface-based contact was examined by considering the geometric nature, mechanical behaviour of the frames and the capabilities of the surface-based contact simulation.

Among two available methods for modelling surface-based contact (“general contact” and “contact pair”), the “contact pair” option with the “master-slave” algorithm was chosen to describe contact between two surfaces, in which one surface was assigned as master surface
and the other was assigned as slave surface. The surface-to-surface discretization with finite-sliding formulation was used to implement contact constraints between the contact pairs. This tracking approach allows for arbitrary relative separation, sliding, and rotation of the contacting surfaces. Appropriate mechanical contact properties were defined and assigned to the contact pairs. They include a constitutive model for the contact pressure-overclosure relationship that represents the contact behaviour of the relevant surfaces normal to the surfaces and a friction model that defines the force resisting the relative tangential motion of the surfaces [12]. For the contact normal direction, “hard” contact was defined, which allows no penetration between two surfaces and provides an allowance for surface separation after contact. Meanwhile, “Penalty” friction formulation was employed for tangential behaviour. The coefficient of friction between two aluminium surfaces was taken as 1.05 [252].

In this study, the contact behaviour was modelled for the central frames only to reduce the computational cost as the deformation of the other two end frames was small and it is unlikely to generate any contact interaction between its components.

4.6.3. Boundary conditions

Boundary conditions play a crucial role in the structural response, and thus they should be modelled as closely as possible to the actual conditions in practice. The boundary conditions including the column base fixity, the lateral bracing at an end frame, and the cross-bracing systems were simulated to represent the actual supports in the experiments.

In the experiments, the columns were connected to the strong floor by using 8 mm thick stainless steel L-plates as shown in Figure 4.20a and Figure 4.20b. Eight M16 bolts were used to fasten the flanges of each column with two L-plates bolted directly to a 25mm thick base plate using two 6mm stainless steel plates serving as washers and four M24 bolts. The base plates were either welded to a base clamp plate (end frames) (see Figure 4.20b) or supported by a load cell system, which was also attached to a thick base clamp plate (test frames) (see Figure 4.20a). The clamp plates were firmly clamped to the I-beam embedded in the concrete strong floor. Full details of the base connections were described in Section 3.2.5.3. The column base is a semi-rigid connection as it is able to resist moment; however, it also rotates under applied loads.
To simulate the semi-rigid boundary condition, the base brackets, the washer plates, and the base plates were modelled using shell elements as presented in Figure 4.20c. The clamp plates and load cell systems were not reproduced in the FE models. M16 bolts used to connect the columns to base brackets were idealised using point-based fasteners assigned to relevant connector sections, whilst the four M24 bolts were simulated by point-based fasteners with rigid MPC section. “ENCASTRE” boundary conditions restraining all the six DoFs were assigned to the lower face of the 25 mm thick base plate to generate a fixity at the base plates. The “Encastre” condition was chosen because there was a negligibly small displacement of the base plate observed and measured under applied load in both the frame tests and the base connection tests. To consider the bending effects, 31 integration points were used through the thickness of the base plate [160]. Contact was also modelled at the base region to simulate the interaction between components of the base connection. Surface-to-surface contact pairs were defined and assigned to the surface pairs including base plate and base bracket, base bracket and washer plate, and base bracket and column.
Figure 4.21. Lateral restraint of the frames: (a) actual restraint; (b) modelling

Figure 4.21a shows the lateral bracing system used to provide lateral restraints for the outer end frame (the end frame on grid A of Figure 3.2). This system not only helped to keep the outer end frame standing in the installation stage but also restrained the end frame against out-of-plane translation at the bottom of the apex bracket during the tests. To represent the lateral restraint condition, a set of nodes, which coincide with the position of the connection between the apex bracket and the bracing bracket, were created. The node-set was restrained laterally (z-direction in the global coordinate system as shown in Figure 4.18) but allowed to displace in all other DoFs as presented in Figure 4.21b.

The frame system was also provided with cross-bracing formed by 16 mm diameter threaded rods of steel as described in Chapter 3. The steel rods were connected to the column base and the top of the column by pinned connections. By considering the behaviour of the actual system, axial connector elements were chosen to model the cross-bracing. These elements can be seen in Figure 4.18.

4.6.4. Loading

In the laboratory, the portal frames were tested by incrementally applying vertical load, which was generated by using a 250 kN capacity hydraulic actuator with a stroke length of 260 mm operated in the displacement control mode. The load spreading system composed of a series of hollow steel beams and steel straps arranged in three levels. The top straps were connected to the loading brackets that were attached to the purlins and purlin cleats (purlin-to-rafter L-plates) symmetrically on either side of the central frame. Further details about the loading arrangement can be found in Section 3.2.7. Under the incremental displacement of the stroke, loads were transferred into the webs of the rafters of the test frames through the purlin cleats. Thus, applied vertical loads could be considered as the concentrated loads applied to the purlin cleats. Therefore, in the FE models, instead of modelling the load spreading system, load conditions can be simulated by vertical loads acting at the purlin cleats. This was achieved by defining multipoint constraints with reference points acting as control points on the purlin cleats, and vertical loads were applied at these coupling constraint reference points. This approach was adopted successfully by Rinchen et al. [160] and Blum et al. [134] in previous
studies also at the University of Sydney. However, by applying this method, only load control analysis can be used, whilst the displacement control, which may help to solve the convergence issues, cannot be employed.

Therefore in this study, the load spreading system was also modelled in the numerical FE simulations to generate the incremental gravity loading as seen in Figure 4.22. While the HSB and straps were modelled using shell elements, the connections of the load system were modelled as pinned connections for simplicity. With the use of this spreading system, the analysis can be done by either displacement control or load control, and it also reflects closer to the actual load transferring conditions.

Figure 4.22. Load spreading system modelling

In Frame Test 3 and Frame Test 5 of seven full-scale tests in the experimental program, in addition to the vertical loads, a horizontal load of 7.5 kN representing wind loads was also applied at the north eave of the portal frames. The horizontal load transferred to the test frame through the custom loading bracket rigidly bolted to the north eave brackets (Figure 4.23a). In the numerical models, in lieu of modelling the loading bracket, a multipoint constraint type beam (a beam MPC) was created on the north eave brackets with the control reference point defined exactly at the connecting point between the steel cable and the loading bracket as shown in Figure 4.23b. When the horizontal load was applied to the reference point, it was distributed to the coupling constraint nodes on the eave brackets.
4.6.5. Incorporation of initial geometric imperfections into models

It is widely acknowledged that the geometric imperfections have a profound influence on the behaviour and strength of thin-walled structures including CRA portal frames in this study. Therefore, the initial geometric imperfections need to be accurately incorporated into the numerical models to simulate the actual behaviour and strength of the frames. In the portal frames, the geometric imperfections can be typically categorised in sectional imperfections, member imperfections and frame imperfections.

Typically, sectional imperfections refer to the type of deviations in cross-sectional shape where component plates of the cross-section displace or rotate leading to the local buckling shaped and/or distortional buckling shaped imperfections as shown in Figure 4.24.

![Figure 4.24. Sectional imperfections: (a) Distortional imperfection; (b) Local imperfection](image)

Member imperfections consist of the type of deviation where the whole cross-section displaces or rotates without distortion. Figure 4.25 shows the member imperfections of a double channel section used in the aluminium portal frames. The camber and bow imperfections also namely as global out-of-straightness imperfections are considered as the strong axis flexure and weak axis flexure shaped imperfections, whilst torsional buckling shape is termed as twist imperfection.

![Figure 4.23. Horizontal load: (a) Custom bracket in test frame; (b) horizontal load modelling](image)
Frame imperfections are the type of deviation from the perfect geometry of the frames. Typically, frame imperfections are associated with sway shaped imperfections, and are defined by an out-of-plumb angle (Figure 4.26).

Geometric imperfections are usually incorporated in the numerical simulation by introducing perturbations in the geometry. For this purpose, in general, three main approaches may be employed to define imperfections in a FE model: (1) as a linear superposition of buckling eigenmodes obtained from a previous buckling analysis, (2) scaling of the deformed shape of the structure obtained from a previous static analysis, or (3) explicit modelling of initial geometric imperfections [12]. Although the two first methods present the ease of numerical implementation, they provide a less accurate representation of the imperfections as compared to the last method where the actual measurements of the imperfections were introduced into the FE models. Therefore, in this study, directly introducing initial geometric imperfections was adopted by replacing the positions of nodes from the perfect geometry by the desired imperfect geometry obtained from imperfection measurements. The geometric imperfections are the sum of sectional, member and frame imperfections. The geometry definition of frames in ABAQUS was processed in two steps: in the “Part” module/level, each part was defined in its own coordinate system and the parts were positioned relative to each other in a global coordinate system using the “Assembly” module/level. This indicates that sectional and member imperfections cannot be introduced at the same stage with the frame imperfections. The
sectional and member imperfections are incorporated in the “Part” level, whilst the frame imperfections (out-of-plumb) are applied in the “Assembly” level. The procedure employed to introduce geometric imperfections into the ABAQUS models is described in the following sections and summarised in detail in Figure 4.27.

![Figure 4.27](image)

**Figure 4.27. The procedure to incorporate geometric imperfections into ABAQUS models**

### 4.6.5.1. Sectional and member imperfections

The imperfections of the columns and rafters were measured in a longitudinal direction at 22 points around the cross-sections, as shown in Figure 3.16, before testing. The measured raw data were decomposed into its respective Fourier series, which are more convenient to use and represent. Two end cross-sections of each member named as A and B were also sketched on papers to account for those non-perfect cross-sections. The details of the imperfection measurements and data processing were provided in Section 3.2.6.2.

In terms of ABAQUS input, when a part has been meshed, its geometry is represented by the coordinates of the mesh nodes. The original coordinates of the nodes represent the perfect geometry of the member. The Fourier expressions of each member (column or rafter) were used to determine the nodal imperfections at the measurement points, whilst imperfections of other
nodes were then calculated by the interpolation algorithm, as fully detailed in Appendix C.1. The measured imperfection incorporation was achieved by manipulating the nodal coordinates of the perfect member with the imperfections data of the nodes.

For that purpose, a Matlab program was developed based on the techniques outlined in Appendix C.1 to incorporate the sectional and member imperfections into the FE models. The Matlab script is also provided in Appendix C.1.

According to the adopted methodology, it is noted that instead of making a clear distinction between the sectional imperfections and the member imperfections, these imperfections were implemented simultaneously in the ABAQUS models. It is also worth mentioning that the member imperfections were implicitly assumed to be unchanged during the frame installation. The reasons for the assumption are that: (1). It is difficult to accurately measure the member imperfections due to the installation process or there is insufficient measurement data of the member imperfections as it is usually only a few points can be measured after the frame completed, and (2). The self-weight of the aluminium rafters is small (the total weight of the rafters, brackets, rafter tie measured in Frame Test 1 is approximately 166 kg), as a result, the bending or twisting of the columns due to the self-weight of the rafters are negligible. Therefore, ignoring the change of member imperfections due to the erection process is acceptable.

4.6.5.2. Frame imperfections

As described in previous sections, frame imperfections or global imperfections consist of the deviation from the nominal geometry of the overall system. They are typically associated with sway shaped imperfections and defined by the out-of-plumb angles. Before performing experiments, the measurements of the global imperfections were carried out. The details of the measurements are provided in Section 3.2.6.3, and the values of out-of-plumbnesses of the columns are given in Table 3.4.

In the numerical finite element models, the instances are assembled together by translating and rotating them to the required locations in the “Assembly” module. As a result, the part position is characterised by a translation specified by a translation vector and/or a rotation specified by two points, defining a rotation axis, plus a right-handed angular rotation around that rotation axis relative to the global origin [12]. In terms of ABAQUS input, the position of each part is defined by two vectors. The first vector contains three values, representing the coordinates of the part’s reference point in the global coordinate system, whilst the second vector contains seven values, corresponding to the coordinates of two points in the global coordinate system and a resultant rotation of the part about an axis defined by the two points.

When the out-of-plumb imperfections were introduced into the FE models, the vertical parts (the columns) were rotated about the axis normal to the frame plan by an angle equal to
the relevant value of the out-of-plumb angles, whereas the horizontal parts were assumed to be translated horizontally by the product of the vertical distance from the part’s reference point to the base plate and the mean of the out-of-plumb angle of the vertical members.

A Matlab program was developed based on the above concepts and techniques to introduce the frame imperfections and generate the new ABAQUS input file in which all sectional, member and frame imperfections are incorporated. The program contains a function namely AxisAngle2.m, which was used to transform the resultant rotations of a part, which include rotations to its normal location plus the imperfection angle, to an axis-angle notation. The Matlab script is provided in Appendix C.1.

4.6.6. Element types and mesh density

4.6.6.1. Element type

In the numerical finite element simulation of the cold-rolled aluminium portal frames, shell elements were chosen to model all main frame members. It is well known that shell elements are the most suitable element type for thin-walled structures as the thickness of the section is significantly small as compared to other dimensions. In addition, the use of solid (continuum) elements can be computationally more intensive due to the larger number of elements. This is because of the need to mesh elements through the thickness to capture all bending and stiffness effects to generate an accurate result. Meanwhile, by using shell elements, the bending effects through the thickness can be simply accounted for if a number of integration points are sufficiently considered. Besides, it may be difficult to obtain the nodal rotations directly from the solid model since there are only three translational degree-of-freedoms at each node of solid elements.

Based on the information provided in the ABAQUS manual [12] and the specific demands of the modelling of the CRA portal frames, of the various types of shell elements available in the ABAQUS library, the 4-noded doubly curves general-purpose shell element with linear interpolation, reduced integration and hourglass control (S4R element) was selected to simulate the prismatic column and rafter sections as well as the connection brackets.

4.6.6.2. Mesh density

A mesh convergence study was performed to determine an appropriate mesh size for numerical models and also to provide an accurate solution with a low computational cost. For that purpose, a full 3D frame model with nominal dimensions was created in ABAQUS. Initial geometric imperfections were not introduced into the model, but the measured material properties and contact were still modelled. Since the two end frames have no primary function apart from providing supports for the purlins, the mesh sizes assigned to their components were kept unchanged during analysis. The use of such the model ensures that the convergence study is performed in a simple and efficient manner with reasonable accuracy.
Five different mesh configurations with the element sizes of 30 x 30 mm, 20 x 20 mm, 15 x 15 mm, 10 x 10 mm, and 8 x 8 mm were considered, respectively. The analyses were completed and the results of the mesh convergence study are given in Table 4.3 and Figure 4.28, in which the CPU time was used to represent the computational efficiency of each mesh configuration. As clearly evident in Table 4.3 and Figure 4.28, the ultimate vertical load converged asymptotically as the mesh size decrease. Once the mesh was refined below 15 x 15 mm, there is a negligible improvement in the solution while computational efficiency dramatically decreases. For instance, the use of the 10 x 10 mm mesh size results in an increase of 74% in the computational costs without any noticeable increase in the accuracy of the result (the difference was around 0.85%) as compared to the use of 15 x 15 mm mesh size. The load versus apex vertical displacement relationship curves are also plotted in Figure 4.29 for the different meshes. It can be seen that there is no observable difference in the structural response of the 15 x 15 mm, 10 x 10 mm, and 8 x 8 mm.

Table 4.3. The influence of the Mesh size

<table>
<thead>
<tr>
<th>Mesh</th>
<th>Element size (mm)</th>
<th>Number of elements*</th>
<th>Ultimate Load (kN)</th>
<th>CPU Time (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>30 x 30</td>
<td>61872</td>
<td>38.66</td>
<td>154479</td>
</tr>
<tr>
<td>2</td>
<td>20 x 20</td>
<td>103554</td>
<td>36.92</td>
<td>212834</td>
</tr>
<tr>
<td>3</td>
<td>15 x 15</td>
<td>194638</td>
<td>35.31</td>
<td>366646</td>
</tr>
<tr>
<td>4</td>
<td>10 x 10</td>
<td>374002</td>
<td>35.01</td>
<td>638067</td>
</tr>
<tr>
<td>5</td>
<td>8 x 8</td>
<td>546000</td>
<td>34.90</td>
<td>1174800</td>
</tr>
</tbody>
</table>

* Note: The number of elements was counted for the central frame only

Figure 4.28. Mesh convergence check
On the basis of the mesh convergence study, the mesh sizes were adopted and chosen for modelling the frame components as follows: For the central frame (test frame), a mesh size of 15 x 15 mm was used to model the flat parts of all members and brackets, whilst the rounded corners and the lips of the channel sections were meshed with 4 elements and 2 elements respectively, resulting in a higher mesh density in these regions. For other parts of the frame system, coarser mesh sizes were utilised to model their components. Specifically, a mesh size of 30mm x 30mm was designated to the end frame components, whereas a 25mm x 25mm mesh size was assigned to the purlins.

4.6.7. Analysis

A geometric and material nonlinear static analysis was performed for each test frame system. There are two non-linear solution schemes, the modified Riks method and the Newton-Raphson, all are available in ABAQUS, to solve difficult non-linear problems. The modified Riks method, which is based on the Newton-Raphson method and uses an automatic arc length constraint in the load-displacement space, is ideal to employ when a snap-through and snap-back response is expected in the load-displacement equilibrium path or when load control is required in the analysis. However, the arc-length method may misjudge the load direction and presents premature analysis by back-tracking when the response shows very high curvature in the load-displacement space. Moreover, it is common to suffer the termination of the analysis without reaching the peak load as a result of convergence deficiencies, which typically stem from the FE models prone to local instabilities usually seen in thin-walled structures. In these cases, damping is generally required to dissipate the strain energy released by localised
buckling. However, a stabilisation algorithm or viscous damping cannot be introduced in the solution when the arc-length method was adopted in ABAQUS [12]. Therefore, in this study, the Newton-Raphson method (i.e. *Static, general) was adopted as the non-linear solver in conjunction with the application of displacement through the load spreading system serving as the loading and the use of automatic stabilisation to deal with localised instabilities. This solver is more stable and transparent as compared to the modified Riks method and is sufficient for the FE models where displacement control was used, and the snap-through and snap-back issues were not presented.

When the automatic stabilisation is employed to overcome the convergence difficulties, it may affect the accuracy of the solution. To minimise the effect of viscous damping on the results, the total strain energy (ALLSE) and the total amount of energy dissipated through artificial damping (ALLSD) were checked to ensure that the ALLSD was maintained at a negligible level compared with the ALLSE.

Two load combinations were employed for analyses of the strength and behaviour of the frames, which include the downward loading (gravity), and combined downward and horizontal loading (combined gravity and wind loads). To simulate these load conditions, a single load step using displacement control was defined for the former case, whilst two load steps were created for the other case, in which a horizontal force of 7.5kN was defined in the first step and displacement control was used in the second step. For controlling the load increments, automatic time incrementation with a small initial step size of 0.001 was chosen because of its efficiency in the analysis.
4.7. COMPARISON OF FE MODELS AND FRAME TEST RESULTS

4.7.1. The ultimate strength

The advanced shell finite element analyses were performed and their results were compared with those of the portal frame tests to validate the appropriateness and accuracy of the modelling method. Table 4.4 provides the frame ultimate strengths predicted by FE models along with the corresponding ultimate load capacities obtained from the experiments.

Table 4.4. Comparison of the ultimate loads

<table>
<thead>
<tr>
<th>Frame</th>
<th>Ultimate strength (kN)</th>
<th>Test</th>
<th>Advanced Analysis</th>
<th>P&lt;sub&gt;Test&lt;/sub&gt;</th>
<th>P&lt;sub&gt;AA&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frame Test 1</td>
<td>33.25</td>
<td>33.73</td>
<td></td>
<td>0.986</td>
<td></td>
</tr>
<tr>
<td>Frame Test 2</td>
<td>32.16</td>
<td>33.32</td>
<td></td>
<td>0.965</td>
<td></td>
</tr>
<tr>
<td>Frame Test 3</td>
<td>18.84</td>
<td>19.30</td>
<td></td>
<td>0.976</td>
<td></td>
</tr>
<tr>
<td>Frame Test 4</td>
<td>39.20</td>
<td>40.21</td>
<td></td>
<td>0.975</td>
<td></td>
</tr>
<tr>
<td>Frame Test 5</td>
<td>27.59</td>
<td>28.52</td>
<td></td>
<td>0.967</td>
<td></td>
</tr>
<tr>
<td>Frame Test 6</td>
<td>27.42</td>
<td>27.57</td>
<td></td>
<td>0.994</td>
<td></td>
</tr>
<tr>
<td>Frame Test 7</td>
<td>60.69</td>
<td>61.46</td>
<td></td>
<td>0.988</td>
<td></td>
</tr>
<tr>
<td><strong>MEAN</strong></td>
<td></td>
<td></td>
<td></td>
<td><strong>0.979</strong></td>
<td></td>
</tr>
<tr>
<td><strong>COV</strong></td>
<td></td>
<td></td>
<td></td>
<td><strong>0.011</strong></td>
<td></td>
</tr>
</tbody>
</table>

As can be evidently seen from Table 4.4, good agreements were presented between experimental ultimate loads and FE predictions. In general, the ultimate strengths predicted by the geometric and material non-linear static analyses (Advanced analyses) are marginally higher than those from the test results. Many contributing factors may lead to these differences, including the uncertainties in the introduction of imperfections, e.g. the member imperfections due to the frame installation were not considered in the FE models, the eccentricity of loads, or varying degrees of introducing pretension in bolts, etc. It is not practical to implement these factors in numerical simulations but they are apparently present in the frame tests. Despite of these sources of variability, the Advanced shell finite element analyses satisfyingly predicted the ultimate strengths of the frame tests. With the mean of test-to-prediction strength ratio of 0.97 and the corresponding low coefficient of variation of 0.017, it indicates that the use of the Advanced Analysis for the design of CRA portal frames is appropriate and potential.

4.7.2. Global deformations of the frames

Apart from predicting the ultimate strengths of the frames, the finite element models also provided to investigate the deformation behaviour of the frames, which were also compared
with the experimental behaviour. Frame Tests 1, 2, 4, 6, and 7 were completed with applied vertical loads only. In a CRA portal frame test, under the incremental vertical loads, the apex deflected downwards while the eaves were pushed outwards and the columns started to displace outwards at the same time. Frame Tests 3 and 5 were completed with applied horizontal and vertical loads. Under the horizontal load applied at the north eave, it could be observed in both Frame Test 3 and 5 that the whole frame displaced northwards. The apex continued to deflect downwards along with the action of vertical testing loads, whilst the columns and eaves began to displace outwards. These deformation behaviours also were observed and captured in the FE models. Typical deformed shapes of Frame Test 1 and Frame Test 5 obtained from numerical simulations are shown in Figure 4.30 and Figure 4.31, respectively. In all the tests, minimal deformation can be observed at the end frames, indicating that no significant loads were transferred to them and that almost applied loads were carried by the central frame.

![Figure 4.30](image1.png)  
Figure 4.30. The deformed Frame (Frame Test 1): (a) Displacement; (b) Rotation

![Figure 4.31](image2.png)  
Figure 4.31. The deformed Frame (Frame Test 5): (a) Displacement; (b) Rotation
Comparisons between the vertical loads applied to the test frames versus vertical displacement of the apex and the corresponding prediction from finite element models for all test frames are shown in Figure 4.32 (for Frame Tests subjected to vertical load only) and Figure 4.33 (for Frame Tests subjected to horizontal and vertical loads). It can be seen that there are good agreements between the structural responses of the experiments and the behaviours predicted by Advanced Analysis. In general, the stiffnesses of the frames predicted by Advanced Analysis are slightly higher than those obtained from experiments. The slightly small discrepancies between the actual behaviours and the numerical results can be explained due to the difficulty of introducing the fully accurate imperfections, the varying degrees of establishing pretension in bolts of the connections, and other uncertainties into the FE models.

Similarly, the horizontal (in-plane) displacements of both south and north eaves predicted by Advanced Analysis for all Frame Tests are also plotted in Figure 4.34 and Figure 4.35 along with those obtained from experiments. There is generally a close agreement between the experiments and the FE models apart from Frame Test 3. In Frame Test 3, under the effect of the constant horizontal load of 7.5 kN applied at the north eaves, the horizontal deflection of the south eaves predicted by the FE model is higher than that obtained from the frame test, whilst the predicted displacement of the north eaves is lower compared to the value obtained by experiment. This discrepancy is attributed to the actual flexibility of the system in Frame Test 3, which indicated by the difference in the horizontal displacement between the south and the north eaves in the tests, which was not fully captured by FE models. The source of uncertainty is discussed in Section 4.9.
Figure 4.32. Vertical applied load versus vertical displacement of apex for the test frames subjected to gravity only.
Figure 4.33. Vertical applied load versus vertical displacement of apex for the test frames subjected to horizontal and gravity loads: (a) the effects of both horizontal and gravity loads (Frame Test 3); (b) the effect of gravity (Frame Test 3); (c) the effects of both horizontal and gravity loads (Frame Test 5); (d) the effect of gravity (Frame Test 5)
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(a) Frame Test 1 - Experiment
  Frame Test 1 - Numerical

(b) Frame Test 2 - Experiment
  Frame Test 2 - Numerical

(c) Frame Test 3 - Experiment
  Frame Test 3 - Numerical

(d) Frame Test 4 - Experiment
  Frame Test 4 - Numerical

(e) Frame Test 5 - Experiment
  Frame Test 5 - Numerical

(f) Frame Test 6 - Experiment
  Frame Test 6 - Numerical
Figure 4.34. Horizontal displacements of south eaves
In Frame Tests 1, 2, 3, and 6, where the columns were not braced laterally, the frame failure mode as observed in the tests and subsequently validated by FE models was lateral-torsional buckling of a column. The failure mode was initiated by out-of-plane displacement of the knee braces and knee brace to column brackets, followed by significant flexural and torsional displacements of the column. As illustrated in Figure 4.36, a typical twist deformation of the north column of Frame Test 2 was observed. After the frames reached the ultimate loads, the columns continued to twist under the incrementally applied displacements until a distortional buckle or a local buckle of the compression flanges and lips occurred, simultaneously followed by a sudden drop of the vertical load. An example of local/ distortional buckling failure that occurred at the north column in Frame Test 2 can be seen in Figure 4.37.
Figure 4.36. Twisting of the north column in Frame Test 2: (a) Experiment; (b) Numerical

Figure 4.37. Local/ distortional buckling failure of the north column in Frame Test 2: (a) Experiment; (b) Numerical

The twist rotation of the column where the failure occurred at the first peak load for all unbraced column frames are plotted in Figure 4.38 to Figure 4.41. While the overall failure of the Frame Test 1 initiated at the south column, the initiating failure exhibited at the north column for other unbraced column frames. As seen in Figure 4.38 to Figure 4.41, the twist
rotation of the columns predicted by Advanced Analysis agrees well with the response obtained from the experiments.

![Graph](image1.png)

**Figure 4.38.** Vertical load versus twist rotation of south column for Frame Test 1: (a) at 125 mm below the knee bracket; (b) at 2.15 m above the base plate

![Graph](image2.png)

**Figure 4.39.** Vertical load versus twist rotation of north column for Frame Test 2: (a) at 125 mm below the knee bracket; (b) at 2.15 m above the base plate
4.7.4. The localised failure mode of columns

In Frame Tests 4, 5, and 7, the columns were laterally restrained by the girts. These experiments aimed at determining the influence of the lateral bracing system of the columns on the strength and behaviour of the frame system. Detailed FE models were also developed to validate the effects of this girt system in the tests. As discussed in Chapter 3, with the presence of the girt system, the columns did not fail in flexural-torsional buckling mode, and thus the strength of the frame system was enhanced significantly. Similar failure phenomena were observed in Experiments 4 and 5, in which the columns failed locally right at the knee brace location as it is the critical section. A representation of this failure mode obtained from both
experiment and numerical model can be seen in Figure 4.42. The Frame Test 7 had the modified columns with sleeve stiffeners to improve the ultimate capacity for the columns, especially at the knee brace position. In this case, the localised failure mode occurred at the bottom end of the sleeve stiffener section of the south column as this is the weakest section of the column as shown in Figure 4.43. As it is evident from Figure 4.42 to Figure 4.43 that all these behaviours were well captured by Advanced Analysis with good agreements between the FE models and the experiments.

Figure 4.42. The localized failure mode of the north column in Frame Test 4: (a) Experiment; (b) Numerical

Figure 4.43. The localized failure mode of the south column in Frame Test 7: (a) Experiment; (b) Numerical
4.8. PARAMETRIC STUDIES

The full 3D cold-rolled aluminium portal frames corresponding to seven frame tests were modelled and analysed by using the software ABAQUS. The strength and behaviour predicted by the Advanced Analysis were in good agreements with those obtained and observed from the tests. Based on the modelling technique calibrated against the experimental results, parametric studies were performed to determine the effect of the column base stiffness and the contribution of the fly bracing on columns and lateral bracing for columns at the knee connection, to create a larger span aluminium portal frame system, and to determine the flexural stiffness of the apex joint for Frame Test 6. In general, FE models generated for parametric studies use the same modelling technique as discussed in previous sections with the exception of introducing initial geometric imperfections into the models. The incorporation of imperfections is detailed in the following section.

4.8.1. Imperfections for parametric studies

Assumed imperfections were used for parametric studies, including sectional, member and frame imperfections. While sectional imperfections consist of local and distortional imperfections, member imperfections include the type of deviation of the perfect geometry of the member where the cross-section is displaced or rotated as a whole (camber, bow and twist imperfections). The frame imperfections are typically associated with sway shaped imperfections and defined by out-of-plumb angles. These imperfections are fully discussed in Section 4.6.5.

Out-of-plumb has a detrimental influence on the load capacity of the frame system as it may increase the second-order effects on the members and trigger sway buckling failure. The Eurocode 9 – Design of aluminium structures [152] includes the provision that imposes the frame imperfections through an initial out-of-plumb angle for global analysis of frames, as follows:

\[
\phi = \phi_h \alpha_h \alpha_m
\]

where:

\[
\phi_h = \frac{1}{200}
\]  \hspace{1cm} (4.5)

\[
\alpha_h = \frac{2}{\sqrt{h}}, \quad \text{but} \quad \frac{2}{3} \leq \alpha_h \leq 1.0
\]  \hspace{1cm} (4.6)

\[
\alpha_m = \sqrt{0.5(1+1/m)}
\]  \hspace{1cm} (4.7)

in which, \( h \) is the height of the column in metre and \( m \) is the number of column in a row including only those columns which carry a vertical load not less than 50% of the average value of the column in the vertical plane considered.
On the other hand, from the measured values of out-of-plumbness of the columns as provided in Table 3.4, the out-of-plumb angle for the south and the north columns are different and vary from $\phi = 1/3323$ to $\phi = 1/191$ with the mean of $\phi = 1/769$. Therefore, to ensure that a parametric study is simply performed with an acceptable level of safety, the initial sway imperfection of $\phi = 1/268$ calculated using Equation (2.19) was imposed.

For member imperfections, each imperfection mode was assumed to form a single sinusoidal function and can be expressed as follows:

$$f_{GL}(z) = G_i \sin \left( \frac{\pi z}{L} \right)$$

where $G_i (i = 1, 2, 3)$ is the amplitude of the respective imperfection mode, $z$ is the coordinate along the length, and $L$ is the length of the member.

For sectional imperfections, the imperfection modes were performed using sinusoidal functions, as follows:

$$f_{Le}(z) = d_1 \sin \left( \frac{\pi z}{L_l} \right)$$

$$f_{Duo}(z) = d_2 \sin \left( \frac{\pi z}{L_d} \right)$$

where $d_1$, $d_2$ represent the maximum amplitudes of the local and distortional imperfections, respectively. $L_l$ represents local buckling length and $L_d$ represents distortional buckling length.

The amplitude imperfections were determined on the basis of the measured imperfection data as provided in Section 3.2.6.2 and are summarised in Table 4.5.

<table>
<thead>
<tr>
<th>Components</th>
<th>$G_1/L$</th>
<th>$G_2/L$</th>
<th>$G_3$</th>
<th>$d_1$</th>
<th>$d_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Degree/m</td>
<td>mm</td>
<td>mm</td>
</tr>
<tr>
<td>Column AC25025</td>
<td>1/5810</td>
<td>1/6677</td>
<td>0.097</td>
<td>0.620</td>
<td>1.734</td>
</tr>
<tr>
<td>Rafter AC25025</td>
<td>1/6041</td>
<td>1/4375</td>
<td>0.134</td>
<td>0.730</td>
<td>2.192</td>
</tr>
</tbody>
</table>

In general, the assumed imperfections were incorporated into the FE models directly using the same procedure as shown in Figure 4.27, except that it is unable to use the interpolation algorithm to introduce the sectional and member imperfections. Therefore, new algorithm and corresponding Matlab code were developed to incorporate the assumed sectional and member imperfections into ABAQUS models. Full details of the algorithm and Matlab program can be seen in Appendix C.2.
4.8.2. Column base stiffness

Base connection is an essential part of the portal frame system. In the tests, the column base connection was composed of two 8mm thick stainless steel L-plates with washers bolted to the flanges of the columns. The L-plates were connected to the base plate system including two 20mm thick steel plates and four load cells to record the base reactions during the test. The lower plate was clamped to the I-steel beam encasted in the concrete floor. Further details of the base connection can be found in Section 3.2.5.3. In practice, the L-cleats and washers would be clamped to cast-in bolts in a concrete slab or footing. This type of connections, which is neither pinned nor fixed, is considered as a semi-rigid connection as it is able to resist moment, also rotates as the load is applied.

Small scale column base rotation tests were completed to quantify the base stiffness for bending about both column-major and minor axes. Various base brackets including 3mm, 6mm and 8mm L-plates and 3mm U-plate were tested to examine their effects on the base stiffness. The details of these experiments and results were provided in Section 3.5.

Full three-dimensional (3D) frame models with various configurations of the base brackets representing different base stiffnesses were performed. Fully rigid and pinned conditions were also created. All the models had the same material properties, geometry, and boundary conditions which are similar to those employed for the models of Frame Tests 1 and 2. However, the members were modelled with the nominal section dimensions and thickness instead of measured sections as implemented in the calibrated models. The models were analysed for the two load combinations: applied vertical loads only or a fixed horizontal load of 7.5 kN followed by vertical load until failure.

The results of the numerical analyses are given in Table 4.6. It can be seen that the variation in base stiffness has a detrimental effect on the vertical load capacity of the frame system, especially in the case of the frame subjected to the combined wind and gravity loads. Changing the base fixities from rigid to pinned results in a decrease in the ultimate vertical load by 22.5% and 53.68% for applied gravity and applied wind and gravity loads, respectively. Likewise, reducing the thickness of the L-shaped base connection brackets from 8 mm to 3 mm decreases the ultimate load by 8.4% for applied vertical load only and 40.4% for applied both horizontal and vertical loads. Therefore, the column base stiffness has a greater effect on the ultimate strength of the frame systems when the wind load is applied.

Furthermore, the structural behaviour of the frame with different base configurations are also plotted in Figure 4.44 and Figure 4.45 for applied vertical load only and applied combined wind and gravity loads, respectively. It can be seen that the variation in the base stiffness also influences the response of the frame system. As a result, enhancing performance of the base connections should be of vital importance to the engineer to improve the performance of the whole portal frame system.
Table 4.6. The effect of the base connection on the strength of the frame systems

<table>
<thead>
<tr>
<th>Loading</th>
<th>Base Connection</th>
<th>Corresponding Base Stiffness (kNm/rad)</th>
<th>Ultimate Vertical Load (kN)</th>
<th>% decrease</th>
</tr>
</thead>
<tbody>
<tr>
<td>V</td>
<td>Fixed condition</td>
<td>Inf</td>
<td>35.96</td>
<td>-</td>
</tr>
<tr>
<td>V</td>
<td>8 mm L-shaped Brackets</td>
<td>912.69</td>
<td>35.32</td>
<td>1.78</td>
</tr>
<tr>
<td>V</td>
<td>6 mm L-shaped Brackets</td>
<td>820.73</td>
<td>35.02</td>
<td>2.62</td>
</tr>
<tr>
<td>V</td>
<td>3 mm U-shaped Bracket</td>
<td>467.28</td>
<td>32.91</td>
<td>8.48</td>
</tr>
<tr>
<td>V</td>
<td>3 mm L-shaped Brackets</td>
<td>504.61</td>
<td>32.35</td>
<td>10.03</td>
</tr>
<tr>
<td>V</td>
<td>Pinned condition</td>
<td>0</td>
<td>27.85</td>
<td>22.55</td>
</tr>
<tr>
<td>H + V</td>
<td>Fixed condition</td>
<td>Inf</td>
<td>25.23</td>
<td>-</td>
</tr>
<tr>
<td>H + V</td>
<td>8 mm L-shaped Brackets</td>
<td>912.69</td>
<td>19.60</td>
<td>22.30</td>
</tr>
<tr>
<td>H + V</td>
<td>6 mm L-shaped Brackets</td>
<td>820.73</td>
<td>18.43</td>
<td>26.96</td>
</tr>
<tr>
<td>H + V</td>
<td>3 mm U-shaped Bracket</td>
<td>467.28</td>
<td>12.17</td>
<td>51.75</td>
</tr>
<tr>
<td>H + V</td>
<td>3 mm L-shaped Brackets</td>
<td>504.61</td>
<td>11.86</td>
<td>52.98</td>
</tr>
<tr>
<td>H + V</td>
<td>Pinned condition</td>
<td>0</td>
<td>11.68</td>
<td>53.68</td>
</tr>
</tbody>
</table>

Note: V = Vertical load & H = 7.5 kN horizontal load

Figure 4.44. Vertical load versus apex vertical displacement with different base stiffness obtained from FE models for applied gravity load only
Figure 4.45. Vertical load versus apex vertical displacement with different base stiffness obtained from FE models for applied horizontal and gravity loads

4.8.3. Different bracing methods for columns

The experimental and numerical results demonstrated that the failure mode of portal frames with unbraced columns was lateral-torsional buckling of the columns, whilst the failure mode of portal frames with braced columns occurred by the formation of a spatial plastic mechanism at critical locations. It was also indicated that the strength of the frame system was enhanced significantly when the columns were braced by the girts connected along the column height, especially when the frames were subjected to combined horizontal and vertical loads. This is due to the fact that the girt system helps to partially prevent the columns from lateral movement and twisting, resulting in a better performance of the frame system.

It is extremely common in practice that fly bracings are also installed in columns/rafters along with girts/purlins to provide extra stability and resist wind load. Therefore, it is necessary to examine the effect of fly bracings on the performance of portal frames.

In this section, the contribution of the fly bracings on columns was evaluated by using reliable finite element models without conducting physical laboratory tests. For this purpose, a full 3D portal frame system was modelled using the same modelling techniques adopted in the calibrated models. The models had the same geometry as Frame Tests 4 and 5 where the columns of the original frame were braced by girts (Figure 4.46). Apart from that, at the locations of the girts, fly bracings were placed to prevent the compression flanges of the columns from twisting, as shown in Figure 4.46. The model was run for two cases of loading: (1) applied vertical load only (Load Case 1) and (2) a fixed horizontal load of 7.5 kN followed by vertical load until failure (Load Case 2). Other modelling inputs including material
properties, fastener characteristics, contact, etc, are the same as those from the calibrated models. The results indicated that the failure mode of the frame was the formation of a spatial plastic mechanism at the knee connection, which is the same behaviour as seen in Frame Tests 4 and 5. The vertical loads versus vertical displacement of the apex relationship curves are plotted for both loading cases as shown in Figure 4.48 and Figure 4.49, respectively (named as “frame with girts and fly bracings”).

As observed in Figure 4.48 and Figure 4.49, the frame, in which the columns were restrained against out-of-plane movement and twist rotation by girts and fly bracings, reached ultimate vertical loads of 43.83 kN and 34.16 kN when subjecting to the vertical load only and the combined horizontal and vertical loads, respectively. This means that a considerable improvement on the frame ultimate vertical load was presented. The ultimate vertical load of the frame for Load Case 1 and Load Case 2 increased by approximately 29.9% and 77% respectively as compared to those of the original frame (the Frame Tests 1 and 2 in which the columns were not restrained). Similarly, compared to the frame with girts only, a 9.1% and 19.8% increase in the ultimate load was shown in the frame for Load Case 1 and Load Case 2, respectively.

Figure 4.46. Finite element model of frames with fly braces for columns

As mentioned previously, the failure mode of the unbraced column frames was observed as lateral-torsional buckling of the columns, which was initiated by the out-of-plane movement of the knee brace-to-column brackets as the knee to column connection was the weak part of the frame. Although the use of girts to provide partial lateral restraints for columns had proven to be efficient in improving the performance of the portal frame system, the installation of girts is not always feasible. For example, in the frames where opening sides are required, other methods of bracing for the columns should be considered. One of the possible methods is to
provide partial lateral restraints for the columns at the knee connection using fly braces as shown in Figure 4.47. On the basis of the above discussions, a similar model as the one for Frame Tests 1 and 2 was created where the columns at the knee connection were partially restrained against out-of-plane deflections by providing fly braces. The layout of the finite element model is shown in Figure 4.47. The analysis results for the two loading conditions (Load Case 1 and Load Case 2) are provided in Figure 4.48 and Figure 4.49, respectively, which is labelled as “frame with lateral braced knee”.

The numerical results demonstrated that the failure mode of the frame with fly braces on the columns at the knee connection for both load cases is lateral-torsional buckling of the columns. The twist deformation of the columns in the Load Case 1 after the frame reached the ultimate load is shown in Figure 4.50. As seen in Figure 4.48 and Figure 4.49, the introduction of fly braces resulted in a slight increase of 2.4 % on the frame ultimate vertical load as compared to the original frame configuration. More notably, the improvement in the performance of the frame system under the effect of the horizontal load is obviously seen. In Load Case 2, the frame failed at 23.64 kN, which is a 22.5% increase in ultimate vertical load a compared to the initial frame. Hence, the application of fly braces on columns at the knee connection is potential.

Figure 4.47, Finite element model of frames with fly braces at knee connections
Figure 4.48. Vertical load versus vertical displacement of apex for frames subjected to vertical load only

Figure 4.49. Vertical load versus vertical displacement of apex for frames subjected to combined horizontal and vertical loads
Larger spans of Aluminium portal frames

The behaviour and strength of a larger span subjected to vertical load can be obtained by numerical finite element models. For this purpose, a full 3D frame system with a centerline span of 28 m was modelled in ABAQUS using the same modelling technique as discussed in the preceding sections. The layout of the finite element model is presented in Figure 4.51.
The frame systems consisted of three portal frames connected by purlins spanning between the rafters, forming a free-standing two-bay frame system with the bay distance of 3.66 m, which is the same as the test frames. In the modelled frames, the height of eaves from the base plate was 5.4 m while the pitch of 10° was adopted, leading to the height of the centre of the apex joint of 7.83 m. The sections comprised of back-to-back double channels bolted through the webs for the columns, rafters, knee braces and rafter ties. While the columns and rafters were selected using 2AC30030 channel sections with member lengths of 5.22 m and 14.29 m, respectively, the knee braces and rafter ties were 2AC20025 channel sections. The purlins profiles are C20025 channel sections and were placed along the rafters at 1750 mm at the intermediate regions and 750 mm at the apex and eave regions. The same configurations were adopted for the brackets used in the respective connections of the frame system, but these brackets were scaled in other dimensions to fit with the utilised sections, whilst their plate thickness was 6 mm as used in the experimental program. In addition to the knee braces and rafter tie as used in the Frame Tests, a rafter tie dropper connected to the bottom of the apex brackets and the rafter tie through L-brackets was also utilised. The dropper profiles of back-to-back C15025 channel sections were chosen. The column base connection consisted of two 8 mm thick L-shaped brackets bolted to the flanges of the column through 6 bolts for each side.

The material properties obtained from coupon tests were defined and assigned to the respective components as discussed in Section 4.2, whilst point-based fasteners were used to model the bolted connections in the frames. In this case, the material properties and fastener
characteristics were assumed to be the same as those obtained from the test program. Besides, the members were modelled with the nominal section dimensions and thickness.

A geometric and material nonlinear static analysis was performed for two cases of loading: (1) vertical load only (Load Case 1) and (2) a fixed horizontal load of 7.5 kN followed by vertical load until failure (Load Case 2). The loading system was modelled using similar method to that used in the calibrated models, in which the incrementally applied loads were simulated by using the displacement control through a load spreading system as shown in Figure 4.52.

![Figure 4.52. Modelling of the load spreading system for larger span](image)

The results show that failure of the frame for both the load cases was lateral-torsional buckling of the north column. The lateral displacement and twist rotation of the frame subjected to gravity load alone at failure are presented in Figure 4.53. The frame reached an ultimate vertical load of 49.57 kN and 39.86 for Load Case 1 and Load Case 2, respectively. The vertical load versus the apex vertical displacement responses for both load cases are plotted in Figure 4.54 along with the corresponding graph of the FE model for Frame Test 1, which had the same configuration but using different sections and spans from the considered frame. As seen in Figure 4.54, although the span of the modelled frame was doubled as compared to Frame Test 1, the ultimate vertical load was significantly higher. This is because the frame using a thicker and larger section in addition to the changes in the knee brace and rafter tie connections. Besides, the apex deflections of the 28 m span frame, as expected, are greater than those of the 14 m span frame.
Figure 4.53. Deformation of 28 m span frames: (a) Lateral displacement; (b) Twist rotation of the columns

Figure 4.54. Load versus vertical displacement of apex: (a) For applied vertical load only; (b) For applied horizontal and vertical loads
4.8.5. The stiffness of the apex connection in Frame Test 6

As often used in design practice in the reality design situations, the portal frames are typically modelled using beam elements instead of complex 3D shell elements. In a beam element model, the semi-rigid connections are generally represented by the elastic stiffness of the connections, which can be determined from the respective connection tests or numerical models. In this study, the connections between members in the portal frames, except for the apex connection of Frame Test 6, can be assumed as pinned connections. This assumption is based on the minimal impact of the true spring stiffness of these connections, which is the result of the effective triangles formed between the knee brace, the column, and the rafter at the eaves connection and between the rafter tie and the rafters at the ridge connection. However, in Frame Test 6, the rafter tie was removed, as a consequence, the assumption of the pinned joint at the apex is no longer maintained. In this case, the in-plane bending stiffness of the apex connection must be determined to produce an accurate beam element model.

The apex connection was modelled using shell elements in ABAQUS to determine the apex flexural stiffness to incorporate in a beam element model. The apex connection model was created with two double-C AC25025 sections serving as rafters, back-to-back apex brackets, apex stiffeners, and purlin brackets. The double-channel sections had a length of 660 mm, which was long enough to replicate the bending moment at the apex joint in the full-scale tests, but also short enough to eliminate the effects of bending in rafters. The introduction of purlin cleats in the model allows the lateral restraints provided by purlins at the apex connection to be implemented by applying out-of-plane restraints at the edges of the purlin cleats, resulting in a more realistic model. An end plate defined as a rigid body was attached to the end of each rafter using a surface-to-surface tie constraint. Boundary conditions were applied to the end plates at the rigid body reference points to prevent all translational degree of freedoms at the left end and prevent vertical and lateral (out-of-plane) movement at the other end. Bolted connections were modelled using the same technique as previously described in Section 4.4, whilst contacts between components were also modelled using the same method when simulating contacts in the full-scale frame tests. Moments were applied to the rigid body reference points to load the connection in bending.
Once the FE analysis was completed, the rotation of the apex connection can be determined. By using the FE model, both the downward deflections and rotation could be obtained. This means that the rotation of each rafter can be determined directly from the rotation results. However, to minimise the effect of rafter bending, the rotation of each rafter was determined based on the vertical displacements as follows: the downward deflections at the centerline of the rafters for a 100 mm length starting at the end of the apex brackets were extracted from the output of the numerical model. The rotation of the apex connection on each side is the deviation of the recorded deflections from the assumed deflections of a fully rigid connection. This approach allows the local effects of the connection and the effect of rafter bending to be eliminated while the characteristic rotation is still obtained. The total rotation of the connection was achieved by adding the rotations together. The moment-rotation relations of the apex connection, considered herein, are shown in Figure 4.56.

As seen in Figure 4.56, the moment-rotation relation of the apex connection is essentially linear up to an applied moment of about 17.5 kNm. A linear regression analysis was performed on the graph to determine the stiffness of the connection and the resulting stiffness of the connection was 1412.5 kNm/rad. It is worth noting that, in the full-scale frame test, there was no failure and/or significant bolt slip observed in the apex connection. Hence it is assumed that the connection remained in the elastic range during the test, and thus the elastic stiffness of 1412.5 kNm/rad can be used in the beam element model for Frame Test 6.
Chapter 4 – Numerical Modelling

Figure 4.56. Moment versus rotation of apex connection obtained from the FE model

4.9. DISCUSSIONS AND CONCLUSIONS

In this Chapter, the advanced nonlinear finite element analysis of portal frame systems composed of cold-rolled aluminium (CRA) sections is presented. All sources of major nonlinear actions are included in the analysis, notably material and fastener nonlinearities and initial geometric imperfections.

The nonlinear stress-strain relations obtained from coupon tests were converted into true stress-strain characteristics to model the material nonlinearity, whilst deformable mesh-independent point-based fasteners were utilised to idealise the physical bolted connections in the frame systems. The nonlinear behaviour of the individual fasteners was defined by using connector properties assigned to force-deformation relations, which were calibrated against the results provided by the respective point fastener connection tests.

The initial geometric imperfections including sectional, member, and frame imperfections were thoroughly incorporated into the FE models by using the explicit modelling approach, i.e. shifting the position of nodes from the perfect geometry to the desired imperfect geometry. The imperfections measured from experiments were adopted for the calibrated models, whilst statistical data was used for parametric studies.

Further, all contacts, boundary conditions, and load spreading system were also simulated to create the realistic models of the CRA portal frames in all seven frame tests. It was demonstrated that the ultimate strength and structural behaviour predicted by the Advanced Analysis were in good agreements with the actual strength and behaviour monitored from experiments. With the mean of the test-to-prediction strength ratio equal to 0.979 and a low coefficient of variation of the strength ratio, it indicated the promising use of the Advanced Analysis for the design of portal frame systems comprised of CRA sections. The strength
predicted by the Advanced Analysis combined with the actual strength from experiments also provided essentially valuable statistical data for undertaking the system reliability analysis of the CRA portal frames. Besides, the numerical analyses revealed that flexural-torsional buckling deformation of the columns is a primary structural behaviour in the frames with unbraced columns, and the ultimate failure is the local/distortional buckle of the compression flanges and lips while the formation of a spatial plastic mechanism in the columns at critical locations is the ultimate failure of the braced column frames. These responses are identical to those observed in the physical tests.

Parametric studies were also performed on the basis of the calibrated modelling technique (i) to determine the effect of the column base stiffness on the ultimate strength of the portal frame systems, (ii) to evaluate different methods of bracing for columns that were used to improve the performance of the frame systems, (iii) to examine the applicability of larger span cold-rolled aluminium portal frames in practice, and (iv) to quantify the flexural stiffness of the apex joint for Frame Test 6, which is necessary to produce a beam element model later on. It was indicated that the column base stiffness has a profound effect on the strength of the portal frame systems especially when the frames were applied both horizontal and vertical loading. Therefore, the base connections need to be carefully designed and fabricated to reduce the variations in the base stiffness. When lateral and torsional restraints to the columns by either the girts and fly bracings along the height of the columns or fly bracing on the columns at the knee connections, higher frame capacities were obtained. Hence, to optimise the structural performance, where practically possible, column bracing should be provided. In addition, the possibility of constructing larger span frames was also revealed by numerical study if reconfiguring the knee braces and rafter tie for apex joint is conducted.

In general, the structural response and ultimate load predicted by the numerical models do not perfectly match with those from the physical tests. In fact, the fully identical predictions between FE models and experimental results are unrealistic due to a number of reasons that can be highlighted as follows:

1. In test frames, it was difficult to align concentrically bolt centres with bolt hole centres due to bolt-hole clearance, whilst precise locations of bolts can be defined in FE models
2. The difficulty of inducing the uniform pretension in each bolt of the test frames, whereas the same bolt pretension was ideally assigned to all bolts of each type in the numerical models.
3. Imperfect geometry of bolt holes in the actual members and brackets.
4. Additional local imperfections were introduced during the installation process, whilst the sectional and member imperfections incorporated into FE models were measured before the assembly of members.
5. The exhibition of variation and anisotropy in material properties was indicated in coupon tests, in numerical modelling, material properties. However, they were idealised on the basis of the average stress-strain relations and were assumed to be uniform for all directions.

6. The introduction of eccentric loading applied to the frames, which is likely due to the imperfect alignment of the load spreading system, initial twisting of rafters during installation, etc.

7. The difference in the point of measuring displacement between the tests and FE models. In reality, the transducers used to measure the displacement of the test frames were fixed in space, whilst the frame gradually deformed under the effect of loading. As a result, when the frames underwent large deformation, the final points of measurement were at locations that were significantly different from the initial point of measurement. Meanwhile, the FE models allow the displacements of the same point to be extracted.

8. Other factors may affect the measuring results in experiments such as temperature effects, the variation of the electrical signal of the instruments, depending on the length of the cable, etc.

In conclusion, in spite of the existence of a small degree of discrepancies between numerical predictions and experiments, the finite element method with the modelling technique described in this Chapter has been proven to be an efficient tool to predict the strength and behaviour of the CRA portal frames.
CHAPTER 5. DESIGN OF COLD-ROLLED ALUMINIUM PORTAL FRAMES

5.1. INTRODUCTION

One of the key activities in the design of a structure is to determine the structural strength. At present, aluminium portal frames are usually designed on the basis of the member-based design methods such as Effective Thickness Method, Effective Width Method, or Direct Strength Method, etc. This has been mainly motivated by the availability of exhaustive resources of design guidelines available in the current standards/specifications. The design process includes two stages. In the first step, the effects of design actions are usually obtained by second-order structural analysis, in which beam elements are preferred. The strengths of the individual members are subsequently checked/calculated by design equations in the standards. For the design of portal frame systems using member-based design approach, one of the challenges is to determine the accurate effective length factors, which are required for the computation of elastic buckling load for compression members in flexural, torsional, and flexural-torsional buckling and elastic lateral-torsional buckling moment for members in bending. Especially, in the cold-formed section portal frames where their connections are semi-rigid in nature, the determination of the effective length factors become challenging as no proper guidance is available for the calculation of effective length factors for semi-rigid connections.

On the other hand, the system-based design (also known as “Direct Design Method”) using Advanced Analysis (second-order inelastic analysis) has been proven to be the most capable ones to capture the strength and stability of a structural system, in which all sources of major nonlinear actions are included in the analysis [213]. It has been demonstrated that the system design by inelastic analysis provides a number of important advantages over the conventional design methods. By using Advanced Analysis, the system strength can be directly assessed from the analysis without the requirement of determining effective length factors or checking any equation in the codes. Furthermore, the Direct Design Method using Advanced Analysis may allow lighter and more economic structures to be designed. In this respect, there has been widespread growth of research studies on system-based design mainly for hot-rolled section steel structures [153, 178, 212, 254, 255]. However, the method has not become the routine design method for structures, especially the structures consisting of cold-formed sections, due to the relatively rare knowledge base.

In this chapter, the current international aluminium design standards/specifications are evaluated for the design of cold-rolled aluminium (CRA) portal frames based on the solid basis of experimental and numerical investigations conducted in Chapters 3 and 4. For this purpose, different approaches to determine the elastic buckling load of the column in compression subjected to flexural, torsional, and flexural-torsional buckling ($N_{oc}$) and elastic buckling moment of the column in flexural-torsional buckling ($M_o$) are presented, and the interaction equations of the member in combined compression and bending are subsequently used to
determine the nominal capacities of the frame systems. The Direct Design Method using Advanced Analysis is described in the next part, where the ultimate capacities of the cold-rolled aluminium portal frames are compared with those obtained from the conventional design methods. The system reliability analysis is also performed to derive the system resistance factor for the frames.

5.2. ELASTIC BUCKLING OF A COLUMN IN THE FRAME SYSTEMS

The conventional aluminium design method currently incorporated in the AS/NZS 1664.1:1997 [197], American Aluminium Design Manual AA2015 [198], and European Code BS EN1999:2007 [194] is based on elastic analysis and safety checks of individual components, and the system effects are only implicitly reflected in design through the use of effective length factor. For the design of CRA portal frames using the conventional method, one of the challenges is determining the accurate effective length factor required for the determination of buckling capacities. Therefore, in this Section, the buckling capacities of the column are determined and provided.

5.2.1. Finite strip buckling analysis of back-to-back channel section

Finite strip buckling analysis was performed using Thin-Wall-2 software [33] to obtain the minimum elastic local and distortional buckling stresses of back-to-back channel section AC25025 for members in compression and in bending. Shear buckling stress was also provided for members subjected to shear. Based on these buckling stresses, the corresponding buckling loads were calculated. The respective buckling stresses are presented in Table 5.1. The signature curves obtained from the finite strip buckling analysis for member subjected to axial compression and major axis bending moment are also given in Appendix D.1.

<table>
<thead>
<tr>
<th>Force condition</th>
<th>Local buckling stress $f_{ol}$ (MPa)</th>
<th>Distortional buckling stress $f_{od}$ (MPa)</th>
<th>Shear buckling stress $f_v$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Member in compression</td>
<td>35.77</td>
<td>64.79</td>
<td>-</td>
</tr>
<tr>
<td>Member in major axis bending</td>
<td>193.70</td>
<td>184.74</td>
<td>-</td>
</tr>
<tr>
<td>Member in minor axis bending</td>
<td>487.90</td>
<td>164.48</td>
<td>-</td>
</tr>
<tr>
<td>Member subjected to shear</td>
<td>-</td>
<td>-</td>
<td>48.17</td>
</tr>
</tbody>
</table>
5.2.2. Determination of elastic buckling load \( (N_{oc}) \) for the compression member undergoing flexural, torsional or flexural-torsional buckling and the elastic lateral-torsional buckling moment \( (M_o) \) based on the member effective length

5.2.2.1. Effective lengths

For the design of members subjected to compression and bending, the elastic buckling load \( (N_{oc}) \) for the compression member undergoing flexural, torsional or flexural-torsional buckling and the elastic lateral-torsional buckling moment \( (M_o) \) must be determined. To determine \( N_{oc} \) and \( M_o \), the effective lengths for buckling about the x-axis \( (L_{ex}) \), for buckling about the y-axis \( (L_{ey}) \), and for twisting \( (L_{ez}) \) are required. However, the effective lengths of columns, \( L_{ex}, L_{ey}, \) and \( L_{ez} \), are unknowns and would be difficult to accurately quantify, especially when considering the effects of the knee connection. For the CRA portal frames, no proper guidelines are available to incorporate the effects of the intermediate torsional restraint of the knee brace connections, and to consider the effects of semi-rigid connections on the determination of the effective lengths of the column. Therefore, in this section, assumptions were adopted to idealise the end restrained conditions, and the effect of the intermediate torsional restraint was ignored in the computation of effective lengths.

Based on the recommendations available in Chapter 4 of the Design guide for portal frame steel sheds and garages [256], AS/NZS 1664.1:1997 [197], American Aluminium Design Manual AA2015 [198], AS/NZS 4600 [146], and AS 4100 [214] the effective length factors for buckling about the x-axis \( (k_{ex}) \), buckling about the y-axis \( (k_{ey}) \), and twisting \( (k_{ez}) \) for the Frame Tests are determined as follows:

The effective length factors depend on the ratios \( \gamma \) of the compression member stiffness to the end restraint stiffnesses. The stiffness ratios shall be calculated as follows:

\[
\gamma = \frac{\sum \left( \frac{l}{l} \right)_c}{\sum \beta_e \left( \frac{l}{l} \right)_b}
\]

where \( \sum \left( \frac{l}{l} \right)_c \) and \( \sum \left( \frac{l}{l} \right)_b \) are the flexural stiffness at the member ends, and \( \beta_e \) is the modifying factor, which accounts for the conditions at the far ends of the beams.

After determining the stiffness ratios, the charts presented in Figure 4.6.3.3 in the AS 4100 [214] were used to determine the effective length factors. In the tests, the frames were unbraced in-plane but braced in the out-of-plane direction by purlins and the cross-bracing system.

To quantify the effective length factor \( k_{ex} \), the effective length factor chart for sway frame was adopted. The stiffness ratio at the base end of the column was calculated as the ratio of the column flexural stiffness about the major axis \( (4EI_x/L) \) to the base connection stiffness of 912.69
kNm/rad obtained from the tests. Meanwhile, when calculating the stiffness ratio at the top of the column, Equation (5.1) was used with an assumption that the length of the rafter is taken as the total developed length of the rafter pair from column to column or taken as span/cosθ (θ is the roof slope) [93, 114]. Due to the effective triangle formed between the knee brace, column and rafter at the eave connections, fixity condition at the far end of the rafter can be considered as rigidly connected to a column.

For determining the effective length factors $k_{ey}$ and $k_{ez}$, the stiffness ratio at the base was calculated as the ratio of the column flexural stiffness about the minor axis ($4EI/L$) to the base connection stiffness of 370.74 kNm/rad obtained from the minor base connection tests. The top end of the column was assumed to be pinned end as no significant rotational restraint was provided leading to the stiffness ratio equal to infinite. In addition, Table 17 in the Design guide for portal frame steel sheds and garages [256] suggests that the effective lengths $L_{ey}$ and $L_{ez}$ are the maximum length between adjacent bracing points. For Frame Test 1, 2, 3, and 6, where the columns were not braced in the out-of-plane direction, the adjacent bracing points are the eave connection and the base connection. In Frame test 4, 5, and 7, the columns were laterally restrained by girts with the spacing of girts of 1.03 m. In these cases, the effective length $L_{ey}$ was taken as 1.03 m while a certain degree of torsional restraints existed, but it was difficult to quantify and thus, the same effective length $L_{ez}$ was adopted as the unbraced column frames to provide a conservative design.

Based on the above discussions, the effective length factors and corresponding effective lengths of the column for all Frame Tests are calculated and presented in Table 5.2.

### Table 5.2. Effective length factors and effective lengths of column used for design

<table>
<thead>
<tr>
<th>Test Series</th>
<th>Effective length factors</th>
<th>Effective lengths (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$k_{ex}$</td>
<td>$k_{ey}$</td>
</tr>
<tr>
<td>Frame Test 1</td>
<td>1.58</td>
<td>0.82</td>
</tr>
<tr>
<td>Frame Test 2</td>
<td>1.58</td>
<td>0.82</td>
</tr>
<tr>
<td>Frame Test 3</td>
<td>1.58</td>
<td>0.82</td>
</tr>
<tr>
<td>Frame Test 4</td>
<td>1.58</td>
<td>1.00</td>
</tr>
<tr>
<td>Frame Test 5</td>
<td>1.58</td>
<td>1.00</td>
</tr>
<tr>
<td>Frame Test 6</td>
<td>1.58</td>
<td>0.82</td>
</tr>
<tr>
<td>Frame Test 7</td>
<td>1.58</td>
<td>1.00</td>
</tr>
</tbody>
</table>

#### 5.2.2.2. Member in compression

For members in compression, the elastic buckling load $N_{oc}$ is the least of the elastic compression member buckling load in flexural, torsional or flexural-torsional buckling, and can be calculated as follows:
where $A_g$ is the gross area of the cross-section and $f_{oc}$ is the elastic buckling stress.

For the back-to-back channel section considered herein, the elastic buckling $f_{oc}$ is the minimum of $f_{ox}, f_{oy},$ and $f_{oz}$ where:

\[
\begin{align*}
  f_{ox} &= \frac{\pi^2 E}{(L_{ex}/r_x)^2} \\
  f_{oy} &= \frac{\pi^2 E}{(L_{ey}/r_y)^2} \\
  f_{oz} &= \frac{GJ}{A_g r_{0i}^2} \left(1 + \frac{\pi^2 EI_{ez}}{GJ L_{ez}^2}\right) \\
  r_{0i} &= \sqrt{r_x^2 + r_y^2 + x_0^2 + y_0^2}
\end{align*}
\]

where

$L_{ex}, L_{ey}, L_{ez}$ are the effective lengths for buckling about the x- and y-axes, and for twisting, respectively.

$r_x, r_y$ are the radii of gyration of the cross-section about the x- and y-axes, respectively.

$x_0, y_0$ are the coordinates of the shear centre of the cross-section. For a doubly symmetric section $x_0 = y_0 = 0.$

5.2.2.3. Member in bending

The elastic buckling moment, $M_o,$ for the back-to-back channel section can be calculated from:

\[
M_o = C_b A_g r_{0i} \sqrt{f_{oy} f_{oz}}
\]

where

$C_b$ is a factor accounting for bending moment distributions in the laterally unbraced segments and is given by,

\[
C_b = \frac{12.5 M_{\text{max}}}{2.5 M_{\text{max}} + 3 M_3 + 4 M_4 + 3 M_5}
\]

$M_{\text{max}}, M_3, M_4, M_5$ are the absolute values of the maximum moment, the moment at the quarter-point, the moment at mid-point, and the moment at three-quarter point of the unbraced segment, respectively.
Based on the calculated effective lengths and the above equations, the elastic buckling load ($N_{oc}$) and the elastic lateral-torsional buckling moment ($M_o$) are determined and provided in Table 5.3.

Table 5.3. Elastic global buckling load and moment based on the effective length method

<table>
<thead>
<tr>
<th>Test Series</th>
<th>$N_{oc}$ kN</th>
<th>$M_o$ kNm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frame Test 1</td>
<td>113.26</td>
<td>22.12</td>
</tr>
<tr>
<td>Frame Test 2</td>
<td>113.26</td>
<td>22.12</td>
</tr>
<tr>
<td>Frame Test 3</td>
<td>113.26</td>
<td>22.38</td>
</tr>
<tr>
<td>Frame Test 4</td>
<td>123.28</td>
<td>91.10</td>
</tr>
<tr>
<td>Frame Test 5</td>
<td>123.28</td>
<td>96.58</td>
</tr>
<tr>
<td>Frame Test 6</td>
<td>113.26</td>
<td>22.54</td>
</tr>
<tr>
<td>Frame Test 7</td>
<td>123.28</td>
<td>91.10</td>
</tr>
</tbody>
</table>

5.2.3. The elastic lateral-torsional buckling moment ($M_o$) based on the energy method

It was proved that the knee brace serving as an intermediate discrete elastic torsional restraint contributes significantly to the buckling capacity of the column, and thus results in a substantial increase of the frame ultimate strength [257]. However, the effects of the knee brace connection are not explicitly reflected in the current design codes. It may be considered through the use of effective length, but no proper guidelines are available to correctly quantify the effective length accounting for the effects of this connection. As a result, the degree of torsional restraints provided by the knee brace is often neglected in the determination of the effective lengths, as presented in the previous section. By using the energy method, Blum and Rasmussen [257] successfully calculated the column buckling capacity considering the influence of the knee brace-to-column connection. In this section, similar determination of the column buckling capacity for Frame Tests 1, 2, 3, and 6 with unbrace columns using the energy approach is described.

5.2.3.1. Internal actions of the column

5.2.3.1.1. Frames subjected to gravity loads

A beam element model of the central frame in the tests where the columns were out-of-plane unbraced is considered as shown in Figure 5.1. This is a symmetrical plane frame subjected to uniformly distributed vertical load, $q$, on the rafters. The portal frame has a span of $2L_s$ with the length of the columns of $h_c$, and the total height from the base to the apex of $h_t$. The knee braces with the length of $L_k$ are connected to the column at a distance of $\beta h_c$ from the base, in which $\beta$ is the ratio of the vertical distance between the column base and the knee
connection to the total length of the column. The rafters are inclined at an angle of \( \alpha \) from the horizontal, and the angle between the column and the knee brace is \( \theta \).

It is assumed that the connections between all members are pinned connections. In fact, the connections between members are likely semi-rigid connections. However, as discussed previously, the actual spring stiffnesses of these connections have negligible impacts. Further, the column bases are modelled as semi-rigid connections defined by an in-plane flexural spring stiffness \( k_b \) as shown in Figure 5.1. Based on the base connection tests, as described in Chapter 3, the in-plane flexural stiffness of the bases, \( k_b \), is equal to 912.69 kNm/rad.

Other symbols used to represent the frame geometry and layout are defined as follows:

\( S_1 \) = the distance between the eave connection and knee brace to rafter connection.

\( S_2 \) = the distance between the knee brace to rafter connection and rafter tie-to-rafter connection.

\( S_3 \) = the distance between the rafter tie-to-rafter connection and apex connection

\( h_1 \) = the vertical distance between the eave connection and knee brace to rafter connection.

\( h_2 \) = the vertical distance between the knee brace to rafter connection and rafter tie-to-rafter connection.

\( h_3 \) = the vertical distance between the rafter tie-to-rafter connection and apex connection

\( L_{t1} \) = the horizontal distance between the eave connection and knee brace to rafter connection.

\( L_2 \) = the horizontal distance between the knee brace to rafter connection and rafter tie-to-rafter connection.

\( L_{rt} \) = the length of the rafter tie used to brace the apex connection

The values of these notations are calculated based on the geometry of the test frames and provided as follows:

\[
\begin{align*}
L_s &= 7 \ (m) \quad h_i = 6.634 \ (m) \quad h_e = 5.4 \ (m) \\
\alpha &= 10^0 \quad \theta = 50^0 \quad \beta = 0.826 \\
S_1 &= 1.444 \ (m) \quad S_2 = 2.821 \ (m) \quad S_3 = 2.843 \ (m) \\
h_1 &= 0.249 \ (m) \quad h_2 = 0.491 \ (m) \quad h_3 = 0.494 \ (m) \\
L_s &= 1.856 \ (m) \quad L_{t1} = 1.422 \ (m) \quad L_2 = 2.778 \ (m) \\
L_{rt} &= 5.6 \ (m)
\end{align*}
\]
The 1st order analysis of the frame is performed to obtain the reactions at the bases and the internal actions of the columns. It can be recognised in Figure 5.1 that the frame is a statically indeterminate structure, and thus the use of the equilibrium equations only is not sufficient enough to obtain the solution. Therefore, the force method of analysis, using equilibrium equations and compatibility conditions, is adopted to determine the unknowns in the frame. By using the above geometric values and the force method of analysis, the unknowns including the reactions at the base \( (P, H, M) \), the horizontal reaction at the apex \( (H_A) \) and the axial force in the rafter tie \( (N_{rt}) \), as shown in Figure 5.2a, are obtained as:

\[
P = 7q \quad (5.9)
\]
\[
H = 2.950q = 0.421P \quad (5.10)
\]
\[
M = 3.669q = 0.524P \quad (5.11)
\]
\[
H_A = 20.360q = 2.909P \quad (5.12)
\]
\[
N_{rt} = 17.409q = 2.487P \quad (5.13)
\]

The free-body diagram (FBD) of the column is shown in Figure 5.2b, whilst Figure 5.2c presents the axial forces acting in the column. In the FBD of the column (Figure 5.2b), \( P_E \) and \( H_E \) are the reactions at the knee connection, and \( F \) is the compression force in the knee brace. By applying the principle of equilibrium, the values of \( F, P_E, \) and \( H_E \) are determined as shown in Equations (5.14) to (5.16). It is noted that the vertical reaction force, \( P = P_{cr} \), is the column buckling load when \( P = P_{cr} \), and thus other variables are expressed in terms of \( P \).
\[
F = \frac{H}{(1-\beta)\sin \theta} - \frac{M}{(1-\beta)h_c \sin \theta} = 2.428P \quad (5.14)
\]

\[
P_E = P - F \cos \theta = -0.561P \quad (5.15)
\]

\[
H_E = H - F \sin \theta = -1.438P \quad (5.16)
\]

The column axial force and bending moment distributions can now be determined and are presented in Figure 5.3. The mathematical expressions of the axial force and bending moment distributions along the column height are given by:

\[
N(z) = \begin{cases} 
  P & \text{for } 0 \leq z \leq \beta h_c \\
  P - F \cos \theta = -0.561P & \text{for } \beta h_c \leq z \leq h_c 
\end{cases} \quad (5.17)
\]

\[
M(z) = \begin{cases} 
  H_z - M = 0.421P_z - 0.524P & \text{for } 0 \leq z \leq \beta h_c \\
  -H_E(h_z - z) = -1.438P_z + 7.766P & \text{for } 0 \leq z \leq \beta h_c 
\end{cases} \quad (5.18)
\]
In Frame Test 6, the rafter tie was removed from the original configuration. In this case, the in-plane bending stiffness of the apex connection needs be incorporated to produce an accurate beam element model. To represent the semi-rigid condition at the apex connection, the elastic flexural stiffness $k_{f,apex}$ of 1412.5 kNm/rad, as determined from the FE model (Section 4.8.5), was assigned to the apex connection. The geometry and layout for Frame Test 6 are shown in Figure 5.4.
The same process as implemented in the original configuration was performed for Frame Test 6 to derive the column axial force and bending moment distributions. Full details are not provided herein as they are very similar to those obtained from the above expression.

5.2.3.1.2. Frames subjected to wind loads

In experiments, to simulate the wind loads acting on the frames, a horizontal load of 7.5 kN were generated at the north eave of the test frames. This horizontal load is also reproduced in the beam element model of the frames. The same portal frame as shown in Figure 5.1 is considered. For load application instead of applying uniformly distributed vertical load \( q \) on the rafters, a horizontal load, \( W \), is applied at the right eave connections (the north eave in experiments) as shown in Figure 5.5.

![Figure 5.5. Frame geometry and layout for applied wind loads](image)

Similar to the frame with vertical loads only, the 1st order analysis of the frame subjected to wind load is also performed to obtain the reactions at the bases and the internal actions of the columns. By adopting the force method of analysis, the reactions at the north base \((P_w, H_w, M_w)\), the horizontal reaction at the apex \((H^w_A)\) and the axial force in the rafter tie \((N^w_{rt})\), as shown in Figure 5.2a, are obtained as:

\[
P_w = 0.208W \\
H_w = 0.550W = 2.646P_w \\
M_w = 1.309W = 6.302P_w \\
H^w_A = 0.261W = 1.257P_w \\
N^w_{rt} = 0.712W = 3.426P_w
\]

The axial force and bending moment diagrams of the north column are also obtained and plotted in Figure 5.7. It is worth noting that the north column is considered as the critical one.
in terms of buckling analysis. Figure 5.6b and Figure 5.6c show the free-body diagram of the north column and axial forces in the north column, respectively, for the frame subjected to horizontal load, where \( F_w \) (Equation (5.24)) is the compression force in the knee brace, and \( P_{wE} \) (Equation (5.25)) and \( H_{wE} \) (Equation (5.26)) are the reactions at the knee connection.

\[
F_w = \frac{H_w}{(1-\beta)\sin \theta} - \frac{M_w}{(1-\beta)h_e \sin \theta} = 2.298W = 11.066P_w \tag{5.24}
\]

\[
P_{wE} = P_w - F_w \cos \theta = -1.270W = -6.113P_w \tag{5.25}
\]

\[
H_{wE} = H_w - F_w \sin \theta = -1.211W = -5.831P_w \tag{5.26}
\]

Figure 5.6. FBD of (a) partial frame, (b) the column, and (c) axial forces in the north column, for applied wind loads.

Figure 5.7. Internal actions of the column for applied wind loads: (a) axial force diagram; (b) bending moment diagram.
The mathematical expressions of the axial force and bending moment distributions along the column height are given by:

\[
N_w(z) = \begin{cases} 
P_w & \text{for } 0 \leq z \leq \beta h_c \\
0 & \text{for } \beta h_c \leq z \leq h_c 
\end{cases} 
\]

\[
P_w^{\beta} = -6.113P_w 
\]

\[
M_w(z) = \begin{cases} 
H_wz - M_w = 2.646P_wz - 6.302P_w & \text{for } 0 \leq z \leq \beta h_c \\
-H_w^{\beta}(h_c - z) = -5.831P_wz + 31.490P_w & \text{for } 0 \leq z \leq \beta h_c 
\end{cases} 
\]

\[\text{(5.27)}\]

\[\text{(5.28)}\]

5.2.3.2. Work done by forces acting at the knee connection

As discussed in Section 3.2, for Frame Tests unbraced laterally by girts, the columns failed in flexural-torsional buckling, which was initiated by out-of-plane movement of the knee brace. The compression force in the knee brace can be defined as the directed loading [257, 258] that may have significant effects on the buckling resistance of the column. To incorporate the effect of this directed loading into the column buckling capacity, the work done by the knee brace force needs to be calculated. This work was determined and presented by Blum and Rasmussen [257]. The following paragraphs describe the process to determine the work done on the column by the force in the knee brace.

A frame in the x-y-z coordinate system with the x-y plane is the plane of the frame is considered as shown in Figure 4.18. It is assumed that the knee brace buckles out-of-plane at an angle \(\alpha\) as presented in Figure 5.8a. The compression force in the knee brace can be decomposed into components (Figure 5.8a) as follows:

\[
\vec{F} = F_x \hat{i} + F_y \hat{j} + F_z \hat{k} 
\]

where \(\hat{i}, \hat{j}, \hat{k}\) are the unit vector in the x-, y-, and z directions, respectively; and \(F_x, F_y, F_z\) are the corresponding scalar components of the vector \(\vec{F}\).

The scalar components are determined as follows: first resolve \(\vec{F}\) into a force acting out-of-plane, \(F_z\), and a force in the frame plane, \(F_{xy}\):

\[
F_z = F \sin \alpha 
\]

\[\text{(5.30)}\]

\[
F_{xy} = F \cos \alpha 
\]

\[\text{(5.31)}\]

Then resolve \(F_{xy}\) into x- and y-components:

\[
F_x = F \cos \alpha \sin \theta 
\]

\[\text{(5.32)}\]

\[
F_y = F \cos \alpha \cos \theta 
\]

\[\text{(5.33)}\]

where \(F\) is the magnitude of \(\vec{F}\), which was determined in the preceding Section.
The knee brace is connected to the column at the web-flange junction of the back-to-back channels. As a consequence, it can be assumed that the position of the compression force $F$ on the column is at the web-flange junction. Hence, to determine the work done by the force $F$, the displacements of the column at the web-flange junction, $v_{kc}$ and $u_{kc}$, must be determined. Due to the effects of the knee brace force, the column twists and displaces laterally. The displacements of the column at the location of the knee connection are shown in Figure 5.8b, where $v_k$ and $\phi_k$ are the lateral displacement and twist of the column, respectively. Using the graphical solution, the displacements of the column at the web-flange junction, $v_{kc}$ and $u_{kc}$, are calculated in Equations (5.34) and (5.35).

$$
v_{kc} = v_k + d_s \sin \phi_k \approx v_k + d_s \phi_k \quad (5.34)
$$

$$
u_{kc} = d_s (1 - \cos \phi_k) \approx \frac{1}{2} d_s \phi_k^2 \quad (5.35)
$$

where $d_s$ is the distances from the load application point (the web-flange junction) to the shear centre of the column section. Note that small angle approximations are considered for $\phi_k$ in the above equations.

The work done by the force in the knee brace, $W_k$, is given by:

$$
W_k = F_u u_{kc} + F_v v_{kc} \quad (5.36)
$$

By substituting Equations (5.30), (5.32), (5.34), and (5.35) into Equation (5.36) and utilising that $\sin \alpha \approx \frac{v_{kc}}{L_k}$ and $\cos \alpha \approx 1$ as small-angle approximations are used for $\alpha$, the work done by the knee brace force is obtained as:

$$
W_k = \frac{F (v_k + d_s \phi_k)^2}{L_k} + \frac{1}{2} F \sin \theta d_s \phi_k^2 \quad (5.37)
$$

where $L_k$ is the length of the knee brace.
Figure 5.8. Work done by knee connection (a) projection of knee brace into x-z plane, and column resultant forces due to knee brace force and (b) column movement at knee connection (plane view) [257]

5.2.3.3. Strain energy stored in the knee brace connection plate

In the frames, the knee brace-to-column connection plate induces restraining actions which restrict the twisting of the column and thus increase its buckling resistance. This discrete torsional restraint is assumed to be elastic, in which it is characterized by its elastic stiffness. The effect of the restraint on the buckling load of the column is incorporated by accounting for the strain energy, $U_k$, stored in the knee brace to column plate, which is calculated as follows:

$$U_k = \frac{1}{2} k_\phi (\Delta \theta_{kc})^2$$

(5.38)

where $k_\phi$ is the torsional stiffness of the plate and $\Delta \theta_{kc}$ is the total rotation of the plate.

As the knee brace to column bracket is comprised of two single plates bolted together, the plate torsional stiffness can be calculated by doubling that of a single plate and is given by [257]:

$$k_\phi = 2 \left[ \frac{E t_i^3}{12(1-\nu^2)} \right] \left( \frac{h}{b} \right)$$

(5.39)

where

$E$ is the elastic modulus.

$\nu$ is Poisson’s ratio.
\( t_k \) is the thickness of a single plate. \( t_k = 6 \text{ mm} \)

\( h \) is the plate height

\( b \) is the plate width over which the plate bending rotation occurs.

The definition of the plate is presented in Figure 5.9. For the knee brace connection considered herein, the values of \( h \) and \( b \) are 137 mm and 69.5 mm, respectively.

![Figure 5.9. The knee-to-column connection plate definition](image)

The total rotation of the plate, \( \Delta \theta_{kc} \), is the sum of the rotation of the column at the knee brace connection and the rotation due to the knee brace deflection, which can be deduced from Figure 5.8a. Hence, \( \Delta \theta_{kc} \) is given by:

\[
\Delta \theta_{kc} = \phi_k + \frac{v_{kc}}{L_k \sin \theta}
\]  

(5.40)

5.2.3.4. The buckling equation for the column

The flexural-torsional buckling load of the column is to be determined using energy analysis. The total energy, \( V_T \), is the sum of the lateral, warping and uniform torsional strain energy stored in the column, the potential energy affiliated with the internal bending moment \( M(z) \) and axial force \( N(z) \), and the potential energy of the knee brace force and the energy stored in the discrete torsional spring. Therefore, the energy equation for flexural-torsional buckling of the column with doubly symmetric back-to-back channel sections is given by:

\[
\delta V_T = 0 \text{ where } V_T = \frac{1}{2} \int_0^h \left( EI_y v'^2 + EI_w \phi^2 + GJ \phi^2 \right) dz + \frac{1}{2} \int_0^h M \left( 2\phi v' \right) dz
\]

\[
- \frac{1}{2} \int_0^h N \left( v'^2 + r_s^2 \phi'^2 \right) dz - \left( \frac{F(v_k + d_S \phi_k)^2}{L_k} + \frac{1}{2} F \sin \theta d_S \phi_k^2 \right) + \frac{1}{2} k_s (\Delta \theta_{kc})^2
\]  

(5.41)

where \( r_s^2 = \frac{I_x + I_y}{A} \)
5.2.3.5. Displacement functions

The first step in solving the energy equation (5.41) to obtain the buckling load of the column is to propose the appropriate buckled shape of the column (the displacement fields). It is well known that the accuracy of the buckling load calculated by the energy method is highly dependent on the accuracy of the assumed buckled shape [75, 76]. Ideally, the displacement fields should satisfy all the boundary conditions imposed by the method of support and constraint of the structure. These boundary conditions are usually described as being either kinematic associated with geometrical constraint at the supports of the structure or static, which relate to the values of stress resultants such as moments, shears, bi-moments and warping torques at the supports. Of these, the kinematic boundary conditions are more important than the static conditions [76].

In the tests, the displacements of columns were measured at various positions along the length as described in Chapter 3. After post-processing experimental data, the twist rotations of the column at 180 mm above the base plate, at the mid-length, and at the location of 125 mm below the knee brace connection were obtained. Meanwhile, the lateral displacements were measured at mid-length, at 125 mm below the knee connection, and at the top of the column. Based on the obtained data and the deformation of the column observed during the tests (the typical buckled shapes of the columns in Frame Test 1, 2, and 6 are shown in Figure 5.10), it is indicated that the buckling displacement can be demonstrated by a limited sine series where the maximum displacement occurs at the position approximately 3.65 m from the base plate. On the basis of that, the assumed displacement fields for \( v \) and \( \phi \) satisfied the experimental measurements are given by:

\[
v = A_v \left( \sin \frac{\pi z}{L} - 0.35 \sin \frac{2\pi z}{L} \right) \\
\phi = A_\phi \left( \sin \frac{\pi z}{L} - 0.35 \sin \frac{2\pi z}{L} \right)
\]  

(5.42)  

(5.43)
5.2.3.6. The Southwell and Meck plots

In Frame Tests 1, 2, 3, and 6, where the columns were not out-of-plane braced by girts, the experimental results and the behaviour of the frames observed during the tests indicated that the column failed in flexural-torsional elastic buckling, and thus the ultimate load, $P_u$, is approximately equal to the elastic buckling load of the column.

It has been proved that it is able to determine the elastic critical load of a structure from experiments. The “Southwell plot” is a well-known technique, originally proposed in 1932 by Southwell [259], to determine the elastic flexural buckling of a column. As the column is loaded axially by an incremental load $P$, the lateral deflection $\delta$ of the column, due to flexure, is measured at a location where it is predicted to be the largest. After that a plot is made of $\delta/P$ against $\delta$, which yields a straight line. The slope of that line is equal to $1/P_{cr}$ where $P_{cr}$ is the critical buckling load. By applying this method, the elastic buckling load can be experimentally determined without a need to subject the structure to loading in the vicinity of critical [260]. Several studies, for example as in [257, 260-262], indicate that the technique can be extended to flexural-torsional buckling by potting as $\nu/P$ (or $\nu/M$) against $\nu$ or $\phi/P$ (or $\phi/M$) against $\phi$, which yield linear relationships and the inverse slopes of these lines are in good agreement with the elastic buckling load.
An alternative version of the Southwell plot proposed by Meck [263] was discussed by Mandal and Calladine [260]. In this method, two plots are made where a plot of $\nu/P$ (or $\nu/M$) against $\phi$, and another of $\phi/P$ (or $\phi/M$) against $\nu$ are considered. The slopes of the linear portions of these two plots are $1/\beta$ and $1/\alpha$, respectively. The critical buckling load can be calculated by the geometric mean of these slopes as shown in Equation (5.44):

$$P_{cr} = (\alpha \beta)^{0.5} \quad (5.44)$$

It has been shown in [257] that the Southwell and Meck plots can be used to estimate the global elastic buckling of the column in portal frames. It is also agreed that the Meck plot technique is more reliable than the Southwell plot method. However, the Southwell plot still provides a satisfactory scheme for assessing the buckling load for lateral-torsional buckling even when only $\nu$ or $\phi$ is measured, whilst the Meck plot requires both kinds of the displacement to be measured.

In this study, both Southwell and Meck plots were created for Frame Tests 1, 2, 3, and 6 and are shown in Figure 5.11 to Figure 5.14. The critical buckling loads of a column determined on the basis of the Southwell and Meck plots are provided in Table 5.4 along with the relevant column ultimate loads obtained from experiments. As evidently seen from Table 5.4, both plotting methods precisely predict the experimental column buckling loads, taken as the ultimate loads. The results can serve to validate the buckling load and moment obtained from the energy method.
Figure 5.11. Southwell and Meck plots: (a) $v/P$ against $v$; (b) $\phi/P$ against $\phi$; (c) $\phi/P$ against $v$; and (d) $v/P$ against $\phi$ for Frame Test 1.
Figure 5.12. Southwell and Meck plots: (a) $v/P$ against $v$; (b) $\phi/P$ against $\phi$; (c) $\phi/P$ against $v$; and (d) $v/P$ against $\phi$ for Frame Test 2.
Figure 5.13. Southwell and Meck plots: (a) \( \frac{v}{P} \) against \( v \); (b) \( \frac{\phi}{P} \) against \( \phi \); (c) \( \frac{\phi}{P} \) against \( v \); and (d) \( \frac{v}{P} \) against \( \phi \) for Frame Test 3
Figure 5.14. Southwell and Meck plots: (a) $v/P$ against $v$; (b) $\phi/P$ against $\phi$; (c) $\phi/P$ against $v$; and (d) $v/P$ against $\phi$ for Frame Test 6

Table 5.4. Comparison of the column buckling loads obtained from Southwell and Meck plots to the experimental column ultimate loads

<table>
<thead>
<tr>
<th>Test series</th>
<th>$P_u$ (kN)</th>
<th>$P_{cr}$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Southwell $v$</td>
<td>Southwell $\phi$</td>
</tr>
<tr>
<td>Frame Test 1</td>
<td>16.51</td>
<td>16.75</td>
</tr>
<tr>
<td>Frame Test 2</td>
<td>16.12</td>
<td>16.89</td>
</tr>
<tr>
<td>Frame Test 3</td>
<td>11.12</td>
<td>10.96</td>
</tr>
<tr>
<td>Frame Test 6</td>
<td>13.90</td>
<td>13.72</td>
</tr>
</tbody>
</table>
5.2.3.7. Buckling loads obtained by energy analysis

The column buckling load was determined by solving the energy equation (5.41) with the utilisation of the displacement functions for lateral displacement $v$ and twisting $\phi$ given in Equations (5.42) and (5.43), respectively. By using the non-trivial solutions as shown in Equation (5.45), the results of the energy analyses were obtained and given in Table 5.5 along with the experimental column ultimate loads.

\[
\frac{\partial V_I v}{\partial A_v} - \frac{\partial V_I \phi}{\partial A_\phi} = 0 \tag{5.45}
\]

<table>
<thead>
<tr>
<th>Test Series</th>
<th>$P_u$ kN</th>
<th>$P_{cr}$ kN</th>
<th>% difference</th>
<th>$M_o$ kNm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frame Test 1</td>
<td>16.51</td>
<td>15.78</td>
<td>4.41</td>
<td>21.38</td>
</tr>
<tr>
<td>Frame Test 2</td>
<td>16.12</td>
<td>15.78</td>
<td>2.11</td>
<td>21.38</td>
</tr>
<tr>
<td>Frame Test 3</td>
<td>11.12</td>
<td>10.82</td>
<td>2.70</td>
<td>23.22</td>
</tr>
<tr>
<td>Frame Test 6</td>
<td>13.90</td>
<td>13.80</td>
<td>0.72</td>
<td>21.99</td>
</tr>
</tbody>
</table>

As seen in Table 5.5, the difference between the experimental ultimate loads and the energy method critical loads is less than 5% for all Frame Tests considered herein. The energy method also predicted the critical buckling loads comparable to that obtained from the Southwell and Meck plotting methods, as presented in the preceding section. It means that the aforementioned energy method is considered as a suitable one to predict the critical buckling loads and thus the elastic lateral-torsional buckling moment ($M_o$) can be obtained. The global buckling bending moment ($M_o$) is the maximum moment in the column when $P = P_{cr}$, which occurs at the location of the knee brace-to-column connection, as shown in Figure 5.3 and Figure 5.7. For the frames subjected to vertical loads only, the critical bending moment ($M_o$) can be calculated from $P_{cr}$ by using Equations (5.18), whilst for the Frame Test 3, which subjected to the combination of vertical and horizontal loads, the critical bending moment ($M_o$) in the column can be calculated by adding the bending moment caused by the fixed horizontal load of 7.5 kN as calculated by Equation (5.28). The respective elastic flexural-torsional buckling moments for the column are provided in Table 5.5.

5.2.4. The determination of elastic buckling load ($N_{oe}$) for the compression member undergoing flexural, torsional or flexural-torsional buckling and the elastic lateral-torsional buckling moment ($M_o$) based on the buckling analysis.

In the previous section, the energy approach of analysing lateral-torsional buckling was presented in a form which is suitable for hand use. Such the method is limited to problems where their solutions do not require excessive computational effort for the analyst. As a result,
the problems must be comparatively simple with moderate acceptable accuracy of solutions, and thus it does not allow a wide range of structures to be solved with an order of accuracy. In lieu of performing the hand use of energy method, the computer method for elastic buckling analysis, which can be applied to a very wide range of problems with a high level of accuracy, can also be used to determine the global elastic buckling load ($N_{oe}$) and the elastic lateral-torsional buckling moment ($M_o$) for the columns in the portal frames. For this purpose, the non-commercial computer program MASTAN2 [13] was utilised to perform elastic buckling analyses for the frame models of CRA portal frames. Full details of the frame modelling are described in the following sections.

5.2.4.1. Frame modelling

The portal frames with the same geometry and configurations as described in Chapter 3 and Chapter 4 are considered and summarised in Table 5.6. The buckling analysis in three-dimensional space was carried out using MASTAN2. For this, beam element models of the frame tests were developed in MASTAN2 using the schematic frame model as shown in Figure 5.15. Only the central frames were created while the end frames and purlins were not modelled. By this way, the models are simplified as often seen in design situations in the practice, so that the analysis can be done quickly and efficiently. However, the effects of lateral restraints on the rafters provided by purlins were still considered. The lateral pinned supports were assigned to the locations on the rafters coinciding with the location of the purlins. Section properties were defined and assigned for all members in the models as given in Table 5.7. In fact, the stiffness at the member ends was enhanced thanks to the contribution of the brackets used in the connections. The stiffness enhancement can be modelled by using properties of the compound sections consisting of member sections and bracket sections. In the models, only the effects of apex and base brackets were considered as the contribution of the others is insignificant (Figure 5.15). In Frame Test 7, sleeve stiffeners with a length of 2.4 m were placed in the column. The experimental results indicated that full composite action between the constituent channel sections can be assumed for calculations. Therefore, the properties of composite sections including the back-to-back column sections and stiffeners sections were calculated and assigned to the stiffened parts of the columns.

For material properties, the measured elastic modulus of 70249 MPa, the yields stress of 210.41 MPa obtained from the AC25025 longitudinal flat coupon tests, and Poisson’s ratio of 0.3 were used to assign to all elements in the models.
Table 5.6. Categories of the frame models

<table>
<thead>
<tr>
<th>Frame models</th>
<th>Loading</th>
<th>Configuration</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frame Test 1</td>
<td>Gravity Load</td>
<td>Original frame system</td>
</tr>
<tr>
<td>Frame Test 2</td>
<td>Gravity Load</td>
<td>Original frame system</td>
</tr>
<tr>
<td>Frame Test 3</td>
<td>Gravity Load + Wind Load</td>
<td>Original frame system</td>
</tr>
<tr>
<td>Frame Test 4</td>
<td>Gravity Load</td>
<td>Original frame system added girts</td>
</tr>
<tr>
<td>Frame Test 5</td>
<td>Gravity Load + Wind Load</td>
<td>Original frame system added girts</td>
</tr>
<tr>
<td>Frame Test 6</td>
<td>Gravity Load</td>
<td>Modified frame system</td>
</tr>
<tr>
<td>Frame Test 7</td>
<td>Gravity Load</td>
<td>Original frame system added girts and sleeve stiffeners</td>
</tr>
</tbody>
</table>

The connections between all members were assumed to be pinned connections as discussed in previous sections. An exception was considered in Frame Test 6 where the test frame configuration was modified by removing the rafter tie member from the original frame. In this case, to represent the semi-rigid condition at the apex connection, the elastic flexural stiffness $k_{f,\text{apex}}$ of 1412.5 kNm/rad, determined from the FE model as described in Section 4.8.5, was assigned to the member ends at the apex nodes (the schematic frame model can be seen in Figure 5.16).

The connections between all members were assumed to be pinned connections as discussed in previous sections. An exception was considered in Frame Test 6 where the test frame configuration was modified by removing the rafter tie member from the original frame. In this case, to represent the semi-rigid condition at the apex connection, the elastic flexural stiffness $k_{f,\text{apex}}$ of 1412.5 kNm/rad, determined from the FE model as described in Section 4.8.5, was assigned to the member ends at the apex nodes (the schematic frame model can be seen in Figure 5.16).

Support conditions at the bases include all translation degrees of freedom restrained while elastic rotational restraints were implemented on the rotational degrees of freedom by assigning the respective flexural stiffness, $(k_{f,\text{base}})$, about the major and minor axes obtained from the base connection tests. The warping condition at the bases was assumed to be “free warping”. In
experiments, the warping condition was neither “fully restrained” nor “free” but “partially restrained”. However, since only the flanges of the columns were bolted to the base brackets and the base brackets themselves were quite flexible, the warping free condition at the bases would be more realistic.

Figure 5.16. Schematic frame model for Frame Test 6

Table 5.7. Section properties of frame members

<table>
<thead>
<tr>
<th>Members</th>
<th>Area (mm$^2$)</th>
<th>$I_{11}$ (mm$^4$)</th>
<th>$I_{22}$ (mm$^4$)</th>
<th>$J$ (mm$^4$)</th>
<th>$I_w$ (mm$^6$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Columns/Rafters</td>
<td>2181.35</td>
<td>20.638×10$^6$</td>
<td>3.203×10$^6$</td>
<td>4545</td>
<td>3.463×10$^{10}$</td>
</tr>
<tr>
<td>Knee Braces</td>
<td>1519.12</td>
<td>5.540×10$^6$</td>
<td>1.870×10$^6$</td>
<td>3165</td>
<td>7.141×10$^9$</td>
</tr>
<tr>
<td>Rafter tie</td>
<td>1934.12</td>
<td>12.347×10$^6$</td>
<td>3.200×10$^6$</td>
<td>4029</td>
<td>2.303×10$^{10}$</td>
</tr>
<tr>
<td>Apex end section</td>
<td>6608.46</td>
<td>55.319×10$^6$</td>
<td>8.163×10$^6$</td>
<td>164370</td>
<td>6.175×10$^{10}$</td>
</tr>
<tr>
<td>Base end section</td>
<td>4765.35</td>
<td>65.335×10$^6$</td>
<td>8.820×10$^6$</td>
<td>59670</td>
<td>2.192×10$^{11}$</td>
</tr>
<tr>
<td>stiffened section</td>
<td>4536.00</td>
<td>40.560×10$^6$</td>
<td>5.575×10$^6$</td>
<td>11610</td>
<td>5.216×10$^{10}$</td>
</tr>
</tbody>
</table>

5.2.4.2. Elastic buckling analysis

The buckling analysis was performed for all frame configurations to obtain the elastic buckling load ($N_{oc}$) for the column in compression undergoing flexural, torsional or flexural-torsional buckling and the elastic lateral-torsional buckling moment ($M_{oc}$) for the column in bending.
To simulate a column in the frames subjected to compression, a vertical concentrated load was applied to a column while no loads were applied to other members. However, to determine the elastic buckling moment ($M_o$) for a column in bending, the applied loads were remodelled for each frame test. For Frame Tests under gravity only, vertical concentrated loads were applied to the nodes coincident with the purlin positions as seen in experiments. Meanwhile, for Frame Tests subjected to both vertical and horizontal loads, in addition to the vertical loads, a horizontal concentrated force was applied to the north eave of the frames.

The elastic buckling analyses were carried out on the respective models. The lowest elastic buckling load ($N_{oc}$) and the flexural-torsional buckling moment ($M_o$) attained by buckling analysis are provided in Table 5.8.

Table 5.8. Elastic global buckling loads and moments obtained from buckling analysis

<table>
<thead>
<tr>
<th>Test Series</th>
<th>$N_{oc}$ kN</th>
<th>$M_o$ kNm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frame Test 1</td>
<td>117.30</td>
<td>23.46</td>
</tr>
<tr>
<td>Frame Test 2</td>
<td>117.30</td>
<td>23.46</td>
</tr>
<tr>
<td>Frame Test 3</td>
<td>117.30</td>
<td>25.82</td>
</tr>
<tr>
<td>Frame Test 4</td>
<td>135.29</td>
<td>87.52</td>
</tr>
<tr>
<td>Frame Test 5</td>
<td>135.29</td>
<td>93.77</td>
</tr>
<tr>
<td>Frame Test 6</td>
<td>123.29</td>
<td>24.81</td>
</tr>
<tr>
<td>Frame Test 7</td>
<td>135.29</td>
<td>87.69</td>
</tr>
</tbody>
</table>

The main advantage of the analysis method is that $N_{oc}$ and $M_o$ can be determined directly without a need to calculate the effective length factors, which are not easy to accurately quantify. Especially, when the effects of the intermediate discrete elastic torsional restraints and restraint conditions at the member ends due to semi-rigid behaviour of the connection need to be considered. By using the analysis method, the results as given in Table 5.8 provide pure elastic buckling load ($N_{oc}$) in columns, but the buckling moment ($M_o$) was coupled with axial compression in columns. However, the magnitude of the axial force was relatively small, so that the value of $M_o$ can be used for design calculations.

5.2.5. Summary

Various approaches to the determination of the elastic compression member buckling load in flexural, torsional, and flexural-torsional buckling ($N_{oc}$) and the elastic lateral-torsional buckling moment ($M_o$) for the column in the portal frame systems were presented. The results of these methods are summarised in Table 5.9. It is noted that the aforementioned energy method only provided the elastic lateral-torsional buckling moment ($M_o$), and thus the elastic compression member buckling load ($N_{oc}$) obtained from elastic buckling analysis is assumed to
be used for the energy method. These results will be used in the design of portal frames, as fully described in the following Sections. In the subsequent sections, the calculations related to \( N_{oc} \) and \( M_o \) obtained from effective length method are referred to as “ELM”, that using energy method as “ENG”, and that using elastic buckling analysis as “EBA”.

<table>
<thead>
<tr>
<th>Test Series</th>
<th>Effective length method (ELM)</th>
<th>Energy method (ENG)</th>
<th>Elastic buckling analysis (EBA)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( N_{oc} )</td>
<td>( M_o )</td>
<td>( N_{oc} )</td>
</tr>
<tr>
<td>Frame Test 1</td>
<td>113.26</td>
<td>22.12</td>
<td>117.30</td>
</tr>
<tr>
<td>Frame Test 2</td>
<td>113.26</td>
<td>22.12</td>
<td>117.30</td>
</tr>
<tr>
<td>Frame Test 3</td>
<td>113.26</td>
<td>23.38</td>
<td>117.30</td>
</tr>
<tr>
<td>Frame Test 4</td>
<td>123.28</td>
<td>91.10</td>
<td>-</td>
</tr>
<tr>
<td>Frame Test 5</td>
<td>123.28</td>
<td>96.58</td>
<td>-</td>
</tr>
<tr>
<td>Frame Test 6</td>
<td>113.26</td>
<td>22.54</td>
<td>123.29</td>
</tr>
<tr>
<td>Frame Test 7</td>
<td>123.28</td>
<td>91.10</td>
<td>-</td>
</tr>
</tbody>
</table>

### 5.3. Evaluation of Current Design Standards/Specifications

The determination of frame strength is one of the key targets in the design of the portal frame system. Currently, aluminium structures are usually designed on the basis of conventional design methods (component-based), where the design process consists of two steps: (1) structural analysis using either the first-order analysis with the moment amplification to account for the second-order effects or second-order elastic analysis directly to obtain the internal actions such as bending moments, axial forces and shear forces followed by (2) design safety checks to verify that each component satisfies the design requirements based on current standards/specifications. The current standards/specifications considered herein include the Australian/ New Zealand standard AS/NZS 1664.1: 1997 [197], the American Aluminium design manual AA2015 [198], and the European code BS EN: 1999-2007 [194]. All these standards are for the aluminium sections using the extrusion process, whilst there are no available design guidelines for CRA structures except some recommendations provided in the part 1-4 of the European code BS EN: 1999-2007 for cold-formed structural sheeting. However, the use of these codes to predict the strength of the frames is necessary to assess the suitability of these standards for the design of the CRA structures. Besides, the AS/NZS 4600:2018 standard for cold-formed steel (CFS) structures, which show the similar behaviours as that made of CRA sections, and the DSM for CRA alloy members recently proposed by researchers at the University of Sydney [25, 205] are also evaluated.
5.3.1. Internal actions

The frame models were developed in MASTAN2 comprised of beam elements to determine the internal actions of the frames. The modelling details of all frame tests are provided in Section 5.2.4.1. The frame imperfections (out-of-plumb) have a profound effect on the frame strengths since it may increase second-order effects on the members. Therefore, the frame imperfections need to be included in the structural models. The MASTAN2 program allows users to introduce the initial geometric imperfections directly by updating the perfect frame geometry to the desired imperfect geometry. This was achieved by two steps: (1) performing the analysis of the perfectly geometric frames with a notional horizontal concentrated force applied to one of the eaves followed by (2) updating the geometry by scaling the deflected shape with regard to the eave node where required initial sway of $\phi = 1/268$, as discussed in Section 4.8.1, was imposed.

After modelling the frame tests, the second-order analysis was performed to determine the bending moments, axial forces, and shear forces of members in the frames with respective load combinations according to the Australian/New Zeland standard AS/NZS 1170.0:2002 [264] as follows:

- $1.2G + 1.5Q$, for Frame Tests 1, 2, 4, 6 and 7 with gravity load only
- $1.2G + W_u$, for Frame Tests 3 and 5 where both gravity and wind loads were applied in which $G$, $Q$, and $W_u$ are the dead load, live load, and wind load, respectively.

The design loads and load combinations in accordance with AS/NZS 1170.0:2002 were calculated as follows:

1. Dead load [265]:

- **Sheeting**: 0.7 mm thick Permalite Alspan sheeting 2.679 kg/m$^2$ [266]:

$$w_{G1} = 2.679 \times 9.81 \times 10^{-3} = 0.0263 \left( kN/m^2 \right) \quad (5.46)$$

- **Purlins**: AC25025 2.046 kg/m [267]:

$$w_{G2} = \frac{2.046 \times 9.81 \times 10^{-3}}{1.75} = 0.0115 \left( kN/m^2 \right) \quad (5.47)$$

- **Rafters, packing plates, and brackets**: 11.669 kg/m (measured from experiments):

$$G_R = 11.669 \times 9.81 \times 10^{-3} = 0.114 \left( kN/m \right) \quad (5.48)$$

2. Live load [265]:

- **Live load on roof**
\[ w_Q = \max \left( \frac{1.8}{A} + 0.12; 0.25 \right) = 0.25 \left( kN/m^2 \right) \]  \hspace{1cm} (5.49)

where \( A \) is the plan projection of the surface area of the roof.

- **Concentrated load at the ridge:**

\[ Q_{\text{Ridge}} = 1.4 \left( kN \right) \]  \hspace{1cm} (5.50)

3. **Wind load** [268]:

- **Site wind speed**

\[ V_{n.i.p} = V_R M_d \left( M_{z, \text{cat}} M_s M_t \right) \]  \hspace{1cm} (5.51)

where:

- \( V_R \) is the regional wind speed
- \( M_d \) is the wind direction multiplier
- \( M_{z, \text{cat}} \) is the terrain/height multiplier
- \( M_s \) is the shielding multiplier
- \( M_t \) is the topographic multiplier

+ **The regional wind speed \( V_R \)**

The regional wind speed \( V_R \) is determined based on exceedance probability. It was assumed that the frames are located in “Region A2” and designed with the importance level 1. Hence, the annual probability of exceedance of the non-cyclonic wind speed of 1:100 for the ultimate limit state can be adopted. As a result, \( V_R = 41 \text{ m/s} \) is used to determine the site wind speed.

+ **The wind direction multiplier \( M_d \)**

For structures in Region A and their final orientations are unknown, the wind direction multiplier \( M_d = 1 \) is used.

+ **The terrain/height multiplier \( M_{z, \text{cat}} \)**

It was assumed that the terrain category of the frames, considered herein, is TC3 (industrial area) so that the terrain multiplier of 0.83 was applied for wind speed calculations.

+ **The shielding multiplier \( M_s \)**

The shielding multiplier is a local development effect, which reduces the design wind speed by taking into account the protection afforded by upwind local building. In this case, a multiplier of 1.0 was applied.
It was assumed that terrain around the structure is flat so that $M_t = 1.0$ was adopted. Hence,

$$V_{sit,po} = 41 \times 1.0 \times (0.83 \times 1.0 \times 1.0) = 34.03 \text{ (m/s)}$$

(5.52)

- **Design wind speed**

According to Clause 2.3 of AS/NZS 1170.2:2011 [268], the design wind speed, $V_{des}$, is the maximum cardinal direction site wind speed, and must not be less than 30 m/s. Therefore, a design wind speed, $V_{des}$, of 35 m/s was utilised for the ultimate limit state.

- **Design wind pressure for ultimate limit state**

$$w_u = 0.6V_{des}^2 \times 10^{-3} = 0.735 \text{ (kN/m}^2\text{)}$$

(5.53)

4. **Load combination 1.2G+1.5Q [264]:**

$$1.2G + 1.5Q = \begin{cases} 
2.69 \text{ (kN)} \text{(Concentrated load at purlin locations)} \\
0.14 \text{ (kN/m)} \text{(Uniform downward loading)} \\
2.1 \text{ (kN)} \text{(Concentrated load at the ridge)} 
\end{cases}$$

(5.54)

5. **Load combination 1.2G+W_u [264]:**

$$1.2G + W_u = \begin{cases} 
0.29 \text{ (kN)} \text{(Concentrated load at purlin locations)} \\
0.14 \text{ (kN/m)} \text{(Uniform downward loading)} \\
7.5 \text{ (kN)} \text{(Concentrated load at the north eaves)} 
\end{cases}$$

(5.55)
5.3.2. Design capacities of members in compression and bending

5.3.2.1. Australian/New Zealand Standard AS/NZS 1664.1:1997

The AS/NZS 1664.1:1997 [197] provides design formulae for aluminium alloy members in compression and bending. The design compression and bending capacities are determined on the basis of the limit state stresses as described below for the respective Sections.

5.3.2.1.1. Design capacity in compression

The member capacity of a member in compression \( N_c \) according to AS/NZS 1664.1:1997 can be calculated as follows:

\[
N_c = A_g F_L
\]  

(5.56)

where \( A_g \) is the gross cross-sectional area and \( F_L \) is the limit state design stress. The limit state design stress \( F_L \) is the minimum of the limit state stress of a member in compression due to global buckling \( F_{Le} \) and local buckling \( F_{Ll} \).

5.3.2.1.1.1. Member buckling

The limit state stress of a member in compression \( F_{Le} \) for flexural, torsional or flexural-torsional buckling is given by:

\[
F_{Le} = \phi_{cc} F_{cy} / k_c \quad \text{for} \quad \lambda \leq S^*_1
\]

\[
F_{Le} = \phi_{cc} (B_c - D_c \lambda) \quad \text{for} \quad S^*_1 < \lambda < S^*_2
\]

\[
F_{Le} = \phi_{cc} \left( \lambda^2 \right) F_{cy} \quad \text{for} \quad \lambda \geq S^*_2
\]

(5.57)

in which

\( S^*_1, S^*_2 \) are the slenderness limits and are given by:

\[
S^*_1 = \frac{B_c - \frac{F_{cy}}{k_c}}{D_c}
\]  

(5.58)

\[
S^*_2 = \frac{C}{\pi} \sqrt{\frac{F_{cy}}{E}}
\]  

(5.59)

\( \phi_{cc} \) is the capacity factor for compression members which accounts for the uncertainties inherent in the determination of the limit state stress. Since both the section properties and material properties used in design calculations are measured values, \( \phi_{cc} \) is taken as unity.
\[ D_e^* = \pi D_s \sqrt{\frac{E}{F_{cy}}} \]  

(5.60)

\(B_c, C_c, \) and \(D_c\) are the buckling constants (Table 3.3(C) of AS/NZS 1664.1:199)

\(k_c\) is the coefficient for compression member (Table 3.4(B) of AS/NZS 1664.1:199)

\(\lambda\) is the equivalent slenderness, which is calculated as:

\[
\lambda = \left( \frac{kL}{r} \right) \left( \frac{1}{\pi} \right) \sqrt{\frac{F_{cy}}{E}}
\]  

(5.61)

\(F_{cy}\) is the compressive yield strength

\[
\left( \frac{kL}{r} \right) \text{ can be calculated as:}
\]

\[
\left( \frac{kL}{r} \right) = \pi \sqrt{\frac{E}{F_e}}
\]  

(5.62)

\(F_e\) is the elastic critical stress determined as follows:

\[
F_e = \frac{N_{oc}}{A_g}
\]  

(5.63)

\(N_{oc}\) is the elastic global buckling load of the member in compression (Section 5.2.5).

5.3.2.1.1.2. Local buckling

The limit state stress of a member in compression \((f_{ll})\) for local buckling can be calculated by using the average weight method, as follows:

\[
F_{ll} = \frac{\sum_{i=1}^{n} F_{Li} A_i + F_{cy} \left( A_g - \sum_{i=1}^{n} A_i \right)}{A_g}
\]  

(5.64)

where \(F_{Li}\) is the local buckling stress of element \(i\) and \(A_i\) is the area of element \(i\)

The local buckling stress of elements are determined depending on the element type as follows:
**For flat elements supported along one edge – whose buckling axis is an axis of symmetry**

\[
F_L = \frac{\phi_c F_{cy}}{k_c} \quad \text{for } \frac{b}{t} < S_1
\]

\[
F_L = \phi_c \left( B_p - 5.1D_p \frac{b}{t} \right) \quad \text{for } S_1 \leq \frac{b}{t} \leq S_2
\]

\[
F_L = \frac{\phi_c k_2 \sqrt{B_p E}}{5.1b/t} \quad \text{for } \frac{b}{t} > S_2
\]

in which, the slenderness limits \( S_1 \) and \( S_2 \) are given by:

\[
S_1 = \frac{B_p - \phi_c F_{cy}}{\phi_c k_c} \quad \text{(5.66)}
\]

\[
S_2 = \frac{k_1 B_p}{5.1D_p} \quad \text{(5.67)}
\]

\( \phi_c \) and \( \phi_y \) are the capacity factors taken as unity.

\( B_p, C_p, \) and \( D_p \) are the buckling constants (Table 3.3(C) of AS/NZS 1664.1:199)

\( k_c \) is the coefficient for compression member (Table 3.4(B) of AS/NZS 1664.1:199)

\( k_1, k_2 \) are the post-buckling constants (Table 3.3(C) of AS/NZS 1664.1:199)

\( b \) is the distance from the unsupported edge of the element to toe of the fillet or bend

\( t \) is the thickness of the element

**For flat elements with both edges supported**

\[
F_L = \frac{\phi_c F_{cy}}{k_c} \quad \text{for } \frac{b}{t} < S_1
\]

\[
F_L = \phi_c \left( B_p - 1.6D_p \frac{b}{t} \right) \quad \text{for } S_1 \leq \frac{b}{t} \leq S_2
\]

\[
F_L = \frac{\phi_c k_2 \sqrt{B_p E}}{1.6b/t} \quad \text{for } \frac{b}{t} > S_2
\]

in which, the slenderness limits \( S_1 \) and \( S_2 \) are given by:
\[ S_1 = \frac{B_p - \phi_c F_{cy}}{1.6D_p} \]

\[ S_2 = \frac{k_c B_p}{1.6D_p} \]

\( \phi_c \) and \( \phi_y \) are the capacity factors taken as unity.

\( B_p, C_p, \) and \( D_p \) are the buckling constants (Table 3.3(C) of AS/NZS 1664.1:199)

\( k_c \) is the coefficient for compression member (Table 3.4(B) of AS/NZS 1664.1:199)

\( b \) is the distance between the toe of the fillet or bends of the supporting elements.

\( t \) is the thickness of the element

For flat elements with one edge supported and the other edge with stiffener (Apply when \( D_s/b \leq 0.8 \))

\[ F_L = \frac{\phi_y F_{cy}}{k_c} \]

\[ \text{or} \]

\[ F_L = F_{UT} + (F_{ST} - F_{UT})R \leq F_{ST} \]

where

\( F_{UT} \) is the limit state stress calculated as the flat element supported along one edge by neglecting the stiffener

\( F_{ST} \) is the limit state stress calculated as the flat element with both edges supported

\( R \) is a ratio to be determined as follows:

\[ R = 1.0 \quad \text{for } b/t \leq S/3 \]

\[ R = \frac{r_s}{9t \left( \frac{b/t}{S - 1/3} \right)} \leq 1.0 \quad \text{for } S/3 < b/t \leq S \]  \hspace{1cm} (5.72)

\[ R = \frac{r_s}{1.5t \left( \frac{b/t}{S + 3} \right)} \leq 1.0 \quad \text{for } S < b/t < 2S \]

\( r_s \) is the radius of gyration of the stiffener determined as follows (see Figure 3.4.10.2(A) in AS/NZS 1664.1:199):
\[ r_z = \frac{d \sin \theta}{\sqrt{3}} \quad (5.73) \]

\[ S = 1.28 \sqrt{\frac{E}{F_{cy}}} \quad (5.74) \]

5.3.2.1.1.3. Interaction between member buckling and local buckling

If the elastic local buckling stress \( F_{ol} \) is less than the member buckling stress \( F_{ce} \), the effects of local buckling on member performance should be considered. In this case, the limit state stress of the member shall not exceed:

\[ F_{ol} = \left[ \frac{\pi^2 E}{(kL/r)^2} \right]^{\frac{2}{3}} F_{ol}^{2/3} \quad (5.75) \]

in which, the elastic local buckling stress \( F_{ol} \) is the smallest elastic local buckling stress for all elements of the cross-section determined by Table 4.7.1 of the AS/NZS 1664.1:199.

5.3.2.1.2. Design capacity in bending

For the design capacity of members subjected to bending, the member capacity \( (M_b) \) according to AS/NZS 1664.1:1997 can be determined as follows:

\[ M_b = W_{el} F_L \quad (5.76) \]

where \( W_{el} \) is the elastic section modulus and \( F_L \) is the limit state design stress. The limit state design stress \( (F_L) \) is the minimum of the limit state stress of a member in bending due to member buckling \( (F_{Le}) \) and local buckling \( (F_{Ll}) \).

5.3.2.1.2.1. Member buckling

The limit state stress of a member in bending \( (F_{Le}) \) for flexural, torsional or flexural-torsional buckling is given by:

\[
F_{Le} = \phi_b F_{cy} \quad \text{for} \quad \lambda \leq S_1 \\
F_{Le} = \phi_b \left( B_c - \frac{D_c \lambda}{1.2} \right) \quad \text{for} \quad S_1 < \lambda < S_2 \\
F_{Le} = \phi_b \frac{\pi^2 E}{\left( \frac{\lambda}{1.2} \right)^2} \quad \text{for} \quad \lambda \geq S_2
\]

(5.77)

where

\( S_1, S_2 \) are the slenderness limits determined as:
\[ S_1 = \frac{1.2 \left( B_c - \frac{\phi_c F_{ty}}{\phi_b} \right)}{D_c} \]  
(5.78)

\[ S_2 = 1.2 C_c \]  
(5.79)

\( \phi_b \) and \( \phi_y \) are the capacity factors taken as unity.

\( B_c, C_c, \) and \( D_c \) are the buckling constants (Table 3.3(C) of AS/NZS 1664.1:199)

\( \lambda \) is the slenderness, given by:

\[ \lambda = \frac{L_b}{r_{ye} \sqrt{C_b}} \]  
(5.80)

\( L_b \) is the length of the beam between bracing points or between a brace point and the free end of a cantilever beam.

\( r_{ye} \) is the effective radius of gyration of the beam about the y-axis (the axis parallel to the web), given as:

\[ r_{ye} = \frac{L_b}{1.2 \pi} \sqrt{\frac{M_o}{EZ_c}} \]  
(5.81)

\( M_o \) is the elastic critical moment (Section 5.2.5)

\( Z_c \) is the section modulus for the extreme compression fibre for bending about the x-axis

\( C_b \) is a factor accounting for bending moment distributions in the laterally unbraced segments and is given by,

\[ C_b = \frac{12.5 M_{max}}{2.5 M_{max} + 3 M_3 + 4 M_4 + 3 M_5} \]  
(5.82)

\( M_{max}, M_3, M_4, M_5 \) are the absolute values of the maximum moment, the moment at the quarter-point, the moment at mid-point, and the moment at three-quarter point of the unbraced segment, respectively

5.3.2.1.2.2. Local buckling

The limit state stress of a member in bending \( (f_{Ll}) \) for local buckling can be calculated by using the average weight method in accordance with AS/NZS 1664.1:1997, as follows:
\[
F_{L,l} = \frac{F_{l,l}A_{l,l} + F_{l,cl}A_{c,l} + \frac{1}{3}F_{l,w}A_{c,w}}{A_{l,l} + A_{c,l} + \frac{1}{3}A_{c,w}}
\] (5.83)

where \(F_{l,l}, F_{l,cl}, F_{l,w}\) are the limit state compressive stresses for the compression flanges, lips and the webs while \(A_{l,l}, A_{c,l}, A_{c,w}\) are the corresponding areas.

The limit state stresses of the components are determined depending on the element type as follows:

**For flat plates supported along one edge under uniform compression**

\[
F_L = \phi_y F_{cy} \quad \text{for } \frac{b}{t} < S_1
\]

\[
F_L = \phi_b \left( B_p - 5.1D_p \frac{b}{t} \right) \quad \text{for } S_1 \leq \frac{b}{t} \leq S_2
\] (5.84)

\[
F_L = \frac{\phi_b k_2 \sqrt{B_p E}}{5.1b/t} \quad \text{for } \frac{b}{t} > S_2
\]

in which, the slenderness limits \(S_1\) and \(S_2\) are given by:

\[
S_1 = \frac{B_p - \phi_y F_{cy}}{5.1D_p}
\] (5.85)

\[
S_2 = \frac{k_1B_p}{5.1D_p}
\] (5.86)

\(\phi_b\) and \(\phi_y\) are the capacity factors taken as unity.

\(B_p, C_p,\) and \(D_p\) are the buckling constants (Table 3.3(C) of AS/NZS 1664.1:199)

\(k_1, k_2\) are the post-buckling constants (Table 3.3(C) of AS/NZS 1664.1:199)

\(b\) is the distance from the unsupported edge of the element to toe of the fillet or bend

\(t\) is the thickness of the element

**For flat elements with both edges supported under uniform compression**

\[
F_L = \phi_y F_{cy} \quad \text{for } \frac{b}{t} < S_1
\]

\[
F_L = \phi_b \left( B_p - 1.6D_p \frac{b}{t} \right) \quad \text{for } S_1 \leq \frac{b}{t} \leq S_2
\] (5.87)

\[
F_L = \frac{\phi_b k_2 \sqrt{B_p E}}{1.6b/t} \quad \text{for } \frac{b}{t} > S_2
\]

in which, the slenderness limits \(S_1\) and \(S_2\) are given by:
\[
S_1 = \frac{B_p - \phi_y F_{cy}}{\phi_b 1.6D_p}
\]

(5.88)

\[
S_2 = \frac{k_i B_p}{1.6D_p}
\]

(5.89)

\(\phi_c\) and \(\phi_y\) are the capacity factors taken as unity.

\(B_p, C_p,\) and \(D_p\) are the buckling constants (Table 3.3(C) of AS/NZS 1664.1:199)

\(b\) is the distance between the toe of the fillet or bends of the supporting elements.

\(t\) is the thickness of the element

For flat elements with one edge supported and the other edge with stiffener under uniform compression

\[
F_L = \phi_y F_{cy}
\]

or

\[
F_L = F_{UT} + (F_{ST} - F_{UT})R \leq F_{ST}
\]

(5.90)

where

\(F_{UT}\) is the limit state stress calculated as the flat element supported along one edge by neglecting the stiffener

\(F_{ST}\) is the limit state stress calculated as the flat element with both edges supported

\(R\) is a ratio to be determined as follows:

\[
R = 1.0 \quad \text{for } b/t \leq S/3
\]

\[
R = \frac{r_s}{9t \left( \frac{b/t}{S} - \frac{1}{3} \right)} \leq 1.0 \quad \text{for } S/3 < b/t \leq S
\]

(5.91)

\[
R = \frac{r_s}{1.5t \left( \frac{b/t}{S} + 3 \right)} \leq 1.0 \quad \text{for } S < b/t < 2S
\]

\(r_s\) is the radius of gyration of the stiffener determined as follows (see Figure 3.4.10.2(A) in AS/NZS 1664.1:199):

\[
r_s = \frac{d_s \sin \theta}{\sqrt{3}}
\]

(5.92)

\[
S = 1.28 \sqrt{\frac{E}{F_{cy}}}
\]

(5.93)
For flat elements with both edges supported under bending in own plane (the webs)

\[ F_L = 1.3\phi_b F_{cy} \quad \text{for} \quad \frac{h}{t} < S_1 \]
\[ F_L = \phi_b \left( B_{br} - 0.67D_{br} \frac{h}{t} \right) \quad \text{for} \quad S_1 \leq \frac{h}{t} \leq S_2 \]  
\[ F_L = \frac{\phi_b k_B E}{0.67 h/t} \quad \text{for} \quad \frac{h}{t} > S_2 \]  

(5.94)

in which, the slenderness limits \( S_1 \) and \( S_2 \) are given by:

\[ S_1 = \frac{B_{br} - 1.3\phi_b F_{cy}}{\phi_b - 0.67D_{br}} \]  
\[ S_2 = \frac{k_B B_{br}}{0.67 D_{br}} \]  

(5.95)

(5.96)

\( \phi_b \) and \( \phi_y \) are the capacity factors taken as unity.

\( B_{br}, C_{br}, \) and \( D_{br} \) are the buckling constants (Table 3.3(C) of AS/NZS 1664.1:199)

\( h \) is the clear web height

\( t \) is the thickness of the element

5.3.2.1.2.3. Interaction between member buckling and local buckling

If the flange’s elastic local buckling stress (\( F_{ol} \)) is less than the lateral-torsional buckling stress \( F_{be} \), the effects of local buckling on member performance should be considered. In this case, the limit state stress of the member in bending shall not exceed:

\[ M_{bi} = \left[ \frac{\pi^2 E}{\left( \frac{L_b}{1.2r_{ye}\sqrt{C_b}} \right)^2} \right]^{1/3} F_{ol}^{2/3} \]  

(5.97)

5.3.2.2. American Aluminium Design Manual AA2015

The part I of the Aluminium Design Manual [198], referred to as the “Specification for Aluminium Structures”, applies to the design of Aluminium structures, members, and connections, in which the design of members in compression and bending are presented in Chapter E and F respectively. The relevant equations from the specification provided below for the respective Sections.
5.3.2.2.1. Design capacity in compression

In accordance with Chapter E of AA2015 [197], the compressive strength of members is the least of the strengths for the limit states of member buckling \((N_{ce})\), local buckling \((N_{cl})\) and the interaction between member buckling and local buckling \((N_{ci})\).

5.3.2.2.1.1. Member buckling

The nominal member buckling strength \(N_{ce}\) is:

\[
N_{ce} = A_g F_{ce}
\]  
(5.98)

where \(F_{ce}\) is the stress corresponding to the uniform compressive strength of member without welds. \(F_{ce}\) is given by:

\[
F_{ce} = \begin{cases} 
F_{cy} & \text{for } \lambda \leq \lambda_1 \\
(B_c - D_c \lambda) \left(0.85 + 0.15 \frac{C_c - \lambda}{C_c - \lambda_1}\right) & \text{for } \lambda_1 < \lambda < \lambda_2 \\
0.85 \pi^2 E \frac{\lambda^2}{\lambda^2} & \text{for } \lambda_{eq} \geq \lambda_2
\end{cases}
\]  
(5.99)

where

\[
\lambda_1, \lambda_2 \text{ are the slenderness limits given by:}
\]

\[
\lambda_1 = \frac{B_c - F_{cy}}{D_c}
\]  
(5.100)

\[
\lambda_2 = C_c
\]  
(5.101)

\(B_c, C_c, \text{ and } D_c\) are the buckling constants (Table B. 4.1 of AA2015)

\(\lambda\) is the greatest column slenderness, which is calculated by:

\[
\lambda = \pi \sqrt{\frac{E}{F_e}}
\]  
(5.102)

\(F_e\) is the elastic critical stress determined as follows:

\[
F_e = \frac{N_{oc}}{A_g}
\]  
(5.103)

\(N_{oc}\) is the elastic global buckling load of the member in compression (Section 5.2.5).

\(F_{cy}\) is the compressive yield strength
5.3.2.2.1.2. Local buckling

According to AA2015, the local buckling strength of members in compression without welds can be determined by either the weighted average method or direct strength method (DSM).

- The weighted average local buckling strength is:

\[ N_{el} = \sum_{i=1}^{n} F_{cli} A_i + F_{cy} \left( A_g - \sum_{i=1}^{n} A_i \right) \]  

(5.104)

where

\[ F_{cli} \] is the local buckling stress of element \( i \)

\[ A_i \] is the area of element \( i \)

The local buckling stress \( F_{cli} \) is determined using Sections B.5.4.1 through B.5.4.3 of the Aluminium Design Manual and provided as follows:

For flat elements supported along one edge

The local buckling stress \( F_{el} \) corresponding to the uniform compressive strength of flat elements supported on one edge is:

\[ F_{el} = F_{cy} \quad \text{for} \quad \frac{b}{t} \leq \lambda_1 \]

\[ F_{el} = B_p - 5.0D_p \frac{b}{t} \quad \text{for} \quad \lambda_1 < \frac{b}{t} < \lambda_2 \]  

(5.105)

\[ F_{el} = \frac{k_2 \sqrt{B_p E}}{5.0b/t} \quad \text{for} \quad \frac{b}{t} \geq \lambda_2 \]

Where

\[ \lambda_1 = \frac{B_p - F_{cy}}{5.0D_p} \]  

(5.106)

\[ \lambda_2 = \frac{B_p - F_{cy}}{1.6D_p} \]  

(5.107)

For flat elements supported on both edges

The local buckling stress \( F_{el} \) corresponding to the uniform compressive strength of flat elements supported on both edges is:
\[ F_{cl} = F_{cy} \quad \text{for} \quad \frac{b}{t} \leq \lambda_1 \]
\[ F_{cl} = B_p - 1.6D_p \frac{b}{t} \quad \text{for} \quad \lambda_1 < \frac{b}{t} < \lambda_2 \]  \tag{5.108}
\[ F_{cl} = \frac{k_2 \sqrt{B_p E}}{1.6b/t} \quad \text{for} \quad \frac{b}{t} \geq \lambda_2 \]

Where
\[ \lambda_1 = \frac{B_p - F_{cy}}{1.6D_p} \]  \tag{5.109}
\[ \lambda_2 = \frac{k_2 B_p}{1.6D_p} \]  \tag{5.110}

For flat elements supported on one edge and with a stiffener on the other edge when \( D_s \leq 0.8b \)

The local buckling stress \( F_{cl} \) corresponding to the uniform compressive strength of flat elements with one edge supported and the other edge with stiffener, where their thicknesses are not greater than the stiffener’s thickness and the depth of stiffener \( D_s \leq 0.8b \), is:
\[ F_{cl} = F_{UT} + \left( F_{ST} - F_{UT} \right) \rho_{ST} \leq F_{UT} \]  \tag{5.111}

where
\( F_{UT} \) is the stress calculated as the flat element supported along one edge and neglecting the stiffener
\( F_{ST} \) is the stress calculated as the flat element with both edges supported
\( \rho_{ST} \) is the stiffener effectiveness ratio to be determined as follows:
\[ \rho_{ST} = 1.0 \quad \text{for} \quad \frac{b}{t} \leq \frac{\lambda_c}{3} \]
\[ \rho_{ST} = \frac{r_s}{9t \left( \frac{b/t - \frac{1}{3}}{\lambda_c} \right)} \leq 1.0 \quad \text{for} \quad \frac{\lambda_c}{3} < \frac{b}{t} \leq \frac{\lambda_c}{2} \]  \tag{5.112}
\[ \rho_{ST} = \frac{r_s}{1.5t \left( \frac{b/t}{\lambda_c} + \frac{3}{\lambda_c} \right)} \leq 1.0 \quad \text{for} \quad \frac{\lambda_c}{2} < \frac{b}{t} \leq \frac{2\lambda_c}{3} \]

\( r_s \) is the radius of gyration of the stiffener determined as follows (parameters defined in Figure B.5.3 in AA2015):
\[
\rho_s = \frac{d_s \sin \theta_s}{\sqrt{3}} \tag{5.113}
\]

\[
\lambda_e = 1.28 \sqrt{\frac{E}{F_{cy}}} \tag{5.114}
\]

\(D_s\) and \(d_s\) are defined in Figure B.5.3 in AA2015

As an alternative to the weighted average method, the local buckling strength of the column sections can be determined by the direct strength method as follows:

\[
N_{cl} = A_g F_{cl} \tag{5.115}
\]

where

\(F_{cl}\) is the stress corresponding to the uniform compressive strength of flat elements without welds and given by:

\[
F_{cl} = F_{cy} \quad \text{for} \quad \lambda_{eq} \leq \lambda_1
\]

\[
F_{cl} = B_p - D_p \lambda_{eq} \quad \text{for} \quad \lambda_1 < \lambda_{eq} < \lambda_2 \tag{5.116}
\]

\[
F_{cl} = \frac{k_2 B_p E}{\lambda_{eq}} \quad \text{for} \quad \lambda_{eq} \geq \lambda_2
\]

\(\lambda_1, \lambda_2\) are the slenderness limits.

\[
\lambda_1 = \frac{B_p - F_{cy}}{D_p} \tag{5.117}
\]

\[
\lambda_2 = \frac{k_1 B_p}{D_p} \tag{5.118}
\]

\(B_p, C_p,\) and \(D_p\) are the buckling constants (Table B. 4.1 of AA2015)

\(k_1\) and \(k_2\) are the post-buckling constants (Table B. 4.3 of AA2015)

\(\bar{\lambda}\) is the equivalent slenderness calculated by:

\[
\bar{\lambda} = \pi \sqrt{\frac{E}{f_{ol}}} \tag{5.119}
\]

\(f_{ol}\) is the elastic local buckling stress determined by analysis (Section 5.2.1)

5.3.2.2.1.3. Interaction between member buckling and local buckling

If the elastic local buckling stress \(f_{ol}\) is less than the member buckling stress \(F_{ce}\), the effects of local buckling on member performance should be considered. In this case, the nominal compressive strength of the member shall not exceed:
in which, the elastic local buckling stress $F_{ol}$ is the smallest elastic local buckling stress for all elements of the cross-section determined by Table B.5.1 in the AA2015 if the local buckling strength, $N_{cl}$, is determined by the weighted average method, whereas $F_{ol}$ is determined by analysis if the local buckling strength, $N_{cl}$, is determined by the DSM.

5.3.2.2.2. Design of members for bending

In accordance with Chapter F of AA2015 [198], the available flexural strength of aluminium alloy members is the least of the available strengths for the limit states of Yielding, Rupture, local buckling, flexural-torsional buckling, and the interaction of local and flexural-torsional buckling. The relevant equations using to determine the nominal flexural strengths are provided below.

5.3.2.2.2.1. Yielding and rupture

- For the limit state of yielding, the nominal flexural strength $M_{bp}$ is:

$$M_{bp} = \min \left( Z F_{cy} ; 1.5 S_t F_{ty} ; 1.5 S_c F_{cy} \right)$$

(5.121)

where

- $Z$ is the plastic section modulus
- $S_t$ is the section modulus on the tension side of the neutral axis
- $S_c$ is the section modulus on the compression side of the neutral axis
- $F_{ty}$ is the tensile yield strength
- $F_{cy}$ is the compressive yield strength

- For the limit state of rupture, the nominal flexural strength $M_{bu}$ is:

$$M_{bu} = Z F_{tu} / k_t$$

(5.122)

where

- $F_{tu}$ is the tensile ultimate strength
- $k_t$ is the tension coefficient. For aluminium alloy 5052-H36, $k_t = 1$ (Table A.3.3 of AA2015)

5.3.2.2.2.2. Lateral-torsional buckling

The nominal member moment capacity ($M_{be}$) for lateral-torsional buckling limit state is given by:
\[ M_{be} = M_{bp} \left(1 - \frac{\lambda}{C_c}\right) + \frac{\pi^2 E \lambda S_{xc}}{C_c^3} \quad \text{for } \lambda < C_c \]  
\[ M_{be} = \frac{\pi^2 ES_{xc}}{\lambda^2} \quad \text{for } \lambda \geq C_c \]  

where

- \( S_{xc} \) is the section modulus about the compressive side of the x-axis
- \( \lambda \) is the slenderness for lateral-torsional buckling, calculated by:
  \[ \lambda = \pi \sqrt{\frac{ES_{xc}}{M_o}} \]  

\( M_o \) is the elastic flexural-torsional buckling moment (Section 5.2.5)

### 5.3.2.2.2.3. Local buckling

For cross-sections composed of flat or curved elements, the nominal flexural strength \( (M_{bl}) \) for the limit state of local buckling can be determined by either the weighted average method, DSM, or limiting element method [198]. In this section, the weighted average method and the DSM are described as follows:

- The weighted average local buckling strength can be calculated as:
  \[ M_{bl} = \frac{F_c I_f}{c_{cf}} + \frac{F_{bl} I_w}{c_{cw}} \]  

where

- \( F_c \) is the stress corresponding to the strength of an element in uniform compression
- \( F_{bl} \) is the stress corresponding to the strength of an element in flexural compression
- \( c_{cf} \) is the distance from the centerline of a uniform compression element to the cross section's neutral axis
- \( c_{cw} \) is the distance from a flexural compression element's extreme compression fibre to the cross section's neutral axis
- \( I_f \) is the moment of inertia of the uniform stress elements about the cross section's neutral axis. These elements include the elements in uniform compression and the elements in uniform tension and their edge or intermediate stiffeners
- \( I_w \) is the moment of inertia of the flexural compression elements about the cross section's neutral axis.

While the relevant equations to determine \( F_c \) have been provided in Section 5.3.2.2.1.2, the equations needed to determine \( F_{bl} \) are provided below.
For flat elements supported on both edges

The local buckling stress \( F_b \) corresponding to the flexural compressive strength of flat elements supported on both edges is:

\[
F_{bl} = 1.5F_{cy} \quad \text{for} \quad \frac{b}{t} \leq \lambda_1
\]

\[
F_{bl} = B_{br} - mD_{br} \frac{b}{t} \quad \text{for} \quad \lambda_1 < \frac{b}{t} < \lambda_2
\]

\[
F_{bl} = \frac{k_2\sqrt{B_{br}E}}{mb/t} \quad \text{for} \quad \frac{b}{t} \geq \lambda_2
\]

Where

\[
\lambda_1 = \frac{B_{br} - 1.5F_{cx}}{mD_{br}} \quad (5.127)
\]

\[
\lambda_2 = \frac{k_1B_{br}}{mD_{br}} \quad (5.128)
\]

\[
m = 1.15 + \frac{c_o}{2c_c} \quad \text{for} \quad -1 < \frac{c_o}{c_c} < 1
\]

\[
m = \frac{1.3}{1-c_o/c_c} \quad \text{for} \quad \frac{c_o}{c_c} \leq -1 \quad (5.129)
\]

\[
m = 0.65 \quad \text{for} \quad c_o = -c_c
\]

\( c_c \) is the distance from the neutral axis to the element extreme fibre with the greatest compressive stress

\( c_o \) is the distance from the neutral axis to the extreme fiber of element

- The nominal flexural strength for the local buckling limit state \( (M_{bl}) \) can be determined by DSM, as follows:

\[
M_{bl} = F_{bl}S_{x_t} \quad (5.130)
\]

where

\( F_{bl} \) is the stress corresponding to the strength of an element in flexural compression

\[
F_{bl} = \frac{M_{lp}}{S_{x_t}} \quad \text{for} \quad \lambda_{eq} \leq \lambda_1
\]

\[
F_{bl} = \frac{M_{lp}}{S_{x_t}} - \left( \frac{M_{lp}}{S_{x_t}} - \frac{\pi^2E}{C_p^2} \right) \left( \frac{\lambda_{eq} - \lambda_1}{C_p - \lambda_1} \right) \quad \text{for} \quad \lambda_1 < \lambda_{eq} < \lambda_2 \quad (5.131)
\]

\[
F_{bl} = \frac{k_2\sqrt{B_{br}E}}{\lambda_{eq}} \quad \text{for} \quad \lambda_{eq} \geq \lambda_2
\]

Where
\( \lambda_1, \lambda_2 \) are the slenderness limits.

\[
\lambda_1 = \frac{B_p - F_{cy}}{D_p} \\
\lambda_2 = C_p
\]  
(5.132) \( \lambda_2 \) are the buckling limits (Table B. 4.1 of AA2015)

\[ B_p, C_p, \text{ and } D_p \] are the buckling constants (Table B. 4.3 of AA2015)

\( k_1 \) and \( k_2 \) are the post-buckling constants (Table B. 4.3 of AA2015)

\( \lambda \) is the equivalent slenderness calculated by:

\[
\lambda = \pi \sqrt{\frac{E}{f_{ol}}}
\]  
(5.134) 

\( f_{ol} \) is the elastic local buckling stress determined by analysis (Section 5.2.1)

5.3.2.2.4. Interaction between member buckling and local buckling

If the flange’s elastic local buckling stress \( (F_{ol}) \) given in Section B.5.6 of the AA2015 is less than the lateral-torsional buckling stress \( F_{be} \), the effects of local buckling on member performance should be considered. In this case, the nominal flexural strength of the member shall not exceed:

\[
M_{nu} = \left[ \sqrt{\frac{\pi^2 E}{r_{ol}} \left( \frac{L}{r_{ol} \sqrt{C_b}} \right)^2} \right]^{2/3} F_{ol}^{2/3} S_{Mc}
\]  
(5.135)

5.3.2.3. European Code BS EN: 1999-2007

The European Code BS EN: 1999-2007 [194], subdivided into five parts, provides rules to the design of buildings and civil engineering and structural works in aluminium, in which the part 4, EN 1999-1-4, presents design requirements for cold-formed trapezoidal aluminium sheeting. By considering the geometric nature of the cold-rolled aluminium back-to-back section and suitability of design requirements for the section, the design rules for determining the resistance of the cross-section in compression and bending are provided in the following Sections.

5.3.2.3.1. Design of members in compression

According to BS EN: 1999-2007 [194], the design resistance of a member in compression is determined by considering the effects of the local and distortional buckling and the member buckling.
5.3.2.3.1.1. Sectional capacity

The design resistance of a cross-section for compression \( (N_{cl}) \) should be determined from:

\[
N_{cl} = A_{eff} f_y \gamma_{M1}
\]  

(5.136)

where

- \( A_{eff} \) is the effective area
- \( f_y \) is the characteristic value of 0.2\% proof strength (the yield stress)
- \( \gamma_{M1} \) is the partial factor for compression members which accounts for the uncertainties inherent in the determination of the resistance. Since both the section properties and material properties used in design calculations are based on actual tests, \( \gamma_{M1} \) is taken as unity.

The local and distortional buckling is considered in the determination of the effective area using the effective thickness method (ETM), which is presented in Clause 5.5 of BS EN: 1999-1-4: 2007 [222].

5.3.2.3.1.2. Member capacity

The design buckling resistance of a compression member \( (N_{ce}) \) can be taken as:

\[
N_{ce} = \chi A_{eff} f_y \gamma_{M1}
\]  

(5.137)

As seen in Equation (5.137), the buckling resistance of an aluminium member is determined by multiplying the sectional resistance by a reduction factor \( \chi \). The reduction factor for a member in compression is given by:

\[
\chi = \frac{1}{\phi + \sqrt{\phi^2 - \lambda^2}} \leq 1.0
\]  

(5.138)

with

\[
\phi = 0.5 \left( 1 + \alpha \left( \overline{x} - \overline{x}_o \right) + \lambda^2 \right)
\]  

(5.139)

where

- \( \alpha \) is an imperfection factor
- \( \overline{x}_o \) is the limit of the horizontal plateau
- \( \lambda \) is the slenderness parameter for the relevant buckling mode (flexural, torsional, or flexural-torsional buckling)
In accordance with Clause 6.2.2 of BS EN: 1999-1-4: 2007 [222], the imperfection factor is \( \alpha = 0.13 \) and the limit of the horizontal plateau is \( \lambda_o = 0.2 \) while the slenderness parameter can be calculated from following:

\[
\lambda = \sqrt{\frac{A_{eff} f_y}{N_{oc}}} \tag{5.140}
\]

where

\( N_{oc} \) is the elastic global buckling load of the member in compression (Section 5.2.5).

5.3.2.3.2. Design of members subjected to bending

The design moment resistance of a member is determined by considering the effects of the local and distortional buckling and the member buckling.

5.3.2.3.2.1. Design moment resistance considered the effects of local and distortional buckling

The design moment resistance of a cross-section for bending (\( M_{bl} \)) should be determined as follows:

\[
M_{bl} = \begin{cases} 
W_{eff} \frac{f_y}{\gamma_{M1}} & \text{for } W_{eff} < W_{el} \\
\frac{f_y}{\gamma_{M1}} \left( W_{el} + \left( W_{pl} - W_{el} \right) \frac{4(1 - \lambda / \lambda_{el})}{\gamma_{M1}} \right) & \text{for } W_{eff} = W_{el}
\end{cases}
\]

where

- \( W_{eff} \) is the effective section modulus in bending
- \( f_y \) is the characteristic value of 0.2% proof stress (the yield stress)
- \( \gamma_{M1} \) is the partial factor accounting for the uncertainties inherent in the determination of the resistance. Since both the section properties and material properties used in design calculations are based on actual tests, \( \gamma_{M1} \) is taken as unity.
- \( W_{el} \) is the elastic modulus of gross section
- \( W_{pl} \) is the plastic modulus of gross section
- \( \lambda \) is the slenderness of the cross-section part which correspond to the largest value of \( \lambda / \lambda_{el} \) where \( \lambda = \lambda_p \) and \( \lambda_{el} = \lambda_{lim} \) (more details can be found in Section 5.5.2 of BS EN: 1999-1-4: 2007)

5.3.2.3.2.2. The design buckling moment resistance

The design buckling resistance moment (\( M_{be} \)) can be taken as [152]:

\[
M_{be} = \chi_{L3} A_{eff,y} \frac{f_y}{\gamma_{M1}} \tag{5.142}
\]

where
\( W_{el,y} \) is the elastic section modulus of the gross section without reduction for local buckling

\( \alpha \) is the shape factor. \( \alpha = \frac{W_{eff}}{W_{el}} \)

\( \chi_{LT} \) is the reduction factor for lateral-torsional buckling, given by

\[
\chi = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \chi_{LT}^2}} \leq 1.0
\]

\( \phi_{LT} \) = \( 0.5 \left( 1 + \alpha_{LT} \left( \chi_{LT} - \chi_{o,LT} \right) + \chi_{LT}^2 \right) \)

\( \alpha_{LT} \) is an imperfection factor

\( \chi_{o,LT} \) is the limit of the horizontal plateau

\( \chi_{LT} \) is the relative slenderness

According to Clause 6.3.2 of BS EN: 1999-1-1: 2007, the imperfection factor is \( \alpha_{LT} = 0.2 \) and the limit of the horizontal plateau is \( \chi_{o,LT} = 0.4 \) (for class 4 cross-section) while the relative slenderness parameter can be calculated from:

\[
\chi_{LT} = \sqrt{\frac{W_{eff} f_y}{M_o}}
\]

\( M_o \) is the elastic lateral-torsional buckling moment (Section 5.2.5).

5.3.2.4. Australian/New Zealand Standard AS/NZS 4600 for Cold-formed Steel Structures

In this Section, the direct strength method (DSM) in accordance with AS/NZS 4600 is presented. Although it applies to the CFS structures, the use of this method to determine the capacities of the CRA member considered herein still has certain values.

5.3.2.4.1. Design capacity in compression

The nominal capacity of a member in compression \((N_c)\) is the minimum of the nominal member capacity of a member in compression \((N_{ce})\) for flexural, torsional or flexural-torsional buckling, the nominal member capacity \((N_{cl})\) for local buckling and the nominal member capacity \((N_{cd})\) for distortional buckling (Section 7.2.1 of AS/NZS 4600)
5.3.2.4.1.1. Flexural, torsional or flexural-torsional buckling

The nominal member capacity of a member in compression \( N_{ce} \) for flexural, torsional or flexural-torsional buckling can be calculated as follows:

\[
N_{ce} = \begin{cases} 
  0.658 \frac{N_y}{\lambda_c^{0.2}} & \text{for } \lambda_c \leq 1.5 \\
  0.877 \frac{N_y}{\lambda_c^{0.2}} & \text{for } \lambda_c > 1.5
\end{cases}
\]  
(5.146)

where

\[ \lambda_c = \sqrt{\frac{N_y}{N_{ce}}} \]  
(5.147)

\( N_{ce} \) is the elastic global buckling load of the member in compression (Section 5.2.5).

5.3.2.4.1.2. Local buckling

The nominal member capacity of a member in compression \( N_{cl} \) for local buckling and the interaction between local and flexural-torsional buckling can be determined as :

\[
N_{cl} = N_{ce} \quad \text{for } \lambda_l \leq 0.776
\]
\[
N_{cl} = \left[ 1 - 0.15 \left( \frac{N_{el}}{N_{ce}} \right)^{0.4} \right] \left( \frac{N_{el}}{N_{ce}} \right)^{0.4} N_{ce} \quad \text{for } \lambda_l > 0.776
\]  
(5.148)

where

\[ \lambda_l = \sqrt{\frac{N_{ce}}{N_{el}}} \]  
(5.149)

\( N_{el} \) is the elastic local buckling load

\[ N_{el} = A_f f_{el} \]  
(5.150)

\( f_{el} \) is the elastic local buckling stress determined by analysis (Section 5.2.1)

5.3.2.4.1.3. Distortional buckling

The nominal member capacity of a member in compression \( N_{cd} \) for distortional buckling is given by:

\[
N_{cd} = N_y \quad \text{for } \lambda_d \leq 0.561
\]
\[
N_{cd} = \left[ 1 - 0.25 \left( \frac{N_{el}}{N_y} \right)^{0.6} \right] \left( \frac{N_{el}}{N_y} \right)^{0.6} N_y \quad \text{for } \lambda_d > 0.561
\]  
(5.151)

where
\[ \lambda_d = \sqrt{\frac{N}{N_{od}}} \]  

(5.152)

\( N_{od} \) is the elastic distortional buckling load

\[ N_{od} = A_g f_{od} \]  

(5.153)

\( f_{od} \) is the elastic distortional buckling stress determined by analysis (Section 5.2.1)

5.3.2.4.2. Design capacity in bending

In accordance with Section 7.2.2 of AS/NZS 4600, the nominal member moment capacity \( (M_b) \) is the minimum of the nominal member moment capacity \( (M_{be}) \) due to flexural-torsional buckling, local buckling \( (M_{bl}) \) and distortional buckling \( (M_{bd}) \).

5.3.2.4.2.1. Flexural-torsional buckling

The nominal member moment capacity \( (M_{be}) \) for flexural-torsional buckling for beams without holes can be calculated as follows:

\[ M_{be} = M_o \quad \text{for} \quad M_o < 0.56M_y \]

\[ M_{be} = \frac{10}{9} M_y \left( 1 - \frac{10M_y}{36M_o} \right) \quad \text{for} \quad 2.78M_y \geq M_o \geq 0.56M_y \]  

(5.154)

\[ M_{be} = M_y \quad \text{for} \quad M_o > 2.78M_y \]

where

\[ M_y = Z_f f_y \]  

(5.155)

\( Z_f \) is the full section modulus of the extreme fibre at first yield

\( M_o \) is the elastic lateral-torsional buckling moment (Section 5.2.5).

5.3.2.4.2.2. Local buckling

The nominal member moment capacity \( (M_{bl}) \) for local buckling and the interaction between local and flexural-torsional buckling can be determined as:

\[ M_{bl} = M_{be} \quad \text{for} \quad \lambda_c \leq 0.776 \]

\[ M_{bl} = \left[ 1 - 0.15 \left( \frac{M_{ol}}{M_{be}} \right)^{0.4} \right] \left( \frac{M_{ol}}{M_{be}} \right)^{0.4} M_{be} \quad \text{for} \quad \lambda_c > 0.776 \]  

(5.156)

where

\[ \lambda_c = \sqrt{\frac{M_{be}}{M_{ol}}} \]  

(5.157)

\( M_{ol} \) is the elastic local buckling moment
\[ M_{ol} = Z_f f_{ol} \]  

(5.158)

\( f_{ol} \) is the elastic local buckling stress determined by analysis (Section 5.2.1)

5.3.2.4.2.3. Distortional buckling

The nominal member moment capacity \( (M_{bd}) \) for distortional buckling can be calculated as follows:

\[
M_{bd} = \begin{cases} 
M_y & \text{for } \lambda_d \leq 0.673 \\
1 - 0.22 \left( \frac{M_{od}}{M_y} \right)^{0.5} \left( \frac{M_{od}}{M_y} \right)^{0.5} M_y & \text{for } \lambda_d > 0.673
\end{cases}
\]  

(5.159)

where

\[
\lambda_d = \frac{M_y}{M_{od}}
\]  

(5.160)

\( M_{od} \) is the elastic distortional buckling moment

\[ M_{od} = Z_f f_{od} \]  

(5.161)

\( f_{od} \) is the elastic distortional buckling stress determined by analysis (Section 5.2.1)

5.3.2.5. Direct Strength Method for Cold-rolled Aluminium Alloy Members Proposed by Researchers at Sydney University

Recently, several research studies on cold-rolled aluminium structural sections and members have been carried out [7, 21, 25, 71-74, 205, 269-275], in which researchers at Sydney University have presented comprehensive studies on the strength and behaviour of CRA sections and members in compression and bending. Based on the experimental and numerical investigations, the Direct Strength Method (DSM) for CRA members has been proposed on the basis of the DSM for CFS structures according to AS/NZS 4600 [146]. These design proposals are provided in this section.

5.3.2.5.1. Design of members in compression

The nominal capacity of a member in compression \( (N_c) \) is the minimum of the nominal member capacity of a member in compression \( (N_{ce}) \) for flexural, torsional or flexural-torsional buckling, the nominal member capacity \( (N_{cd}) \) for local buckling, the nominal member capacity \( (N_{od}) \) for distortional buckling and the nominal member capacity \( (N_{ci}) \) for local-global interaction buckling. Since the AS/NZS 4600 provides good predictions for member buckling strengths [205], the nominal member capacity of a member in compression \( (N_{ce}) \) for flexural, torsional or flexural-torsional buckling is determined using the relevant equations in AS/NZS 4600. For the local-global buckling, new DSM formulae as proposed by Pham, N.H. [205] are used to calculate \( N_{ci} \). In a similar manner, new DSM design formulae for local buckling and
distortional buckling, as proposed by Huynh, L.A.T. [25], are utilised. The details of the above discussions are provided below for the respective sections.

5.3.2.5.1.1. Flexural, torsional or flexural-torsional buckling according to AS/NZS 4600

In accordance with section 7.2.1.2 of AS/NZS 4600, the nominal member capacity of a member in compression \( N_{ce} \) for flexural, torsional or flexural-torsional buckling can be calculated as follows [146]:

\[
N_{ce} = \begin{cases} 
0.658 \lambda_c^2 N_y & \text{for } \lambda_c \leq 1.5 \\
0.877 \left(\frac{N_{ce}}{\lambda_c^2}\right) N_y & \text{for } \lambda_c > 1.5 
\end{cases}
\]

(5.162)

where

\[
\lambda_c = \sqrt{\frac{N_x}{N_{ce}}}
\]  

(5.163)

\( N_{ce} \) is the elastic global buckling load of the member in compression (Section 5.2.5).

5.3.2.5.1.2. Local buckling proposed by Huynh, L.A.T.

Based on the proposal by Huynh, L.A.T. [25], the nominal member capacity of a member in compression \( N_{cl} \) for local buckling can be determined as:

\[
N_{cl} = \begin{cases} 
N_y & \text{for } \lambda_l \leq 0.681 \\
1 - 0.18 \left(\frac{N_{cl}}{N_y}\right)^{0.35} \left(\frac{N_{ol}}{N_y}\right)^{0.35} N_y & \text{for } \lambda_l > 0.681 
\end{cases}
\]

(5.164)

where

\[
\lambda_l = \sqrt{\frac{N_y}{N_{ol}}}
\]  

(5.165)

\( N_{ol} \) is the elastic local buckling load

\[
N_{ol} = A_x f_{ol}
\]

(5.166)

\( f_{ol} \) is the elastic local buckling stress determined by analysis (Section 5.2.1)
5.3.2.5.1.3. Distortional buckling proposed Huynh, L.A.T.

The nominal member capacity of a member in compression \(N_{cd}\) for distortional buckling is given by [25]:

\[
N_{cd} = N_y \\
N_{cd} = \left[ 1 - 0.25 \left( \frac{N_{od}}{N_y} \right)^{0.55} \right] \left( \frac{N_{od}}{N_y} \right)^{0.55} N_y
\]

for \(\lambda_d \leq 0.532\) \hspace{1cm} (5.167)

\[
\lambda_d = \sqrt{\frac{N_y}{N_{cd}}}
\]

\(N_{od}\) is the elastic distortional buckling load

\[
N_{od} = A_y f_{od}
\]

\(f_{od}\) is the elastic distortional buckling stress determined by analysis (Section 5.2.1)

5.3.2.5.1.4. Local-global interaction buckling proposed by Pham, N.H.

The nominal member capacity of a member in compression \(N_{cl,i}\) for local-global interaction buckling is given by [205]:

\[
N_{cl,i} = N_{ce} \\
N_{cl,i} = \left( \frac{1}{\lambda_{l,i}^{0.7}} \right) - 0.18 \left( \frac{N_{ce}}{\lambda_{l,i}^{1.4}} \right) N_{ce}
\]

for \(\lambda_i \leq 0.681\) \hspace{1cm} (5.170)

\[
\lambda_{l,i} = \sqrt{\frac{N_{ce}}{N_{od}}}
\]

\(N_{ce}\) is the member buckling strength

\(N_{0l}\) is the elastic local buckling strength

\[M_{bd,i}\] for distortional-global interaction buckling (proposed by Pham, N.H.).
5.3.2.5.2.1. Lateral-torsional buckling according to AS/NZS 4600

The nominal member moment capacity ($M_{be}$) for lateral-torsional buckling for beams without holes can be calculated as follows [146]:

$$M_{be} = M_o \quad \text{for} \quad M_o < 0.56M_y$$

$$M_{be} = \frac{10}{9} M_y \left( 1 - \frac{10M_y}{36M_o} \right) \quad \text{for} \quad 2.78M_y \geq M_o \geq 0.56M_y$$

(5.172)

$$M_{be} = M_y \quad \text{for} \quad M_o > 2.78M_y$$

where

$$M_y = Z_f f_y$$

(5.173)

$Z_f$ is the full section modulus of the extreme fibre at first yield

$M_o$ is the elastic lateral-torsional buckling moment (Section 5.2.5).

5.3.2.5.2.2. Local buckling proposed by Huynh, L.A.T.

The nominal member moment capacity ($M_{bl}$) for local buckling can be determined as [25]:

$$M_{bl} = M_{yc} \quad \text{for} \quad \lambda_c \leq 0.843$$

$$M_{bl} = \left[ 1 - 0.10 \left( \frac{M_{ol}}{M_{yc}} \right)^{0.35} \right] \left( \frac{M_{ol}}{M_{yc}} \right)^{0.35} M_{yc} \quad \text{for} \quad \lambda_c > 0.843$$

(5.174)

where

$$\lambda_c = \sqrt[0.35]{\frac{M_{yc}}{M_{ol}}}$$

(5.175)

$M_{yc}$ is the yield moment on the compression side

$$M_{yc} = Z_{fc} f_y$$

(5.176)

5.3.2.5.2.3. Distortional buckling proposed by Huynh, L.A.T.

The nominal member moment capacity ($M_{bd}$) for distortional buckling can be calculated as follows [25]:

$$M_{bd} = M_{yc} \quad \text{for} \quad \lambda_d \leq 0.880$$

$$M_{bd} = \left[ 1 - 0.10 \left( \frac{M_{ol}}{M_{yc}} \right)^{0.45} \right] \left( \frac{M_{ol}}{M_{yc}} \right)^{0.45} M_{yc} \quad \text{for} \quad \lambda_d > 0.880$$

(5.177)

where
\lambda_d = \sqrt{\frac{M_{yc}}{M_{od}}} \quad (5.178)

\( M_{yc} \) is the yield moment on the compression side

\( M_{od} \) is the elastic distortional buckling moment

5.3.2.5.2.4. Distortional-global interaction buckling proposed by Pham, N.H.

The nominal moment member capacity \( (M_{bd,i}) \) for distortional-global interaction buckling is given by [205]:

\[
M_{bd-i} = M_{be} \quad \text{for} \quad \lambda_{d-i} \leq 0.532
\]

\[
M_{bd-i} = \left( \frac{0.8}{\lambda_{d-i}^{1.1}} - \frac{0.15}{\lambda_{d-i}^{2.2}} \right) M_{be} \quad \text{for} \quad \lambda_{d-i} > 0.532
\quad (5.179)

where

\[
\lambda_{d-i} = \sqrt{\frac{M_{bd-i}}{M_{od}}}
\quad (5.180)
5.3.3. The determination of member capacities

The results of the experimental and numerical investigations indicated that the failure of the frame tests was initiated with the failure of the columns. It means that the columns are the critical members of the frames. In the frames subjected to gravity loads only, both columns experienced almost identical forces and moments with the most critical region is at the knee brace connection as both axial compression and bending are significantly large at that area. Meanwhile, columns in the frames under the combination of gravity and wind loads experienced unequal forces and moments as a result of the application of constant horizontal load at the north eaves. In these cases, the critical members are the north columns. Therefore, in the present context, to quantify the capacity (strength/resistance) of the frames, the capacity check was focused on the north column.

The capacities of the north column in compression \((N_c)\) and bending \((M_b)\) were calculated on the basis the design capacity check in accordance with Section 5.3.2. All the design calculations were based on the material properties and section properties obtained from the actual tests. The capacities for the flexural, torsional or flexural torsional buckling \((N_{ce})\), local buckling \((N_{cl})\), distortional buckling \((N_{cd})\), and the local-global interaction buckling \((N_{cl-gd})\) of the north column in compression and the lateral-torsional buckling \((M_{ble})\), local buckling \((M_{bl})\), distortional buckling \((M_{bd})\), and distortional-global interaction buckling \((M_{bd-gd})\) of the north column in bending for the Frame Tests 1 and 2, serving as typical frames, are provided in Table 5.10. These capacities for other Frame Tests are provided in Appendix D.3. The capacities of the north column in compression \((N_c)\) and bending \((M_b)\) were obtained and provided in Table 5.11.

Table 5.10. Capacities of the north column for Frame Tests 1 and 2

<table>
<thead>
<tr>
<th>Design type</th>
<th>Items</th>
<th>AA2015</th>
<th>AS/NZS 1664</th>
<th>EUROCODE 9</th>
<th>AS/NZS 4600</th>
<th>PROPOSED DSM</th>
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<tbody>
<tr>
<td></td>
<td></td>
<td>ELM</td>
<td>ENG</td>
<td>EBA</td>
<td>ELM</td>
<td>ENG</td>
</tr>
<tr>
<td>Compression</td>
<td>(N_c)</td>
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<td>99.71</td>
<td>99.71</td>
<td>113.26</td>
<td>117.30</td>
</tr>
<tr>
<td></td>
<td>(N_{cl-ELM})</td>
<td>145.81</td>
<td>145.81</td>
<td>145.81</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>(N_{cl-ENG})</td>
<td>250.73</td>
<td>250.73</td>
<td>250.73</td>
<td>241.41</td>
<td>241.41</td>
</tr>
<tr>
<td></td>
<td>(N_{cd-ELM})</td>
<td>83.69</td>
<td>84.67</td>
<td>84.67</td>
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<td>-</td>
</tr>
<tr>
<td></td>
<td>(N_{cd-ENG})</td>
<td>73.41</td>
<td>74.27</td>
<td>74.27</td>
<td>77.50</td>
<td>78.41</td>
</tr>
<tr>
<td>Bending</td>
<td>(M_{bl-ELM})</td>
<td>0.406</td>
<td>0.406</td>
<td>0.406</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>(M_{bl-ENG})</td>
<td>49.13</td>
<td>49.13</td>
<td>49.13</td>
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<td>-</td>
</tr>
<tr>
<td></td>
<td>(M_{bl-WAM})</td>
<td>19.40</td>
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<td>20.01</td>
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<td></td>
<td>(M_{bl-DSM})</td>
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<td>28.10</td>
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<td>-</td>
</tr>
<tr>
<td></td>
<td>(M_{bl-WAM})</td>
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<td>30.03</td>
<td>30.03</td>
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<tr>
<td></td>
<td>(M_{bd-ELM})</td>
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<td>-</td>
</tr>
<tr>
<td></td>
<td>(M_{bd-ENG})</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>(M_{bd-WAM})</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>(M_{bd-DSM})</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>(M_{bd-DSM})</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

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### Table 5.11. Nominal capacities of the north column for all Frame Tests

<table>
<thead>
<tr>
<th>Design approaches</th>
<th>Frame Test 1 &amp; 2</th>
<th>Frame Test 3</th>
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<td>86.53 20.81</td>
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5.3.4. Prediction of frame capacities

The second-order elastic analyses with different frame configurations and load conditions, as described in Section 5.3.1, were performed to determine the design forces and moments in the respective frames. The design compressive axial forces and moments for the north columns for all Frame Tests are provided in Table 5.12.

Table 5.12. Design forces and moments for the north columns

<table>
<thead>
<tr>
<th>Test Series</th>
<th>Compression, ( N^* ) (kN)</th>
<th>Bending moment, ( M^* ) (kNm)</th>
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<td>Frame Test 7</td>
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</tbody>
</table>

As evidently seen in Table 5.12, the north columns experienced both significant axial compression and bending moment. As a result, the interaction of axial compression and bending moment was required to be checked for the ultimate strength limit state.

All the current standards/specifications provide the interaction equation to check for the members combined axial compression and bending. Although there is a small difference in the expression of these equations between the standards, a general interaction equation can be used for all the standards, as follows:

\[
\frac{N^*}{\phi_c N_c} + \frac{M_x^*}{\phi_{bx} M_{bx}} + \frac{M_y^*}{\phi_{by} M_{by}} \leq 1
\]  

(5.181)

where

- \( x \) is subscript for major principal axis bending
- \( y \) is subscript for major principal axis bending
- \( N^* \) is the design axial compression
- \( N_c \) is the nominal member capacity of the member in compression
- \( M_x^* \) is the design bending moment about the x-axis of the gross section
$M_y^*$ is the design bending moment about the x-axis of the gross section

$M_{bx}$ is the nominal member moment capacity about the x-axis

$M_{by}$ is the nominal member moment capacity about the y-axis

$\phi_b$ and $\phi_c$ are the capacity reduction factors for the member in bending and in compression, respectively.

It is worth noting that $\phi_cN_c$, $\phi_bM_{bx}$, $\phi_bM_{by}$ are the design (available/factored) capacities of the member in compression and in bending about the x- and y-axes, which may be calculated directly by including the design (reduction) factor in the relevant design equations. In other words, the design capacities were determined directly instead of the nominal capacity. However, as discussed earlier, both the section properties and material properties were obtained from the actual tests, so that the capacity reduction factors were taken as unity. Therefore, Equation (5.181) can be used for all standards.

Since the minor axis bending moment is insignificant, Equation (5.181) can be simplified as follows:

$$\frac{N^*}{\phi_cN_c} + \frac{M_y^*}{\phi_bM_{bx}} \leq 1$$

(5.182)

The member capacities of columns in the cold-rolled aluminium portal frames were checked by satisfying the interaction Equation (5.182) for combined compression and bending actions. The column capacities are found if Equation (5.183) holds true for a given load case. Hence, for each frame test, the second-order elastic analyses were completed with various load levels, which were created by changing the magnitude of gravity loads, and a force-moment pair ($N_j^*$, $M_j^*$) was found to satisfy Equation (5.183).

$$\frac{N^*}{\phi_cN_c} + \frac{M_j^*}{\phi_bM_{bx}} = 1$$

(5.183)

The column capacities for all frame tests were determined by this method. The force-moment pairs ($N_j^*$, $M_j^*$) corresponding to the ultimate capacities of the north column for all frame tests are provided in Table 5.13. It can be seen that the behaviour of frames is dominated by bending, which is partly attributed to the large span of the frame systems.

Based on the concept that once the critical columns of the relative frames reach the ultimate capacities, the frame system reaches the ultimate load, so that the ultimate capacities of the frames can be determined and are provided in Table 5.14.

Although three different approaches were utilised to determine $N_{oc}$ and $M_o$, subsequent determination of relevant capacities was based on the same design equations from each the
current standards, resulting in the similar predictions of member capacities, as seen in Table 5.13, and subsequently, the similar predictions of frame capacities, as shown in Table 5.14.

Table 5.13. The ultimate capacities of the north column

<table>
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5.4. DIRECT DESIGN METHOD

5.4.1. Background

The available standards/specifications for aluminium structures provide design rules using a two-stage design process, where the elastic analysis is performed to determine the design actions in each member of a structure and safety checks are carried out for individual components. A structural system is treated as a set of independent components, such as beams, columns, connections, etc. The interaction between the structural system and its members is only implicitly reflected in the design by the effective length factors. However, despite the popular use in the present as a basic design, the approach has its limitations. Firstly, elastic analysis is used to determine the design actions in each member of the system, whereas inelastic analysis is used to quantify the strength of each independent member. There is no verification of the compatibility between the independent member and the member as a part of the structural system [212]. The second limitation is that the simple effective length factors cannot accurately predict the complex interactions between the member and structural system nor can they capture the inelastic load redistributions after the first yield in indeterminate structural systems [8].

Instead of using the conventional design method, the system-based design has been recognised as a realistic approach to the design of structural systems [9, 10, 153, 179, 187, 212, 213, 276-284]. The recent advances in computer technology and structural analysis methods make it possible to treat a structure in design as a system rather than as a set of individual components. Further, the structural behaviour of the system can be most accurately predicted by using the second-order inelastic analysis, which accounts for all sources of important nonlinear actions affecting the structural behaviour, notably second-order effects, material nonlinearities, geometrical imperfections, and semi-rigidity of connections. This design approach is marked as the direct analysis and design method. The development of the direct method to design is referred to as “Advanced Analysis” or “Non-linear Inelastic Analysis”. The use of the Direct Design Method (DDM) using Advanced Analysis provides significant advantages over the conventional member-based design methods. In the direct method, the system ultimate strength is directly assessed without having to compute the effective length factors or checking the specification equations. Moreover, this method may lead to the design of lighter and more economic structures. The DDM is now permitted in the provisions for steel structures such as in the Australian steel standard AS4100 [214], the American steel specification AISC360-16 [191], and the Australian/New Zealand standard for cold-formed steel structures AS/NZS 4600 [146], thus providing further incentive for using the DDM for cold-rolled aluminium portal frames.

5.4.2. Design of cold-rolled aluminium portal frames using DDM

The ultimate strength of each cold-rolled aluminium (CRA) portal frame test described in Chapter 3 was already obtained by using the system-based design approach presented in
Chapter 4. The generic procedure for design by Advanced Analysis with particular reference to the CRA portal frames is described in the following sections.

5.4.2.1. Preliminary member sizing

Before using the advance analysis, the preliminary structural design needs to be carried out rigorously to estimate the initial member sizes of the frames as decisions taken at the preliminary design stage will influence the extent to which the actual structure approximates to the ideal. The initial size of members is basically chosen on the basis of the engineer’s experience by using simplified analysis and design, and/or simply based on the member sizes of the similar structures that were previously designed.

In this context, the initial size of members can be estimated on the basis of conservative assumptions, which make it possible to derive simplified calculations for both analysis and design (using conventional design method), e.g. Semi-rigid connections can be assumed as pinned connections (or fixed ends with careful consideration), and only strength limit state is examined in this stage. The frames are assumed to be subjected to gravity only and beam elements can be used for structural analysis. It is also noted that the size of columns and rafters can be differently chosen by establishing different preliminary design criteria.

5.4.2.2. Geometric modelling

The essence of the Direct Design Method using Advanced Analysis is that the structures are modelled as realistically as possible to simulate accurately the structural response that would achieve in physical tests of the structure. Therefore, full-three dimensional frame models can be created with the same geometry as the physical frames. Since the frames composed of CRA sections, the thickness of the sections is significantly small as compared to other dimensions. As a result, the CRA portal frames are susceptible to local instabilities, and are likely to undergo large deformation. Hence, to capture this structural behaviour, shell element models were chosen as it has been proved to be the most suitable element type for thin-walled structures. The appropriate mesh size for the FE models can be determined by referring to the mesh size used in the similar existed frame models or more efficiently, which can be run by a mesh convergence study on the simplified models.

In the frames, members are connected using bolted connections. To simulate the behaviour of the bolted connections, the bolts can be efficiently modelled by deformable mesh-independent point-based fasteners with prescribed non-linear constitutive relations obtained from the fastener tests or derived by numerical simulations. Contact formulations should also be implemented in places where severe deformations are experienced, triggering components come into contact with others. More details of the modelling can be found in Chapter 4.
5.4.2.3. Material properties

To proceed with the Advanced Analysis, non-linear material properties are required to be assigned to the respective components. The nonlinear material behaviour is often defined by Young’s modulus and Poisson’s ratio for the linear elastic behaviour, and the true stress – plastic strain relations for the material nonlinearity. Although the anisotropy can be omitted and material mechanical properties in the longitudinal direction can be adopted as the variations in physical properties along different molecular axes, the effects are negligible and will not be considered in this study. However, the yield strength enhancement at the corner regions due to cold-forming should be taken into consideration. The nonlinear material properties can be obtained from the coupon tests or simply adopted from the available data collected from past studies. In addition, residual stresses may not need to be included since the membrane residual stress has been proved to introduce an insignificant effect on the strength capacity of CRA alloy members, and the flexural residual stresses are often implicitly assumed to be automatically introduced in the FE models through the assignments of stress-strain relations obtained from the coupon tests.

5.4.2.4. Initial geometric imperfections

Since the initial geometric imperfections have a detrimental effect on the system strength and behaviour, they need to be accurately incorporated into the FE models. Imperfections are usually included in the FE models by introducing perturbations in the geometry, which can be achieved by several methods. A common method of introducing imperfections is through the superposition of buckling eigenmodes obtained from the elastic buckling analysis. The appropriate scale factors can be found in standards or statistical data from the previous studies. This method was presented in past studies, e.g. [159, 160]

Another efficient approach is the explicit modelling of initial imperfection. The geometric imperfections of the frames can be typically categorised into three types: sectional imperfections, member imperfections and frame imperfections. These imperfection types are incorporated into numerical models at different levels. The imperfection data can be collected from the measured imperfections or adopted the statistical data from the past studies for similar material and structures. More details of the method are described in Section 4.6.5 and Section 4.8.1.

Besides, the sensitive study of the imperfections can be performed by completing nonlinear analysis without geometric imperfections and validating the results against that of the imperfect structural system to evaluate the influence of initial imperfections on the strength and behaviour of the structure.

5.4.2.5. Loads

The aluminium portal frame systems are basically subjected to both vertical (gravity) and horizontal (wind) loads. Ignoring the special cases of earthquakes and crane loads, most actions
on the portal frames are due to gravity loads arise from self-weight of the frames, the weight of purlins/girts, roof sheeting, associated fixtures and live loads and horizontal loads arise from wind loads. In certain alpine and sub-alpine areas, it may be necessary to consider the effects of snow actions.

The essence of the modelling actions on the frames for Advanced Analysis is that they must exhibit the load distribution and the loading sequence on the real system. The accuracy of load magnitude may be less significant for Advanced analysis as it requires only the reference load to establish the load-displacement equilibrium path. However, the required/design actions are necessary to confirm the structural adequacy in the routine design.

For particular aluminium Frames considered in the experimental program, the Advanced Analysis was performed with two cases of load conditions: (1) frames under gravity only (Frame Tests 1, 2, 4, 6, and 7) and (2) frames subjected to the combination of gravity and horizontal loads (Frame Tests 3 and 5). The horizontal force was fixed to a constant magnitude of 7.5 kN, whereas the load spreading system was used to generate equally incremental vertical loads at eight load points on the rafters. By applying this approach, it reflects closer to the actual load transferring conditions and thus the obtained results can be comparable to the test results.

5.4.2.6. Nonlinear analysis and system strength assessment

The nonlinear inelastic analysis is performed using the static general option (Newton-Raphson method). As the CRA portal frames are prone to local instabilities with the contact non-linearity, the convergence difficulties may occur, resulting in the termination of the analysis without reaching the peak load. To avoid convergence issues, several techniques can be considered as follows: (1) using displacement control instead of load control if possible; (2) the automatic stabilisation can be employed with rigorously examining the effect of viscous damping on the results by maintaining the energy dissipated through artificial damping (ALLSD) at a negligible level; (3) a small initial step size should be chosen and limits imposed on the maximum allowable increment. Besides, to improve the analysis speed, automatic time incrementation can be used because of its efficiency during the analysis.

After completing the nonlinear inelastic analysis, the nonlinear load-displacement response of the system can be obtained and the ultimate load acting on the system can be extracted, which serves as the nominal strength of the system ($R_n$). The adequacy of the structural system strength can be directly assessed by comparing the ultimate strength ($\phi_s R_n$) to the design actions ($\sum \gamma_i Q_{ni}$) [178], in which $\phi_s$ is the system resistance factor obtained from reliability analysis; $Q_{ni}$ and $\gamma_i$ are the loads and load factors, respectively.

5.4.2.7. Serviceability assessment

The serviceability of the structural system should be also assessed to ensure the adequacy of the structure and its member stiffness under service actions. To obtain the deflection in conforming to serviceability limit state, 1st order elastic analysis can be performed [178] on the
same model with serviceability load combinations. The resulting displacements are then compared to the limited displacements specified in the standards to evaluate the adequacy of the system and its member stiffness.

5.4.2.8. Adjustment of member sizes

If the design requirements are not satisfied, the adjustment of the member sizes needs to be made. Since the Advanced Analysis also provides the stress distribution in the structural system, it is feasible to appropriately adjust the size of members in the structures. Based on the stress distribution, the most highly stressed member can be identified and then it should be replaced first with a stronger member. On the contrary, if the structure is designed over-conservatively, the member with the least stress can be replaced with a smaller size. The process can be performed until the system is optimised. A similar process can be adopted for the serviceability limit state. If the deflections of the structure do not meet the drift requirement, the member sizes can be increased for serviceability or a brace system can be considered instead.
5.5. COMPARISON OF DESIGN RESULTS

The ultimate strengths of portal frame systems composed of CRA back-to-back sections predicted on the basis of the conventional design methods and the DDM using Advanced Analysis are provided in Table 5.14 along with the ultimate loads obtained from experiments.

Table 5.14. Ultimate capacities of the frame systems

<table>
<thead>
<tr>
<th>Design approaches</th>
<th>Frame Test 1&amp;2</th>
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<th>Frame Test 4</th>
<th>Frame Test 5</th>
<th>Frame Test 6</th>
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<td>-</td>
<td>-</td>
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<td>-</td>
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<tr>
<td></td>
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<td></td>
<td>ENG 22.75</td>
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<td>-</td>
</tr>
<tr>
<td></td>
<td>EBA 23.71</td>
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<td>19.35</td>
<td>21.12</td>
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</tr>
<tr>
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<td>-</td>
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<td>-</td>
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<tr>
<td></td>
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<td>-</td>
<td>17.17</td>
<td>-</td>
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<tr>
<td></td>
<td>EBA 20.24</td>
<td>10.07</td>
<td>23.65</td>
<td>12.82</td>
<td>17.92</td>
<td>37.03</td>
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<tr>
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<td>33.73</td>
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<td>40.21</td>
<td>28.52</td>
<td>27.57</td>
<td>61.46</td>
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<td>33.32</td>
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<tr>
<td>TESTS</td>
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<td>18.84</td>
<td>39.20</td>
<td>27.59</td>
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<td>60.69</td>
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<td>32.16</td>
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</tbody>
</table>
A comparison of predicted strengths from the various design approaches to the experimental results is presented in Table 5.15 and plotted in Figure 5.17.

In the conventional design methods, as described in Section 5.3.4, the linear strength interaction equation for the combined bending and axial compression was used to derive the ultimate capacities of the portal frame systems. It can be seen that all the current standards using the conventional design methods underestimate the load capacities of the frame systems. As expected, the frame strengths predicted by the AS/NZS 1664 are very close to those predicted by the Aluminium Design Manual (AA2015) because the AS/NZS 1664 code was technically developed on the basis the older version of the Aluminium design manual. The strengths obtained from their prediction are 23% ÷ 31% lower than the actual strengths. The same phenomenon is also shown in the prediction of the Eurocode 9, in which the predicted strengths are 16% ÷ 37% lower as compared to the experimental capacities. For the unbraced columns frames, the predictions of the AS/NZS 1664 and the AA2015 standards seem to be better than those of the Eurocode 9. In contrast, for the frames with braced columns, the capacities predicted from the Eurocode 9 seem to be closer to the experimental results than the strengths predicted by the other two codes. This is partly attributed to the use of the design rules for cold-formed trapezoidal aluminium sheeting in determining the member capacity of the column in compression (Section 5.3.2.3.1.2). When the strength enhancement of the cold-formed aluminium sections was accounted for, the member capacity of the column in compression was also improved. Along with this, in the frames with braced columns, the influence of the axial compression in the linear interaction equation (5.183) becomes significant. As a result, the predicted ultimate strengths of the frames are higher. Therefore, it is recommended that the strength enhancement due to the cold-working process should be considered in the practice design.

The AS/NZS 4600 standard for CFS structures was also attempted to predict the strength of the CRA portal frames. It is shown that the predictions of the AS/NZS 4600 are quite good. Its predictions for the unbraced column frames are even better than those of the current aluminium specifications. This implies that using design provisions of extruded aluminium profiles for the design of CRA portal frames may not be very appropriate, requiring the development of new design guidelines for cold-rolled aluminium members and structures.
Table 5.15. Comparison of predicted strengths to the test results

<table>
<thead>
<tr>
<th>Design approaches</th>
<th>Frame Test 1&amp;2</th>
<th>Frame Test 3</th>
<th>Frame Test 4</th>
<th>Frame Test 5</th>
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<tr>
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<td>0.77</td>
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<td>-</td>
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</tr>
<tr>
<td>EBA</td>
<td>0.77</td>
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</tr>
<tr>
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<td>0.84</td>
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</tr>
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<td>1.03</td>
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<td>1.01</td>
<td>1.01</td>
</tr>
<tr>
<td>TESTS</td>
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<td>18.84</td>
<td>39.20</td>
<td>27.59</td>
<td>27.42</td>
<td>60.69</td>
</tr>
</tbody>
</table>
On the basis of the DSM for CFS, a member-based design method using the DSM was proposed by Huynh, L.A.T. [25] and Pham, N.H. [205] and was also attempted to predict the frame capacities as shown in Table 5.14. The obtained results were compared to the corresponding test results and are presented in Table 5.15 and Figure 5.17. Although the strength enhancement of the cold-rolled section has been included in the proposals, the predicted capacities are very low as compared to the actual results. Especially, while the predicted strengths for Frame Tests 3 and 5 do not reach half of the experimental strengths, those for other Frame Tests are approximately 60% of the actual strengths. The explanation for this fact is probably due to the consideration of the interaction between distortional and flexural-torsional buckling in the columns, as proposed by Pham, N.H. However, in the CRA portal frames as considered herein, the interaction between distortional and flexural-torsional buckling was not introduced. In the braced column frames, the lateral-torsional buckling of columns was eliminated by the presence of the girts, as described in Chapter 3, and thus there is no need to included the distortional-global interaction in the calculation of the frame strengths. In the unbraced column frames, the columns underwent the lateral-torsional buckling that initiated the overall collapse of frames. The ultimate loads were reached when an occurrence of the distortional/local buckling in the compression flanges and lips usually presented in the mid-length of the column as demonstrated in Section 3.2.10. In terms of
moment distribution in the columns, the maximum moment occurred at the knee connection while the moment at mid-length is much smaller (This can be seen in the respective bending moment diagrams plotted in Appendix D.2). Meanwhile, the distortional-global interaction equations were proposed based on the experimental and numerical investigations of simply support single-channel section under a uniform moment (The maximum moment occurred at the mid-span where the distortional buckling is likely to appear). Hence, in the Frame Tests, the distortional-global interaction did not occur due to the effect of moment gradient. The nominal capacities of the north column under pure compression and pure moment were determined based on the proposed DSM without considering the interaction between distortional and global buckling and are shown in Table 5.16. The capacities of the column subjected to combined bending and axial compression were then determined by using the linear interaction equation and are provided in Table 5.17. Subsequently, the frame strengths were predicted (Table 5.18) and are compared with the corresponding test result (Table 5.19).

Table 5.16. Nominal capacities of the north column based on the proposed DSM without the distortional-global interaction

<table>
<thead>
<tr>
<th>Design approaches</th>
<th>Frame Test 1&amp;2</th>
<th>Frame Test 3</th>
<th>Frame Test 4</th>
<th>Frame Test 5</th>
<th>Frame Test 6</th>
<th>Frame Test 7</th>
</tr>
</thead>
<tbody>
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<td>$M_b$</td>
<td>$N_c$</td>
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<td>78.13</td>
<td>23.96</td>
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</table>

Table 5.17. Capacities of the north column under combined bending and axial compression predicted by the proposed DSM without the distortional-global interaction

<table>
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<tr>
<th>Design approaches</th>
<th>Frame Test 1&amp;2</th>
<th>Frame Test 3</th>
<th>Frame Test 4</th>
<th>Frame Test 5</th>
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<td>$M_b$</td>
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</table>
Table 5.18. Capacities of the Frame Tests predicted by the proposed DSM without the distortional-global interaction

<table>
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<tr>
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<th>Frame Test 4</th>
<th>Frame Test 5</th>
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<tr>
<td>ENG</td>
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<td>39.20</td>
<td>27.59</td>
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</table>

Table 5.19. Comparison of predicted strengths from the proposed DSM without the distortional-global interaction to the test results

<table>
<thead>
<tr>
<th>Design approaches</th>
<th>Frame Test 1&amp;2</th>
<th>Frame Test 3</th>
<th>Frame Test 4</th>
<th>Frame Test 5</th>
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<td></td>
</tr>
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<td>0.74</td>
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<td>0.77</td>
</tr>
<tr>
<td>ENG</td>
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<td>-</td>
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<td>18.84</td>
<td>39.20</td>
<td>27.59</td>
<td>27.42</td>
<td>60.69</td>
</tr>
</tbody>
</table>

As seen in Table 5.19, the proposed DSM for cold-rolled aluminium members predicted quite accurately the strength of the CRA portal frames with a lower degree of variation as compared to the other member-based design guidelines. However, the proposed DSM predictions are still conservative.

The underestimated frame strength of the conventional design methods is attributed to many contributing factors. These may include inaccurately predicting the complex interactions between the system and its members, and incapacity of capturing the load redistributions in the frame through the use of the effective length factors only. Furthermore, in the traditional design methods, the frame ultimate capacities were derived on the basis of the linear strength interaction equation of the critical column for the combined bending and axial compression (Equation (5.183)), in which the member capacities in compression and bending were determined separately and independently from each other. This may not reflect the actual interaction between design actions. A numerical study was conducted to determine appropriate
interaction relations for a beam-column composed of back-to-back CRA channel sections (2AC25025). For this purpose, FE models for different spans (slenderness) and axial compression ratio were created using ABAQUS. The results of the Advanced Analyses are shown in Figure 5.18.

![Interaction curves of the double channel CRA 2AC25025 beam-column](image)

**Figure 5.18. Interaction curves of the double channel CRA 2AC25025 beam-column**

It is revealed that nonlinear interactions should be used to estimate the strength of members under the effect of combined bending and axial compression design actions. By using the nonlinear interaction, which was derived based on the numerical data shown in Figure 5.18, the predicted strengths of the CRA portal frames can increase about 5% ÷ 10% as compared to the use of linear interaction equation, and thus they are closer to the actual strengths. The frame strengths predicted based on the proposed DSM using a nonlinear interaction and the comparison with the test results are shown in Table 5.20 and Table 5.21, respectively.
Table 5.20. Frame strengths predicted by the proposed DSM using a nonlinear interaction

<table>
<thead>
<tr>
<th>Design approaches</th>
<th>Frame Test 1&amp;2</th>
<th>Frame Test 3</th>
<th>Frame Test 4</th>
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<td><strong>27.57</strong></td>
<td><strong>61.46</strong></td>
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<td><strong>27.42</strong></td>
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</table>

Table 5.21. Comparison of predicted strengths from the proposed DSM using a nonlinear interaction to the test results

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<td>0.83</td>
<td>0.93</td>
<td>-</td>
<td>-</td>
<td>0.87</td>
<td>-</td>
</tr>
<tr>
<td>EBA</td>
<td>0.88</td>
<td>1.02</td>
<td>0.87</td>
<td>0.84</td>
<td>0.93</td>
<td>0.88</td>
</tr>
<tr>
<td>DDM</td>
<td><strong>1.03</strong></td>
<td><strong>1.02</strong></td>
<td><strong>1.03</strong></td>
<td><strong>1.03</strong></td>
<td><strong>1.01</strong></td>
<td><strong>1.01</strong></td>
</tr>
<tr>
<td>TESTS</td>
<td><strong>33.25</strong></td>
<td><strong>18.84</strong></td>
<td><strong>39.20</strong></td>
<td><strong>27.59</strong></td>
<td><strong>27.42</strong></td>
<td><strong>60.69</strong></td>
</tr>
</tbody>
</table>

On the other hand, in the DDM, the ultimate loads were extracted directly from the load-displacement responses obtained from Advanced Analysis of the complete frame systems. All sources of major nonlinear actions, notably the material nonlinearity, the deformable fastener connections, and the initial geometric imperfections, are explicitly included in the shell model of the frame systems. Despite this, the predicted strengths from the DDM are on average 2.2% higher than the actual strengths. However, with the mean of test-to-prediction strength ratio equal to 0.979 and the corresponding low coefficient of variation of 0.011 (Table 4.4), it indicates that the strengths predicted by the DDM are more uniform and reliable as compared to the conventional methods employed in the current standards of aluminium sections. Therefore, the use of the DDM adopting Advanced Analysis is very promising for the design of CRA portal frames.
5.6. SYSTEM RELIABILITY ANALYSIS

The conventionally employed methodology to design and evaluate aluminium structural systems based on individual component analysis reflects the current aluminium specifications [152, 197, 198]. A structural system is treated as a set of individual components that must satisfy the design safety checks. According to the Load and Resistance Factor Design (LRFD) format, the structural members must comply with:

\[ \phi_{mem} R_{n}^{mem} \geq \sum \gamma_i Q_{ni} \]  

(5.184)

In which \( R_{n}^{mem} \) is the nominal member strength using the provisions of the standards, \( Q_{ni} \) are the load effects, and \( \phi_{mem} \) and \( \gamma_i \) are the resistance and load factors, respectively, that reflect uncertainties in the resistance and loads.

In consistent with the current LRFD philosophy, if the resistance of the whole system \( R_n \) is determined using advanced analysis, an approximate system resistance factor \( \phi_s \), which accounts for uncertainties influencing the reliability of the system, need to be applied. Hence, the LRFD format can be applied in an integral sense for the system-based design:

\[ \phi_s R_n \geq \sum \gamma_i Q_{ni} \]  

(5.185)

where \( \sum \gamma_i Q_{ni} \) represents the total load effect on the structure.

In the following sections, system reliability analysis is performed to establish relations between the reliability index or safety index (\( \beta \)) that can serve as a comparative measure of reliability or safety of a structural system and the system resistance factor (\( \phi_s \)) for the cold-rolled aluminium portal frames.

Let \( R \) denote the system strength variable and \( Q \) represents the load design variable, a performance function, or limit state function defined by equation (5.186) can be used to establish the safe and unsafe region [219].

\[ g(R, Q) = R - Q \]  

(5.186)

Structural safety is a function of the resistance, \( R \), of the system and the load effects, \( Q \). The resistance and the load effects are assumed to be random variables because of the uncertainties correlated with their inherent randomness. The probability of failure, \( P_f \), can be derived by considering the probability density functions (PDFs) of the random variables \( R \) and \( Q \) as shown in Figure 5.19.
The structures “fail” when the load effects exceed the resistance. The probability of this event is:

\[ P_f = P(g(R,Q) < 0) \]  \hspace{1cm} (5.187)

The calculation of the probability of safety, or probability of failure, requires the information of the distributions \( f_R(r) \) and \( f_Q(q) \), or the joint distribution \( f_{R,Q}(r,q) \) \[219, 285\]. In practice, this information is often unavailable or difficult to obtain due to insufficient data. In some cases, the available information or data may be sufficient only to evaluate the first and second moments, i.e. the mean values and the variances of the respective random variables. Therefore, practical reliability analysis must often be limited to the functions of these first two moments so that the implementation of reliability concepts must necessarily be limited to a formulation based on the first and second moments of the random variables- that is restricted to the second-moment formulation \[286\]. Although the true mean and coefficient of variation (COV) of the variables (the resistance and load effects) should be employed when measuring reliability, these data generally are not known precisely in structural engineering problems because of the lack of data and information. Instead, the estimates of the mean and COV of the resistance and load variables are computed from the idealized models and data gathered from carefully controlled experiments and sampled load data \[11, 285\].

In this study, reliability analysis is performed for CRA portal frames under gravity loads, using the First-order second-moment probabilistic analysis. The statistical parameters for the resistance of the system were determined based on data collected from the experimental investigations demonstrated in Chapter 3 and the corresponding numerical analysis in Chapter 4. Besides, the statistical parameters from Aluminium Design Manual \[198\] were also attempted. To calculate the reliability index (\( \beta \)), two methods including the Rackwitz-Fiessler procedure \[287\], which used the iteration algorithm and began with the calculation of equivalent normal values of the mean and standard deviation for non-normal random variables, and the approximate expression that was mentioned in the current aluminium specifications \[152, 197, 198\].
5.6.1. Target system reliability index

Probabilistic limit state design and evaluation are built around the notation of a “target” reliability or the acceptable probability of failure (the acceptable risk) as a quantitative measure of structural safety. The identification of acceptable risks and selection of appropriate target reliabilities are complicated and challenging due to the small probabilities that are usually involved, limitation in supporting data, and the severe consequences of structural failure [288]. The values of reliability targets are not readily available and need to be selected by engineering judgement or based on assessing existing structures acknowledged to be efficiently designed [283]. In this study, a range of acceptable system reliability index (β) was chosen to perform the reliability analysis based on the following considerations [283, 289]:

- The reliability of most structural systems is greater than that of the most critical member in the structures. This is due to the favourable effect of structural redundancy, allowing the structures to redistribute force and moment under increasing applied load, is likely to more than compensate for the undesirable influence of the fact that “the more components the system comprises, the larger the chance of one component failing”.

- In statically determine systems, which commonly exist in practice, the system fails when the first member fails so that the system reliability is equal or slightly less than the member reliability. Hence, it is supposed for such systems that the system reliability index may be adopted as current member safety indices.

- The structural system does not collapse when one member reaches a limit state and, similarly, the hazard of a member failing is probably lower than that of a system failing.

- The importance level of a structure plays a crucial role in the judgement of an acceptable probability of structural failure.

On the basis of these considerations, a minimum system reliability index of 2.5 is suggested for gravity loads, which is the member reliability index recommended in [146, 191].

Further, for the ultimate limit state, a range of target system reliability indices (β), that depends on the level of damage and the likelihood of the loss of human life, was proposed by Galambos [289], ranging from β = 2.0 assigned to the case of slight damage of the system to β = 4.0 assigned to the limit states which results in the completed destruction of the system and its occupants.

In this study, the system resistance factor (ϕ_s) will be calibrated with the system safety indices (β) ranging from 2.5 to 3.5 for gravity load combination.
5.6.2. Random variables

5.6.2.1. Resistance

The randomness of the resistance \( R \) of a structural system in association with the variabilities inherent in the mechanical properties of the material, the variation in geometric properties, and the uncertainties in the prediction of the system strength. The mean strength \( (R_m) \) and the coefficient of variation \((V_R)\) of the resistance are defined as follows [221, 285, 290]:

\[
R_m = R_n (M_m F_m P_m)
\]

\[
V_R = \sqrt{V_M^2 + V_F^2 + V_P^2}
\]

(5.188)  
(5.189)

In which, \( R_n \) is the nominal resistance of the structural system, and \( M_m, F_m \) and \( P_m \) are the mean values of the random variable ratios reflecting abbreviations for Material, Fabrication and Professional, and \( V_M, V_F, V_P \) are the corresponding coefficient of variations. Statistical data for these parameters are provided in Table 5.22.

\( M_m \) denotes the mean ratio of actual mechanical properties (i.e \( F_y, F_u, E \), etc) obtained from the coupon tests to the corresponding specified values (normal values). In this study, two cases concerning material factor will be considered. They are the material factor based on Appendix 1 of American Aluminium Design Manual 2015 (AA2015) [198] in case of data established from a sufficient number of results on material properties do not exist for the system (Material case 2), and material factor determined from data obtained by actual coupon test results (Section 3.3) and data collected from [21] (Material case 1). These data are given in Appendix D.4 including statistics of elastic modulus and yield stress. The dominant material parameter for elastic buckling is the modulus of elasticity, whilst the yield stress is the dominant material parameter for inelastic buckling [291]. For reliability analysis of aluminium portal frames, the ratio of static yield stress obtained from real data to the nominal yield stress was adopted. Material case 1 pertains to BlueScope Permalite Aluminium material that was used in test frames, while Material case 2 represents the “general category” of extruded aluminium.

\( F_m \) is the mean ratio of actual to the nominal dimensions of the cross-section. In this study, the thickness was selected as a dominant parameter as it is the most important dimension for thin-wall sections [292]. The statistical data for fabrication factor was presented in Appendix D.4 and rearranged in Table 5.22. As seen in Table 5.22, BlueScope Permalite material has a lower coefficient of variation for fabrication and material factors as compared to that of AA2015.

\( P_m \) reflects the accuracy of the model and is the ratio of the mean value of the tests divided by the advanced analysis model. The information for \( P_m \) was given in Table 4.4.
Based on Equations (5.188),(5.189) and the statistical information provided in Table 5.22, the mean strength \( R_m \) and the coefficient of variation \( V_R \) of the resistance and Material Case 2 are \( R_m = 1.045R_n; \ V_R = 0.044 \) and \( R_m = 1.077R_n; \ V_R = 0.079 \), respectively. The Log-normal distribution that is widely accepted for the statistical model of resistance is also used in this study.

**Table 5.22. Statistical parameters for material, fabrication and professional factors**

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Material Case 1</th>
<th>Source</th>
<th>Material Case 2</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>( M_m )</td>
<td>1.093</td>
<td>Table D.9</td>
<td>1.100</td>
<td>Appendix 1 of the AA2015</td>
</tr>
<tr>
<td>( F_m )</td>
<td>0.977</td>
<td>Table D.11</td>
<td>1.000</td>
<td></td>
</tr>
<tr>
<td>( P_m )</td>
<td>0.979</td>
<td>Table 4.4</td>
<td>0.979</td>
<td></td>
</tr>
<tr>
<td>( V_M )</td>
<td>0.042</td>
<td>Table D.9</td>
<td>0.060</td>
<td></td>
</tr>
<tr>
<td>( V_F )</td>
<td>0.005</td>
<td>Table D.11</td>
<td>0.050</td>
<td></td>
</tr>
<tr>
<td>( V_P )</td>
<td>0.011</td>
<td>Table 4.4</td>
<td>0.011</td>
<td></td>
</tr>
</tbody>
</table>

\[
R_m/R_n = 1.045 \quad 1.077 \\
V_R = 0.044 \quad 0.079
\]

### 5.6.2.2. Load and Load effects

In this study, system reliability analysis was performed for CRA portal frames subjected to gravity loads only. This load combination includes Dead load (permanent load) and Live load (roof imposed load). Therefore, the variability of these structural loads needs to be examined.

The dead (or permanent) loads acting on a portal framed building arise its self-weight including finishes and from any other permanent construction or equipment. The dead load will vary during construction but is assumed to remain constant thereafter [265]. The statistical parameter of the dead load was investigated and proposed by many researchers. Ellingwood et al. [285] investigated the results of several American studies on dead load and propose that the probability distribution of dead load is a normal distribution with the ratio of mean load to nominal load \( D_m/D_n = 1.05 \) and the coefficient of variation \( V_D = 0.10 \), which were adopted in ASCE/SEI 7 [293]. Meanwhile, the statistical model for permanent load according to AS/NZS 1170.1 [265] was given by L Pham et al. [294, 295]. It also follows a normal distribution with the ratio of mean load to nominal load \( D_m/D_n = 1.05 \) and the coefficient of variation \( V_D = 0.10 \).

In design, there are normally two cases of design live load including (1) maximum value for a certain return period (typically 50 years) and (2) point-in-time value to be considered. The former case is associated with a sustained live load where the value of the live load is most
likely to be found acting on the structure at any point in time, whereas the latter case corresponds to the expected maximum value of live load that is likely to act on the system during a reference period. The live loads (or imposed load) acting on the roof of a portal frame building arise mainly from the maintenance loads where new or old roof sheeting may be stacked in concentrated areas [139, 293]. Hence, the live load model can be considered as intermittent live load, which is associated with occupancy situations where the imposed load is increased by a short to very short of time [282]. Due to the absence of statistical data for roof imposed load as pointed out in [282] and accepted in [11], the following assumption was made for reliability analysis in this study: maximum live load on the roof has the similar statistical characteristics as the occupancy live load that have been proposed in [285, 295, 296]. It means that the maximum roof imposed load follows an Extreme Type I distribution with mean-to-nominal ratios of 0.8 and 1.0 for the live loads accorded with AS/NZS 1170.1 [265] and ASCE/SEI 7 [293], respectively. The coefficient of variation of 0.25 [285, 296] was used for both the load combinations according to AS/NZS 1170.0 [264] and ASCE/SEI 7 [293].

The statistical parameters of dead load and live load used in this study are presented in Table 5.23.

Table 5.23. Statistical parameters for load effects

<table>
<thead>
<tr>
<th>Load type</th>
<th>AS/NZS 1170.0 Load combination</th>
<th>ASCE/SEI 7 Load combination</th>
<th>Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mean-to-nominal ratio</td>
<td>COV</td>
<td>Mean-to-nominal ratio</td>
</tr>
<tr>
<td>Dead Load</td>
<td>1.05</td>
<td>0.10</td>
<td>1.05</td>
</tr>
<tr>
<td>Live load (50 years)</td>
<td>0.80</td>
<td>0.25</td>
<td>1.00</td>
</tr>
</tbody>
</table>

5.6.3. Cold-rolled Aluminium portal frames under gravity loads only

5.6.3.1. Limit state function

For the CRA portal frames subjected to gravity loads, the ultimate limit state design equations with combinations of load effects in accordance with different current standards are given as follows:

- Considering the load criteria as per AS/NZS 1170.0 [264]:

\[
\phi_s R_n \geq 1.35D_n
\]  
\[
\phi_s R_n \geq 1.2D_n + 1.5L_n
\]

- And as per ASCE/SEI 7 [293]:
\[ \phi_s R_n \geq 1.4 D_n \]  
\[ \phi_s R_n \geq 1.2 D_n + 1.6 L_n \]

where \( D_n \) and \( L_n \) are the nominal dead load and nominal live load, respectively.

The general form of these design equations can be written as:

\[ \phi_s R_n \geq \gamma_d D_n + \gamma_l L_n \]

in which, \( \gamma_d \) is the dead load factor and \( \gamma_l \) is the live load factor.

The limit state function \( g \) is given by:

\[ g(R, D, L) = R - (D + L) \]

At the limit state surface \( g = 0 \), which is the boundary between the “safe domain” and the “failure domain”, and the design Equation (5.194) becomes:

\[ \phi_s R_n = \gamma_d D_n + \gamma_l L_n \]

Equation (5.196) can be rewritten in terms of mean-to-nominal ratios of loads and resistance, as follows:

\[ \frac{\phi_s R_m}{(R_m / R_n)} = \gamma_d \frac{D_m}{(D_m / D_n)} + \gamma_l \frac{L_m}{(L_m / L_n)} \]

where \((D_m/D_n)\) and \((L_m/L_n)\) are the mean-to-nominal ratios of dead load and live load respectively, while \((R_m/R_n)\) defines the ratio of the mean resistance to the nominal resistance.

Introducing \((L_m/D_n) = k\), then the relations between the mean live load \((L_m)\) to the mean dead load \((D_m)\) is:

\[ L_m = \left( \frac{L_m / L_n}{D_m / D_n} \right) k D_m \]

And thus,

\[ R_m = \left[ (\gamma_d + \gamma_l k) \frac{D_m}{(D_m / D_n)} \right] \left( \frac{R_m / R_n}{\phi_s} \right) \]

5.6.3.2. Reliability analysis

The limit state function given by Equation (5.195) is composed of non-normal variables. As a result, a closed-form solution for the system reliability index \( \beta \) cannot be obtained, and thus, the First-order second-moment (FORM) probabilistic analysis using the Rackwitz-Fiessler procedure [287] was performed to derive an estimation of \( \beta \). The procedure used the
iteration algorithm and began with the calculation of equivalent normal values of the mean and standard deviation.

Based on the two different load combinations in Table 5.23 along with two cases of material factors as shown in Table 5.22, four cases for the reliability analysis can be considered as summarised in Table 5.24.

Table 5.24. Cases considered for reliability analysis

<table>
<thead>
<tr>
<th>Cases</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>AS/NZS 1170.0 Load combination with Material case 1</td>
</tr>
<tr>
<td>Case 2</td>
<td>AS/NZS 1170.0 Load combination with Material case 2</td>
</tr>
<tr>
<td>Case 3</td>
<td>ASCE/SEI 7 Load combination with Material case 1</td>
</tr>
<tr>
<td>Case 4</td>
<td>ASCE/SEI 7 Load combination with Material case 2</td>
</tr>
</tbody>
</table>

The steps in the matrix procedure implementing the Rackwitz-Fiessler modification linking system resistance factors ($\phi_s$) to system reliability indices ($\beta$) can be summarised as follows [219]:

1. Formulate the limit state function. The limit state function for cold-rolled aluminium portal frames subjected to gravity loads was given by Equation (5.195) and is rewritten as:

$$ g(R, D, L) = R - D - L $$

in which, $R$ is the system strength, $D$ is the dead load and $L$ is the roof imposed load.

2. Determine the probability distributions and appropriate parameters for the system resistance ($R$). As discussed in the above sections, the lognormal random variable is adopted for the probabilistic model of the system strength of cold-rolled aluminium portal frames with the mean $R_m$ and standard deviation $\sigma_R = V_R R_m$, i.e.,

$$ R \sim \text{Lognormal} \left( \mu_R = R_m, \sigma_R = V_R R_m \right) $$

By substituting Equation (5.199) into Equation (5.201), the probabilistic model of the system strength becomes:

$$ R \sim \text{Lognormal} \left( \mu_R = \left( \gamma_d + \gamma R \right) \frac{D_m}{(D_m/D_n)} \left( \frac{R_m/R_n}{\phi_s} \right), \sigma_R = V_R \left( \gamma_d + \gamma R \right) \frac{D_m}{(D_m/D_n)} \left( \frac{R_m/R_n}{\phi_s} \right) \right) $$

(5.202)

3. Estimate the probabilistic model for the variables permanent load ($D$) and roof imposed load ($L$). As given in Table 5.23, it is established that the dead load follows the normal distribution with the mean $D_m$ and standard deviation $\sigma_D = V_D D_m$ while roof live load follows the extreme type I distribution with the mean $L_m$ and standard deviation $\sigma_L = V_L L_m$, i.e.,
\[ D \sim \text{Normal} \left( \mu_D = D_m, \sigma_D = V_D D_m \right) \]  

\[ L \sim \text{Extreme type I} \left( \mu_L = L_m, \sigma_L = V_L L_m \right) \]

Using Equation (5.198), Equation (5.204) becomes:

\[ L \sim \text{Extreme type I} \left( \mu_L = \left( \frac{L_m/L_n}{D_m/D_n} \right) kD_m, \sigma_L = V_L \left( \frac{L_m/L_n}{D_m/D_n} \right) kD_m \right) \]  

4. Acquire an initial design point \( \{x^*_i\} = \{r^*, d^*, l^*\}^{-1} \) by assuming:

\[ r^* = R_m \]  

\[ l^* = L_m \]  

From the limit state equation \( g = 0 \) which ensures that the design point is on the failure boundary, then:

\[ \Rightarrow d^* = r^* - l^* \]  

5. Determine the equivalent normal mean and standard deviation for the system strength \( (R) \) and the roof imposed load \( (L) \) as they are non-normal distribution.

- For the system strength \( (R) \) – Lognormal distribution:

\[ \sigma^*_R = r^* \sigma_{\ln R} \]  

\[ \mu^*_R = r^* \left[ 1 - \ln \left( r^* \right) + \mu_{\ln R} \right] \]  

where:

\[ \sigma^2_{\ln R} = \ln \left[ 1 + \frac{\sigma^2_R}{\mu_R} \right] \]  

\[ \mu_{\ln R} = \ln \left( \mu_R \right) - 0.5 \sigma^2_{\ln R} \]

- For the live load \( (L) \) – Extreme type I distribution:

\[ F_L \left( l^* \right) = \exp \left[ -\exp \left( -\alpha \left( l^* - u \right) \right) \right] \]  

\[ f_L \left( l^* \right) = \alpha \left\{ \exp \left[ -\exp \left( -\alpha \left( l^* - u \right) \right) \right] \right\} \exp \left( -\alpha \left( l^* - u \right) \right) \]  

where:
\[ \alpha = \sqrt{\frac{\pi^2}{6\sigma^2}} \]  
(5.215)

\[ u = \mu - \frac{0.5772}{\alpha} \]  
(5.216)

Hence,

\[ \sigma_L = \frac{1}{f_L (l')} \Phi^{-1} \left( F_L (l') \right) \]  
(5.217)

\[ \mu_L = l' - \sigma_L \Phi^{-1} \left( F_L (l') \right) \]  
(5.218)

6. Determine the reduced variables \( \{ z_i^* \} \) corresponding to the design point \( \{ x_i^* \} = \{ r^*, d^*, l^* \}^{-1} \) as follows:

\[ z_i^* = \frac{x_i^* - \mu_{x_i}^*}{\sigma_{x_i}^*} \quad \text{i.e.} \quad z_1^* = \frac{r^* - \mu_r^*}{\sigma_r^*} ; z_2^* = \frac{d^* - \mu_d^*}{\sigma_d^*} ; z_3^* = \frac{l^* - \mu_l^*}{\sigma_l^*} \]  
(5.219)

7. Determine the partial derivatives of the limit state function with respect to the reduced variables.

\[ \{ G \} = \begin{bmatrix} G_1 \\ G_2 \\ G_3 \end{bmatrix} \quad \text{with} \quad G_i = -\frac{\partial g}{\partial z_i} \bigg|_{\{ z_i^* \}} \quad \text{evaluated at design point} \]  
(5.220)

\[ G_1 = -\frac{\partial g}{\partial z_1} \bigg|_{\{ z_i^* \}} = -\frac{\partial g}{\partial R} \bigg|_{\{ z_i^* \}} \quad \sigma^*_R = -1 \sigma^*_R \]

i.e.

\[ G_2 = -\frac{\partial g}{\partial z_2} \bigg|_{\{ z_i^* \}} = -\frac{\partial g}{\partial D} \bigg|_{\{ z_i^* \}} \quad \sigma^*_D = +1 \sigma^*_D \]

\[ G_3 = -\frac{\partial g}{\partial z_3} \bigg|_{\{ z_i^* \}} = -\frac{\partial g}{\partial L} \bigg|_{\{ z_i^* \}} \quad \sigma^*_L = +1 \sigma^*_L \]  
(5.221)

8. Calculate an estimate of \( \beta \) using the following formula:

\[ \beta = \frac{\{ G \}^T \{ z^* \}}{\sqrt{\{ G \}^T \{ G \}}} \quad \text{where} \quad \{ z^* \} = \begin{bmatrix} z_1^* \\ z_2^* \\ z_3^* \end{bmatrix} \]  
(5.222)

For linear performance functions, Equation (5.222) simplifies to
\[
\beta = \frac{a_0 + \sum_{i=1}^{n} a_i \mu_{x_i}}{\sqrt{\sum_{i=1}^{n} (a_i \sigma_{x_i})^2}} = \frac{\mu_R - \mu_D - \mu_L}{\sqrt{\sigma_R^2 + (\sigma_D)^2 + (\sigma_L)^2}}
\] (5.223)

9. Calculate a column vector containing the sensitivity factors using:

\[
\{\alpha\} = \frac{\{G\}^T \{G\}}{\sqrt{\{G\}^T \{G\}}
\] (5.224)

10. Determine a new design point in reduced variables for \((n - 1)\) of the variables using:

\[
z^*_i = \alpha_i \beta
\] (5.225)

11. Determine the corresponding design point values in original coordinates for the \((n - 1)\) values in Step 9 using:

\[
x^*_i = \mu_{x_i} + z^*_i \sigma_{x_i}
\] (5.226)

12. Determine the value of the remaining random variable (i.e., the one not found in steps 10 and 11) by solving the limit state function \(g = 0\).

13. Repeat steps 5 \div 12 until \(\beta\) and the design point \(\{x^*_i\}\) converge.

Equation (5.222) or Equation (5.223) shows the relation between the system resistance factor \(\phi_s\) and the system reliability index \(\beta\). For each imposed-to-permanent nominal load ratio \(k\), a range of \(\phi_s\) from 0.6 to 1.0 was considered to obtain numerically corresponding estimations for the system safety indices \(\beta\). A Matlab function was developed to carry out techniques introduced from step 1 to step 13 for all cases of reliability analysis that were categorised in Table 5.24. The main part of this Matlab function is provided in Appendix D.4. Since it is preferable to derive \(\beta\) for fixed values of \(\phi_s\), linear interpolation can be implemented between the adjacent \((\phi_s, \beta)\) points.

### 5.6.3.3. System resistance factor and reliability index relations

Reliability analysis was employed and results were obtained on the basis of the procedures described in the preceding Section. The system reliability index \(\beta\) versus the system resistance factor \(\phi_s\) curves for four reliability analysis cases are plotted in Figure 5.20 to Figure 5.23.

The system resistance factor \(\phi_s\) for each target system reliability index \(\beta\) and the corresponding ratio of the imposed-to-permanent nominal load \(k = L_n / D_n\) in the range of 0.25 to 5.0 are also presented in Table 5.25 to Table 5.28. In these tables, the overall weighted system resistance factor \(\phi_{s,w}\) was determined for each target \(\beta\) by taking into account the likelihood of occurrence of different \(L_n / D_n\) situations as per Equation (5.227).
where \( w_i \) is the frequency (weight) of the corresponding \((L_n/D_n)\) that was taken for aluminium material from Table 5.2a in [285], \( \phi_{si} \) is the system resistance factor corresponding to \((L_n/D_n)\).

\[
\phi_{s,w} = \sum w_i \phi_{si}
\]

(5.227)

Figure 5.20. System resistance factor and reliability index relations for Case 1

Figure 5.21. System resistance factor and reliability index relations for Case 2
Figure 5.22. System resistance factor and reliability index relations for Case 3

Figure 5.23. System resistance factor and reliability index relations for Case 4
Table 5.25. System resistance factor for Case 1

<table>
<thead>
<tr>
<th>$L_{ol}/D_n = 0.5$</th>
<th>Weight %</th>
<th>$\phi_s$</th>
<th>$\beta = 2.5$</th>
<th>$\beta = 2.75$</th>
<th>$\beta = 3.0$</th>
<th>$\beta = 3.25$</th>
<th>$\beta = 3.5$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25</td>
<td>0</td>
<td>1.038</td>
<td>1.014</td>
<td>0.990</td>
<td>0.966</td>
<td>0.943</td>
<td></td>
</tr>
<tr>
<td>0.5</td>
<td>0</td>
<td>1.072</td>
<td>1.038</td>
<td>1.004</td>
<td>0.970</td>
<td>0.936</td>
<td></td>
</tr>
<tr>
<td>1.0</td>
<td>6</td>
<td>1.076</td>
<td>1.029</td>
<td>0.983</td>
<td>0.938</td>
<td>0.894</td>
<td></td>
</tr>
<tr>
<td>1.5</td>
<td>17</td>
<td>1.071</td>
<td>1.017</td>
<td>0.965</td>
<td>0.915</td>
<td>0.866</td>
<td></td>
</tr>
<tr>
<td>2.0</td>
<td>22</td>
<td>1.066</td>
<td>1.008</td>
<td>0.952</td>
<td>0.899</td>
<td>0.848</td>
<td></td>
</tr>
<tr>
<td>3.0</td>
<td>33</td>
<td>1.058</td>
<td>0.996</td>
<td>0.936</td>
<td>0.880</td>
<td>0.826</td>
<td></td>
</tr>
<tr>
<td>5.0</td>
<td>22</td>
<td>1.051</td>
<td>0.984</td>
<td>0.921</td>
<td>0.861</td>
<td>0.806</td>
<td></td>
</tr>
<tr>
<td>Overall weighted $\phi_{s,w}$</td>
<td></td>
<td>1.062</td>
<td>1.002</td>
<td>0.944</td>
<td>0.889</td>
<td>0.837</td>
<td></td>
</tr>
</tbody>
</table>

Table 5.26. System resistance factor for Case 2

<table>
<thead>
<tr>
<th>$L_{ol}/D_n = 0.5$</th>
<th>Weight %</th>
<th>$\phi_s$</th>
<th>$\beta = 2.5$</th>
<th>$\beta = 2.75$</th>
<th>$\beta = 3.0$</th>
<th>$\beta = 3.25$</th>
<th>$\beta = 3.5$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25</td>
<td>0</td>
<td>1.015</td>
<td>0.986</td>
<td>0.958</td>
<td>0.931</td>
<td>0.904</td>
<td></td>
</tr>
<tr>
<td>0.5</td>
<td>0</td>
<td>1.059</td>
<td>1.022</td>
<td>0.986</td>
<td>0.951</td>
<td>0.915</td>
<td></td>
</tr>
<tr>
<td>1.0</td>
<td>6</td>
<td>1.074</td>
<td>1.025</td>
<td>0.977</td>
<td>0.931</td>
<td>0.885</td>
<td></td>
</tr>
<tr>
<td>1.5</td>
<td>17</td>
<td>1.073</td>
<td>1.017</td>
<td>0.963</td>
<td>0.911</td>
<td>0.861</td>
<td></td>
</tr>
<tr>
<td>2.0</td>
<td>22</td>
<td>1.070</td>
<td>1.010</td>
<td>0.953</td>
<td>0.897</td>
<td>0.845</td>
<td></td>
</tr>
<tr>
<td>3.0</td>
<td>33</td>
<td>1.065</td>
<td>1.000</td>
<td>0.939</td>
<td>0.880</td>
<td>0.825</td>
<td></td>
</tr>
<tr>
<td>5.0</td>
<td>22</td>
<td>1.059</td>
<td>0.990</td>
<td>0.924</td>
<td>0.863</td>
<td>0.806</td>
<td></td>
</tr>
<tr>
<td>Overall weighted $\phi_{s,w}$</td>
<td></td>
<td>1.067</td>
<td>1.005</td>
<td>0.945</td>
<td>0.889</td>
<td>0.835</td>
<td></td>
</tr>
</tbody>
</table>

Table 5.27. System resistance factor for Case 3

<table>
<thead>
<tr>
<th>$L_{ol}/D_n = 0.5$</th>
<th>Weight %</th>
<th>$\phi_s$</th>
<th>$\beta = 2.5$</th>
<th>$\beta = 2.75$</th>
<th>$\beta = 3.0$</th>
<th>$\beta = 3.25$</th>
<th>$\beta = 3.5$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25</td>
<td>0</td>
<td>1.009</td>
<td>0.983</td>
<td>0.958</td>
<td>0.933</td>
<td>0.908</td>
<td></td>
</tr>
<tr>
<td>0.5</td>
<td>0</td>
<td>1.007</td>
<td>0.971</td>
<td>0.936</td>
<td>0.900</td>
<td>0.865</td>
<td></td>
</tr>
<tr>
<td>1.0</td>
<td>6</td>
<td>0.978</td>
<td>0.931</td>
<td>0.886</td>
<td>0.842</td>
<td>0.800</td>
<td></td>
</tr>
<tr>
<td>1.5</td>
<td>17</td>
<td>0.957</td>
<td>0.906</td>
<td>0.857</td>
<td>0.810</td>
<td>0.764</td>
<td></td>
</tr>
<tr>
<td>2.0</td>
<td>22</td>
<td>0.944</td>
<td>0.890</td>
<td>0.838</td>
<td>0.789</td>
<td>0.743</td>
<td></td>
</tr>
<tr>
<td>3.0</td>
<td>33</td>
<td>0.927</td>
<td>0.870</td>
<td>0.817</td>
<td>0.766</td>
<td>0.718</td>
<td></td>
</tr>
<tr>
<td>5.0</td>
<td>22</td>
<td>0.912</td>
<td>0.852</td>
<td>0.796</td>
<td>0.744</td>
<td>0.695</td>
<td></td>
</tr>
<tr>
<td>Overall weighted $\phi_{s,w}$</td>
<td></td>
<td>0.936</td>
<td>0.880</td>
<td>0.828</td>
<td>0.778</td>
<td>0.731</td>
<td></td>
</tr>
</tbody>
</table>
### Table 5.28. System resistance factor for Case 4

<table>
<thead>
<tr>
<th>$L_n/D_n = 0.5$</th>
<th>Weight %</th>
<th>$\phi_s$</th>
<th>$\beta = 2.5$</th>
<th>$\beta = 2.75$</th>
<th>$\beta = 3.0$</th>
<th>$\beta = 3.25$</th>
<th>$\beta = 3.5$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25</td>
<td>0</td>
<td>0.988</td>
<td>0.959</td>
<td>0.931</td>
<td>0.903</td>
<td>0.875</td>
<td></td>
</tr>
<tr>
<td>0.50</td>
<td>0</td>
<td>0.998</td>
<td>0.961</td>
<td>0.923</td>
<td>0.886</td>
<td>0.850</td>
<td></td>
</tr>
<tr>
<td>1.0</td>
<td>6</td>
<td>0.978</td>
<td>0.930</td>
<td>0.883</td>
<td>0.838</td>
<td>0.794</td>
<td></td>
</tr>
<tr>
<td>1.5</td>
<td>17</td>
<td>0.961</td>
<td>0.908</td>
<td>0.857</td>
<td>0.808</td>
<td>0.761</td>
<td></td>
</tr>
<tr>
<td>2.0</td>
<td>22</td>
<td>0.949</td>
<td>0.893</td>
<td>0.840</td>
<td>0.789</td>
<td>0.741</td>
<td></td>
</tr>
<tr>
<td>3.0</td>
<td>33</td>
<td>0.934</td>
<td>0.875</td>
<td>0.819</td>
<td>0.767</td>
<td>0.717</td>
<td></td>
</tr>
<tr>
<td>5.0</td>
<td>22</td>
<td>0.919</td>
<td>0.858</td>
<td>0.800</td>
<td>0.746</td>
<td>0.696</td>
<td></td>
</tr>
<tr>
<td>Overall weighted $\phi_{s,w}$</td>
<td></td>
<td>0.941</td>
<td>0.884</td>
<td>0.830</td>
<td>0.778</td>
<td>0.730</td>
<td></td>
</tr>
</tbody>
</table>

#### 5.6.3.4. Simplified reliability analysis

Instead of using the iteration algorithm as described in the preceding Sections, a simplified expression, which is similar to the widely used equation for member-based design in current standards, can be employed for the calculation of the system resistance factor. For the LRFD criteria as given in Equation (5.185), the reliability index $\beta$ can be expressed as:

$$
\beta = \frac{\ln \left( \frac{R_m}{Q_m} \right)}{\sqrt{V_R^2 + V_Q^2}}
$$

(5.228)

In the above equation, $R_m$ and $Q_m$ are mean values of the resistance and load effects, respectively, and $(V_R, V_Q)$ are their corresponding coefficients of variations. In this development, no mention has been made of probability distributions where $\beta$ depends only on measures of central tendency and dispersion in the limit state function [285]. However, $\beta$ is still a useful comparative measure of reliability and can serve to evaluate the relative safety between various design methods.

The expressions of $R_m$ and $V_R$ were given in Equation (5.188) and Equation (5.189), respectively. At the limit state surface, by combining Equations ((5.188), (5.199)), the mean value of the resistance is obtained as:

$$
R_m = \left[ (\gamma_d + \gamma_h k) \frac{D_m}{(D_m/D_n)} \right] \left( \frac{M_n F_m P_m}{\phi_s} \right)
$$

(5.229)

The mean load effects, $Q_m$, and its coefficient of variation, $V_Q$ for a combination of dead and live loads, are defined as follows [290]:
where $D$ and $L$ are random variables representing the dead load and live load intensities, $D_{m}$ and $L_{m}$ are the mean values of dead load and live load, respectively, and $(V_D, V_L)$ are their corresponding coefficients of variations.

By using Equation (5.198), Equation (5.230) becomes:

$$Q_m = 1 + \left(\frac{L_m}{L_n}\right) k D_m$$

(5.232)

And Equation (5.231) can be rewritten in terms of the ratio of mean to nominal parameters as:

$$V_Q = \sqrt{\left(\frac{D_m}{D_n}\right)^2 V_D^2 + \left[k\left(\frac{L_m}{L_n}\right)\right]^2 V_L^2 \over \left(\frac{D_m}{D_n}\right) + k\left(\frac{L_m}{L_n}\right)}$$

(5.233)

By substituting Equations ((5.189), (5.229), (5.232),(5.233)) into Equation (5.228), the relations between the reliability index $\beta$ and the system resistance factor $\phi_s$ is obtained as follows:

$$\beta = \ln \left( \frac{1}{\phi_s} \left( M_m F_m P_m \right) \left(\frac{\left(\gamma_d + \gamma_l k\right)}{\left(\frac{D_m}{D_n}\right) + k\left(\frac{L_m}{L_n}\right)}\right) \right)$$

(5.234)

or

$$\phi_s = \frac{\left(\frac{\left(\gamma_d + \gamma_l k\right)}{\left(\frac{D_m}{D_n}\right) + k\left(\frac{L_m}{L_n}\right)}\right)}{\left( M_m F_m P_m \right) e^{-\beta \sqrt{V_D^2 + V_L^2 + V_P^2}}}$$

(5.235)

In these above equations, $(D_m/D_n)$ and $(L_m/L_n)$ are the mean-to-nominal ratios of dead load and live load respectively, and $k = (L_n/D_n)$ is the live load to dead load ratio.

If the number of tests is taken into account, a correction factor $C_p$ defined by (5.238) [198] can be used and thus the simple approximate expression for the estimation of the system resistance factor can be defined as:
\[ \phi_s = C_{\phi_s} \left( M_m F_m P_m \right) e^{-\beta \sqrt{\gamma_d^2 + \gamma_l^2} + C_P \gamma_d^2 + \gamma_l^2} \]  

(5.236)

where \( C_{\phi_s} \) is a calibration coefficient that depends on the Standards/Specifications due to the differences in load combinations and load factors.

\[ C_{\phi_s} = \frac{\left[ (\gamma_d + \gamma_l k) \right]}{\left( \frac{D_m}{D_n} + k \left( \frac{L_m}{L_n} \right) \right)} \]  

(5.237)

\[ C_p = \frac{n^2 - 1}{n^2 - 3n} \]  

(5.238)

in which \( n \) is the number of tests. In this study, \( C_p = 1.714 \), which was calculated based on the number of seven portal frame tests, can be used for all cases of reliability analysis.

By using the simplified expression (5.236), reliability analysis can be performed for four cases presented in Table 5.24. All needed parameters for the calculation of the system resistance factor are provided in Table 5.29, in which the statistical data for material, fabrication and professional factors were taken in Appendix D.4 and Table 4.4 and the stochastic models for the loads were obtained Table 5.23. The live load to dead load ratio \( L_n / D_n = 5.0 \) is assumed [198] in this study while the dead load factor \( \gamma_d \) and the live load factor \( \gamma_l \) were obtained properly from the AS/NZS 1170.0 [264] and the ACSE/SEI 7 [293]. The system resistance factor results for all cases are provided in Table 5.30 corresponding to the target system reliability indices \( \beta \) in the range of 2.5 to 3.5.

### Table 5.29. Parameters used in the simple approximate expression

<table>
<thead>
<tr>
<th>Cases</th>
<th>( M_m )</th>
<th>( V_M )</th>
<th>( F_m )</th>
<th>( V_F )</th>
<th>( P_m )</th>
<th>( V_P )</th>
<th>( \gamma_D )</th>
<th>( \gamma_L )</th>
<th>( \frac{L_m}{D_n} )</th>
<th>( \frac{D_m}{D_n} )</th>
<th>( V_D )</th>
<th>( \frac{L_m}{D_n} )</th>
<th>( V_L )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>1.09</td>
<td>0.04</td>
<td>0.98</td>
<td>0.01</td>
<td>0.98</td>
<td>0.01</td>
<td>1.20</td>
<td>1.50</td>
<td>5.00</td>
<td>1.05</td>
<td>0.10</td>
<td>0.80</td>
<td>0.25</td>
</tr>
<tr>
<td>Case 2</td>
<td>1.10</td>
<td>0.06</td>
<td>1.00</td>
<td>0.05</td>
<td>0.98</td>
<td>0.01</td>
<td>1.20</td>
<td>1.50</td>
<td>5.00</td>
<td>1.05</td>
<td>0.10</td>
<td>0.80</td>
<td>0.25</td>
</tr>
<tr>
<td>Case 3</td>
<td>1.09</td>
<td>0.04</td>
<td>0.98</td>
<td>0.01</td>
<td>0.98</td>
<td>0.01</td>
<td>1.20</td>
<td>1.60</td>
<td>5.00</td>
<td>1.05</td>
<td>0.10</td>
<td>1.00</td>
<td>0.25</td>
</tr>
<tr>
<td>Case 4</td>
<td>1.10</td>
<td>0.06</td>
<td>1.00</td>
<td>0.05</td>
<td>0.98</td>
<td>0.01</td>
<td>1.20</td>
<td>1.60</td>
<td>5.00</td>
<td>1.05</td>
<td>0.10</td>
<td>1.00</td>
<td>0.25</td>
</tr>
</tbody>
</table>

### Table 5.30. System resistance factors based on simplified expression

<table>
<thead>
<tr>
<th>Reliability analysis Cases</th>
<th>( V_Q )</th>
<th>( C_{\phi_s} )</th>
<th>( C_P )</th>
<th>( \phi_s )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( \beta = 2.5 )</td>
<td>( \beta = 2.75 )</td>
<td>( \beta = 3.0 )</td>
<td>( \beta = 3.25 )</td>
</tr>
<tr>
<td>Case 1</td>
<td>0.199</td>
<td>1.723</td>
<td>1.714</td>
<td>1.080</td>
</tr>
<tr>
<td>Case 2</td>
<td>0.199</td>
<td>1.723</td>
<td>1.714</td>
<td>1.085</td>
</tr>
<tr>
<td>Case 3</td>
<td>0.207</td>
<td>1.521</td>
<td>1.714</td>
<td>0.935</td>
</tr>
<tr>
<td>Case 4</td>
<td>0.207</td>
<td>1.521</td>
<td>1.714</td>
<td>0.940</td>
</tr>
</tbody>
</table>
### 5.6.3.5. The system resistance factor

The weighted average system resistance factor ($\phi_s$) is plotted against the system reliability index ($\beta$) in Figure 5.24 for different analysis cases (Table 5.24) using the FORM. The $\phi_s$-$\beta$ relations derived from the simplified standardised equation (Equation (5.236)) is also presented in Figure 5.24, represented by the dashed lines.

**Figure 5.24. Plots of weighted $\phi_{s,w}$ versus $\beta$ for cold-rolled aluminium portal frames**

As can be seen in Figure 5.24, for a given system resistance factor, the AS/NZS 1170.0 load combination provides a higher safety level indicated by higher values of $\beta$ compared to the ASCE/SEI 7-10 load combination. The reason for this is mainly due to the use of different mean-live-load to nominal-live-load ratio and the live load factor in the respective load combinations. The same characteristic is also presented in results derived from Equation (5.236). However, it is clear that the $\phi_s$-$\beta$ relations predicted by Equation (5.236) did not accurately align with the $\phi_s$-$\beta$ curves obtained from FORM analysis, especially when the relations were derived based on the AS/NZS 1170.0 load combination. This means that it may not appropriate to use Equation (5.236) to derived the system resistance factors for all analysis cases.

It is well known that the target reliability index of 2.5 is often adopted for the design of cold-formed members. Hence, the same value is also assumed to be used for the evaluation of system strength of CRA portal frames. For this, the system resistance factors corresponding to the four analysis cases were determined for achieving $\beta$ of 2.5 and are provided in Table 5.31.
Table 5.31. Summary of system resistance factor for $\beta = 2.5$

<table>
<thead>
<tr>
<th>Reliability analysis Cases</th>
<th>$\Phi_s$ (FORM)</th>
<th>$\Phi_s$ (Simplified Equation)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>1.000</td>
<td>1.000</td>
</tr>
<tr>
<td>Case 2</td>
<td>1.000</td>
<td>1.000</td>
</tr>
<tr>
<td>Case 3</td>
<td>0.936</td>
<td>0.935</td>
</tr>
<tr>
<td>Case 4</td>
<td>0.941</td>
<td>0.940</td>
</tr>
</tbody>
</table>

In Figure 5.24 and Table 5.31, the analysis Case 1 and Case 3 represent the respective AS/NZS 1170.0 and ASCE/SEI 7-10 load combinations with statistical parameters determined based on the measured material properties and geometry from experiments carried in this study, referred to as “BlueScope Permalite material Case”, while the analysis Case 2 and Case 4 pertain to the respective AS/NZS 1170.0 and ASCE/SEI 7-10 load combinations with statistical parameters related to the general extruded aluminium structural sections, which are provided in AA2015 [198]. It is revealed that the system resistance factor is sensitive to the statical parameters, live load to dead load ratio, and the load combinations. Although the utilise of the small sample size, Case 1 and Case 3 yield slightly lower values of system resistance factors compared to respective Case 2 and Case 4 for achieving $\beta$ of 2.5. The reason for this is that the mean-to-nominal ratio for fabrication and material properties of CRA sections are lower than the extruded sections. Hence, it is recommended that the use of statistical data based on extruded aluminium sections may not be satisfactory for the determination of system resistance factors for CRA frames in particular and CRA members and structures in general. This emphasizes the importance of future studies on the statistical analysis of the material, geometrical, and imperfection characteristics of CRA sections.

5.7. DISCUSSIONS AND CONCLUSIONS

In this chapter, the determination of the ultimate strength of cold-rolled aluminium portal frames based on the conventional methods and the Direct Design Method (DDM) using Advanced Analysis is presented.

In the traditional design approaches, the structural systems are treated as individual members and components and it is assumed that the system reaches the ultimate capacities when the critical member reaches its capacities. Hence, towards determining the frame strength, the capacities of the critical member are required. For this purpose, the elastic buckling load ($N_{oc}$) for members in compression and elastic buckling moment ($M_o$) for members in bending are determined from three approaches including the effective length, the energy, and the buckling analysis methods. Although different methods were used, the obtained elastic buckling loads and moments are comparable, resulting in similar member capacities for each design.
standards. The capacities of the member in compression and bending were calculated separately and independently, and the interaction of axial compression and bending effects was then examined through the use of linear interaction equations. It is found that all the current standards of aluminium sections are conservatively predicted the ultimate strength of the portal frame systems with a high degree of variation. Therefore, using design guidelines of extruded aluminium profiles for the design of CRA portal frames may not be very appropriate.

The design features that are relevant to the DDM using Advanced Analysis in predicting the nominal strength of CRA portal frames are also demonstrated. This emphasizes the need for accurate modelling of material properties, geometric nonlinearities, and nonlinear constitutive relations for deformable fasteners. The shell elements are recommended to use in the full three-dimensional models of the frames due to its capability of capturing all instabilities in the frame systems, which may not always be possible with the use of beam element models. With the actual strength-to-nominal strength ratios are close to 1 along with a low coefficient of variation, the DDM using Advanced Analysis has been proved to be a reliable and promising method for the design of CRA portal frames. The attractiveness of the method is its capability of including all nonlinearities in a single-step analysis by which the system effect can be fully captured and the frame capacities can be obtained directly from the load-displacement response without a need to go through a lengthy process as in the conventional methods.

A system reliability analysis was performed to determine the system resistance factors for the CRA portal frames subjected to gravity load combinations. It was revealed that the system resistance factor is sensitive to the statical parameters, live load to dead load ratio, and the load combinations. Although there is the existence of a certain degree of variation in the system resistance factor between different analysis cases, the obtained system resistance factors for all cases is higher than 0.9 for achieving the minimum target system reliability of 2.5. Therefore, a system resistance factor of $\phi_s = 0.9$ is recommended for the design of CRA portal frames.
CHAPTER 6. CONCLUSIONS

A comprehensive study on cold-rolled aluminium back-to-back channel section portal frames was performed. This included an extensive experimental study on full-scale portal frames and components along with numerical modelling, design, and system reliability analysis of the cold-rolled aluminium frame systems.

Although portal frames have been commonly used in the construction industry from steel structures, this end-user application has not been previously attempted by any producer of cold-rolled aluminium (CRA) profiles. Hence, a preliminary design is required to develop a full-scale portal frame system with the main columns and rafter made from CRA double C-sections. In this study, the system consisted of two-bay single-span portal frames connected in parallel by a series of purlins along the rafters to simulate a realistic free-standing frame was designed for testing. The knee braces and rafter tie were provided to transfer large moment between the rafters and columns and between the rafters. This frame configuration can serve as a benchmark for portal frame systems composed of CRA sections.

The original/typical frame configuration, as described above, was tested to investigate the frame behaviour and determine the corresponding ultimate frame strength for both vertical loading and combined horizontal and vertical loading conditions. It was demonstrated that the failure mode of the frames is identical for both loading conditions as the frames exhibited the dominance of flexural-torsional deformation of the columns.

The first modification to the original frame configuration was conducted by adding girts to the columns along their heights, resulting in partial restraint to the column web laterally and torsionally. It was determined that the proving girt system resulted in significant increases in the frame ultimate vertical loads, especially when the frames were subjected to combined horizontal and vertical loads, while the twist rotations of the columns were reduced substantially. This indicates that column bracing is essential, where practically possible, to improve the frame system performance.

Other modifications were also introduced to evaluate the effect of the rafter tie on the strength and deformation of the frame system and examine the possibility of optimising the frame strength. In particular, the rafter tie was removed from the original frame configuration in Frame Test 6 while, in contrast, sleeve stiffeners were placed to the column at the knee connections in Frame Test 7. The test results indicated that the introduction of rafter tie to the cold-rolled aluminium portal frame not only helps to enhance the ultimate vertical load but also reduce the deflections of the frame system. Therefore, it was recommended to utilise the rafter tie in the portal frames, especially in the systems with larger span. It was also determined that the addition of sleeve stiffeners to the columns provided an impressively higher ultimate load capacity, and the south column and rafter were failed at the same time by the formation of a
spatial plastic mechanism, indicating that this frame configuration is optimal in terms of the structural design.

In all Frame Tests, there was no failure occurred at any connection of the frame system. This indicated that the connections used in the experimental program were sufficiently designed, and thus they can serve as benchmarks.

Besides the full-scale frame tests, component tests were conducted to obtain the necessary data to further studies. A series of coupon tests to determine the actual material properties of brackets and cold-rolled aluminium sections used in the experimental investigation. It was shown that a certain degree of anisotropy in mechanical properties of the AC25025 section was exhibited and the strength of corner regions is enhanced considerably due to the cold-working process. The material properties of the flat parts of the cross-section for compression and tension, however, are almost identical. The properties derived from the coupon tests also showed that the material used for brackets was slightly stiffer and stronger compared with that used for the cold-rolled sections. The constitutive relations of the bolted connections were established by the tests on point fastener connections in shear and under torque. These characteristics were used to define the fastener properties adopted in numerical models.

The flexural stiffness of the column base connections was measured in both the full-frame tests and the isolated component tests. It was shown that a substantial dispersion was exhibited in the base stiffness despite of having identical geometry and adopting the same method to fasten connection components. The experiments also indicated that the base connection brackets play an important role in the formation of the base connection stiffness. The flexural stiffness is an important parameter in representing the semi-rigid behaviour of the base connection especially in frames modelled with beam elements.

Numerical finite element models using shell elements and advanced analysis of the cold-rolled aluminium portal frames were developed and calibrated. It was indicated that the shell element-based advanced analysis can accurately predict both the ultimate strength and behaviour of the frame systems obtained from the full-scale frame tests. The nearly perfect agreement between the numerical prediction and the actual behaviour was partly attributed to the incorporation of nonlinear material properties obtained from coupon tests and geometric imperfections measured from experiments. Another important factor is the use of deformable mesh-independent point-based fasteners representing the behaviour of bolted connections. Hence, it is apparent that the utilise of deformable point-based fasteners has proven to be an efficient method of modelling bolts in FE models.

In the context of system strength and behaviour, good agreements were achieved between the FE models and experimental results where neither residual stresses nor the anisotropy of material properties was modelled. Therefore, the conclusion may be drawn upon that the unfavourable effects of residual stresses and material anisotropy are negligible and thus, modelling of these features may be not required.
Calibrated modelling technique was used to carry out parametric studies. The results demonstrated that the column base stiffness had a considerable influence on the ultimate vertical loads and deformations of the frame system, especially when the frames were subjected to combined wind and gravity loads. Therefore, methods to enhance the base stiffness and reduce the variation in stiffness should be further investigated. Also, column bracing is essential to improve the structural performance of the frame system. This was evident by the remarkable contribution of either fly bracings along the columns or fly braces at knee connection to the load capacity of the overall frame system. In addition, a larger span frame with unbraced columns was created by numerical modelling to predict the ultimate strength and behaviour of the system. The results showed that the frame failed due to flexural-torsional buckling of the columns which is the same as the experimental results of unbraced column frames of a shorter span. Therefore, numerical finite element modelling is an efficient tool to develop new configurations of CRA portal frames.

The current standards/specifications of extruded aluminium structures were evaluated whether they are applicable to the design of portal frames composed of CRA sections. It was described that the current design codes were over-conservative in predictions of the ultimate strength of the CRA portal frames. This could be explained that these guidelines are premised on research on extruded sections and do not allow the increase in strength produced by cold-forming to be accounted for. In addition, the conventional methods in these standards were member-based so that the frame strength was predicted based on the prediction of member capacity for members under stress resulting from combined compression and bending. The system effect was only reflected through the use of effective length factors which may not sufficient to capture the complex interactions between members of a structural system.

In contrast, the DDM using Advanced Analysis presents a more realistic approach to the design of CRA portal frames, which allows not only the ultimate frame strength but also the behaviour of the frame systems to be accurately predicted. The prediction of the nominal strength of CRA frame systems by advanced analysis is based on the complete model of the frames, in which all aspects of uncertainties including the fastener properties, connection flexibilities, imperfections and non-linear material properties can be explicitly incorporated. This also allows the complex system effect and load redistribution to be captured. The subsequent advantage offered by the Advanced Analysis is that the nominal strength of the frame system is directly determined in a single-step process without requiring the lengthy process of checking individual member strengths. Therefore, it can be stated that DDM using Advanced Analysis is a robust, routine design method in the near future for all types of structures including those comprised of CRA sections.

A system reliability analysis was performed for the CRA portal frames subjected to gravity load combinations.
Recommendations for further research

It is recommended that further research could be carried out to improve the cold-rolled aluminium portal frame system studied in this thesis and also extend the knowledge base in this field, as follows:

1. In this study, the cross-section of the main members was back-to-back channels, in which full composite action between the constituent channel sections was implicitly assumed. The sections, however, were formed by connecting two channel sections through their webs using discrete bolts having a longitudinal spacing of 1030 mm. As a result, the full composite action assumption may not be entirely accurate. This requires further studies to be conducted to quantify the effectiveness of these built-up sections.

2. The base connections have a profound effect on the performance of frame systems, but it was indicated that there is a certain level of variation in the base stiffness. Therefore, studying methods to improve the performance of the base connections and reduce the dispersion of the base stiffness is necessary.

3. Single C-section portal frames are potentially in use for a small to moderate span. However, the behaviour of this configuration may relatively differ from that of double symmetric section portal frames. Hence, further studies should be completed to determine the strength and behaviour of CRA single C-section portal frames.

4. The experiments completed in this study showed that cold-rolled aluminium portal frames exhibited good performance in the structural point of view, especially when column bracings and stiffeners were provided. Therefore, further research on strengthening methods for columns and rafter at critical locations should be performed to optimise the strength of the frame system. Also, the failure mode of unbraced column frames was flexural-torsional buckling of the columns. The numerical study indicated that providing a lateral restraint to the columns at the knee connection can improve the frame strength. Therefore, more investigations on the bracing for columns should be performed without the reduction in clear space due to the presence of bracings.

5. Studies should be performed to optimise the connection to reduce the total costs of the structure. There was no sign of any failure that occurred at the connections in all experiments indicating that the load capacity of the designed connections was higher than that of the members. Although it is the expectation in the design of frame systems, it is also possible that these connections were over-designed. Therefore, the optimisation in the design of connection is necessary to reduce the total cost of the structure.

6. It is also recommended that larger spans should be tested to evaluate the applicability of larger span portal frames composed of cold-rolled aluminium sections in practice.
7. The effect of stress skin diagram action from roof sheeting and wall cladding on cold-rolled aluminium portal frames should also be carried out.

8. For the design of CRA portal frames based on DDM using Advanced Analysis, one of the challenges is to determine the system resistance factors using a system reliability analysis, which is mainly contributed to the lack of statistical data. In this research, the statistical parameters related to the frame strength were obtained from the experiments and their corresponding numerical models while the statistical parameters for BlueScope Permalite’s materials were obtained only from measured data from experiments of the study. Therefore, statistical data should be expanded to cover a wider range of structure members. In the same manner, the load combinations considered in this study are limited to gravity load combinations. More loading types such as wind, snow, and earthquake loads are required to consider for deriving the system resistance factors.
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### APPENDIX A. ADDITIONAL MATERIALS FOR FULL-SCALE FRAME TESTS AND SUPPLEMENTARY RESULTS

#### A.1. FRAME DRAWINGS

Table A.1. List of drawings for the experimental program in this study

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<td>Structural drawing</td>
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<td>Apex bracket for AC250 Rafters</td>
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<td>AC150 Knee brace to AC250 rafter bracket (KBR)</td>
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<td>AC200 Rafter tie to AC250 Rafter bracket</td>
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<td>Frame elevation with cross-bracings</td>
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<td>Loading arrangement and details</td>
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<td>Plate dimensions for M12 snug-tightened connections in shear</td>
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<td>Plate dimensions for M16 slip-resistance connections under torque</td>
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BOLT HOLE LOCATION AT TOP COLUMN AND KNEE BRACE REGION (COLUMN ON GRID 1)

BOLT HOLE LOCATION AT TOP COLUMN AND KNEE BRACE REGION (COLUMN ON GRID 3)

BOLT HOLE LOCATION AT COLUMN BASE (COLUMN ON GRID 1 AND 3)
RAFTER C25025 (SPANNING FROM GRID 3 TO APEX)

BOLT HOLE LOCATION ON RAFTER (SPANNING FROM GRID 3 TO APEX)
PURLIN C15025

BOLT HOLE LOCATION ON PURLIN

END FRAME REGION

PURLIN C15025 SECTION

SIDE VIEW

END VIEW

TEST FRAME REGION

BACK (WEB SIDE)

FRONT (UP SIDE)

TOP SIDE

BOTTOM SIDE

PURLIN DETAILS

DR. CAO HUNG PHAM
PROF. TIM RASMUSSEN

Scale 1:12

3038

2.5 MAY 2018
# Material Requirement for the Experimental Testing of Back-to-Back Channel Section Cold-Rolled Aluminium Portal Frames

## Statistical Table of Members

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GRADE | 5083-H321/H116
THICKNESS | 6mm
NOTE | NL29 - AL-L - AS DRAWN
NL29 - AL-R - OPPOSITE HAND
MATERIAL: ALUMINIUM
GRADE: 5083 = H321/H116
THICKNESS: 6mm
NOTE: RT20L = AS DRAWN
      RT20R = OPPOSITE HAND

COLD-ROLLED ALUMINIUM DOUBLE CHANNEL Portal Frame

DRAWN BY: NGUYEN
PRINTED BY: PERMALITE
SCHOOL OF CIVIL ENGINEERING
THE UNIVERSITY OF SYDNEY

Scale: 1:8
Date: 23 MAY 2018
FIGURE

BASE CLEAT

4 HOLES ø18

BASE CLEAT WASHER

2 HOLES ø24

FLAT DEVELOPMENT

MATERIAL: STAINLESS STEEL

GRADE: 2205

THICKNESS: 8mm

NOTE: BC25-SS & BW25-SS

TABLE
**Material:** STEEL

**Grade:** 2205

**Thickness:** 8mm

**Note:** BC25-SS
# Statistical Table of Cleats

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![Diagram of a steel bracket with dimensions and notes.](image-url)
BASE CLEAT WASHER

BASE CLEAT

4 HOLES #18

BASE CLEAT - BC25-SS

2 HOLES #24

BASE CLEAT WASHER - BW25-SS

Material: STAINLESS STEEL
Grade: 2205
Thickness: 6mm
Note: BC25-SS & BW25-SS
BASE CLEAT WASHER

4 HOLES #18

BASE CLEAT - BC25-SS

2 HOLES #24

BASE CLEAT WASHER - BW25-SS

MATERIAL: STAINLESS STEEL

GRADE: 2205

THICKNESS: 3mm

NOTE: BC25-SS & BW25-SS

FLAT DEVELOPMENT
COLD-ROLLED ALUMINIUM PLATE
EXTRACTED FROM AC25025
CR-AL-PL1

ALUMINIUM PLATE EXTRACTED
FROM BRACKETS
AL-P1
COLD-ROLLED ALUMINIUM PLATE
EXTRACTED FROM AC25025
CR-AL-PL2

2 Ø16x22 LONG
ELONGATED HOLES

325
35
40 40
80

ALUMINIUM PLATE EXTRACTED
FROM BRACKETS
AL-PL2

2 Ø16x30 LONG
ELONGATED HOLES

325
160
160
40 40
80
A.2. LOAD CELL CALIBRATION

In Frame Tests, the reactions under each column were calculated based on the reactions from four supporting load cells, as described in Section 3.2.10.9.1. For this purpose, the load cells were calibrated before performing the full-scale frame tests to provide reactions based on the strains that recorded during testing by four strain gauges attached to the shank of the load cell. The load cell arrangement and the development of the force – strain relations for the load cells are demonstrated below.

A.2.1. Load cell arrangement

The columns were attached to the strong floor through the base connection, as shown in Figure A.1. The L-shaped base brackets were bolted to a 25mm thick upper base plate which is supported on four cylindrical load cells. These load cells were arranged on the lower base plate clamped to the universal beam on the strong floor. The load cell configuration on the base plate is shown in Figure A.2.

Figure A.1. Details of the base connection
Four strain gauges, which have a gauge length of 5 mm, gauge resistance of 120 Ω, and a gauge factor of 2.10, were attached to each load cell. These four strain gauges are distributed equidistantly from each other around the periphery of the shank of load cells.

Figure A.2. Load cells arrangement: (a) The base connection in Full-scale tests; (b) Load cell arrangement on the lower base plate; (c) Single load cell showing strain gauge attachment; (d) Calibration of load cell

A.2.2. Calibration procedure

The load cells were calibrated in the Sintech Tensile Testing Machine. The incremental vertical load was applied to the load cell through a smooth cylindrical metallic cushion with a movable upper part of hemispherical-shape (Figure A.2d). The use of the cylindrical cushion not only allows the load to be uniformly distributed on the cross-section of the load cell but also prevent the load cell from being damaged during calibrating. The load was gradually applied to each load cell from 0 to until 10kN. The reason for the limitation of 10kN is that it is crucial not to subject the load cell to higher loads lest it yields and renders useless. The average of the four strain gauge readings in each load cell corresponding to vertical loads are presented in Figure A.3 and Figure A.4.
The linear regression line was run through the four readings to obtain the force-strain relationship, producing:

\[ \text{Force (kN)} = 0.1369 \times \text{microStrain} \quad (A.1) \]

\[ \text{Force (kN)} = 0.1377 \times \text{microStrain} \quad (A.2) \]
A.3. MEASURED SECTION GEOMETRY

A.3.1. Measured sectional dimensions
## Table A.2. Measurement of the sectional dimensions for Frame Test 1

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Measured values are provided for the thickness, depth, and flange dimensions, with respective nominal values given for comparison. The thickness is measured and compared to the nominal thickness, and similarly for depth and flange.
Table A.8. Measurement of the sectional dimensions for Frame Test 7

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A.3.2. Measured sectional and member imperfections

Figure A.5. Imperfections at the two ends for the south column of Frame Test 1: (a) End section A; (b) End section B
### Appendix A

#### Line 5

![Line 5 Graph](image)

#### Line 6

![Line 6 Graph](image)

#### Line 7

![Line 7 Graph](image)

#### Line 8

![Line 8 Graph](image)

#### Line 9

![Line 9 Graph](image)

#### Line 10

![Line 10 Graph](image)

#### Line 11

![Line 11 Graph](image)

#### Line 12

![Line 12 Graph](image)

#### Line 13

![Line 13 Graph](image)

#### Line 14

![Line 14 Graph](image)
Figure A.6. Sectional and member imperfections for the south column of Frame Test 1
Figure A.7. Imperfections at the two ends for the north column of Frame Test 1: (a) End section A; (b) End section B
Figure A.8. Sectional and member imperfections for the north column of Frame Test 1
Figure A.9. Imperfections at the two ends for the south rafter of Frame Test 1: (a) End section A; (b) End section B
Figure A.10. Sectional and member imperfections for the south rafter of Frame Test 1
Figure A.11. Imperfections at the two ends for the north rafter of Frame Test 1: (a) End section A; (b) End section B

Line 1

Line 2

Line 3

Line 4

Line 5

Line 6
Figure A.12. Sectional and member imperfections for the north rafter of Frame Test 1
Figure A.13. Imperfections at the two ends for the south column of Frame Test 2: (a) End section A; (b) End section B
Figure A.14. Sectional and member imperfections for the south column of Frame Test 2
Figure A.15. Imperfections at the two ends for the north column of Frame Test 2: (a) End section A; (b) End section B
Figure A.16. Sectional and member imperfections for the north column of Frame Test 2
Appendix A

Figure A.17. Imperfections at the two ends for the south rafter of Frame Test 2: (a) End section A; (b) End section B

Line 1

Line 2

Line 3

Line 4

Line 5

Line 6
Figure A.18. Sectional and member imperfections for the south rafter of Frame Test 2
Figure A.19. Imperfections at the two ends for the north rafter of Frame Test 2: (a) End section A; (b) End section B
Figure A.20. Sectional and member imperfections for the north rafter of Frame Test 2
Figure A.21. Imperfections at the two ends for the south column of Frame Test 3: (a) End section A; (b) End section B
Figure A.22. Sectional and member imperfections for the south column of Frame Test 3
Figure A.23. Imperfections at the two ends for the north column of Frame Test 3: (a) End section A; (b) End section B
Figure A.24. Sectional and member imperfections for the north column of Frame Test 3
Figure A.25. Imperfections at the two ends for the south rafter of Frame Test 3: (a) End section A; (b) End section B
Appendix A

Line 7

Line 8

Line 9

Line 10

Line 11

Line 12

Line 13

Line 14

Line 15

Line 16
Figure A.26. Sectional and member imperfections for the south rafter of Frame Test 3
Figure A.27. Imperfections at the two ends for the north rafter of Frame Test 3: (a) End section A; (b) End section B
Figure A.28. Sectional and member imperfections for the north rafter of Frame Test 3
Figure A.29. Imperfections at the two ends for the south column of Frame Test 4: (a) End section A; (b) End section B
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Figure A.30. Sectional and member imperfections for the south column of Frame Test 4
Figure A.31. Imperfections at the two ends for the north column of Frame Test 4: (a) End section A; (b) End section B
Figure A.32. Sectional and member imperfections for the north column of Frame Test 4
Figure A.33. Imperfections at the two ends for the south rafter of Frame Test 4: (a) End section A; (b) End section B
Figure A.34. Sectional and member imperfections for the south rafter of Frame Test 4
Figure A.35. Imperfections at the two ends for the north rafter of Frame Test 4: (a) End section A; (b) End section B
Appendix A

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Line 13

Line 14

Line 15

Line 16
Figure A.36. Sectional and member imperfections for the north rafter of Frame Test 4
Figure A.37. Imperfections at the two ends for the south column of Frame Test 5: (a) End section A; (b) End section B.
Figure A.38. Sectional and member imperfections for the south column of Frame Test 5
Figure A.39. Imperfections at the two ends for the north column of Frame Test 5: (a) End section A; (b) End section B
Figure A.40. Sectional and member imperfections for the north column of Frame Test 5
Figure A.41. Imperfections at the two ends for the south rafter of Frame Test 5: (a) End section A; (b) End section B
Figure A.42. Sectional and member imperfections for the south rafter of Frame Test 5
Figure A.43. Imperfections at the two ends for the north rafter of Frame Test 5: (a) End section A; (b) End section B

(Chart A.43 continued)
Figure A.44. Sectional and member imperfections for the north rafter of Frame Test 5
Figure A.45. Imperfections at the two ends for the south column of Frame Test 6: (a) End section A; (b) End section B

Line 1

Line 2

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Line 6
Figure A.46. Sectional and member imperfections for the south column of Frame Test 6
Figure A.47. Imperfections at the two ends for the north column of Frame Test 6: (a) End section A; (b) End section B
Figure A.48. Sectional and member imperfections for the north column of Frame Test 6
Figure A.49. Imperfections at the two ends for the south rafter of Frame Test 6: (a) End section A; (b) End section B
Figure A.50. Sectional and member imperfections for the south rafter of Frame Test 6
Figure A.51. Imperfections at the two ends for the north rafter of Frame Test 6: (a) End section A; (b) End section B
Figure A.52. Sectional and member imperfections for the north rafter of Frame Test 6
Figure A.53. Imperfections at the two ends for the south column of Frame Test 7: (a) End section A; (b) End section B
## Appendix A

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Figure A.54. Sectional and member imperfections for the south column of Frame Test 7
Figure A.55. Imperfections at the two ends for the north column of Frame Test 7: (a) End section A; (b) End section B
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<tr>
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<td>Imperfection (mm)</td>
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<tr>
<td>Distance along the specimen (m)</td>
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<td>Imperfection (mm)</td>
</tr>
<tr>
<td>Distance along the specimen (m)</td>
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<td>Imperfection (mm)</td>
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<tr>
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Figure A.56. Sectional and member imperfections for the north column of Frame Test 7
Figure A.57. Imperfections at the two ends for the south rafter of Frame Test 7: (a) End section A; (b) End section B
Figure A.58. Sectional and member imperfections for the south rafter of Frame Test 7.
Figure A.59. Imperfections at the two ends for the north rafter of Frame Test 7: (a) End section A; (b) End section B

Line 1

Line 2

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Appendix A

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Line 15

Line 16

Imperfection (mm) vs. Distance along the specimen (m) for different lines and measurements compared to Fourier results.

- Line 7: Imperfection vs. Distance along the specimen
- Line 8: Imperfection vs. Distance along the specimen
- Line 9: Imperfection vs. Distance along the specimen
- Line 10: Imperfection vs. Distance along the specimen
- Line 11: Imperfection vs. Distance along the specimen
- Line 12: Imperfection vs. Distance along the specimen
- Line 13: Imperfection vs. Distance along the specimen
- Line 14: Imperfection vs. Distance along the specimen
- Line 15: Imperfection vs. Distance along the specimen
- Line 16: Imperfection vs. Distance along the specimen

Each line shows the comparison between measurement and Fourier results, with the x-axis representing distance along the specimen and the y-axis representing imperfection in millimeters.
Figure A.60. Sectional and member imperfections for the north rafter of Frame Test 7
### A.3.3. Statistic of representative parameters

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<th>Specimen</th>
<th>$G_1$ (mm)</th>
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<th>$G_1/L$</th>
<th>$G_2/L$</th>
<th>$G_3/L$ (deg/m)</th>
<th>$d_1$ (mm)</th>
<th>$d_2$ (mm)</th>
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<tr>
<td>South Column 1</td>
<td>0.626</td>
<td>0.367</td>
<td>1.066</td>
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<td>1/14267</td>
<td>0.204</td>
<td>0.706</td>
<td>1.971</td>
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<td>0.638</td>
<td>1.288</td>
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<td>1/8194</td>
<td>0.246</td>
<td>0.565</td>
<td>1.842</td>
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<td>0.307</td>
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<td>1/8608</td>
<td>0.059</td>
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<tr>
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<td>1.029</td>
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<td>0.069</td>
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<td>1.149</td>
<td>0.470</td>
<td>1/5156</td>
<td>1/4552</td>
<td>0.090</td>
<td>0.715</td>
<td>1.776</td>
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<tr>
<td>North Column 3</td>
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<td>1.107</td>
<td>0.238</td>
<td>1/10097</td>
<td>1/4727</td>
<td>0.046</td>
<td>0.678</td>
<td>1.831</td>
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<td>South Column 4</td>
<td>0.820</td>
<td>1.086</td>
<td>0.300</td>
<td>1/6379</td>
<td>1/4820</td>
<td>0.057</td>
<td>0.472</td>
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<td>North Column 4</td>
<td>0.595</td>
<td>0.551</td>
<td>0.481</td>
<td>1/8795</td>
<td>1/9488</td>
<td>0.092</td>
<td>0.469</td>
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<td>0.879</td>
<td>0.608</td>
<td>0.431</td>
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<td>0.082</td>
<td>0.496</td>
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<td>North Column 5</td>
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<td>0.091</td>
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<td>1.000</td>
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<td>1/5232</td>
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Table A.10. Sectional and member imperfections for the rafters

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<th>( G_3 ) (deg)</th>
<th>( G_1/L )</th>
<th>( G_2/L )</th>
<th>( G_3/L ) (deg/m)</th>
<th>( d_1 ) (mm)</th>
<th>( d_2 ) (mm)</th>
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<td>1.331</td>
<td>0.618</td>
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<td>0.362</td>
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<td>North Rafter 1</td>
<td>0.321</td>
<td>1.587</td>
<td>0.581</td>
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<td>1/4525</td>
<td>0.081</td>
<td>1.013</td>
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<tr>
<td>South Rafter 2</td>
<td>1.211</td>
<td>0.653</td>
<td>1.546</td>
<td>1/5933</td>
<td>1/11006</td>
<td>0.215</td>
<td>0.583</td>
<td>1.925</td>
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<tr>
<td>North Rafter 2</td>
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<td>2.326</td>
<td>0.774</td>
<td>1/17146</td>
<td>1/3088</td>
<td>0.108</td>
<td>0.550</td>
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<tr>
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<td>0.055</td>
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<td>1/2141</td>
<td>0.081</td>
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<td>1/2907</td>
<td>0.041</td>
<td>0.774</td>
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<td>0.598</td>
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<td>1/5612</td>
<td>0.101</td>
<td>0.612</td>
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<tr>
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<td>1.963</td>
<td>1.154</td>
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<td>0.624</td>
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<td>0.151</td>
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<td>0.202</td>
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<td>1/10247</td>
<td>0.116</td>
<td>0.942</td>
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<tr>
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<td>1.055</td>
<td>0.970</td>
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<td>1/6810</td>
<td>0.135</td>
<td>0.638</td>
<td>2.037</td>
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| Mean           | 1/6041         | 1/4375         | 0.134          | 0.730       | 2.192       |
| COV            | 0.708          | 0.570          | 0.628          | 0.250       | 0.128       |
### A.4. WEIGHT OF LOADING SYSTEM AND SELF-WEIGHT OF THE FRAME TESTS

**Table A.11. Self-weight on rafter**

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<th>Total Weight (kg)</th>
</tr>
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**Total self-weight on rafter: 247.82**
## Table A.12. Weight of loading rig

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**Total weight of load spreading system** 489.61
### Table A.13. Self-weight of frame (Frames 1, 2, and 3)

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**Total weight of the frame** 437.20
Table A.14. Self-weight of frame (Frames 4 and 5)

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**Total weight of the frame** 515.52
## Table A.15. Self-weight of frame 6

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**Total weight of the frame** 544.65
A.5. ADDITIONAL EXPERIMENTAL RESULTS

Figure A.61. Applied load versus horizontal (in-plane) displacement of the apex

Figure A.62. Applied load versus lateral (out-of-plane) displacement of the apex
Figure A.63. Applied load versus twist rotation of columns at 180 mm above the base plate

Figure A.64. Applied load versus in-plane displacement of columns at 180 mm above the base plate
Figure A.65. Applied load versus local displacement of outside flanges of the south column

Figure A.66. Applied load versus local displacement of inside flanges of the south column
Figure A.67. Applied load versus local displacement of outside flanges of the north column

Figure A.68. Applied load versus local displacement of inside flanges of the north column
Figure A.69. Applied load versus local displacement of top flanges of the south rafter

Figure A.70. Applied load versus local displacement of bottom flanges of the south rafter
Figure A.71. Applied load versus local displacement of top flanges of the north rafter

Figure A.72. Applied load versus local displacement of bottom flanges of the north rafter
Appendix A

Figure A.73. Applied load versus out-of-plane displacement of the south knee brace measured at the KBC bracket

Figure A.74. Applied load versus out-of-plane displacement of the south knee brace measured at the mid-length
Figure A.75. Applied load versus out-of-plane displacement of the south knee brace measured at the KBR bracket.

Table A.17. Summary of experiments conducted in this study

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## APPENDIX B. ANCILLARY TEST RESULTS

### B.1. COUPON TEST DATA AND RESULTS

#### Table B.1. Dimensions of aluminium bracket coupons

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<th>Width (mm)</th>
<th>Area (mm²)</th>
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<td>ApexB_4</td>
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<td>0.032</td>
<td>0.020</td>
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<td>0.046</td>
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<td>Bracket (Mean)</td>
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<td>COV</td>
<td>-</td>
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<td>0.040</td>
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</table>
Figure B.1. Stress-strain curves of column and rafter section coupons from the longitudinal direction
Figure B.2. Stress-strain curves of column and rafter section coupons from the transverse direction

Figure B.3. Stress-strain curves of column and rafter section coupons from the diagonal direction
Figure B.4. Stress-strain curves of coupons from aluminium brackets
B.2. POINT FASTENER TEST DATA AND RESULTS

B.2.1. Measured dimensions of single-lap-joint slip-resistant M16 shear connections

![Diagram of single-lap-joint slip-resistant M16 shear connection specimen]

**Figure B.5. Single-lap-joint slip-resistant M16 shear connection specimen**

**Table B.4. Measured dimensions of single-lap-joint M16 shear connection specimens**

<table>
<thead>
<tr>
<th>Specimens</th>
<th>$w_b$ (mm)</th>
<th>$w_t$ (mm)</th>
<th>$t_b$ (mm)</th>
<th>$t_t$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SBF-1</td>
<td>79.81</td>
<td>80.11</td>
<td>5.82</td>
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<td>SBF-2</td>
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<td>79.94</td>
<td>5.81</td>
<td>2.42</td>
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<tr>
<td>SBF-3</td>
<td>79.81</td>
<td>80.32</td>
<td>5.81</td>
<td>2.43</td>
</tr>
</tbody>
</table>

Note:  
- $w_b$ is the width of the bottom plate  
- $w_t$ is the width of the top plate  
- $t_b$ is the thickness of the bottom plate  
- $t_t$ is the thickness of the top plate
B.2.2. Measured dimensions of double-lap-joint slip-resistant M16 shear connections

![Diagram of double-lap-joint slip-resistant M16 shear connection specimen]

**Figure B.6.** Double-lap-joint slip-resistant M16 shear connection specimen

**Table B.5.** Measured dimensions of double-lap-joint M16 shear connection specimens

<table>
<thead>
<tr>
<th>Specimens</th>
<th>$w_{bA}$ (mm)</th>
<th>$w_{bB}$ (mm)</th>
<th>$w_{tA}$ (mm)</th>
<th>$w_{tB}$ (mm)</th>
<th>$t_{bA}$ (mm)</th>
<th>$t_{bB}$ (mm)</th>
<th>$t_{tA}$ (mm)</th>
<th>$t_{tB}$ (mm)</th>
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</thead>
<tbody>
<tr>
<td>DBF-1</td>
<td>79.84</td>
<td>79.99</td>
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<td>80.56</td>
<td>5.83</td>
<td>5.86</td>
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<td>DBF-2</td>
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<td>5.84</td>
<td>5.79</td>
<td>2.42</td>
<td>2.43</td>
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</table>

Note: $w_{bA}$ is the width of the bottom plate A (Figure B.6)

$w_{bB}$ is the width of the bottom plate B (Figure B.6)

$w_{tA}$ is the width of the top plate A (Figure B.6)

$w_{tB}$ is the width of the top plate B (Figure B.6)

$t_{bA}$ is the thickness of the bottom plate A (Figure B.6)

$t_{bB}$ is the thickness of the bottom plate B (Figure B.6)

$t_{tA}$ is the thickness of the top plate A (Figure B.6)

$t_{tB}$ is the thickness of the top plate B (Figure B.6)
B.2.3. Measured dimensions of snug-tightened M12 shear connections

Figure B.7. Snug-tightened M12 shear connection specimen

Table B.6. Measured dimensions of snug-tightened M12 shear connection specimens

<table>
<thead>
<tr>
<th>Specimens</th>
<th>$w_b$ (mm)</th>
<th>$w_t$ (mm)</th>
<th>$t_b$ (mm)</th>
<th>$t_t$ (mm)</th>
</tr>
</thead>
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<td>79.87</td>
<td>5.83</td>
<td>2.44</td>
</tr>
<tr>
<td>M12-3</td>
<td>79.95</td>
<td>80.18</td>
<td>5.81</td>
<td>2.44</td>
</tr>
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</table>

Note: $w_b$ is the width of the bottom plate

$w_t$ is the width of the top plate

$t_b$ is the thickness of the bottom plate

$t_t$ is the thickness of the top plate
B.2.4. Failure of the shear connections

Figure B.8. Typical failure of slip-resistant M16 shear connection samples: (a) Single-lap-joint; (b) Double-lap-joint

B.2.5. Slip-resistant M16 moment connections

B.2.5.1. Measured initial dimensions of slip-resistant M16 moment connections

Figure B.9. Dimensions of the M16 moment connection

Table B.7. Measured initial dimensions of M16 moment connection specimens

<table>
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<th></th>
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<th></th>
<th></th>
<th>Double lap-joint</th>
<th></th>
<th></th>
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</thead>
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<td>$l_{bo}$ (mm)</td>
<td>$l_{lo}$ (mm)</td>
<td>$l_o$ (mm)</td>
<td>Test specimens</td>
<td>$l_{bo}$ (mm)</td>
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<td>176.8</td>
<td>254.0</td>
<td>MDF-1</td>
<td>181.5</td>
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<td>MDF-2</td>
<td>181.8</td>
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<td>178.0</td>
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B.2.5.2. Calculation of torque at the centre of the bolt

The torque at the centre of the bolt and corresponding bolt rotation at each data point can be determined based on the initial dimensions and the reading data. As seen in Figure B.10, the moment at the centre of the bolt at each data point can be computed as follows:

\[ M_j = F_j h_j \]  \hspace{1cm} (B.1)

where \( F_j \) is the applied load at each data point, \( h_j \) is the moment arm at each data point.

Using the cosine law, the initial angles and moment arm can be obtained as:

\[ \theta_{Ao} = \cos^{-1} \left( \frac{l^2_{bo} + l^2_o - l^2_{ho}}{2l_{bo}l_o} \right) \]  \hspace{1cm} (B.2)

\[ \theta_{Bo} = \cos^{-1} \left( \frac{l^2_{bo} + l^2_o - l^2_{ho}}{2l_{bo}l_o} \right) \]  \hspace{1cm} (B.3)

\[ \theta_{Co} = \cos^{-1} \left( \frac{l^2_{bo} + l^2_o - l^2_{ho}}{2l_{bo}l_o} \right) \]  \hspace{1cm} (B.4)
Based on the measured inclination of the two plates, the moment arm, $h_j$, at each recording can be determined as:

$$h_j = l_{to} \sin \left( \theta_{to} - \Delta \theta_{Cj} \right)$$

or

$$h_j = l_{bo} \sin \left( \theta_{bo} - \Delta \theta_{Bj} \right)$$

Where $\Delta \theta_{Bj}$ and $\Delta \theta_{Cj}$ are the angles recorded by the two inclinometers.

Figure B.11. Applied load versus vertical displacement of the lower end of M16 bolted connection subject to torque: (a) Single-lap-joint; (b) Double-lap-joint
B.3. ADDITIONAL BASE CONNECTION TEST RESULTS

B.3.1. Major axis bending tests

Figure B.12. Load versus horizontal displacement above the base: (a) specimen BC_Lplate_8_MA-1; (b) specimen BC_Lplate_8_MA-2

Figure B.13. Load versus horizontal displacement above the base: (a) specimen BC_Lplate_6_MA-1; (b) specimen BC_Lplate_6_MA-2
Figure B.14. Load versus horizontal displacement above the base: (a) specimen BC_Lplate_3_MA-1; (b) specimen BC_Lplate_3_MA-2

Figure B.15. Load versus horizontal displacement above the base: (a) specimen BC_Uplate_3_MA-1; (b) specimen BC_Uplate_3_MA-2
Figure B.16. Load versus lateral displacement of the columns: (a) at 480 mm above the base plate; (b) at loading point

Figure B.17. Load versus twist rotation of the columns at loading point
B.3.2. Minor axis bending tests

Figure B.18. Load versus horizontal displacement above the base: (a) specimen BC_Lplate_8_MI-1; (b) specimen BC_Lplate_8_MI-2

Figure B.19. Load versus horizontal displacement above the base: (a) specimen BC_Lplate_6_MI-1; (b) specimen BC_Lplate_6_MI-2
Figure B.20. Load versus horizontal displacement above the base: (a) specimen BC_Lplate_3_MI-1; (b) specimen BC_Lplate_3_MI-2

Figure B.21. Load versus horizontal displacement above the base: (a) specimen BC_Uplate_3_MI-1; (b) specimen BC_Uplate_3_MI-2
Figure B.22. Load versus lateral displacement of the columns: (a) at 480 mm above the base plate; (b) at loading point

Figure B.23. Load versus twist rotation of the columns at loading point
APPENDIX C. FINITE ELEMENT MODELLING AND RESULTS

C.1. INCORPORATION OF MEASURED GEOMETRIC IMPERFECTIONS INTO FINITE ELEMENT MODELS

C.1.1. The interpolation algorithm used to determine the sectional and member nodal imperfections

The double lipped channel cross-sections of the columns and rafters were divided into 18 individual regions which are defined by edges labelled from E1 to E20 as shown in Figure C.1. The nodal coordinates are then manipulated based on the node position within the region it lies on and the relevant imperfection values attributed to its longitudinal location.

![Figure C.1. Sectional and member imperfections: (a) Location of imperfection measurements; (b) divided regions](image)

C.1.1.1. Determine the imperfections of the edges and reference points

The nodal imperfections are calculated as follows:

1. The top – left flange (defining by edges 3 and 4)

![Figure C.2. Nodal imperfections of the top-left flange](image)
Appendix C

\[ tg \theta_{f_1} \approx \frac{IMP(2) - IMP(4)}{x_4 - x_2} \]  \hspace{1cm} (C.1)

\[ Icr\_E3 \approx IMP(4) + (x_4 - x_3)tg \theta_{f_1} \]  \hspace{1cm} (C.2)

\[ Icr\_E4 \approx IMP(4) - (x_4 - x_3)tg \theta_{f_1} \]  \hspace{1cm} (C.3)

\[ IMP\_Lc3 \approx IMP(3) - (IMP(4) + (x_4 - x_3)tg \theta_{f_1}) \]  \hspace{1cm} (C.4)

2. The bottom – left flange (defining by edges 7 and 8)

\[ tg \theta_{f_2} \approx \frac{IMP(10) - IMP(8)}{x_8 - x_{10}} \]  \hspace{1cm} (C.5)

\[ Icr\_E7 \approx IMP(8) - (x_7 - x_8)tg \theta_{f_2} \]  \hspace{1cm} (C.6)

\[ Icr\_E8 \approx IMP(8) + (x_8 - x_{e8})tg \theta_{f_2} \]  \hspace{1cm} (C.7)

\[ IMP\_Lc9 \approx IMP(9) - (IMP(8) + (x_8 - x_{e8})tg \theta_{f_2}) \]  \hspace{1cm} (C.8)

3. The top – right flange (defining by edges 13 and 14)

\[ tg \theta_{f_3} \approx \frac{IMP(13) - IMP(15)}{x_{13} - x_{15}} \]  \hspace{1cm} (C.9)
Appendix C

\[ Icr\_E13 \approx IMP(15) + (x_{e13} - x_{i3})tg\theta_{f3} \]  \hspace{1cm} (C.10)

\[ Icr\_E14 \approx IMP(15) - (x_{i3} - x_{e14})tg\theta_{f3} \]  \hspace{1cm} (C.11)

\[ IMP\_Lc14 \approx IMP(14) - \left( IMP(15) + (x_{i4} - x_{i5})tg\theta_{f3} \right) \]  \hspace{1cm} (C.12)

4. The bottom – right flange (defining by edges 17 and 18)

\[ tg\theta_{f4} \approx \frac{IMP(21) - IMP(19)}{x_{21} - x_{19}} \]  \hspace{1cm} (C.13)

\[ Icr\_E17 \approx IMP(19) - (x_{i9} - x_{e17})tg\theta_{f4} \]  \hspace{1cm} (C.14)

\[ Icr\_E18 \approx IMP(19) + (x_{e18} - x_{i9})tg\theta_{f4} \]  \hspace{1cm} (C.15)

\[ IMP\_Lc20 \approx IMP(20) - \left( IMP(19) + (x_{i20} - x_{i19})tg\theta_{f4} \right) \]  \hspace{1cm} (C.16)

5. The left web (defining by edges 5 and 6)

\[ Icr\_E5 \approx IMP(5) - \left( IMP(6) + (x_{i5} - x_{i6})tg\theta_{w1} \right) \]  \hspace{1cm} (C.17)

Figure C.5. Nodal imperfections of the bottom-right flange

Figure C.6. Nodal imperfections of the left web
Appendix C

\[ tg \theta_{w1} \approx \frac{IMP(7) - IMP(15)}{y_5 - y_7} \]  \hspace{1cm} (C.17)

\[ Icr\_E5 \approx IMP(5) - (y_{e5} - y_5)tg \theta_{w1} \]  \hspace{1cm} (C.18)

\[ Icr\_E6 \approx IMP(7) + (y_7 - y_{e6})tg \theta_{w1} \]  \hspace{1cm} (C.19)

\[ IMP\_Lc6 \approx IMP(6) - (IMP(5) + (y_5 - y_{e6})tg \theta_{w1}) \]  \hspace{1cm} (C.20)

6. The right web (defining by edges 15 and 16)

7. The top – left lip and the top – left lip-flange corner (defining by edges 1 and 2, and edges 2 and 3, respectively)

\[ tg \theta_{w2} \approx \frac{IMP(18) - IMP(16)}{y_{e16} - y_{18}} \]  \hspace{1cm} (C.21)

\[ Icr\_E15 \approx IMP(16) - (y_{e16} - y_{16})tg \theta_{w2} \]  \hspace{1cm} (C.22)

\[ Icr\_E16 \approx IMP(18) + (y_{18} - y_{e16})tg \theta_{w2} \]  \hspace{1cm} (C.23)

\[ IMP\_Lc17 \approx IMP(17) - (IMP(16) + (y_{16} - y_{e17})tg \theta_{w2}) \]  \hspace{1cm} (C.24)
Figure C.8. Nodal imperfections of the top-left lip and the top—left lip-flange corner

\[
\tan \theta_{L,1} \approx \frac{\text{IMP}(1)}{y_{x,2} - y_{1}}
\]  \hspace{1cm} (C.25)

8. The bottom—left lip and the bottom—left lip-flange corner (defining by edges 9 and 10, and edges 8 and 9, respectively)

Figure C.9. Nodal imperfections of the bottom-left lip and the bottom—left lip-flange corner

\[
\tan \theta_{L,2} \approx \frac{\text{IMP}(11)}{y_{11} - y_{9}}
\]  \hspace{1cm} (C.26)

9. The top—right lip and the top—right lip-flange corner (defining by edges 11 and 12, and edges 12 and 13, respectively)

Figure C.10. Nodal imperfections of the top-right lip and the top—right lip-flange corner
Appendix C

\[ \tan \theta_{L3} \approx \frac{IMP(12)}{y_{e12} - y_{12}} \]  
(C.27)

10. The bottom – right lip and the bottom – right lip-flange corner (defining by edges 19 and 20, and edges 18 and 19, respectively)

![Diagram of nodal imperfections](image)

Figure C.11. Nodal imperfections of the bottom-right lip and the bottom-right lip-flange corner

\[ \tan \theta_{L4} \approx \frac{IMP(22)}{y_{22} - y_{e19}} \]  
(C.28)

**C.1.1.2. Determine the nodal imperfections and new coordinates of imperfect nodes**

1. The region 1 - the top – left lip (defining by edges 1 and 2)

For the nodes lying on region 1, the imperfect coordinates are calculated relative to the coordinates of the edge 4 (Figure C.8), as follows:

\[ x' = \Delta x \cos \theta_{f1} - \Delta y \sin \theta_{f1} - (y_{e2} - y_i) \tan \theta_{L1} \]  
(C.29)

\[ y' = \Delta x \sin \theta_{f1} + \Delta y \cos \theta_{f1} \]  
(C.30)

\[ x_{i, new} = x_{e4} - x' + Icr \_E5 \]  
(C.31)

\[ y_{i, new} = y_{e4} - y' - Icr \_E4 \]  
(C.32)

where

\[ \Delta x = x_{e4} - x_i \]  
(C.33)

\[ \Delta y = y_{e4} - y_i \]  
(C.34)

2. The region 2 - the top – left lip-flange corner (defining by edges 2 and 3)

The new coordinates of the nodes lying on region 2 are:

\[ x' = \Delta x \cos \theta_{f1} - \Delta y \sin \theta_{f1} \]  
(C.35)
\[ y' = \Delta x \sin \theta_{f1} + \Delta y \cos \theta_{f1} \]  
(C.36)

\[ x_{i\_new} = x_{e4} - x' + Icr\_E5 \]  
(C.37)

\[ y_{i\_new} = y_{e4} - y' - Icr\_E4 \]  
(C.38)

where

\[ \Delta x = x_{e4} - x_i \]  
(C.39)

\[ \Delta y = y_{e4} - y_i \]  
(C.40)

3. The region 3 - the top – left flange (defining by edges 3 and 4)

For the nodes belonging to region 3, as shown in Figure C.2, the updated coordinates are determined as follows:

\[ Icr\_x_i = Icr\_E5 \]  
(C.41)

\[ Icr\_Lci = Lagrange\ interpolation([x_{e3} \ x_3 \ x_{e4}],[0 \ IMP\_Lc3 \ 0],x_i) \]  
(C.42)

\[ Icr\_y_i = Icr\_E4 + (x_{e4} - x_i)tg\theta_{f1} + Icr\_Lci \]  
(C.43)

\[ x_{i\_new} = x_i + Icr\_x_i \]  
(C.44)

\[ y_{i\_new} = y_i - Icr\_y_i \]  
(C.45)

4. The region 4 - the top – left flange - web corner (defining by edges 4 and 5)

The new coordinates of the nodes lying on region 4 are:

\[ x_{i\_new} = x_i + Icr\_E5 \]  
(C.46)

\[ y_{i\_new} = y_i - Icr\_E4 \]  
(C.47)

5. The region 5 - the left web (defining by edges 5 and 6)

For the nodes lying on region 5, as shown in Figure C.6, the updated coordinates are determined as follows:

\[ Icr\_Lci = Lagrange\ interpolation([y_{e5} \ y_6 \ y_{e6}],[0 \ IMP\_Lc6 \ 0],y_i) \]  
(C.48)

\[ Icr\_x_i = Icr\_E5 + (y_{e5} - y_i)tg\theta_{el} + Icr\_Lci \]  
(C.49)

\[ Icr\_y_i = Icr\_E4 \]  
(C.50)
Appendix C

\[ x_{i\_\text{new}} = x_i - Icr\_x_i \]  \hfill (C.51)

\[ y_{i\_\text{new}} = y_i - Icr\_y_i \]  \hfill (C.52)

6. The region 6 - the bottom – left flange - web corner (defining by edges 6 and 7)

The new coordinates of the nodes lying on region 6 are:

\[ x_{i\_\text{new}} = x_i + Icr\_E6 \]  \hfill (C.53)

\[ y_{i\_\text{new}} = y_i + Icr\_E7 \]  \hfill (C.54)

7. The region 7 - the bottom – left flange (defining by edges 7 and 8)

For the nodes belonging to region 7, as shown in Figure C.3, the updated coordinates are calculated as follows:

\[ Icr\_x_i = Icr\_E6 \]  \hfill (C.55)

\[ Icr\_Lci = \text{Lagrange interpolation} \left( [x_{e7} \quad x_9 \quad x_{e8}] ; [0 \quad IMP\_Lc9 \quad 0] , x_i \right) \]  \hfill (C.56)

\[ Icr\_y_i = Icr\_E7 + (x_{e7} - x_i)\tan(\theta_{f2}) + Icr\_Lci \]  \hfill (C.57)

\[ x_{i\_\text{new}} = x_i + Icr\_x_i \]  \hfill (C.58)

\[ y_{i\_\text{new}} = y_i + Icr\_y_i \]  \hfill (C.59)

8. The region 8 - the bottom – left lip-flange corner (defining by edges 8 and 9)

The new coordinates of the nodes lying on region 8 are:

\[ x' = \Delta x \cos \theta_{f2} - \Delta y \sin \theta_{f2} \]  \hfill (C.60)

\[ y' = \Delta x \sin \theta_{f2} + \Delta y \cos \theta_{f2} \]  \hfill (C.61)

\[ x_{i\_\text{new}} = x_{e7} - x' + Icr\_E6 \]  \hfill (C.62)

\[ y_{i\_\text{new}} = y_{e7} + y' + Icr\_E7 \]  \hfill (C.63)

where

\[ \Delta x = x_{e7} - x_i \]  \hfill (C.64)

\[ \Delta y = y_i - y_{e7} \]  \hfill (C.65)

9. The region 9 - the bottom – left lip (defining by edges 9 and 10)
For the nodes lying on region 9, the imperfect coordinates are calculated relative to the coordinates of the edge 7 (Figure C.9), as follows:

\[
x' = \Delta x \cos \theta_{j_2} - \Delta y \sin \theta_{j_2} - (y_i - y_{e_7}) \tan \theta_{j_2}
\]
(C.66)

\[
y' = \Delta x \sin \theta_{j_2} + \Delta y \cos \theta_{j_2}
\]
(C.67)

\[
x_{i,\text{new}} = x_{e_7} - x' + Icr_{-} E6
\]
(C.68)

\[
y_{i,\text{new}} = y_{e_7} + y' + Icr_{-} E7
\]
(C.69)

where

\[
\Delta x = x_{e_i} - x_i
\]
(C.70)

\[
\Delta y = y_i - y_{e_7}
\]
(C.71)

10. The region 10 - the top – right lip (defining by edges 11 and 12)

For the nodes lying on region 10, the imperfect coordinates are calculated relative to the coordinates of the edge 14 (Figure C.10), as follows:

\[
x' = \Delta x \cos \theta_{j_3} - \Delta y \sin \theta_{j_3} - (y_{e_12} - y_i) \tan \theta_{j_3}
\]
(C.72)

\[
y' = \Delta x \sin \theta_{j_3} + \Delta y \cos \theta_{j_3}
\]
(C.73)

\[
x_{i,\text{new}} = x_{e_14} + x' - Icr_{-} E15
\]
(C.74)

\[
y_{i,\text{new}} = y_{e_14} - y' - Icr_{-} E14
\]
(C.75)

where

\[
\Delta x = x_i - x_{e_14}
\]
(C.76)

\[
\Delta y = y_{e_14} - y_i
\]
(C.77)

11. The region 11 - the top – right lip-flange corner (defining by edges 12 and 13)

The new coordinates of the nodes lying on region 2 are:

\[
x' = \Delta x \cos \theta_{j_3} - \Delta y \sin \theta_{j_3}
\]
(C.78)

\[
y' = \Delta x \sin \theta_{j_3} + \Delta y \cos \theta_{j_3}
\]
(C.79)

\[
x_{i,\text{new}} = x_{e_14} + x' - Icr_{-} E15
\]
(C.80)

\[
y_{i,\text{new}} = y_{e_14} - y' - Icr_{-} E14
\]
(C.81)
where
\[
\Delta x = x_i - x_{e14}
\] (C.82)
\[
\Delta y = y_{e14} - y_i
\] (C.83)

12. The region 12 - the top – right flange (defining by edges 13 and 14)

For the nodes belonging to region 12, as shown in Figure C.4, the updated coordinates are determined as follows:

\[
Icr\_x_i = Icr\_E15
\] (C.84)

\[
Icr\_Lci = \text{Lagrange interpolation}([x_{e13} \ x_{e14} \ x_{e15}], [0 \ IMP\_Lc14 \ 0], x_i)
\] (C.85)

\[
Icr\_y_i = Icr\_E14 + (x_i - x_{e14})tg\theta + Icr\_Lci
\] (C.86)

\[
x_{i,\text{new}} = x_i - Icr\_x_i
\] (C.87)

\[
y_{i,\text{new}} = y_i - Icr\_y_i
\] (C.88)

13. The region 13 - the top – right flange - web corner (defining by edges 14 and 15)

The new coordinates of the nodes lying on region 13 are:

\[
x_{i,\text{new}} = x_i - Icr\_E15
\] (C.89)

\[
y_{i,\text{new}} = y_i - Icr\_E14
\] (C.90)

14. The region 14 - the right web (defining by edges 15 and 16)

For the nodes lying on region 14, as shown in Figure C.7, the updated coordinates are determined as follows:

\[
Icr\_Lci = \text{Lagrange interpolation}([y_{e15} \ y_{e16}], [0 \ IMP\_Lc17 \ 0], y_i)
\] (C.91)

\[
Icr\_x_i = Icr\_E15 + (y_{e15} - y_i)tg\theta + Icr\_Lci
\] (C.92)

\[
Icr\_y_i = Icr\_E14
\] (C.93)

\[
x_{i,\text{new}} = x_i - Icr\_x_i
\] (C.94)

\[
y_{i,\text{new}} = y_i - Icr\_y_i
\] (C.95)

15. The region 15 - the bottom – right flange - web corner (defining by edges 16 and 17)
The new coordinates of the nodes lying on region 15 are:

\[ x_{i,\text{new}} = x_i - Icr \_E16 \]  \hfill (C.96)

\[ y_{i,\text{new}} = y_i + Icr \_E17 \]  \hfill (C.97)

16. The region 16 - the bottom – right flange (defining by edges 17 and 18)

For the nodes belonging to region 16 as shown in Figure C.5, the updated coordinates are calculated as follows:

\[ Icr \_x_i = Icr \_E16 \]  \hfill (C.98)

\[ Icr \_Lci = \text{Lagrange interpolation}([x_{e17}, x_{e18}, 0 \text{ IMP } Lc20, 0], x_i) \]  \hfill (C.99)

\[ Icr \_y_i = Icr \_E17 + (x_i - x_{e17})tg\theta_{f4} + Icr \_Lci \]  \hfill (C.100)

\[ x_{i,\text{new}} = x_i - Icr \_x_i \]  \hfill (C.101)

\[ y_{i,\text{new}} = y_i + Icr \_y_i \]  \hfill (C.102)

17. The region 17 - the bottom – right lip-flange corner (defining by edges 18 and 19)

The new coordinates of the nodes lying on region 17 are:

\[ x' = \Delta x \cos \theta_{f4} - \Delta y \sin \theta_{f4} \]  \hfill (C.103)

\[ y' = \Delta x \sin \theta_{f4} + \Delta y \cos \theta_{f4} \]  \hfill (C.104)

\[ x_{i,\text{new}} = x_{e17} + x' - Icr \_E16 \]  \hfill (C.105)

\[ y_{i,\text{new}} = y_{e17} + y' + Icr \_E17 \]  \hfill (C.106)

where

\[ \Delta x = x_i - x_{e17} \]  \hfill (C.107)

\[ \Delta y = y_i - y_{e17} \]  \hfill (C.108)

18. The region 18 - the bottom – right lip (defining by edges 19 and 20)

For the nodes lying on region 18, the imperfect coordinates are calculated relative to the coordinates of the edge 17 (Figure C.11), as follows:

\[ x' = \Delta x \cos \theta_{f4} - \Delta y \sin \theta_{f4} - (y_i - y_{e19})tg\theta_{L4} \]  \hfill (C.109)

\[ y' = \Delta x \sin \theta_{f4} + \Delta y \cos \theta_{f4} \]  \hfill (C.110)
$$x_{i,\text{new}} = x_{el7} + x' - lcr \cdot E_{16} \quad (C.111)$$

$$y_{i,\text{new}} = y_{el7} + y' + lcr \cdot E_{17} \quad (C.112)$$

where

$$\Delta x = x_i - x_{el7} \quad (C.113)$$

$$\Delta y = y_i - y_{el7} \quad (C.114)$$

### Table C.1. The coordinates of measurement points

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<th>y (mm)</th>
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### Table C.2. The coordinates of edge points

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### C.1.2. Matlab code for the incorporation of sectional and member imperfections

```matlab
% HOAI CUONG NGUYEN
% This program uses the member perfect geometry to impose member and sectional imperfections For Cold-rolled Aluminium PFrame
clc, clearvars
% Create a n number of files
Nfra=7; % Number of Frame Tests
Ncol=2; % Number of columns
Nraf=2; % Number of rafters
n_lines=22;
```
n=35;
L_col=5232; % The length of Column
L_Raf=7182; % The length of Rafter

% --> Member and sectional imperfections parameters <--

for i = 1:Nfra
    for j=1:Ncol
        % open the reading and writing files of columns
        filename1=sprintf('Original-Column%d%d.inp',j,i);
        fin=fopen(filename1,'r'); % open the reading file
        filename2=sprintf('Updated-Column%d_%d.inp',j,i);
        fout=fopen(filename2,'w'); % open the writing file
        K=xlsread(['fourier_coefficientsC' num2str(j) num2str(i) '.xlsx'],1); % Fourier Coefficients for column
        a=xlsread(['Imperfection_magnitude_endA_C' num2str(j) num2str(i) '.xlsx'],1); % True imperfection at end A for column
        b=xlsread(['Imperfection_magnitude_endB_C' num2str(j) num2str(i) '.xlsx'],1); % True imperfection at end B for column
        
        % -- Member and sectional imperfections parameters of the columns <--
        
        while ~feof(fin) % while doesn't get to the end of the file
            t=str2num(fgetl(fin)); % store line values in vector t
            t contains node perfect coordinates {ID X Y Z}
            L=L_col; % Height of the column
            Imp=K*Vsine(n,L,t(4))+a+(b-a)/L*t(4); % Section dimensions of the members for sectional imperfection purposes
            hc= 252.5; % Column height
            bc= 161.5; % Column width
            rc= 6.25; % Column fillet radius
            tc= 2.5; % Column thickness
            yc= 0; % "coordinate" y of centroid
            gapc=14.5; % gap between channels
            Sc=[0 0]; % Coordinates of the shear centre
            
            % Original Coordinates of Edges and Measuring points
            Coord_Edges=xlsread('Original Coordinates.xlsx',1,'F3:G20'); % Coordinates of the edges
            Coord_Lasers=xlsread('Original Coordinates.xlsx',1,'B3:C24'); % Coordinates of the measuring points
            x = Coord_Lasers(:,1); % Coordinate x of the measuring points
            y = Coord_Lasers(:,2); % Coordinate y of the measuring points
            xe = Coord_Edges(:,1); % Coordinate x of the edges
            ye = Coord_Edges(:,2); % Coordinate y of the edges
            
            % Determine the rotation angles of flanges and lips, local imperfections of flanges and webs and the increment of edges
            % Left - top flange, conner and lip
            tan_thetaF1 = (Imp(2)-Imp(4))/(x(4) - xe(3));
            tan_thetaL1 = Imp(1)/(ye(2) - y(1));
            incr_e3 = Imp(4)+(x(4)- xe(3))*tan_thetaF1;
            incr_e4 = Imp(4)-(xe(4)- x(4))*tan_thetaF1;
            Imp_lc3 = Imp(3)- (Imp(4)+ (x(4)-x(3))*tan_thetaF1); % Left - bottom flange, conner and lip
            tan_thetaF2 = (Imp(10)-Imp(8))/(x(8) - x(10));
            tan_thetaL2 = Imp(11)/(y(11) - ye(9));
            incr_e7 = Imp(8)-(xe(7)- x(8))*tan_thetaF2;
            incr_e8 = Imp(8)+(x(8)- xe(8))*tan_thetaF2;
            Imp_lc9 = Imp(9)- (Imp(8)+ (x(8)-x(9))*tan_thetaF2); % Right - top flange, conner and lip
            tan_thetaF3 = (Imp(13)-Imp(15))/(x(13) - x(15));
\[
\tan_{\theta_{L3}} = \frac{\text{Imp}(12)}{\text{ye}(12) - y(12)};
\]
\[
\text{incr}_{e13} = \text{Imp}(15) + (x(13) - x(15)) \tan_{\theta_{F3}};
\]
\[
\text{Imp}_{lc14} = \text{Imp}(14) - (\text{Imp}(15) + (x(14) - x(15)) \tan_{\theta_{F3}});
\]
% Right - bottom flange, conner and lip
\[
\tan_{\theta_{F4}} = \frac{\text{Imp}(21) - \text{Imp}(19)}{x(21) - x(19)};
\]
\[
\text{incr}_{e17} = \text{Imp}(19) - (x(19) - xe(17)) \tan_{\theta_{F3}};
\]
\[
\text{incr}_{e18} = \text{Imp}(19) + (x(18) - x(19)) \tan_{\theta_{F4}};
\]
\[
\text{Imp}_{lc20} = \text{Imp}(20) - (\text{Imp}(19) + (x(20) - x(19)) \tan_{\theta_{F4}});
\]
% Left web
\[
\tan_{\theta_{W1}} = \frac{\text{Imp}(7) - \text{Imp}(5)}{y(5) - y(7)};
\]
\[
\text{incr}_{e5} = \text{Imp}(5) - (y(5) - ye(5)) \tan_{\theta_{W1}};
\]
\[
\text{incr}_{e6} = \text{Imp}(7) + (y(7) - ye(6)) \tan_{\theta_{W1}};
\]
\[
\text{Imp}_{lc6} = \text{Imp}(6) - (\text{Imp}(5) + (y(5) - y(6)) \tan_{\theta_{W1}});
\]
% Right web
\[
\tan_{\theta_{W2}} = \frac{\text{Imp}(18) - \text{Imp}(16)}{y(16) - y(18)};
\]
\[
\text{incr}_{e15} = \text{Imp}(16) - (y(16) - ye(16)) \tan_{\theta_{W2}};
\]
\[
\text{incr}_{e16} = \text{Imp}(18) + (y(18) - ye(16)) \tan_{\theta_{W2}};
\]
\[
\text{Imp}_{lc17} = \text{Imp}(17) - (\text{Imp}(16) + (y(16) - y(17)) \tan_{\theta_{W2}});
\]

% Imperfection modes start here
if (t(2)<(-gapc/2-rc) && t(2)>(-bc/2+rc) && t(3)>(hc/2-rc)) % Left flange + Top % Check limits
Increment_x = incr_e5;
Increment_y = lagrange_interpolation([xe(3) x(3) xe(4)], [0 Imp_{lc3} 0], t(2));
t(2) = t(2) + Increment_x; % New coordinate X
% New coordinate Y

elseif (t(2)<(-gapc/2-rc) && t(2)>(-bc/2+rc) && t(3)<(-hc/2+rc)) % Left flange + Bottom % Check limits (edit)
Increment_x = incr_e6;
Increment_y = lagrange_interpolation([xe(7) x(9) xe(8)], [0 Imp_{lc9} 0], t(2));
t(2) = t(2) + Increment_x; % New coordinate X
% New coordinate Y

elseif (t(2)>(gapc/2+rc) && t(2)<(bc/2-rc) && t(3)>(hc/2-rc)) % Right flange + Top % Check limits
Increment_x = incr_e15;
Increment_y = lagrange_interpolation([xe(13) x(14) xe(14)], [0 Imp_{lc14} 0], t(2));
t(2) = t(2) - Increment_x; % New coordinate X
% New coordinate Y

elseif (t(2)>(gapc/2+rc) && t(2)<(bc/2-rc) && t(3)<(-hc/2+rc)) % Right flange + Bottom % Check limits
Increment_x = incr_e16;
Increment_y = lagrange_interpolation([xe(17) x(20) xe(18)], [0 Imp_{lc20} 0], t(2));
t(2) = t(2) - Increment_x; % New coordinate X
% New coordinate Y

elseif (t(2)< 0 && t(2)>(-gapc/2-rc) && t(3)<(hc/2-rc) && t(3)>(-hc/2+rc)) % Left Web
Increment_x = incr_e5 + (ye(5) - y(6)) \tan_{\theta_{W1}} + incr_lci;
Increment_y = incr_e4;
t(2) = t(2) + Increment_x; \% New coordinate X
\t(3) = t(3) - Increment_y; \% New coordinate Y

\textbf{elseif} (t(2) < \text{gapc/2+rc} \text{ } \&\text{ } t(2) > 0 \text{ } \&\text{ } t(3) < \text{hc/2-rc} \text{ } \&\text{ } t(3) > (-hc/2+rc)) \% Right Web
\begin{align*}
&\text{incr\_lc1} = \text{lagrange\_interpolation([ye(15) \text{ } y(17) \text{ } ye(16)]),[0 \text{ } Imp\_lc17 \text{ } 0],t(3));} \\
&\text{Increment\_x} = \text{incr\_el5} + (\text{ye(15) - t(3)})*\text{tan\_thetaW2} + \text{incr\_lc1}; \\
&t(2) = t(2) - \text{Increment\_x}; \% \text{New coordinate X} \\
&t(3) = t(3) - \text{Increment\_y}; \% \text{New coordinate Y}
\end{align*}

\textbf{elseif} (t(2) < (-bc/2+rc) \text{ } \&\text{ } t(3) < (hc/2-rc) \text{ } \&\text{ } t(3) > 0) \% Left lip + Top
\begin{align*}
&\text{delta\_x} = \text{xe(4) - t(2)}; \\
&\text{delta\_y} = \text{ye(4) - t(3)}; \\
&\text{thetaF1} = \text{atan}(\text{tan\_thetaF1}); \\
&\text{xdash} = \text{delta\_x*\cos(thetaF1) - delta\_y*\sin(thetaF1) - (ye(2) - t(3))*\tan\_thetaL1}; \\
&\text{ydash} = -\text{delta\_x*\sin(thetaF1) + delta\_y*\cos(thetaF1)}; \\
&t(2) = (\text{xe(4) - xdash}) + \text{incr\_e5}; \% \text{New coordinate X} \\
&t(3) = (\text{ye(4) - ydash}) - \text{incr\_e4}; \% \text{New coordinate Y}
\end{align*}

\textbf{elseif} (t(2) < (-bc/2+rc) \text{ } \&\text{ } t(3) < 0 \text{ } \&\text{ } t(3) > (-hc/2+rc)) \% Left lip + Bottom
\begin{align*}
&\text{delta\_x} = \text{xe(4) - t(2)}; \\
&\text{delta\_y} = \text{ye(4) - t(3)}; \\
&\text{thetaF2} = \text{atan}(\text{tan\_thetaF2}); \\
&\text{xdash} = \text{delta\_x*\cos(thetaF2) - delta\_y*\sin(thetaF2) - (t(3) - ye(9))*\tan\_thetaL2}; \\
&\text{ydash} = -\text{delta\_x*\sin(thetaF2) + delta\_y*\cos(thetaF2)}; \\
&t(2) = (\text{xe(4) + xdash}) + \text{incr\_e6}; \% \text{New coordinate X} \\
&t(3) = (\text{ye(4) + ydash}) + \text{incr\_e7}; \% \text{New coordinate Y}
\end{align*}

\textbf{elseif} (t(2) > (bc/2-rc) \text{ } \&\text{ } t(3) < (hc/2-rc) \text{ } \&\text{ } t(3) > 0) \% Right lip + Top
\begin{align*}
&\text{delta\_x} = \text{xe(4) - t(2)}; \\
&\text{delta\_y} = \text{ye(4) - t(3)}; \\
&\text{thetaF3} = \text{atan}(\text{tan\_thetaF3}); \\
&\text{xdash} = \text{delta\_x*\cos(thetaF3) - delta\_y*\sin(thetaF3) - (t(3) - ye(12))*\tan\_thetaL3}; \\
&\text{ydash} = -\text{delta\_x*\sin(thetaF3) + delta\_y*\cos(thetaF3)}; \\
&t(2) = (\text{xe(4) + xdash}) - \text{incr\_e15}; \% \text{New coordinate X} \\
&t(3) = (\text{ye(4) - ydash}) - \text{incr\_e14}; \% \text{New coordinate Y}
\end{align*}

\textbf{elseif} (t(2) > (bc/2-rc) \text{ } \&\text{ } t(3) < 0 \text{ } \&\text{ } t(3) > (-hc/2+rc)) \% Right lip + Bottom
\begin{align*}
&\text{delta\_x} = \text{xe(4) - t(2)}; \\
&\text{delta\_y} = \text{ye(4) - t(3)}; \\
&\text{thetaF4} = \text{atan}(\text{tan\_thetaF4}); \\
&\text{xdash} = \text{delta\_x*\cos(thetaF4) - delta\_y*\sin(thetaF4) - (t(3) - ye(19))*\tan\_thetaL4}; \\
&\text{ydash} = -\text{delta\_x*\sin(thetaF4) + delta\_y*\cos(thetaF4)}; \\
&t(2) = (\text{xe(4) + xdash}) - \text{incr\_e16}; \% \text{New coordinate X} \\
&t(3) = (\text{ye(4) + ydash}) + \text{incr\_e17}; \% \text{New coordinate Y}
\end{align*}

\textbf{elseif} (t(2) <= (-bc/2+rc) \text{ } \&\text{ } t(3) >= (hc/2-rc)) \% Left corner of top lip and flange
\begin{align*}
&\text{delta\_x} = \text{xe(4) - t(2)}; \\
&\text{delta\_y} = \text{ye(4) - t(3)}; \\
&\text{thetaF1} = \text{atan}(\text{tan\_thetaF1}); \\
&\text{xdash} = \text{delta\_x*\cos(thetaF1) - delta\_y*\sin(thetaF1)}; \\
&\text{ydash} = -\text{delta\_x*\sin(thetaF1) + delta\_y*\cos(thetaF1)}; \\
&t(2) = (\text{xe(4) - xdash}) + \text{incr\_e5}; \% \text{New coordinate X} \\
&t(3) = (\text{ye(4) - ydash}) - \text{incr\_e4}; \% \text{New coordinate Y}
\end{align*}
elseif (t(2)<=(-bc/2+rc) && t(3)<=(-hc/2+rc)) % Left conner of bottom lip and flange
delta_x = xe(7) - t(2);
delta_y = t(3) - ye(7);
thetaF2 = atan(tan_thetaF2);
xdash = delta_x*cos(thetaF2) - delta_y*sin(thetaF2);
ydash = delta_x*sin(thetaF2) + delta_y*cos(thetaF2);
t(2)=xe(7) - xdash + incr_e6; % New coordinate X
t(3)=(ye(7) + ydash) + incr_e7; % New coordinate Y

elseif (t(2)>= (bc/2-rc) && t(3)>= (hc/2-rc)) % Right conner of top lip and flange
delta_x = t(2) - xe(14);
delta_y = ye(14) - t(3);
thetaF3 = atan(tan_thetaF3);
xdash = delta_x*cos(thetaF3) - delta_y*sin(thetaF3);
ydash = delta_x*sin(thetaF3) + delta_y*cos(thetaF3);
t(2)=(xe(14) + xdash) - incr_e15; % New coordinate X
t(3)=(ye(14) - ydash) - incr_e14; % New coordinate Y

elseif (t(2)>=(bc/2-rc) && t(3)<=(-hc/2+rc)) % Right conner of bottom lip and flange
delta_x = t(2) - xe(17);
delta_y = t(3) - ye(17);
thetaF4 = atan(tan_thetaF4);
xdash = delta_x*cos(thetaF4) - delta_y*sin(thetaF4);
ydash = delta_x*sin(thetaF4) + delta_y*cos(thetaF4);
t(2)=(xe(17) + xdash) - incr_e16; % New coordinate X
t(3)=(ye(17) + ydash) + incr_e17; % New coordinate Y

elseif (t(2)< 0 && t(2)>= (-gapc/2-rc) && t(3)>= (hc/2-rc)) % Left conner of top flange and web
t(2)=t(2) + incr_e5; % New coordinate X
t(3)=t(3) - incr_e4; % New coordinate Y

elseif (t(2)< 0 && t(2)>= (-gapc/2-rc) && t(3)<=(-hc/2+rc)) % Left conner of bottom flange and web
t(2)=t(2) + incr_e6; % New coordinate X
t(3)=t(3) + incr_e7; % New coordinate Y

elseif (t(2)<= (gapc/2+rc) && t(2)> 0 && t(3)>= (hc/2-rc)) % Right conner of top flange and web
t(2)=t(2) - incr_e15; % New coordinate X
t(3)=t(3) - incr_e14; % New coordinate Y

else % Right conner of bottom flange and web
t(2)=t(2) - incr_e16; % New coordinate X
t(3)=t(3) + incr_e17; % New coordinate Y
end

% Sectional imperfection modes end here
fprintf(fout, ' %g, %12f, %12f, %12f
', t(1), t(2), t(3), t(4));
end
fclose(fin); %closes the file
fclose(fout); %closes the file
end

for j=1:Nraf % open the reading and writing files of rafters
filename1=sprintf('Original-Rafter%d%d.inp',j,i);
fin=fopen(filename1, 'r'); % open the reading file
Appendix C

filename2=sprintf('Updated-Rafter%d%d.inp',j,i);
fout=fopen(filename2,'w'); % open the writing file
K=xlsread(['fourier_coefficientsR' num2str(j) num2str(i) '.xlsx'],1); % Fourier Coeffients for rafter
a=xlsread(['Imperfection_magnitude_endA_C' num2str(j) num2str(i) '.xlsx'],1); % True imperfection at end A for rafter
b=xlsread(['Imperfection_magnitude_endB_C' num2str(j) num2str(i) '.xlsx'],1); % True imperfection at end B for rafter
% --> Member and sectional imperfections parameters of the rafters <--
while ~feof(fin) % while doesn't get to the end of the file
t=str2num(fgetl(fin));
% t contains node perfect coordinates {ID X Y Z}
L=L_Raf; % Height of the column

Imp=K*Vsine(n,L,t(4))+a+(b-a)/L*t(4);
% Section dimensions of the members for sectional imperfection purposes
hr= 252.5; % Rafter height
br= 161.5; % Rafter width
rr= 6.25; % Rafter fillet radius
tr= 2.5; % Rafter thickness
yr= 0; % "coordinate" y of centroid
gapr=14.5; % gap between channels
Sr=[0 0]; % Coordinates of the shear centre

% Original Coordinates of Edges and Measuring points
Coord_Edges=xlsread('Original Coordinates.xlsx',1,'F3:G20'); % Coordinates of the edges
Coord_Lasers=xlsread('Original Coordinates.xlsx',1,'B3:C24'); % Coordinates of the measuring points
x = Coord_Lasers(:,1); % Coordinate x of the measuring points
y = Coord_Lasers(:,2); % Coordinate y of the measuring points
xe = Coord_Edges(:,1); % Coordinate x of the edges
ye = Coord_Edges(:,2); % Coordinate y of the edges

% Determine the rotation angles of flanges and lips, local imperfections of flanges and webs and the increment of edges
% Left - top flange, conner and lip
tan_thetaF1 = (Imp(2)-Imp(4))/(x(4) - x(2));
tan_thetaL1 = Imp(1)/(ye(2) - y(1));
incr_e3 = Imp(4)+(x(4)- xe(3))*tan_thetaF1;
incr_e4 = Imp(4)-(xe(4)- x(4))*tan_thetaF1;
Imp_lc3 = Imp(3)- (Imp(4)+ (x(4)-x(3))*tan_thetaF1);
% Left - bottom flange, conner and lip
tan_thetaF2 = (Imp(10)-Imp(8))/(x(8) - x(10));
tan_thetaL2 = Imp(11)/(y(11) - ye(9));
incr_e7 = Imp(8)-(xe(7)- x(8))*tan_thetaF2;
incr_e8 = Imp(8)+(x(8)-xe(8))*tan_thetaF2;
Imp_lc9 = Imp(9)- (Imp(8)+ (x(8)-x(9))*tan_thetaF2);
% Right - top flange, conner and lip
tan_thetaF3 = (Imp(13)-Imp(15))/(x(13) - x(15));
tan_thetaL3 = Imp(12)/(ye(12) - y(12));
incr_e13 = Imp(15)+(xe(13)- x(15))*tan_thetaF3;
incr_e14 = Imp(15)-(x(15)- xe(14))*tan_thetaF3;
Imp_lc14 = Imp(14)- (Imp(15)+ (x(14)-x(15))*tan_thetaF3);
% Right - bottom flange, conner and lip
tan_thetaF4 = (Imp(21)-Imp(19))/(x(21) - x(19));
tan_thetaL4 = Imp(22)/(y(22) - ye(19));
incr_e17 = Imp(19)-(x(19)- xe(17))*tan_thetaF4;
\[ \text{incr}_e_{18} = \text{Imp}(19) + (\text{xe}(18) - \text{x}(19)) \cdot \tan \theta_{F4}; \]
\[ \text{Imp}_c_{20} = \text{Imp}(20) - (\text{Imp}(19) + (\text{x}(20) - \text{x}(19)) \cdot \tan \theta_{F4}); \]

\% Left web
\[ \tan \theta_{W1} = (\text{Imp}(7) - \text{Imp}(5)) / (\text{y}(5) - \text{y}(7)); \]
\[ \text{incr}_e_{5} = \text{Imp}(5) - (\text{ye}(5) - \text{y}(5)) \cdot \tan \theta_{W1}; \]
\[ \text{incr}_e_{6} = \text{Imp}(7) + (\text{y}(7) - \text{ye}(6)) \cdot \tan \theta_{W1}; \]
\[ \text{Imp}_c_{6} = \text{Imp}(6) - (\text{Imp}(5) + (\text{y}(5) - \text{y}(6)) \cdot \tan \theta_{W1}); \]

\% Right web
\[ \text{tan}_\theta_{W2} = (\text{Imp}(18) - \text{Imp}(16)) / (\text{y}(16) - \text{y}(18)); \]
\[ \text{incr}_e_{15} = \text{Imp}(16) - (\text{ye}(15) - \text{y}(16)) \cdot \tan \theta_{W2}; \]
\[ \text{incr}_e_{16} = \text{Imp}(18) + (\text{y}(18) - \text{ye}(16)) \cdot \tan \theta_{W2}; \]
\[ \text{Imp}_c_{17} = \text{Imp}(17) - (\text{Imp}(16) + (\text{y}(16) - \text{y}(17)) \cdot \tan \theta_{W2}); \]

\% Imperfection modes start here
\[ \text{if} \ (t(2) < (-\text{gap}/2 - \text{rr}) \&\& t(2) > (-\text{br}/2 + \text{rr}) \&\& t(3) > (\text{hr}/2 - \text{rr}) \) \% Left flange + Top \% Check limits
\]
Increment_x = incr_e5;
\[ \text{incr}_\text{lci} = \text{lagrange interpolation}((\text{xe}(3), \text{x}(3), \text{xe}(4)), [0 \ \text{Imp}_c_{3} 0], t(2)); \]
Increment_y = incr_e4 + (\text{xe}(4) - t(2)) \cdot \tan \theta_{F1} + \text{incr}_\text{lci};
\[ t(2) = t(2) + \text{Increment}_x; \% \text{New coordinate X} \]
\[ t(3) = t(3) - \text{Increment}_y; \% \text{New coordinate Y} \]
\]
\% --
\[ \text{elseif} \ (t(2) < (-\text{gap}/2 - \text{rr}) \&\& t(2) > (-\text{br}/2 + \text{rr}) \&\& t(3) < (-\text{hr}/2 + \text{rr}) \) \% Left flange + Bottom \% Check limits
\]
Increment_x = incr_e6;
\[ \text{incr}_\text{lci} = \text{lagrange interpolation}((\text{xe}(7), \text{x}(9), \text{xe}(8)), [0 \ \text{Imp}_c_{9} 0], t(2)); \]
Increment_y = incr_e7 + (\text{xe}(7) - t(2)) \cdot \tan \theta_{F2} + \text{incr}_\text{lci};
\[ t(2) = t(2) + \text{Increment}_x; \% \text{New coordinate X} \]
\[ t(3) = t(3) - \text{Increment}_y; \% \text{New coordinate Y} \]
\]
\% --
\[ \text{elseif} \ (t(2) > (\text{gap}/2 + \text{rr}) \&\& t(2) < (\text{br}/2 - \text{rr}) \&\& t(3) > (\text{hr}/2 - \text{rr}) \) \% Right flange + Top \% Check limits
\]
Increment_x = incr_e15;
\[ \text{incr}_\text{lci} = \text{lagrange interpolation}((\text{xe}(13), \text{x}(14), \text{xe}(14)), [0 \ \text{Imp}_c_{1} 0], t(2)); \]
Increment_y = incr_e14 + (\text{xe}(14) - t(2)) \cdot \tan \theta_{F3} + \text{incr}_\text{lci};
\[ t(2) = t(2) - \text{Increment}_x; \% \text{New coordinate X} \]
\[ t(3) = t(3) - \text{Increment}_y; \% \text{New coordinate Y} \]
\]
\% --
\[ \text{elseif} \ (t(2) > (\text{gap}/2 + \text{rr}) \&\& t(2) < (\text{br}/2 - \text{rr}) \&\& t(3) < (-\text{hr}/2 + \text{rr}) \) \% Right flange + Bottom \% Check limits
\]
Increment_x = incr_e16;
\[ \text{incr}_\text{lci} = \text{lagrange interpolation}((\text{xe}(17), \text{x}(20), \text{xe}(18)), [0 \ \text{Imp}_c_{20} 0], t(2)); \]
Increment_y = incr_e17 + (\text{xe}(18) - t(2)) \cdot \tan \theta_{F4} + \text{incr}_\text{lci};
\[ t(2) = t(2) + \text{Increment}_x; \% \text{New coordinate X} \]
\[ t(3) = t(3) + \text{Increment}_y; \% \text{New coordinate Y} \]
\]
\% --
\[ \text{elseif} \ (t(2) < 0 \&\& t(2) > (-\text{gap}/2 - \text{rr}) \&\& t(3) < (\text{hr}/2 - \text{rr}) \&\& t(3) > (-\text{hr}/2 + \text{rr}) \) \% Left Web
\]
\[ \text{incr}_\text{lci} = \text{lagrange interpolation}((\text{ye}(5), \text{y}(6), \text{ye}(6)), [0 \ \text{Imp}_c_{6} 0], t(3)); \]
Increment_x = incr_e5 + (\text{ye}(6) - t(3)) \cdot \tan \theta_{W1} + \text{incr}_\text{lci};
Increment_y = incr_e4;
\[ t(2) = t(2) + \text{Increment}_x; \% \text{New coordinate X} \]
\[ t(3) = t(3) - \text{Increment}_y; \% \text{New coordinate Y} \]
\]
\% --
\[ \text{elseif} \ (t(2) > (\text{gap}/2 + \text{rr}) \&\& t(2) < 0 \&\& t(3) < (\text{hr}/2 - \text{rr}) \&\& t(3) > (-\text{hr}/2 + \text{rr}) \) \% Right Web
\]
\[ \text{incr}_\text{lci} = \text{lagrange interpolation}((\text{ye}(15), \text{y}(17), \text{ye}(16)), [0 \ \text{Imp}_c_{17} 0], t(3)); \]
Increment_x = incr_e15 + (\text{ye}(15) - t(3)) \cdot \tan \theta_{W2} + \text{incr}_\text{lci};
Increment_y = incr_e14;
t(2)=t(2) - Increment_x;  % New coordinate X
t(3)=t(3) - Increment_y;  % New coordinate Y

\[\text{elseif } (t(2) < (-br/2+rr) \&\& \ t(3) < (hr/2-rr) \&\& \ t(3) > 0) \text{ } \% \text{ Left lip + Top} \]
delta_x = xe(4) - t(2);
delta_y = ye(4) - t(3);
thetaF1 = atan(tan_thetaF1);
xdash = delta_x*costhetaF1 - delta_y*sinthetaF1 - (ye(2) - t(3))*tanthetaL1;
ydash = delta_x*sinthetaF1 + delta_y*costhetaF1;
t(2)=xe(4) - xdash + incr_e5;  % New coordinate X
t(3)=ye(4) - ydash - incr_e4;  % New coordinate Y

\[\text{elseif } (t(2) < (-br/2+rr) \&\& \ t(3) < 0 \&\& \ t(3) > (hr/2-rr)) \text{ } \% \text{ Left lip + Bottom} \]
delta_x = (xe(7) - t(2));
delta_y = t(3) - ye(7);
thetaF2 = atan(tan_thetaF2);
xdash = delta_x*costhetaF2 - delta_y*sinthetaF2 - (t(3) - ye(9))*tanthetaL2;
ydash = delta_x*sinthetaF2 + delta_y*costhetaF2;
t(2)=xe(7) - xdash + incr_e6;  % New coordinate X
t(3)=ye(7) + ydash + incr_e7;  % New coordinate Y

\[\text{elseif } (t(2) > (br/2-rr) \&\& \ t(3) < (hr/2-rr) \&\& \ t(3) > 0) \text{ } \% \text{ Right lip + Top} \]
delta_x = t(2) - xe(14);
delta_y = ye(14) - t(3);
thetaF3 = atan(tan_thetaF3);
xdash = delta_x*costhetaF3 - delta_y*sinthetaF3 - (ye(12) - t(3))*tanthetaL3;
ydash = delta_x*sinthetaF3 + delta_y*costhetaF3;
t(2)=xe(14) + xdash - incr_e15;  % New coordinate X
t(3)=ye(14) - ydash - incr_e14;  % New coordinate Y

\[\text{elseif } (t(2) > (br/2-rr) \&\& \ t(3) < 0 \&\& \ t(3) > (hr/2-rr)) \text{ } \% \text{ Right lip + Bottom} \]
delta_x = (xe(17) - t(2));
delta_y = t(3) - ye(17);
thetaF4 = atan(tan_thetaF4);
xdash = delta_x*costhetaF4 - delta_y*sinthetaF4 - (t(3) - ye(19))*tanthetaL4;
ydash = delta_x*sinthetaF4 + delta_y*costhetaF4;
t(2)=xe(17) + xdash - incr_e16;  % New coordinate X
t(3)=ye(17) + ydash + incr_e17;  % New coordinate Y

\[\text{elseif } (t(2) <= (-br/2+rr) \&\& \ t(3) >= (hr/2-rr)) \text{ } \% \text{ Left conner of top lip and flange} \]
delta_x = xe(4) - t(2);
delta_y = ye(4) - t(3);
thetaF1 = atan(tan_thetaF1);
xdash = delta_x*costhetaF1 - delta_y*sinthetaF1;
ydash = delta_x*sinthetaF1 + delta_y*costhetaF1;
t(2)=xe(4) - xdash + incr_e5;  % New coordinate X
t(3)=ye(4) - ydash - incr_e4;  % New coordinate Y

\[\text{elseif } (t(2) <= (-br/2+rr) \&\& \ t(3) <= (-hr/2-rr)) \text{ } \% \text{ Left conner of bottom lip and flange} \]
delta_x = xe(7) - t(2);
delta_y = t(3) - ye(7);
thetaF2 = atan(tan_thetaF2);
xdash = delta_x*costhetaF2 - delta_y*sinthetaF2;
ydash = delta_x*sinthetaF2 + delta_y*costhetaF2;
t(2)=xe(7) - xdash + incr_e6;  % New coordinate X
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\[ t(3) = (y_e(7) + \text{ydash}) + \text{incr}_e7; \] % New coordinate Y

\% New coordinate X

\begin{verbatim}
elseif (t(2) >= (br/2-rr) && t(3) >= (hr/2-rr)) % Right conner of top lip and flange
delta_x = t(2) - xe(14);
delta_y = ye(14) - t(3);
thetaF3 = atan(tan_thetaF3);
xdash = delta_x*cos(thetaF3) - delta_y*sin(thetaF3);
ydash = delta_x*sin(thetaF3) + delta_y*cos(thetaF3);
t(2) = (xe(14) + xdash) - incr_e15; % New coordinate X
t(3) = (ye(14) - ydash) - incr_e14; % New coordinate Y
\end{verbatim}

\begin{verbatim}
elseif (t(2) >= (br/2-rr) && t(3) <= (-hr/2+rr)) % Right conner of bottom lip and flange
delta_x = t(2) - xe(17);
delta_y = t(3) - ye(17);
thetaF4 = atan(tan_thetaF4);
xdash = delta_x*cos(thetaF4) - delta_y*sin(thetaF4);
ydash = delta_x*sin(thetaF4) + delta_y*cos(thetaF4);
t(2) = (xe(17) + xdash) - incr_e16; % New coordinate X
t(3) = (ye(17) + ydash) + incr_e17; % New coordinate Y
\end{verbatim}

\begin{verbatim}
elseif (t(2) < 0 && t(2) >= (-gapr/2-rr) && t(3) >= (hr/2-rr)) % Left conner of top flange and web
t(2) = t(2) + incr_e5; % New coordinate X
t(3) = t(3) - incr_e4; % New coordinate Y
\end{verbatim}

\begin{verbatim}
elseif (t(2) < 0 && t(2) >= (-gapr/2-rr) && t(3) <= (-hr/2+rr)) % Left conner of bottom flange and web
t(2) = t(2) + incr_e6; % New coordinate X
t(3) = t(3) + incr_e7; % New coordinate Y
\end{verbatim}

\begin{verbatim}
elseif (t(2) <= (gapr/2+rr) && t(2) > 0 && t(3) >= (hr/2-rr)) % Right conner of top flange and web
t(2) = t(2) - incr_e15; % New coordinate X
t(3) = t(3) - incr_e14; % New coordinate Y
\end{verbatim}

\begin{verbatim}
else % Right conner of bottom flange and web
t(2) = t(2) - incr_e16; % New coordinate X
t(3) = t(3) + incr_e17; % New coordinate Y
end
\end{verbatim}

\begin{verbatim}
\% Sectional imperfection modes end here
fprintf(fout,' %g, %12f, %12f, %12f
',t(1),t(2),t(3),t(4));
end
fclose(fin); %closes the file
fclose(fout); %closes the file
end
end
\end{verbatim}

C.1.3. Matlab code for the incorporation of frame imperfections

clc; clear all;
\%open the reading and writing files
Nfra = 7; % Number of test frames
OPlumbness1=[-0.0015 -0.0043 0.0013 -0.0041 -0.0007 -0.0003 -0.0019]; % Out-of-Plumbness of the south Column
OPlumbness2=[0.0013 0.0023 0.0051 0.0013 -0.0014 0.0024 -0.0027]; % Out-of-Plumbness of the North Column
for i = 1:Nfra
fileID = sprintf('FrameTest%d.inp',i);
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```c
fin=fopen(fileID, 'r'); % open the reading file
filename=sprintf('UpdatedFrameTest-%d.inp', i);
% open the writing file
outofplumb1=OPlumbness1(i);
outofplumb2=OPlumbness2(i);
k=0;
while ~feof(fin) % while doesn't get to the end of the file
if k==0
t=fgetl(fin); % store text of the line in s
if strncmp(t, '*Instance, name=MD_Column_L', 27)
    fprintf(fout, '%s
', t);
k=1; % sets a different value of k to change the value next to this line
elseif strncmp(t, '*Instance, name=MD_Column_R', 27)
    fprintf(fout, '%s
', t);
k=2; % sets a different value of k to change the value next to this line
elseif strncmp(t, '*Instance, name=MD_Rafter_L', 27)
    fprintf(fout, '%s
', t);
k=3; % sets a different value of k to change the value next to this line
elseif strncmp(t, '*Instance, name=MD_Rafter_R', 27)
    fprintf(fout, '%s
', t);
k=4; % sets a different value of k to change the value next to this line
elseif strncmp(t, '*Instance, name="MD_Knee Bracket _R"', 36)
    fprintf(fout, '%s
', t);
k=5; % sets a different value of k to change the value next to this line
elseif strncmp(t, '*Instance, name="MD_KneeBracket AC15025_L"', 40)
    fprintf(fout, '%s
', t);
k=6; % sets a different value of k to change the value next to this line
elseif strncmp(t, '*Instance, name="MD_KneeBracket AC15025_R"', 40)
    fprintf(fout, '%s
', t);
k=7; % sets a different value of k to change the value next to this line
elseif strncmp(t, '*Instance, name="MD_Apex Brackets"', 34)
    fprintf(fout, '%s
', t);
k=8; % sets a different value of k to change the value next to this line
elseif strncmp(t, '*Instance, name="MD_Knee Bracket _L"', 36)
    fprintf(fout, '%s
', t);
k=9; % sets a different value of k to change the value next to this line
elseif strncmp(t, '*Instance, name="MD_PackingCleat CL1"', 38)
    fprintf(fout, '%s
', t);
k=10; % sets a different value of k to change the value next to this line
elseif strncmp(t, '*Instance, name="MD_PackingCleat CL2"', 38)
    fprintf(fout, '%s
', t);
k=11; % sets a different value of k to change the value next to this line
elseif strncmp(t, '*Instance, name="MD_Apex Brackets"', 34)
    fprintf(fout, '%s
', t);
k=12; % sets a different value of k to change the value next to this line
elseif strncmp(t, '*Instance, name="MD_RafterTie Cleat_L"', 36)
    fprintf(fout, '%s
', t);
k=13; % sets a different value of k to change the value next to this line
elseif strncmp(t, '*Instance, name="MD_RafterTie Cleat_R"', 36)
    fprintf(fout, '%s
', t);
k=14; % sets a different value of k to change the value next to this line
elseif strncmp(t, '*Instance, name="MD_Apex Brackets"', 34)
    fprintf(fout, '%s
', t);
k=15; % sets a different value of k to change the value next to this line
elseif strncmp(t, '*Instance, name="MD_KneeBracket AC15025_L"', 36)
    fprintf(fout, '%s
', t);
k=16; % sets a different value of k to change the value next to this line
elseif strncmp(t, '*Instance, name="MD_PackingCleat CL1"', 38)
    fprintf(fout, '%s
', t);
k=17; % sets a different value of k to change the value next to this line
else
    fprintf(fout, '%s
', t);
end
```

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```matlab
k=18; % sets a different value of k to change the value next to this line
elseif strncmp(t,'*Instance, name="MD_Packing Cleat-CL3"',38)
    fprintf(fout,'%s
',t);
    k=19; % sets a different value of k to change the value next to this line
elseif strncmp(t,'*Instance, name="MD_Packing Cleat-CL4"',38)
    fprintf(fout,'%s
',t);
    k=20; % sets a different value of k to change the value next to this line
elseif strncmp(t,'*Instance, name="MD_Packing Cleat-CR1"',38)
    fprintf(fout,'%s
',t);
    k=21; % sets a different value of k to change the value next to this line
elseif strncmp(t,'*Instance, name="MD_Packing Cleat-CR2"',38)
    fprintf(fout,'%s
',t);
    k=22; % sets a different value of k to change the value next to this line
elseif strncmp(t,'*Instance, name="MD_Packing Cleat-CR3"',38)
    fprintf(fout,'%s
',t);
    k=23; % sets a different value of k to change the value next to this line
elseif strncmp(t,'*Instance, name="MD_Packing Cleat-CR4"',38)
    fprintf(fout,'%s
',t);
    k=24; % sets a different value of k to change the value next to this line
else
    fprintf(fout,'%s
',t); %write content of line in new file
end

% open path k=1
elseif k==1
    t=fgetl(fin);
    fprintf(fout,'%s
',t);
    a=str2num(t);
    t=fgetl(fin);
    d=AxisAngle2(-outofplumb1);
    fprintf(fout,' %.12e, %.12e, %.12e, %.12e, %.12e,
                %.12e,%.12e
',a(1),a(2),a(3),a(1)+d(2),a(2)+d(3),a(3)+d(4),d(1));
    t=fgetl(fin);
    t=sprintf('Updated-Column%d_%d.inp',1,i);
    fprintf(fout,'*Node, Input=%s
',t);
k=0;

% open path k=2
elseif k==2
    t=fgetl(fin);
    fprintf(fout,'%s
',t);
    a=str2num(t);
    t=fgetl(fin);
    d=AxisAngle2(-outofplumb2);
    fprintf(fout,' %.12e, %.12e, %.12e, %.12e, %.12e,
                %.12e,%.12e
',a(1),a(2),a(3),a(1)+d(2),a(2)+d(3),a(3)+d(4),d(1));
    t=fgetl(fin);
    t=sprintf('Updated-Column%d_%d.inp',2,i);
    fprintf(fout,'*Node, Input=%s
',t);
k=0;

% open path k=3
elseif k==3
    t=str2num(fgetl(fin));
    t(1)= t(1) - t(2)*(outofplumb1+outofplumb2)/2;
    fprintf(fout,' %.12e, %.12e, %.12e
',t(1),t(2),t(3));
    t=str2num(fgetl(fin));
    t(1)= t(1) - t(2)*(outofplumb1+outofplumb2)/2;
    t(4)= t(4) - t(2)*(outofplumb1+outofplumb2)/2;
    fprintf(fout,' %.12e, %.12e, %.12e, %.12e, %.12e,
                %.12e,%.12e
',t(1),t(2),t(3),t(4),t(5),t(6),t(7));
    t=fgets(fin);
    t=sprintf('Updated-Rafter%d_%d.inp',1,i);
    fprintf(fout,'*Node, Input=%s
',t);```

% open path k=4

elseif k==4
    t=str2num(fgetl(fin));
    t(1)= t(1) - t(2)*(outofplumb1+outofplumb2)/2;
    fprintf(fout, ' %.12e, %.12e, %.12e\n',t(1),t(2),t(3));
    t=str2num(fgetl(fin));
    t(1)= t(1) - t(2)*(outofplumb1+outofplumb2)/2;
    t(4)= t(4) - t(2)*(outofplumb1+outofplumb2)/2;
    fprintf(fout, ' %.12e, %.12e, %.12e, %.12e, %.12e, %.12e, %.12e\n',t(1),t(2),t(3),t(4),t(5),t(6),t(7));
    t=fgets(fin);
    t=sprintf('Updated-Rafter%d_%d.inp',2,i);
    fprintf(fout,'*Node, Input=%s\n',t);
    k=0;

% open path k=5

elseif k==5
    t=str2num(fgetl(fin));
    t(1)= t(1) + t(2)*outofplumb2;
    fprintf(fout, ' %.12e, %.12e, %.12e\n',t(1),t(2),t(3));
    t=str2num(fgetl(fin));
    t(1)= t(1) + t(2)*outofplumb2;
    t(4)= t(4) + t(2)*outofplumb2;
    fprintf(fout, ' %.12e, %.12e, %.12e, %.12e, %.12e, %.12e, %.12e\n',t(1),t(2),t(3),t(4),t(5),t(6),t(7));
    k=0;

% open path k=6

elseif k==6
    t=str2num(fgetl(fin));
    t(1)= t(1) + t(2)*outofplumb2;
    fprintf(fout, ' %.12e, %.12e, %.12e\n',t(1),t(2),t(3));
    t=str2num(fgetl(fin));
    t(1)= t(1) + t(2)*outofplumb2;
    t(4)= t(4) + t(2)*outofplumb2;
    fprintf(fout, ' %.12e, %.12e, %.12e, %.12e, %.12e, %.12e, %.12e\n',t(1),t(2),t(3),t(4),t(5),t(6),t(7));
    k=0;

% open path k=7

elseif k==7
    t=str2num(fgetl(fin));
    t(1)= t(1) + t(2)*outofplumb2;
    fprintf(fout, ' %.12e, %.12e, %.12e\n',t(1),t(2),t(3));
    t=str2num(fgetl(fin));
    t(1)= t(1) + t(2)*outofplumb2;
    t(4)= t(4) + t(2)*outofplumb2;
    fprintf(fout, ' %.12e, %.12e, %.12e, %.12e, %.12e, %.12e, %.12e\n',t(1),t(2),t(3),t(4),t(5),t(6),t(7));
    k=0;

% open path k=8

elseif k==8
    t=str2num(fgetl(fin));
    t(1)= t(1) + t(2)*outofplumb2;
    fprintf(fout, ' %.12e, %.12e, %.12e\n',t(1),t(2),t(3));
    t=str2num(fgetl(fin));
    t(1)= t(1) + t(2)*outofplumb2;
    t(4)= t(4) + t(2)*outofplumb2;
    fprintf(fout, ' %.12e, %.12e, %.12e, %.12e, %.12e, %.12e, %.12e\n',t(1),t(2),t(3),t(4),t(5),t(6),t(7));
    k=0;

% open path k=9

elseif k==9
    t=str2num(fgetl(fin));
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t(1) = t(1) + t(2)*(outofplumb1+outofplumb2)/2;
fprintf(fout,' %.12e, %.12e, %.12e
',t(1),t(2),t(3));
t=str2num(fgetl(fin));
t(1) = t(1) + t(2)*(outofplumb1+outofplumb2)/2;
t(4) = t(4) + t(2)*(outofplumb1+outofplumb2)/2;
fprintf(fout,' %.12e, %.12e, %.12e
',t(1),t(2),t(3),t(4),t(5),t(6),t(7));
k=0;
% open path k=10
elseif k==10
    t=str2num(fgetl(fin));
    t(1) = t(1) + t(2)*(outofplumb1+outofplumb2)/2;
    fprintf(fout,' %.12e, %.12e, %.12e
',t(1),t(2),t(3));
t=str2num(fgetl(fin));
t(1) = t(1) + t(2)*(outofplumb1+outofplumb2)/2;
t(4) = t(4) + t(2)*(outofplumb1+outofplumb2)/2;
    fprintf(fout,' %.12e, %.12e, %.12e, %.12e, %.12e, %.12e,%.12e
',t(1),t(2),t(3),t(4),t(5),t(6),t(7));
k=0;
% open path k=11
elseif k==11
    t=str2num(fgetl(fin));
    t(1) = t(1) + t(2)*(outofplumb1+outofplumb2)/2;
    fprintf(fout,' %.12e, %.12e, %.12e
',t(1),t(2),t(3));
t=str2num(fgetl(fin));
    t(1) = t(1) + t(2)*(outofplumb1+outofplumb2)/2;
    t(4) = t(4) + t(2)*(outofplumb1+outofplumb2)/2;
    fprintf(fout,' %.12e, %.12e, %.12e, %.12e, %.12e, %.12e,%.12e
',t(1),t(2),t(3),t(4),t(5),t(6),t(7));
k=0;
% open path k=12
elseif k==12
    t=str2num(fgetl(fin));
    t(1) = t(1) + t(2)*(outofplumb1+outofplumb2)/2;
    fprintf(fout,' %.12e, %.12e, %.12e
',t(1),t(2),t(3));
t=str2num(fgetl(fin));
    t(1) = t(1) + t(2)*(outofplumb1+outofplumb2)/2;
    t(4) = t(4) + t(2)*(outofplumb1+outofplumb2)/2;
    fprintf(fout,' %.12e, %.12e, %.12e, %.12e, %.12e, %.12e,%.12e
',t(1),t(2),t(3),t(4),t(5),t(6),t(7));
k=0;
% open path k=13
elseif k==13
    t=str2num(fgetl(fin));
    t(1) = t(1) + t(2)*(outofplumb1+outofplumb2)/2;
    fprintf(fout,' %.12e, %.12e, %.12e
',t(1),t(2),t(3));
t=str2num(fgetl(fin));
    t(1) = t(1) + t(2)*(outofplumb1+outofplumb2)/2;
    t(4) = t(4) + t(2)*(outofplumb1+outofplumb2)/2;
    fprintf(fout,' %.12e, %.12e, %.12e, %.12e, %.12e, %.12e,%.12e
',t(1),t(2),t(3),t(4),t(5),t(6),t(7));
k=0;
% open path k=14
elseif k==14
    t=str2num(fgetl(fin));
    t(1) = t(1) + t(2)*(outofplumb1+outofplumb2)/2;
    fprintf(fout,' %.12e, %.12e, %.12e
',t(1),t(2),t(3));
t=str2num(fgetl(fin));
    t(1) = t(1) + t(2)*(outofplumb1+outofplumb2)/2;
    t(4) = t(4) + t(2)*(outofplumb1+outofplumb2)/2;
    fprintf(fout,' %.12e, %.12e, %.12e, %.12e, %.12e, %.12e,%.12e
',t(1),t(2),t(3),t(4),t(5),t(6),t(7));
k=0;
% open path k=15
elseif k==15
    t=str2num(fgetl(fin));
    t(1)= t(1) - t(2)*(outofplumb1+outofplumb2)/2;
    fprintf(fout, ' %.12e, %.12e, %.12e
', t(1), t(2), t(3));
    t=str2num(fgetl(fin));
    t(1)= t(1) - t(2)*(outofplumb1+outofplumb2)/2;
    t(4)= t(4) - t(2)*(outofplumb1+outofplumb2)/2;
    fprintf(fout, ' %.12e, %.12e, %.12e, %.12e, %.12e, %.12e
', t(1), t(2), t(3), t(4), t(5), t(6), t(7));
    k=0;
% open path k=16
elseif k==16
    t=str2num(fgets(fin));
    t(1)= t(1) + t(2)*outofplumb1;
    fprintf(fout, ' %.12e, %.12e, %.12e
', t(1), t(2), t(3));
    t=str2num(fgets(fin));
    t(1)= t(1) + t(2)*outofplumb1;
    t(4)= t(4) + t(2)*outofplumb1;
    fprintf(fout, ' %.12e, %.12e, %.12e, %.12e, %.12e, %.12e
', t(1), t(2), t(3), t(4), t(5), t(6), t(7));
    k=0;
% open path k=17
elseif k==17
    t=str2num(fgets(fin));
    t(1)= t(1) + t(2)*outofplumb1;
    fprintf(fout, ' %.12e, %.12e, %.12e
', t(1), t(2), t(3));
    k=0;
% open path k=18
elseif k==18
    t=str2num(fgets(fin));
    t(1)= t(1) + t(2)*outofplumb1;
    fprintf(fout, ' %.12e, %.12e, %.12e
', t(1), t(2), t(3));
    k=0;
% open path k=19
elseif k==19
    t=str2num(fgets(fin));
    t(1)= t(1) + t(2)*outofplumb1;
    fprintf(fout, ' %.12e, %.12e, %.12e
', t(1), t(2), t(3));
    k=0;
% open path k=20
elseif k==20
    t=str2num(fgets(fin));
    t(1)= t(1) + t(2)*outofplumb1;
    fprintf(fout, ' %.12e, %.12e, %.12e
', t(1), t(2), t(3));
    k=0;
% open path k=21
elseif k==21
    t=str2num(fgetl(fin));
    t(1)= t(1) + t(2)*outofplumb2;
    fprintf(fout, ' %.12e, %.12e, %.12e
', t(1), t(2), t(3));
    k=0;
% open path k=22
elseif k==22
    t=str2num(fgetl(fin));
    t(1)= t(1) + t(2)*outofplumb2;
    fprintf(fout, ' %.12e, %.12e, %.12e
', t(1), t(2), t(3));
    k=0;
% open path k=23
elseif k==23
    t=str2num(fgetl(fin));
    t(1)= t(1) + t(2)*outofplumb2;
    fprintf(fout, ' %.12e, %.12e, %.12e
', t(1), t(2), t(3));
    k=0;
t(1)= t(1) + t(2)*outofplumb2;
fprintf(fout,' %.12e, %.12e, %.12e\n',t(1),t(2),t(3));
k=0;
% open path k=24
elseif k==24
t=str2num(fgetl(fin));
t(1)= t(1) + t(2)*outofplumb2;
fprintf(fout,' %.12e, %.12e, %.12e\n',t(1),t(2),t(3));
k=0;
end
end
% End of modifications
% Closing the files
fclose(fin);
%fclose(fin2);
fclose(fout);
end
% Exit

C.2. INCORPORATION OF ASSUMED GEOMETRIC IMPERFECTIONS INTO FINITE ELEMENT MODELS FOR PARAMETRIC STUDIES

C.2.1. The formulations used to modify the node coordinates to account for sectional and member imperfections

Sectional and member imperfections are characterised by a certain cross-sectional displacement that is scaled by a certain amplitude varying longitudinally. In term of mathematical, each of the sectional imperfections (i.e. distortional and local imperfections) and member imperfections (i.e. bow, camber, and twist imperfections) can be represented by the product of a cross-sectional shape function \( \phi(x, y) \) dependent on cross-section coordinates \( x \) and \( y \) and an amplitude function \( f(z) \) dependent only on axial coordinate \( z \), as per Equation (C.115). The total sectional and member imperfections \( \delta_{SM,total} \) is the sum of all member and sectional imperfections, as shown in Equation (C.116).

\[
\delta_i(x, y, z) = \phi_i(x, y) \times f_i(z) \tag{C.115}
\]

\[
\delta_{SM,total}(x, y, z) = \sum \delta_i(x, y, z) \tag{C.116}
\]

where \( i \) represents the type of imperfection – distortional, local, camber, bow, and twist.

The imperfections were incorporated into the FE models by adding the total sectional and member imperfections \( \delta_{SM,total} \) to the perfect geometry configuration \( u_0(x_0, y_0, z_0) \) to obtain the imperfect geometry configuration \( u_f(x_f, y_f, z_f) \), as presented in Equation (C.117).

\[
u_f(x_f, y_f, z_f) = \delta_{SM,total}(x, y, z) + u_0(x_0, y_0, z_0) \tag{C.117}
\]

The details of the cross-sectional shape and amplitude functions for each type of imperfection are described below.

C.2.1.1. Member imperfection functions
Bow and camper imperfections have the shape of a pure translation in the direction of strong and weak axes, respectively. The cross-section shape functions are given by Equations (C.118) and (C.119).

\[
\begin{align*}
\varphi_{\text{bow}}(x,y) &= \begin{bmatrix} 1 \\ 0 \end{bmatrix} \\
\varphi_{\text{camber}}(x,y) &= \begin{bmatrix} 0 \\ 1 \end{bmatrix}
\end{align*}
\]

As discussed in Section 4.8.1, the amplitude functions of the bow and camber imperfections were assumed to be a single sinusoidal function, and can be expressed as follows:

\[
\begin{align*}
f_{\text{bow}}(z) &= G_1 \sin \left( \frac{\pi z}{L} \right) \\
f_{\text{camber}}(z) &= G_2 \sin \left( \frac{\pi z}{L} \right)
\end{align*}
\]

in which, \( G_1 \) and \( G_2 \) are the amplitude of the bow and camber imperfections and \( L \) is the global half-wavelength.

The twist imperfection has the shape of a pure rotation. Mathematically, the twist imperfection can be expressed directly by Equation (C.122).

\[
\delta_{\text{twist}}(x, y) = \begin{bmatrix} d_0 (\cos(\theta_0 + f_{\text{twist}}) - \cos \theta_0) \\ d_0 (\sin(\theta_0 + f_{\text{twist}}) - \sin \theta_0) \end{bmatrix}
\]

where

\[
\begin{align*}
d_0 &= \sqrt{(x-x_s)^2 + (y-y_s)^2} \\
\theta_0 &= \tan^{-1} \left( \frac{y-y_s}{x-x_s} \right)
\end{align*}
\]

\( x_s, y_s \) are the coordinates of the shear centre, and \( f_{\text{twist}} \) represents the amplification factor.

The amplitude functions of twist imperfection, as described in Section 4.8.1, can be expressed by Equation (C.125).

\[
f_{\text{twist}}(z) = G_3 \sin \left( \frac{\pi z}{L} \right)
\]

in which, \( G_3 \) is the amplitude of the twist imperfection and \( L \) is the global half-wavelength.

**C.2.1.1. Sectional imperfection functions**

Sectional imperfections imply a distortion of the cross-section and thus, cross-section shape functions can be expressed in terms of a reference displacement that typically takes the maximum value of imperfection within the distorted cross-section.
For the double lipped channel sections, as shown in Figure C.12, the cross-section shape functions are:

\[ \varphi_{l_a}(x, y) = \begin{bmatrix} 0 & \frac{x + \Delta/2 + r_c}{b_f - 2r_c} \\ 0 & \frac{x - \Delta/2 - r_c}{b_f - 2r_c} \\ 0 & -\frac{x + \Delta/2 + r_c}{b_f - 2r_c} \\ 0 & -\frac{x - \Delta/2 - r_c}{b_f - 2r_c} \\ -\frac{y^2}{(D - 2r_c)(b_f - 2r_c)} + \frac{(D - 2r_c)}{4((b_f - 2r_c))} & 0 \\ 0 & 0 \end{bmatrix} \]

- for top left flange and lip
- for top right flange and lip
- for bottom left flange and lip
- for bottom right flange and lip
- for webs
- for flange – web junctions

\[ \varphi_{l_b}(x, y) = \begin{bmatrix} 0 & \frac{b_f - 2r_c}{D - 2r_c} \left( \frac{4(x + b_f/2 + \Delta/2)^2}{(b_f - 2r_c)^2} - 1 \right) \\ 0 & -\frac{b_f - 2r_c}{D - 2r_c} \left( \frac{4(x - b_f/2 - \Delta/2)^2}{(b_f - 2r_c)^2} + 1 \right) \\ \frac{4(y - (D/2 - r_c))}{D - 2r_c} & 0 \\ \frac{4(y - (D/2 - r_c))}{D - 2r_c} & 0 \\ \frac{-4y^2}{(D - 2r_c)^2} + 1 & 0 \\ 0 & 0 \end{bmatrix} \]

- for top left flange
- for top right flange
- for top lip
- for top right lip
- for webs
- for flange – web junctions

\[ \varphi_{l_c}(x, y) = \begin{bmatrix} 0 & -\frac{b_f - 2r_c}{D - 2r_c} \left( \frac{4(x + b_f/2 + \Delta/2)^2}{(b_f - 2r_c)^2} + 1 \right) \\ 0 & \frac{b_f - 2r_c}{D - 2r_c} \left( \frac{4(x - b_f/2 - \Delta/2)^2}{(b_f - 2r_c)^2} - 1 \right) \\ -\frac{4(y + (D/2 - r_c))}{D - 2r_c} & 0 \\ -\frac{4(y + (D/2 - r_c))}{D - 2r_c} & 0 \\ 0 & 0 \end{bmatrix} \]

- for bottom left flange
- for bottom right flange
- for bottom left lip
- for bottom right lip
- for corners
where $D$ is the depth of the section, $b_f$ is the width of the flange, $\Delta$ is the gap between the two channels, and $r_c$ is the radius of the corner, as shown in Figure C.12.

![Figure C.12. Sectional imperfections: (a) Distortional imperfection; (b) Local imperfection](image)

The amplitude functions of distortional and local imperfections are given in Equations (C.128) and (C.129).

\[
f_{\text{Dist}}(z) = d_2 \sin \left( \frac{\pi z}{L_d} \right) \tag{C.128}
\]

\[
f_{\text{Lc}}(z) = d_1 \sin \left( \frac{\pi z}{L_l} \right) \tag{C.129}
\]

in which, $d_1$ and $d_2$ represent the maximum amplitudes of the local and distortional imperfections, respectively, and $L_l$ represents the local buckling length and $L_d$ represents the distortional buckling length.

### C.2.2. Typical matlab code developed to incorporate the sectional and member imperfections for parametric studies

```matlab
% This program uses the member perfect geometry to impose member and sectional imperfections
% For Cold-rolled Aluminium Portal Frames
clc, clearvars
Ncol = 2;  % Number of columns
L_col = 5232;  % The length of Column
G1 = 0.900;  % Amplitude of the bow imperfection
G2 = 0.784;  % Amplitude of the camber imperfection
G3 = 0.009;  % Amplitude of the twist imperfection (in radian)
d1 = 0.62;  % Amplitude of the local imperfection
d2 = 1.734;  % Amplitude of the Distortional imperfection

% --> Member and sectional imperfections parameters
for i=1:Ncol
    % open the reading and writing files of uprights
    filename1 = sprintf('PFrame-Column%d.inp',i);
    fin = fopen(filename1,'r');  % open the reading file
    filename2 = sprintf('PFrame-ColumnNewA_%d.inp',i);
    fout = fopen(filename2,'w');  % open the writing file
    % --> Member and sectional imperfections parameters of the columns
```

569
while ~feof(fin) % while doesn't get to the end of the file
    t = str2num(fgetl(fin)); % store line values in vector t
    % t contains node perfect coordinates {ID X Y Z}
    L = L_col; % Height of the column
    Ld = 670; % Identify the distortional buckling half-wavelength
    Ll = 140; % Identify the local buckling half-wavelength
    BowFc = G1*sin(pi*t(4)/L);
    CamberFc = G2*sin(pi*t(4)/L);
    TwistFc = G3*sin(pi*t(4)/L);
    DistFc = d2*sin(pi*t(4)/Ld);
    LocalFc = d1*sin(pi*t(4)/Ll);
    % ------------------------------------------------------------------------
    % Section dimensions of the members for sectional imperfection purposes
    hc = 252.5; % Column height
    bc = 161.5; % Column width
    rc = 6.25; % Column fillet radius
    tc = 2.5; % Column thickness
    yc = 0; % "coordinate" y of centroid
    gapc = 14.5; % gap between channels
    Sc = [0 0]; % Coordinates of the shear centre
    d = ((t(3)-Sc(2))^2+(t(2)-Sc(1))^2)^(1/2); % Distance of point to shear centre
    teta0 = atan2(t(3)-Sc(2),t(2)-Sc(1)); % Initial angle of point in radians
    Dbow = BowFc;
    Dcamber = CamberFc;
    Dtwist = [d*(cos(teta0+TwistFc)-cos(teta0))
                d*(sin(teta0+TwistFc)-sin(teta0))]; % Twist in radians
    % Member imperfection modes end here
    % Sectional imperfection modes start here
    Dteta = DistFc/(bc/2-gapc/2-2*rc);
    if (t(2)<(-gapc/2-rc) && t(3)> 0) % Left flange + Left Lip + Top % Check limits
        Ddist(1)= 0;
        Ddist(2)= (t(2)+(gapc/2+rc))* Dteta; % coordinate y increment
    elseif (t(2)<(-gapc/2-rc) && t(3)<0) % Left flange + Left Lip + Bottom % Check limits
        Ddist(1)= 0;
        Ddist(2)= -(t(2)+(gapc/2+rc))* Dteta; % coordinate y increment
    elseif (t(2)> (gapc/2+rc) && t(3)> 0) % Right flange + Right Lip + Top % Check limits
        Ddist(1)= 0;
        Ddist(2)= (t(2)-(gapc/2+rc))* Dteta; % coordinate y increment
    elseif (t(2)> (gapc/2+rc) && t(3)<0) % Right flange + Right Lip + Bottom % Check limits
        Ddist(1)= 0;
        Ddist(2)= -(t(2)-(gapc/2+rc))* Dteta; % coordinate y increment
    elseif (t(2)< (gapc/2+rc) && t(2)> (-gapc/2-rc) && t(3)< (hc/2-rc) && t(2)> (-hc/2+rc)) % Web
        Ddist(1)= Dteta*(t(3))^2/((2*(-hc/2+rc))-(Dteta/2)*(-hc/2+rc)); % coordinate x increment
        Ddist(2)= 0;
    else
        Ddist(1)= 0;
        Ddist(2)= 0;
    end
% Local
if (t(2)<(-gapc/2 - rc) && t(2)>(-bc/2 + rc) && t(3)>hc/2) % Left flange + Top % Check limits
Dlocal(1)=0;
Dlocal(2)= -LocalFc*((bc/2-gapc/2-2*rc)/(2*(hc/2-rc)))+LocalFc*((bc/2-gapc/2-2*rc)/(2*(hc/2-rc)))*t(2)+bc/4+gapc/4)^2/(bc/4-gapc/4)^2);

elseif (t(2)<(-gapc/2 - rc) && t(2)>(-bc/2 + rc) && t(3)<(-hc/2) % Left flange + Bottom % Check limits
Dlocal(1)=0;
Dlocal(2)= LocalFc*(bc/2-gapc/2-2*rc)/(2*(hc/2-rc))-LocalFc*(bc/2-gapc/2-2*rc)/(2*(hc/2-rc)))*t(2)+bc/4+gapc/4)^2/(bc/4-gapc/4)^2);

elseif (t(2)>gapc/2 && t(2)<bc/2 && t(3)>hc/2) % Right flange + Top % Check limits
Dlocal(1)=0;
Dlocal(2)= LocalFc*bc/2-gapc/2-2*rc)/(2*(hc/2-rc))-LocalFc*(bc/2-gapc/2-2*rc)/(2*(hc/2-rc)))*t(2)+bc/4+gapc/4)^2/(bc/4-gapc/4)^2);

elseif (t(2)>gapc/2 && t(2)<bc/2 && t(3)<hc/2) % Right flange + Bottom % Check limits
Dlocal(1)=0;
Dlocal(2)= -LocalFc*(bc/2-gapc/2-2*rc)/(2*(hc/2-rc))+LocalFc*(bc/2-gapc/2-2*rc)/(2*(hc/2-rc)))*t(2)+bc/4+gapc/4)^2/(bc/4-gapc/4)^2);

elseif (t(2)<(gapc/2 + rc) && t(2)>0) % Web
Dlocal(1)= LocalFc-LocalFc*t(3)^2/(hc/2-rc)^2;
Dlocal(2)= 0;

elseif (t(2)<(gapc/2 + rc) && t(3)<0 && t(3)>hc/2) % Left lip + Top
Dlocal(1)= 2*LocalFc*(t(3)+hc/2)/hc/2-rc);% New coordinate X
Dlocal(2)= 0;

elseif (t(2)<0 && t(3)>hc/2) % Left lip + Bottom
Dlocal(1)= -2*LocalFc*(t(3)+hc/2)/hc/2-rc);% New coordinate Y
Dlocal(2)= 0;

elseif (t(2)>0 && t(3)<0 && t(3)>hc/2) % Right lip + Top
Dlocal(1)= 2*LocalFc*(t(3)-hc/2)/hc/2-rc);% New coordinate X
Dlocal(2)= 0;

elseif (t(2)>0 && t(3)<0 && t(3)>hc/2) % Right lip + Bottom
Dlocal(1)= -2*LocalFc*(t(3)+hc/2)/hc/2-rc);% New coordinate Y
Dlocal(2)= 0;

else
Dlocal(1)= 0;
Dlocal(2)= 0;
end

% Sectional imperfection modes end here
t(2)=t(2) + Dbow + Dtwist(1) + Ddist(1) + Dlocal(1); % New coordinate X
t(3)=t(3) + Dcamber + Dtwist(2) + Ddist(2) + Dlocal(2); % New coordinate Y
fprintf(fout,' %g, %12f, %12f, %12f
',t(1),t(2),t(3),t(4));
end
fclose(fin); %closes the file
fclose(fout); %closes the file
APPENDIX D.

ADDITIONAL MATERIAL FOR DESIGN STRENGTH OF COLD-ROLLED ALUMINIUM PORTAL FRAMES

D.1. FINITE STRIP BUCKLING ANALYSIS OF BACK-TO-BACK CHANNEL SECTIONS

The finite strip buckling analysis was performed using Thin-Wall-2_V1_1 software. The relevant inputs/outputs are given below.

Figure D.1. Thin-wall-2 inputs
1. Cold-rolled Aluminium double-channel (2AC25025) member subjected to axial compression

Figure D.2. Double-channel member subjected to axial compression

Figure D.3. The signature curve of 2AC25025 member subjected to axial compression
2. Cold-rolled Aluminium double-channel (2AC25025) member subjected to major axis bending moment

![Figure D.4. Double-channel member subjected to axial compression](image1)

Figure D.4. Double-channel member subjected to axial compression

![Figure D.5. The signature curve of 2AC25025 member subjected to major axis bending](image2)

Figure D.5. The signature curve of 2AC25025 member subjected to major axis bending
D.2. DESIGN ACTION EFFECTS OBTAINED BY MASTAN2

D.2.1. Typical frame model

Figure D.6. Typical frame model in Mastan2 (Frame Tests 1 and 2)

Figure D.7. Typical frame model in Mastan2 (Frame Test 6)
D.2.2. Design action effects

Design axial force and bending moment for Frame Tests 1 and 2

Load combination: $1.2G+1.5Q$

Figure D.8. Frame Tests 1 and 2: (a) Axial force diagram; (b) Bending moment diagram
Design axial force and bending moment for Frame Test 3

Load combination: $1.2G + W_u$

Figure D.9. Frame Test 3: (a) Axial force diagram; (b) Bending moment diagram
Design axial force and bending moment for Frame Test 4

Load combination: $1.2G + 1.5Q$

Figure D.10. Frame Test 4: (a) Axial force diagram; (b) Bending moment diagram
Design axial force and bending moment for Frame Test 5

Load combination: 1.2G+W_u

Figure D.11. Frame Test 5: (a) Axial force diagram; (b) Bending moment diagram
Design axial force and bending moment for Frame Test 6

Load combination: $1.2G + 1.5Q$

Figure D.12. Frame Test 6: (a) Axial force diagram; (b) Bending moment diagram
Appendix D

Design axial force and bending moment for Frame Test 7

Load combination: $1.2G+1.5Q$

Figure D.13. Frame Test 7: (a) Axial force diagram; (b) Bending moment diagram
## D.3. Capacities of the North Column

### Table D.1. Capacities of the North Column for Frame Tests 1&2

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| **Table D.2. Capacities of the North Column for Frame Test 3**

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D.4. ADDITIONAL INFORMATION FOR SYSTEM RELIABILITY ANALYSIS

D.4.1. Statistical data for material and fabrication factors

D.4.1.1. Statistical parameters for material factor

The material factor (Material case 1) was determined from data obtained by actual coupon test results, which is provided in Table D.7, and data collected from [21], which is shown in Table D.8.

Table D.7. Material data obtained from coupon tests conducted in this study

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Appendix D

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<td>C40030_LT_W2_(1)</td>
<td>213 MPa</td>
<td>200 MPa</td>
<td>1.07</td>
<td>68215 MPa</td>
<td>70000 MPa</td>
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<td>C40030_LT_W3_(1)</td>
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<td>1.08</td>
<td>68803 MPa</td>
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<td>0.98</td>
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<td>C40030_LT_W4_(1)</td>
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<td>70230 MPa</td>
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</tr>
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<td>1.09</td>
<td>69449 MPa</td>
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Table D.9. Statistical parameters for material factor

<table>
<thead>
<tr>
<th>Material parameters</th>
<th>Mean-to-nominal ratio</th>
<th>COV</th>
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</thead>
<tbody>
<tr>
<td>Yield stress ($f_y$)</td>
<td>1.093</td>
<td>0.042</td>
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<tr>
<td>Elastic modulus ($E$)</td>
<td>0.998</td>
<td>0.022</td>
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D.4.1.2. Statistical parameters for fabrication factor

In this study, the statistical parameters for fabrication factor were determined based on the measurements of the sectional dimensions of the columns and rafters in Frame Tests and the measurements of the thicknesses in coupon tests. The measured thickness data is shown in Table D.10, whilst the sectional dimension data can be found in Table A.2 to Table A.8.

The statistical parameters for the fabrication factor were determined and are presented in Table D.11.
Table D.10. Measurement of thicknesses for coupon tests

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Measurement location</th>
<th>Thickness (mm)</th>
<th>t&lt;sub&gt;measured&lt;/sub&gt;</th>
<th>t&lt;sub&gt;nominal&lt;/sub&gt;</th>
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<tbody>
<tr>
<td>Column AC25025</td>
<td>Web</td>
<td>2.43</td>
<td>2.50</td>
<td>0.971</td>
</tr>
<tr>
<td>Column AC25025</td>
<td>Web</td>
<td>2.43</td>
<td>2.50</td>
<td>0.971</td>
</tr>
<tr>
<td>Column AC25025</td>
<td>Web</td>
<td>2.43</td>
<td>2.50</td>
<td>0.972</td>
</tr>
<tr>
<td>Column AC25025</td>
<td>Web</td>
<td>2.43</td>
<td>2.50</td>
<td>0.972</td>
</tr>
<tr>
<td>Column AC25025</td>
<td>Flange</td>
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<td>2.50</td>
<td>0.972</td>
</tr>
<tr>
<td>Column AC25025</td>
<td>Flange</td>
<td>2.43</td>
<td>2.50</td>
<td>0.971</td>
</tr>
<tr>
<td>Column AC25025</td>
<td>Flange</td>
<td>2.43</td>
<td>2.50</td>
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</tr>
<tr>
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<td>Flange</td>
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<td>2.50</td>
<td>0.972</td>
</tr>
<tr>
<td>Column AC25025</td>
<td>Lips</td>
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<td>2.50</td>
<td>0.971</td>
</tr>
<tr>
<td>Column AC25025</td>
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<td>2.43</td>
<td>2.50</td>
<td>0.972</td>
</tr>
<tr>
<td>Column AC25025</td>
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<td>2.50</td>
<td>0.969</td>
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<tr>
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</tr>
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<td>2.50</td>
<td>0.972</td>
</tr>
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<td>2.50</td>
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</tr>
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</tr>
<tr>
<td>Rafter AC25025</td>
<td>Lips</td>
<td>2.44</td>
<td>2.50</td>
<td>0.976</td>
</tr>
<tr>
<td>Rafter AC25025</td>
<td>Lips</td>
<td>2.43</td>
<td>2.50</td>
<td>0.972</td>
</tr>
<tr>
<td>Bracket</td>
<td>Apex</td>
<td>5.82</td>
<td>6.00</td>
<td>0.971</td>
</tr>
<tr>
<td>Bracket</td>
<td>Apex</td>
<td>5.83</td>
<td>6.00</td>
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</tr>
<tr>
<td>Bracket</td>
<td>Apex</td>
<td>5.81</td>
<td>6.00</td>
<td>0.968</td>
</tr>
<tr>
<td>Bracket</td>
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<td>6.00</td>
<td>0.969</td>
</tr>
<tr>
<td>Bracket</td>
<td>Knee</td>
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<td>6.00</td>
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</tr>
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<td>6.00</td>
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<tr>
<td>Bracket</td>
<td>Knee</td>
<td>5.82</td>
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<td>Bracket</td>
<td>KneeBrace</td>
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<td>0.972</td>
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<tr>
<td>Bracket</td>
<td>KneeBrace</td>
<td>5.82</td>
<td>6.00</td>
<td>0.969</td>
</tr>
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</table>

Table D.11. Statistical parameters for fabrication factor

<table>
<thead>
<tr>
<th>Dominant parameters</th>
<th>Mean-to-nominal ratio</th>
<th>COV</th>
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</thead>
<tbody>
<tr>
<td>Thickness</td>
<td>0.977</td>
<td>0.005</td>
</tr>
<tr>
<td>All sectional dimensions</td>
<td>0.980</td>
<td>0.069</td>
</tr>
</tbody>
</table>
D.4.2. Matlab script for the system reliability analysis

% Hoai Cuong Nguyen
% This program determines the reliability index for a linear limit state %
% function with non-normal variables
% Notes: Resistance follows a lognormal distribution
% Dead load follows a normal distribution
% Live load follows Extreme Type I distribution
% This function was developed in MATLAB R2017a
% FIRST ORDER SECOND MOMENT
clc, clearvars
format long
%------------------- DEFINE INPUT---------------------------------------------

% Parameters for resistance distribution
mR_Rn = 1.045;  % mR/Rn (mean R divided by nominal R)
VR = 0.044;    % Coefficient of variation of R

% Parameters for dead load distribution
mD_Dn = 1.05;  % mD/Dn (mean dead load divided by nominal dead load)
VD = 0.1;      % Coefficient of variation of dead load

% Parameters for live load distribution
mL_Ln = 0.8;   % mL/Ln (mean live load divided by nominal live load)
VL = 0.25;     % Coefficient of variation for live load

% Load factor (according to AS/NZS 1170)
gammaDead = 1.2; % dead load factor
gammaDeadSmall = 1.35; % dead load factor (for Ln/Dn<0.1)
gammaLive = 1.5; % live load factor

% Ratios Live load/Dead load to study
k = [0.25 0.5 1.5 2 3 5];  % Ratio of Ln/Dn

% System resistance factors to analyse
f = [0.6 0.625 0.65 0.675 0.7 0.725 0.75 0.775 0.8 0.825 0.85 0.875 0.9
0.925 0.95 0.975 1.0];  % Fi - factors

% Set mean of dead load equal 1
mD = 1.0;  % Mean of dead load set to 1
sD = VD*mD; % standard deviation

% Create a report file for reliability results
fout = fopen('AL Frames ReliabilityGravity_Case1.txt','w');
% End of user input
fprintf(fout,'Reliability analysis for Case 1 of Gravity Load 3a\n');

%------------------- FIRST ORDER SECOND MOMENT ITERATION---------------------
for j=1:length(k) % range of Ln/Dn ratios
t = k(j);
fprintf(fout,'Results for L/D=%.3g\n',t);
for i=1:length(f) % range of system capacity factors
%------------------- 1st STEP---------------------------------------------

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% Define limit state function and variables
% g=R-D-L
% Establish relations based on design equation and variables
ml = mL_Ln/mD_Dn*k(j)*mD; % Mean of live load
sl = VL*mL; % Std. of live load
% Mean of resistance
if k(j)<0.1 % Dictated by Australian combination fRn > 1.35 Dn
    mR =((gammaDeadSmall)*mD/mD_Dn)*(mR_Rn/f(i));
else
    mR =((gammaDead+gammaLive*k(j))*mD/mD_Dn)*(mR_Rn/f(i));
end
sR = VR*mR; % Std. of resistance

%----------------------------------- 2nd STEP -------------------------------------
% Obtain initial design point by assuming values for n-1 of the random
% variables X*i (Mean value are often a reasonable choice)
r = mR;
l = mL;
d = r-l;
% Create the iteration stopping criteria
b = 10;
db = 10000;
dx = 10;
while db>0.000001 && dx>0.00001
%----------------------------------- 3rd STEP -------------------------------------
% Determine equivalent normal mean and standard deviation for initial
design points
sRe = r*sqrt(log((sR/mR)^2+1)); % Equivalent standard deviation of R
mRe = r*(1-log(r)+(log(mR)-(1/2)*log((sR/mR)^2+1))); % Equivalent mean of R
% Statistical parameters of extreme type I distribution for live load
a = sqrt(pi^2/(6*sL^2));
u = mL-0.5772/a;
FL = exp(-exp(-a*(l-u)));
fL = a*exp(-exp(-a*(l-u)))*exp(-a*(l-u)); % Probability density function
sLe = (1/fL)*normpdf(norminv(FL)); % Equivalent standard deviation for live load
mLe = l-sLe*norminv(FL); % Equivalent mean for live load

%----------------------------------- 4th STEP -------------------------------------
% Determine the values of the reduced variables
zh = (r-mRe)/sRe;
zd = (d-mD)/sD;
zl = (l-mLe)/sLe;
Z = [zh;zd;zl];
%----------------------------------- 5th STEP -------------------------------------
% Determine the partial derivatives of the limit state function with
% respect to the reduced variables. dg/dZi = dg/dxi*sXi
Gr = -sRe;
Gd = sD;
Gl = sLe;
G = [Gr;Gd;Gl];

%----------------------------------- 6th STEP -------------------------------------
% Estimate reliability index, b=(G^T.Z)/(G^T.G)^(1/2)
b_new =(G.'*Z)/sqrt(G.'*G); % for uncorrelated random variable;
% If correlated random variable then insert rho

%----------------------------------- 7th STEP -------------------------------------
% Determine sensitivity factor
a = G/sqrt(G.'*G);

%------------------------------- 8th STEP -------------------------------
% Obtain new reduced design points
zr = a(1,1)*b_new;
zd = a(2,1)*b_new;
zl = a(3,1)*b_new;

%------------------------------- 9th STEP -------------------------------
% Update design points in original coordinates
r_new = mRe+sRe*zr;
l_new = mLe+sLe*zl;
d_new = r_new-l_new;

% Stopping criterion for the loop
db = abs(b-b_new);
dx = max([abs(r-r_new) abs(d-d_new) abs(l-l_new)]); % update values
r = r_new;
d = d_new;
l = l_new;
b = b_new;
end % loop for iteration

% Print the result of simulations
A(:,j)=[k(j) f(i) b];
fprintf(fout,'f-factor,reliability index target = %.3f,%.4f\n',f(i),b);
results = sprintf('L_B %.3f',k(j));
xlswrite('Reliability_analysis_Gravity_Case1_1.xlsx',A,results);
end% end of system capacity factors

target =[2 2.25 2.5 2.75 3 3.25 3.5 3.75 4];
Q = interp1(A(:,3),A(:,2),target);
B=[target' Q'];
xlswrite('Reliability_analysis_Gravity_Case1_2.xlsx',B,results);
end% end of live load to dead load ratios

fprintf(fout,'End of report');
fclose(fout);
% fout=fclose('AL Frames Reliability.txt','w');
D.4.3. Additional plots

Figure D.14. Live load to dead load ratio versus system resistance factors for Case 1 – FORM

Figure D.15. Live load to dead load ratio versus system resistance factors for Case 2 – FORM
Figure D.16. Live load to dead load ratio versus system resistance factors for Case 3 – FORM

Figure D.17. Live load to dead load ratio versus system resistance factors for Case 4 – FORM
End of Appendices