Chapter 1

INTRODUCTION

1.1 BACKGROUND

There has been increased use of hollow structural sections (HSS) in recent years. In particular, rectangular hollow sections (RHS), square hollow sections (SHS), and circular hollow sections (CHS) are used for their aesthetic appeal. Modern architecture in Australia is dominated by tubular structures (eg Stadium Australia, Sydney Football Stadium, and Darling Harbour - all in Sydney, Australia). However, the market for steel extends well beyond such famous examples.

While the cost of cold-formed hollow sections has been decreasing with advances in manufacturing technology, the cost per tonne is still greater than that of hot-rolled open steel products (eg I-beams). Hot-rolled open sections are more commonly used in industrial structures, such as warehouses, garages and farm sheds.

Despite costing more per tonne than hot-rolled sections, hollow sections have properties (some non-structural) which make them suitable for use in portal frame structures:

- High torsional rigidity reduces the need for lateral bracing
- Less surface area per length of member reduces painting and fire resistance costs
- There is less possibility of unhealthy deposits accumulating within the structure in agricultural applications (eg feathers in a chicken coop, or dust in a grain facility).

Plastic design is a method of design that can result in more economical structures compared to structures designed by traditional elastic methods. Plastic design can provide the most benefit to structures that resist load primarily by bending, and where deflections due to dead load are not a significant design consideration. Portal frame structures (warehouses, garages and farm sheds) can therefore benefit from plastic design.
Plastic design methods were verified experimentally by tests on hot-rolled I-sections only (Chapter 2), and consequently some steel design standards do not permit plastic design in cold-formed hollow sections. In particular, *cold-formed* hollow sections have material properties which are considerably different from hot-rolled sections, and fail to satisfy the *material ductility* requirements for plastic design in some steel design standards.

There has been a significant research program at The University of Sydney since the 1980s into the behaviour of cold-formed RHS (Section 2.8). The aim of the research has been to develop appropriate design rules for cold-formed SHS and RHS for use in the limit states steel design specification in Australia, AS 4100 (Standards Australia 1990a). Much of AS 4100 is now applicable to cold-formed RHS, and one of the few remaining omissions is the use of cold-formed RHS in plastic design. The main aim of this thesis is to investigate the suitability of cold-formed RHS for use in plastic design.
1.2 AIM OF THE THESIS

The primary aim of the thesis is to investigate the suitability of cold-formed rectangular hollow sections for use in plastic design.

The main aim can be subdivided into further sub-sections:

1. To examine experimentally the element slenderness limits of cold-formed RHS for use in plastic design, particularly the web slenderness limit, and any interaction between the web and the flange, by means of simple beam tests.

2. To assess experimentally the suitability for plastic hinge formation of various connection types for the beam-rafter knee joint in cold-formed RHS portal frames, and suggest alternative connection designs.

3. To study experimentally the behaviour of cold-formed RHS portal frames under simulated gravity and combined simulated gravity and wind loading conditions, specifically the ability of the frames to form a plastic collapse mechanism and the rotation capacity required to achieve the plastic collapse mechanism.

4. To simulate the behaviour of the RHS beam tests described in (1) via finite element analysis, and extend the finite element analysis to consider the effects of imperfections, varying material properties, and specimen dimensions not considered in the experimental program.

5. To simulate the behaviour of the portal frame tests described in (3) via various types of structural analysis.

6. To review existing design recommendations and standards and compare the relevant clauses with results obtained from (1) - (5) above and suggest improvements.
1.3 OUTLINE OF THE THESIS

1.3.1 GENERAL

The main body of the thesis contains the introduction, a review of previous research, the experimental and numerical (finite element) investigations, recommendations, and conclusions. There are several appendices which include more comprehensive results.

1.3.2 EXPERIMENTAL INVESTIGATIONS

This majority of the work described in this thesis is experimental, and three series of tests were carried out.

Chapter 3 describes a series of bending tests on a variety of cold-formed RHS. Forty four RHS beams were tested under four point loading to examine their ability to form plastic hinges. Two steel grades were used, Grade C350 (350 MPa nominal yield), and Grade C450 (450 MPa nominal yield). The aim of the experiments was to establish a relationship between the rotation capacity, and web and flange slenderness of the section. A simplified representation of the test apparatus is shown in Figure 1.1

![Test Setup for Bending Tests](image-url)
Chapter 4 details a set of experiments on knee joints constructed from cold-formed RHS, using the testing rig shown in Figure 1.2. A knee joint is the column to rafter connection in a portal frame. Different types of joints were used to examine the ability of the joints to deform plastically, primarily under either opening or closing bending moment, and a small amount of axial force. Connections constructed from Grade C350 and Grade C450 steel were considered.

Figure 1.2: Test Setup for Connection Tests
Tests of three portal frames are described in Chapter 5. The portal frames were constructed from cold-formed RHS and were large scale, being seven metres wide, and four metres high at the apex, as shown in Figure 1.3. Three frames were tested under downwards vertical and a transverse load. Frame 1 was constructed from Grade C350 steel, while Frames 2 and 3 were constructed from Grade C450 RHS. The experimental behaviour is compared to several different forms of structural analysis.

Figure 1.3: Test Setup for Portal Frame Tests
1.3.3 NUMERICAL (FINITE ELEMENT) ANALYSIS

Chapter 6 describes finite element analyses simulating the RHS beam tests in Chapter 3. The finite element analyses were performed using the commercial program, ABAQUS. The stages in the development of the model are described. The main aim of the finite element analysis was to establish significant trends in the behaviour of cold-formed RHS beams, which could be used to help extrapolate and interpolate the experimental results. Part of a typical finite element mesh is shown in Figure 1.4.

![Figure 1.4: Part of a Typical Finite Element Model](image-url)
1.3.4 COMPARISON WITH DESIGN SPECIFICATIONS

Many of the test results are compared with the appropriate clauses in some steel design specifications. The standards most commonly considered in this thesis are:

- Australia: Australian Standard AS 4100 (Standards Australia 1990a),
- Europe: Eurocode 3 (European Committee for Standardisation 1992), and

The 1994 metric edition of the AISC LRFD is usually quoted in this thesis, though it is identical to the 1993 edition (AISC 1993), which is in non-metric units. There has also been a supplement to the AISC LRFD relevant to hollow sections (AISC 1997).

Other design codes which are referred to in this thesis are:

- Great Britain: BS 5950 (British Standards Institution 1990),
- Canada: CAN/CSA-S16.1 (Canadian Standards Association 1994), and

Previous editions, amendments, commentaries and supplements to the aforementioned steel design specifications are also referenced in this thesis.
Chapter 2

INTRODUCTION TO PLASTIC DESIGN, LITERATURE REVIEW, AND CURRENT DESIGN STANDARDS

2.0 CHAPTER SYNOPSIS

The following chapter provides an introduction to plastic design of steel frames, and examines the relevant previous research in plastic design, including aspects such as local buckling, slenderness limits, and behaviour of knee joints in portal frames.

In particular, this chapter investigates the development of the element slenderness \((b/t)\) limits in various steel design specifications. There are inconsistencies in the development of the Class 1 slenderness limit for webs in the AISC LRFD Specification.

The limits for webs in bending have been based on investigations of I-sections, but are applied to both I-sections and RHS. While many researchers have indicated that there is interaction between the flange and web of I-sections, which affects local buckling and rotation capacity, all current steel design codes prescribe flange and web slenderness limits which are independent of each other.
2.1 COLD-FORMED HOLLOW SECTIONS

Figure 2.1 shows the cross section of a typical RHS and defines the dimensions \( d, b, t, \) and \( r_e \) (depth, width, thickness, and external corner radius) of the section. In this thesis the term RHS is used generically, and can represent either a specific rectangular section (where \( b \neq d \)) or a square hollow section (where \( b = d \)). The term SHS specifically refers to a square section \((b = d)\). The term “flange” refers to the smaller side of the RHS, and the term “web” refers to the larger side. Whenever an RHS bends about its major principal axis, the web is the element which experiences a strain gradient from tension to compression.

Cold-formed hollow sections are produced from rolls of thin steel strip and formed into the finished shape by passing through a series of rollers. Figure 2.2 illustrates the forming process.
Considerable work is applied to the strip steel to form the shape, resulting in a change of the mechanical properties of the finished product, compared to the original strip steel. For example, in Australia, BHP Steel produces cold-formed RHS from strip which has a nominal yield stress of 300 MPa. After forming, the RHS have a yield stress of at least 350 MPa. BHP Steel use a proprietary in-line galvanising process which results in RHS with a yield stress of 450 MPa after cold-forming and galvanising.

Three typical stress-strain curves are shown in Figure 2.3. Curve (a) illustrates the behaviour of mild (hot-rolled) steel. Mild steel is characterised by an initial linear elastic region, and a rapid transition to a plastic plateau. At high strains, strain hardening occurs. The stress-strain curve for a cold-formed steel is shown in curve (b) of Figure 2.3. There is a rounded “knee” after the elastic region, and since there is no well defined yield stress, the 0.2% proof stress is commonly used to define a value of yield stress. Typically cold-formed steels have no (or a small) plastic plateau, and strain hardening commences immediately after yielding. The fracture strain is usually less than that of a mild steel, and generally decreases with increasing yield stress. The yield stress of cold-formed steel increases with the amount of cold-working performed to create the section shape. Curve (c) shows the stress-strain curve from a corner of an RHS. The corner of an RHS has a higher yield stress and a lower $f_y/f_{y0}$ ratio than the flat faces due to the extra cold working in the formation of the tight corner radii.
2.2 THE BASICS OF PLASTIC DESIGN

2.2.1 BENDING BEHAVIOUR OF STRUCTURAL STEEIBEAMS AND THE PLASTIC HINGE

When a transverse load ($P$) is applied to a steel beam, as shown in Figure 2.4, there is corresponding deformation, which includes curvature ($\kappa$), induced in the beam. Internal forces, such as bending moments ($M$), occur within the beam.

Figure 2.4: Beam Under Transverse Load
Consider the RHS in bending, the cross-section of which is shown in Figure 2.5. According to simple engineering bending theory, the distribution of strain across the section is linear regardless of the stress state, and the value of the strain at the extreme fibres is proportional to the curvature. Figure 2.5 indicates how the stress distribution changes with increasing levels of curvature for an RHS with either the idealised elastic - plastic - strain hardening material properties, or the curved stress - strain behaviour of cold-formed steel. Initially, in the elastic range, the stress distribution is linear. As the curvature increases, the extreme fibres reach the yield stress at the yield moment \((M_y)\), where \(M_y = f_y Z\) and \(Z\) is the elastic section modulus. At larger curvatures and strains, yielding spreads inwards toward the neutral axis. For the elastic - plastic - strain hardening material, the section yields almost completely and is fully plastic at high values of curvature (theoretically full plasticity can only occur at infinite curvature). The moment at which full yielding occurs is termed the plastic moment \((M_p)\), where \(M_p = f_y S\) and \(S\) is the plastic section modulus. Strain hardening is initiated at even higher curvatures, and the stress can exceed the yield stress. In the case of cold-formed RHS, there is no plastic plateau as strain hardening occurs immediately after yielding, and the stress increases beyond \(f_y\) at lower values of curvature compared to the idealised case.

The resulting idealised moment-curvature relationships of the cross-section are shown in Figure 2.6. For the idealised case, the curve includes a linear range and a transition from the yield moment to the plastic moment. Once the cross-section is fully plastic, increases in curvature can occur without a corresponding moment increase. Not only does the moment reach \(M_p\) but the beam maintains \(M_p\) as the curvature increases. The increasing curvature at constant moment \(M_p\) is termed a plastic hinge and demonstrates the ductility of the steel beam. The moment may rise above the plastic moment due to strain hardening, but the increase in moment is sometimes ignored. The behaviour can be idealised as “rigid plastic”, in which no deformation occurs until the plastic moment is reached. For an RHS with the realistic rounded stress-strain curve in Figure 2.3(b), yielding occurs before the yield moment is reached due to residual stresses and the rounded stress - strain curve. The moment rises above the plastic moment due to the lack of a plastic plateau and early strain hardening. The rigid-plastic assumption is an approximation of the true behaviour of an RHS beam.
Figure 2.5: Stress and Strain Distribution in a Beam Section with Increasing Curvature
(adapted from Rasmussen, Clarke and Hancock (1997))

Figure 2.6: Moment-Curvature Behaviour of an Idealised and a Cold-Formed RHS Beam
2.2.2 ROTATION CAPACITY AND CLASSIFICATION OF SECTIONS

A steel beam cannot sustain infinite curvature, and at some curvature failure occurs. The most common mode of failure is local instability (buckling) of the plate elements in the section, although material fracture is another possible failure mode. It is assumed that there is adequate lateral restraint to ensure that no failure occurs due to (out-of-plane) lateral buckling.

Some beams may fail before reaching the yield moment or the plastic moment. If the beam can reach the plastic moment, the rotation capacity \( R \) is a measure of how much the plastic hinge can rotate before failure occurs. To calculate \( R \), the moment-curvature graph is normalised with respect to the plastic moment and plastic curvature \( (\kappa_p = M_p/EI) \), (where \( E \) is Young’s modulus of elasticity, or elastic modulus, and \( I \) is the second moment of area of the section). Such a non-dimensional curve is given in Figure 2.7. Assuming buckling occurs after the moment increases above \( M_p \), then the moment drops below \( M_p \) at some curvature \( (\kappa_1) \). The rotation capacity is commonly defined as:

\[
R = \frac{\kappa_1}{\kappa_p} - 1 \tag{2.1}
\]

Figure 2.7: Definition of Rotation Capacity
In some cases, the curvature at which the moment starts to decrease, rather than the curvature at which the moment passes back through $M_p$ is used to define the rotation capacity.

Sections are classified into groups depending on their behaviour under bending, (their rotation capacity and maximum moment, $M_{max}$), as illustrated in Figure 2.8. Some design standards use four classes of sections, while other steel standards use only three separate classes.

Figure 2.8: Moment-Curvature Behaviour of Different Types of Steel Sections

(Eurocode 3 classification)
1) Class 1 sections can attain the plastic moment and have plastic rotation capacity sufficient for plastic design. Such sections are sometimes referred to as plastic sections (BS 5950), or compact sections (AS 4100, AISC LRFD). Section 2.2.5 examines the amount of rotation deemed to be “sufficient”.

2) Class 2 sections can develop the plastic moment but have limited rotation capacity and are considered unsuitable for plastic hinge formation. Class 2 sections may be known as compact sections (CSA-S16.1) or compact elastic (Galambos 1976) sections. Confusion may arise with the dual use of the term “compact” for Class 1 in AISC LRFD and AS 4100, and Class 2 in CSA-S16.1.

3) Class 3 sections can reach the yield moment, but cannot reach the plastic moment due to local buckling. Such sections are sometimes called semi-compact (BS 5950), or non-compact (CSA-S16.1).

4) Class 4 sections cannot reach the yield moment due to local buckling. They are also known as slender sections in all standards.

Some specifications, such as AS 4100 and AISC LRFD, group together Class 2 and Class 3 sections, into one single class, commonly referred to as non-compact. Under the AS 4100 and AISC LRFD definition, “non-compact” sections have a moment capacity exceeding the yield moment, and up to and including the plastic moment, but cannot sustain the plastic moment for suitably large rotations. The moment capacity for such sections varies linearly with slenderness from the yield moment to the plastic moment.

To avoid confusion in this thesis, the generic terms “Class 1”, “Class 2”, “Class 3”, and “Class 4” will be used when discussing the different classes of sections. If a specific design standard is being discussed, the terms used in that design standard will be used. Table 2.1 summarises the terminology in some of the more widely known steel design codes.
<table>
<thead>
<tr>
<th>Specification</th>
<th>Class 1</th>
<th>Class 2</th>
<th>Class 3</th>
<th>Class 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eurocode 3</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BS 5950</td>
<td>Plastic</td>
<td>Compact</td>
<td>Semi-Compact</td>
<td>Slender</td>
</tr>
<tr>
<td>CSA S16.1</td>
<td>Plastic, Compact, Non-Compact, Slender, or Class 1 or Class 2 or Class 3 or Class 4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>AS 4100</td>
<td>Compact</td>
<td>Non-Compact</td>
<td></td>
<td></td>
</tr>
<tr>
<td>AISC LRFD</td>
<td>Compact</td>
<td>Non-Compact</td>
<td>Slender</td>
<td></td>
</tr>
</tbody>
</table>

Table 2.1: Section Classification in Various Design Standards

2.2.3 THE BEHAVIOUR OF A SIMPLY SUPPORTED BEAM

Consider a simply supported beam of length $L$, subject to a point load, $P$, at the centre, as shown in Figure 2.9 (adapted from Rasmussen, Clarke and Hancock 1997). The bending moment distribution is triangular, and is not affected by yielding of the steel beam. As the load increases, the stress reaches the yield stress at the most stressed point. Further increase of the load causes yielding to spread through the depth and along the length of the beam until the plastic moment is reached at a load of $P_u = 4M_p/L$. A plastic hinge has now been created and no extra load can be sustained. A mechanism has formed, since there are the two hinges (pins) at the supports and the plastic hinge at the centre. A mechanism is formed when an increase in deflection can occur in any part of the structure, without the addition of any extra load.

At collapse, most of the beam remains elastic and hence has small curvatures. The shaded zone in Figure 2.9 represents the yielded area and large curvatures occur in the shaded region. If the curvature in the elastic parts of the beam is ignored, it is an example of the rigid-plastic concept, whereby all the curvature and rotation occur at the plastic hinge. The curvature is assumed to be infinite at the plastic hinge where discontinuities occur in the beam rotation.
Elastic Rigid

\[ P_u = 4M_p/L \]

\[ M_p = P_u L/4 \]

Bending moment

Elastic-plastic

Curvature

Deflections

(a) Real beam

(b) Rigid-plastic beam

Figure 2.9: Plastic Hinge Forming in a Simply Supported Beam
(adapted from Rasmussen, Clarke and Hancock 1997)
2.2.4 BASIC PLASTIC ANALYSIS

2.2.4.1 Built-In Beam

A statically indeterminate beam does not necessarily fail when a single plastic hinge forms. Load is transferred to other less highly stressed parts of the structure. Additional load is carried, and the first (and any subsequent) hinges maintain the plastic moment and undergo rotation. A collapse mechanism forms (for the entire structure or part of the structure) when there is a sufficient number of plastic hinges. The process of transferring load to other parts of the structure is known as moment redistribution.

An example of moment redistribution in a beam is shown in Figure 2.10 (adapted from Hasan and Hancock 1988). The first plastic hinge forms at point A at load $P = 6.75M_p/L$. A plastic collapse mechanism has not yet been formed, due to the built-in supports at the ends of the beam. Additional load is redistributed until the plastic moment is reached at C, at a load of $P = 8.68M_p/L$. The hinge at A must rotate through $\theta_A = M_pL/14EI$ to redistribute the moment. Further load is carried and redistributed until a hinge forms at C, when $P = 9M_p/L$. There are now hinges at A, B and C, a plastic collapse mechanism forms, and no further load can be carried. The ultimate load, $P_u$, is 33% higher than the load at the formation of the first hinge. Hence plastic design can result in higher ultimate strengths (and deformations) of structures. The true distribution of curvature at collapse is shown in the last diagram of Figure 2.10, where large plastic curvatures are present at the hinge points.

At the formation of the final hinge at B, the rotation at the first hinge is $\theta_A = M_pL/6EI$. At point A, the beam must be able to sustain the plastic moment for the required rotation, or moment redistribution cannot take place. If the beam is unable to achieve the necessary rotation at A, the beam is unsuitable for plastic design.
Elastic moment distribution

Hinge forms at A
\[ P = 6.75 \frac{M_p}{L} \]
\[ \theta_A = 0 \]

Hinge forms at C
\[ P = 8.68 \frac{M_p}{L} \]
\[ \theta_A = \frac{M_p L}{144EI} \]

Hinge forms at B
Plastic collapse mechanism formed
\[ P_u = 9 \frac{M_p}{L} \]
\[ \theta_A = \frac{M_p L}{6EI} \]

Plastic collapse mechanism

True curvature distribution at collapse

Figure 2.10: Formation of Hinges in a Built-In Beam
(adapted from Hasan and Hancock (1988))
2.2.4.2 Simple Portal Frame

Plastic behaviour can be extended from the beam in Section 2.2.4.1 to a more complicated structure, such as the fixed-base rectangular portal frame in Figure 2.11. The example in Figure 2.11 is a modified version of a frame shown in Baker, Horne and Heyman (1956). Figure 2.11 shows the point loads on the frame and the formation of the plastic hinges. The beams and columns are made from the same section, a $150 \times 50 \times 4.0$ RHS, yield stress of 450 MPa, and plastic moment, $M_p = 29.4 \text{ kNm}$. The dimension $h = 3 \text{ m}$.

![Diagram of Plastic Hinges in a Simple Portal Frame](image)

The first plastic hinge forms when $P = 23.8 \text{ kN}$, which would be the maximum load allowed under elastic design. The plastic collapse mechanism forms at the creation of the fourth plastic hinge, when $P = 3M_p/h = 29.4 \text{ kN}$. Thus plastic design provides a 23% increase in load capacity for the case shown compared to elastic “first hinge” design.

It is not the intention of this thesis to give a detailed description of plastic analysis methods. The previous two examples gave a brief illustration of plastic analysis. There are many texts available on plastic methods of analysis. One of the most well-known is *The Steel Skeleton* (Baker, Horne and Heyman 1956).
2.2.5 ROTATION REQUIREMENTS

The example of a built-in beam in Section 2.2.4.1 has shown that plastic hinges must be able to rotate a certain amount to redistribute the bending moment and eventually form a plastic collapse mechanism. The required hinge rotation depends on the nature of the loading, the properties of the member, and the size of the frame or beam. Note that hinge rotation is a dimensionless quantity, and refers to the rotation in radians.

Kerfoot (1965) analysed three span beams with point loadings. It was found that only in extreme cases would a plastic hinge rotation greater than $\frac{ML}{EI}$ be needed to form a plastic collapse mechanism, where $L$ is the length of each span.

Driscoll (1958) considered three span beams (each span of length $L$) with distributed loads. It was shown that a plastic hinge rotation of $0.425 \frac{ML}{EI}$ was the maximum required for the three span beam. Driscoll extended the analysis to frames. For a single span rigid frame $0.475 \frac{ML}{EI}$ was the required plastic hinge rotation, and for gable frames (pitched roof) $1.05 \frac{ML}{EI}$ was the maximum required plastic hinge rotation.

For a highly redundant structure, where many hinges are required to form, the load-deflection response asymptotes towards the maximum load with large deflections. Large rotations are required from some of the hinges to create only small increases in the capacity of the structure. For example, Driscoll (1958) analysed a two bay pitched roof portal frame with gravity and wind loads. At the ultimate load and formation of the plastic collapse mechanism, the first hinge had to rotate $1.52 \frac{ML}{EI}$. At 98% of the ultimate load, the first hinge had rotated $0.54 \frac{ML}{EI}$. In design situations, achieving slightly under the calculated maximum load is acceptable, and therefore the practical rotation capacity requirements can be less than the very large theoretical values of rotation calculated in some highly redundant frames.

The required hinge rotation is the angle through which an idealised plastic hinge must rotate. In reality, there is a plastic zone of high curvature which acts like a hinge (refer to Figure 2.9). Some analysis is needed to convert the hinge rotations (an angle) into a required rotation capacity (essentially a measure of curvature). The conversion depends on many factors such as moment gradient, length of the yielded zone, and the shape factor ($S = \frac{M_p}{M_y}$) of the section.
Since the rotation capacity requirements vary according to the loading and geometry of a structure, it is convenient to establish a representative value of rotation capacity that covers most common practical situations.

The Eurocode 3 Editorial Group (1989) summarise the rotation requirements for a variety of frames and multi-span beams constructed from I-sections and concluded that a value of $R = 3$ was a suitable value to ensure that a plastic collapse mechanism could form. The value of $R = 3$ is used in Eurocode 3.

Yura, Galambos and Ravindra (1978) state that the AISC LRFD specification is based on $R = 3$. AISC LRFD mentions [Commentary to Clause B.5] that in seismic regions greater rotation capacity may be required that the rotation provided by a compact section, and rotation capacities of the order $R = 7 \sim 9$ (Chopra and Newmark 1980) need to be provided.

Korol and Hudoba (1972) considered hollow sections and recommended that a value of $R = 4$ was the minimum necessary to ensure that a mechanism could form. Hasan and Hancock (1988) and Zhao and Hancock (1991) followed the suggestion of Korol and Hudoba, and used the value of $R = 4$, for determining suitability for plastic design in the establishment of values for the Australian Standard AS 4100.

Stranghöner, Sedlacek and Boeraeve (1994) investigated the behaviour of RHS, SHS and CHS beams. Since the shape factor of hollow sections is different to I-sections, the rotation requirements calculated by the authors above might not be appropriate for hollow sections. It was found that $R = 3$ was an acceptable rotation capacity requirement for continuous beams.

To utilise plastic design, the rotation capacity of the member must exceed the rotation requirement at the hinge for the given frame and distribution of loading. Not all members can achieve the required rotation.
2.3 RESEARCH INTO THE PLASTIC BEHAVIOUR OF STEEL

The initial methods of analysis and design of steel structures in the nineteenth century were based on the theory of elasticity. Hooke’s Law, which states that force is proportional to deformation (stress is proportional to strain), is the basis of elastic design.

The earliest tests on steel beams set out to confirm the elastic behaviour of beams. Some papers suggested that the beam had reached its limit at the yield moment, $M_y$, and the results verified $M_y$ as the limit. As Lay (1963) notes, early tests probably confirmed the elastic behaviour of beams due to two factors:

- Non-linear or inelastic behaviour, which would occur after $M_y$ was reached, was regarded as failure by the early researchers, who were investigating elastic behaviour only.
- There may have been insufficient bracing of the test specimens, and yielding often initiated lateral buckling, before the plastic moment could be reached.

Lyse and Godfrey (1934) considered that the first yield of a beam was an appropriate limit for strength design. Lyse and Godfrey wrote

“Since the usefulness of beams is determined by the maximum load it can contain without excessive deflection, the determination of its yield point becomes the most important factor in testing.... The ultimate load has little significance beyond the fact that it is a measure of the toughness of the beam after it has lost its usefulness.... The yield point strength of the beam was used as the criterion for its load-carrying capacity”

Ewing (1899) hypothesised that if the bending moment was increased beyond $M_y$, that:

“the outer layers of the beam are taking permanent set [yielding] while the inner layers are still following Hooke’s Law... and any small addition to the stress produces a relatively very large amount of strain”.

Ewing suggested that the stress distribution across the section would be of the form shown in Figure 2.12. It is not known whether Ewing confirmed his hypotheses by any tests on steel beams.
Probably the earliest recorded instances of the plastic behaviour of beams were the experiments of Meyer (1908). Meyer tested simply supported beams of rectangular cross section, with single point loads. The deflection increased dramatically as $M_p$ was reached.

Kazinczy (1914) may have been the first to suggest the development of a plastic hinge.

The most famous of the early researchers on plastic behaviour of beams was Maier-Leibnitz (1936). He performed tests on simple and continuous beams, and observed the ductile behaviour of steel beams. Some beams eventually failed by lateral buckling.

The next stage was the investigation of the plastic behaviour of complete structures. Baker and Roderick (1938, 1940) describe the series of experiments at the Civil Engineering Department of the University of Bristol, England between 1936 and 1939. Very small scale rectangular portal frames of span 510 mm and height 255 mm were constructed, from I-sections with a depth of only 32 mm and web thickness of approximately 3.2 mm.

Conclusions from the research at Bristol are summarised by Baker, Horne and Heyman (1956):

"... portals subjected to vertical loads had a great reserve of strength beyond the point at which yield was first developed, and that collapse, the growth of large uncontrolled deflections, did not occur until a mechanism had formed by the development of three plastic hinges... The agreement is good ... between the observed and calculated collapse loads of the portals... It was realised, of course, that it was a far cry from calculating the vertical loads which would cause collapse of a rectangular portal frame to deriving an acceptable method of designing redundant structures based on collapse, but ... incomplete though they [the results] were, they formed the basis of much wartime [World War 2] design.”
After World War 2, research into plastic design continued in Cambridge, England with the cooperation of the Steel Structures Research Committee, the British Constructional Steelwork Association, and the British Welding Research Association. There were test series on miniature rectangular (flat roofed) portal frames constructed from solid rectangular sections (Baker and Heyman 1950), and full-scale rectangular portal frames in I-sections (Baker and Roderick 1952). They were followed by tests on pitched roof I-section portals, either symmetrical (Baker and Eickhoff 1956a), or saw-toothed (non-symmetrical roofs) (Baker and Eickhoff 1956b, and Baker, Horne and Heyman 1956). These tests provided the experimental verification of the plastic design method for suitable hot-rolled I-sections. By the 1950s the plastic method of design was being accepted by the engineering community and there was a large number of published papers generated by the Cambridge research (Baker, Horne and Heyman (1956) provides a large list of references).

At Lehigh University in the United States, a comprehensive investigation of plastic design of structures constructed from I-sections was performed from the mid 1940s to the late 1960s. The research was sponsored by the Welding Research Council; the Department of the Navy; the American Institute of Steel Construction; the American Iron and Steel Institute; the Institute of Research, Lehigh University; Column Research Council; Office of Naval Research; Bureau of Ships; and the Bureau of Yards and Docks.

The research program included tests on beams; connections in frames; large scale multi-storey braced and unbraced frames; analysis of structures; slenderness limits for I-sections; material considerations; and, in particular, the development of design aids for use in plastic design. A summary of much of the research at Lehigh University can be found in Driscoll et al (1965) and Galambos (1968). The Lehigh research forms the basis of the plastic design rules found in many steel design specifications.
Some of the conclusions arising from the research project at Lehigh University were summarised by Driscoll (1966):

“1. The [plastic] method presented for the design of braced multi-story frames is successful. A saving of steel and design time is possible.

2. Plastic hinges will develop in high-strength steels... Proper proportions of members will assure adequate rotation capacity for the development of plastic mechanisms in structures.

3. Plastic design of unbraced multi-story frames is feasible... Less savings of steel may be expected than for a braced frame, and sway deflection can govern the design rather than strength considerations alone.”

There have been other test series on portal frame structures constructed from hot-rolled I-sections, such as Dowling et al (1982) and Shanmugan et al (1995).

The first cold-formed open sections produced in the 1960's were light gauge sections, which are not appropriate as the major structural members of portal frames, and were used as secondary elements such as purlins. As larger sections became available, portal frames could be constructed from the cold-formed open sections. Baigent and Hancock (1982) tested pitched roof frames constructed from cold-formed channels. Kirk (1986) constructed portal frames from a unique section known as a Swagebeam. Cold-formed hollow flange beams were used in the portal frames of Heldt and Mahendran (1995, 1998). Avery (1998) used non-compact cold-formed RHS to construct a portal frame. The members used by Avery were not suitable for plastic design.

The use of cold-formed sections in portal frames is accepted for elastic design, but plastic design of cold-formed RHS portal frames is not yet permitted in many modern steel design specifications. This thesis aims to investigate whether compact cold-formed RHS can be used in the plastic design of a portal frame.
2.4 LOCAL BUCKLING AND SLENDERNESS LIMITS

Most structural sections can be idealised as being comprised of individual flat plate elements. An RHS can be considered as four plates joined to form the hollow section. An I-section is constructed from a web plate and two flange plates (or a web plate and four half flange plates).

Plate elements are susceptible to local buckling. In plastic design, the plate elements are required to achieve substantial deformations once they have yielded before local buckling occurs, in order for moment redistribution to take place. To prevent premature local buckling, slenderness limits for the plate elements in members have been established.

2.4.1 ELASTIC LOCAL BUCKLING OF THIN RECTANGULAR PLATES

Consider a long plate of width $b$ and thickness $t$, with in plane stress $f_x$ acting on the plate, as shown in Figure 2.13. The plate in Figure 2.13 is simply supported on all four edges, but any type of edge restraint could be considered. The plate can buckle out-of-plane, with out-of-plane deflections denoted $w$.

![Figure 2.13: Local Buckling of a Rectangular Plate](image-url)
It can be shown that the differential equation for elastic local buckling of the plate is (Bryan 1891):

\[
\frac{E t^3}{12(1-\nu^2)} \left( \frac{\partial^4 w}{\partial x^4} + 2 \frac{\partial^4 w}{\partial x^2 \partial y^2} + \frac{\partial^4 w}{\partial y^4} \right) = -f_o t \frac{\partial^2 w}{\partial x^2} \tag{2.2}
\]

where \(\nu\) is Poisson’s ratio.

The solution for the elastic local buckling stress \(f_o\) is given by:

\[
f_o = \frac{k \pi^2 E}{12(1-\nu^2)(b/t)^2} = \frac{H^2}{(b/t)^2} \tag{2.3}
\]

where \(k\) is the plate buckling coefficient. The equation is simplified by substituting \(H^2 = k \pi^2 E/(12(1-\nu^2))\). The value of \(k\) depends on the nature of the stress distribution across the plate and the support conditions of the plate. Bryan (1891) solved the problem for the case of the plate being simply supported on all sides. Figure 2.14 solved the problem for the case of the plate being simply supported on all sides. Figure 2.14 shows three different combinations of plate support and stress distribution with the appropriate value of \(k\), and shows the type of plate element from a common structural section that each case represents. Under bending, the flange of an I-section can be thought of as a plate in compression with one longitudinal edge free; the flange of a box section or RHS may be considered as a plate in uniform compression simply supported on all sides; and, the web of an I-section, box section, or RHS, can be considered as a plate in bending, with all edges simply supported.
Stress distributions on plates

Plate thickness, \( t \) \( k = 0.425 \)

Stress distribution in bending

Plate elements in structural section

Figure 2.14: Stress Distributions on Plate Elements

To prevent a plate from buckling before it reaches its yield stress (ie to avoid elastic local buckling), then \( f_y \leq f_o \), or from Equation 2.3

\[
f_y \leq \frac{H^2}{(b/t)^2}
\]

(2.4)

which can be written in terms of a slenderness limit for \( b/t \) for some value \( H \) to prevent elastic local buckling:

\[
\frac{b}{t} \leq \frac{H}{\sqrt{f_y}} \quad \text{or} \quad \frac{b}{t} \sqrt{f_y} \leq H
\]

(2.5)
There are different methods of expressing a slenderness limit such as that shown in Equation 2.5. In the AISC LRFD Specification and Eurocode 3, the right hand side of the equation (the limit) is a function of the yield stress \( f_y \) and some constant, while the left hand side (the slenderness) is a geometric ratio of the width or depth to thickness only. Hence the limits change for different values of \( f_y \). AS 4100 specifies a limit (the right hand side) which is constant (ie independent of the yield stress), while the flange and web slenderness values on the left hand side include a term involving \( f_y \). There is no mathematical difference between the two approaches. For simplicity and consistency, this thesis adopts the AS 4100 method, in which the limit is independent of \( f_y \), but the slenderness values are a function of the dimensions and \( f_y \). Hence, in this thesis, the term including \( f_y \) in the limits given in AISC LRFD and Eurocode 3 have been included in the slenderness term, but not the value of the limit.

There are many other sources of information on elastic and inelastic local buckling of plates. Bleich (1952), Johnston (1976), Ostapenko (1983), Galambos (1968), and Timoshenko and Gere (1969) provide significant summaries of plate local buckling.

The following sections will examine slenderness limits in more detail for particular cases relevant to local buckling of plate elements in widely used structural members. The theory of elastic buckling is extended into the inelastic range, since in plastic design the plates have to deform plastically before local buckling occurs.

### 2.4.2 Definition of Slenderness

For a single plate, the definition of the width of the plate is obvious. However, when considering the plate elements in a structural section, the exact definition of the plate width is unclear, particularly if there are fillet welds (eg for a welded I-section) or corner radii (for an RHS) present. There are four possible definitions that may be appropriate as illustrated in Figure 2.15:

i) Full width,  
ii) mid-thickness width  
iii) clear width between supporting elements, or  
iv) flat width
(a) Flanges of hollow sections

(b) Webs of hollow sections

(c) Webs of I-sections

(d) Flanges of I-sections

Figure 2.15: Types of Definitions of Plate Width
Most common structural sections can be loosely described as “thin-walled”, i.e. the dimensions $b$ and $d$ are considerably larger than the thickness $t$. Many hot-formed hollow sections have small corner radii, and hence the four definitions of $b/t$ given above are not significantly different. Due to the cold-forming process for RHS, the corner radius is often in the range $2.0t \leq r_c \leq 2.5t$, and there can be a notable difference between the various width definitions for RHS. Table 2.2 lists the definition of plate width used by various design codes.

<table>
<thead>
<tr>
<th>Specification</th>
<th>RHS web or rolled flange</th>
<th>Rolled I-section web</th>
<th>Welded I-section web</th>
<th>Rolled/welded I-section flange</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eurocode 3</td>
<td>flat$^a$</td>
<td>flat (iv)</td>
<td>flat (iv)</td>
<td>mid (ii)</td>
</tr>
<tr>
<td>BS 5950</td>
<td>flat$^a$</td>
<td>flat (iv)</td>
<td>flat (iv)</td>
<td>mid (ii)</td>
</tr>
<tr>
<td>CSA S16.1</td>
<td>flat (iv)</td>
<td>clear (iii)</td>
<td>clear (iii)</td>
<td>mid (ii)</td>
</tr>
<tr>
<td>AS 4100</td>
<td>clear (iii)</td>
<td>clear (iii)</td>
<td>clear (iii)</td>
<td>clear (iii)</td>
</tr>
<tr>
<td>AISC LRFD</td>
<td>flat (iv)</td>
<td>flat (iv)</td>
<td>clear (iii)</td>
<td>mid (ii)</td>
</tr>
</tbody>
</table>

Notes: (a) Width taken as $b - 3t$, or $d - 3t$, which is the flat width assuming that $r_c = 1.5t$.
(b) Symbol in parentheses refers to definition in Figure 2.15.

Table 2.2: Definition of Width of Plate Elements in Selected Design Specifications

The following sections on slenderness limits will generally use the clear width definition (either $d_{\text{w}}$ or $b_{\text{w}}$) which is used in AS 4100, unless referring to a specific design standard.

### 2.4.3 ELEMENT IN COMPRESSION SUPPORTED ON ONE EDGE

The case of an element supported on one edge is applicable only to open sections such as I-sections or C-sections, and not to hollow sections, and is considered only briefly in this thesis. A fuller explanation is available in Lay (1965).

A plate simply supported on one edge represents a flange outstand of an I-section or a channel section. For elastic buckling of a plate with one longitudinal edge free and the other longitudinal edge simply supported, $k = 0.425, \text{ and } v = 0.3, \text{ and } E = 200000 \text{ MPa, and assuming } f_y \text{ is in MPa, then Equation 2.5 becomes}$
\[
\frac{b_{\text{int}}}{t} \leq \frac{277}{\sqrt{f_y}} \text{ or } \frac{b_{\text{int}}}{t} \sqrt{\frac{f_y}{250}} \leq 17.5 \quad \text{(elastic)}
\] (2.6)

The alternative form \((b_{\text{int}}/t\sqrt{(f_y/250)})\) is given as a direct comparison to the form of the slenderness limits in AS 4100.

Allowing for the effects of initial imperfections in the plate (Johnston 1966), and residual stresses (Ueda and Tall 1967), the elastic slenderness limit can be reduced to

\[
\frac{b_{\text{int}}}{t} \leq \frac{252}{\sqrt{f_y}} \text{ or } \frac{b_{\text{int}}}{t} \sqrt{\frac{f_y}{250}} \leq 16 \quad \text{(elastic)}
\] (2.7)

though the exact value of the elastic limit may change according to varying levels of residual stresses in the different forming processes of various types of steel sections.

In plastic design, the flange must be able to maintain the stress \(f_y\) for considerable deformation as the plastic hinge forms, before local buckling occurs. Lay (1965) derived an effective shear modulus in the inelastic range to establish the resistance against local buckling, and by considering the typical material properties of hot-rolled steel determined the following limit

\[
\frac{b_{\text{int}}}{t} \leq \frac{132}{\sqrt{f_y}} \text{ or } \frac{b_{\text{int}}}{t} \sqrt{\frac{f_y}{250}} \leq 8.3 \quad \text{(plastic)}
\] (2.8)

Lay (1965) assumed specific material properties of steel, including a yield plateau and strain hardening. Consequently, certain material requirements for plastic design were placed in some design standards, such as AS 4100.

Tests on twelve I-section beams under moment gradient by Lukey and Adams (1969), and eleven tests of Kato (1965) were generally in agreement with the limit proposed by Lay (1965).

A summary of the flange slenderness limits for I-sections under \(x\)-axis bending for several steel design specifications can be found in Bild and Kulak (1991).
2.4.4 ELEMENT IN COMPRESSION SUPPORTED ON BOTH EDGES

A flange of a box section or RHS or SHS in $x$-axis bending can be represented by a plate in uniform compression simply supported on all four sides. For elastic buckling of a plate with all sides simply supported, $k = 4$, and taking $v = 0.3$, and $E = 200000$ MPa, and assuming $f_y$ is in MPa, then Equation 2.5 changes to

$$\frac{b_{iii}}{t} \leq \frac{850}{\sqrt{f_y}} \quad \text{or} \quad \frac{b_{iii}}{t} \sqrt{\frac{f_y}{250}} \leq 54 \quad \text{(elastic)} \quad (2.9)$$

There is some uncertainty as to the restraint conditions that should be applied to the edges, since the webs are connected to the flanges and provide some torsional restraint to the flange plates. The edges cannot be considered fully clamped, and it is conservative to assume simply supported edges. The level of restraint depends on the slenderness of the restraining plate.

The limit in Equation 2.9 is reduced to account for residual stresses and imperfections. For the case of stress relieved plates

$$\frac{b_{iii}}{t} \leq \frac{707}{\sqrt{f_y}} \quad \text{or} \quad \frac{b_{iii}}{t} \sqrt{\frac{f_y}{250}} \leq 45 \quad \text{(elastic)} \quad (2.10)$$

The limit in Equation 2.10 may be lower for more severe residual stress, such as heavy welding

$$\frac{b_{iii}}{t} \leq \frac{550}{\sqrt{f_y}} \quad \text{or} \quad \frac{b_{iii}}{t} \sqrt{\frac{f_y}{250}} \leq 35 \quad \text{(elastic)} \quad (2.11)$$

The plastic limit for hot-rolled elements supported on two edges was determined theoretically by Haaijer and Thurlimann (1958) for 36 ksi (248 MPa) steel and assumed an inelastic rotation capacity $R = 3$:

$$\frac{b_{ii}}{t} \leq 32 \quad \text{(plastic)} \quad (2.12)$$
Korol and Hudoba (1972) investigated the behaviour of SHS, RHS and CHS in both hot-formed and cold-formed steel, with a total of 31 tests on single span and three span beams. Due to the lack of strain hardening, some sections did not exceed the plastic moment or exhibit considerable rotation capacity before local buckling occurred. The proposed limit for RHS and SHS flanges was:

\[
\frac{b_{iv}}{t} \leq \frac{394}{\sqrt{f_y}} \text{ or } \frac{b_{iv}}{t} \sqrt{\frac{f_y}{250}} \leq 25 \quad \text{(plastic)}
\]  

(2.13)

though the limit may be increased if the yield stress assumed in plastic design was less than the guaranteed minimum value in the relevant material specification.

Hasan and Hancock (1988) performed nineteen bending tests of Grade C350 cold-formed SHS and RHS under uniform moment. Zhao and Hancock (1991) tested cold-formed SHS and RHS in Grade C450 cold-formed steel. Combining the two sets of results, the flange slenderness limit for plastic design assuming an inelastic rotation capacity \(R = 4\), was given as:

\[
\frac{b_{iii}}{t} \leq \frac{474}{\sqrt{f_y}} \text{ or } \frac{b_{iii}}{t} \sqrt{\frac{f_y}{250}} \leq 30 \quad \text{(plastic)}
\]  

(2.14)

Corona and Vaze (1996) performed bending tests on small SHS with \(b/t\) ranging from 15.4 to 28.6. The most slender specimen, with \(b/t = 28.6\) achieved a rotation capacity of approximately \(R = 3.8\), which roughly fits with the limit proposed by Zhao and Hancock (1991).

Stranghöner (1995) performed tests on both cold-formed and hot-formed hollow sections to examine the flange slenderness limit. Stranghöner found that the hot-formed sections did not achieve the plastic moment, since they had a very large yield plateau. Theoretically, a section with simple elastic-perfectly plastic material properties will only reach the plastic moment at infinite curvature. Hence Stranghöner based the rotation capacity on achieving 95% of the plastic moment.
Table 2.3 gives the flange slenderness limits for RHS under x-axis bending for Eurocode 3, AISC LRFD and AS 4100 for each class of cross section. Note that recently the 1997 Hollow Structural Section Supplement (AISC 1997) to the 1994 AISC LRFD Specification gives a lower Class 1 limit for an RHS flange, compared to a box section flange. The limit in the 1997 Steel Hollow Section Supplement was lowered to match the values in the Canadian Standard CSA-S16.1, which was based on the results of Korol and Hudoba (1972). A more complete summary of limits in several steel design specifications is in Bild and Kulak (1991).

<table>
<thead>
<tr>
<th>Specification</th>
<th>Flange slenderness definition ( (\lambda_i) )</th>
<th>Flange slenderness limits</th>
</tr>
</thead>
<tbody>
<tr>
<td>AS 4100</td>
<td>( b - 2t \sqrt{\frac{f_y}{250}} )</td>
<td>Class 1, or Compact¹</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Class 2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Class 3, or Non-Compact³</td>
</tr>
<tr>
<td>Eurocode 3</td>
<td>( b - 3t \sqrt{\frac{f_y}{235}} )</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>40</td>
</tr>
<tr>
<td>AISC LRFD HSS (1997)</td>
<td>( b - 2r_e \sqrt{\frac{f_y}{E}} )</td>
<td>33</td>
</tr>
<tr>
<td></td>
<td></td>
<td>38</td>
</tr>
<tr>
<td></td>
<td></td>
<td>42</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(32.0)²</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(38.8)²</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(40.7)²</td>
</tr>
<tr>
<td>AISC LRFD (1994)³</td>
<td>( b - 2r_e \sqrt{\frac{f_y}{E}} )</td>
<td>9.39</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(26.6)²</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(39.6)²</td>
</tr>
</tbody>
</table>

**Notes:**
1. AS 4100 and AISC LRFD definition of “compact” is Class 1.
2. Applies to \( f_y = 250 \text{ MPa} \) and \( E = 200000 \text{ MPa} \) for comparison with the AS 4100 limits.
3. Sections exceeding the Class 3 or Non-Compact limit are Class 4 or Slender respectively.
5. 1994 AISC LRFD applies to flanges of box sections.

Table 2.3: Summary of RHS Flange Slenderness Limits for x-axis Bending
2.4.5 ELEMENT IN BENDING SUPPORTED ON BOTH EDGES

In this section the web slenderness is referred to as \( d/t \) to avoid confusion with the flange slenderness. This thesis is concerned with the behaviour of RHS, but in current design standards the same slenderness limits apply to the webs of RHS and I-sections. Most research on web slenderness limits considered tests of simple plates or I-sections.

For an I-section or RHS bending about its major principal axis, the web is in bending. Under bending, the webs of I-sections and RHS have a linear variation of strain across the web and are supported on both longitudinal edges, but the nature of any restraint provided by the flange is likely to be different for I-sections and RHS, since the centre of the flange of an I-section restraints the web, whereas the ends of the flange restrain the webs of an RHS. Another difference between RHS and I-sections, is that for RHS the area of the two webs ranges from about 50% of the total area of the section (for an SHS, \( d/b = 1 \)) to about 75% of the area (for an RHS with aspect ratio \( d/b = 3 \)), whereas for an I-section the web is typically 35 - 45% of the total area. Hence the influence of the web is greater in RHS and SHS than in I-sections.

A web in bending is a simple case of a web in bending and compression, assuming no compression. Many of the investigations outlined below were examinations of bending and compression, but only the results relevant to bending are outlined. Since this thesis is primarily concerned with the behaviour of webs in bending only, the case of bending and compression is not treated in as much detail (Section 2.4.6).

A plate simply supported on both edges with a stress gradient varying from tension to compression represents the condition of the web. For elastic buckling in this case, \( k = 23.9 \), and hence the limit based on elastic buckling (Equation 2.5) becomes:

\[
\frac{d_{\text{ii}}}{t} \leq \frac{2078}{\sqrt{f_y}} \quad \text{or} \quad \frac{d_{\text{ii}}}{t} \sqrt{\frac{f_y}{250}} \leq 131 \quad \text{(elastic)}
\]  

(2.15)

It is usually conservative to assume simply supported conditions on the longitudinal edges, since the flanges normally provide some torsional restraint to the longitudinal edges of the web.
The elastic limit is affected by residual stresses and imperfections:

\[
\frac{d_{\text{iii}}}{t} \leq \frac{1818}{\sqrt{f_y}} \quad \text{or} \quad \frac{d_{\text{iii}}}{t} \sqrt{\frac{f_y}{250}} \leq 115 \quad \text{(elastic)} \tag{2.16}
\]

Initial investigations of webs were performed by Lyse and Godfrey (1934) who tested fourteen I-sections in shear and bending. The trends in their results “indicated that buckling of the web might occur at a depth-thickness of approximately 80” for steel with yield stress approximately \(f_y \approx 235 - 250\) MPa. Lyse and Godfrey did not provide a definitive limit, as the specimens tested had a maximum \(d/t = 70\).

The work of Lyse and Godfrey was extended by Haaijer (1957), and Haaijer and Thurlimann (1958). Haaijer (1957) performed pure bending and pure compression tests on six different I-sections. Haaijer and Thurlimann (1958) extended the plate buckling theory outlined in Haaijer (1957) to include residual stresses, and determined values of the plate buckling coefficient, \(k\), for a fully plastified web plate under bending and compression. From the values of \(k\), recommendations for slenderness limits of webs were put forward. Bending and compression tests were not performed to verify the proposal, but the proposal agreed with the initial tests of Haaijer (1957). Assuming \(\epsilon_m/\epsilon_y = 4\), where \(\epsilon_m\) is the maximum strain and \(\epsilon_y\) the yield strain, (corresponding to \(R = 3\)), and using the mid-width thickness definition of \(d\) in Figure 2.15(c)(ii), the following limit was suggested for 33 ksi (230 MPa) steel:

\[
\frac{d_{\text{iii}}}{t} \leq 64 \quad \text{(plastic)} \tag{2.17}
\]

Lower limits were given if greater rotation capacity was required.

The limit was altered by the ASCE (1961) and AISC (1963) for 33 ksi (230 MPa) to be:

\[
\frac{d_{\text{iii}}}{t} \leq 70 \quad \text{(plastic)} \tag{2.18}
\]

The ASCE limit is slightly higher than Haaijer and Thurlimann’s limit, since ASCE used the clear width definition for \(d\) shown in Figure 2.15(c)(iii).
The 1969 version of the Canadian Steel Specification, CSA-S16 (Canadian Standards Association 1969), listed one web slenderness limit, namely the “Plastic” (Class 1) limit. To improve the data, there was a series of tests on I-sections to examine the Class 2 and Class 3 limits for webs under pure bending, and bending and compression, at the University of Alberta, Canada. Holtz and Kulak (1973, 1975) performed pure bending tests on a range of I-sections to examine the Class 2 and Class 3 limits under pure bending. Perlynn and Kulak (1974) performed bending and compression tests on three I-sections with various values of web slenderness and constant flange slenderness, at three different levels of axial force. Nash and Kulak (1976) performed six compression and bending tests on I-sections at three levels of axial compression. Compact (Class 2) and non-compact (Class 3) limits for I-sections webs under pure bending, and bending and compression were produced, but the most relevant limit is the Class 2 limit for pure bending which is given in Table 2.4. The recommendations were incorporated into the 1974 and 1978 editions of the Canadian Steel Standard (Canadian Standards Association 1974, 1978).

Della-Croce (1970) and Costley (1970) performed tests on continuous I-section beams at the University of Texas at Austin. Della-Croce tested eight welded plate girders, continuous over three spans, with different point loading configurations. The sections with web slenderness up to $d/t = 123$ were capable of achieving the plastic moment. In many of Della-Croce’s tests, the beams carried extra load after the initiation of web buckling, indicating the substantial post buckling reserve of strength. Della-Croce recommended that the $d/t$ limit for 36 ksi (248 MPa) steel could be increased to 125 for both compact (Class 2) and plastic (Class 1) I-sections. Costley performed tests on eleven continuous rolled steel beams to examine lateral stability and bracing length in plastic design.

Edinger and Haaijer (1984) summarise the work of Della-Croce and Costley as follows:

“Della-Croce investigated the behaviour of continuous welded girders designed with web and flange slenderness considerably in excess of the limits prescribed in Part 2 [the AISC ASD rules for plastic design]. All of the specimens tested developed the full plastic mechanism and, due to the effect of strain hardening, reached an ultimate load in excess of the classical plastic design ultimate load. The stability of plate girders with thin unstiffened webs was also confirmed [by Costley] on tests of nine full size plate girders.”
Dawe and Kulak (1981, 1984a, 1984b, 1986) produced a computer program to analyse local buckling of I-sections to include flange-web interaction, residual stresses, yielding and strain hardening. The program was based on virtual work and a Rayleigh-Ritz technique. Dawe and Kulak calibrated the finite element program against the series of tests on I-sections performed at the University of Alberta by Holtz and Kulak (1973, 1975), Nash and Kulak (1976), and Perlynn and Kulak (1974) described above. Based on the analysis, Dawe and Kulak (1986) suggested a new set of limits for Class 2 and Class 3 webs of I-sections, as shown in Table 2.4. The recommendations were incorporated, in a slightly altered form, into the 1989 edition of the Canadian Steel Specification (Canadian Standards Association 1989). Dawe and Kulak (1986) suggested only a minor change to the Class 1 limit for webs in bending, and so the Class 1 limit for webs in bending has remained unchanged since 1969 (though there was a change to the Class 1 limit for webs in bending and compression in 1989). The web slenderness limits in the most recent 1994 edition of the Canadian Steel Specification are unchanged from the values given in the 1989 edition.

Dawe, Elgabry and Grondin (1985) subsequently used the computer program to examine the behaviour of SHS and RHS columns.

The 1969 AISC ASD Specification introduced differentiation between “compact” (Class 2) and “plastic” (Class 1) sections. However, the 1969 Specification gave the same limits for plastic (Class 1) and compact (Class 2) sections.

Based on tests by Della-Croce (1970) and Costley (1970), Supplement No 3 (1974) to the 1969 AISC ASD Specification increased the compact (Class 2) limit, but did not alter the plastic (Class 1) limit. There have been no changes to the plastic (Class 1) and compact (Class 2) web limits in subsequent revisions of the AISC ASD Specification in 1978, and the latest edition published in 1989. In AISC ASD the term “compact” specifically refers to Class 2 sections, as noted in AISC ASD (1989) [Commentary to Clause N.1]: “compact sections are proportioned so that the cross section may be strained in bending to the degree necessary to achieve full plastification of the cross section, however the reserve for inelastic strains is adequate only to achieve modest redistribution of moments”
Galambos (1976) prepared a draft LRFD version of the AISC Specification. For what Galambos termed the “compact elastic” (Class 2) web limit, the compact (Class 2) limit in the 1974 Supplement to the AISC ASD was used. Galambos defines “compact elastic design” for “indeterminate beams where the moments are determined by elastic analysis and in determinate beams”. Galambos increased the plastic (Class 1) limit to the recommendations of Perlynn and Kulak (1974). Galambos clearly identifies that the limit is appropriate for “indeterminate beams where the moments are determined by plastic analysis” (Class 1), while Perlynn and Kulak’s recommendation was for “compact” (Class 2) sections.

A further draft version of the LRFD was prepared by the AISC in 1983 (AISC 1983). In the 1983 Draft, the meaning of the term “compact” changes from the definition of Galambos (1976) and the AISC ASD. “Compact” sections are deemed suitable for plastic design (Class 1) in the 1983 draft. The “compact” (Class 1) limit in the 1983 draft is the “compact elastic” (Class 2) of Galambos (1976). Edinger and Haaijer (1984) summarise the plastic design requirements in the 1983 AISC draft, and write that the work of Della-Croce (1970) and Costley (1970) provide the justification for increasing the compact (Class 1) limit from the plastic (Class 1) limit in the 1978 AISC ASD.

In the finally published AISC LRFD Specification (1986, 1993, 1994), the term “compact” refers to plastic design (Class 1). AISC LRFD (1994) states [Commentary to Clause B.5] that “compact sections are capable of developing a fully plastic stress distribution and they possess a rotational capacity of approximately 3 before the onset of local buckling” and Clause B5.2 states that “plastic design is permitted when flanges ... and webs have a width-thickness ratio less than or equal to the limiting $\lambda_p$ [compact limit]”. The “compact” (Class 1) limit is AISC LRFD (1986, 1993, 1994) is unchanged from that in the 1983 draft.
AISC LRFD (1994) includes a “seismic compact” limit. AISC LRFD (1994) mentions [Commentary to Clause B.5] that in seismic regions greater rotation capacity may be required that the rotation provided by a compact section, and rotation capacities of the order $R = 7 \sim 9$ (Chopra and Newmark 1980) need to be provided. Consequently, a more stringent “seismic compact” limit is necessary. The “seismic compact” (Seismic Class 1) limit in AISC LRFD, is the same as the plastic (Class 1) limit in Galambos (1976). Commentary to Clause B5 in the 1993 AISC LRFD specifically states that the “seismic compact” limits have been taken from Galambos (1976), yet Galambos makes no reference to seismic behaviour and extra stringent rotation requirements.

Most recently, the AISC has produced a supplement to the 1994 AISC LRFD, specifically for hollow sections, “Specification for the Design of Steel Hollow Structural Sections” (AISC 1997). The “Compact” (Class 1) limit for RHS webs is unchanged from the 1994 AISC LRFD.

To summarise, for the webs of I-sections, the “compact elastic” (Class 2) limit of Galambos (1976) became the “compact” (Class 1) limit in AISC LRFD (1986, 1993, 1994). The “compact” (Class 2) limit of Perlynn and Kulak became the “plastic” (Class 1) limit of Galambos, which then became the “compact seismic” limit (Seismic Class 1) in AISC LRFD (1986, 1993, 1994).

Similarly for the flange limits for I-sections, the “plastic” (Class 1) limit of Galambos, became the “compact seismic” limit (Seismic Class 1) in AISC LRFD, and Galambos’ “compact elastic” (Class 2) limit became the “compact” (Class 1) limit in AISC LRFD.

It appears as if there are inconsistencies in the development of the limits in AISC LRFD (1986, 1993, 1994), and that the web slenderness limits for plastic design (Class 1) are different in the AISC ASD (1989) and AISC LRFD, as indicated in Table 2.4.
The British Standard BS 5950 gives a higher Class 1 limit \( \frac{d_w}{t} = 1310 / \sqrt{f_y} \) or \( \frac{d_w}{t} \sqrt{f_y/250} = 83 \) than the ASCE (1961) limit. The exact derivation of the British limit is unclear, but it is possible that it is based on buckling of the web in shear. Kerensky, Flint and Brown (1956) refer to a limit of \( \frac{d_w}{t} = 85 \) for 235 MPa steel (corresponds to \( \frac{d_w}{t} = 1303 / \sqrt{f_y} \) or \( \frac{d_w}{t} \sqrt{f_y/250} = 82.4 \), which was based on the web yielding before buckling in shear. Longbottom and Heyman (1956) explain the basis of the limit as:

“A previous paper [Heyman and Dutton 1954] reported tests on small (3-in deep) plate girders with thick webs. The results enabled rules to be formulated for the plastic design of such plate girders, provided the \( d/t \) ratio for the web was not greater than about 72 [for 235 MPa steel]... The figure of 72 was arrived at by considering reports by other investigators, but it was felt at the time that the figure might well be increased, certainly up to the figure of 85 proposed in the draft BS 153 for unstiffened webs.”

The limit of approximately \( \frac{d_w}{t} = 1303 / \sqrt{f_y} \) or \( \frac{d_w}{t} \sqrt{f_y/250} = 82 \) for shear is noted in other references (Australian Standard AS 1250 (Standards Australia 1975), Australian Standard AS 4100, Lay 1970, Lay 1982, and Trahair and Bradford 1991).

Horne (1979) identified that the Class 1 web limit for bending in the British Standard was higher than those in the United States [AISC ASD], and stated:

“...At the higher [web slenderness] ratios used in the UK as opposed to the USA practice, webs tend to undergo some degree of buckling and a slight fall-off in mean stress below the yield value during the plastic deformation of a “plastic hinge” section, but the fall-off in moment of resistance is insufficient to affect significantly the carrying capacity of the structure containing the member in question”

In the upgrade of the working stress Australian Standard AS 1250 to the limit state Australian Standard AS 4100, the plastic web limit was increased to coincide with the British Standard BS 5950.

\[
\frac{d}{t} \sqrt{\frac{f_y}{250}}
\]
<table>
<thead>
<tr>
<th>Specification</th>
<th>Def.</th>
<th>Web slenderness limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASCE (1961) “Plastic” (Class 1)</td>
<td>iii</td>
<td>70</td>
</tr>
<tr>
<td>AISC ASD (1969) “Compact” (Class 2)</td>
<td>iv</td>
<td>$1081 / \sqrt{f_y}$</td>
</tr>
<tr>
<td>AISC ASD (1969, 1978, 1989) “Plastic” (Class 1)</td>
<td>iv</td>
<td>$1081 / \sqrt{f_y}$</td>
</tr>
<tr>
<td>AISC ASD (1969) Supplement 3 (1974) “Compact” (Class 2)</td>
<td>iv</td>
<td>$1679 / \sqrt{f_y}$</td>
</tr>
<tr>
<td>AISC LRFD Draft Galambos (1976) “Compact elastic” (Class 2)</td>
<td>vi</td>
<td>$1679 / \sqrt{f_y}$</td>
</tr>
<tr>
<td>AISC LRFD Draft Galambos (1976) “Plastic” (Class 1)</td>
<td>iv</td>
<td>$1364 / \sqrt{f_y}$</td>
</tr>
<tr>
<td>AISC LRFD Draft (1983) “Plastic” (Class 1)</td>
<td>iv</td>
<td>$1679 / \sqrt{f_y}$</td>
</tr>
<tr>
<td>AISC LRFD (1986, 1993, 1994) “Compact” (Class 1)</td>
<td>iv</td>
<td>$1679 / \sqrt{f_y}$</td>
</tr>
<tr>
<td>AISC LRFD (1986, 1993, 1994) “Compact Seismic” (“Seismic” Class 1)</td>
<td>iv</td>
<td>$1364 / \sqrt{f_y}$</td>
</tr>
<tr>
<td>Canadian Standard S16.1 (1969) “Plastic” (Class 1) (no Class 2 or Class 3 limit specified in 1969)</td>
<td>iv</td>
<td>$1100 / \sqrt{f_y}$</td>
</tr>
<tr>
<td>Canadian Standard S16.1 (1989, 1994) “Compact” (Class 2)</td>
<td>iv</td>
<td>$1700 / \sqrt{f_y}$</td>
</tr>
<tr>
<td>Canadian Standard S16.1 (1989, 1994) “Non-Compact” (Class 3)</td>
<td>iv</td>
<td>$1900 / \sqrt{f_y}$</td>
</tr>
<tr>
<td>Eurocode 3 (Class 1)</td>
<td>iv*</td>
<td>$1103 / \sqrt{f_y}$</td>
</tr>
<tr>
<td>British Standard BS 5950 “Plastic” (Class 1)</td>
<td>iv*</td>
<td>$1310 / \sqrt{f_y}$</td>
</tr>
<tr>
<td>Australian Standard AS 1250 (1975) “Plastic” (Class 1)</td>
<td>iii</td>
<td>$1120 / \sqrt{f_y}$</td>
</tr>
<tr>
<td>Australian Standard AS 4100 (1991) “Compact” (Class 1)</td>
<td>iii</td>
<td>$1296 / \sqrt{f_y}$</td>
</tr>
<tr>
<td>Perlynn and Kulak (1974) “Compact” (Class 2)</td>
<td>iv</td>
<td>$1364 / \sqrt{f_y}$</td>
</tr>
<tr>
<td>Nash and Kulak (1976) “Non-Compact” (Class 3)</td>
<td>iv</td>
<td>$1810 / \sqrt{f_y}$</td>
</tr>
<tr>
<td>Dawe and Kulak (1986) “Plastic” (Class 1)</td>
<td>iv</td>
<td>$1129 / \sqrt{f_y}$</td>
</tr>
<tr>
<td>Dawe and Kulak (1986) “Compact” (Class 2)</td>
<td>iv</td>
<td>$1733 / \sqrt{f_y}$</td>
</tr>
<tr>
<td>Dawe and Kulak (1986) “Non-Compact” (Class 3)</td>
<td>iv</td>
<td>$1904 / \sqrt{f_y}$</td>
</tr>
</tbody>
</table>

Notes: Column \(d\) refers to the definition of the web depth defined in Section 2.4.2.  
(1) Applied to 33 ksi (230 MPa) steel.  
#: Eurocode 3 and BS 5950 use a flat width definition of the RHS web depth assuming that \(r_e = 1.5t\).
Finite strip analysis of I-beams by Bradford (1987) indicated that the Class 2 web slenderness limit in BS 5950 was non-conservative. Hence the Compact (Class 1) value in AISC LRFD is also non-conservative by Bradford’s results.

All of the previous research listed above indicates that the web slenderness limits are based on tests of I-sections. As each web of an RHS has similar support conditions to an I-section web (although there are two webs in an RHS), the slenderness limits in current design standards are deemed to apply to both RHS and I-sections, although it may not be justified as discussed below.

When RHS were first manufactured, they were either square (ie SHS) or rectangular with low aspect ratios. The slenderness limits summarised in Table 2.4 above indicate that the limits for the webs of RHS were more than twice the flange slenderness limits for RHS. So for SHS and low aspect ratio RHS in bending, flange buckling would be more critical than web buckling. Hence there was no great need to investigate the web slenderness limit for RHS.

More recently, RHS have been produced with higher aspect ratios, such as 3.0 (Tubemakers 1994). The webs of such sections are considerably more slender than the flange, and the possibility of web local buckling before flange buckling is increased. Zhao and Hancock (1992) observed inelastic web local buckling in a 102 × 51 × 2.0 C350 RHS. The local buckling occurred at low rotation values for specimens with flange and web slenderness values below the limits set in current design standards for plastic design \((b-2t)/t\sqrt{f_y/250} = 29.7 < 30\), and \((d-2t)/t\sqrt{f_y/250} = 62 < 82\), where 30 and 82 are the Class 1 limits in AS 4100). The results of Zhao and Hancock provided part of the impetus for the current series of tests on higher aspect ratio RHS.

Table 2.5 gives the web slenderness limits for RHS under \(x\)-axis bending for Eurocode 3, AISC LRFD and AS 4100 for each class of cross section, and a more complete summary of several steel design specifications is in Bild and Kulak (1991).
### Table 2.5: Summary of RHS Web Slenderness Limits for x-axis Bending

<table>
<thead>
<tr>
<th>Specification</th>
<th>Web slenderness definition ((\lambda_w))</th>
<th>Web slenderness limits</th>
</tr>
</thead>
<tbody>
<tr>
<td>AS 4100</td>
<td>(\frac{d - 2t}{t} \sqrt{\frac{f_y}{250}})</td>
<td>82 () 115</td>
</tr>
<tr>
<td>Eurocode 3</td>
<td>(\frac{d - 3t}{t} \sqrt{\frac{f_y}{235}})</td>
<td>72 () 83 () 124</td>
</tr>
<tr>
<td>AISC LRFD</td>
<td>(\frac{d - 2r_e}{t} \sqrt{\frac{f_y}{E}})</td>
<td>3.76 () 5.7 () (161)^2</td>
</tr>
</tbody>
</table>

**Notes:**

1. AS 4100 and AISC LRFD definition of “compact” is Class 1.
2. Applies to \(f_y = 250\) MPa and \(E = 200000\) MPa for comparison with the AS 4100 limits.
3. Sections exceeding the Class 3 or Non-Compact limit are Class 4 or Slender respectively.

#### 2.4.6 ELEMENT IN BENDING AND COMPRESSION SUPPORTED ON BOTH EDGES

If an I-section or RHS is bent about its major principal axis, and there is net axial compression, the distribution of stress and strain in the web is changed from the case of pure bending. The neutral axis shifts and more than half of the web is in compression. Figure 2.16 illustrates the elastic and fully plastic stress and strain distributions in webs for the cases of pure bending, and bending and compression. Since more of the web is in compression, the web is more likely to experience local buckling compared to the case of pure bending. Hence web slenderness limits become lower with an increase in compression.
The case of a web in bending and compression is a logical extension of the case of bending only, which was considered in the previous section. Many of the test programs outlined in Section 2.4.5 were examinations of bending and compression, but only the results relevant to bending were outlined. Since this thesis is primarily concerned with the behaviour of webs in bending, the case of bending and compression is not treated in as much detail.

The plastic limit for webs in bending and compression was derived by Haaijer (1957), and Haaijer and Thurlimann (1958), and incorporated the original tests of Lyse and Godfrey (1934). Haaijer (1957), and Haaijer and Thurlimann (1958) performed pure bending and pure compression tests on six different I-sections, and assumed a rotation capacity of $R \approx 3$. Theoretical analysis (outlined previously) was performed to interpolate the limit between pure bending and pure compression.
Haaijer and Thurlimann’s recommendations were incorporated into ASCE (1961) and AISC (1963) for 33 ksi (230 MPa) steel only as:

\[
\frac{d}{t} \leq 70 - 100 \frac{N}{N_y} \quad \text{for} \quad \frac{N}{N_y} \leq 0.27
\]

\[
\frac{d}{t} \leq 43 \quad \text{for} \quad \frac{N}{N_y} > 0.27 \quad \text{(plastic)}
\]

(2.19) where \(N\) is the net axial compression and \(N_y\) is the yield (squash) load of the member. The limit for \(d/t\) drops linearly from 70 to 43 as the axial load increases to 27% of the squash load. For higher values of compression the limit remains constant. The abbreviation \(n = N/N_y\) is commonly used. For the case of pure bending \(n = 0\), and limit reverts to the limit for pure bending as given in Section 2.4.5.

A major investigation of slenderness limits of I-section webs in bending and compression was performed at the University of Alberta (outlined in Section 2.4.5). The tests of Holtz and Kulak (1973, 1975), Nash and Kulak (1976), and Perlynn and Kulak (1974), were followed by the numerical simulation of Dawe and Kulak (1981, 1984a, 1984b, 1986). The recommendations were incorporated into the 1974, 1978, and 1989 editions of the Canadian Steel Standard.

A summary of Class 1 and Class 2 slenderness limits for webs in bending and compression is given in Tables 2.6(a) and 2.6(b), and shown graphically in Figures 2.17 and 2.18. A summary of several steel design specifications can be found in Bild and Kulak (1991).

The common practice is to define the slenderness limit for webs as a function of the net axial compression \((N/N_y)\) in the section, as highlighted by the limit defined by ASCE (1961) in Equation 2.19. There can be the case where a section is Class 1 for low axial load, but Class 2 for higher axial loads. There is a potential for confusion if a section can change classes. The Australian Standard AS 4100 uses a different approach. The equation is rearranged so that the maximum axial load is a function of the value of \(d/t\). The Australian approach means that a member cannot change classes depending on the axial load, but the axial compression is limited according the value of \(d/t\). Both approaches yield the same outcome. For consistency, this thesis uses the more common approach in which the \(d/t\) limit is expressed as a function of \(N/N_y\), and the equations from AS 4100 have been rearranged into that format.
<table>
<thead>
<tr>
<th>Specification</th>
<th>Def. d</th>
<th>Web slenderness limit</th>
<th>Limit for $f_y = 250$ MPa &amp; $n = 0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASCE (1961) “Plastic” (Class 1)</td>
<td>i</td>
<td>$70 - 100n$ $n &lt; 0.27$</td>
<td>68.4</td>
</tr>
<tr>
<td>AISC ASD (1969, 1989) “Plastic” (Class 1)</td>
<td>iv</td>
<td>$1081(1 - 1.4n)/\sqrt{f_y}$ $n &lt; 0.27$</td>
<td>68.4</td>
</tr>
<tr>
<td>AISC ASD (1969) “Compact” (Class 2)</td>
<td>iv</td>
<td>$1081(1 - 2.33f'\alpha)/\sqrt{f_y}$ $f' &gt; 0.16$</td>
<td>106</td>
</tr>
<tr>
<td>AISC ASD (Supplement 3 1974, 1989) “Compact” (Class 2)</td>
<td>iv</td>
<td>$1680(1 - 3.74f'\alpha)/\sqrt{f_y}$ $f' &gt; 0.16$</td>
<td>106</td>
</tr>
<tr>
<td>LRFD Draft Galambos (1976) “Plastic” (Class 1)</td>
<td>iv</td>
<td>$1365(1 - 1.54n)/\sqrt{f_y}$ $n &lt; 0.125$</td>
<td>86.5</td>
</tr>
<tr>
<td>LRFD Draft Galambos (1976) “Compact elastic” (Class 2)</td>
<td>iv</td>
<td>$1680(1 - 2.75n)/\sqrt{f_y}$ $n &lt; 0.125$</td>
<td>86.5</td>
</tr>
<tr>
<td>AISC LRFD Draft (1983) “Compact” (Class 1)</td>
<td>iv</td>
<td>$1680(1 - 2.75n)/\sqrt{f_y}$ $n &lt; 0.125$</td>
<td>86.3</td>
</tr>
<tr>
<td>AISC LRFD (1986, 1993, 1994, 1997) “Compact” (Class 1)</td>
<td>iv</td>
<td>$1680(1 - 2.75n)/\sqrt{f_y}$ $n &lt; 0.125$</td>
<td>86.3</td>
</tr>
<tr>
<td>AISC LRFD (1986) “Compact Seismic” (“Seismic” Class 1)</td>
<td>iv</td>
<td>$1365(1 - 1.54n)/\sqrt{f_y}$ $n &lt; 0.125$</td>
<td>86.3</td>
</tr>
<tr>
<td>AISC LRFD (1993, 1994) “Compact Seismic” (“Seismic” Class 1)</td>
<td>iv</td>
<td>$1365(1 - 1.54n)/\sqrt{f_y}$ $n &lt; 0.125$</td>
<td>86.3</td>
</tr>
<tr>
<td>Eurocode 3 (Class 1)</td>
<td>iv*</td>
<td>$6070/(13\alpha - 1)\sqrt{f_y}$ All $\alpha$</td>
<td>69.8</td>
</tr>
<tr>
<td>Eurocode 3 (Class 2)</td>
<td>iv*</td>
<td>$6990/(13\alpha - 1)\sqrt{f_y}$ All $\alpha$</td>
<td>69.8</td>
</tr>
<tr>
<td>British Standard BS 5950 “Plastic” (Class 1)</td>
<td>iv*</td>
<td>$1310/(1.2\alpha + 0.4)\sqrt{f_y}$ All $\alpha$</td>
<td>82.6</td>
</tr>
<tr>
<td>British Standard BS 5950 “Compact” (Class 2)</td>
<td>iv*</td>
<td>$813/(\alpha\sqrt{f_y})$ All $\alpha$</td>
<td>102</td>
</tr>
</tbody>
</table>

Notes:  
$n = N/N_y$, ratio of axial force to squash load.
$f'$ refers to $f_y/f_{Py}$ in terms of allowable stress design. It is common to assume that $0.6n = f'$.
Column $d$ refers to the definition of the web depth defined in Section 2.4.2.
#: Eurocode 3 and BS 5950 use a flat width definition of the RHS web depth assuming that $r_e = 1.5t$.
§: $\alpha$ is the proportion of the web in compression, approximated as $\alpha = (n + 1)/2$.

Table 2.6 (a): Summary of Web Slenderness Limits Under Bending and Compression
### Table 2.6 (b): Summary of Web Slenderness Limits Under Bending and Compression

<table>
<thead>
<tr>
<th>Specification</th>
<th>Def. $d$</th>
<th>Web slenderness limit</th>
<th>Limit for $f_y = 250$ MPa &amp; $n = 0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Canadian Standard S16.1 (1969, 1974, 1978, 1984) “Plastic” (Class 1)</td>
<td>iv</td>
<td>$1100(1 - 1.4n)/\sqrt{f_y}$</td>
<td>$n &lt; 0.28$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$670/\sqrt{f_y}$</td>
<td>$n &gt; 0.28$</td>
</tr>
<tr>
<td>Canadian Standard S16.1 (1974, 1978, 1984) “Compact” (Class 2)</td>
<td>iv</td>
<td>$1370(1 - 1.28n)/\sqrt{f_y}$</td>
<td>$n &lt; 0.15$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$1180(1 - 0.43n)/\sqrt{f_y}$</td>
<td>$n &gt; 0.15$</td>
</tr>
<tr>
<td>Canadian Standard S16.1 (1974) “Non-Compact” (Class 3)</td>
<td>iv</td>
<td>$1810(1 - 2.6n)/\sqrt{f_y}$</td>
<td>$n &lt; 0.15$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$1180(1 - 0.43n)/\sqrt{f_y}$</td>
<td>$n &gt; 0.15$</td>
</tr>
<tr>
<td>Canadian Standard S16.1 (1978, 1984) “Non-Compact” (Class 3)</td>
<td>iv</td>
<td>$1810(1 - 1.69n)/\sqrt{f_y}$</td>
<td>$n &lt; 0.15$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$1470(1 - 0.54n)/\sqrt{f_y}$</td>
<td>$n &gt; 0.15$</td>
</tr>
<tr>
<td>Canadian Standard S16.1 (1989, 1994) “Plastic” (Class 1)</td>
<td>iv</td>
<td>$1100(1 - 0.39n)/\sqrt{f_y}$</td>
<td>All $n$</td>
</tr>
<tr>
<td>Canadian Standard S16.1 (1989, 1994) “Compact” (Class 2)</td>
<td>iv</td>
<td>$1700(1 - 0.61n)/\sqrt{f_y}$</td>
<td>All $n$</td>
</tr>
<tr>
<td>Canadian Standard S16.1 (1989, 1994) “Non-Compact” (Class 3)</td>
<td>iv</td>
<td>$1900(1 - 0.65n)/\sqrt{f_y}$</td>
<td>All $n$</td>
</tr>
<tr>
<td>Australian Standard AS 1250 (1975) “Plastic” (Class 1)</td>
<td>iii</td>
<td>$1120(1 - 1.42n)/\sqrt{f_y}$</td>
<td>$n &lt; 0.27$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$688/\sqrt{f_y}$</td>
<td>$n &gt; 0.27$</td>
</tr>
<tr>
<td>Australian Standard AS 4100 (1991) “Compact” (Class 1)</td>
<td>iii</td>
<td>$1296(1 - 1.67n)/\sqrt{f_y}$</td>
<td>$n &lt; 0.27$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$826(1 - 0.52n)/\sqrt{f_y}$</td>
<td>$n &gt; 0.27$</td>
</tr>
<tr>
<td>Perlynn and Kulak (1974) (Class 2)</td>
<td>iv</td>
<td>$1365(1 - 1.54n)/\sqrt{f_y}$</td>
<td>$n &lt; 0.125$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$399(2.89 - n)/\sqrt{f_y}$</td>
<td>$n &gt; 0.125$</td>
</tr>
<tr>
<td>Nash and Kulak (1976) (Class 3)</td>
<td>iv</td>
<td>$1810(1 - 1.69n)/\sqrt{f_y}$</td>
<td>$n &lt; 0.15$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$1404(1 - 0.28n)/\sqrt{f_y}$</td>
<td>$n &gt; 0.15$</td>
</tr>
<tr>
<td>Dawe and Kulak (1986) (Class 1)</td>
<td>iv</td>
<td>$1128(1 - 0.93n)/\sqrt{f_y}$</td>
<td>$n &lt; 0.15$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$1002(1 - 0.22n)/\sqrt{f_y}$</td>
<td>$n &gt; 0.15$</td>
</tr>
<tr>
<td>Dawe and Kulak (1986) (Class 2)</td>
<td>iv</td>
<td>$1732(1 - 0.546n)/\sqrt{f_y}$</td>
<td>All $n$</td>
</tr>
<tr>
<td>Dawe and Kulak (1986) (Class 3)</td>
<td>iv</td>
<td>$1903(1 - 0.586n)/\sqrt{f_y}$</td>
<td>All $n$</td>
</tr>
</tbody>
</table>

**Notes:** $n = N/N_c$, ratio of axial force to squash load. Column $d$ refers to the definition of the web depth defined in Section 2.4.2.
Figure 2.17: Summary of Class 1 Web Slenderness Limits Under Bending and Compression

Figure 2.18: Summary of Class 2 Web Slenderness Limits Under Bending and Compression
There have been bending and compression tests on hollow sections, such as Dwyer and Galambos (1965) and Sully and Hancock (1996), but they examined member behaviour (overall flexural buckling) as opposed to local buckling and slenderness limits.

2.4.7 INTERACTION EFFECTS IN LOCAL BUCKLING

Sections 2.4.4 and 2.4.5 considered local buckling of elements supported on both edges. But, there is some uncertainty as to the restraint conditions that should be applied to the edges, since the webs are connected to the flanges and provide some torsional restraint. The edges cannot be considered fully clamped, and hence it is conservative to assume simply supported edges. The level of restraint will depend on the slenderness of the restraining plate. It is logical that there will be some type of interaction between the flange and the web which affects local buckling. In all current design specifications web and flange slenderness limits are independent of each other.

Kato (1989, 1990) investigated the behaviour of I-sections, and noted that the flange and web slenderness limits were independent of each other, and wrote:

“Obviously, flange is restrained by web, and vice versa, web is restrained by flanges, and therefore an independent limitation is unreasonable.”

Analysing the results of several test series on I-sections, Kato (1989, 1990) produced elliptical limits to consider flange - web interaction for various classes of sections, such as the Class 1 limit for I-sections as:

\[
\left( \frac{b}{t_f} \right)^2 + \left( \frac{d}{t_w} \right)^2 = 1 \quad \text{(plastic)} \tag{2.20}
\]

Kuhlmann (1989) established a relationship for I-sections between flange slenderness and rotation capacity which included interaction with the web based on a series of 24 tests on I-sections.
Beg and Hladnik (1996) performed tests on ten I-sections and then used finite element analysis to produce Class 3 limits for I-sections that incorporated interaction between the flange and the web.

Kemp (1985, 1986, 1996) produced flange and web slenderness limits that accounted for interaction between lateral buckling, and local buckling of the flanges and webs of I-sections.

Reviewing the tests on I-sections by Lukey and Adams (1969), and Kuhlmann (1989), and incorporating the predictions of Kato (1989), Daali and Korol (1995) proposed a limit that incorporated flange - web interaction for I-sections. To achieve a rotation capacity of $R = 4$, Daali and Korol (1995) put forward a Class 1 web limit that varied from $dlt\sqrt{f_y/250} = 7.63$ (for flange slenderness $blt\sqrt{f_y/250} = 9.85$), to $dlt\sqrt{f_y/250} = 76$ (for flange slenderness $blt\sqrt{f_y/250} = 9.85$), to $dlt\sqrt{f_y/250} = 92$ (for flange slenderness $blt\sqrt{f_y/250} = 5.45$).

Stewart and Sivakumaran (1997) used the finite strip method to extend the work of Dawe and Kulak (1981, 1984a, 1984b, 1986). Stewart and Sivakumaran produced Class 1, 2 and 3 limits for I-section beams that accounted for flange - web interaction.
2.5 MATERIAL PROPERTIES IN PLASTIC DESIGN

The tests on frames which verified the plastic method of design were performed on hot-rolled I-sections (Section 2.3). Section 2.1 illustrated the difference in stress-strain behaviour of hot-rolled sections compared to cold-formed sections. In particular, cold-formed steels are less ductile, and do not exhibit a plastic plateau before the onset of strain hardening.

There have been several investigations into the effect of material properties on the suitability of a section for plastic design. Specifically, there must be a sufficient amount of strain hardening available to allow for moment redistribution. The basic theory of plastic design detailed in Section 2.3 assumes that plastic hinges form on a zero length, with infinite strains and curvatures occurring at the hinge point. In reality, strains and curvatures must be finite, and the hinges are plastic zones of finite length. In most practical situations where there is a moment gradient, the moment has to exceed $M_p$ for the plastic zone to form properly. Hence some degree of strain hardening is required.

An analytical investigation by Lay and Smith (1965) on I-sections concluded that:

“it is necessary to postulate that a member being used in plastic design be comprised of an elasto-plastic-strain hardening material. If the more conventional assumption of elasto-plastic material is made, it will not be possible for the member to form a collapse mechanism”.

Hrennikoff (1965) performed tests on high strength aluminium I-section beams, as the aluminium exhibited only very limited strain hardening properties. It was concluded that plastic design would not be suitable for sections made from material with little or no strain hardening capability.

Adams and Galambos (1966) investigated the significance of the length of the yield plateau. Adams and Galambos performed an analytical study of a three span beam constructed from an I-section, and examined various parameters concerning the material properties. It was concluded that steel sections with larger yield plateaus would have a larger rotation capacity before the formation of the plastic hinge.
Lay (1965) developed the slenderness limit for plastic design for an I-section flange which was dependent on the strain hardening modulus, and the yield plateau. The higher the strain hardening modulus, the greater the resistance to flange local buckling in the plastic range. Consequently, certain material requirements for plastic design are specified in some design standards, such as AS 4100.

McDermott (1969) tested I-section beams of high strength ASTM A514 steel. Such steel has high yield stress (approximately 700 MPa), does not have a plastic plateau, and has a small strain hardening modulus compared to mild steels. Tests on seven beams indicated that the beams exhibited plastic rotation suitable for plastic design provided the flange slenderness limit of Lay (1965) was satisfied.

Kato (1990) analysed the behaviour of I-section beams under moment gradient and concluded that the plastic rotation capacity was dependent on the material properties, and that steels with high \( f_y / f_u \) ratios would not exhibit substantial plastic deformation.

As a result of the investigations mentioned above, current design standards enforce material property requirements for plastic design which are outlined below.

Clause 3.2.2.2 of Eurocode 3 has the following criteria:

\[
\begin{align*}
& f_u / f_y > 1.2 \\
& e_t > 15\% \\
& e_u / e_y > 20
\end{align*}
\]

where \( f_y \) is the yield stress, \( f_u \) is the tensile strength, \( e_y \) is the yield strain, \( e_u \) is the strain at the ultimate tensile strength, and \( e_t \) is the percentage strain (elongation) after failure measured across the fracture surface of a tensile coupon on a gauge length of \( 5.65 \sqrt{S_o} \) where \( S_o \) is the cross sectional area of the undeformed tensile coupon.
Clause 4.5.2 of AS 4100 specifies the following for plastic design:

- \( f_u / f_y \geq 1.2 \)
- \( e_t \geq 15\% \)
  - the length of the yield plateau is greater than 6\( e_y \)
  - the steel exhibits strain-hardening

and specifically excludes cold-formed steel, only permitting hot-formed, doubly symmetric I-sections to be used in plastic design.

The only material requirement for plastic design in AISC LRFD is that \( f_y \leq 450 \text{ MPa} \) (Clause A5.1). However cold-formed tubes must satisfy the requirements of ASTM A 500 (ASTM 1993) to be considered an approved steel by Clause A3.1.1a of AISC LRFD. Table 2 of ASTM A 500 specifies:

- \( e_{50} \geq 21\% \)

for Grade C steels (and slightly higher values for other steel grades), where \( e_{50} \) is the elongation on a 2 inch (50.8 mm) gauge length.

It may seem that the ASTM A 500 requirement of \( e_{50} \geq 21\% \) is more stringent than say the requirement in AS 4100 that \( e_t \geq 15\% \). However, the standard relating to cold-formed tubes in Australia, AS 1163 (Standards Australia 1991) specifies that the hollow sections must be strain aged before coupon testing, while under ASTM A 500, strain ageing is not required. Strain ageing reduces the ductility of steel.

High strength, cold-formed RHS in Australia are manufactured to Australian Standard AS 1163. RHS in Grade C450 (\( f_y = 450 \text{ MPa} \)) satisfying nominal mechanical properties of AS 1163, do not satisfy the material requirements for plastic design of Eurocode 3, and are specifically excluded for plastic design by AS 4100. It is not feasible to assess whether hollow sections to AS 1163 could be used plastically under the AISC LRFD, as it is not possible to compare the ductility requirements directly. However, cold-formed RHS to ASTM A 500 may be used in plastic design according to AISC LRFD.
2.6 KNEE JOINTS IN PORTAL FRAMES

Plastic hinges can be often required to form at the connections of frames, since joints are sometimes points of maximum moment. Local effects may reduce the rotation capacity of a connection compared to the rotation capacity of the member itself. In the typical portal frame shown in Figure 2.19, a plastic hinge may often form in the knee connection (sometimes referred to as an L-joint) between the column and the rafter. Under the gravity loads shown in Figure 2.19, the knee connection experiences a closing moment. However, under wind uplift loading, the connection experiences an opening moment. Depending on the loads on the structure, the knee connection may need to be able to form plastic hinges under both closing and opening moments.

![Figure 2.19: Column Rafter Knee Joint in a Portal Frame](image)

Tests on knee connections made from I-sections were carried out as part of the Lehigh University research program into plastic design. Design recommendations were made for straight corner connections (right angled), haunches, tapered haunches, and curved haunches. A summary is available in the *ASCE Commentary on Plastic Design in Steel* (ASCE 1961).

There has only been a limited number of tests on hollow section knee connections. Tests on knee joints are carried out by applying tension or compression to ends of the connections, and inducing an opening or closing moment respectively at the connection centre-line.
A single test on an unstiffened welded RHS, $254 \times 152 \times 9.5$, under compression (closing moment) indicated that the very stocky hollow section connection could reach the plastic moment and showed some rotation capacity (School of Civil Engineering 1975). Full details are unavailable, so it is unclear whether the joint was able to maintain the plastic moment for a sufficiently large rotation to be suitable for plastic design.

CIDECT has produced a design guide (Packer et al 1992) for RHS connections, which includes recommended details for stiffened and unstiffened welded knee connections suitable for a rigid portal frame. Typical stiffened and unstiffened welded knee connections are shown in Figure 2.20. The CIDECT design guide recommendations are based on tests by Mang et al (1980) which were performed on *hot-formed* hollow sections.

![Typical Welded Knee Connections (L-joints) in RHS](image)

**Figure 2.20: Typical Welded Knee Connections (L-joints) in RHS**

Mang *et al* tested the knee connections under compression (closing moment) only. It was found that ovalisation (or distortion) occurred across the junction of the connection in the unstiffened case. There was a non-linear distribution of stresses across the section at the junction. It was highlighted that the unstiffened connections exhibited shell behaviour at the junction of the two legs due to lack of stiffness at the junction of the two legs.
The installation of a stiffening plate ensured that the full moment capacity could be reached. The distribution of stresses across the section was linear, demonstrating typical engineering bending behaviour. It was notable that the cross-section exhibiting the greatest stresses and hence greatest bending moment was a finite distance away from the actual connection centre-line as shown in Figure 2.21. The cross-section of maximum moment coincided with the location of the local buckle. Traditional static analysis of the connection would put the location of highest moment at the connection centre-line, but such analysis ignores the complex stress distribution at the junction of the two legs, which is not pure bending.

![Cross section experiencing the highest bending moment](image)

Figure 2.21: Bending Moment in a Knee Connection
Mang et al gave validity limits for welded knee joints which are summarised in Table 2.7.

<table>
<thead>
<tr>
<th>Stiffened</th>
<th>Unstiffened</th>
</tr>
</thead>
<tbody>
<tr>
<td>( b \leq 400 \text{ mm} )</td>
<td>( b \leq 300 \text{ mm} )</td>
</tr>
<tr>
<td>( d \leq 400 \text{ mm} )</td>
<td>( d \leq 300 \text{ mm} )</td>
</tr>
<tr>
<td>( 0.3 \leq d/b \leq 3.0 )</td>
<td>( 0.3 \leq d/b \leq 3.0 )</td>
</tr>
<tr>
<td>( t \geq 3.0 \text{ mm} )</td>
<td>( t \geq 3.0 \text{ mm} )</td>
</tr>
<tr>
<td>( t \sqrt{2551f_y} \leq 30 \text{ mm}^a )</td>
<td>( t \sqrt{2551f_y} \leq 30 \text{ mm}^a )</td>
</tr>
<tr>
<td>((b/t) \sqrt{2551f_y} \leq 43^a )</td>
<td>((b/t) \sqrt{2551f_y} \leq 43^a )</td>
</tr>
</tbody>
</table>

Note: (a) Mang et al gave limits for 37 ksi (255 MPa) and 52 ksi (358 MPa) steel only. The limits of Mang et al have been modified to be included in a single limit incorporating the yield stress term.

Table 2.7: Validity Limits for RHS Welded Knee Connections

Additional requirements for the connection experiencing an axial force \( N^* \), a bending moment \( M^* \) and a shear force \( V^* \) are:

\[
\frac{N^*}{N_{ri}} + \frac{M^*}{M_{ri}} \leq \alpha_{knee}
\]  

(2.21)

where \( \alpha_{knee} \) is a stress reduction factor given by:

\( \alpha_{knee} = 1 \) for stiffened connections (a standard strength interaction between axial force and moment),

\( \alpha_{knee} < 1 \) for unstiffened connections and is a function of the specimen dimensions \((b/t\) and \(d/b\)) and the angle \( \theta \) between the legs of the connection (design charts are given to determine the value of \( \alpha_{knee} \)).

and \( N_{ri} = \) the design section capacity in either tension \((N_{i,Rd} \text{ (Eurocode 3, or } \phi N_i \text{ (AS 4100))})\) or compression \((N_{c,Rd} \text{ (Eurocode 3, or } \phi N_i \text{ (AS 4100))})\) (as appropriate),

\( M_{ri} = \) the design section moment capacity \((M_{c,Rd} \text{ (Eurocode 3, or } \phi M_s \text{ (AS 4100))})\).
It was specified in the CIDECT guide that the shear force in the connection should be limited by
\[ \frac{V^*}{V_p} \leq 0.5 \]  
(2.22)
where \( V_p = 2d t f_y \sqrt{3} \) and is the plastic shear yield capacity of the section.

The CIDECT design guide gives the following recommendations:

“... it could be expected that the rotation capacity of some unstiffened connections might be low, and in structures in which reasonable rotational capacity is required, a stiffened knee connection should be used... It is suggested to use, for unstiffened connections, only RHS members which satisfy plastic design requirements for rigid frames.”

In addition, there has been recent research funded by CIDECT on the static behaviour of knee joints made from circular hollow sections (CHS) (CIDECT Project 5BE: Mang et al 1997); and the fatigue behaviour of knee connections in both RHS and CHS (CIDECT Project 7S: Puthli et al 1998).

As an alternative for the knee connection, a haunch can be positioned on the inside of the knee as shown in Figure 2.22. The haunch should have the same width as the two legs of the connection, and is usually made by cutting some of the RHS used for the beams and columns of the structure. The CIDECT design guide states:

“Provided the haunch length is sufficient to ensure that the bending moment does not exceed the section capacity in either main member, the connection resistance will be adequate and does not require checking.”
Zhao and Hancock (1995a, 1995b) performed simple tension tests (not knee joints) of cold-formed RHS connected by butt welds and fillet welds. The fully plastic capacity of the sections could be sustained by both the Grade C350 and Grade C450 RHS. Once the plastic load was reached the high strength Grade C450 specimens displayed considerably less plastic deformation capacity than the Grade C350 specimens.
2.7 RESEARCH BY CIDECT

CIDECT is a French acronym for Comité International pour le Développement et l’Étude de la Construction Tubulaire (International Committee for the Development and Study of Tubular Construction). CIDECT was founded in the 1960s and initiates and participates in many research projects concerning tubular structures. The research projects by CIDECT which are relevant to this thesis have been referred to previously in this chapter. Readers may be interested in other areas of research by CIDECT which include stability, concrete filled tubes, connections, fatigue, and fire protection. Information can be obtained from CIDECT (CIDECT 1998).

2.8 RESEARCH AT THE UNIVERSITY OF SYDNEY

There has been a collective body of research into cold-formed RHS and SHS at The University of Sydney since the 1980s. The main aim has been to determine appropriate design rules for cold-formed RHS and SHS for use in the limit states version of the steel design specification in Australia (AS 4100). The specific research which is relevant to this thesis have been referred to previously in this chapter.

Some of the areas of previous research into cold-formed RHS and SHS at The University of Sydney are:

- Determination of the appropriate column curve for RHS and SHS columns (Key and Hancock 1985, Key 1988, and Key, Hasan and Hancock 1988),
- Examination of plate slenderness limits for columns and beams (Key and Hancock 1985, Hasan and Hancock 1988, and Zhao and Hancock 1991),
- Behaviour of SHS and RHS under concentrated force, bending moment, and combined bending moment and concentrated force (Zhao and Hancock 1992),
- Use of SHS in stressed-arch frames (Clarke and Hancock 1995a, 1995b),
- Lateral buckling of RHS (Zhao, Hancock and Trahair 1995),
- Welds in cold-formed RHS (Zhao and Hancock 1995a, 1995b),
- Member behaviour of hollow sections under axial force and bending moment (Sully and Hancock 1996), and
2.9 SUMMARY

This chapter has provided an introduction to the plastic design of steel frames, and examined the relevant previous research into plastic design, local buckling and slenderness limits, and the behaviour of knee joints in portal frames.

The slenderness limits for plastic design (Class 1) specified in various design standards have been compared. The “Compact” (Class 1) limit for webs in bending in the AISC LRFD Specification (AISC 1993) is considerably higher than the limit prescribed in comparable standards, such as Eurocode 3. Similarly, the “Compact” (Class 1) limit in the Australian Standard AS 4100 is slightly higher than the value prescribed in Eurocode 3. There are inconsistencies in the development of the Class 1 slenderness limit for webs in the AISC LRFD Specification.

The limits for webs in bending have been based on investigations of I-sections, but are applied to both I-sections and RHS.

While many researchers have indicated that there is interaction between the flange and web of I-sections, which affects local buckling, all current steel design codes prescribe flange and web slenderness limits which are independent of each other.
Chapter 3

BENDING TESTS OF COLD-FORMED RECTANGULAR HOLLOW SECTIONS

3.0 CHAPTER SYNOPSIS

The following chapter describes bending tests to examine the influence of web slenderness on the rotation capacity of cold-formed RHS. The results indicate that the Class 1 web slenderness limits in design standards, which are based on tests of I-sections, are unconservative for RHS. The rotation capacity is a function of both the web slenderness and flange slenderness, which is shown by iso-rotation curves. The common approach of specifying independent slenderness limits for the flange and web is inappropriate. A linear interaction formula between the web and flange slenderness for the Class 1 limits of RHS is proposed.
3.1 INTRODUCTION

Chapter 2 introduced the concept of various classes of cross-section. Sections are classified according to the effect local buckling has on the bending behaviour of a beam. The main factor that affects local buckling is the slenderness of the plate elements (b/t or d/t) that make up the section. For plastic design, a section must be able to reach its plastic moment and deform for a sufficiently large rotation, while maintaining the plastic moment. Such a section is known as a Class 1 section (referred to as “compact” or “plastic” by some design standards). Figure 3.1 demonstrates the different types of bending behaviour of steel sections.

![Figure 3.1: Types of Behaviour of RHS Beams (Eurocode 3 Classification)](image)

Steel design specifications give slenderness limits to classify cross-sections. The slenderness limits for webs were derived from tests of I-sections and plates, and are applied to both I-section and RHS webs. Zhao and Hancock (1991, 1992) observed inelastic web local buckling in some RHS with an aspect ratio of 2.0. The local buckling occurred at low rotation values for specimens with flange and web slenderness values below the limits set in current design standards for plastic design. The results of Zhao and Hancock (1991, 1992) provided the impetus for the current series of tests in higher aspect ratio RHS.
3.2 TEST SPECIMENS AND MATERIAL PROPERTIES

3.2.1 RHS PROPERTIES

A variety of cold-formed RHS was chosen for the test series. The RHS were manufactured by Tubemakers of Australia Limited (now known as BHP Structural and Pipeline Products). Two strength grades were selected, Grade C350L0 and C450L0 (nominal yield stress \( f_{yw} \) of 350 MPa and 450 MPa respectively and nominal tensile strength \( f_{ut} \) of 430 MPa and 500 MPa respectively), manufactured to Australian Standard AS 1163 (Standards Australia 1991a). The L0 suffix indicates the material has Charpy V-notch impact properties for use at temperatures below 0°C. The Grade C450 specimens are known as DuraGal sections, produced using a proprietary cold-forming and in-line galvanizing process. In-line galvanizing provides strength enhancement and corrosion protection. The samples were artificially strain aged in a furnace at 170°C for 20 minutes to avoid strain ageing with time during the test program. The original strip steel for both the C350 and C450 sections was TF 300 (TUBEFORM 300, nominal yield stress 300 MPa), produced by BHP Steel.

Figure 3.2 shows a typical RHS and defines the dimensions \( d, b, t, \) and \( r_e \) (depth, width, thickness, and external corner radius) of the section. The names given to each of the flat faces of the RHS: “weld”, “opposite”, “adjacent 1”, and “adjacent 2” is included. For most RHS, the longitudinal weld was located on one of the shorter faces (the flange). Since the weld was normally slightly off-centre, the adjacent face of the RHS which was closer to the weld is labelled “adjacent 1”.

The typical chemical composition of the cold-formed tubes is shown in Table 3.1.
### 3.2.2 TENSILE COUPON TESTS

Three coupons were taken from the centre of the flats of each tube. One was cut from the face opposite the weld, and one from each of the sides adjacent to the weld. Corner coupons were cut from selected RHS. The coupons were prepared and tested in accordance with AS 1391 (Standards Australia 1991b) in a 250 kN capacity INSTRON Universal Testing Machine, or a 300 kN SINTECH Testing Machine. Full details on the tensile coupon tests and the stress - strain curves are given in Appendix C. Typical stress - strain curves obtained from an RHS are shown in Figure 3.3.
Since the steel was cold-formed, the yielding was gradual, so that the reported yield stress was the 0.2% proof stress. The yield stress of the opposite face was on average 10% higher than that of the adjacent faces. The yield stress obtained from the corner coupons was on average 10% higher than that of the opposite face. The variability of material properties was a result of the cold-forming process and has been identified previously (CASE 1992a).

The average of the yield stress from both of the adjacent faces was assumed for the entire section, and used in the determination of plastic moment and slenderness values. The average of the Young’s modulus of elasticity from both of the adjacent faces was used in stiffness calculations. Values of $f_y$, $f_u$ and $e_i$ are included in Table 3.2(a) (later in this chapter) and are the means of the corresponding values obtained from the adjacent faces of the RHS.

All measured yield stresses and ultimate strengths reported are static values obtained by stopping the test for approximately one minute near the yield and ultimate loads. The static values are often considerably lower than commercial test values, which are usually dynamic.
3.2.3 FULL SECTION TENSILE TESTS

The method of determining the yield stress of cold-formed sections varies amongst various international specifications. AS 1163 permits the use of tension tests on coupons cut from the flats of the RHS for the determination of the yield stress of a section. ASTM A 500 (ASTM 1993) also allows coupon tests on the flat faces of a cold-formed RHS. Eurocode 3 states that the average yield stress ($f_{yz}$) of a cold-formed hollow section should be determined from a full size section tensile test or by applying a formula to the yield stress of the original strip.

Full section tensile tests were performed on two RHS to compare the various methods of determining the yield stress of a cold-formed RHS. The yield stress of the adjacent face was slightly lower than the yield stress of the whole section. The Eurocode 3 formula also gave values of average yield stress below the experimentally observed value for the entire section. Full details and results are given in Appendix D.

3.2.4 STUB COLUMN TESTS

Under bending, the stress in the webs of an RHS varies from compression at one edge to tension at the other, while under axial compression, the web experiences uniform compression. A small series of stub column tests was undertaken to examine the slenderness of RHS webs under uniform compression.

Full details of the test procedure and results are given in Appendix E. The main conclusion from the stub column tests is that the yield element slenderness limits (which delineate Slender and Non-Compact sections, or Class 3 and Class 4) for RHS webs under uniform compression are satisfactory.
RHS are not perfect rectangular specimens. The cold-forming process, welding, handling and other factors can introduce geometric imperfections. Local imperfections can affect the moment and rotation capacity (Key and Hancock 1993). Hence the local imperfections of the RHS were measured as part of the test program. The nature of the imperfection is important in subsequent finite element modelling of the bending tests. Full details on the procedure used and the imperfection profiles obtained can be found in Appendix F.

Most RHS exhibited a “bow-out” (as shown in Figure 3.4) or “bow-in” on each flat face of the section that was reasonably constant along the length of the RHS. It was most common for there to be a bow-out on each web, and a bow-in on each flange. The magnitude of the imperfection was approximately constant along the length of the specimen, that is, no significant wave-like imperfections along the length were detected. It was not possible to measure imperfections close to the loading plates that were welded to the specimen (refer to Figure 3.5). It was anticipated that the clamping and heating during the welding process would produce deformations in the RHS. Visual inspection of the specimens indicated that there was some imperfection induced into the specimen next to the welds connecting the loading plates to the RHS, but it was difficult to quantify the magnitude or shape of these imperfections.

![Figure 3.4: Typical “Bow-Out” Imperfection of an RHS (note: bow-out exaggerated)](image)
3.3 BENDING TEST PROCEDURE

The bending tests were performed in a 2000 kN capacity DARTEC testing machine, using a servo-controlled hydraulic ram. A diagram of the test set-up is shown in Figure 3.5. The four point bending arrangement provided a central region of uniform bending moment and zero shear force. Specimens were supported on half rounds resting on greased Teflon pads which simulated a set of simple supports. The members were loaded symmetrically at two points via a centrally loaded spreader beam. Three methods of transferring the force from the spreader beam to the RHS were employed.

The initial loading method, the “parallel plate” method, was used in previous tests (Hasan and Hancock 1988, Zhao and Hancock 1991) and involved welding plates parallel to the webs of the RHS beam as shown in Figure 3.5. A greased Teflon pad was placed between the bottom of the spreader beam and the half round, allowing for the half rounds to move due to the axial shortening of the beam caused by curvature, without inducing axial strain into the RHS. The half round bore upon a thick load transfer plate, which in turn transmitted the force to the loading plates. The loading plates and the load transfer plate were connected by bolts and an angle section, but the bolted angle connection did not transfer load and was for safety purposes only.

For the “perpendicular plate” loading method, the loading plates were welded perpendicular to the web of the RHS as shown in Figure 3.6. Two plates were welded on each web at each loading point. Care was taken to ensure full contact for the bearing between these plates and the load transfer plate. In all other respects the loading mechanism is the same as the first. The perpendicular plate method was used to see whether the parallel plate method inadvertently strengthened the section.

The “pin loading” system involved a steel pin through the neutral axis of the RHS. A hole was drilled through the RHS and short channel sections either side of the RHS and a pin inserted as shown in Figure 3.7. Load was transferred by bearing from the spreader beam to the channel sections, and in turn from the channel to the pin and the bending specimen. The pin allowed rotation of the beam. Teflon pads between the spreader beam and the channels allowed longitudinal movement. The webs of both the RHS and the channels were reinforced with additional plates to avoid local bearing failure.
Longitudinal strain gauges were placed mid-span on each flange, and linear displacement transducers were positioned mid-span and directly below the loading plates, enabling the curvature to be calculated from both the strain gauge and displacement measurements (Hasan and Hancock 1988). For some of the later tests, curvature was determined from the displacements only. Load, deflection, and strain measurements were recorded by a SPECTRA data acquisition system.

The lengths of the specimens were chosen to avoid lateral buckling (Zhao, Hancock and Trahair 1995), and shear failure in the end spans. For RHS with depth $d \geq 100$ mm, the length between the loading points ($L_1$) was 800 mm, and the distance between the supports ($L_2$) was 1700 mm. For sections with $d \leq 75$ mm, $L_1$ was 500 mm, and $L_2$ was 1300 mm.

A photograph of a typical specimen in the test rig is shown in Figure 3.8.
Figure 3.5: The Parallel Plate Loading Method for Plastic Bending Tests
Figure 3.6: The Perpendicular Plate Loading Method
Figure 3.7: The Pin Loading Method
3.4 RESULTS

For most sizes of RHS, more than one test was performed. There was a degree of variability in results for some samples in the same size range (eg 150 × 50 × 2.5 C450: BS04A - $R = 2.2$, BS04B - $R = 1.4$, BS04C - $R = 1.2$, and BS16A - $R = 1.1$; 100 × 50 × 2.0 C450: BS06A - $R = 1.3$, BS06B - $R = 0.8$, BS06C - $R = 0.8$, and BS17A - $R = 1.6$). While the absolute value of the variability in $R$ is reasonably small in the two previous cases, the percentage variability is significant. For other samples, the variability in the results for the same size is not significant.

All specimens, except the two 150 × 50 × 5.0 C450 specimens (BS01B and BS01C) and the 100 × 100 × 3.0 C450 (BS19A, BS19B and BS19C) samples, experienced web local buckling which produced a rapid shedding of load with increased deflection. Each web buckled and compatibility of rotation at the corner caused deformation of the flange. A typical buckled specimen is shown in Figure 3.9. In all cases, the buckle formed adjacent to one of the loading plates. BS01B and BS01C exhibited large deflections and an inelastic lateral deformation was
observed at high curvatures ($\kappa/\kappa_p$) greater than 6. There was no sudden unloading associated with the lateral deflection. BS19A, BS19B and BS19C were SHS and, as expected, failed by flange local buckling. Specimen BS08C was not loaded to failure. No specimen failed due to insufficient material ductility.

![Specimen with Locally Buckled Web](image)

Figure 3.9: Specimen with Locally Buckled Web

The results of the plastic bending tests are presented in Tables 3.2(a) and 3.2(b). Table 3.2 lists the nominal section size, the measured dimensions and measured yield stress, the ratio $M_{max}/M_p$ and $R$. $M_{max}$ is the maximum static moment reached during the test and $M_p$ is the plastic moment of the section based on the measured dimensions and the mean measured yield stress of the adjacent faces. The static moment was obtained when the test machine was halted for approximately one minute in the vicinity of the ultimate load. The nominal plastic moment ($M_{pn}$) (based on nominal properties) is listed. The individual moment - curvature graphs for each test are included in Appendix A.
Four typical non-dimensional moment-curvature curves are shown in Figure 3.1, and are representative of the performance of the test specimens. Class 1, Class 2, and Class 3 behaviour, as shown in Figure 3.1, were observed. No specimen behaved as a Class 4 section, but the behaviour of a typical slender member is included in Figure 3.1 for completeness. The individual moment-curvature graphs for each test are included in Appendix A.

Curvature (κ) was calculated in two ways. The readings of strain on the top and bottom flange of the RHS can be used to determine curvature as indicated in Figure 3.10. The strain method gives the curvature at the strain gauge location only, which is the mid-span of the beam.

$$\tan \kappa = \frac{e_1 + e_2}{d}$$

Figure 3.10: Calculation of Curvature from Strains

Alternatively, the average curvature over a finite length of the beam can be calculated from displacement values. As the moment is constant in the beam between the loading plates, the curvature is constant in the same region. Constant curvature in a segment means that the segment forms an arc of a circle (constant radius), and hence κ can be determined as shown in Figure 3.11. For the beams tested, the length over which the curvature was calculated was $L_1$, the length between the loading plates.
Figure 3.11: Determination of Curvature from Deflection

Figure 3.12 compares the two methods of curvature determination for one of the test specimens. The two methods give nearly identical curvature values up to the point of local buckling. Once the beam has locally buckled, the curvature is not constant in the centre region of the beam, and curvature becomes concentrated at the location of the buckle, as illustrated in the photograph of the buckled beam in Figure 3.9. The buckled section has a reduced moment capacity, and the beam is required to shed load. Since the strain gauge location is at mid-span, not at the buckle location, the elastic unloading at the strain gauge location is seen as the load reduces. The curvature based on deflection after buckling is an average curvature over the length of the beam \( L \). If the deflection curvature was calculated over a shorter length including the buckled region, the post buckling curvatures would be notably higher, resulting in a higher value of rotation capacity.
The web slenderness is calculated slightly differently in AS 4100, Eurocode 3, and AISC LRFD as demonstrated in Table 2.5. Figure 3.13 displays the rotation capacity versus web slenderness for AISC LRFD and differentiates between the different loading methods. Figure 3.14 graphs the results with respect to AS 4100, and distinguishes the different aspect ratios of the specimens. The results calculated according to Eurocode 3 are shown in Figure 3.15, which compares the results of the different steel grades. In each figure, the slenderness is calculated with the measured dimensions and mean yield stress from the adjacent faces.
Speci-

Cut from

d

b

t

men
BS01B
BS01C
BS02B
BS02C
BS02A
BF02
BS03A
BS03B
BS03C
BS04B
BS04C
BS04A
BS16A
BS05A
BS05B
BS05C
BS06B
BS06C
BS06A
BS17A
BS07B
BS07C
BS08B
BS08C
BS09B
BS09C
BS09A
BS10B
BS10C
BS11B
BS11C
BS20A
BS20B
BS12B
BS12C
BS13B
BS13C
BS13A
BS21A
BS19A
BS19B
BS19C
BJ07
BF01

section
150×50×5.0 C450
150×50×5.0 C450
150×50×4.0 C450
150×50×4.0 C450
150×50×4.0 C450
150×50×4.0 C450
150×50×3.0 C450
150×50×3.0 C450
150×50×3.0 C450
150×50×2.5 C450
150×50×2.5 C450
150×50×2.5 C450
150×50×2.5 C450
150×50×2.3 C450
150×50×2.3 C450
150×50×2.3 C450
100×50×2.0 C450
100×50×2.0 C450
100×50×2.0 C450
100×50×2.0 C450
75×50×2.0 C450
75×50×2.0 C450
75×25×2.0 C450
75×25×2.0 C450
75×25×1.6 C450
75×25×1.6 C450
75×25×1.6 C450
75×25×1.6 C350
75×25×1.6 C350
150×50×3.0 C350
150×50×3.0 C350
150×50×3.0 C350
150×50×3.0 C350
100×50×2.0 C350
100×50×2.0 C350
125×75×3.0 C350
125×75×3.0 C350
125×75×3.0 C350
125×75×2.5 C350
100×100×3.0 C450
100×100×3.0 C450
100×100×3.0 C450
150×50×4.0 C350
150×50×4.0 C350

(mm)
151.04
150.92
150.43
150.44
150.42
150.21
150.47
150.79
150.80
150.43
150.39
150.40
150.31
150.65
150.51
150.37
100.45
100.49
100.46
100.45
75.48
75.63
75.31
75.33
75.24
74.90
74.98
75.27
75.19
150.46
150.50
150.45
150.38
100.91
100.83
125.56
125.40
125.40
125.40
100.43
100.53
100.53
150.32
150.39

(mm)
50.25
50.41
50.27
50.40
50.11
50.16
50.22
50.01
50.34
50.15
50.41
50.23
50.40
50.64
50.57
50.70
50.70
50.55
50.24
50.22
50.10
50.31
25.28
25.23
25.12
25.20
25.08
25.12
25.25
50.13
50.19
50.51
50.51
50.43
50.52
75.84
75.74
75.56
75.10
100.27
100.33
100.25
50.21
50.57

re

d 2rH b 2rH

t
t
(mm) (mm)
4.92 9.9 26.67 6.19
4.90 10.7 26.43 5.92
3.92 6.8 34.91 9.35
3.87 7.3 35.10 9.25
3.89 7.3 34.92 9.13
3.89 5.4 35.84 10.12
2.97 5.9 46.69 12.94
2.95 5.8 47.18 13.02
2.96 5.7 47.09 13.16
2.60 4.6 54.32 15.75
2.57 4.6 54.94 16.04
2.59 4.8 54.34 15.69
2.64 5.3 52.92 15.08
2.25 4.6 62.87 18.42
2.28 4.2 62.33 18.50
2.26 4.8 62.29 18.19
2.06 3.8 45.07 20.92
2.07 3.9 44.78 20.65
2.04 4.7 44.64 20.02
2.04 3.4 45.91 21.28
1.94 4.4 34.37 21.29
1.95 4.4 34.27 21.29
1.98 3.7 34.30 9.03
1.95 4.0 34.53 8.84
1.54 3.1 44.83 12.29
1.54 3.4 44.22 11.95
1.56 3.9 43.06 11.08
1.55 3.4 44.17 11.82
1.56 3.4 43.84 11.83
3.00 6.2 46.02 12.58
2.96 6.5 46.45 12.56
3.00 6.8 45.62 12.30
3.00 6.3 45.93 12.64
2.06 3.6 45.49 20.99
2.05 3.8 45.48 20.94
2.92 6.6 38.48 21.45
2.93 6.9 38.09 21.14
2.91 7.1 38.21 21.09
2.53 3.9 46.48 26.60
2.88 5.2 31.26 31.20
2.91 5.0 31.11 31.04
2.86 5.2 31.51 31.42
3.90 7.9 34.49 8.82
3.85 7.5 35.17 9.24

fy

fu

ef

(MPa)
441
441
457
457
457
423
444
444
444
446
446
446
440
444
444
444
449
449
449
423
411
411
457
457
439
439
439
422
422
370
370
382
382
400
400
397
397
397
374
445
445
445
349
410

(MPa)
495
495
527
527
527
480
513
513
513
523
523
523
506
518
518
518
499
499
499
479
484
484
514
514
511
511
511
456
456
429
429
430
430
450
450
449
449
449
441
502
502
502
437
464

(%)
17.1
17.1
19.3
19.3
19.3
28.0
18.2
18.2
18.2
15.7
15.7
15.7
26.5
17.3
17.3
17.3
11.9
11.9
11.9
25.8
12.5
12.5
12.6
12.6
19.5
19.5
19.5
16.6
16.6
30.2
30.2
30.7
30.7
20.1
20.1
26.7
26.7
26.7
34.5
24.3
24.3
24.3
36.0
35.0

Table 3.2(a): Summary of Results of Plastic Bending Tests

87


<table>
<thead>
<tr>
<th>Specimen</th>
<th>Cut from section</th>
<th>Loading method</th>
<th>$M_{pn}$ (kNm)</th>
<th>$M_p$ (kNm)</th>
<th>$M_{max}$ (kNm)</th>
<th>$M_{\ast\ast\ast}$</th>
<th>R ratio</th>
<th>Rotation</th>
</tr>
</thead>
<tbody>
<tr>
<td>BS01B</td>
<td>150x50x5.0 C450</td>
<td>Parallel</td>
<td>35.49</td>
<td>35.51</td>
<td>43.80</td>
<td>1.23</td>
<td>&gt;13.0</td>
<td></td>
</tr>
<tr>
<td>BS01C</td>
<td>150x50x5.0 C450</td>
<td>Parallel</td>
<td>35.49</td>
<td>35.14</td>
<td>41.10</td>
<td>1.17</td>
<td>&gt;9.0</td>
<td></td>
</tr>
<tr>
<td>BS02B</td>
<td>150x50x4.0 C450</td>
<td>Parallel</td>
<td>29.45</td>
<td>30.32</td>
<td>38.60</td>
<td>1.27</td>
<td>6.6</td>
<td></td>
</tr>
<tr>
<td>BS02C</td>
<td>150x50x4.0 C450</td>
<td>Parallel</td>
<td>29.45</td>
<td>29.86</td>
<td>35.50</td>
<td>1.19</td>
<td>7.7</td>
<td></td>
</tr>
<tr>
<td>BF02</td>
<td>150x50x4.0 C450</td>
<td>Parallel</td>
<td>29.44</td>
<td>28.06</td>
<td>33.02</td>
<td>1.17</td>
<td>9.5</td>
<td></td>
</tr>
<tr>
<td>BS03A</td>
<td>150x50x3.0 C450</td>
<td>Parallel</td>
<td>23.14</td>
<td>22.76</td>
<td>26.20</td>
<td>1.15</td>
<td>2.7</td>
<td></td>
</tr>
<tr>
<td>BS03B</td>
<td>150x50x3.0 C450</td>
<td>Parallel</td>
<td>23.14</td>
<td>22.68</td>
<td>26.30</td>
<td>1.16</td>
<td>2.3</td>
<td></td>
</tr>
<tr>
<td>BS03C</td>
<td>150x50x3.0 C450</td>
<td>Parallel</td>
<td>23.14</td>
<td>22.81</td>
<td>25.80</td>
<td>1.13</td>
<td>2.9</td>
<td></td>
</tr>
<tr>
<td>BS04B</td>
<td>150x50x2.5 C450</td>
<td>Parallel</td>
<td>19.58</td>
<td>20.30</td>
<td>20.80</td>
<td>1.02</td>
<td>1.4</td>
<td></td>
</tr>
<tr>
<td>BS04C</td>
<td>150x50x2.5 C450</td>
<td>Parallel</td>
<td>19.58</td>
<td>20.13</td>
<td>20.20</td>
<td>1.00</td>
<td>1.2</td>
<td></td>
</tr>
<tr>
<td>BS04A</td>
<td>150x50x2.5 C450</td>
<td>Perp</td>
<td>19.58</td>
<td>20.18</td>
<td>21.80</td>
<td>1.10</td>
<td>2.2</td>
<td>1.39</td>
</tr>
<tr>
<td>BS16A</td>
<td>150x50x2.5 C450</td>
<td>Pin</td>
<td>19.58</td>
<td>20.28</td>
<td>22.60</td>
<td>1.11</td>
<td>1.1</td>
<td>0.91</td>
</tr>
<tr>
<td>BS05A</td>
<td>150x50x2.3 C450</td>
<td>Parallel</td>
<td>18.13</td>
<td>17.73</td>
<td>17.40</td>
<td>0.98</td>
<td>0.0</td>
<td></td>
</tr>
<tr>
<td>BS05B</td>
<td>150x50x2.3 C450</td>
<td>Parallel</td>
<td>18.13</td>
<td>17.96</td>
<td>18.20</td>
<td>1.01</td>
<td>0.6</td>
<td></td>
</tr>
<tr>
<td>BS05C</td>
<td>150x50x2.3 C450</td>
<td>Parallel</td>
<td>18.13</td>
<td>17.71</td>
<td>17.30</td>
<td>0.98</td>
<td>0.0</td>
<td></td>
</tr>
<tr>
<td>BS06B</td>
<td>100x50x2.0 C450</td>
<td>Parallel</td>
<td>8.33</td>
<td>8.70</td>
<td>9.30</td>
<td>1.07</td>
<td>0.8</td>
<td></td>
</tr>
<tr>
<td>BS06C</td>
<td>100x50x2.0 C450</td>
<td>Parallel</td>
<td>8.33</td>
<td>8.71</td>
<td>8.80</td>
<td>1.01</td>
<td>0.8</td>
<td></td>
</tr>
<tr>
<td>BS06A</td>
<td>100x50x2.0 C450</td>
<td>Perp</td>
<td>8.33</td>
<td>8.47</td>
<td>9.30</td>
<td>1.07</td>
<td>1.3</td>
<td>1.28</td>
</tr>
<tr>
<td>BS17A</td>
<td>100x50x2.0 C450</td>
<td>Pin</td>
<td>8.33</td>
<td>7.91</td>
<td>8.75</td>
<td>1.08</td>
<td>1.6</td>
<td>1.44</td>
</tr>
<tr>
<td>BS07B</td>
<td>75x50x2.0 C450</td>
<td>Parallel</td>
<td>5.38</td>
<td>4.80</td>
<td>5.00</td>
<td>1.04</td>
<td>1.7</td>
<td></td>
</tr>
<tr>
<td>BS07C</td>
<td>75x50x2.0 C450</td>
<td>Parallel</td>
<td>5.38</td>
<td>4.86</td>
<td>4.96</td>
<td>1.02</td>
<td>1.9</td>
<td></td>
</tr>
<tr>
<td>BS08B</td>
<td>75x25x2.0 C450</td>
<td>Parallel</td>
<td>3.74</td>
<td>3.82</td>
<td>4.24</td>
<td>1.11</td>
<td>5.7</td>
<td></td>
</tr>
<tr>
<td>BS08C</td>
<td>75x25x2.0 C450</td>
<td>Parallel</td>
<td>3.74</td>
<td>3.75</td>
<td>4.25</td>
<td>1.13</td>
<td>*</td>
<td></td>
</tr>
<tr>
<td>BS09B</td>
<td>75x25x1.6 C450</td>
<td>Parallel</td>
<td>3.07</td>
<td>2.84</td>
<td>3.16</td>
<td>1.08</td>
<td>2.2</td>
<td></td>
</tr>
<tr>
<td>BS09C</td>
<td>75x25x1.6 C450</td>
<td>Parallel</td>
<td>3.07</td>
<td>2.82</td>
<td>3.25</td>
<td>1.13</td>
<td>2.5</td>
<td></td>
</tr>
<tr>
<td>BS09A</td>
<td>75x25x1.6 C450</td>
<td>Perp</td>
<td>3.07</td>
<td>2.83</td>
<td>3.10</td>
<td>1.09</td>
<td>2.5</td>
<td>1.04</td>
</tr>
<tr>
<td>BS10B</td>
<td>75x25x1.6 C350</td>
<td>Parallel</td>
<td>2.39</td>
<td>2.81</td>
<td>2.90</td>
<td>1.03</td>
<td>1.9</td>
<td></td>
</tr>
<tr>
<td>BS10C</td>
<td>75x25x1.6 C350</td>
<td>Parallel</td>
<td>2.39</td>
<td>2.82</td>
<td>2.82</td>
<td>1.00</td>
<td>2.6</td>
<td></td>
</tr>
<tr>
<td>BS11B</td>
<td>150x50x3.0 C350</td>
<td>Parallel</td>
<td>18.00</td>
<td>19.11</td>
<td>23.20</td>
<td>1.21</td>
<td>4.1</td>
<td></td>
</tr>
<tr>
<td>BS11C</td>
<td>150x50x3.0 C350</td>
<td>Parallel</td>
<td>18.00</td>
<td>18.84</td>
<td>21.70</td>
<td>1.15</td>
<td>3.6</td>
<td></td>
</tr>
<tr>
<td>BS20A</td>
<td>150x50x3.0 C350</td>
<td>Perp</td>
<td>18.00</td>
<td>19.69</td>
<td>23.20</td>
<td>1.18</td>
<td>3.2</td>
<td>0.87</td>
</tr>
<tr>
<td>BS20B</td>
<td>150x50x3.0 C350</td>
<td>Perp</td>
<td>18.00</td>
<td>19.79</td>
<td>23.90</td>
<td>1.21</td>
<td>3.6</td>
<td>0.95</td>
</tr>
<tr>
<td>BS12B</td>
<td>100x50x2.0 C350</td>
<td>Parallel</td>
<td>6.48</td>
<td>7.77</td>
<td>7.70</td>
<td>1.00</td>
<td>1.2</td>
<td></td>
</tr>
<tr>
<td>BS12C</td>
<td>100x50x2.0 C350</td>
<td>Parallel</td>
<td>6.48</td>
<td>7.75</td>
<td>7.75</td>
<td>1.00</td>
<td>1.3</td>
<td></td>
</tr>
<tr>
<td>BS13B</td>
<td>125x75x3.0 C350</td>
<td>Parallel</td>
<td>16.54</td>
<td>18.42</td>
<td>18.90</td>
<td>1.03</td>
<td>1.5</td>
<td></td>
</tr>
<tr>
<td>BS13C</td>
<td>125x75x3.0 C350</td>
<td>Parallel</td>
<td>16.54</td>
<td>18.40</td>
<td>19.10</td>
<td>1.04</td>
<td>1.6</td>
<td></td>
</tr>
<tr>
<td>BS13A</td>
<td>125x75x3.0 C350</td>
<td>Perp</td>
<td>16.54</td>
<td>18.20</td>
<td>18.70</td>
<td>1.03</td>
<td>1.4</td>
<td>0.94</td>
</tr>
<tr>
<td>BS21A</td>
<td>125x75x2.5 C350</td>
<td>Pin</td>
<td>13.99</td>
<td>15.32</td>
<td>16.25</td>
<td>1.06</td>
<td>1.1</td>
<td></td>
</tr>
<tr>
<td>BS19A</td>
<td>100x100x3.0 C450</td>
<td>Parallel</td>
<td>18.54</td>
<td>17.86</td>
<td>18.16</td>
<td>1.02</td>
<td>0.8</td>
<td></td>
</tr>
<tr>
<td>BS19B</td>
<td>100x100x3.0 C450</td>
<td>Pin</td>
<td>18.54</td>
<td>18.09</td>
<td>17.30</td>
<td>0.95</td>
<td>0.0</td>
<td>0.94</td>
</tr>
<tr>
<td>BS19C</td>
<td>100x100x3.0 C450</td>
<td>Perp</td>
<td>18.54</td>
<td>17.78</td>
<td>18.40</td>
<td>1.03</td>
<td>0.9</td>
<td>1.06</td>
</tr>
<tr>
<td>BJ07</td>
<td>150x50x4.0 C350</td>
<td>Parallel</td>
<td>22.90</td>
<td>22.80</td>
<td>29.70</td>
<td>1.28</td>
<td>12.9</td>
<td></td>
</tr>
<tr>
<td>BF01</td>
<td>150x50x4.0 C350</td>
<td>Parallel</td>
<td>18.54</td>
<td>26.63</td>
<td>31.78</td>
<td>1.19</td>
<td>10.7</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>Mean</th>
<th>Standard deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1.09</td>
<td>0.09</td>
</tr>
</tbody>
</table>

**Note:** * Specimen BS08C not tested to failure.

Table 3.2(b): Summary of Results of Plastic Bending Tests
Figure 3.13: Rotation Capacity Slenderness Relationship (AISC LRFD)
(“Compact” in AISC LRFD is equivalent to Class 1)

Figure 3.14: Rotation Capacity Slenderness Relationship (AS 4100)
(“Compact” in AS 4100 is equivalent to Class 1)
3.5 DISCUSSION

The results in Figures 3.13, 3.14 and 3.15 indicate that the current web slenderness limits in AISC LRFD, AS 4100 and Eurocode 3 respectively for Compact or Class 1 sections are unconservative. A number of sections, which are currently classified as Compact or Class 1, demonstrated insufficient rotation capacity for plastic design. The Compact (Class 1) limit for the AISC LRFD specification appears to be the most unconservative. Many of the sections classified by AS 4100 and Eurocode 3 as Compact or Class 1 failed to reach the required rotations of $R = 3$ or $R = 4$ as shown in Figures 3.14 and 3.15.

As indicated in Figure 3.14, several of the sections (100 × 50 × 2.0 C450, 75 × 50 × 2.0 C450, and 125 × 75 × 3.0 C450) have Non-Compact flanges according to the AS 4100 flange limit. However, the sections have Compact or Class 1 flanges with respect to the AISC LRFD and Eurocode 3 limits. The sections just exceed the Compact (Class 1) flange limit ($\lambda = 30$ defined in AS 4100) and it is reasonable to expect that they would behave almost identically to sections with Compact (Class 1) flanges.
Figure 3.13 distinguishes between the loading methods. There is a column in Table 3.2(b), called the “Rotation ratio”. The rotation ratio is defined as \( \frac{\kappa_{\text{pin}}}{\kappa_{\text{parallel}}} \), i.e., the rotation ratio compares the curvature when the moment falls below \( M_p \) of the specimen tested under the pin or perpendicular loading method, with the curvature of the specimen tested by the parallel plate method. Section 6.9.2.4 details why it is more appropriate to compare values of \( \kappa_{\text{pin}} \), as opposed to \( R \). On average the perpendicular plate method increased the curvature at buckling by 10%. For the pin loading method, the corresponding increase was 5%. The averages have been boosted by three tests (BS04A - 150 × 50 × 2.5 C450 - perpendicular, BS06A - 100 × 50 × 2.0 C450 - perpendicular, and BS17A - 100 × 50 × 2.0 C450 - pin) which had rotation ratios of 1.39, 1.28, and 1.44 respectively. For all the other tests, there was no significant effect on the rotation capacity. The three aforementioned specimens exhibit Non-Compact or Class 2 behaviour (\( M_p \) is reached but the rotation capacity is insufficient). The loading method did not change the classification of the behaviour from Class 2 to Class 1. \( M_{\text{max}} \) was barely affected by altering the loading arrangement. It can be concluded that the loading method did not have a major affect on rotation capacity.

Figure 3.14 compares the different aspect ratios \( \frac{d}{b} \) of the RHS with respect to web slenderness and rotation capacity. The majority of sections, which had an aspect ratio of 3, exhibit a relationship between rotation capacity and web slenderness, which is close to linear, or perhaps bilinear. For sections with the same web slenderness, the RHS with higher aspect ratios (which corresponds to a lower flange slenderness) exhibit higher rotation capacities. For example, one may compare the 150 × 50 × 3 C450 RHS \( (d/l = 50, b/t = 17, d/l = 3, R \approx 2.5) \), with the 100 × 50 × 2 C450 RHS \( (d/l = 50, b/t = 25, d/l = 2, R \approx 1.0) \). The different rotation capacity for sections with the same web slenderness indicates that both the web slenderness and flange slenderness affect the rotation capacity of RHS.

Figure 3.15 distinguishes between the different steel grades. Separate linear regression lines for the Grade C350 and Grade C450 are plotted in Figure 3.15. It can be seen from the linear regression lines that to achieve a rotation capacity of approximately \( R = 4 \), a similar web slenderness is required from both Grade C350 and Grade C450 RHS. The regression lines show that at higher web slenderness, more rotation capacity is available from the Grade C450 RHS compared to the Grade C350 specimens of the same slenderness. However the range of results for the Grade C350 specimens is not as large as for the Grade C450 samples, and there were no
Grade C350 samples tested in the higher slenderness range. Hence any trends shown by the Grade C350 results should not be considered conclusive. Zhao and Hancock (1991) observed similar behaviour with respect to the flange slenderness of RHS.

The most appropriate way to illustrate the interaction between the flange and the web is by means of iso-rotation curves, which plot contours of equal rotation capacity against the flange and web slenderness of each section. Figure 3.16 shows approximate iso-rotation curves for the RHS tested. Since some tests were repeated, the rotation capacity shown in Figure 3.16 is the average rotation from the two (or three) tests performed on a given RHS size. Only the results of sections tested by the parallel plate method are shown in Figure 3.16. To incorporate the effects of flange buckling, the results of Hasan and Hancock (1988) and Zhao and Hancock (1991) are included in Figure 3.16. Most of the tests by Hasan and Hancock, and Zhao and Hancock were performed on SHS, with an aspect ratio of 1.0, and several RHS with an aspect ratio of 2.0 were tested. All tests used the parallel plate method of loading.

![Figure 3.16: Approximate Iso-Rotation Plot (AS 4100)](image-url)
Figure 3.16 distinguishes the two steel grades. It can be seen that near the contour $R = 4$, there is not much difference between the two steel grades. However, in the higher contour regions, the Grade C350 RHS achieve a higher rotation than the Grade C450 RHS of similar flange and web slenderness. Hence for most stocky sections, steel grade does appear to affect the rotation capacity, but in the vicinity of $R = 4$, the steel grade does not have considerable effect on $R$. However, as the range Grade C350 specimens is not as wide as for the Grade C450 samples, and there were no Grade C350 samples tested in the higher slenderness range, any trends shown by the Grade C350 results should not be considered conclusive.

The iso-rotation curves in Figure 3.16 give the best indication of the relationship between the flange and web slenderness, and rotation capacity. The nature of the rotation contours highlight the interaction of flange and web slenderness. The sections with higher aspect ratio have a relatively stiffer flange which provides restraint against web local buckling. A section with a similar web but having a less stiff flange has less resistance against local buckling and hence a lower rotation capacity. The current “rectangular” independent flange and web limits, which assume no interaction, are inappropriate for the type of behaviour observed in the tests.

The results shown in the iso-rotation curves have to be interpreted to give design recommendations for new slenderness limits. The iso-rotation contours in Figure 3.16 were drawn to fit the trend of the results without any rational statistical basis. Figure 3.14 does show, that for a given aspect ratio, there may be a linear relationship between web slenderness and rotation capacity. Figure 3.17 plots rotation capacity against web slenderness for aspect ratios of $d/b = 3.0$, 2.0, 1.66 and 1.0 respectively, and incorporates the results of Hasan and Hancock (1988) and Zhao and Hancock (1991). In Figure 3.17 the slenderness is calculated in accordance with AS 4100. The relationships for Eurocode 3 and AISC LRFD are not shown, but are similar.
Particularly for sections with $dl/b = 2.0$ and $dl/b = 1.66$, it is difficult to establish the type of trend (ie linear, logarithmic etc) between web slenderness and rotation capacity, since there are few data points. For sections with $dl/b = 3.0$, a single least-squares regression line would not give the best approximation of the results. A bi-linear approximation, as indicated in Figure 3.18, fits the experimental data better than a linear regression. The two parts of the bi-linear approximations were chosen by separate least squares regression for sections with higher slenderness and lower slenderness separately (for $dl/b = 3.0$ and $dl/b = 1.0$ only). The large range of experimental results for $dl/b = 3.0$ and $dl/b = 1.0$ displayed similarly shaped bi-linear relationships between web slenderness - rotation capacity. It is reasonable to assume that bi-linear curves would also be appropriate for $dl/b = 2.0$ and $dl/b = 1.66$ despite the limited experimental data, as indicated in Figure 3.17.

Figure 3.17: Web Slenderness against Rotation Capacity incorporating Hasan and Hancock (1988) and Zhao and Hancock (1991)
Sully (1996) encountered a similar bi-linear relationship between the flange slenderness of SHS and the critical buckling strain (similar to rotation capacity).

In Figure 3.18, the bi-linear approximation gives a relationship between the rotation capacity and the web slenderness, and the corresponding flange slenderness is calculated from the aspect ratio. From the set of flange and web slenderness and rotation capacity values, it is possible to construct iso-rotation curves based on the bi-linear approximation for each aspect ratio.

Iso-rotation plots, based on the bi-linear approximation for each aspect ratio, are plotted for each steel design specification: AS 4100, Eurocode 3, and AISC LRFD, in Figures 3.19, 3.20, and 3.21.
Figure 3.19: Iso-Rotation Plot and Suggested New Compact Limit for AS 4100

Figure 3.20: Iso-Rotation Plot and Suggested New Compact Limit for Eurocode 3
To be suitable for plastic design, sections must demonstrate sufficient rotation capacity once the plastic moment has been reached. Section 2.2.5 noted that AS 4100 used \( R = 4 \), and AISC LRFD and Eurocode 3 used \( R = 3 \) as the rotation requirement for plastic design. Hence, it is appropriate to examine the contour lines representing \( R = 3 \), or \( R = 4 \).

Figures 3.19, 3.20, and 3.21 indicate that for sections with stocky webs, which buckle predominately in the flange, the Compact or Class 1 flange limits reasonably accurately model the behaviour for RHS and SHS. The results suggest that a Compact (Class 1) limit of \( (b-2t)/t\sqrt{(f_y/250)} = 35 \) would be suitable in AS 4100. Zhao and Hancock (1991) used the same data to propose a limit of \( (b-2t)/t\sqrt{(f_y/250)} = 30 \), but Zhao and Hancock’s recommendation was based on the lower bound of the test results, while the contour lines in Figure 3.19 come from the mean of the test results, giving a higher limit. The AISC LRFD imposed a lower limit for RHS flanges, \( (b-2r_e)/t\sqrt{(f_y/E)} = 0.939 \), in the 1997 Steel Hollow Section Supplement, compared to the original value of \( (b-2r_e)/t\sqrt{(f_y/E)} = 1.12 \) in the 1993 Specification which currently only applies to box sections. The results in Figure 3.21 suggest that the previous flange limit of \( (b-2r_e)/t\sqrt{(f_y/E)} = 1.12 \) was adequate to provide a rotation capacity of \( R = 3 \) for the cold-formed
SHS tested in this thesis, and by Hasan and Hancock (1988) and Zhao and Hancock (1991). However, the limit in the 1997 Steel Hollow Section Supplement was lowered to match the values in the Canadian Standard CSA-S16.1, which was based on the results of Korol and Hudoba (1972).

Figures 3.19, 3.20, and 3.21 indicate that there is a significant region for members with more slender webs in which all three design specifications are unconservative. The contour line for $R = 3$, or $R = 4$ as the flange slenderness increases, the web slenderness would need to decrease to ensure that the section has adequate rotation capacity.

A simple bi-linear formula to account for flange-web interaction may be appropriate to classify sections as suitable for plastic design (Class 1). Such a line is shown in Figure 3.19 and Equation 3.1 for AS 4100. Similar bi-linear proposals Eurocode 3 and the AISC LRFD are indicated in Figures 3.20 and 3.21, and Equations 3.2 and 3.3. The bi-linear curve has been chosen initially due to its simplicity, while still accounting for the flange-web interaction. The proposed limit may be overly conservative, particularly in the region of the “bump” on the $R = 3$ or $R = 4$ contour. A multi-linear curve or an elliptical Class 1 limit may also be appropriate, and could be less conservative than the initially proposed bi-linear limit.

\[
\lambda_{\phi} \leq 70 - \frac{5\lambda_{\phi}}{6}, \quad \lambda_{\phi} \leq 30 \quad \text{ (AS 4100: } R = 4) \] \hspace{1cm} (3.1)

\[
\lambda_{\phi} \leq 72 - \frac{9\lambda_{\phi}}{11}, \quad \lambda_{\phi} \leq 33 \quad \text{ (Eurocode 3: } R = 3) \] \hspace{1cm} (3.2)

\[
\lambda_{\phi} \leq 2.5 - \frac{\lambda_{\phi}}{0.939}, \quad \lambda_{\phi} \leq 0.939 \quad \text{ (AISC LRFD: } R = 3) \] \hspace{1cm} (3.3)

Section 2.4.2 outlined the definition of “slenderness” in various design codes. With regards to RHS, two definitions of plate width are most common:

- the “flat width” definition $(d-2r_e$ or $b-2r_e$), used in AISC LRFD and CSA-S16.1,
- the “clear width” $(d-2t$ or $b-2t$), used in AS 4100.
Eurocode 3 defines the width as $d-1.5t$ or $b-1.5t$, which is similar to the flat width definition, assuming that $r_e = 1.5t$. For hot-formed RHS, the corners are reasonably tight, and $r_e = 1.5t$ is a suitable approximation, but cold-formed RHS have larger corner radii, typically, $r_e = 2.0t - 2.5t$. Hence, it is the corner radius that creates the difference in the varying definitions of slenderness.

There were no specimens which varied only in corner radius size, so there can be no direct comparison between two specimens. Figure 3.13 plotted $R$ against $\lambda_w$ (flat width - AISC LRFD), while Figure 3.15 plotted $R$ against $\lambda_w$ (clear width - AS 4100). There is no evidence from either graph that one method of determining web slenderness gives a better relationship with $R$, than the other method. Section J.4 investigates the effect of the corner radius on rotation capacity using finite element analysis.

The cold-formed RHS do not satisfy the material ductility requirements of AS 4100 (Clause 4.5.2) or Eurocode 3 (Clause 3.2.2.2) for plastic design outlined in Section 2.5. Local instability (web buckling) was the failure mode of most of the sections, not material failure. Section BS01B, which did not experience local buckling, exhibited large rotations and strains, indicating that ductility may not present a problem. Hence the limitation on cold-formed RHS for plastic design based on insufficient material ductility may not be justified.

### 3.6 BENDING TESTS OF HOT-FORMED RHS

At the request of British Steel Tube and Pipes, some hot-formed RHS were tested as part of the CIDECT project (which forms the basis of the thesis), to provide a comparison with the cold-formed steel. Two samples, $200 \times 150 \times 6.3$ and $250 \times 150 \times 6.3$, in Grade S275J0H, manufactured to EN 10210 (European Committee for Standardisation 1994), were selected. Full details of the test results are available in Appendix B.
3.7 CONCLUSIONS

3.7.1 SUMMARY

The results of the plastic bending tests on a range of cold-formed RHS with different web and flange slenderness, aspect ratio and yield stress grade have been presented. The major finding is that the Class 1 web slenderness limits for RHS, which were based on tests of I-section beams, are unconservative for RHS. Some sections which satisfy the current Class 1 slenderness limits of AS 4100, Eurocode 3 and AISC LRFD do not exhibit the rotation capacity suitable for plastic design. The rotation capacity depends on both the flange slenderness and the web slenderness, so the flange and web slenderness limits must be related. The interaction has been identified in the iso-rotation plots from which a simple bi-linear interaction curve for the Class 1 limits of RHS has been proposed. Alternative forms of limits that account for flange-web interaction, such as multilinear or elliptical, maybe less conservative than the simple bi-linear limits proposed.

3.7.2 FURTHER STUDY

The results have shown that flange - web interaction should be considered in the Class 1 slenderness limits for RHS. Further study is required to examine the Class 2 and Class 3 limits of cold-formed RHS in bending, to establish if new slenderness limits which incorporate flange - web interaction are required.

The web slenderness limits for members in bending and compression are lower than the limits for bending alone, since a greater proportion of the web is in compression for the case of bending and compression. Given that the web limits for bending alone have been shown to be unconservative, the web slenderness limits for RHS in bending and compression must also be examined.
Chapter 4

TESTS OF KNEE JOINTS IN COLD-FORMED RECTANGULAR HOLLOW SECTIONS

4.0 CHAPTER SYNOPSIS

This chapter describes tests on various types of knee joints for portal frames constructed from cold-formed RHS. The major aim of the connection tests was to examine the ability of various joint types to form plastic hinges. Welded stiffened and unstiffened knee joints, bolted knee joints with end plates, and connections with a fabricated internal sleeve, were included in the experimental investigation. Most connections tested under opening moment failed by fracture in the heat affected zone of the RHS near the weld. The connections tested under closing moment failed by web local buckling which occurred near the connection.

While the stiffened and unstiffened welded connections satisfied the strength interaction requirements in the available design guides, the connections did not maintain the plastic moment for sufficiently large rotation to be considered suitable for a plastic hinge location. The unstiffened welded joints were not able to reach the plastic moment.

The use of an internal sleeve moved the plastic hinge in the connection away from the connection centre-line and reduced the stress on the weld between the legs of the connection. It was found that sleeve connections were capable of sustaining the plastic moment for large rotations considered suitable for plastic design.
4.1 INTRODUCTION

In plastic design of steel structures, various regions in the structure are required to form plastic hinges. The hinges must be capable of reaching the plastic moment \( (M_p) \) and maintaining the plastic moment for a suitable amount of plastic hinge rotation, to allow for redistribution of bending moments in a frame. Chapter 3 examined the behaviour of RHS beams, and the relationship between web slenderness, flange slenderness and rotation capacity.

Plastic hinges can be required to form at the connections of frames, since joints are sometimes points of maximum moment. Local effects within the connection may reduce the rotation capacity of a connection compared to the rotation capacity of the member itself. In a typical portal frame shown in Figure 4.1, a plastic hinge may often form in the knee connection (often called an L-joint) between the column and the rafter. Under the gravity loads shown in Figure 4.1, the knee connection experiences a closing moment. Under wind uplift loading, the connection experiences an opening moment. Wind uplift is frequently the critical load case for low-rise structures in Australia. Depending on the loads on the structure, the knee connection may need to be able to form plastic hinges under both closing and opening moments.

![Figure 4.1: Column Rafter Knee Joint in an RHS Portal Frame](image)

This chapter reports on tests to examine the ability of the portal frame knee connections to act as plastic hinges in a portal frame has been investigated. The connections tested were unstiffened welded, stiffened welded, bolted end plate and internal sleeve connections.

Chapter 2 included a review of previous literature of portal frame knee connections.
4.2 TEST PROGRAM

4.2.1 MATERIAL PROPERTIES

4.2.1.1 RHS Properties

Following the plastic bending tests (Chapter 3), the 150 × 50 × 4.0 RHS was chosen as an appropriate size for the joint tests. The 150 × 50 × 4.0 RHS had exhibited rotation capacity in pure bending deemed suitable for plastic design. As part of the investigation into connection behaviour, tests were performed on the 150 × 50 × 4.0 RHS, and the thicker 150 × 50 × 5.0 RHS.

The RHS were manufactured by Tubemakers of Australia Limited (now known as BHP Steel Structural and Pipeline Products). Two strength grades were selected, Grade C350L0 and C450L0 (nominal yield stress ($f_y$) of 350 MPa and 450 MPa respectively and nominal tensile strength ($f_u$) of 430 MPa and 500 MPa respectively), manufactured to Australian Standard AS 1163 (Standards Australia 1991a). More information on the RHS properties can be found in Section 3.2.1.

Figure 4.2 shows the cross section of a typical RHS and defines the dimensions $d$, $b$, $t$ and $r_e$ (depth, width, thickness and external corner radius) of the section. Figure 4.2 indicates the names given to each of the faces of the RHS: “weld”, “opposite”, “adjacent 1”, and “adjacent 2”. For most RHS the longitudinal weld is located on one of the shorter faces (the flange). Since the weld is normally slightly off centre, the adjacent face of the RHS which is closer to the weld is labelled “Adjacent 1”. In some cases (KJ01, KJ02, KJ03, KJ04, KJ05, KJ06 - all cut from the same length of specimen) the weld was located in the corner of the section. The unusual location of the seam weld is an exception to the rule which was probably caused by some twisting of the coil during the forming process.
4.2.1.2 Tensile Coupon Tests

Three coupons were taken from the flats of each tube. One was cut from the face opposite the weld, and one from each of the sides adjacent to the weld. Corner coupons were cut from the first RHS. The coupons were prepared and tested in accordance with AS 1391 (Standards Australia 1991b) in a 250 kN capacity INSTRON Universal Testing Machine or a 300 kN capacity SINTECH Testing Machine.

Since the steel was cold-formed, the yielding was gradual. Hence the reported yield stress ($f_y$) was the 0.2% proof stress. The average of the static yield stress from both of the adjacent faces was used in the determination of plastic moment ($M_p$) and slenderness values. The average of the elastic modulus ($E$) from both of the adjacent faces was used in stiffness calculations. The yield stress of the opposite face was on average 10% higher than that of the adjacent faces. The variation of yield stress was a result of the cold-forming process and has been previously identified (CASE 1992a).
Full details of the test method and the results are given in Appendix C. The values of the yield stress are included in the full results, which are presented in Table 4.2 (later in this Chapter).

### 4.2.1.3 Plate Properties

The 10 mm steel plate in the connections was Grade 350 to Australian Standard AS 3678 (Standards Australia 1990b). The plate had nominal yield stress \( f_{yn} = 360 \) MPa and nominal ultimate tensile strength \( f_{un} = 450 \) MPa. The measured material properties were yield stress \( f_{y} = 382 \) MPa and ultimate tensile strength \( f_{u} = 485 \) MPa. The coupon tests are described in Appendix C.

### 4.2.1.4 Bolt Properties

High strength structural bolts (M16: Grade 8.8) manufactured to Australian Standard AS 1252 (Standards Australia 1981) were used in the bolted end plate connection. Grade 8.8 bolts have minimum tensile stress \( f_{ut} = 830 \) MPa. Standard structural grade bolts (M16: Grade 4.6) to Australian Standard AS 1111 (Standards Australia 1980) were employed in the bolted sleeve joint. Grade 4.6 bolts have minimum tensile stress \( f_{ut} = 410 \) MPa. All nuts, bolts and washers were produced by Ajax Spurway Fasteners.
4.2.2 CONNECTION DETAILS

4.2.2.1 Stiffened Welded Connection (SW)

The stiffened welded (SW) connection was based on the knee joint in the CIDECT design guide (Packer et al 1992, Mang et al 1980). One end of each leg of the connection was cut at an angle of 52.5° to create a total of 105°. A 10 mm stiffening plate was welded between the two legs. Different welding procedures were used incorporating butt and fillet welds. The stiffening plate protruded 10 mm from each face of the RHS, as indicated in Figures 4.3 and 4.4. The acute angle between the plate and the RHS on the inside corner of the knee (Detail B of Figure 4.3) was difficult to weld. An alternative detail was then adopted in which the plate did not extend beyond the inside corner of the knee (Alternative Detail B of Figure 4.3).

Figure 4.3: Stiffened Welded Connection - Butt Weld
4.2.2.2 Unstiffened Welded Connection (UW)

The CIDECT design guide included an unstiffened mitred knee connection, denoted type UW. The UW connection is almost identical to the stiffened joint, except that the two legs are butt welded directly together without a stiffening plate. The detail is shown in Figure 4.5.
4.2.2.3 Bolted End Plate (BP)

The bolted end plate connection is denoted type BP. One RHS leg of the connection was mitre cut at 15°. A 10 mm thick steel plate was butt welded onto this mitred end. The plate was oversized so that there was sufficient space to allow for bolts outside the RHS on the plate. A second, identical 10 mm plate was welded to the flange of the other RHS leg. The second RHS leg was not mitred. The two sections were bolted together with eight fully tensioned high strength bolts. Coronet® load indicating washers ensured correct tensioning of the bolts. 6 mm stiffeners under the flange plate were employed against local bearing failure of the RHS web. Figure 4.6 shows the details of the BP joint.
Figure 4.6: Bolted End Plate Knee Connection
4.2.2.4 Welded Internal Sleeve (WS)

An internal hollow steel sleeve was fabricated to fit inside the mitred knee connection, and is called type WS. Two “L” shaped pieces of 10 mm steel were profile cut at the same angle (105°) as the whole joint. Flange plates were fillet welded to these pieces to create a box section. After fabrication, an angle grinder ground the edges of the sleeve to ensure that it would fit into the RHS legs. A groove was cut into the sleeve to allow for the internal seam weld of the RHS. The sleeve was fitted snugly inside each RHS leg of the joint. The sleeve was slightly oversized so that sledge hammer blows were required to insert the sleeve into the RHS, and ensured that load could be transferred mainly by friction. The sleeve thickness was chosen so that the plastic moment of the sleeve itself was slightly larger than the plastic moment of the RHS. Finally, the two legs were butt welded together with the sleeve acting as a backing plate for the weld. It was anticipated that the weld would not carry significant load since friction and bearing would transfer the load from the RHS into the sleeve. The weld would also help reduce slip between the sleeve and the RHS and hence stiffen the connection. The WS connection is shown diagrammatically in Figure 4.7.
4.2.2.5 Bolted Internal Sleeve (BS)

The internal sleeve in this connection (denoted type BS) is almost identical to the sleeve in the WS connection described in Section 4.2.2.4. However the two RHS legs were not welded. Four Grade 4.6, M16 bolts passed through each leg and the internal sleeve. Details of the joint are given in Figure 4.8.

![Diagram of Bolted Internal Sleeve Connection]

Figure 4.8: Bolted Internal Sleeve Connection

4.2.3 WELDING PROCEDURES

Welding procedures may be pre-qualified by Clause 4.3 of Australian / New Zealand Standard AS/NZS 1554.1 (Standards Australia 1995). The joint preparation, welding consumables, workmanship and welding techniques need to be pre-qualified to AS/NZS 1554.1 and are summarised in a welding procedure sheet.
A variety of welding procedures was used. The welding procedure sheets are shown in Appendix G. Some samples were welded by an independent fabricator, R. K. Engineering, (Procedure RKE - 138) to provide a useful benchmark as to the quality of welding that would be found in practice as performed by a typical steel fabricator. All other welding was performed in the Department of Civil Engineering, The University of Sydney. Table 4.1 specifies the welding procedures used for each specimen.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Cut from Section</th>
<th>Welding Procedure</th>
<th>Connection Type</th>
<th>Load¹</th>
</tr>
</thead>
<tbody>
<tr>
<td>KJ01</td>
<td>150 × 50 × 4.0 C450</td>
<td>RKE - 138</td>
<td>Stiffened Welded (SW)</td>
<td>C</td>
</tr>
<tr>
<td>KJ02</td>
<td>150 × 50 × 4.0 C450</td>
<td>RKE - 138</td>
<td>Stiffened Welded (SW)</td>
<td>T</td>
</tr>
<tr>
<td>KJ03</td>
<td>150 × 50 × 4.0 C450</td>
<td>TJW01</td>
<td>Stiffened Welded (SW)</td>
<td>C</td>
</tr>
<tr>
<td>KJ04</td>
<td>150 × 50 × 4.0 C450</td>
<td>TJW01</td>
<td>Stiffened Welded (SW)</td>
<td>T</td>
</tr>
<tr>
<td>KJ05</td>
<td>150 × 50 × 4.0 C450</td>
<td>TJW02</td>
<td>Unstiffened Welded (UW)</td>
<td>C</td>
</tr>
<tr>
<td>KJ06</td>
<td>150 × 50 × 4.0 C450</td>
<td>TJW02</td>
<td>Unstiffened Welded (UW)</td>
<td>T</td>
</tr>
<tr>
<td>KJ07</td>
<td>150 × 50 × 4.0 C350</td>
<td>TJW04</td>
<td>Stiffened Welded (SW)</td>
<td>C</td>
</tr>
<tr>
<td>KJ08</td>
<td>150 × 50 × 4.0 C350</td>
<td>TJW04</td>
<td>Stiffened Welded (SW)</td>
<td>T</td>
</tr>
<tr>
<td>KJ09</td>
<td>150 × 50 × 5.0 C350</td>
<td>TJW03</td>
<td>Stiffened Welded (SW)</td>
<td>C</td>
</tr>
<tr>
<td>KJ10</td>
<td>150 × 50 × 5.0 C350</td>
<td>TJW03</td>
<td>Stiffened Welded (SW)</td>
<td>T</td>
</tr>
<tr>
<td>KJ11</td>
<td>150 × 50 × 4.0 C450</td>
<td>TJW05</td>
<td>Stiffened Welded² (SW)</td>
<td>T</td>
</tr>
<tr>
<td>KJ12</td>
<td>150 × 50 × 4.0 C350</td>
<td>TJW04</td>
<td>Stiffened Welded³ (SW)</td>
<td>T</td>
</tr>
<tr>
<td>KJ13</td>
<td>150 × 50 × 4.0 C350</td>
<td>TJW07</td>
<td>Welded Sleeve (WS)</td>
<td>C</td>
</tr>
<tr>
<td>KJ14</td>
<td>150 × 50 × 4.0 C350</td>
<td>TJW07</td>
<td>Welded Sleeve (WS)</td>
<td>T</td>
</tr>
<tr>
<td>KJ15</td>
<td>150 × 50 × 4.0 C350</td>
<td>TJW04 TJW06⁴</td>
<td>Bolted End Plate (BP)</td>
<td>C</td>
</tr>
<tr>
<td>KJ16</td>
<td>150 × 50 × 4.0 C350</td>
<td>TJW04 TJW06⁴</td>
<td>Bolted End Plate (BP)</td>
<td>T</td>
</tr>
<tr>
<td>KJ17</td>
<td>150 × 50 × 4.0 C450</td>
<td>na</td>
<td>Bolted Sleeve (BS)</td>
<td>C</td>
</tr>
<tr>
<td>KJ18</td>
<td>150 × 50 × 4.0 C450</td>
<td>na</td>
<td>Bolted Sleeve (BP)</td>
<td>T</td>
</tr>
</tbody>
</table>

Notes: (1) Load is either tensile (T) (opening moment) or compressive (C) (closing moment).
(2) Fillet weld.
(3) Alternate detail.
(4) Procedure TJW04 for RHS to end plate butt weld, procedure TJW06 for RHS flange to plate fillet weld.

Table 4.1: Summary of Knee Joint Details and Welding Procedures
4.2.4 TEST PROCEDURE

Tests were performed in a 2000 kN capacity DARTEC servo-controlled universal testing machine. A schematic diagram of the general test set up is shown in Figure 4.9. A photograph of a typical test is shown in Figure 4.10. A compression or tension load was applied through pins inserted through the webs of the RHS, and the eccentricity of the load with respect to the joint itself produced a bending moment (a closing or an opening moment respectively) about the major principal x-axis at the joint centre-line.

Figure 4.9: Schematic Details for Knee Joint Test
The aim of the connection tests was to simulate the behaviour of a knee joint in a portal frame. It can be seen from Figure 4.9 that unequal leg lengths have been used. The point of contraflexure (zero moment) in the rafter of a portal frame under typical loads is closer to the joint than the base of the column, particularly for frames with pinned bases (refer to Figure 4.11). Hence unequal leg lengths more accurately simulate the distribution of bending moment, axial force and shear force in the connection than an equal leg specimen.
A constant crosshead speed of 0.03 mm/s was used at all times. Near the ultimate load, the test was halted for approximately 1 minute to obtain a value of the static load. A transducer measured the horizontal displacement at the joint, and the total vertical stroke of the DARTEC ram was obtained from the testing machine. Data was recorded by a SPECTRA data acquisition system.

Strain gauges were placed near the centre-line of each joint to determine the curvature. Figure 4.12 shows the location of the strain gauges. The gauges were placed on opposite flanges of the RHS, a finite distance away from the connection centre-line. As was indicated in Figure 2.21, it was expected that the cross-section carrying the maximum moment was a small distance from the centre-line. Hence the gauges were placed at a small distance away from the centre-line. It can be noted that for the welded sleeve connections, the gauges were in the part of the connection that included the sleeve, while for the bolted sleeve connection, the gauges were placed outside the sleeve region.
Figure 4.12: Strain Gauge Locations
4.2.5 TESTS OF BOLTED MOMENT END PLATE CONNECTIONS

As an addition to the test series on knee joints, there were three tests on bolted moment end plate connections between beams. This aim was to investigate the behaviour of prototype joints for the apex connection between the two rafters of the portal frame. Appendix H contains a full description of the connections details and the results.

4.3 RESULTS

4.3.1 GENERAL

The results of the knee joint tests are summarised in Tables 4.2, 4.3 and 4.4. The results have been separated into tension and compression tests (opening and closing moments). \( P_{\text{max}} \) is the maximum (vertical) load applied by the testing machine. The maximum moment (\( M_{\text{max}} \)) reached at the knee and the maximum axial force (\( N_{\text{max}} \)) and shear force (\( V_{\text{max}} \)) induced at the connection are listed. \( M_{\text{max}} \) is the second order moment and includes the effect of changing eccentricity at the joint, and is calculated using the eccentricity to the joint centre-line, which was initially 561 mm. \( N_{\text{max}} \) and \( V_{\text{max}} \) were obtained from a first order elastic analysis of the joint and the applied load. These are compared to the plastic moment (\( M_p \)), the squash load (\( A_{gf} \)) and the shear yield load (\( V_p = 2 dtf_y / \sqrt{3} \)). The maximum loads are static loads, which represent the recorded load after the testing machine had been halted for approximately one minute.

The web slenderness (\( \lambda_w \)) and flange slenderness (\( \lambda_f \)) determined according to Eurocode 3 are tabulated, and the mode of failure is described. The proportion of the web in compression (\( \alpha \)), assuming full plasticity is given for compression tests only. The value of \( \alpha \) is important, since the Class 1 web slenderness limit in Eurocode 3 is reduced from 72 when webs are under bending and compression (Table 5.3.1 of Eurocode 3). The reduced Class 1 limit (\( \lambda_{w1} \)), and the rotation capacity based on the curvature at the strain gauges (\( R_g \)) and the connection rotation (\( R_c \)) are also given (refer to Sections 4.3.4 and 4.3.5 respectively).
<table>
<thead>
<tr>
<th>Specimen</th>
<th>Cut from section</th>
<th>Welding Procedure</th>
<th>Type</th>
<th>d (mm)</th>
<th>b (mm)</th>
<th>t (mm)</th>
<th>( r_c ) (mm)</th>
<th>( f_y ) (MPa)</th>
<th>( f_u ) (MPa)</th>
<th>( \epsilon_i ) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>COMPRESSION (CLOSING MOMENT)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>KJ01</td>
<td>150×50×4.0 C450</td>
<td>RKE - 138</td>
<td>SW</td>
<td>150.33</td>
<td>50.35</td>
<td>3.91</td>
<td>8.5</td>
<td>436</td>
<td>462</td>
<td>24.6</td>
</tr>
<tr>
<td>KJ03</td>
<td>150×50×4.0 C450</td>
<td>TJW01</td>
<td>SW</td>
<td>150.38</td>
<td>50.38</td>
<td>3.91</td>
<td>9.2</td>
<td>436</td>
<td>462</td>
<td>24.6</td>
</tr>
<tr>
<td>KJ05</td>
<td>150×50×4.0 C450</td>
<td>TJW02</td>
<td>UW</td>
<td>150.23</td>
<td>50.24</td>
<td>3.93</td>
<td>8.9</td>
<td>436</td>
<td>462</td>
<td>24.6</td>
</tr>
<tr>
<td>KJ07</td>
<td>150×50×4.0 C350</td>
<td>TJW04</td>
<td>SW</td>
<td>150.37</td>
<td>50.23</td>
<td>3.91</td>
<td>8.0</td>
<td>349</td>
<td>437</td>
<td>36.3</td>
</tr>
<tr>
<td>KJ09</td>
<td>150×50×5.0 C350</td>
<td>TJW03</td>
<td>SW</td>
<td>150.38</td>
<td>50.07</td>
<td>4.90</td>
<td>10.0</td>
<td>348</td>
<td>419</td>
<td>38.3</td>
</tr>
<tr>
<td>KJ13</td>
<td>150×50×4.0 C350</td>
<td>TJW07</td>
<td>WS</td>
<td>150.66</td>
<td>50.56</td>
<td>3.89</td>
<td>7.5</td>
<td>429</td>
<td>503</td>
<td>22.5</td>
</tr>
<tr>
<td>KJ15</td>
<td>150×50×4.0 C350</td>
<td>TJW04, TJW06</td>
<td>BP</td>
<td>150.45</td>
<td>50.67</td>
<td>3.90</td>
<td>8.0</td>
<td>429</td>
<td>503</td>
<td>22.5</td>
</tr>
<tr>
<td>KJ17</td>
<td>150×50×4.0 C450</td>
<td>n/a</td>
<td>BS</td>
<td>150.43</td>
<td>50.25</td>
<td>3.93</td>
<td>8.5</td>
<td>436</td>
<td>491</td>
<td>23.1</td>
</tr>
<tr>
<td><strong>TENSION (OPENING MOMENT)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>KJ02</td>
<td>150×50×4.0 C450</td>
<td>RKE - 138</td>
<td>SW</td>
<td>150.18</td>
<td>50.05</td>
<td>3.90</td>
<td>8.8</td>
<td>436</td>
<td>462</td>
<td>24.6</td>
</tr>
<tr>
<td>KJ04</td>
<td>150×50×4.0 C450</td>
<td>TJW01</td>
<td>SW</td>
<td>150.41</td>
<td>50.25</td>
<td>3.86</td>
<td>9.0</td>
<td>436</td>
<td>462</td>
<td>24.6</td>
</tr>
<tr>
<td>KJ06</td>
<td>150×50×4.0 C450</td>
<td>TJW02</td>
<td>UW</td>
<td>150.35</td>
<td>50.26</td>
<td>3.93</td>
<td>8.5</td>
<td>436</td>
<td>462</td>
<td>24.6</td>
</tr>
<tr>
<td>KJ08</td>
<td>150×50×4.0 C350</td>
<td>TJW04</td>
<td>SW</td>
<td>150.30</td>
<td>50.08</td>
<td>3.89</td>
<td>8.3</td>
<td>349</td>
<td>437</td>
<td>36.3</td>
</tr>
<tr>
<td>KJ10</td>
<td>150×50×5.0 C350</td>
<td>TJW03</td>
<td>SW</td>
<td>150.29</td>
<td>50.04</td>
<td>4.90</td>
<td>10.1</td>
<td>348</td>
<td>419</td>
<td>38.3</td>
</tr>
<tr>
<td>KJ11</td>
<td>150×50×4.0 C450</td>
<td>TJW05</td>
<td>SW</td>
<td>150.31</td>
<td>50.20</td>
<td>3.87</td>
<td>8.4</td>
<td>436</td>
<td>462</td>
<td>24.6</td>
</tr>
<tr>
<td>KJ12</td>
<td>150×50×4.0 C350</td>
<td>TJW04</td>
<td>SW</td>
<td>150.29</td>
<td>51.71</td>
<td>3.87</td>
<td>9.0</td>
<td>349</td>
<td>437</td>
<td>36.3</td>
</tr>
<tr>
<td>KJ14</td>
<td>150×50×4.0 C350</td>
<td>TJW07</td>
<td>WS</td>
<td>150.60</td>
<td>50.60</td>
<td>3.85</td>
<td>7.5</td>
<td>429</td>
<td>503</td>
<td>22.5</td>
</tr>
<tr>
<td>KJ16</td>
<td>150×50×4.0 C350</td>
<td>TJW04, TJW06</td>
<td>BP</td>
<td>150.52</td>
<td>50.65</td>
<td>3.90</td>
<td>8.0</td>
<td>429</td>
<td>503</td>
<td>22.5</td>
</tr>
<tr>
<td>KJ18</td>
<td>150×50×4.0 C450</td>
<td>n/a</td>
<td>BP</td>
<td>150.47</td>
<td>50.43</td>
<td>3.95</td>
<td>8.5</td>
<td>436</td>
<td>491</td>
<td>23.1</td>
</tr>
</tbody>
</table>

Note: (a) \( F_y, F_u \) and \( \epsilon_i \) are the average values for coupons cut from the two flat faces of the RHS adjacent to the longitudinal seam weld.

Table 4.2: Summary of Results for Knee Joint Tests (1)
<table>
<thead>
<tr>
<th>Specimen</th>
<th>$P_{\text{max}}$ (kN)</th>
<th>$P_p$ (kN)</th>
<th>$\delta_p$ (mm)</th>
<th>$\lambda_{w}$</th>
<th>$\lambda_{r}$</th>
<th>$\alpha$</th>
<th>$M_{\text{max}}$ (kNm)</th>
<th>$M_{\text{max}}/M_p$</th>
<th>$N_{\text{max}}$ (kN)</th>
<th>$N_{y}$ (kN)</th>
<th>$V_{\text{max}}$</th>
<th>$V_{\text{max}}/V_p$</th>
</tr>
</thead>
<tbody>
<tr>
<td>KJ01</td>
<td>56.4</td>
<td>50.70</td>
<td>12.24</td>
<td>48.3</td>
<td>13.5</td>
<td>0.55</td>
<td>64.5</td>
<td>32.6</td>
<td>1.15</td>
<td>47.8</td>
<td>0.075</td>
<td>52.5</td>
</tr>
<tr>
<td>KJ03</td>
<td>59.2</td>
<td>50.54</td>
<td>12.26</td>
<td>48.2</td>
<td>13.4</td>
<td>0.55</td>
<td>64.2</td>
<td>34.1</td>
<td>1.20</td>
<td>50.2</td>
<td>0.079</td>
<td>55.1</td>
</tr>
<tr>
<td>KJ05</td>
<td>36.1</td>
<td>50.66</td>
<td>12.26</td>
<td>48.0</td>
<td>13.3</td>
<td>0.53</td>
<td>67.0</td>
<td>20.6</td>
<td>0.72</td>
<td>30.6</td>
<td>0.048</td>
<td>33.6</td>
</tr>
<tr>
<td>KJ07</td>
<td>57.9</td>
<td>40.73</td>
<td>10.44</td>
<td>43.2</td>
<td>12.0</td>
<td>0.56</td>
<td>62.7</td>
<td>33.8</td>
<td>1.48</td>
<td>49.1</td>
<td>0.096</td>
<td>53.9</td>
</tr>
<tr>
<td>KJ09</td>
<td>68.2</td>
<td>49.22</td>
<td>10.80</td>
<td>33.7</td>
<td>8.8</td>
<td>0.56</td>
<td>63.0</td>
<td>39.8</td>
<td>1.44</td>
<td>57.8</td>
<td>0.092</td>
<td>63.4</td>
</tr>
<tr>
<td>KJ13</td>
<td>75.5</td>
<td>50.33</td>
<td>12.45</td>
<td>48.3</td>
<td>13.5</td>
<td>0.57</td>
<td>62.1</td>
<td>44.2</td>
<td>1.57</td>
<td>64.0</td>
<td>0.10</td>
<td>70.2</td>
</tr>
<tr>
<td>KJ15</td>
<td>57.1</td>
<td>53.55</td>
<td>12.48</td>
<td>48.0</td>
<td>13.5</td>
<td>0.55</td>
<td>64.3</td>
<td>34.2</td>
<td>1.21</td>
<td>48.4</td>
<td>0.077</td>
<td>53.1</td>
</tr>
<tr>
<td>KJ17</td>
<td>74.8</td>
<td>51.01</td>
<td>12.75</td>
<td>48.0</td>
<td>13.3</td>
<td>0.56</td>
<td>62.4</td>
<td>47.0</td>
<td>1.64</td>
<td>63.4</td>
<td>0.099</td>
<td>69.6</td>
</tr>
</tbody>
</table>

**COMPRESSION (CLOSING MOMENT)**

**TENSION (OPENING MOMENT)**

Note: (a) Web slenderness ($\lambda_{w}$) and flange slenderness ($\lambda_{r}$) determined according to Eurocode 3

Table 4.3: Summary of Results of Knee Joint Tests (2)
<table>
<thead>
<tr>
<th>Specimen</th>
<th>$R_x$</th>
<th>Heat Input</th>
<th>$\frac{N_{\text{max}}}{N_{\text{ri}}}$</th>
<th>$\frac{M_{\text{max}}}{M_{\text{ri}}}$</th>
<th>$\frac{N_{\text{max}}}{N_{\text{y}}}$</th>
<th>$\frac{M_{\text{max}}}{M_{p}}$</th>
<th>$\alpha_{\text{knee}}$</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>(kJ/mm)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>COMPRESSION (CLOSING MOMENT)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>KJ01</td>
<td>0.9</td>
<td>2.8</td>
<td>1.0/1.0</td>
<td>1.35</td>
<td>1.22</td>
<td>1.0</td>
<td>Local buckle inside of knee</td>
<td></td>
</tr>
<tr>
<td>KJ03</td>
<td>0.9</td>
<td>3.3</td>
<td>1.5/1.2</td>
<td>1.42</td>
<td>1.28</td>
<td>1.0</td>
<td>Local buckle inside of knee</td>
<td></td>
</tr>
<tr>
<td>KJ05</td>
<td>-</td>
<td>-</td>
<td>0.5</td>
<td>0.86</td>
<td>0.77</td>
<td>0.87</td>
<td>Local buckle across joint</td>
<td></td>
</tr>
<tr>
<td>KJ07</td>
<td>2.8</td>
<td>6.0</td>
<td>1.3/1.3</td>
<td>1.74</td>
<td>1.58</td>
<td>1.0</td>
<td>Local buckle inside of knee</td>
<td></td>
</tr>
<tr>
<td>KJ09</td>
<td>5.2</td>
<td>8.5</td>
<td>1.4/1.2</td>
<td>1.69</td>
<td>1.53</td>
<td>1.0</td>
<td>Local buckle inside of knee</td>
<td></td>
</tr>
<tr>
<td>KJ13</td>
<td>1.7</td>
<td>3.9</td>
<td>n/a</td>
<td>1.85</td>
<td>1.67</td>
<td>n/a</td>
<td>Local buckle at end of sleeve</td>
<td></td>
</tr>
<tr>
<td>KJ15</td>
<td>1.0</td>
<td>5.1</td>
<td>1.3/1.3</td>
<td>1.43</td>
<td>1.29</td>
<td>n/a</td>
<td>Local buckle at end of flange</td>
<td></td>
</tr>
<tr>
<td>KJ17</td>
<td>2.0</td>
<td>12</td>
<td>n/a</td>
<td>1.93</td>
<td>1.74</td>
<td>n/a</td>
<td>Local buckle at end of sleeve</td>
<td></td>
</tr>
<tr>
<td><strong>TENSION (OPENING MOMENT)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>KJ02</td>
<td>1.1</td>
<td>2.1</td>
<td>1.0/1.0</td>
<td>1.64</td>
<td>1.47</td>
<td>1.0</td>
<td>Tube fracture in HAZ</td>
<td></td>
</tr>
<tr>
<td>KJ04</td>
<td>1.8</td>
<td>2.6</td>
<td>1.5/1.2</td>
<td>1.71</td>
<td>1.53</td>
<td>1.0</td>
<td>Tube fracture in HAZ</td>
<td></td>
</tr>
<tr>
<td>KJ06</td>
<td>-</td>
<td>-</td>
<td>0.5</td>
<td>0.97</td>
<td>0.87</td>
<td>0.87</td>
<td>Tube fracture in HAZ</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Tearing of butt weld</td>
<td></td>
</tr>
<tr>
<td>KJ08</td>
<td>10</td>
<td>5.0</td>
<td>1.3/1.3</td>
<td>1.84</td>
<td>1.67</td>
<td>1.0</td>
<td>Weld tear-out from plate</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Local buckle outside of knee</td>
<td></td>
</tr>
<tr>
<td>KJ10</td>
<td>0.9</td>
<td>1.0</td>
<td>1.4/1.2</td>
<td>1.49</td>
<td>1.35</td>
<td>1.0</td>
<td>Weld tear-out from plate</td>
<td></td>
</tr>
<tr>
<td>KJ11</td>
<td>2.7</td>
<td>2.8</td>
<td>0.9</td>
<td>1.53</td>
<td>1.37</td>
<td>1.0</td>
<td>Tube fracture in HAZ</td>
<td></td>
</tr>
<tr>
<td>KJ12</td>
<td>2.5</td>
<td>3.1</td>
<td>1.3/1.3</td>
<td>1.73</td>
<td>1.57</td>
<td>1.0</td>
<td>Tube fracture in HAZ</td>
<td></td>
</tr>
<tr>
<td>KJ14</td>
<td>1.0</td>
<td>5.9</td>
<td>n/a</td>
<td>2.19</td>
<td>1.98</td>
<td>n/a</td>
<td>Tube fracture at end of sleeve</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>(internal bearing)</td>
<td></td>
</tr>
<tr>
<td>KJ16</td>
<td>0.2</td>
<td>3.4</td>
<td>1.3/1.3</td>
<td>1.37</td>
<td>1.24</td>
<td>n/a</td>
<td>Tube fracture in HAZ (butt weld)</td>
<td></td>
</tr>
<tr>
<td>KJ18</td>
<td>0.3</td>
<td>6.2</td>
<td>n/a</td>
<td>1.71</td>
<td>1.53</td>
<td>n/a</td>
<td>Sleeve fracture</td>
<td></td>
</tr>
</tbody>
</table>

Notes: (1) For welding procedures involving two welding runs, two values of heat input are given.
(2) Section capacities $M_n$ and $N_n$ determined according to Eurocode 3 ($N_{c,Rd}$, $N_{c,Rd}$, or $M_{c,Rd}$ as appropriate).

Table 4.4: Summary of Results for Knee Joint Tests (3)
4.3.2 FAILURE MODES

The mode of failure of each specimen is stated in Table 4.4. Figure 4.13 and Appendix I illustrate the failure modes described in Table 4.4.

Under **compression** loads (closing moment) web local buckling was the failure mode in all cases. For the unstiffened connection the buckle formed across the butt weld. Local buckling occurred on the inside of the knee (the compression side due to the applied moment) for all other connection types. The local buckle formed in the lower leg adjacent to the stiffening plate for all stiffened welded joints, and the bolted end plate connection. For both the welded and bolted sleeve joints, the buckle formed just outside the limit of the sleeve, whilst on the opposite tension face, the sleeve had pushed against the inside face of the flange of the RHS and caused a small outwards bulge in that face. As the load increased on the bolted sleeve connection, the two legs of the connection separated at the sleeve as shown in Figure 4.14. After local buckling, the applied load dropped slowly with increased deflection. For beam sections of the same size described in Chapter 3, the load shedding was considerably quicker (refer to Chapter 3). There was no evidence of lateral buckling or out-of-plane deflections in any test.

Most **tension** (opening moment) joints failed at or near the weld. Failure always occurred on the inside of the knee as the inside had the higher tension due to the axial force and bending moment. There was a sudden drop in load whenever there was a weld failure, due to the reduction in cross-sectional area resisting the load. Tube fracture occurred in the heat affected zone (HAZ) next to the weld. The HAZ fracture always initiated in the corner of the RHS. The tensile coupon tests (Appendix C) indicate that coupons cut from the corners of the RHS are considerably less ductile than coupons cut from the flat portions of the RHS. Hence, it is not unexpected that the fracture began in the corners of the RHS. Another mode of failure observed was tear-out of the weld from the stiffening plate (Specimen KJ08). For the sleeve connections, there was considerable bearing from the sleeve onto the inside of the flange of the RHS as the load was applied. A bulge was clearly evident in the flange (Figure 4.14). The bulging (internal bearing) caused a fracture in the RHS tube for the welded sleeve. The sleeve itself fractured at the ultimate load in the bolted sleeve connection. There was no evidence of lateral buckling or out-of-plane deflections in any test.
HAZ fracture or Weld tearout (tension)
Local buckle (compression)

Weld tearout from plate

HAZ fracture (initiated in corner)

(a) Type SW
(b) Type SW (tension)

Local buckle & ovalisation (compression)
Weld failure (tension)

Sleeve

(c) Type UW
(d) Type WS

Sleeve fracture (Tension)
Local buckle (Compression)

Sleeve

(e) Type BS
(f) Type BP

Stiffeners

HAZ fracture (tension)
Local buckle (compression)

Figure 4.13: Failure Modes for the Knee Joints
4.3.3 LOAD - STROKE CURVES

The load - stroke (total vertical displacement) curves are given in Figure 4.15 (Compression) and Figure 4.16 (Tension). The load has been normalised with respect to the plastic load $P_p$, where $P_p = M_p/e$, and $e = 561$ mm, the initial eccentricity of the joint, and $M_p$ is the plastic moment of the RHS only. The stroke has been normalised with respect to $\delta_p$, where $\delta_p$ is the displacement of the joint at a load $P_p$ based on a first order linear elastic analysis. Values of $P_p$ and $\delta_p$ have been included in Table 4.3.
Figure 4.15: Load Displacement Curves for Knee Joint Tests (Compression)

Figure 4.16: Load Displacement Curves for Knee Joint Tests (Tension)

(In Figures 4.15 and 4.16, the load and displacement have been normalised with respect to $P_p$ and $\delta_p$ respectively. Refer to Table 4.3 for values of $P_p$ and $\delta_p$.)
4.3.4 MOMENT - CURVATURE CURVES

The moment - curvature relations are plotted in Figures 4.17 and 4.18. The graphs show the second order moment (derived by taking the product of the load and the eccentricity of the strain gauge position) and the curvature at the strain gauge location on the lower leg.

Since the strain gauges were placed a finite distance from the connection apex (Figure 4.12) and not necessarily at the position of maximum curvature (or the buckle), the graphs do not indicate the curvature and moment at the point of failure. For the SW, UW and BP connections, the position of maximum curvature (refer to Figure 2.21) was closer to the connection centre-line than the strain gauges were to the centre-line. Due to the extra eccentricity at the point of maximum curvature, the moment would be higher than at the strain gauge location. More plasticity, and hence higher curvatures, occur closer to the joint centre-line than at the gauge location. Therefore, the moment - curvature graphs in Figures 4.17 and 4.18 underestimate both the curvature and the moment at the critical section of the joints. For both the welded sleeve and bolted sleeve connections, the curvature was highly concentrated just past the end of the internal sleeve. Since the strain gauges for the WS connection were in the very stiff sleeve zone, the curvatures shown in Figures 4.17 and 4.18 are considerably lower than those at the critical cross section where the curvature is concentrated. Hence, it can be seen in Figures 4.17 and 4.18 that the curves representing the WS connection are stiffer than those for the other connections.

An initial measure of the rotation capacity, $R_g$, is based on the moment - curvature relationship at the strain gauge location. For the compression tests $R_g = \kappa / \kappa_p - 1$, where $\kappa$ is the curvature at which the moment (at the strain gauge position) drops below $M_p$ (the definition of $R$ used for the beam tests). For the tension tests $R_g = \kappa / \kappa_p - 1$, where $\kappa$ is the curvature at which fracture or failure occurred. Since the strain gauges were not at the point of maximum curvature, and the value of $M$ at the gauge location is less than the maximum moment, $R_g$ underestimates the “true” rotation capacity of the connection. Values of $R_g$ are reported in Table 4.4.
Note: Moment and curvature at the strain gauge location on the lower leg (Figure 4.12). 
$M_p$ based on the measured dimensions of the RHS only.

Figure 4.17: Moment - Curvature Curves for Knee Joint Tests (Compression)

Note: Moment and curvature at the strain gauge location on the lower leg (Figure 4.12). 
$M_p$ based on the measured dimensions of the RHS only.

Figure 4.18: Moment - Curvature Curves for Knee Joint Tests (Tension)
4.3.5 MOMENT - ROTATION CURVES

Section 4.3.4 has highlighted that it was difficult to obtain a reasonable estimate of the rotation capacity of the various connections from the curvature at the strain gauge locations. Consequently an alternative method was devised, by considering the rotation of the plastic hinge.

Figure 4.19(a) shows the elastic distribution of curvature in a typical test specimen. After yielding occurs, curvature is highly concentrated near the centre-line of the connection as shown in Figure 4.19(b). Most of the specimen remains elastic, and the elastic curvature is considerably smaller than the curvature at the plastic hinge. The behaviour can idealised using the rigid plastic assumption, which is demonstrated in Figure 4.19(c). It is assumed that infinite curvature occurs at a plastic hinge, which rotates through some angle, while zero curvature occurs at all other points. The rotation of the hinge point can then be calculated from the deflection of the specimen. A measure of the rotation capacity of the connection can be obtained from the rotation at the hinge point.

Figure 4.19: Distribution of Curvature in the Test Connections
In Figure 4.20(a), the angle of the undeformed member \((\alpha_0 = 105^\circ)\) is shown. Using a first order elastic analysis, the angle \((\alpha_p)\) at which the plastic moment is reached at the connection centre-line is given in Figure 4.20(b). Figure 4.20(c) shows the angle at any load \((\alpha_d)\) after the plastic moment has been reached. Each angle is calculated assuming that all rotation occurs at a single point at the joint centre-line. The normalised rotation \((\bar{\alpha})\) is given by:

\[
\bar{\alpha} = \frac{\alpha_d - \alpha_0}{\alpha_p - \alpha_0}
\]  

(4.1)

![Diagram showing rotation for knee connections](image)

(a) Undisplaced  
(b) Displaced at the plastic moment  
(c) Displaced at any load

Figure 4.20 Definition of Rotation for the Knee Connections
For the sleeve connection, a slightly more complex model of rotation is needed, since the point of maximum curvature occurs well away from the connection centre-line, at the end of the internal sleeve. While the maximum moment still occurs at the connection apex, since the sleeve locally stiffens and strengthens the section, the curvature concentrates just outside the sleeve region. The rotation model for the sleeve connection is shown in Figure 4.21. Equation 4.1 is again used to define the normalised rotation, \( \tilde{\alpha} \), for the sleeve connections.

Figure 4.21: Definition of Rotation for Sleeve Joints

Figures 4.22 and 4.23 show the normalised rotation versus moment graphs for the joints in compression and tension respectively. The moment and rotation are calculated at the assumed hinge points defined in Figures 4.20 and 4.21. From Figures 4.22 and 4.23, the rotation capacity based on the rotation \( R_\alpha \) was calculated. Values of \( R_\alpha \) are given in Table 4.4. \( R_\alpha \) is based on the behaviour of the whole joint and is not a “localised” value like \( R_\gamma \). For tension tests, \( R_\alpha \) is defined as \( R_\alpha = \tilde{\alpha}_f - 1 \), where fracture occurred at a normalised rotation of \( \tilde{\alpha}_f \), while in compression \( R_\alpha = \tilde{\alpha}_i - 1 \), where the moment drops below \( M_p \) at \( \tilde{\alpha}_i \).
Note: Moment and rotation calculated at the assumed hinge (Figures 20 and 21). $M_p$ based on the measured dimensions of the RHS only.

Figure 4.22: Moment - Rotation Curves for Knee Joint Tests (Compression)

Note: Moment and rotation calculated at the assumed hinge (Figures 20 and 21). $M_p$ based on the measured dimensions of the RHS only.

Figure 4.23: Moment - Rotation Curves for Knee Joint Tests (Tension)
4.4 DISCUSSION

4.4.1 STIFFENED AND UNSTIFFENED WELDED JOINTS

Table 4.3 indicates that the axial force in the RHS is low (approximately 5 - 10 % of the squash load $N_y = A_f f_y$). All stiffened connections reached a moment greater than $M_{p}$. Combined compression and bending reduces the Eurocode 3 Class 1 limit for the slenderness of RHS webs from the value of 72 for bending alone. All sections tested can still be classified as Class 1 and are deemed suitable for plastic design. The sections tested had also exhibited large rotation capacities ($R > 6.6$) as beams as described in Chapter 2. The ratios of the shear force to the shear load capacity are less than the 0.5 limit given in the CIDECT design guide (Packer et al 1992).

The unstiffened connections (KJ05 and KJ06) are unsuitable for plastic hinge formation. Both failed at approximately 70-80 % of the plastic moment and hence cannot be used at a plastic hinge location. They cannot be classified as Class 1 or Class 2, or Compact connections.

It is evident that the SW connections exceeded the plastic moment by a considerable amount. The values of $M_{max}/M_p$ range from 1.08 to 1.56. By comparison, for the simple beam tests (Chapter 3), the highest value of $M_{max}/M_p$ was approximately 1.25. It is expected that the maximum moment can exceed the plastic moment for two reasons:

- There is significant strain hardening in the cold-formed sections after yielding.
- The values of $M_p$ are based on the yield stress of the adjacent face of the RHS. The tensile coupon tests indicate that the yield stress of the opposite face and the corners of the RHS are 10 - 20 % higher than that of the adjacent face.

The maximum moment ($M_{max}$ in Table 4.3) in the connections was calculated from the eccentricity of the loading points with respect to the centre-line of the connection (shown as $e$ in Figure 4.24). However, as was noted in Mang et al (1980), the cross section that resists the greatest bending moment is not at the connection centre-line, and hence the eccentricity is slightly lower ($e_1$ in Figure 4.24, the exact value of which is unknown). At the connection centre-line, load is resisted by bending moment and axial force, and interaction with the stiffening plate, and shell action. Hence the maximum moment experienced in the specimen is lower than that reported, but the exact value is unknown. Therefore the values of $M_{max}/M_p$ are higher than anticipated based on a simple centre-line model.
Figure 4.24: Eccentricity of the Knee Joint

Mang et al (1980) and the CIDECT design guide gave validity limits for welded knee joints which are summarised in Table 4.5.

<table>
<thead>
<tr>
<th>Stiffened</th>
<th>Unstiffened</th>
</tr>
</thead>
<tbody>
<tr>
<td>$b \leq 400 \text{ mm}$</td>
<td>$b \leq 300 \text{ mm}$</td>
</tr>
<tr>
<td>$d \leq 400 \text{ mm}$</td>
<td>$d \leq 300 \text{ mm}$</td>
</tr>
<tr>
<td>$0.3 \leq d/b \leq 3.0$</td>
<td>$0.3 \leq d/b \leq 3.0$</td>
</tr>
<tr>
<td>$t \geq 3.0 \text{ mm}$</td>
<td>$t \geq 3.0 \text{ mm}$</td>
</tr>
<tr>
<td>$t \sqrt{255/f_y} \leq 30 \text{ mm}^a$</td>
<td>$t \sqrt{255/f_y} \leq 30 \text{ mm}^a$</td>
</tr>
<tr>
<td>$(b/t) \sqrt{255/f_y} \leq 43^a$</td>
<td>$(b/t) \sqrt{255/f_y} \leq 43^a$</td>
</tr>
</tbody>
</table>

Note: (a) Mang et al gave limits for 37 ksi (255 MPa) and 52 ksi (358 MPa) steel only.

Table 4.5: Validity Limits for RHS Welded Knee Connections

Additional requirements for the connection experiencing an axial force $N^*$, a bending moment $M^*$ and a shear force $V^*$ are:
\[ \frac{N^*}{N_{ri}} + \frac{M^*}{M_{ri}} \leq \alpha_{\text{knee}} \]  
\[ (4.2) \]

where \( \alpha_{\text{knee}} \) is a stress reduction factor given by:

\[ \alpha_{\text{knee}} = 1 \quad \text{for stiffened connections (a standard strength interaction between axial force and moment),} \]

\[ \alpha_{\text{knee}} < 1 \quad \text{for unstiffened connections and is a function of the specimen dimensions} \quad \text{(} \frac{b}{t} \text{ and } \frac{d}{h} \text{)} \quad \text{and the angle } \theta \text{ between the legs of the connection (design charts are given to determine the value of } \alpha_{\text{knee}}), \]

and \( N_{ri} = \) the design section capacity in either tension \((N_{t,Rd} (\text{Eurocode 3}, \text{ or } \phi N_t (\text{AS 4100})))\) or compression \((N_{c,Rd} (\text{Eurocode 3}) \text{ or } \phi N_c (\text{AS 4100}))\) (as appropriate),

\[ M_{ri} = \] the design section moment capacity \((M_{c,Rd} (\text{Eurocode 3}) \text{ or } \phi M_c (\text{AS 4100})).\]

It was specified in the CIDECT guide that the shear force in the connection should be limited by

\[ \frac{V^*}{V_p} \leq 0.5 \]
\[ (4.3) \]

where \( V_p = 2d tf_y \sqrt{3} \) and is the plastic shear yield capacity of the section.

Table 4.4 lists the interaction values for the stiffened and unstiffened connections and compares them to the value of \( \alpha_{\text{knee}}. \ \alpha_{\text{knee}} \) was calculated from the design charts in the CIDECT design guide (Packer et al 1992).

Table 4.4 indicates that all but one of the welded connections satisfied Equation 4.2. Specimen KJ05, an unstiffened welded connection under closing moment, just failed to satisfy Equation 4.2, by less than 1 per cent. All sections satisfied the validity ranges given in Table 4.5.

There was considerable variation in the rotation behaviour of the stiffened welded connections in both tension and compression (opening and closing moment).

Under **compression (closing moment)**, the 150 \( \times \) 50 \( \times \) 4.0 Grade C450 specimens (KJ01 and KJ03) exhibited notably less rotation capacity \((R_g \text{ and } R_c)\) than the corresponding 150 \( \times \) 50 \( \times \) 4.0 Grade C350 specimen (KJ07). The thicker 5mm specimen in Grade C350 (KJ09) displayed more rotation capacity \((R_g \text{ and } R_c)\) than the 4 mm thick specimen (KJ07), as would be expected for a
stockier section. Only the stiffened welded connections in Grade C350 for both thicknesses (KJ07 and KJ09) were suitable for maintaining a plastic hinge under closing moment for a sufficiently large rotation ($R_\alpha > 4$) suitable for plastic design.

The stiffened welded connections exhibited less rotation capacity than simple beams under pure bending constructed from the same sections, as summarised in Table 4.6.

<table>
<thead>
<tr>
<th>Section</th>
<th>Beam only $R$</th>
<th>SW Connection (closing moment) $R_g$</th>
<th>SW Connection (closing moment) $R_\alpha$</th>
</tr>
</thead>
<tbody>
<tr>
<td>150×50×4.0 C350</td>
<td>12.9, 10.7</td>
<td>2.8</td>
<td>6.0</td>
</tr>
<tr>
<td>150×50×4.0 C450</td>
<td>6.6, 7.7, 9.1</td>
<td>0.9, 0.9</td>
<td>2.8, 3.3</td>
</tr>
</tbody>
</table>

Table 4.6: Comparison of Rotation Capacity of Beams and SW Connections

Possible reasons for the reduced rotation capacities of the connections under closing moment compared to beams under pure bending include:

- Local buckling behaviour is different for members under uniform moment compared to members under a moment gradient, as shown by tests of I-section beams (Lay and Galambos 1965, 1967).
- There is net axial compression in the section which will initiate local buckling earlier compared to bending alone.
- Imperfections induced into the section by the welding process can affect the local buckling behaviour of the RHS.
- The values of $R_g$ underestimate the rotation capacity occurring at the point of maximum curvature, since the strain gauges were not necessarily at the point of maximum curvature.

Under tension (opening moment), the failure was always associated with the weld. It is difficult to establish the effect that the different welding procedures had on the rotation capacity of the connections. There can be small differences between each welding run, even using the same procedure, that can affect the performance of the connection. Imperfections in the weld, or small regions where extra heat was input, can affect the ductility of the connection markedly.
It has been shown previously that welded connections in C450 *DuraGal* have less ductility than Grade C350 (Zhao and Hancock 1995a, b). The results in this report are generally in agreement with Zhao and Hancock. The Grade C350 specimen KJ10 had a low rotation capacity, but the failure was weld tear-out from the plate, indicating poor welding was the probable cause. Table 4.4 includes the heat input of the various welding procedures used in the manufacture of the different connections (Appendix G).

HAZ fracture was the main cause of failure in the tension (opening moment) tests. The heat input during welding is usually considered an important parameter in assessing the likelihood of HAZ failure. However specimen KJ02, which was welded by an independent fabricator, had the lowest heat input for the butt welded specimens, yet had the second lowest rotation capacity ($R_g$ and $R_c$). Specimen KJ08 had a very large rotation capacity, and did not experience HAZ fracture, but failed eventually by weld tear-out. KJ08 gave an extraordinarily large value of $R_g$ compared to all other specimens (4 - 10 times as large). One reason for this is that the strain gauge position may have coincided with the point of maximum curvature more closely than the other tests. The value of $R_c$ for KJ08 was only 2 - 5 times as large as the other specimens. Regardless of the actual values of $R_c$ and $R_g$, it is evident that KJ08 deformed more than any other stiffened welded connection, and eventually formed a local buckle on the outside of the knee, despite the net axial tension on the section, before the weld ultimately failed. The detail of specimen KJ08 was slightly modified to Alternative Detail B (Figure 4.3) and the new connection (KJ12) experienced a much lower rotation capacity ($R_g$ and $R_c$), despite the fact the new detail was designed to make welding easier. The fillet welded C450 connection (KJ11) had less heat input than the other Grade C450 specimens and exhibited slightly higher rotation capacity ($R_g$ and $R_c$), but still failed by HAZ fracture. Only one specimen (KJ08) showed sufficient rotation capacity to be suitable for plastic hinge formation and maintenance ($R_c > 4$). The variability in behaviour meant that it was not possible to guarantee that the rotation capacity could be repeated.

Samples KJ01 and KJ02 were examined by BHP Structural and Pipeline Products after testing and the welds were closely examined. Vickers Hardness tests showed that on average the hardness of the parent metal was 177 HV 5, while in the heat affected zone (HAZ) the average hardness was 150 HV 5. The drop in hardness indicates that the HAZ experienced softening, corresponding to a reduction in the ultimate stress ($f_u$) of approximately 80 MPa, and a reduction of at least 80 MPa in the yield stress ($f_y$).
As indicated in the welding procedures in Appendix G, the circumferential welds of the RHS were performed as separate runs on each of the flat faces of the RHS, with each weld starting and ending at the corner of the RHS as shown in Figure 4.25(a). Advice after the completion of the test program indicates that this was not the best welding procedure to use. It would be more prudent to alter the weld run sequence to avoid welds starting and ending in the corners as shown in Figure 4.25(b).

It is not relevant to compare the rotation capacities of the welded stiffened and unstiffened connections under opening moment with the capacities of the beams under pure bending, as the mode of failure was different, being associated with fracture as opposed to local instability.
4.4.2 BOLTED PLATE AND SLEEVE KNEE CONNECTIONS

All connections reached a moment greater than $M_p$ (of the RHS only). Since the sleeve connections had a much thicker region at the apex, the maximum moment reached at the centreline is notably higher than $M_p$ of the RHS. The axial force in the RHS is low (approximately 5 - 10 % of the squash load). In addition, all sections can still be classified as Class 1, even though the Class 1 limit has been reduced due to the presence of axial compression, and hence would be classified as suitable for plastic design by Eurocode 3.

It is difficult to assess the rotation capacity of the different joints, particularly the sleeve connections. Since the point of maximum curvature in the sleeve connections was not at the apex, but at the end of the sleeve, the total vertical deflection of the test specimen was lower compared to a connection in which the curvature was concentrated at the apex. The total deflection of the bolted connection was increased by the separation of the connection at the junction of the two legs (refer to Figure 4.14). The welded sleeve connection was stiffer than either the bolted sleeve or the bolted plate connections.

The values of rotation capacity based on the strain gauges ($R_g$) do not give reliable estimates of the true rotation capacity of any of the connections. The values shown in Table 4.4 are very low, since the strain gauges were not at the point of maximum curvature. The curvature was highly localised at the end of the sleeve, or the end of the plates. Consequently, the rotation capacity based on angle change ($R_a$) is a more appropriate measure to use. The moment-rotation curves in Figures 4.22 and 4.23 were normalised with respect to $M_p$ and $\bar{\alpha}_p$, and $\bar{\alpha}_p$ was calculated from a first order elastic analysis. If the assumptions in the rotation analysis were correct, then the initial slope of the moment-rotation curves should be unity (1.0). It can be seen in Figures 4.22 and 4.23 that the sleeve connections, particularly the bolted sleeve, are less stiff than all the other connections.

The lower stiffness of the sleeve joints is due to the rotation analysis described in Section 4.3.5. It is assumed that all plastic rotation occurs at the end of one sleeve. Curvatures were observed at the ends of both sleeves, but more rotation was visible at the end of the sleeve in the lower leg. The end of the sleeve in the lower leg was more eccentric to the line of action of the applied load than the upper leg, and yields before the other end of the sleeve. The deformations of the bolted sleeve connection were increased as the connection began to open up (refer to Figure 4.14). The
analyses calculates the rotation assuming that all rotation occurs at only one hinge point. The assumption that all plastic rotation occurs at one end point is slightly incorrect. It is difficult to model the behaviour of the connection as the two legs separated. Since only small curvatures were observed at the end of the sleeve on the upper end compared to the end of the sleeve on the lower leg, and since the initial slopes of the moment-rotation curves for the welded sleeve connection are close to one, then it suggests that the rotation analysis for the WS connections is a reasonable measure of the behaviour. The initial slopes of the moment rotation curves for the bolted sleeve connections were notably less than one, caused mainly by the opening of the sleeve. As the opening of the sleeves was not monitored during each test, it is not possible to account for it in the analysis.

It is also unknown whether the RHS and sleeve act prismatic or non-prismatic. (Prismatic action is defined as the two objects acting as a unit with the stiffness of the combined unit, while non-prismatic action occurs when the two elements act independently and slip occurs between the two). The analysis assumed that there was prismatic action, whereas the true behaviour was probably a mixture of prismatic and non-prismatic action.

Regardless of the difficulties in quantifying the rotation capacity exactly, it is evident that large plastic rotations, much larger than those of the stiffened connections, were achieved by the sleeve connections.

The bolted plate connection (BP) displayed plasticity, however the problem of weld or heat affected zone failure under tension loads was not fully overcome. The BP connection exhibited less stiffness in tension compared to compression, caused by the small opening up of the two plates during the test.

One major benefit of the welded sleeve connection was that it forced the plastic hinge to form away from the junction of the two RHS. The stress and strain on the weld was reduced, since the sleeve transferred most of the load between the two legs of the connection.
It was decided that the welded sleeve connection was the most suitable structurally for the portal frames. However, this would pose difficulties with the construction of the frame. It was hard to insert the sleeve into the long column and rafter of the frame (both over 3 metres long). It was important that the sleeve fitted snugly into the RHS legs. The angle grinding of the sleeve was a labour intensive operation. At present, the sleeve connection may not be a commercially viable possibility. Since the sleeve fits inside the RHS, different sizes are required for the same outer size RHS, since the different thickness of RHS results in different inner dimensions. It is labour intensive and expensive to produce.
4.5 CONCLUSIONS

4.5.1 SUMMARY

The plastic behaviour of various types of knee joints constructed from cold-formed RHS for use in portal frames was examined. Welded stiffened and unstiffened knee joints, bolted knee joints with end plates, and connections with a fabricated internal sleeve, were included in the experimental investigation. It was found that in tension (opening moment), there was often fracture in the heat affected zone of the cold-formed RHS at low rotation values. In compression (closing moment), web local buckling occurred near the connection. The major finding of the experimental study was the unexpected fracture of the welded stiffened joints near the heat affected zone.

The stiffened and unstiffened welded connections satisfied the strength interaction requirements of the CIDECT RHS connection design guide, yet did not prove entirely suitable for plastic hinge formation. There was variability in the rotation behaviour of the stiffened welded connection under opening moment, while under closing moments there was insufficient rotation prior to local buckling for some of the sections. It is not suggested that the CIDECT design recommendations for stiffened welded L-joints are incorrect, as they were based on tests of hot-formed HSS, but the recommendations may not be suitable for cold-formed HSS. The unstiffened welded joints were not able to reach the plastic moment.

The bolted end plate connection was less stiff than the other connections, due to plate separation. It did not demonstrate sufficient rotation capacity to be considered suitable for plastic design.

The internal sleeve connections exhibited the most suitable behaviour for plastic design. The plastic hinge was moved from the apex of the joint due to the extra strength and stiffness provided by the internal sleeve. The bolted sleeve connection opened up as load was increased on the specimen. The welded sleeve did not open up like the bolted sleeve and hence was stiffer. The welded sleeve connection provided sufficient rotation capacity to be considered suitable for a plastic hinge location in a plastically designed frame.
4.5.2 FURTHER STUDY

An important observation in the connection tests was the fracture of the welded stiffened connections at low rotation values. It is recommended that there be further examination of the ductility of welded joints in cold-formed sections to find alternative connection details, welding procedures or even steel metallurgy, that can improve the performance of such connections, and allow them to form plastic hinges.

It is suggested that there could be significant refinement of the internal sleeve connections to allow for the possibility of cheaper production of the internal sleeve joints.
Chapter 5

TESTS OF COLD-FORMED RHS PORTAL FRAMES

5.0 CHAPTER SYNOPSIS

This chapter describes tests on three large scale portal frames manufactured from cold-formed RHS in either Grade C350 or Grade C450 steel. The loading simulated gravity and wind loads. The connections, lateral restraint mechanism, loading method, test procedure, and results are explained.

The results indicate that a plastic collapse mechanism was formed in each frame. There was no failure associated with any of the joints. Simple plastic analysis provided a reasonable estimate of the ultimate load, even though the analysis did not include second-order effects or non-linear material properties. A plastic zone analysis which included second order effects, material non-linearity, and member imperfections slightly over predicted the strength of the frame. The plastic zone analysis did not account for the small loss of connection rigidity (semi rigid connection behaviour), and hence underestimated the deflections and second-order effects, thereby over predicting the strength. A second order inelastic analysis that did not account for member imperfections provided the best estimates of the strengths of the frames. The distribution of curvature throughout the portal frame was analysed. The results indicate that for the portal frames tested a rotation capacity of $R = 4$ is adequate to form the plastic collapse mechanism.

Although the cold-formed RHS do not satisfy the material ductility requirements for plastic design, there was no failure associated with lack of material ductility in the three frames tested, demonstrating that the restriction on cold-formed RHS from plastic design based on insufficient material ductility is unwarranted provided that both the members and connections can achieve the required plastic rotations.
5.1 INTRODUCTION

Plastic design of statically indeterminate frames can lead to higher ultimate loads with associated higher deformations, compared to traditional elastic design methods, particularly when wind load is the major design load, and deflection due to dead load is not critical. Plastic design can offer more economical structures in such cases, and can be applied to structures such as warehouses, garages, and rural structures. Cold-formed rectangular hollow sections (RHS) have properties, such as high torsional stability, which result in less bracing, that could be exploited beneficially in plastic design. However, plastic design methods were verified experimentally on hot-rolled open sections, and some the element slenderness limits for plastic design assumed typical hot-rolled material properties (refer to Chapter 2). As cold-formed RHS have material properties that are considerably different to hot-rolled sections, some current design standards do not permit plastic design for structures constructed from cold-formed RHS. This chapter details a test series on portal frames constructed from cold-formed RHS. The aim is to show that cold-formed RHS portal frames can be designed plastically.

Chapter 3 outlined tests on RHS beams to examine the element slenderness limits for RHS to ensure suitable rotation capacity. Chapter 4 described tests on connections to assess the suitability on various portal frame knee joints to act as plastic hinges. This chapter describes the next experimental part of this project: a series of tests of portal frames constructed from cold-formed RHS.
5.2 MATERIAL PROPERTIES

5.2.1 SECTION SELECTION

For a plastically designed structure, a Compact or Class 1 section is required. Such a section can achieve the plastic moment and sustain the plastic moment for large rotations before inelastic local buckling occurs. Following a series of bending tests (Chapter 3) and connection tests (Chapter 4), a $150 \times 50 \times 4.0$ RHS was selected for the portal frames. The $150 \times 50 \times 4.0$ RHS had exhibited rotation capacity suitable for plastic design ($R > 4$).

A typical RHS was shown in Figure 3.2. The dimensions $d, b, t, r_e$ (depth, width, thickness, and external corner radius) were defined. Section 3.2.1 outlined the general properties of the Grade C350 and Grade C450 ($DuraGal$) RHS.

5.2.2 TENSILE COUPON TESTS

Tensile coupons were cut from the flats and corners of the RHS, and were prepared and tested in accordance with AS 1391 (Standards Australia 1991b) in a 250 kN capacity INSTRON Universal Testing Machine or a 300 kN capacity SINTECH testing machine.

Since the steel was cold-formed, the yielding was gradual. Accordingly the reported yield stress ($f_y$) was the 0.2% proof stress. The average of the static yield stress from both of the adjacent faces was used in the determination of plastic moment ($M_p$). Full details of the test method and the results are given in Appendix C. The results are summarised in Table 5.1.
<table>
<thead>
<tr>
<th>Specimen</th>
<th>Cut from section</th>
<th>$f_y^1$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frame 1</td>
<td>150 × 50 × 4.0 C350</td>
<td>411</td>
</tr>
<tr>
<td>Frame 2/3</td>
<td>150 × 50 × 4.0 C450</td>
<td>438</td>
</tr>
</tbody>
</table>

*Notes:* (1) The average value of $f_y$ for both adjacent sides is used for the whole section.
(2) Nominal strengths: Grade C450: $f_{yn} = 450$ MPa and $f_{un} = 500$ MPa,
Grade C350: $f_{yn} = 350$ MPa and $f_{un} = 430$ MPa.

Table 5.1: Summary of Tensile Test Results

### 5.2.3 PLASTIC BENDING TESTS

Before assessing the performance of the entire frame, plastic bending tests were performed on simple beams constructed from the same cold-formed RHS that were to be used in the portal frames. The plastic bending tests were described in Chapter 3. The conclusion from the results was that the RHS chosen for the portal frames, 150 × 50 × 4, in either Grade C350 or Grade C450, were capable of maintaining a plastic hinge for significant rotations and were suitable for plastic design.

### 5.2.4 CONNECTION TESTS

Before the portal frame tests could commence, a series of pilot connection tests was carried out, to find a suitable knee connection between the column and rafter of the portal frame. Initial analyses had shown that this connection would need to support a plastic hinge. The test series was described in Chapter 4. The main conclusion was that an internal sleeve knee connection was required at the plastic hinge region.
5.2.5 WELDING PROCEDURES

Welding procedures may be pre-qualified by Clause 4.3 of Australian / New Zealand Standard AS/NZS 1554.1 (Standards Australia 1995). The joint preparation, welding consumables, workmanship and welding techniques need to be pre-qualified to AS/NZS 1554.1 and are summarised in a welding procedure sheet.

A variety of welding procedures was used. The procedures are listed in Table 5.2, and are shown in Appendix G. All welding of the portal frames was performed in the Department of Civil Engineering, The University of Sydney.

<table>
<thead>
<tr>
<th></th>
<th>Bolted moment end plate at apex</th>
<th>Internal sleeve knee connection</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frame 1 (C350)</td>
<td>TJW04</td>
<td>TJW07</td>
</tr>
<tr>
<td>Frame 2 (C450)</td>
<td>TJW01</td>
<td>TJW07</td>
</tr>
<tr>
<td>Frame 3 (C450)</td>
<td>TJW01</td>
<td>TJW07</td>
</tr>
</tbody>
</table>

Table 5.2: List of Welding Procedures for the Connections in the Portal Frames
5.3 PORTAL FRAME DETAILS

5.3.1 GENERAL DETAILS AND NOMINAL DIMENSIONS

The general layout of the frames is shown in Figure 5.1, and a photograph is shown in Figure 5.2. Each frame spanned 7 metres, with an eaves height of 3 metres, and a total height of 4 metres. The RHS were arranged so that in-plane bending of the frame resulted in major principal axis (x-axis) bending of the RHS. There was a collar tie which joined the midpoint of each rafter. The collar tie was constructed from a pair of hot-rolled channel (C) sections, and was needed to apply load within the span of the rafters. Each frame was pin based. The frame was laterally braced at several critical locations. Subsequent subsections of this chapter describe the various components of the portal frames.

Figure 5.1: Layout of Portal Frame
Figure 5.1 shows that a vertical load can be applied by the gravity load simulator. The nature of the simulator means that downwards vertical loads can be applied easily, but large upwards vertical loads require significant bracing of the gravity load simulator. Horizontal load can be applied to the column reasonably easily.

A typical portal frame is subjected to gravity and wind loads. Figure 5.3 shows a typical distribution of gravity loads on a portal frame, resulting from some combination of dead, live and snow loads. A typical distribution of pressures from a wind (Australian Standard AS 1170.2 (Standards Australia 1989b)) would result in uplift on a roof, but it was stated that upwards forces would be difficult to apply. However, for a horizontal wind and a significant opening on the leeward or side wall of a structure, the uplift caused by the wind blowing over the roof, is approximately cancelled by the internal suction caused by the opening, and there are net pressures on the leeward and windward walls. Hence one wind load case can be represented by the pressure distribution shown in Figure 5.4.
Typical dead/live/snow load distribution

Figure 5.3: Typical Gravity Loads on a Portal Frame

Typical wind load distribution for dominant opening on the leeward or side wall

Figure 5.4: Typical Loads on a Portal Frame

Therefore, a combination of wind and gravity loads can be simulated by applying a single vertical load ($V$) to the collar tie, and a horizontal load ($H$) to one column of the test frame as shown in Figure 5.5.
Point load simulation

Figure 5.5: Position of loads

Three portal frames were tested, as summarised in Table 5.3 below.

<table>
<thead>
<tr>
<th>Frame</th>
<th>Made from section</th>
<th>Ratio of vertical to horizontal load $V/H$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frame 1</td>
<td>$150 \times 50 \times 4.0$ C350</td>
<td>40</td>
</tr>
<tr>
<td>Frame 2</td>
<td>$150 \times 50 \times 4.0$ C450</td>
<td>40</td>
</tr>
<tr>
<td>Frame 3</td>
<td>$150 \times 50 \times 4.0$ C450</td>
<td>3.3</td>
</tr>
</tbody>
</table>

Table 5.3: Summary of Loading in Portal Frame Tests
5.3.2 CONNECTIONS

5.3.2.1 Knee Joint (Column - Rafter Connection)

Plastic hinges often form at the connections in frames, and hence the joint must be able to produce large rotations while maintaining the plastic moment. The series of prototype connection tests (Chapter 4) indicated that even with compact sections, some connection types, such as stiffened welded connections, were unable to achieve sufficiently large rotations suitable for plastic design, under either opening moment (due to fracture at the weld during the plastic rotation), or closing moment (due to local buckling). Local stress concentrations and effects of welding may have reduced the rotation capacity of a connection compared to the rotation capacity of a simple beam of the same size RHS. The solution was found by employing an internal sleeve connection.

The welded internal sleeve connection is shown in Figure 5.6. An internal hollow steel sleeve was fabricated to fit inside the mitred knee connection. Two open “V” shaped pieces of 10 mm steel were profile cut at the same angle (106°) as the whole joint. Flange plates were fillet welded to “V” pieces to create a box section. After fabrication, the edges of the sleeve were ground to ensure that the sleeve would fit into the RHS legs. A groove was cut into the sleeve to allow for the internal seam weld of the RHS. The sleeve was fitted snugly inside each RHS leg of the joint. The sleeve was slightly oversized so that substantial sledgehammer blows were required to insert the sleeve into the RHS. The tight fit ensured that load could be transferred between the RHS and the sleeve mainly by friction. The sleeve thickness of 10 mm was chosen so that the plastic moment of the RHS was slightly smaller than the plastic moment of the sleeve itself. Finally, the two legs were butt welded together with the sleeve acting as a backing plate for the weld (refer to Appendix G for welding procedures).
Since the junction of the two legs was strengthened by the sleeve, the plastic moment at the junction was approximately twice the value of $M_p$ of the RHS alone. Hence, the plastic zone would be forced to form in the RHS outside the region of the sleeve and away from the weld. The tests on stiffened welded connections (Chapter 4) suggested that there was insufficient ductility at or near the weld to maintain a plastic hinge. Since the region near the weld was much stiffer, and the plastic zone was forced to move away from the weld, it was anticipated that the strain on the weld would be significantly reduced, and probably elastic. Figure 5.7 illustrates the distribution of extreme fibre stress in knee connections with and without an internal sleeve. Therefore, the possibility of failure at the weld was reduced.
5.3.2.2 Apex Joint

A moment resisting bolted end plate connection was used at the apex of the frame. The end plate connection involved a 10 mm plate (Grade 350 to Australian Standard AS 3678) butt welded to each end of the RHS (refer to Appendix G for welding procedures). High strength fully tensioned bolts (Category 8.8/TF in AS 4100), manufactured to Australian Standard AS 1252, join each half of the connection. A minimum bolt tension of 95 kN in the installed bolts was ensured by the use of Coronet Load Indicating Washers. The connection is shown in Figure 5.8.

Appendix H outlines tests on prototype bolted moment end plate connections, however the connections tested in Appendix H were straight joints (180°), whereas the connections in the portal frame had an angle of 148° to account for the pitched roof. The geometry of the frame ensured that the apex connection did not experience large loads, and initial analyses showed that the moment at the apex would reach approximately one half of the plastic capacity of the RHS, at the plastic collapse load of the frame.
Fully tensioned high Strength M16 bolts (only 2 of 8 shown)

Detail A

Section B-B

Figure 5.8: Apex Joint
5.3.2.3  Base

A common connection for portal frames is the bolted base plate connection to a concrete footing. The column base plate is a flexible connection which is considered pinned by designers. An idealised pinned connection was chosen for the portal frame tests. The webs of the RHS were stiffened locally by a steel plate, and a 30 mm diameter high strength steel pin was inserted through the neutral axis of the RHS, similar to the “pin loading” method used in the plastic bending tests (Chapter 3). A thick steel base plate supported the pin. The base plate was securely fastened to the strong floor of the laboratory. The connection was greased to reduce friction. Details of the connection are shown in Figure 5.9.

Figure 5.9: Base Connection
5.3.2.4 Channel Collar Tie

Many portal frames have a collar tie. The collar tie joins the two rafters between their midpoints. The collar tie is often constructed from the same section as the whole frame. Under gravity load, the tie is in tension and the moment at the apex joint is reduced. The collar tie is also used for support of services in a commercial structure.

In the experimental frames, the collar tie transferred the vertical load from a hydraulic jack to the rafters. The simulated gravity load acted at the centre of the tie, and the collar tie transferred load to the frame at the ends of the tie. Since the collar tie was carrying additional load in the experimental structure compared to a prototype frame, extra strength was required for the collar. Hence, a pair of channels was used, one channel on each side of the RHS frame. Steel pins inserted through a hole cut through the centre of the RHS webs and the channels connected the rafters to the channels, as shown in Figure 5.10. The ends of the channels were mitred to allow space for the lateral restraint system.

Figure 5.10: Rafter - Collar Tie Connection
5.3.3 LATERAL RESTRAINTS

Plastic design requires that no out-of-plane (lateral) buckling occurs in the frame. In an actual multi-bay portal frame structure, purlins linking adjacent frames provide lateral restraint to each frame. Often the final bay in a structure has cross bracing which restrains the entire frame against out-of-plane deflections. For the single frame being tested, a system was required which would restrain the frame out-of-plane for the large deflections expected in-plane. A single restraint fixed at one end is suitable only for small in-plane deflections. It was anticipated that large in-plane deflections of the order of 300 - 500 mm would occur in the frame.

The system chosen was used in large scale frame tests at Lehigh University (Yarimci et al 1967). There are two parallel levers which are pin jointed together at one end by a coupler. The other end of each lever is pin ended to a fixed support. As shown in Figure 5.11, the midpoint of the coupler traces out an approximate straight line as the mechanism moves. For very large movements the locus of the midpoint deviates slightly from the straight line. The mechanism can easily be applied to a three dimensional system if the joints allow rotation in every plane. Hence the system can be used as a lateral restraint system if the midpoint of the coupler is connected to the frame. Lateral deflections of the restraint point are prevented by the mechanism. Figure 5.11 also compares the performance of the mechanism compared to a single element restraint.

Holes were drilled through the minor axis of symmetry of the RHS and a threaded rod welded into position. The threaded end was always on the outside of the frame. A length of angle (the coupler) was placed over the rod and secured in place by bolts. The cross section through the RHS at a restraint point is shown in Figure 5.12. The coupler was connected to the two lever arms with a ball and socket joint. The lever arms were connected to auxiliary columns fixed to adjacent strong floor beams. Each lever and the coupler was approximately 1 metre in length. The geometry allowed for in-plane deflections of the order of 600 mm while producing out-of-plane deflections of less than 1 mm.
Figure 5.11: Lateral Restraint Mechanism (adapted from Yarimci et al. 1967)

Figure 5.12: RHS Cross Section at Restraint Point
The locations of the lateral restraints are shown in Figure 5.13. The potential plastic hinge locations (the knee joint and the mid points of the rafters) and the horizontal loading point in the north column were restrained. For Frame 1, the north column was restrained at the loading point while the south column was not restrained. Some minor axis bending of the south column was observed during testing for Frame 1. Hence the south column was restrained in Frame 2 and Frame 3, at approximately the same height as the restraint on the north column.

![Figure 5.13: Lateral Restraint Locations](image)

The high torsional rigidity of RHS means that full lateral restraint can be achieved with widely spaced lateral restraints. Zhao, Hancock and Trahair (1995) give an equation for restraint spacing based on the moment gradient of the member. Figure 5.14 shows the bending moment distribution in the portal frame for Frame 1 and Frame 2. For the columns, the ratio of end moments ($\beta$) is $\beta = 0$. For such a moment gradient the bracing interval is $L_0 \approx 26$ m and is significantly greater than the height of the columns. Between the midpoint of the rafter and the apex or the knee joints, the rafter is in double curvature. In such a case, $\beta = 1$, and $L_0 \approx 27$ m. The most severe case for lateral buckling is the uniform moment case ($\beta = -1$ and $L_0 \approx 3$ m) does not occur in any segment of the portal frame, and is significantly greater than the unbraced lengths in the rafters.
Figure 5.14: Bending Moment Distribution at Formation of First Plastic Hinge

(Normalised with respect to the Plastic Moment)

(Collar Tie between midpoints of rafters not shown)
5.3.4 CONSTRUCTION SEQUENCE

The two columns and two rafters were fabricated separately. The RHS were supplied in standard lengths and then saw cut to correct length. The ends of the sections were mitre cut with a band saw at the appropriate angle (for the knee and apex connections). Stiffening plates for the loading points were welded in place and the holes for the pins and restraints were drilled. The end plates for the apex joint were butt welded to the rafters and the internal sleeves were manufactured to fit snugly into the columns and rafters. Strain gauges were fixed to the RHS at the appropriate locations, and dimensions of the sections were measured.

The sleeves were hammered into each rafter. It was a tight fit and required substantial hitting with a sledge hammer. The other end of each sleeve was then hammered into the column. Each knee joint was welded together. The two halves of the frame were bolted together at the apex joint. At all stages during the previous sequence, the frame remained in a horizontal plane on the ground. Before the pair of channels that made up the collar tie was pinned to the rafters, the exact locations of the pin holes in the channels were identified and cut, to minimise any lack of fit moments induced in the frame during the construction process. The channel tie was connected to the frame with the pins.

The constructed frame was lifted by a crane into place. One pin base had been securely fixed to the strong floor previously and one column was connected to the base. The other base was then slid into the correct position to avoid lack of fit moments being induced. The second base was fixed to the strong floor and the other column connected to the base. In a commercially erected frame, moments can be induced due to lack of fit since it is not possible to move concrete footings once they have been poured.

The lateral restraints were connected to the frame. Finally overall imperfection measurements were made.

Appendix I includes some photographs of the frames during construction.
5.4 TEST PROCEDURE

5.4.1 INSTRUMENTATION

Strain gauges were placed at important locations on the frame as shown in Figure 5.15. At each location one gauge was placed at the centre of the top flange and another at the centre of the bottom flange of the RHS. Curvature and net axial strain could be calculated from the strain values on the top and bottom of the RHS.

Electronic displacement measurement devices, such as Linearly Varying Displacement Transducers (LVDT) were unsuitable for measuring the large expected horizontal and vertical displacements of the frame. Firstly, a fixed datum is needed, which would require an auxiliary frame to be constructed. Secondly, the device would have to sway with the frame if, for example, vertical displacements were being measured.

As a solution, measuring tapes were hung from the frame with a small weight on the bottom of each tape. The measuring tapes remained vertical as the frame swayed. The tapes were hung from the apex joint, and the mid point of each rafter as shown in Figure 5.16. A Wild NA2 precise level was used to read the measuring tape at various stages during the test. It was possible to estimate readings to 0.1 mm using the Wild NA2 level.
The horizontal in-plane sway deflections of the north knee joint were measured with a Wild ZL Zenith Plummet. The Zenith Plummet contains a mirror and views vertically upwards. A graduated rule was attached to the knee joint of the frame and displacements readings were taken to an estimate of 0.1 mm.

Out of plane deflections were monitored by a Sokkisha TM1A theodolite with the horizontal circle of the instrument fixed. A graduated rule (or lateral scale) was connected perpendicular to the frame at the apex joint. It was possible to measure lateral deflections at the apex to an estimation of 0.1 mm.

An LVDT was positioned near the base of the frame at the north column. It was possible to monitor the rotation of the column base connection from the displacement values. The MTS Microconsole provided information on the load and stroke of the hydraulic actuator.

During each test, all electronic readings from the strain gauges, LVDT, and MTS Microconsole were recorded using a SPECTRA data acquisition system. Theodolite and level measurements were recorded manually.
5.4.2 DIMENSIONS AND IMPERFECTION MEASUREMENTS

The dimensions of the RHS for each frame were measured, and the average values are given in Table 5.4. The mean of the yield stress values obtained from the coupons cut from the adjacent faces is also included, as well as the plastic moment based on nominal and measured properties. Note that since the material for Frame 2 and Frame 3 were from the same rolling procedure, that the properties for both these frames are the same.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Cut from section (mm)</th>
<th>d (mm)</th>
<th>b (mm)</th>
<th>t (mm)</th>
<th>$r_e$ (mm)</th>
<th>$f_y$ (MPa)</th>
<th>$M_{pn}$ (kNm)</th>
<th>$M_p$ (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frame 1</td>
<td>150×50×4.0 C350</td>
<td>150.5</td>
<td>50.4</td>
<td>3.94</td>
<td>7.5</td>
<td>411</td>
<td>22.9</td>
<td>27.3</td>
</tr>
<tr>
<td>Frame 2</td>
<td>150×50×4.0 C450</td>
<td>150.6</td>
<td>50.4</td>
<td>3.92</td>
<td>7.5</td>
<td>438</td>
<td>29.4</td>
<td>29.0</td>
</tr>
<tr>
<td>Frame 3</td>
<td>150×50×4.0 C450</td>
<td>150.6</td>
<td>50.4</td>
<td>3.92</td>
<td>7.5</td>
<td>438</td>
<td>29.4</td>
<td>29.0</td>
</tr>
</tbody>
</table>

Table 5.4: Summary of RHS Dimensions for Portal Frame Tests

The overall dimensions (span and height), and individual member imperfections were measured, after the frame had been erected. Figure 5.17 and Table 5.5 define and list the various measurements of the frame.

The most notable imperfections were the out-of-plumb measurements of the columns. There was little allowance for adjustment at the knee joint, so if the ends of RHS were mitre cut at a slightly incorrect angle, the columns would not be vertical, unless they were forced to be vertical. A 1° change in angle at the apex causes the column base to move approximately 70 mm. Forcing the columns to be vertical would have induced lack-of-fit moments, and consequently it was decided not to force them to be vertical.
Figure 5.17: Definition of Dimensions and Imperfections

Table 5.5: Dimensions and Imperfections

<table>
<thead>
<tr>
<th>Span</th>
<th>Height</th>
<th>Out of plumb</th>
<th>Member imperfections</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frame 1</td>
<td>6982 2996 4007 3000</td>
<td>0 -4 0 -4 -4 0</td>
<td></td>
</tr>
<tr>
<td>Frame 2</td>
<td>6900 2998 3975 2995</td>
<td>33 -61 0 2 0 -2</td>
<td></td>
</tr>
<tr>
<td>Frame 3</td>
<td>6920 3002 4002 3005</td>
<td>60 45 1 -1 -1 0</td>
<td></td>
</tr>
</tbody>
</table>

Note: All dimensions are in mm.
The initial out-of-plane imperfections were measured at various positions on each frame after the lateral restraints had been attached. The restraints were slightly adjustable, but it was virtually impossible to ensure that all the restraints were aligned along the centre of the frame. The out-of-plane imperfections are shown in Figure 5.18.

Figure 5.18: Initial Out-of-Plane Imperfections
5.4.3 LOADING METHOD

A combination of vertical gravity and horizontal wind loads was selected for the test series. The first two tests had a large vertical load compared to the horizontal load \((V/H = 40)\), and the final test had a larger horizontal load component \((V/H = 3.3)\). In a typical portal frame, gravity and wind loads are transferred to the frame from the sheeting via purlins and girts, similar to a distributed load. It is difficult to replicate distributed loads in a laboratory, and hence the loads were applied as point loads.

The vertical load was applied at the midpoint of the collar tie. Large in-plane sway displacements were anticipated and it was essential that the downwards loads remained vertical. True gravity loads could have been achieved by hanging dead weights from the structure. However, it was not feasible to use dead weights for the total load required (approximately 70 kN). A hydraulic jack was chosen to achieve the large load. A jack connected directly to the strong floor of the laboratory would not produce a vertical load due to the sway deflections of the frame (refer to Figure 5.19).

![Diagram](image)

- (a) True gravity load
- (b) Non-vertical load caused by fixed base load actuator

Figure 5.19: Gravity Loads in Sway Frames (adapted from Yarimci et al 1967)
A gravity load simulator (Yarimci *et al* 1967) is a device which allows the jack to move with the sway of the structure. The simulator is attached to the strong floor of the laboratory. It has two inclined members, pinned together by a rigid triangular unit, and pinned at the other end of the inclined members to the base. The jack is connected to another vertical member, which is pinned to the base of the triangular section.

As the unit sways, the instantaneous centre of rotation (ICR) of the two inclined arms moves as indicated in Figure 5.20. To maintain equilibrium the line of action of the force must act through the instantaneous centre of rotation. The bottom vertex of the triangle remains almost vertically below the instantaneous centre and hence the direction of the load remains vertical. The bottom vertex of the triangle moves in a horizontal plane (Figure 5.21). For large sways, the line of action deviates slightly from the vertical, as can be seen in Figure 5.21.
A 150 × 50 × 5.0 C450 RHS was pinned to the bottom vertex of the triangular section of the simulator. The other end of the RHS was bolted to a 250 kN capacity MTS Actuator (Model 244.31) controlled by an MTS 458.20 Microconsole. The actuator was bolted to the channel tie of the portal frame. The gravity load simulator - actuator - channel connection is detailed in Figure 5.22.

Figure 5.22: Connection of Gravity Load Simulator to Portal Frame
The horizontal load was much smaller than the gravity load and was achieved with dead weights as shown in Figure 5.23. A pin was inserted through the north column of the frame. A pair of cables was attached to the pin and slung over a roller bearing (supported by an auxiliary frame) to a cradle upon which lead weights were placed.

Figure 5.23: Horizontal Load Apparatus
5.4.4 LOADING PROCEDURE

Before testing, the actuator ram was bolted to the channel tie. A small amount of pre-load, less than 1 kN (~ 1 - 2 % of the maximum load), was placed on and removed from each frame to reduce any possible effects of “settlement” or take up of fit in the connections.

During testing, the ram was lowered at a constant rate of approximately 0.06 mm/s. For the first two tests, the ratio of the vertical load to the horizontal load was 40. At increments of 10 kN in vertical load, a lead block of approximately 0.25 kN was added to the side load cradle. Closer to the ultimate load, lead weights of 0.125 kN were added for every 5 kN of vertical load. Similarly for the third test, lead blocks were loaded periodically to maintain a ratio of vertical to horizontal load of approximately 3.3.

At regular intervals during the test, the ram was halted for approximately two minutes. After pausing, load measurements were made to obtain a value of the static load. At these times, all displacement measurements were taken.

The test continued until well past the ultimate load. The actuator ram was near its maximum stroke of 200 mm, and the large sway deflection of about 300 mm was near the limit of sway for the gravity load simulator. The horizontal load was removed and the ram raised until the vertical load was zero.
5.5 RESULTS

5.5.1 OBSERVATIONS

Once loading commenced, the deflections initially increased elastically with the load. Frame 2 was unloaded after the end of the elastic region and then reloaded back to the previous position and testing continued. The need to unload was caused by difficulties with the pin base on the north column, where the connection was about to stick due to lateral movement, preventing rotation at the base. The load was removed and the joint was corrected, allowing rotation.

For each frame, large plastic rotations became visible at the channel loading point in the north rafter (Local buckle 1 in Figure 5.24). Sway deflections in the frames increased dramatically once the load to reach first yield was reached. Very large deflections were visible in the south column. A local buckle formed in the web of the north rafter (Figure 5.24) for Frame 1 and Frame 2. The buckle did not form exactly at the loading point (predicted as a hinge point by plastic analysis) since there was a small stiffening plate on the RHS at the connection (see Figure 5.10). The buckle formed just past the stiffening plate towards the apex of the frame, at the point of lateral restraint. In Frames 2 and 3, a local buckle was noticed in the south column just below the limit of the sleeve. Figure 5.25 shows a photograph of Frame 3, and the local buckle near the top of the south column is visible.

In Frame 1, out-of-plane deflections of the south column became visible in the vicinity of the ultimate load. Consequently, Frames 2 and 3 had an additional brace point in the south column. No out-of-plane deflections occurred in the south column in Frame 2. For all frames, loading continued past the ultimate load. Very large sway deflections occurred. Frames 1 and 3 reached the displacement limit of the gravity load simulator or the actuator ram before a second local buckle could form. However, for all three frames, two regions of large curvature were clearly identifiable.

There was no sign of failure or distress at either the apex joint or the welded internal sleeve knee joint.
Figure 5.24: Buckle/Hinge Locations for Portal Frame Tests

Figure 5.25: Local Buckle in South Column of Frame 3
5.5.2 ULTIMATE LOADS AND LOAD DEFLECTION CURVES

The horizontal and vertical loads at ultimate for each of the three portal frames tested are summarised in Table 5.6.

<table>
<thead>
<tr>
<th>Frame</th>
<th>Made from section</th>
<th>Load at ultimate (kN)</th>
<th>Vertical</th>
<th>Horizontal</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frame 1</td>
<td>150 × 50 × 4.0 C350</td>
<td>68.2</td>
<td>1.75</td>
<td></td>
</tr>
<tr>
<td>Frame 2</td>
<td>150 × 50 × 4.0 C450</td>
<td>71.5</td>
<td>1.87</td>
<td></td>
</tr>
<tr>
<td>Frame 3</td>
<td>150 × 50 × 4.0 C450</td>
<td>45.7</td>
<td>13.8</td>
<td></td>
</tr>
</tbody>
</table>

Table 5.6: Summary of Ultimate Loads

The load deflection curves are shown in Figures 5.26 and 5.27. Figure 5.26 represents the vertical deflection at the apex of the frame, and Figure 5.27 is the horizontal (in-plane) deflection at the north knee. The unloading and reloading of Frame 2 have been removed from the graphs.
Figure 5.26: Load - Vertical Deflection Graphs

Figure 5.27: Load - Vertical Deflection Graphs
Since Frame 1 and Frame 2 had the same ratio of vertical to horizontal load, it is not unexpected that the load deflection curves have similar shapes. The slightly different stiffness of Frame 1 and Frame 2 is identifiable in the curves. The rigidity of the knee connection and apex connection is an important factor in the frame stiffness. Small variations in bolt tension, or the tightness of the fit of the sleeve in the RHS, can affect the total frame stiffness. As Frame 3 had a greater horizontal load, the larger horizontal deflections are evident.

Out-of-plane (lateral) displacements were monitored at the apex of the frame. Figure 5.28 displays the vertical load and against lateral deflection relationships for each frame.

![Figure 5.28: Out-of-plane Deflections at the Frame Apex](image)

For each frame, the lateral deflections were less than 5 mm (approximately 0.07% of the span) until after the ultimate load had been reached. In Frame 2, there was a slight slippage of one of the restraints (at large deflections - near the effective limit of the lateral restraint mechanisms), which caused the sudden change in lateral deflection.
5.5.3 CURVATURE

In plastic design, plastic hinges are required to rotate for sufficiently large rotations to allow for the plastic redistribution of moment. The wide variety of loading patterns and structural frame shapes results in a large range of required plastic rotations to form the plastic collapse mechanism. However, it is appropriate to adopt a representative value of rotation capacity which is satisfactory for most practical situations. In Chapters 3 and 4, suitability for plastic design was based on a rotation capacity of $R = 4$. Korol and Hudoba (1972) put forward a recommendation of $R = 4$. Eurocode 3 Class 1 and AISC LRFD Compact limits are based on a rotation capacity of $R = 3$. However AISC LRFD states that greater rotation capacity may be required in seismic regions. A value of $R = 4$ was adopted in the determination of the RHS flange slenderness limit in AS 4100 (Hasan and Hancock 1988, Zhao and Hancock 1991) as appropriate for plastic design.

Hence, it is appropriate to investigate the rotation requirements for the portal frames, by examining the curvature at various points in the frame at the ultimate load.

Curvature is calculated from the strain readings on the top and bottom flanges of the RHS as shown in Figure 5.29. The curvature is normalized with respect to the plastic curvature $\kappa_p$, where $\kappa_p = M_p/EI$.

$\tan \kappa = (e_1 + e_2)/d$

Figure 5.29: Calculation of Curvature from Strains
Figures 5.30, 5.31 and 5.32 show the non-dimensional curvatures at the plastic hinge points for each frame. The strain gauge locations are given in Figure 5.15.

Figure 5.30: Load - Curvature Graph (Frame 1)

Figure 5.31: Load - Curvature Graph (Frame 2)
As mentioned above, one hinge was predicted at the collar tie - rafter connection in the north rafter (gauge 4), but since the connection was locally stiffened (Figure 5.10), curvature was concentrated slightly away from the location of the strain gauge. As the curvature was not concentrated exactly at the location of gauge 4, the curvature at the true hinge point was much greater than that measured by the strain gauges. Hence it was necessary to approximate the curvature at the hinge point using the computer finite element program ABAQUS (Hibbit, Karlson and Sorensen 1995). A section of the rafter was modelled, including the stiffening plate at the connection to the collar-tie. It was possible to obtain a numerical relationship between the curvature at the strain gauge location, to a point just adjacent to the stiffening plate, where maximum curvature and local buckling occurred. The adjusted curvature is shown in Figures 5.30, 5.31 and 5.32 as “Gauge 4 modified”.

Figure 5.32: Load - Curvature Graph (Frame 3)
It should be stressed that the analysis was performed on a simple ABAQUS model and it did not consider the formation of the local buckle in the rafter. The finite element simulation produced a relationship between the curvatures at the two adjacent points for an unbuckled beam. Hence the relationship obtained is not valid past the ultimate load since a local buckle had formed. Consequently, the “modified” curve is only graphed up to the ultimate load. Wood (1968) provides a discussion on the distribution of curvature in an element with a falling moment characteristic (such as a local buckle).

Only relatively small curvatures were calculated at gauge 4, for the reasons described above. However, the adjusted curvature values next to gauge 4 demonstrate the formation of the plastic hinge next to gauge 4. The large rotations occurring at the top of the south column (gauge 9) are clearly evident. The reversal of curvature at gauge 4 (for Frame 1) is probably due to the formation of the local buckle adjacent to that point. The curvatures at other strain gauge locations are not shown, but the curvatures indicate that the frame was still elastic at most points.

To investigate the rotation capacity requirements of portal frames, Table 5.7 lists the non-dimensional curvature ($\kappa/\kappa_p$) at the hinge points at various load levels. The most important value is the curvature at the ultimate load ($V_u$), as it gives a measure of the required rotation capacity ($R = \kappa/\kappa_p - 1$). However, particularly for structures with many plastic hinges, very large curvatures are required at the hinges that form first in order for the last hinge to form, while there may be an insignificant increase in load in the formation of the last hinges. Consequently the curvature at approximately 99% of $V_u$ is also listed. For completeness, the maximum curvature at the hinge points (which occurred after the ultimate was reached) is included in Table 5.7.
The distribution of curvature throughout the portal frame was analysed. The points of high curvature required a rotation capacity of $R < 3$ to achieve a load of 99% of the ultimate load. All but one point (the “modified” gauge 4 location for Frame 2) required a rotation capacity of $R < 4$ to achieve the ultimate load.

The values of $\kappa/\kappa_p$ indicate that for the pin-based portal frame, a rotation capacity requirement of $R = 4$ is a sufficient criterion. However, for different framing structures additional rotation may be required. Since the frames tested required only 2 plastic hinges to form a plastic collapse mechanism, the rotation capacity requirement is fairly low compared to other, more highly redundant structures. The background documentation to Eurocode 3 (Sedlacek and Feldmann 1995) gives some rotation capacity requirements for various framing systems.

It should be noted that the curvatures (and deflections) increase considerably from 99% of the ultimate load to the ultimate load. A rotation requirement of $R = 3$ is sufficient to achieve 99% of the maximum load for these frames.
5.5.4 DUCTILITY

Large rotations, and hence large strains, are required from sections in a plastically designed frame in order to form a plastic collapse mechanism. Therefore design standards have material ductility requirements for the suitability of sections for plastic design (Clause 3.2.2.2 of Eurocode 3 and Clause 4.5.2 of AS 4100).

There was no failure associated with lack of material ductility in the three frames tested, indicating that the restriction on cold-formed RHS from plastic design is unwarranted provided that the connections are sufficiently ductile for any required plastic rotations. Before the tests were stopped, the curvature at gauge 9 was at least $\kappa/\kappa_p > 7$, and in the simple bending tests curvatures of the order of $\kappa/\kappa_p = 10$ were obtained, further illustrating that the sections alone (not necessarily the connections) can form large hinges without tearing or fracture. A normalised curvature of $\kappa/\kappa_p = 10$ corresponds to an extreme fibre strain of approximately 3% for a Grade C450 150 × 50 × 4.0 RHS under pure bending. As indicated in Appendix C, the strain at failure $(e_t)$ for tensile coupons cut from the RHS in the portal frames was at least 25% for the flat faces, and approximately 15% for the corners of the RHS. However, in the past there have been examples of cold-formed hollow sections with low values of ductility, such as $e_u = 5.4\%$ in the flats and 2.8% (corners) (Key 1988), where $e_u$ is the percentage elongation at the ultimate stress. The current specification in Australia, AS 1163, specifies that $e_t \geq 16\%$ (Grade C350), and $e_t \geq 14\%$ (Grade C450).
5.5.5 SERVICEABILITY AND DEFLECTIONS

5.5.5.1 The Serviceability Limit State

The serviceability limit state is often the critical limit state in the design of steel structures. The magnitude of deflections of a structure under service loads may be as an important a design consideration as the strength of a structure under ultimate loads.

Deflections in a high-strength RHS portal frame can be more critical than the deflections in a frame constructed from I-sections for three main reasons:

- The increase in strength of structural steel results in lighter, less stiff structures. The use of high strength steel (450 MPa) can give smaller member sizes for strength, but increased deflections.

- In plastic design, the extra strength of the structure after the first plastic hinge has formed is caused by the large plastic deformations of the hinges. Designing a portal frame plastically results in significantly higher deflections compared to elastic design methods. Deflections increase more rapidly after the formation of the first plastic hinge.

- A typical I-section has a shape factor \((S = M_y / M_y)\) in the range \(1.1 \sim 1.2\), while an RHS with an aspect ratio \((d/b)\) of 3, has a shape factor \(S \sim 1.3\). Hence a RHS deflects more (compared to an I-section) as the moment increases from \(M_y\) to \(M_y\).

The deflection of a structure under service or working loads is a criterion for determining the serviceability limit state. The service loads are notably smaller than the ultimate loads on a structure. For example, the Australian Standard AS 1170.1 (Standards Australia 1989a) gives a gravity load for the strength limit state as \(1.25G + 1.5Q\) where \(G\) is the dead load and \(Q\) is the live load. For the serviceability limit state, AS 1170.1 gives the typical gravity load as \(G + \psi_s Q\) where \(\psi_s\) is a load factor and \(\psi_s = 0.7\). As an initial guide, it is common practice to assume that the serviceability loads are 66% of the ultimate loads in a structure.

The portal frames tested did exhibit large deflections in the vicinity of the ultimate load. However, as the serviceability loads are lower than the ultimate loads, it is possible the frames were still elastic under the service loads and hence the deflections were much smaller than the deflection at the ultimate load.
5.5.5.2 Deflection Limits

There is some guidance in design specifications for the deflection limits for columns and beams in structures. In Australia, Appendix B of AS 4100 gives suggested limits for deflections, including limiting the vertical deflections of beams (δv), and the relative horizontal deflection between adjacent frames at eaves level (δh,rel). The AS 4100 limits are derived from Woolcock and Kitipornchai (1986). Another useful source of information on deflection limits is ASCE (1988). The AS 4100 recommendations are summarised in Table 5.8.

<table>
<thead>
<tr>
<th>Structural Element Type</th>
<th>Suggested Limit</th>
</tr>
</thead>
</table>
| Beams                   | \[
\frac{\delta_v}{\text{beam span, } s} \leq \frac{1}{250}
\]
| Columns                 | \[
\frac{\delta_{h,\text{rel}}}{\text{column height, } h} \leq \frac{1}{150}
\]

Table 5.8: Suggested Deflection Limits (AS 4100)

The limits in AS 4100 are not mandatory, but merely suggested limits. It is left to the designer to consider appropriate limits for the structure depending on its nature and purpose. Woolcock and Kitipornchai (1986) indicate that designers would consider a less severe horizontal deflection limit (than 1/150) for structures without gantry frames.

The beam deflection limit of 1/250 is more appropriate for beams supporting floors as opposed to roof rafters. Firstly, one purpose of the limit of 1/250 is to prevent the dynamic response of slender floor systems. Secondly, the vertical deflections of a portal frame roof are increased as the eaves deflect outwards. The survey in Woolcock and Kitipornchai (1986) indicates that the majority of designers would use less severe deflection limits for dead and live load than the 1/250 limit suggested in AS 4100 for a portal frame structure.
### 5.5.5.3 Experimental Deflections

The deflections of the portal frames at serviceability loads can be compared with the deflection limits given Table 5.8 above.

Since the total span \(s\) of the two inclined rafters is 7000 mm, the suggested vertical deflection limit of \(1/250\) is 28 mm. Table 5.9 lists the vertical deflection of the apex at 66\% of the ultimate load \(\delta_{v,66\%}\) and compares \(\delta_{v,66\%}\) to the span \(s\). Shading in Table 5.9 indicates that the suggested deflection limit of \(1/250\) was exceeded. The load \(V_{s/250}\) at which serviceability deflection limit was reached, and the ratio \(V_{s/250}/V_u\) are included in Table 5.9. Figure 5.33 shows the load deflection curves for the three portal frames and indicates the serviceability deflection limit.

<table>
<thead>
<tr>
<th></th>
<th>(V_u)</th>
<th>(s/250)</th>
<th>(\delta_{v,66%})</th>
<th>(\frac{s}{\delta_{v,66%}})</th>
<th>(V_{s/250})</th>
<th>(\frac{V_{s/250}}{V_u})</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frame 1</td>
<td>68.2</td>
<td>28</td>
<td>43</td>
<td>163</td>
<td>30.8</td>
<td>0.452</td>
</tr>
<tr>
<td>Frame 2</td>
<td>71.5</td>
<td>28</td>
<td>50</td>
<td>140</td>
<td>27.5</td>
<td>0.385</td>
</tr>
<tr>
<td>Frame 3</td>
<td>45.7</td>
<td>28</td>
<td>31</td>
<td>226</td>
<td>27.3</td>
<td>0.600</td>
</tr>
</tbody>
</table>

Table 5.9: Vertical Deflections Under Serviceability Loads
As the eaves height of the columns \((h)\) is 3000 mm, the suggested relative horizontal deflection limit of \(1/150\) is 20 mm. As only one frame was tested at a time, it is not possible to determine a relative deflection between adjacent frames in a structure. However, it is reasonable to assume that in a multi-frame structure each of the internal frames would be required to support twice as much load as the end bays of the structure, since the internal bays would have twice the tributary area for wind, snow and roof live loads. Under service loads therefore, an internal bay would be carrying 66\% of the ultimate load, and the end bays would be required to support 33\% of the ultimate load (assuming the end bay is constructed from the same sized sections as the internal bays). But, the end bays may be braced, and are most likely to have an infill (eg girts and sheeting, and possibly bracing), making the end bays very stiff, with small horizontal deflections. An estimate of the relative deflection of the end bay to the adjacent internal bay would be the horizontal deflection at 66\% of the ultimate load, which conservatively assumes that the end bay does not deflect horizontally.

Figure 5.33: Vertical Deflections Compared to the Serviceability Limit
Table 5.10 lists the horizontal deflection of the apex at 66% and 33% of the ultimate load ($\delta_{h, \text{66\%}}$, $\delta_{h, \text{33\%}}$). The relative deflection, $\delta_{h, \text{rel}}$, is the same as $\delta_{h, \text{66\%}}$, and is compared to the column height ($h$). Shading indicates that the deflection limit has been exceeded. Figure 5.34 shows the load deflection curves for the three portal frames and indicates the serviceability deflection limit, though it must be recognised that the horizontal deflection limit is for relative deflections of adjacent bays, rather than absolute deflections.

<table>
<thead>
<tr>
<th>$V_u$</th>
<th>$h/150$</th>
<th>$\delta_{h, \text{66%}} = \delta_{h, \text{rel}}$</th>
<th>$\delta_{h, \text{33%}}$</th>
<th>$\frac{h}{\delta_{h, \text{rel}}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>(kN)</td>
<td>(mm)</td>
<td>(mm)</td>
<td>(mm)</td>
<td></td>
</tr>
<tr>
<td>Frame 1</td>
<td>68.2</td>
<td>20</td>
<td>4.5</td>
<td>1.3</td>
</tr>
<tr>
<td>Frame 2</td>
<td>71.5</td>
<td>20</td>
<td>-3.8</td>
<td>-4.2</td>
</tr>
<tr>
<td>Frame 3</td>
<td>45.7</td>
<td>20</td>
<td>70</td>
<td>22.5</td>
</tr>
</tbody>
</table>

Table 5.10: Horizontal Deflections Under Serviceability Loads

Figure 5.34: Horizontal Deflections Compared to the Serviceability Limit
5.5.5.4 Serviceability Discussion

Initial observations indicate that the suggested vertical deflection limit of AS 4100 was exceeded by all three frames, and the suggested horizontal deflection limit was exceeded by Frame 3. However, several important points must be considered:

- Many designers would use less severe deflection limits for structures such as farm buildings and garages without gantry cranes (Woolcock and Kitipornchai 1986).
- The vertical deflection limit is more applicable to beams supporting floors, than roof rafters.
- A fully completed structure has purlins, girts and sheeting, and also bracing systems which would reduce deflections compared to that of the single bare frames tested.
- The point loads acting on the test frames produce higher deflections than the distributed loads that act on structures in the field.
- The typical column base plate connection exhibits semi-rigid behaviour, compared to the true pinned connections in the test frames. A structure with semi-rigid, rather than pinned, base connection would have less deflections. There would be even less deflections in a fixed base frame.

While a real structure has semi-rigid base connections, distributed loads, and sheeting and bracing systems that would increase the frame stiffness compared to a bare frame, a real structure would also have a higher capacity. Though the frame is stiffer, the higher loads that the frame could sustain would increase the deflections, counteracting the benefit of the higher stiffness.

Heldt and Mahendran (1998) performed a series of tests of portal frame building systems, comprising of multiple bays, cladding and cross bracing. In particular, they were able to compare the deflections of a bare frame without cladding, to the deflections of a fully clad frame, as would occur in practice. For example, at working loads, the lateral deflection of the windward knee of the centre frame was 51 mm for a bare frame, 35 mm for a frame with cladding, and 14 mm for cladding and bracing in the end frame.
Some relevant observations and conclusions of Heldt and Mahendran (1998) were:

“... deflections were significantly different between the bare frames and fully clad frames when they were under lateral loads due to cross winds... Adding the cladding alone did not cause significant reductions to deflections... It was effective only when end frame bracing was added to the experimental building. This observation agrees well with the basic stressed skin behaviour.

For load cases with significant net racking load [horizontal wind load], normal base conditions [semi-rigid behaviour of the column base connection as opposed to an idealised pin behaviour] significantly reduced racking displacements.

It was found that even though portal frame deflections exceeded the current deflection limits significantly, it caused no problems to structural integrity or the confidence of occupants. Purlin and girt deflections were more noticeable than frame deflections. It is important that current deflection limits are reviewed to improve this situation.”

It is left to the individual designer to determine appropriate deflection limits for the structure. It has always been acknowledged that plastically designed structures have considerably larger deflections than elastically designed structures, in order to achieve the extra design strength afforded by plastic design.
5.6 COMPARISON WITH METHODS OF ANALYSIS

5.6.1 FIRST ORDER ELASTIC ANALYSIS

Simple elastic analysis, using the computer program PRFSA (CASE 1996), was performed. The aim of the first order analysis was to examine the elastic distribution of bending moment in the frame.

At a low level of load, approximately 25% of the ultimate load, the frames were still elastic. Based on the applied loads at the low load level, an elastic analysis was performed, and the distribution of bending moment obtained. At the same time, the curvature at the strain gauge locations was calculated as shown in Figure 5.29. For an elastic member $M = EI\kappa$, and since $\kappa_p = M_p/EI$, then $M/M_p = \kappa/\kappa_p$ while the frames were elastic. Figures 5.36, 5.37 and 5.38 plot $\kappa/\kappa_p$ (analysis) and $\kappa/\kappa_p$ (experimental) for each frame.

The curvature distribution shown in the figures below is discontinuous at the connections since the strain gauges were placed a finite distance away from the connection. Hence it was not possible to calculate the curvature at the connections. In an elastic frame with point loads only, the curvature and moment vary linearly between the connections and loading points.

![Figure 5.35: Distance Along Frame (in mm) (for use in Figures 5.36, 5.37 and 5.38)](image_url)
Figure 5.36: Elastic Curvature in Frame 1

Figure 5.37: Elastic Curvature in Frame 2
Figure 5.38: Elastic Curvature in Frame 3

The general shape of the experimental distribution of curvature matched the curvature distribution predicted by a first order elastic analysis. Most notably, the experimental curvature values, and hence the experimental bending moments, were slightly lower than the analysis values at the connections. One reason for lower moments is that the connections were not perfectly rigid. The bolted moment connection at the apex started to separate slightly, reducing its stiffness, and the moment it carried. Similarly, the sleeve connection may have slipped slightly, and not behaved prismatically as assumed by the analysis. The lower than expected stiffness results in higher than predicted deformations in the frame and lower moments at the same load.
5.6.2 SECOND ORDER ELASTIC ANALYSIS

Second order elastic analysis, using the computer program NIFA (Clarke and Zablotskii 1995), was performed. A second order analysis includes the effects of the deformed shape of a frame when calculating the design actions and deformations. It is an iterative and incremental analysis. The aim of the second order analysis was to provide an estimate of the elastic deflections and initial stiffness of the frame. There were large second order effects, particularly for Frame 3. The results of the second order analysis can be seen in Section 5.6.5.

5.6.3 SIMPLE PLASTIC ANALYSIS

Simple plastic analysis, using the computer program PRFSA (CASE 1996), was performed on a model of each frame. The analysis assumed no interaction between axial force and moment (ie the plastic moment was not reduced in the presence of axial force). Only elastic - plastic (no strain hardening) material properties were considered. Second order effects were not included. The absence of second order effects and moment - axial force interaction would tend to give a higher ultimate load. However, the lack of strain hardening in the material properties would tend to give a lower ultimate load than the experimental values. These two effects counteract each other.

Two analyses were made on each frame:
(1) Based on nominal dimensions and nominal yield stress (either 350 MPa or 450 MPa)
(2) Based on measured dimensions and measured yield stress.

The results of the PRFSA plastic analysis are summarised in Table 5.11. The hinge locations are shown in Figure 5.39. Table 5.11 lists the ultimate load, which is the load at plastic collapse. The pin-based portal frame was a redundant structure, and two plastic hinges were required for a plastic collapse mechanism to form. The ultimate load occurred at the formation of the second hinge. The ratio of the ultimate load to the load at the formation of the first hinge represents the increase in load capacity afforded by plastic analysis when compared to an elastic analysis, where the section is compact so that the first hinge is allowed to form.
Table 5.11: Summary of Plastic Analysis

<table>
<thead>
<tr>
<th>Frame</th>
<th>Load at ultimate (kN)</th>
<th>Ultimate load</th>
<th>Ultimate load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Vertical</td>
<td>Horizontal</td>
<td>First hinge load</td>
</tr>
<tr>
<td>Frame 1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>nominal</td>
<td>57.6</td>
<td>1.44</td>
<td>1.05</td>
</tr>
<tr>
<td>measured</td>
<td>68.4</td>
<td>1.71</td>
<td>1.05</td>
</tr>
<tr>
<td>experimental</td>
<td>68.2</td>
<td>1.75</td>
<td></td>
</tr>
<tr>
<td>Frame 2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>nominal</td>
<td>74.0</td>
<td>1.85</td>
<td>1.05</td>
</tr>
<tr>
<td>measured</td>
<td>72.8</td>
<td>1.82</td>
<td>1.05</td>
</tr>
<tr>
<td>experimental</td>
<td>71.5</td>
<td>1.87</td>
<td></td>
</tr>
<tr>
<td>Frame 3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>nominal</td>
<td>50.8</td>
<td>15.4</td>
<td>1.09</td>
</tr>
<tr>
<td>measured</td>
<td>50.0</td>
<td>15.1</td>
<td>1.09</td>
</tr>
<tr>
<td>experimental</td>
<td>45.7</td>
<td>13.8</td>
<td></td>
</tr>
</tbody>
</table>

Figure 5.39: Plastic Hinge Locations Predicted by Plastic Analysis

Note: LF = Load factor
5.6.4 PLASTIC ZONE ANALYSIS

Second order elastic and inelastic analyses, using the computer program NIFA (Clarke and Zablotskii 1995), were performed on models of the portal frames. NIFA uses the finite element method and the principal of virtual displacements to obtain equilibrium equations. An incremental and iterative method is used to solve the equations. Yielding (or plasticity) is accounted for by a plastic zone approach, rather than assuming infinitely small plastic hinges. The plastic zone approach can model the spread of yielding throughout a given cross section and along the length of a member. The reduction of moment capacity due to the presence of axial force is also included. Second order effects can be excluded or included. Member imperfections can be included if required.

Several different types of analysis were performed using NIFA:

- Second order elastic (refer to Section 5.6.2).
- First order plastic zone using simple elastic-plastic material properties.
- Second order plastic zone using simple elastic-plastic material properties.
- Second order plastic zone using different multi-linear material properties (based on the tensile coupon tests) for the flange, web and corners.
- Second order plastic zone using different multi-linear material properties (based on the tensile coupon tests) for the flange, web and corners, and including the geometric imperfections of the structure.

It should be noted that NIFA cannot model the local buckles in the frame. It is not suitable to estimate the deformations in the frame past the ultimate load (when the first local buckle occurred), due to the falling moment characteristic of the local buckle (Wood 1968). Hence all NIFA analyses were terminated soon after the ultimate load was reached.

Section 5.6.5 provides a detailed discussion on the results of the different types of analysis.
5.6.5 COMPARISON AND DISCUSSION

Figures 5.40 - 5.45 plot the load - deflection curves predicted by the various analyses. Table 5.12 summarises the ultimate loads and compares them to the experimental ultimate load. Table 5.13 summarises the deflections at the ultimate load.

Figure 5.40: Portal Frame 1: Vertical Deflections at Apex

Figure 5.41: Portal Frame 1: Horizontal Deflections at North Knee
Figure 5.42: Portal Frame 2: Vertical Deflections at Apex

Figure 5.43: Portal Frame 2: Horizontal Deflections at North Knee
Figure 5.44: Portal Frame 3: Vertical Deflections at Apex

Figure 5.45: Portal Frame 3: Horizontal Deflections at North Knee
<table>
<thead>
<tr>
<th>Program</th>
<th>Type^1</th>
<th>Props^2</th>
<th>Mat^3</th>
<th>Imp^4</th>
<th>Frame 1</th>
<th>Frame 2</th>
<th>Frame 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Vert^5</td>
<td>Ratio^6</td>
<td>Vert^5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>68.2</td>
<td>71.5</td>
<td>45.7</td>
</tr>
<tr>
<td>PRFSA</td>
<td>1^{st} plast</td>
<td>nom</td>
<td>e-p</td>
<td>n</td>
<td>57.64</td>
<td>0.85</td>
<td>74.03</td>
</tr>
<tr>
<td>PRFSA</td>
<td>1^{st} plast</td>
<td>meas</td>
<td>e-p</td>
<td>n</td>
<td>68.41</td>
<td>1.00</td>
<td>72.76</td>
</tr>
<tr>
<td>NIFA</td>
<td>1^{st} pz</td>
<td>nom</td>
<td>e-p</td>
<td>n</td>
<td>57.18</td>
<td>0.84</td>
<td>73.51</td>
</tr>
<tr>
<td>NIFA</td>
<td>1^{st} pz</td>
<td>meas</td>
<td>e-p</td>
<td>n</td>
<td>66.89</td>
<td>0.98</td>
<td>70.85</td>
</tr>
<tr>
<td>NIFA</td>
<td>2^{nd} pz</td>
<td>meas</td>
<td>e-p</td>
<td>n</td>
<td>61.49</td>
<td>0.90</td>
<td>64.79</td>
</tr>
<tr>
<td>NIFA</td>
<td>2^{nd} pz</td>
<td>meas</td>
<td>mul</td>
<td>n</td>
<td>67.16</td>
<td>0.98</td>
<td>73.14</td>
</tr>
<tr>
<td>NIFA</td>
<td>2^{nd} pz</td>
<td>meas</td>
<td>mul</td>
<td>y</td>
<td>68.84</td>
<td>1.01</td>
<td>77.05</td>
</tr>
</tbody>
</table>

**Notes:**
1. Type of analysis is either 1^{st} order simple plastic, 1^{st} order plastic zone, or 2^{nd} order plastic zone.
2. Dimensions and yield stress are based on either nominal or measured properties.
3. Material properties are either elastic-plastic with the properties of the webs used for the entire cross section, or multi-linear approximation to the measured properties with separate properties for the webs, flanges and corners.
4. The frame imperfections are either included or excluded.
5. Ultimate vertical load (kN).
6. Ratio of the analysis load to the experimental load.

Table 5.12: Summary of Analysis Results
| Program Type | Props | Mat | Imp | Frame 1 | | Frame 2 | | Frame 3 |
|--------------|-------|-----|-----|--------|--------|--------|--------|
|              |       |     |     | Vert¹  | Rat³   | Horz²  | Rat³   | Vert¹  | Rat³   | Horz²  | Rat³   |
| Experimental |       |     |     | 94.1   | 1.00   | 43.0   | 1.00   | 108.0  | 1.00   | 84.0   | 1.00   |
| PRFSA        | 1st   | plast| nom | e-p n  | 50.7   | 0.54   | 4.5    | 0.10   | 65.1   | 0.60   | 5.7    | 0.07   |
| PRFSA        | 1st   | plast| meas| e-p n  | 59.1   | 0.63   | 5.3    | 0.12   | 63.0   | 0.58   | 5.7    | 0.07   |
| NIFA¹        | 1st   | pz   | nom | e-p n  | 59.4   | 0.63   | 24.3   | 0.57   | 77.1   | 0.71   | 32.9   | 0.39   |
| NIFA¹        | 1st   | pz   | meas| e-p n  | 85.9   | 0.91   | 88.7   | 2.06   | 106.3  | 0.98   | 103.8  | 1.24   |
| NIFA²        | 2nd   | pz   | meas| e-p n  | 70.1   | 0.74   | 29.9   | 0.70   | 71.9   | 0.67   | 26.3   | 0.31   |
| NIFA²        | 2nd   | pz   | meas| mul n  | 106.2  | 1.13   | 117.5  | 2.73   | 202.7  | 1.88   | 345.7  | 4.12   |
| NIFA²        | 2nd   | pz   | meas| mul y  | 81.3   | 0.65   | 27.2   | 0.63   | 65.3   | 0.60   | 30.6   | 0.36   |
| NIFA²        | 2nd   | pz   | meas| mul n  | 80.7   | 0.86   | 60.0   | 1.40   | 86.8   | 0.80   | 64.9   | 0.77   |
| NIFA²        | 2nd   | pz   | meas| mul y  | 82.1   | 0.87   | 59.4   | 1.38   | 87.0   | 0.81   | 35.9   | 0.43   |

**Notes:**
1. Vertical deformation in millimetres at the apex at the ultimate load.
2. Horizontal deformation in millimetres at the north knee at the ultimate load.
3. Ratio of the deflection predicted by the analysis to the observed experimental deflection at the ultimate load.
4. The first order plastic zone analysis exhibited asymptotic deflection behaviour as the load increased as illustrated in Figures 5.40 - 5.45.

The two rows of values represent the range of deflections between 99 and 100 % of the ultimate load.

Table 5.13: Summary of Deflections at Ultimate Load
5.6.5.1 Elastic Response

The initial (elastic) response of the frame was similar for each of the analyses, with the exception of the advanced analysis including the structural imperfections. The observed vertical and horizontal deflections in Frame 2 and Frame 3 diverged from the predictions at a low load. The divergence was most likely caused by two reasons. Firstly, and most significantly, the take up of fit, or “settlement” in the connections, resulted in a less than anticipated stiffness, despite a small pre-load being applied to each structure before testing to reduce the effects of take up of fit. After the initial period the slope of the experimental curve was close to (but slightly less than) the analysis predictions. Secondly, initial imperfections in the frame caused different deflections from a geometrically perfect frame. For example, the initial negative horizontal deflection of Frame 2 was only predicted by the analysis which included the imperfections. Imperfections are discussed in more detail in Section 5.6.5.6. Residual stresses in the experimental frames may have caused premature yielding, and hence contributed to a lower than anticipated stiffness at low levels of load.

As expected, the second order elastic analysis showed softening as the load increased, and could provide a measure of the deflections at service load. However, once a hinge had formed, the deflections increased dramatically, and the second order elastic analysis was not appropriate to predict deflections.

5.6.5.2 Plastic Analysis

The simple plastic analysis using the measured yield stress gave a reasonable estimate of the ultimate load, despite all the deficiencies of such an analysis. The absence of second order effects and moment - axial force interaction, would tend to give a higher ultimate load than that observed, but the lack of strain hardening in the material properties would produce a lower load than the experimental values. The two aforementioned effects counteracted each other. For Frames 1 and 2, the simple plastic analysis predicted the frame strength to within 2%. However, in Frame 3, where the second order effects were greatest, the plastic analysis overestimated the strength by nearly 10%, as the significance of the second order effects was more considerable than strain hardening.
The plastic analysis also significantly under predicted the deflections at the formation of the final plastic hinge and ultimate load, compared to the observed experimental deflection at ultimate load. The simple plastic analysis is essentially a series of first order elastic analyses, adding an extra plastic hinge each iteration. Hence the plastic analysis did not give an accurate prediction of deflection at ultimate load. However, deflections at serviceability loads (often taken as 66 % of the ultimate load) are normally considered important by designers. As the frames were still (mainly) elastic at serviceability loads, the predictions of the plastic analysis were the same as those of the first order elastic analysis.

For Frame 1 there was a considerable difference between the analyses based on nominal material properties compared to measured properties. The measured results were notably higher, by approximately 15%. For Frame 2 and Frame 3, the two sets of results were very similar, mainly due to the measured yield stress being close to the nominal value. For Frame 1, the measured yield stress was considerable higher than the nominal yield stress ($f_{\text{yn}} = 350$ MPa and $f_{\gamma} = 411$ MPa, approximately +15 % difference), whereas for Frames 2 and 3 the measured yield stress was approximately equal to the nominal yield stress ($f_{\text{yn}} = 450$ MPa and $f_{\gamma} = 438$ MPa, about -2.5 % difference). Similar observations can be made in the comparison of the first order plastic zone analyses (NIFA), with either nominal or measured elastic plastic material properties. Therefore it is important to consider any possible difference between observed and nominal material properties when considering frame strength.
5.6.5.3 Interaction of Bending Moment and Axial Force (First order plastic zone analysis)

The comparison between the first order simple plastic analysis (PRFSA) and the first order plastic zone analysis (NIFA) highlights the effect of interaction of bending moment with axial force. The simple plastic analysis did not account for the reduction of plastic moment in the presence with axial force, while the plastic zone did consider interaction. In addition, the NIFA analysis models the spread of plasticity in the members, while the PRFSA plastic analysis merely considered plastic hinges at distinct points.

Since the levels of axial force were relatively low, there was only a small reduction in the moment capacity of the members. Hence an analysis including interaction between axial force and moment capacity gave only a slightly lower value than the simple plastic analysis. For Frames 1 and 2, the inclusion of axial force moment interaction reduced the predicted ultimate load by no more than 3%. For the Frame 3, the corresponding drop in ultimate load was between 4 and 6%. It is surprising that moment-axial force interaction had a more significant effect on Frame 3, since the axial loads were lower compared to Frames 1 and 2, and second order effects were not considered. It may be due to the different distribution of plasticity in Frame 3 compared to Frames 1 and 2 (refer to Figure 5.39). The deflections from the NIFA first order plastic zone analysis asymptotically approached a value just below the simple plastic predictions of PRFSA. The analysis underpredicted the deflections as the load approached the ultimate load, but due to the asymptotic nature of the response, the deflections subsequently increased markedly as the load increased marginally.

The first order plastic zone analysis did not consider second order effects, member imperfections, or strain hardening, yet predicted the ultimate strengths of the frames to within 3% of the experimental values.

While the NIFA first and second order plastic zone analyses did not predict actual hinges, there were regions (zones) of highly concentrated curvature, which were equivalent to hinges. The NIFA analysis predicted these “hinges” in the same locations as those predicted by the simple plastic analysis (Figure 5.39). These locations corresponded to the regions of high curvature and local buckles on the experimental portal frames.
5.6.5.4 Second Order Effects (Second order plastic zone analysis)

The significance of second order effects was identified by comparing the first and second order NIFA analyses. For Frames 1 and 2 the ultimate load dropped approximately 8% when second order effects were included. In Frame 3, the horizontal force, and hence the horizontal deflections and second order effects, were greater than in Frames 1 and 2. The predicted ultimate load for Frame 3 was reduced by about 15% when second order effects were considered.

When second order effects, but not non-linear material properties and strain hardening, were included, the analysis underestimated the strength of the frame by 9 - 12%. Hence exclusion of non-linear material properties and strain hardening can produce unnecessarily conservative estimates of the frame strength.

The deflections predicted by the analysis at the ultimate load were less than those in the actual tests.

5.6.5.5 Material Non-Linearity and Strain Hardening (Second order plastic zone analysis with multilinear stress - strain curves)

There was considerable strain hardening in the cold-formed RHS, and after yielding the stresses exceeded the 0.2% proof stress that was quoted as the yield stress (Appendix C). A multi-linear stress - strain curve approximating the true response was required to obtain useful predictions of the frame behaviour. Approximately six to ten different points on the true stress - strain curve were extracted, and the analysis assumed a linear variation between the points. Lau and Hancock (1990) showed that the stress - strain curves of cold-formed steel could not be accurately approximated by the traditional Ramberg-Osgood equation, and derived a modified Ramberg-Osgood formula. Currently NIFA can only use the traditional Ramberg-Osgood equation, and so the multilinear approximation was used.
As was identified in Appendix C, due to the different amounts of cold-work in different parts of the RHS, the material properties vary between the flange, web and corners of the section. On average the yield stress of the flat face (the flanges) opposite to the weld face is about 10% higher than the flat faces (webs) adjacent to the weld face. The corners of the RHS have a value of $f_y$ about 10% higher than the opposite face. Three different stress-strain curves were used to model the different parts of the RHS. However, the change in material properties around the section was a gradual transition from flange to corner to web (Hancock 1998). It was not possible to model the gradual transition of material properties in the NIFA analysis.

The second order plastic zone analysis that considered second order effects and material non-linearity, but not imperfections, provided very good estimates of the ultimate strength of each frame, within 2% of the experimental value, for all three frames. The second order plastic zone analysis without frame imperfections provided the best estimates of the ultimate loads of the three frames. While the beneficial effects of the unsympathetic imperfections were omitted (refer to Section 5.6.5.6 for a discussion of imperfections), the omission was counteracted by under predicting the sway deflections and hence the magnitude of the second order effects.

The deflections predicted by the analysis at the ultimate load were less than those in the actual tests.

5.6.5.6 Imperfections (Second order plastic zone analysis with member imperfections)

Inclusion of member imperfections is an important factor in an advanced analysis. The nature of the deflections and the ultimate loads can be affected by initial imperfections. Typically, if an imperfection is in the same direction as the deflections of a structure, it will tend to reduce the ultimate load. Such an imperfection is sometimes referred to as a sympathetic imperfection. Conversely, an unsympathetic imperfection tends to increase the ultimate load.
The second order plastic zone analyses were repeated, including the initial imperfections measured in Section 5.4.2. It was assumed that the member imperfections were sinusoidal, with maximum imperfection at the element midpoint, and zero imperfection at the member ends. The magnitudes of the imperfections are listed in Table 5.5. Table 5.5 shows that the individual member imperfections were quite small, but the out-of-plumb imperfections were significant.

Allowing for take up of fit, the initial horizontal stiffness of Frames 2 and 3 did not closely match the analysis predictions. When the imperfections were taken into account in Frame 2, the plastic zone analysis did predict the initial negative horizontal stiffness of the frame. However for Frame 3, the analysis including the imperfection still did not match the initial experimental stiffness, which suggested that while the magnitude of the imperfections was modelled, the precise imperfection shape assumed in the analysis did not match the true imperfection distribution. Figure 5.17 shows the measured imperfections, and it was assumed that the member imperfections were sinusoidal. The true imperfect structural shape may not have been sinusoidal, which resulted in the different initial stiffness.

When the imperfections were included in the analysis, the ultimate load increased, by 3 % for Frame 1, 6 % for Frame 2, and 2 % for Frame 3, compared to the analysis without imperfections. In each case, the analysis over predicted the ultimate load, by 1 %, 8 %, and 2 % respectively.

In Frame 1, there was a small unsympathetic out-of-plumb imperfection in the north column and hence there was a small increase in ultimate load. For Frame 2, both columns had significant unsympathetic out-of-plumb imperfections, and there was a 6 % increase in ultimate load. In Frame 3, an unsympathetic imperfection in the south column counteracted a slightly smaller sympathetic imperfection in the south column, resulting in a small 2 % increase in load.
The imperfections also affected the nature of the deflections. For Frame 2, the experimental horizontal deflections did not match the analysis excluding imperfections. The experimental deflections were initially negative, but all the analyses without imperfections predicted positive deflections. Only the imperfection analysis predicted the initial negative deflections. However, in Frame 3, the imperfection analysis closely matched the analysis without imperfections, and did not correctly predict the initial portion of the load deflection curve.

The deflections predicted by the analysis at the ultimate load were less than those in the actual tests. Hence the second order effects were less than those that occurred in the experimental frames. It is most likely that the underestimation of the deflections and the second order effects, caused the analysis to over predict the ultimate load.

5.6.5.7 Joint Flexibility (Deformations)

Table 5.13 shows that the analyses consistently underestimated the magnitude of the deflections, and consequently the second order effects. Hence the second order plastic zone analysis which incorporated material non-linearity and imperfections over predicted the frame strength. The most likely cause was the non-inclusion of flexible joint behaviour. Each analysis assumed that the joints had the same rigidity as the plain members, and any additional non-linearity in the joint behaviour (perhaps caused by slippage, bolts, yielding etc) was not accounted for in the analysis. Chapter 4 described tests on the connections used in the portal frame behaviour. The connections exhibited non-linear behaviour, with slight loss of rigidity compared to a plain member (though the observed behaviour would not be described as “semi-rigid”). It was also unknown whether the RHS and sleeve acted prismatically or non-prismatically. (Prismatic action is defined as the two objects acting as a unit with the stiffness of the combined unit, while non-prismatic action occurs when the two elements act independently and slip occurs between the two). The analysis assumed that there was prismatic action, whereas the true behaviour was probably a mixture of prismatic and non-prismatic action. Assuming prismatic action would give smaller deflections. Therefore, in the experimental frames, there was more deformation at each connection, particularly the sleeve joint, than was calculated in the analyses.
Generally, modern structural analysis software is capable of including flexible joint behaviour, but the lack of information of the moment-curvature behaviour of various connections prevents its widespread inclusion. The sleeve connections exhibited particularly complex behaviour, which included possible slippage of the sleeve, a combination of prismatic and non prismatic behaviour, and the formation of the plastic hinge at the end of the sleeve rather than at the junction of the two members, making it difficult to obtain a useful moment-curvature relationship that could be incorporated into an analysis. The experimental data collected on the behaviour of the sleeve connections would not necessarily be sufficient, as the strain gauges were not always located at the point of maximum curvature (refer to Chapter 4).

If an advanced analysis is to provide a highly accurate prediction of the frame behaviour, the flexible or semi-rigid behaviour of the connections ought to be considered. However, significant information on the flexible joint behaviour needs to be gathered before flexible behaviour can be included in the analysis.

5.6.5.8 Summary

Each type of analysis considered underestimated the deflections (due mainly to the non-inclusion of flexible joint behaviour). The 2nd order plastic zone analysis including the imperfections and strain hardening, overestimated the ultimate load, and provided a less accurate prediction of the ultimate load than the same analysis without the imperfections. Excluding the imperfections gave a more accurate result because the effect of underestimating the deflections counteracted the effect of omitting the beneficial effects of the mainly unsympathetic imperfections.
5.7 CONCLUSIONS

5.7.1 SUMMARY

This chapter has described tests on three portal frames constructed from cold-formed RHS in Grade C350 or Grade C450 steel. The details of the frames, the loading methods and the results were discussed.

The results indicate that a plastic collapse mechanism was formed in each frame. The connections of the frames, particularly the welded internal sleeve knee joints, performed adequately.

The various forms of structural analyses predicted the locations of the plastic hinges which coincided with the places of high curvature and local buckles in the experimental frames. All analyses underestimated the magnitude of the deflections of the frame. Factors such loss of rigidity in joints, and slippage in various places in the frame, contributed to the underestimation of deflections. The analysis showed that the second order effects were considerable, particularly for Frame 3, where they accounted for a drop in ultimate load of approximately 15 %. The measured imperfections of the frame were unsympathetic and increased the load carrying capacity of the frames compared to a perfect frame. An advanced plastic zone structural analysis, which included second effects, gradual yielding, multilinear stress - strain curves varying around the cross section, and the structural imperfections slightly overestimated the strength of the frame. The main cause for the over prediction of strength was the underestimation of the deflections and the hence the magnitude of the second order effects. The second order plastic zone analysis without frame imperfections provided the best estimates of the ultimate loads of the three frames, within 2 % of the experimental values.

The distribution of curvature throughout the portal frame was analysed. The points of high curvature required a rotation capacity of $R < 3$ to achieve a load of 99 % of the ultimate load. All but one point required a rotation capacity of $R < 4$ to achieve the ultimate load, indicating that for the portal frames tested a rotation capacity of $R = 4$ is adequate.
Although the cold-formed RHS do not satisfy the material ductility requirements of AS 4100 or Eurocode 3, there was no failure associated with material ductility in the three frames tested, demonstrating that the restriction on cold-formed RHS from plastic design based on insufficient material ductility is unwarranted provided that the connections are sufficiently ductile for any required plastic rotations.

5.7.2 FURTHER STUDY

The tests have shown that cold-formed RHS portal frames could deform plastically, and form a plastic collapse mechanism, and attain the simple plastic collapse load. However, for the pin-based portal frames considered, only two plastic hinges were required to form the collapse mechanism. It would be desirable to consider a more complex system, such as a fixed based frame, where four hinges are usually required to form a mechanism. It would also be beneficial to consider the plastic behaviour of a frame under wind uplift loading, and the behaviour of a complete system, incorporating purlins, sheeting, and bracing.

The deformations, particularly the vertical deflections of the apex, exceeded the recommended deflection limits for beams in AS 4100. However, it was shown that the limits in AS 4100 were suited more for beams supporting floor systems, rather than portal frame rafters. It is recommended that serviceability limits for portal frame structures be reviewed.
Chapter 6

FINITE ELEMENT ANALYSIS OF COLD-FORMED RHS BEAMS

6.0 CHAPTER SYNOPSIS

This chapter describes the finite element analysis of RHS beams, to simulate the bending tests of Chapter 3. The finite element program ABAQUS was used for the analysis. The main aim of the analysis was to determine trends and to understand the occurrence of inelastic instability in cold-formed RHS beams. The analysis also supplemented the experimental data gathered in Chapter 3.

The maximum loads predicted were slightly lower than those observed experimentally, since the numerical model assumed only three distinct material properties in the RHS. In reality, the variation of material properties around the RHS cross-section is gradual.

Introducing geometric imperfections into the model was essential to obtain results that were close to the experimental results. A perfect specimen without imperfections achieved rotation capacities much higher than those observed experimentally. Introducing a bow-out imperfection, constant along the length of the beam, as was measured (approximately) experimentally, did not affect the numerical results significantly. To simulate the effect of the imperfections induced by welding the loading plates to the beams in the experiments, the amplitude of the bow-out imperfection was varied sinusoidally along the length of the beam. The size of the imperfections had an unexpectedly large influence on the rotation capacity of the specimens. Larger imperfections were required on the more slender sections to simulate the experimental results. The sensitivity to imperfection size increased as the aspect ratio of the RHS decreased.

The finite element analysis determined similar trends as observed experimentally, namely that the rotation capacity was a function of both the flange and web slenderness, and that for a given aspect ratio, the relationship between web slenderness and rotation capacity was non-linear, and the slope of the line describing the relationship increased as the web slenderness decreased.
6.1 INTRODUCTION

Chapter 3 described tests on cold-formed RHS beams to examine the Class 1 flange and web slenderness limits. Forty four (44) bending tests were performed on 16 different specimens. Some specimens of the same type were tested more than once to examine the effect of loading method. The sections represented a broad range of web and flange slenderness values, but it would have been desirable to test a much larger selection of specimens. However, a more extensive test program would have been expensive and time consuming.

Numerical or finite element analysis provides a relatively inexpensive, and time efficient alternative to physical experiments. It is vital to have a sound set of experimental data upon which to calibrate a finite element model. It is then possible to investigate a wide range of parameters within the model.

In order to model the plastic bending tests, the finite element program should include the effects of material and geometric non-linearity, residual stresses, imperfections, and local buckling. The program ABAQUS (Version 5.7-1) (Hibbit, Karlsson and Sorensen 1997), installed on Digital Alpha WorkStations in the Department of Civil Engineering, The University of Sydney, performed the numerical analysis.

This chapter describes the essential stages in the development of the model to simulate the bending tests on RHS. A wider parametric study was also performed on other variables in the model, not essential for the simulation of the beam tests. The parametric studies are included in Appendix J.
6.2 DEVELOPMENT OF THE FINITE ELEMENT MODEL

6.2.1 PHYSICAL MODEL

The physical situation being modelled must be considered first. The plastic bending tests outlined in Chapter 3 were modelled. Several general factors were considered:

- the RHS itself,
- the method of loading,
- nature of restraints.

Figure 6.1 is a simplified diagrammatic representation of the experimental layout (refer to Figure 3.5). Unless otherwise specified, the lengths of the specimen analysed were those shown in Figure 6.1. The cross-sectional dimensions \((d, b, t\) and \(r_e\)) of the RHS were changeable. The axis system for the beam is shown in Figure 6.2.

![Figure 6.1: Simplified Representation the Plastic Bending Tests](image)

![Figure 6.2: Axis System](image)
6.2.2 SYMMETRY AND BOUNDARY CONDITIONS

The size of a finite element model can be reduced significantly by using symmetry in the body being analysed.

There was symmetry along the length of the beam (the longitudinal axis, or the 1-axis in the finite element model). The loading was symmetric about the middle of the beam. The support conditions were almost symmetric about the middle cross section of the beam, but only one support provides restraint against 1-axis translation. It was possible to consider only half the length of the beam, and apply the boundary conditions, as shown in Figure 6.3, to all nodes at the middle section of the beam. 1-axis translation was prevented at the middle cross section.

Symmetry in a plane perpendicular to the 1-axis only applied when the deflected shape was symmetric about that plane. The deflections were symmetric before the beam experienced local buckling. After buckling, the deflected shape was no longer symmetric about a plane perpendicular to the 1-axis, as shown in Figure 6.4 and Figure 3.9, since only one local buckle formed in the experimental situation. Using symmetry after the formation of the local buckle gave an incorrect deflected shape, as the analysis assumed that two buckles, one on each side of the symmetry plane, had formed. However, the major aim of the finite element analysis was to predict...
the rotation capacity, not the post-buckling response. Hence, the inability to model the post buckling deflections very precisely was not considered a deficiency in the model.

Figure 6.4: Loss of Longitudinal Symmetry after Local Buckling

The two principal axes of the RHS are axes of symmetry as shown in Figure 6.5. The major principal axis (sometimes referred to as the x-axis) was the 2-axis in the finite element model, but could not be used as an axis of symmetry for the finite element analysis. Since the beam was bending about the 2-axis, the top half of the cross-section was in compression, and the bottom half was in tension, and symmetry did not occur. The minor principal axis (y-axis) is the 3-axis
in the ABAQUS model. The stress distribution was the same on either side of the 3-axis, and only half the cross-section needed to be analysed provided the boundary conditions indicated in Figure 6.5 were prescribed. One boundary condition along the 3-axis was that translation in the 2-direction \( (u_2) \) was zero, which prevented overall out-of-plane (lateral) deflections of the entire beam. In plastic design, it is usually required that there is adequate bracing to stop lateral buckling. Therefore restricting lateral deflections of the beam \( (u_2) \) was appropriate within the finite element model.

Figure 6.5: Symmetry Within a Section

Figure 6.6 shows the typical cross-section shape after local buckling. The buckled shape remained symmetric about a plane perpendicular to the 2-axis.

Figure 6.6: Symmetrical Locally Buckled Shape
6.2.3 CHOICE OF ELEMENT TYPE

ABAQUS has several element types suitable for numerical analysis: solid two and three dimensional elements, membrane and truss elements, beam elements, and shell elements. The major aim of the analysis was to predict the formation of inelastic local instabilities in a cross section and the corresponding rotation capacity. Beam, membrane and truss elements are not appropriate for the buckling problem. Solid three dimensional elements (“brick” elements) may be suitable, but the solid elements have only translation degrees of freedom at each node, and require a fine mesh to model regions of high curvature. A finer mesh does not necessarily imply more total degrees of freedom, as one must consider the number of elements and the degrees of freedom of each element.

The most appropriate element type is the shell element. ABAQUS has “thick” and “thin” shells. “Thick” shells should be used in applications where the shell thickness is more than 1/15th of a characteristic length on the surface of the shell. When modelling the local buckling of RHS beams, the characteristic length is the flange or web width. In the experimental program, values of \( \frac{d}{t} \) and \( \frac{b}{t} \) varied from about 10 to 75. Only a few of the stockiest sections could be classed as “thick”. Therefore, “thin” shells are acceptable in the analysis, but there may be some loss of accuracy for the stockiest sections, as the ratio of 1/15 is exceeded in some stocky cases.

The S4R5 element was used in the finite element analysis. The S4R5 element is defined as “4-node doubly curved general purpose shell, reduced integration with hourglass control, using five degrees of freedom per node” (Hibbit, Karlson, and Sorensen 1997). One rotational degree of freedom per node is normally not considered, but ABAQUS will automatically include the third rotation if it is required.

The loading plates attached to the RHS beam (refer to the “parallel plate” method of loading in Section 3.3) were modelled as 3-dimensional brick elements, type C3D8 (8 node linear brick). The weld between the RHS and the loading plate was element type C3D6 (6 node linear triangular prism). The RHS was joined to the loading plates only by the weld elements.

Figure 6.7 shows a typical finite element mesh. There were three zones along the length of the beam, each with different mesh densities. The most important zone was that between the loading
plate and the symmetric end of the half-length model. In this zone, the moment was constant and at its maximum, and the shear force was zero. Local buckling was observed in this zone in the experiments. The mesh density was highest in this middle zone, as it was important to be able to model the formation of the local buckle. The length of the beam between the support plate and the loading plate, and from the support plate to the end were of lesser importance in the model, and the mesh density was reduced in those zones.

Figure 6.7: Finite Element Model

Figure 6.8 shows the mesh distribution around the cross-section of a typical RHS. The S4R5 elements are flat elements, and it was necessary to define the rounded corner of the RHS as a series of flat elements. Unless otherwise mentioned, the size of the external corner radius \( r_e \) was assigned according to the Australian Standard AS 1163. For sections with \( t \leq 3 \, \text{mm}, r_e = 2t \); otherwise for \( t > 3 \, \text{mm}, r_e = 2.5t \). The shell elements had zero thickness, but a “thickness” was assigned as a shell element property within ABAQUS. The shell elements were modelling an RHS which had finite thickness. The shell model followed the mid-thickness line of the “real” RHS as illustrated in Figure 6.9.
6.2.4 LOADING

ABAQUS allows loads to be applied as enforced displacements at nodes, much like in a displacement controlled experiment. A translation in the 3-direction was prescribed as a boundary condition. The enforced displacement was applied of the top of the loading plate as illustrated in Figure 6.10. Rotations were permitted at the point of application.
The method of loading simulated the experimental conditions as much as possible. The “parallel plate” method of loading, described in Section 3.3, was used. The load was applied to the top of the loading plate. The loading plate was connected by solid triangular elements, simulating fillet welds, to the RHS beams, as detailed in Figure 6.11.
6.2.5 PRE and POST PROCESSING

ABAQUS requires an input file which defines the nodes, elements, material properties, boundary
conditions and loadings. A preprocessor was developed in FORTRAN 77 to prepare the
ABAQUS input file. The user input parameters such as RHS dimensions, mesh size,
imperfections, and material properties, and an ABAQUS input file was generated. It was possible
to create many files quickly, in order to investigate the effects of various parameters efficiently
and easily. The ABAQUS INTERACTIVE option was employed, so that a large number of files
could be analysed consecutively without user intervention.

The finite element analysis generated vast amounts of data. A typical model contained 2150
elements, 2480 nodes and 13215 degrees of freedom. Each analysis usually consisted of 30 to
50 increments. It was possible for the output to contain full details of deformations, stresses and
strains in each direction for each node during every increment. Only a fraction of the available
output was required to obtain the moment - curvature relation for the beam being analysed.

Equilibrium in the vertical (3-axis) direction dictated that the force applied at the loading point
equalled the vertical reaction at the support for the half-beam being analysed. The bending
moment in the central region of the beam (between the loading plate and the “symmetric” loading
plate at the other end of the beam) was obtained from the support reaction. The curvature in the
central region was calculated from the deflection at the middle cross-section of the beam and the
deflection at the loading plate, similar to the experimental method outlined in Section 3.4. The
method for determining the curvature from the deflections is shown in Figure 6.12.
Figure 6.12: Determination of Curvature from Deflection

As was highlighted in Section 6.6.2, once a local buckle formed, the deflected shape was no longer symmetric about the mid-length of the beam, and the curvature became concentrated at the local buckle. The above method calculated an *average* curvature over the middle section of the beam after local buckling had occurred. The curvature calculated did not represent the value of localised curvature at the buckled section.

The ABAQUS output was restricted to the vertical (3-axis) deflection at two points, and the vertical reaction at the support. A small post processor was written to extract these values from the ABAQUS output file (*.dat), into a form which could be easily inserted into a spreadsheet. A macro within Microsoft Excel processed the extracted data into a moment - curvature plot (such as those seen in this chapter) and calculated the rotation capacity.
6.2.6 MATERIAL PROPERTIES

Different material properties could be included in the numerical analysis. The material properties obtained from a standard coupon test were input to the ABAQUS model as a set of points on the stress - strain curve. ABAQUS uses true stress and true strain, and hence the values of engineering stress and engineering strain from a standard coupon test were modified before being inserted into the model using the following equations:

\[
\sigma_{\text{eff}} = \sigma_{\text{true}} \left( 1 + e_{\text{true}} \right) \tag{6.1}
\]

\[
e_{\text{eff}} = \ln \left( 1 + e_{\text{true}} \right) \frac{\sigma_{\text{true}}}{E} \tag{6.2}
\]

The coupon tests of the RHS (Appendix C) showed that there was no distinct yield point in the cold-formed steel. The stress - strain curves were rounded and the value of yield stress reported was the 0.2% offset stress. There was variability of yield stress around the section. On average, the yield stress of the corner was approximately 10% higher than the yield stress at the centre of the flat face opposite the seam weld, which in turn was approximately 10% higher than the yield stress at the centre of the faces adjacent to the seam weld. Figure 6.13 illustrates the typical variation of mechanical properties within an RHS.

Figure 6.13: Typical Stress - Strain Curve for Cold-Formed RHS
In the numerical model, the RHS section was broken into three regions, representing the flange, web and corner of the section as shown in Figure 6.14. Different material properties were assigned to each part separately.

Figure 6.14: Three Regions of RHS with Different Material Properties

Figure 6.15 shows the different moment - curvature responses of a 150 × 50 × 3.0 C450 RHS for different material properties. The experimental curve exceeded the plastic moment due to strain hardening, and also since the plastic moment was determined as the product of the plastic section modulus and the yield stress of the adjacent face. The ABAQUS simulation, which incorporated simple elastic - plastic material properties, is shown. The response for the ABAQUS simulation, which incorporated simple elastic - plastic material properties, approached the plastic moment asymptotically. Theoretically a beam with elastic - plastic properties can never reach the plastic moment. A section with elastic - plastic - strain hardening properties is shown. The response was identical to the response of the elastic - plastic specimen until a curvature $\kappa = 3.5\kappa_p$, at which point strain hardening commenced, and the moment increased beyond the plastic moment. A specimen with the measured material properties is also included. The initial response was similar to the observed experimental behaviour, but after yielding the moment in the ABAQUS model was slightly lower than in the experiment by approximately 3 %. The numerical model assumed the same material properties across the whole flange, web or corner. There was discontinuity of properties at the junction of the regions. In reality, the variation of material properties around the
RHS was gradual, with a smoother increase of yield stress from the centre of a flat face, to a maximum in the corner. The numerical model assigned the measured properties from the coupon cut from the centre of the face (which were the lowest across the face) to the entire face. Hence, the numerical model underestimated the moment within the section. The slight error in predicted moment was not considered important, as the main aim of the analysis was to predict the rotation capacity. Provided that the shape of the response was modelled correctly, the under prediction of strength was not of concern.

The ABAQUS plots included in Figure 6.15 show that buckling occurred at much higher curvatures in the model, compared to the experiment. The numerical models in Figure 6.15 were for RHS with “perfect” geometry, and hence did not predict the local buckling correctly. The aim was to consider the effects of material properties on the shape and value of the predicted moment-curvature responses. The conclusion is that the measured stress-strain curves from the coupon tests needed to be included in the model, to simulate the magnitude of the moment response.

Figure 6.15: Moment - Curvature Response of RHS Beams with Different Material Properties
6.2.7 RESIDUAL STRESSES

During the formation process, residual stresses and strains are induced within the cross-section. While the net effect of residual stresses and strains must be zero for equilibrium, the presence of residual stresses can result in premature yielding and affect elastic local buckling of plate elements. Residual stresses were not explicitly included in the numerical model. Typically RHS have three types of residual stresses:

- Membrane residual stresses, which are uniform through the thickness of a plate element,
- Bending residual stresses, which vary linearly through the thickness, and
- Layering residual stresses, which vary irregularly through the thickness.

Membrane and bending residual stresses can be measured using the sectioning technique (Key 1988). The residual strains are released when the section is cut into strips (such as when a tensile coupon is cut from an RHS). The coupons exhibit a longitudinal curvature after cutting, indicating the released residual strain. When coupons are tested the sections are forced back into the uncurved shape as the grips are tightened and the specimen tensioned initially, reintroducing the bending residual stresses. The use of coupon data, therefore, implies that the bending residual stresses are accounted for within the stress-strain curves. The membrane residual stress is not reintroduced during coupon testing, but the magnitude of membrane residual stresses is much smaller than those of the bending residual stresses (Key 1988).

Layering residual stresses are difficult and expensive to measure. A method such as the spark erosion layering technique (Key 1988) is required to determine the layering residual stresses. However layering residual stresses are included in the results of a typical tensile coupon test.

Key found that in cold-formed SHS, the magnitude of the maximum membrane residual stresses was approximately 80 MPa, the maximum bending residual stresses was about 300 MPa, and the maximum layering residual stresses was approximately 100 MPa. Key investigated the effect of the various types of residual stresses on a finite strip analysis of SHS stub columns. The membrane residual stresses had an insignificant effect, the layering residual stress had a small impact, but the bending residual stresses had the major impact on stub column behaviour. The model used in the present analysis incorporated the bending and layering residual stresses, and that was deemed to be sufficient, based on Key’s findings.
6.2.8 MESH REFINEMENT

It is desirable to minimise the number of elements within a model, provided the results of the analysis are not unduly affected by removing elements. One also must consider the shape of the body being modelled and what mesh size is needed to represent the body adequately.

The S4R5 elements are quadrilateral shaped elements and, for simplicity of modelling, they were all defined as rectangles within the current analysis. If the aspect ratio of the rectangles becomes too high, the shell elements become less accurate. For example, Sully (1997) mentions that Schafer (1995) found errors in maximum load of up to 4% when he used shell elements with a high aspect ratio of 5.

The model had to simulate the formation of local buckles, and allow for wave-like local imperfections (detailed in Section 6.2.9) along the length of the beam. Experimental observations indicated that local buckles had half-wavelengths as small as 40 - 50 mm. Local imperfections with wavelengths in the range of 40 - 100 mm had to be included in the model. Modelling a half sine wave required at least four straight lines, any less made it difficult to simulate the peaks and troughs accurately. Hence the longitudinal length of an element had to be no greater than approximately 10 mm. The critical central zone between the edge of the loading plate end the symmetric and of the half-length beam was 325 mm (allowing for the 150 mm length of the loading plate), so at least 30 elements ($n_{el}$), were required longitudinally (the 1-axis direction).

To model the corner of the RHS with straight elements, at least two elements were required. The number of elements in the flange and web of the model could vary somewhat, but there was a need to ensure that the aspect ratio of the elements was not too high. It was also desirable to use the same mesh distribution for RHS with different aspect ratios.

Figures 6.16 and 6.17 show the different moment - curvature responses of two RHS with varying mesh distributions. The code used in the legend accounts for $d$, $b$, $10t$, $n_{el}$, $n_{elw}$, $n_{elf}$, and $n_{ele}$, hence 150_50_40_s_20_w_10_f_5_c_2 would refer to a 150 × 50 × 4.0 RHS, with 20 elements along the length of the beam from the loading plate to the symmetric end, 10 elements in the web, 5 elements in the half-flange, and 2 elements in the corner.
Figure 6.16: Effect of Mesh Size on 150 × 50 × 4.0 RHS

Figure 6.17: Effect of Mesh Size on 75 × 25 × 1.6 RHS

234
Figures 6.16 and 6.17 show that changing the mesh sizes did not have a major effect on the moment-curvature behaviour. The value of the moment reached was barely changed, but the rotation capacity was affected slightly. Most curves fell within a reasonably tight band. For the 75 x 25 x 1.6 specimen, the mesh which had only 4 elements in the web gave a response markedly different from the others. The mesh distribution adopted encompassed 40 elements along the length from the loading plate to the symmetric end, 10 elements in the web, 5 in the flange, and 2 elements in the corner of the RHS.

6.2.9 GEOMETRIC IMPERFECTIONS

The initial numerical analyses were performed on geometrically perfect specimens. It is known that imperfections must be included in a finite element model to simulate the true shape of the specimen and introduce some inherent instability into the model.

6.2.9.1 Bow-out Imperfection

The first imperfection considered was the constant bow-out imperfection which was observed and measured in the tests on RHS beams (Section 3.2.5 and Appendix F). The imperfection was approximately uniform along the length of the specimen, and typically there was a bow-out on the web, and a bow-in on the flange. However, the nature of the imperfection immediately adjacent to the loading plate was unknown, as it was not possible to measure the imperfections close to the loading plate. The process of joining a flat plate to a web with a slight concave bow-out imperfection is certain to induce local imperfections close to the plate.

Figure 6.18 shows a typical cross section with the bow-out imperfection included. A constant web imperfection of $\delta_w$, and a flange imperfection of $\delta_f$ were incorporated into the model. The imperfect shape of the flange and web was assumed to be sinusoidal across the flange or web face, with maximum imperfection of either $\delta_w$ or $\delta_f$. The corner of the RHS was not affected by the imperfection. The bow-out imperfection was constant along the length of the beam.
Figure 6.18: Specimen with bow-out Flange and Web Imperfections
(Imperfection magnified for illustration purposes)

Figure 6.19 shows the moment - curvature relationships obtained for a series of analyses on 150 × 50 × 3.0 C450 RHS with a variety of imperfections. A comparison with the experimental result is illustrated. The code used in the legend of Figure 6.19 accounts for $d$, $b$, $t$, $d_i\delta_w$, and $b/d_i$, hence 150_50_3_w+1000_f-1000 refers to a 150 × 50 × 3.0 C450 RHS, with web imperfection of 150/+1000 = +0.15 mm (bow-out positive), and flange imperfection of 50/-1000 = -0.05 mm (bow-in negative). The imperfection observed in the experimental specimen was $\delta_w = +0.3 \text{ mm}$, and $\delta_i = -0.1 \text{ mm}$. The material properties used were those of the tensile coupons cut from specimen BS03 (see Appendix C). The results are summarised in Table 6.1.
Figure 6.19: ABAQUS Simulations for Specimens with Different Bow-Out Imperfections
<table>
<thead>
<tr>
<th>Section</th>
<th>Web imperfection</th>
<th>Flange imperfection</th>
<th>$R$</th>
</tr>
</thead>
<tbody>
<tr>
<td>150 × 50 × 3.0</td>
<td>0</td>
<td>0</td>
<td>4.13</td>
</tr>
<tr>
<td>150 × 50 × 3.0</td>
<td>$+d/1500$</td>
<td>$-b/1500$</td>
<td>4.13</td>
</tr>
<tr>
<td>150 × 50 × 3.0</td>
<td>$+d/1000$</td>
<td>$-b/1000$</td>
<td>4.13</td>
</tr>
<tr>
<td>150 × 50 × 3.0</td>
<td>$+d/500$</td>
<td>$-b/500$</td>
<td>4.14</td>
</tr>
<tr>
<td>150 × 50 × 3.0</td>
<td>$+d/250$</td>
<td>$-b/250$</td>
<td>4.23</td>
</tr>
<tr>
<td>150 × 50 × 3.0</td>
<td>$+d/150$</td>
<td>$-b/150$</td>
<td>4.27</td>
</tr>
<tr>
<td>150 × 50 × 3.0</td>
<td>$+d/75$</td>
<td>$-b/75$</td>
<td>4.38</td>
</tr>
<tr>
<td>150 × 50 × 3.0</td>
<td>$+d/500$</td>
<td>$-b/500$</td>
<td>4.14</td>
</tr>
<tr>
<td>150 × 50 × 3.0</td>
<td>$-d/500$</td>
<td>$+b/500$</td>
<td>4.17</td>
</tr>
<tr>
<td>150 × 50 × 3.0</td>
<td>$+d/500$</td>
<td>$+b/500$</td>
<td>4.05</td>
</tr>
<tr>
<td>150 × 50 × 3.0</td>
<td>$-d/500$</td>
<td>$-b/500$</td>
<td>4.27</td>
</tr>
</tbody>
</table>

Experimental | BS03A | 2.7 |
Experimental | BS03B | 2.3 |
Experimental | BS03C | 2.9 |

Table 6.1: Summary of Results for Bow-out Imperfection

The measured imperfections for a 150 × 50 × 3.0 were approximately 1/500 of the web and flange dimensions. Different magnitudes of imperfection were analysed, ranging from 1/2000 to 1/75. In some cases, the signs of the imperfections were also changed from bow-outs to bow-ins. Compared to a specimen with no imperfection, the magnitude of the bow-out imperfection had a minor effect on the rotation capacity. The rotation capacity increased slightly as the imperfection increased. Even when the bow-out imperfections were included, the numerical results exceeded the observed rotation capacity by a significant amount. The conclusion is that the bow-out imperfection was not the most suitable type of imperfection to include in the model.
6.2.9.2 Continuous Sinusoidal Imperfection

It is more common to include imperfections that follow the buckled shape of a “perfect” specimen, such as by linear superposition of various eigenmodes. The approach taken was to include the bow-out imperfection, but vary the magnitude of the imperfection sinusoidally along the length of the specimen, which more closely modelled the shape of local buckles. The half wavelength of the imperfection is defined as $L_w$. A typical specimen with the varying bow-out along the length is shown in Figure 6.20.

![Figure 6.20: Specimen with Wave-like Flange and Web Imperfections](Imperfection magnified for illustration purposes)

A series of analyses was carried out to determine the appropriate half-wavelength of the imperfection to be included in the analysis. Figures 6.21 to 6.24 show a selection of results for three different size RHS. The code in the legend describes $d$, $b$, $t$, and $L_w$, hence 150_50_3_lw_090 refers to a 150 × 50 × 3.0 C450 RHS, with a half wavelength in the imperfection of 90 mm. The stress-strain curves used in the numerical analysis were those of the tensile coupons cut from relevant specimen (refer to Appendix C). In each case the magnitude of the imperfection was $+d/500$ (web) and $-b/500$ (flange).
Figure 6.21: Effect of Imperfection Half-Wavelength on 150 × 50 × 5.0 C450 RHS

Figure 6.22: Effect of Imperfection Half-Wavelength on 150 × 50 × 3.0 C450 RHS
Some analyses terminated immediately after buckling due to convergence problems.

Figure 6.23: Effect of Imperfection Half-Wavelength on 150 × 50 × 2.0 C450 RHS

Figure 6.24: Effect of Imperfection Half-Wavelength on 100 × 50 × 5.0 C450 RHS
The half wavelength of the local imperfection had a major impact on the rotation capacity of the sections. By choosing an appropriate wavelength the behaviour predicted in the numerical analysis approached the behaviour observed experimentally. Several important observations were made from the analysis of different imperfection wavelengths:

The imperfection wavelength had little effect on a more slender section where the rotation capacity was close to $R = 1$ (150 × 50 × 2.0 RHS).

For very stocky sections (150 × 50 × 5.0 RHS), the imperfection wavelength did alter the rotation slightly, but as such stocky sections had large rotation capacities, the small change was not particularly significant.

Imperfection wavelength had the greatest effect on sections with mid-range rotation capacities ($R = 2$ to $R = 5$) (eg 150 × 50 × 3.0 RHS, 100 × 50 × 2.0 RHS).

A half wavelength of approximately $d/2$ ($d$ is the depth of the RHS web) tended to yield the lowest rotation capacity and most closely matched the experimental behaviour (eg 150 × 50 × 3.0 RHS - $L_w = 70$ or 80 mm; 100 × 50 × 2.0 RHS - $L_w = 40$ or 50 mm). A half wavelength of $d/2$ was approximately equal to the half wavelength of the local buckle observed experimentally and in the ABAQUS simulations.

The post buckling response varied between specimens. In Figure 6.24, Specimens 100_50_2_lw_050 and 100_50_2_lw_040, which differed only by 10 mm in the imperfection half wavelength, displayed different post buckling behaviour. Both specimens buckled at approximately the same curvature, but the buckled shape was different. Specimen 100_50_2_lw_040 had one local buckle, formed adjacent to the loading plate as shown in Figure 6.25, which was the same location as the local buckle observed in tests. The post buckling drop in load was sharp in both the observed behaviour and the simulated behaviour. Specimen 100_50_2_lw_050 experienced two local buckles along the length of the beam. Since the curvature was not concentrated at one point, the moment dropped off more slowly as the average curvature in the beam increased. The double buckled shape is shown in Figure 6.26, and does not represent the observed buckled shape in any of the test samples.
For the sections with only one local buckle, the half wavelength of the buckle predicted by ABAQUS was independent of the half wavelength of the imperfection. Such a buckle is shown in Figure 6.25. The buckle half wavelength was slightly longer in the web of the RHS than in the flange. A half wavelength of $d/2$ was approximately equal to the half wavelength of the local buckle observed experimentally and in the ABAQUS simulations.
6.2.9.3 Single Sinusoidal Imperfection

The previous section highlighted the fact that in some cases the numerical analysis did not predict the buckled shape correctly. In the experiments, the local buckle always formed adjacent to the loading plates. It was most likely that imperfections caused by welding the loading plates to the RHS caused the buckle to form adjacent to the plates. The RHS were clamped during the welding process, and differential cooling after welding may have induced imperfections next to the loading plates. It was not practical to measure imperfections next to the loading plates, and the imperfection profiles in Appendix F did not demonstrate significant imperfections near the plates (it was only practical to measure the RHS profile to within about 25 mm of the loading plates). However visual inspection indicated some imperfection near the loading plates.

The solution to the problem was to introduce the sinusoidal imperfection in the RHS only next to the loading plates, and not along the entire length of the beam. Three half wavelengths of imperfection were included in the model, but only a single sinusoidal imperfection (two half wavelengths) was in the central region of the beam. The extra half wavelength of imperfection was within the beam length that was attached to the loading plate, to ensure that there was a bow-out immediately next to the loading plate and then to ensure continuity of the imperfect shape next to the loading plate. Figure 6.27 shows a typical RHS section with the modified imperfection pattern.

![Figure 6.27: RHS with Imperfection Close to the Loading Plate Only](image)

(Imperfection magnified for illustration purposes)
Figure 6.28 compares the moment - curvature response of specimens with either a continuous imperfection or the single imperfection. The section simulated was a 150 × 50 × 3 C450 RHS with material properties for specimen S03 (refer to Appendix C). The curvature at which the local buckle formed was barely affected by the nature of the imperfection, but the buckled shape, and the post-buckling response was altered. The specimens with the continuous imperfection experienced two local buckles (Figure 6.26). The specimens with the imperfection limited to near the loading plate, only one buckle formed, close to the loading plate (Figure 6.25), matching the location of the buckle observed experimentally, and the post buckling moment - curvature behaviour was more similar to the observed response.

Figure 6.28: Effect of Type of Imperfection
6.9.2.4 Comparing Rotation Capacities

Before comparing the complete sets of results, an appropriate measure of comparison was required. Chapter 2 defined the rotation capacity as $R = \kappa_1/\kappa_p - 1$, where $\kappa_1$ was the curvature at which the moment dropped below the plastic moment, $M_p$. Figure 6.29 shows two hypothetical moment-curvature relationships, which are similar, but one sample had a slightly larger rotation capacity than the other. The ratio of the rotation capacities is $R_1/R_2 = 0.51$, suggesting that the two results were considerably different. The discrepancy is caused by the definition of $R$ in which 1 is subtracted from the normalised curvature. It is more appropriate to consider the ratios of the curvatures of the two samples. In this example, $(\kappa_1/\kappa_p)_1/(\kappa_1/\kappa_p)_2 = 0.87$, which is a much better measure of the relative closeness of the two responses. Hence, when the results are considered in subsequent sections, the values of $\kappa_1/\kappa_p$ will be compared. For some cases the specimen buckles before $M_p$ is reached, and consequently the value $\kappa_1$ is undefined. In such instances, the curvature at buckling ($\kappa$), is used.

Figure 6.29: Hypothetical ABAQUS Simulations
6.2.9.5 Imperfection Size

The imperfection profiles in Appendix F indicated that the average bow-out imperfections for 150 × 50 RHS were \( \delta_w \approx +0.3 \text{ mm}, \) and \( \delta_f \approx -0.1 \text{ mm}, \) which correspond to \( \delta_w = d/500, \) and \( \delta_f = -b/500. \) For other sized sections, there was less data available on the nature of the imperfections. Analyses were carried out to examine the effect of varying the size of the imperfection.

Figures 6.30 and 6.31 show moment - curvature graphs for RHS with size 150 × 75. The code in the legend refers to \( d, \) \( b, \) 10\( t, \) and the ratio \( d/\delta_w \) or \( b/\delta_f. \) Therefore the specimen denoted as 150_75_50_imp_1000 represents a 150 × 75 × 5.0 RHS with imperfections \( \delta_w = d/1000, \) and \( \delta_f = -b/1000. \) For the analyses, the half wavelength of the imperfection was \( d/2 \) and the material properties were taken as those of specimen S02 (see Appendix C).

Figure 6.30: Effect of Imperfection Size 150 × 75 × 3.0 RHS
Figures 6.30 and 6.31 show how the magnitude of the imperfection affected the rotation capacity. The rotation capacity decreased as the imperfection increased. When the imperfection was small (d/2000 and d/1500), changing the size of the imperfection had less affect on the rotation capacity than changing the imperfection size when the imperfections were bigger (d/1000 and d/500).

The finite element model included imperfections and material properties which have been found to produce results that closely matched the maximum moment, the location of the local buckle, and the shape of the post-buckling response observed experimentally. However, it was of most interest to see how closely the predicted rotation capacities compared with the observed results. The effect of imperfection size, and the comparison of the finite element results with the observed results are shown in Figures 6.32 to 6.36. Sections analysed were either 150 × 150 (d/b = 1.0), 150 × 90 (d/b = 1.66), 150 × 75 (d/b = 2.0), 150 × 50 (d/b = 3.0), and 150 × 37.5 (d/b = 4.0), with a variety of thicknesses, and different imperfection sizes: d/250, d/500, d/1000, d/1500, or d/2000 (for the web), and b/250, b/500, b/1000, b/1500, or b/2000 (for the flange). The material properties assumed were those for specimen BS02 (see Appendix C). The experimental results included in the graphs are those given in Chapter 3 conducted as part of this thesis, and the tests by Hasan and Hancock (1988), and Zhao and Hancock (1991). It needs to be reinforced that the
ABAQUS analyses in Figures 6.32 to 6.36 were all performed on RHS with web depth $d = 150$ mm and material properties for specimen BS02. The experimental results shown in comparison were from a variety of RHS with varying dimensions and material properties. Note that the following figures use the AS 4100 definition of web slenderness, where $\lambda_w = (d - 2t)/t \cdot \sqrt{(f_c/250)}$.

For easy reference, the results in Figures 6.32 to 6.36 are also tabulated in Table 6.2. Table 6.2 also compares the normalised curvature ($\kappa_t/\kappa_p$) for each specimen for a given imperfection size, to the value of $\kappa_t/\kappa_p$ for the same sized specimen with an imperfection of $d/2000$ and $b/2000$. The comparison provides a measure of the relative effect of the imperfection size. Figure 6.37 compares the relationship between rotation capacity and web slenderness for each aspect ratio for a constant imperfection size of $d/1000$ and $b/1000$.

![Graph showing relationship between rotation capacity and web slenderness for different imperfection sizes.](image)

Figure 6.32: Effect of Imperfection Size, $d/b = 1.0$
Figure 6.33: Effect of Imperfection Size, $\frac{d}{b} = 1.66$

Figure 6.34: Effect of Imperfection Size, $\frac{d}{b} = 2.0$
Figure 6.35: Effect of Imperfection Size, \(d/b = 3.0\)

Figure 6.36: Effect of Imperfection Size, \(d/b = 4.0\)

251
<table>
<thead>
<tr>
<th>Section</th>
<th>Imp 1/250</th>
<th>Imp 1/500</th>
<th>Imp 1/1000</th>
<th>Imp 1/1500</th>
<th>Imp 1/2000</th>
</tr>
</thead>
<tbody>
<tr>
<td>150 × 37.5 × 1.5</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>150 × 37.5 × 2.0</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>150 × 37.5 × 2.5</td>
<td>0.90</td>
<td>1.26</td>
<td>1.59</td>
<td>1.58</td>
<td>1.78</td>
</tr>
<tr>
<td>150 × 37.5 × 3.0</td>
<td>1.82</td>
<td>2.51</td>
<td>3.36</td>
<td>3.50</td>
<td>3.89</td>
</tr>
<tr>
<td>150 × 37.5 × 3.5</td>
<td>2.96</td>
<td>4.49</td>
<td>5.75</td>
<td>6.26</td>
<td>6.71</td>
</tr>
<tr>
<td>150 × 37.5 × 4.0</td>
<td>5.13</td>
<td>6.85</td>
<td>8.01</td>
<td>8.76</td>
<td>8.83</td>
</tr>
<tr>
<td>150 × 37.5 × 4.5</td>
<td>7.06</td>
<td>8.60</td>
<td>9.74</td>
<td>9.92</td>
<td>10.01</td>
</tr>
<tr>
<td>Average</td>
<td>0.69</td>
<td>0.82</td>
<td>0.92</td>
<td>0.96</td>
<td>1.00</td>
</tr>
<tr>
<td>150 × 50 × 2.0</td>
<td>0.00</td>
<td>0.00</td>
<td>0.46</td>
<td>0.54</td>
<td>0.58</td>
</tr>
<tr>
<td>150 × 50 × 2.5</td>
<td>0.77</td>
<td>1.05</td>
<td>1.38</td>
<td>1.57</td>
<td>1.72</td>
</tr>
<tr>
<td>150 × 50 × 3.0</td>
<td>1.62</td>
<td>2.33</td>
<td>2.91</td>
<td>3.36</td>
<td>3.40</td>
</tr>
<tr>
<td>150 × 50 × 3.5</td>
<td>2.99</td>
<td>3.95</td>
<td>5.00</td>
<td>5.67</td>
<td>5.95</td>
</tr>
<tr>
<td>150 × 50 × 4.0</td>
<td>4.40</td>
<td>6.17</td>
<td>7.27</td>
<td>7.88</td>
<td>8.33</td>
</tr>
<tr>
<td>150 × 50 × 4.5</td>
<td>6.57</td>
<td>8.16</td>
<td>9.17</td>
<td>9.71</td>
<td>9.90</td>
</tr>
<tr>
<td>150 × 50 × 5.0</td>
<td>8.22</td>
<td>9.53</td>
<td>10.44</td>
<td>10.69</td>
<td>10.82</td>
</tr>
<tr>
<td>Average</td>
<td>0.67</td>
<td>0.80</td>
<td>0.91</td>
<td>0.97</td>
<td>1.00</td>
</tr>
<tr>
<td>150 × 75 × 2.0</td>
<td>0.00</td>
<td>0.00</td>
<td>0.46</td>
<td>0.54</td>
<td>0.58</td>
</tr>
<tr>
<td>150 × 75 × 2.5</td>
<td>0.77</td>
<td>1.05</td>
<td>1.38</td>
<td>1.57</td>
<td>1.72</td>
</tr>
<tr>
<td>150 × 75 × 3.0</td>
<td>1.62</td>
<td>2.33</td>
<td>2.91</td>
<td>3.36</td>
<td>3.40</td>
</tr>
<tr>
<td>150 × 75 × 3.5</td>
<td>2.99</td>
<td>3.95</td>
<td>5.00</td>
<td>5.67</td>
<td>5.95</td>
</tr>
<tr>
<td>150 × 75 × 4.0</td>
<td>4.40</td>
<td>6.17</td>
<td>7.27</td>
<td>7.88</td>
<td>8.33</td>
</tr>
<tr>
<td>150 × 75 × 4.5</td>
<td>6.57</td>
<td>8.16</td>
<td>9.17</td>
<td>9.71</td>
<td>9.90</td>
</tr>
<tr>
<td>150 × 75 × 5.0</td>
<td>8.22</td>
<td>9.53</td>
<td>10.44</td>
<td>10.69</td>
<td>10.82</td>
</tr>
<tr>
<td>Average</td>
<td>0.67</td>
<td>0.80</td>
<td>0.91</td>
<td>0.97</td>
<td>1.00</td>
</tr>
<tr>
<td>150 × 90 × 2.0</td>
<td>0.00</td>
<td>0.00</td>
<td>0.46</td>
<td>0.54</td>
<td>0.58</td>
</tr>
<tr>
<td>150 × 90 × 2.5</td>
<td>0.77</td>
<td>1.05</td>
<td>1.38</td>
<td>1.57</td>
<td>1.72</td>
</tr>
<tr>
<td>150 × 90 × 3.0</td>
<td>1.62</td>
<td>2.33</td>
<td>2.91</td>
<td>3.36</td>
<td>3.40</td>
</tr>
<tr>
<td>150 × 90 × 3.5</td>
<td>2.99</td>
<td>3.95</td>
<td>5.00</td>
<td>5.67</td>
<td>5.95</td>
</tr>
<tr>
<td>150 × 90 × 4.0</td>
<td>4.40</td>
<td>6.17</td>
<td>7.27</td>
<td>7.88</td>
<td>8.33</td>
</tr>
<tr>
<td>150 × 90 × 4.5</td>
<td>6.57</td>
<td>8.16</td>
<td>9.17</td>
<td>9.71</td>
<td>9.90</td>
</tr>
<tr>
<td>150 × 90 × 5.0</td>
<td>8.22</td>
<td>9.53</td>
<td>10.44</td>
<td>10.69</td>
<td>10.82</td>
</tr>
<tr>
<td>Average</td>
<td>0.67</td>
<td>0.80</td>
<td>0.91</td>
<td>0.97</td>
<td>1.00</td>
</tr>
<tr>
<td>150 × 90 × 6.0</td>
<td>0.00</td>
<td>0.00</td>
<td>0.46</td>
<td>0.54</td>
<td>0.58</td>
</tr>
<tr>
<td>150 × 90 × 7.0</td>
<td>0.00</td>
<td>0.00</td>
<td>0.46</td>
<td>0.54</td>
<td>0.58</td>
</tr>
<tr>
<td>150 × 90 × 8.0</td>
<td>0.00</td>
<td>0.00</td>
<td>0.46</td>
<td>0.54</td>
<td>0.58</td>
</tr>
<tr>
<td>Average</td>
<td>0.67</td>
<td>0.80</td>
<td>0.91</td>
<td>0.97</td>
<td>1.00</td>
</tr>
<tr>
<td>Standard deviation</td>
<td>0.106</td>
<td>0.071</td>
<td>0.035</td>
<td>0.020</td>
<td>0.0</td>
</tr>
</tbody>
</table>

Table 6.2: Summary of Numerical Analyses to Investigate Effect of Imperfection Size
Several observations can be made from Figures 6.32 to 6.37:

The size of the imperfection has a significant effect on the rotation capacity. Imperfection size has a lesser effect on the more slender sections \((R < 1)\), and has a greater effect for stockier sections. For a given aspect ratio, the band of results, encompassing the varying imperfection sizes, widens as the slenderness decreases. The effect of imperfections on stockier sections is an unexpected result of the study.

In general, the rotation capacity increased as the imperfection magnitude decreased. There were some rare occasions when the rotation capacity decreased when the imperfection size was decreased. On such occurrences, the values of \(R\) were low, and there was some lack of precision in the value of \(R\) due to size of the increment relative to the curvature at buckling. Hence such anomalies were not considered important.
There is a clear non-linear trend between the web slenderness and rotation capacity for a given aspect ratio and imperfection size. The trend resembles the shape of a hyperbola. It may be possible to simplify the trend by a bi-linear relationship: a steep line for lower slenderness, and a line of less gradient at higher slenderness values. For higher slenderness sections, the increase in rotation capacity with decreasing web slenderness is small, but for the stockier sections, rotation capacity increases more rapidly with reduced slenderness. The trend displayed by the ABAQUS results fits well with the experimental results for sections with $d/b = 3.0$, in which the slope of the line that approximates the relationship between rotation capacity and web slenderness increases as the web slenderness increases. Sully (1996) found a similar bi-linear trend when comparing the critical local buckling strain of SHS (under pure compression or pure bending) to the plate slenderness.

The flange - web interaction that was identified in the experiments is also apparent in the numerical simulations. For example, compare the rotation capacities for an imperfection size of 1/1000 for $150 \times 50 \times 3$ ($R = 2.91$), $150 \times 75 \times 3$ ($R = 1.78$), and $150 \times 90 \times 3$ ($R = 0.97$). Each specimen has the same web slenderness, but different flange slenderness. The rotation capacity decreases as the flange slenderness increases for a constant web slenderness.

No single line, which corresponds to a given imperfection size, is a very good match for the experimental results. For example, the ABAQUS results for $d/b = 3.0$ and imperfection of 1/250 match the experimental results well when $\lambda_w \approx 48$, while in the range $58 < \lambda_w < 65$, the results for an imperfection of 1/500 provide a reasonable estimation of the experimental results, and for $75 < \lambda_w < 85$ an imperfection of 1/2000 gives results closest to the experimental values. For $d/b = 1.0$, the ABAQUS results for an imperfection of 1/500 are close to the experimental results in the range $37 < \lambda_w < 48$, while in the range $25 < \lambda_w < 35$, an imperfection of 1/2000 more accurately simulates the test results, but still underestimates the values of $R$. This suggests considerable variability in the imperfections with changing aspect ratios and slenderness, and that as the slenderness increase, larger imperfections are required to simulate the experimental behaviour. There is no reason why the same relative magnitude of imperfections should be applicable to sections with a range of slenderness values.
The measured imperfection profiles (Appendix F) indicated that the bow-out imperfections were roughly constant along the length of the beam, but imperfections that were not practical to measure were introduced by welding the loading plates to the RHS. It is likely that the stockier (thicker) sections would not be as severely affected (i.e., size of imperfections induced) by the welding process compared to the more slender sections, and hence the imperfections in the stockier sections near the loading plates is smaller.

The sensitivity to imperfection is unaffected by the aspect ratio, and can be identified in Table 6.2. For example, regardless of the aspect ratio, the normalised curvature $\kappa_c/\kappa_p$ for a section with an imperfection of $d/250$ or $b/250$ is approximately 70% of that for a specimen with an imperfection of $d/2000$ or $b/2000$.

It should be recalled that the elements being used in the analysis, S4R5, are defined as “thin” shells (Section 6.2.3), most appropriate for use when $d/t, b/t > 15$. The stockiest sections analysed sometimes exceed that limit, so there is some uncertainty to the accuracy of some of the results. However, such sections clearly demonstrate considerable rotation capacity, and if there is some uncertainty in the results, that is not of major concern.

For sections with $d/b = 1.0$, the values of $R$ predicted by ABAQUS are consistently below the observed experimental values, even when large imperfections are imposed. The experimental values are predominately those of Hasan and Hancock (1988) and Zhao and Hancock (1991), while the ABAQUS simulations were performed with material properties taken from the specimens tested as part of this thesis. It is likely that the different material properties (in particular the strain hardening behaviour) were the cause of the difference. The effect of material properties for this case is discussed in more detail in Section 6.3.
6.2.9.6 Effect of Scale Factor

The previous section considered the effect of imperfection size on the rotation capacity of RHS beams. All the numerical analyses were performed on RHS with web depth \( d = 150 \) mm, and varying flange width and thickness. However, the analysis did not consider if there was a scale factor that could affect the rotation capacity. In other words, is the rotation capacity the same for a \( 150 \times 50 \times 4 \) RHS and a \( 75 \times 25 \times 2 \) RHS, which vary by a scale factor of 2?

Figures 6.38 and 6.39 show the numerical moment-curvature relationships for two series of RHS which vary only by a scale factor in cross-sectional dimensions. The imperfections imposed were \( \delta_w = d/500 \), and \( \delta_t = -b/500 \), with a half-wavelength \( L_w = d/2 \). The material properties assumed were those for specimen BS02 (see Appendix C). The length of the beam was not scaled, so for each analysis the length of the beam is that defined in Figure 6.1. The mesh discretisation was not changed. The code in the legend of each graph refers to \( d, b, \) and \( t \).

![Figure 6.38: Scale Effect](image-url)
Figures 6.38 and 6.39 indicate that the scale of the model did not affect the curvature at which the local buckle forms. However, the post buckling response was slightly different. The curvature dropped more sharply for sections with smaller cross-section dimensions. The size of the local buckle changed with the scale of the beam. Since the beam length was constant, the relative size of the buckle with respect to the beam length reduced, as the section was smaller. Since the curvature was calculated as an average curvature over the beam length (refer to Figure 6.12), the curvature drops off much more steeply for smaller sections after the buckle has formed.

6.2.9.7 Effect of Steel Grade

In Section 6.2.9.5, Figures 6.32 to 6.36 showed the relationship between web slenderness and rotation capacity for beams with different aspect ratios and imperfection sizes. All the analyses were performed assuming typical section properties of a Grade C450 specimen (Specimen BS02: $f_y = 457$ MPa). Selected analyses were repeated using typical material properties for a Grade C350 specimen (Specimen BS11: $f_y = 370$ MPa - refer to Appendix C for the stress-strain
curves). Only two aspect ratios were considered, and sinusoidally varying imperfections of \( \delta_w = d/1000 \), and \( \delta_t = -b/1000 \), with a half-wavelength \( L_w = d/2 \), were prescribed. The resulting relationship between web slenderness and rotation capacity is shown in Figure 6.40.

Figure 6.40: Effect of Steel Grade on Rotation Capacity

Figure 6.40 demonstrates that the two different steel properties considered had a minor effect on the slenderness - rotation capacity relationship. Particularly as the sections became stockier, and \( R > 3 \), the different steel grades gave slightly different results, but the variance is not very significant. Changing the steel grade does alter the rotation capacity of a given a section - increasing the yield stress reduces the rotation capacity, but the slenderness also increases (as it is dependent on \( f_y \)), so the slenderness - rotation capacity relationship is not affected considerably. However, there is no reason to assume that the relationship would be the same if properties of cold-formed steel from a different supplier were used. Table 6.3 summarises the values graphed in Figure 6.40.
<table>
<thead>
<tr>
<th>Section</th>
<th>Grade C350</th>
<th>Grade C450</th>
<th>Section</th>
<th>Grade C350</th>
<th>Grade C450</th>
</tr>
</thead>
<tbody>
<tr>
<td>R</td>
<td>R</td>
<td>R</td>
<td>R</td>
<td>R</td>
<td>R</td>
</tr>
<tr>
<td>150 × 50 × 2.0</td>
<td>0.00</td>
<td>0.46</td>
<td>150 × 90 × 2.5</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>150 × 50 × 2.5</td>
<td>2.15</td>
<td>1.38</td>
<td>150 × 90 × 3.0</td>
<td>1.43</td>
<td>0.97</td>
</tr>
<tr>
<td>150 × 50 × 3.0</td>
<td>4.63</td>
<td>2.91</td>
<td>150 × 90 × 3.5</td>
<td>2.86</td>
<td>1.90</td>
</tr>
<tr>
<td>150 × 50 × 3.5</td>
<td>7.44</td>
<td>5.00</td>
<td>150 × 90 × 4.0</td>
<td>4.72</td>
<td>3.24</td>
</tr>
<tr>
<td>150 × 50 × 4.0</td>
<td>10.12</td>
<td>7.27</td>
<td>150 × 90 × 4.5</td>
<td>7.16</td>
<td>4.85</td>
</tr>
<tr>
<td>150 × 50 × 4.5</td>
<td>12.24</td>
<td>9.17</td>
<td>150 × 90 × 5.0</td>
<td>9.42</td>
<td>6.66</td>
</tr>
<tr>
<td>150 × 50 × 5.0</td>
<td>14+</td>
<td>10.44</td>
<td>150 × 90 × 5.5</td>
<td>11.75</td>
<td>8.69</td>
</tr>
<tr>
<td>150 × 90 × 6.0</td>
<td>13.21</td>
<td>9.99</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 6.3: Comparison of Rotation Capacity with Steel Grade

The analysis was extended to encompass the material properties from all the specimens in the bending tests. A 150 × 50 × 3.0 RHS, with imperfections of $\delta_w = d/500$, and $\delta_x = -b/500$, and a half-wavelength $L_w = d/2$, was examined, and each analysis encompassed each of the different material properties from Appendix C. The results are shown in Table 6.4 and Figure 6.41.
<table>
<thead>
<tr>
<th>Material Properties</th>
<th>Yield Stress $f_y$ (MPa)</th>
<th>Rotation Capacity $R$</th>
</tr>
</thead>
<tbody>
<tr>
<td>S01</td>
<td>441</td>
<td>2.72</td>
</tr>
<tr>
<td>S02</td>
<td>457</td>
<td>2.33</td>
</tr>
<tr>
<td>S03</td>
<td>444</td>
<td>2.57</td>
</tr>
<tr>
<td>S04</td>
<td>446</td>
<td>2.54</td>
</tr>
<tr>
<td>S05</td>
<td>444</td>
<td>2.54</td>
</tr>
<tr>
<td>S06</td>
<td>449</td>
<td>2.40</td>
</tr>
<tr>
<td>S07</td>
<td>411</td>
<td>2.94</td>
</tr>
<tr>
<td>S08</td>
<td>457</td>
<td>2.31</td>
</tr>
<tr>
<td>S09</td>
<td>439</td>
<td>2.39</td>
</tr>
<tr>
<td>S10</td>
<td>422</td>
<td>2.49</td>
</tr>
<tr>
<td>S11</td>
<td>370</td>
<td>3.27</td>
</tr>
<tr>
<td>S12</td>
<td>400</td>
<td>2.96</td>
</tr>
<tr>
<td>S13</td>
<td>397</td>
<td>2.88</td>
</tr>
<tr>
<td>S19</td>
<td>445</td>
<td>2.10</td>
</tr>
<tr>
<td>F01</td>
<td>410</td>
<td>2.98</td>
</tr>
<tr>
<td>F02</td>
<td>423</td>
<td>2.67</td>
</tr>
<tr>
<td>J07</td>
<td>349</td>
<td>3.81</td>
</tr>
</tbody>
</table>

Table 6.4: Summary of Variation of Rotation Capacity with Material Properties
Figure 6.41: Variation of Rotation Capacity with Different Material Properties

Figure 6.41 indicates that while changing the material properties affects the rotation capacity, there is no significant impact on the relationship between rotation capacity and web slenderness, provided that the appropriate value of $f_y$ is used to calculate the slenderness and the plastic moment. The material properties considered had similar shapes (the gradual yielding and strain hardening), but varying values. The similar properties are expected, as all samples were produced by the same manufacturer. Other material properties, such as those for hot-formed steel, have not been considered. Appendix J briefly examines the effect of different material properties, and the strain hardening behaviour has considerable influence on the rotation capacity. It is likely that steels with different material properties to those examined in this thesis, would yield a different correlation between web slenderness and rotation capacity. For example, Appendix B describes tests on two hot-formed RHS, and those sections displayed rotation capacity notably different to the trends established by the cold-formed RHS.
6.3 SIMULATION OF THE BENDING TESTS

6.3.1 GENERAL

Section 6.2 described the stages of development of the finite element model in order to simulate the bending tests on cold-formed RHS referred to in Chapter 3. It has been established that the material properties obtained from coupon tests need to be considered, and that local imperfections near the loading plate, with a half-wavelength $L_w = d/2$ should be included. The rotation capacity achieved in the analysis depends on the magnitude of the imperfections, which have been varied from $\delta_w = d/2000$ to $\delta_w = d/250$, and $\delta_i = -b/2000$ to $\delta_i = -b/250$.

A set of ABAQUS simulations was performed on specimens to model the bending tests outlined in Chapter 3. The exact measured dimensions, including the measured corner radius, $r_e$, (refer to Chapter 3) and the measured material properties reported in Appendix C, were used in the analysis. Only the beams tested by the “parallel plate” method were simulated by ABAQUS. The tests by Hasan and Hancock (1988), and Zhao and Hancock (1991) were also modelled numerically. However, as the exact distribution of material properties around the cross-section were not available, the material properties input into the models of Hasan and Hancock’s Grade C350 RHS/SHS were those from Specimen BS11, and for Zhao and Hancock’s Grade C450 RHS/SHS the material properties of Specimen BS02 were utilised. The magnitude of the imperfection varied from $\delta_w = d/2000$ to $\delta_w = d/250$, and $\delta_i = -b/2000$ to $\delta_i = -b/250$.

6.3.2 RESULTS

Figures 6.42 to 6.44 compare the experimental moment - curvature relationships of three of the RHS beams with ABAQUS simulations. The code in the legend of the graphs indicates the magnitude of the imperfection in the simulation. The ABAQUS simulations give slightly lower predictions of the maximum moment than obtained experimentally. It was explained in Section 6.2.6 that the yield stress varies continuously around the section, while the ABAQUS model incorporates only three separate material definitions in the flange, web, and corner of the section. The variability of rotation capacity with imperfection size is evident.
Figure 6.42: ABAQUS Simulations for BS02C: 150 × 50 × 4.0 C450

Figure 6.43: ABAQUS Simulations for BS03B: 150 × 50 × 3.0 C450
Tables 6.5 to 6.7 summarise the results of the numerical analyses simulating the bending tests. The rotation capacity achieved for each of five different imperfection sizes is listed as well as the comparison between the value of $\kappa_p/\kappa_1$ achieved in each simulation with the value obtained experimentally. Separate tables are given for the experiments performed as part of this thesis, and those of Hasan and Hancock (1988), and Zhao and Hancock (1991). The tables highlight the size of imperfection that is closest to the bow-out imperfection measured.
<table>
<thead>
<tr>
<th>Section</th>
<th>Cut from section</th>
<th>$R_{exp}$</th>
<th>$R_{AB}$</th>
<th>$R_{AB}$</th>
<th>$R_{AB}$</th>
<th>$R_{AB}$</th>
<th>$R_{AB}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Imp 1/250</td>
<td>Imp 1/500</td>
<td>Imp 1/1000</td>
<td>Imp 1/1500</td>
<td>Imp 1/2000</td>
<td></td>
</tr>
<tr>
<td>BS01B</td>
<td>150×50×5.0 C450</td>
<td>13+</td>
<td>7.75</td>
<td>9.32</td>
<td>10.19</td>
<td>10.62</td>
<td>10.87</td>
</tr>
<tr>
<td>BS01C</td>
<td>150×50×5.0 C450</td>
<td>9+</td>
<td>7.68</td>
<td>9.32</td>
<td>10.12</td>
<td>10.56</td>
<td>10.8</td>
</tr>
<tr>
<td>BS02B</td>
<td>150×50×4.0 C450</td>
<td>6.6</td>
<td>4.30</td>
<td>0.70</td>
<td>5.92</td>
<td>0.91</td>
<td>7.16</td>
</tr>
<tr>
<td>BS02C</td>
<td>150×50×4.0 C450</td>
<td>7.7</td>
<td>4.05</td>
<td>0.58</td>
<td>5.60</td>
<td>0.76</td>
<td>6.84</td>
</tr>
<tr>
<td>BF02</td>
<td>150×50×4.0 C450</td>
<td>9.5</td>
<td>5.16</td>
<td>0.59</td>
<td>6.79</td>
<td>0.74</td>
<td>7.97</td>
</tr>
<tr>
<td>BS03A</td>
<td>150×50×3.0 C450</td>
<td>2.7</td>
<td>1.82</td>
<td>0.76</td>
<td>2.40</td>
<td>0.92</td>
<td>3.09</td>
</tr>
<tr>
<td>BS03B</td>
<td>150×50×3.0 C450</td>
<td>2.3</td>
<td>1.67</td>
<td>0.81</td>
<td>2.33</td>
<td>1.01</td>
<td>2.94</td>
</tr>
<tr>
<td>BS03C</td>
<td>150×50×3.0 C450</td>
<td>2.9</td>
<td>1.67</td>
<td>0.68</td>
<td>2.29</td>
<td>0.84</td>
<td>2.75</td>
</tr>
<tr>
<td>BS04B</td>
<td>150×50×2.5 C450</td>
<td>1.4</td>
<td>1.14</td>
<td>0.89</td>
<td>1.55</td>
<td>1.06</td>
<td>1.87</td>
</tr>
<tr>
<td>BS04C</td>
<td>150×50×2.5 C450</td>
<td>1.2</td>
<td>1.12</td>
<td>0.96</td>
<td>1.46</td>
<td>1.12</td>
<td>1.70</td>
</tr>
<tr>
<td>BS05A</td>
<td>150×50×2.3 C450</td>
<td>0</td>
<td>0.98</td>
<td>0.98</td>
<td>0.84</td>
<td>1.08</td>
<td>1.02</td>
</tr>
<tr>
<td>BS05B</td>
<td>150×50×2.3 C450</td>
<td>0.6</td>
<td>1.05</td>
<td>0.91</td>
<td>1.12</td>
<td>1.33</td>
<td>1.23</td>
</tr>
<tr>
<td>BS05C</td>
<td>150×50×2.3 C450</td>
<td>0</td>
<td>1.02</td>
<td>1.02</td>
<td>0.87</td>
<td>1.15</td>
<td>1.07</td>
</tr>
<tr>
<td>BS06B</td>
<td>100×50×2.0 C450</td>
<td>0.8</td>
<td>0.88</td>
<td>1.04</td>
<td>1.40</td>
<td>1.33</td>
<td>1.96</td>
</tr>
<tr>
<td>BS06C</td>
<td>100×50×2.0 C450</td>
<td>0.8</td>
<td>0.89</td>
<td>1.05</td>
<td>1.42</td>
<td>1.34</td>
<td>1.88</td>
</tr>
<tr>
<td>BS07B</td>
<td>75×50×2.0 C450</td>
<td>1.7</td>
<td>1.10</td>
<td>0.78</td>
<td>1.55</td>
<td>0.94</td>
<td>2.11</td>
</tr>
<tr>
<td>BS07C</td>
<td>75×50×2.0 C450</td>
<td>1.9</td>
<td>1.12</td>
<td>0.73</td>
<td>1.59</td>
<td>0.89</td>
<td>2.22</td>
</tr>
<tr>
<td>BS08B</td>
<td>75×25×2.0 C450</td>
<td>5.7</td>
<td>4.50</td>
<td>0.82</td>
<td>5.48</td>
<td>0.97</td>
<td>6.61</td>
</tr>
<tr>
<td>BS08C</td>
<td>75×25×2.0 C450</td>
<td>5.7</td>
<td>3.94</td>
<td>0.74</td>
<td>5.35</td>
<td>0.95</td>
<td>6.38</td>
</tr>
<tr>
<td>BS09B</td>
<td>75×25×1.6 C450</td>
<td>2.2</td>
<td>1.61</td>
<td>0.82</td>
<td>2.32</td>
<td>1.04</td>
<td>3.18</td>
</tr>
<tr>
<td>BS09C</td>
<td>75×25×1.6 C450</td>
<td>2.5</td>
<td>1.67</td>
<td>0.76</td>
<td>2.39</td>
<td>0.97</td>
<td>3.27</td>
</tr>
<tr>
<td>BS10B</td>
<td>75×25×1.6 C350</td>
<td>1.9</td>
<td>1.78</td>
<td>0.96</td>
<td>2.53</td>
<td>1.22</td>
<td>3.15</td>
</tr>
<tr>
<td>BS10C</td>
<td>75×25×1.6 C350</td>
<td>2.6</td>
<td>1.83</td>
<td>0.79</td>
<td>2.55</td>
<td>0.99</td>
<td>3.18</td>
</tr>
<tr>
<td>BS11B</td>
<td>150×50×3.0 C350</td>
<td>4.1</td>
<td>2.45</td>
<td>0.68</td>
<td>4.56</td>
<td>1.09</td>
<td>4.66</td>
</tr>
<tr>
<td>BS11C</td>
<td>150×50×3.0 C350</td>
<td>3.6</td>
<td>2.40</td>
<td>0.74</td>
<td>3.21</td>
<td>0.92</td>
<td>4.33</td>
</tr>
<tr>
<td>BS12B</td>
<td>150×50×2.0 C350</td>
<td>12.9</td>
<td>6.54</td>
<td>0.54</td>
<td>8.38</td>
<td>0.67</td>
<td>9.97</td>
</tr>
<tr>
<td>BS12C</td>
<td>100×50×2.0 C350</td>
<td>1.2</td>
<td>1.16</td>
<td>0.98</td>
<td>1.80</td>
<td>1.27</td>
<td>2.43</td>
</tr>
<tr>
<td>BS13B</td>
<td>125×75×3.0 C350</td>
<td>1.5</td>
<td>1.21</td>
<td>0.88</td>
<td>1.83</td>
<td>1.13</td>
<td>2.32</td>
</tr>
<tr>
<td>BS13C</td>
<td>125×75×3.0 C350</td>
<td>1.6</td>
<td>1.24</td>
<td>0.86</td>
<td>1.82</td>
<td>1.08</td>
<td>2.44</td>
</tr>
<tr>
<td>BS19A</td>
<td>100×100×3.0 C450</td>
<td>0.8</td>
<td>0.78</td>
<td>0.81</td>
<td>0.42</td>
<td>0.79</td>
<td>0.50</td>
</tr>
<tr>
<td>BJ07</td>
<td>150×50×4.0 C350</td>
<td>12.9</td>
<td>6.54</td>
<td>0.54</td>
<td>8.38</td>
<td>0.67</td>
<td>9.97</td>
</tr>
<tr>
<td>BF01</td>
<td>150×50×4.0 C350</td>
<td>10.7</td>
<td>4.90</td>
<td>0.50</td>
<td>6.54</td>
<td>0.64</td>
<td>7.65</td>
</tr>
</tbody>
</table>

Average: 0.84 1.03 1.23 1.33 1.40 0.80
Standard deviation: 0.14 0.16 0.20 0.22 0.24

Notes: (*) Local buckling not observed in the test, hence curvatures at buckling cannot be compared.
(#) $R = 0$ implies that $M_p$ was not reached, and hence $\kappa_i$ is not defined. $\kappa_i$ is used instead, where $\kappa_i$ is the curvature at which buckling occurred.
- Shading indicates the imperfection size in the ABAQUS analysis that most closely matched the observed imperfection size. If two columns are shaded, the observed imperfection was between the two values.
Unshaded rows imply that imperfection measurements were not made for that specimen.
- Bold indicates the ABAQUS analysis which most closely matches the observed result.

Table 6.5 Comparison of Experimental Results to ABAQUS Simulations
### Table 6.6: Comparison of Zhao and Hancock (1991) to ABAQUS Simulations

<table>
<thead>
<tr>
<th>Section Cut from section</th>
<th>( R_{\exp} )</th>
<th>Imp 1/250</th>
<th>Imp 1/500</th>
<th>Imp 1/1000</th>
<th>Imp 1/1500</th>
<th>Imp 1/2000</th>
</tr>
</thead>
<tbody>
<tr>
<td>BS1 100×100×3.8</td>
<td>4.17</td>
<td>1.51</td>
<td>1.62</td>
<td>2.11</td>
<td>2.32</td>
<td>2.45</td>
</tr>
<tr>
<td>BS2 100×100×3.3</td>
<td>3.55</td>
<td>1.06</td>
<td>1.31</td>
<td>1.56</td>
<td>1.75</td>
<td>1.85</td>
</tr>
<tr>
<td>BS3 100×100×2.8</td>
<td>0</td>
<td>0</td>
<td>0.85</td>
<td>0.46</td>
<td>0.51</td>
<td>0.55</td>
</tr>
<tr>
<td>BS4 75×75×3.3</td>
<td>5.60</td>
<td>2.08</td>
<td>2.76</td>
<td>3.31</td>
<td>3.80</td>
<td>4.06</td>
</tr>
<tr>
<td>BS5 75×75×2.8</td>
<td>2.76</td>
<td>1.13</td>
<td>1.48</td>
<td>1.71</td>
<td>1.90</td>
<td>1.90</td>
</tr>
<tr>
<td>BS6 75×75×2.3</td>
<td>1.41</td>
<td>0.57</td>
<td>0.71</td>
<td>0.80</td>
<td>0.96</td>
<td>0.87</td>
</tr>
<tr>
<td>BS7 65×65×2.3</td>
<td>2.00</td>
<td>0.90</td>
<td>1.02</td>
<td>1.36</td>
<td>1.57</td>
<td>1.62</td>
</tr>
<tr>
<td>BS8 125×75×3.8</td>
<td>5.97</td>
<td>2.71</td>
<td>3.80</td>
<td>4.76</td>
<td>4.98</td>
<td>5.37</td>
</tr>
<tr>
<td>BS9 125×75×3.3</td>
<td>4.79</td>
<td>1.72</td>
<td>2.31</td>
<td>2.63</td>
<td>3.02</td>
<td>3.21</td>
</tr>
<tr>
<td>BS10 100×50×2.8</td>
<td>7.35</td>
<td>3.17</td>
<td>4.59</td>
<td>6.00</td>
<td>6.44</td>
<td>6.92</td>
</tr>
</tbody>
</table>

Notes: (#) \( R = 0 \) implies that \( M_p \) was not reached, and hence \( \kappa_i \) is not defined. \( \kappa_o \) is used instead, where \( \kappa_o \) is the curvature at which buckling occurred.

**Bold** indicates the ABAQUS analysis which most closely matches the observed result.

### Table 6.7: Comparison of Hasan and Hancock (1988) to ABAQUS Simulations

<table>
<thead>
<tr>
<th>Section Cut from section</th>
<th>( R_{\exp} )</th>
<th>Imp 1/250</th>
<th>Imp 1/500</th>
<th>Imp 1/1000</th>
<th>Imp 1/1500</th>
<th>Imp 1/2000</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pilot 152×152×4.9</td>
<td>1.2</td>
<td><strong>1.00</strong></td>
<td>1.57</td>
<td>1.95</td>
<td>2.15</td>
<td>2.31</td>
</tr>
<tr>
<td>1/2(^{8}) 254×254×9.5</td>
<td>5.0</td>
<td>2.72</td>
<td>3.42</td>
<td>3.99</td>
<td>4.39</td>
<td><strong>4.51</strong></td>
</tr>
<tr>
<td>3/4(^{8}) 203×102×9.5</td>
<td>14+</td>
<td>15+</td>
<td>15+</td>
<td>15+</td>
<td>15+</td>
<td>15+</td>
</tr>
<tr>
<td>5/6(^{8}) 203×152×6.3</td>
<td>9.05</td>
<td>3.80</td>
<td>4.52</td>
<td>5.44</td>
<td>5.85</td>
<td><strong>6.01</strong></td>
</tr>
<tr>
<td>7/8(^{8}) 127×127×1.9</td>
<td>6.5</td>
<td>2.45</td>
<td>3.17</td>
<td>3.7</td>
<td>4.68</td>
<td><strong>4.39</strong></td>
</tr>
<tr>
<td>9/10(^{8}) 102×102×4.0</td>
<td>8.0</td>
<td>2.00</td>
<td>3.41</td>
<td>4.49</td>
<td>4.92</td>
<td><strong>5.29</strong></td>
</tr>
<tr>
<td>11/12(^{8}) 102×76×3.6</td>
<td>11.7</td>
<td>4.50</td>
<td>5.84</td>
<td>6.71</td>
<td>7.52</td>
<td><strong>7.89</strong></td>
</tr>
<tr>
<td>13/14(^{8}) 89×89×3.6</td>
<td>9.0</td>
<td>2.72</td>
<td>3.70</td>
<td>4.84</td>
<td>5.31</td>
<td><strong>5.60</strong></td>
</tr>
<tr>
<td>15/16(^{8}) 76×76×3.2</td>
<td>8.5</td>
<td>2.82</td>
<td>4.06</td>
<td>5.15</td>
<td>5.48</td>
<td><strong>5.78</strong></td>
</tr>
<tr>
<td>17/18(^{8}) 76×76×2.6</td>
<td>4.25</td>
<td>0.87</td>
<td>1.32</td>
<td>1.64</td>
<td>1.82</td>
<td><strong>1.82</strong></td>
</tr>
</tbody>
</table>

Average     | **0.48** | **0.61** | **0.71** | **0.77** | **0.79** |
Standard deviation | 0.18 | 0.23 | 0.25 | 0.27 | 0.28 |

Notes: (*) Local buckling not observed in the test, hence curvatures at buckling cannot be compared.

- **Bold** indicates the ABAQUS analysis which most closely matches the observed result.

### Summary for all tests: (This thesis, Hasan and Hancock, and Zhao and Hancock)

| Average     | 0.70 | 0.86 | 1.00 | 1.08 | 1.13 |
| Standard deviation | 0.21 | 0.26 | 0.31 | 0.34 | 0.37 |

Notes: (*) Local buckling not observed in the test, hence curvatures at buckling cannot be compared.

- **Bold** indicates the ABAQUS analysis which most closely matches the observed result.
6.3.3 DISCUSSION

The results in Table 6.5 indicate that different imperfections are required to simulate the experimental results. The trend can be identified by considering the variety of 150 × 50 C450 RHS. For the thickest section, 150 × 50 × 5 RHS, buckling was not observed experimentally. For 150 × 50 × 4 RHS, an imperfection of 1/2000 or 1/1000 gave results closest to those observed. Larger imperfections of 1/1000 or 1/500 were required to simulate the response of the 150 × 50 × 3 RHS. For the more slender section, 150 × 50 × 2.5 RHS, an imperfection of 1/500 or 1/250 was required, and 1/250 was needed to replicate the results of the most slender section, 150 × 50 × 2.3 RHS.

A possible explanation is that the imperfections caused by the welding of the loading plate to the RHS were initiating the local buckling. A thinner section was deformed more by a similar heat input, hence larger imperfections were induced. The sinusoidally varying bow-out imperfections simulated the effect of the imperfections caused by the weld, and hence greater imperfections were required as the slenderness increased.

For most of the tests by Hasan and Hancock (1988), and Zhao and Hancock (1991), the ABAQUS simulations underestimated the rotation capacity, even when the smallest imperfections of 1/2000 was prescribed. A source of inaccuracy in the analyses was the material properties used. The exact values of the stress-strain curves were not known ($f_y$ and $f_u$ were reported, but the exact shape of the curve, particularly strain hardening, was unclear), and consequently the material properties from RHS tested as part of this thesis were used (either S02 for Grade C450, and S11 for Grade C350). It was shown in Section 6.2.9.6 that the rotation capacity generally decreased with increasing yield stress. However the yield stress assumed by the material properties, were not significantly different from the yield stress reported, as summarised in Table 6.8. In many cases, the yield stress assumed was less than that reported, which would tend to yield a higher rotation capacity in the simulation.
Table 6.8: Summary of Yield Stresses

<table>
<thead>
<tr>
<th>Section</th>
<th>$f_y$ (MPa)</th>
<th>Section</th>
<th>$f_y$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BS1</td>
<td>459</td>
<td>Pilot</td>
<td>-</td>
</tr>
<tr>
<td>BS2</td>
<td>435</td>
<td>1/2§</td>
<td>438</td>
</tr>
<tr>
<td>BS3</td>
<td>466</td>
<td>3/4§</td>
<td>438</td>
</tr>
<tr>
<td>BS4</td>
<td>462</td>
<td>5/6§</td>
<td>368</td>
</tr>
<tr>
<td>BS5</td>
<td>490</td>
<td>7/8§</td>
<td>378</td>
</tr>
<tr>
<td>BS6</td>
<td>469</td>
<td>9/10§</td>
<td>373</td>
</tr>
<tr>
<td>BS7</td>
<td>479</td>
<td>11/12§</td>
<td>341</td>
</tr>
<tr>
<td>BS8</td>
<td>448</td>
<td>13/14§</td>
<td>358</td>
</tr>
<tr>
<td>BS9</td>
<td>452</td>
<td>15/16§</td>
<td>344</td>
</tr>
<tr>
<td>BS10</td>
<td>451</td>
<td>17/18§</td>
<td>347</td>
</tr>
<tr>
<td>Assumed Value</td>
<td>458</td>
<td>Assumed Value</td>
<td>370</td>
</tr>
</tbody>
</table>

Notes: (§) Hasan and Hancock performed two tests on each size specimen. The yield stress is the same for each.

Section J.3 shows the effect of strain hardening on rotation capacity. If the strain hardening portion of the material properties assumed is different from the “true” response of the sections of Hasan and Hancock, and Zhao and Hancock, the numerical simulations are likely to produce inaccurate results. In particular, Zhao and Hancock used Grade C450 specimens from a different supplier, Palmer Tube Mills Australia Pty Ltd, which were not in-line galvanised, so it is reasonable to assume that the material properties were different to those used in the ABAQUS simulations.

The importance of material properties is a notable finding of the finite element study. The physical isolation of Australia makes it difficult to test RHS made from a variety of suppliers with a range of material properties. It is recommended that further numerical simulations be performed using a variety of properties from different steels to examine the effect of the varying material properties.
6.4 PARAMETRIC STUDY

Appendix J examines the effects of various parameters, such as material properties, corner radius, and the effect of welding, which are not immediately relevant to the simulation of the bending tests.

Appendix J considers the effects of idealised material properties: elastic - plastic, elastic - strain hardening, and elastic - plastic - strain hardening. The most important observation was that increasing strain hardening modulus increased the rotation capacity.

Several RHS sections, which varied only by the size of the corner radius, were analysed, to consider whether the flat width or clear width definition of slenderness was more appropriate to use to describe the slenderness - rotation capacity relationship. The limited results suggest that the trend between web slenderness and rotation capacity is described better when the clear width definition of slenderness is used, rather than the flat width definition.

Some preliminary analyses were made to examine the effect of the weld. The weld was initially hot, and as the weld metal cooled, it induced deformations into the RHS, but the deformations induced were considerably smaller than the typical initial imperfections. The amplitude of the sinusoidally varying bow-out imperfections is a more critical factor in simulating the experimental results.

Full details are available in Appendix J.
6.5 CONCLUSIONS

6.5.1 SUMMARY

This chapter has described the finite element analysis of RHS beams. The finite element program ABAQUS was used for the analysis. The main aim of the analysis was to establish any trends in the plastic behaviour of cold-formed RHS beams that could supplement the experimental data gathered in Chapter 3.

The maximum loads predicted were slightly lower than those observed experimentally, since the numerical model assumed the same material properties across the whole flange, web or corner of the RHS. In reality, the variation of material properties is gradual, with a smooth increase of yield stress from the centre of a flat face, to a maximum in the corner. The slight error in predicted moment is not considered important, as a major aim of the analysis was to predict the rotation capacity.

A perfect specimen without imperfections achieved rotation capacities much higher than those observed experimentally. Introducing a bow-out imperfection, constant along the length of the beam, as was measured experimentally (approximately), did not affect the numerical results significantly. In order to simulate the effect of the imperfections induced by welding the loading plates to the beams in the experiments, the amplitude of the bow-out imperfection was varied sinusoidally along the length of the beam. The size of the imperfections had an unexpectedly large influence on the rotation capacity of the specimens.

It is likely that the imperfection caused by welding the loading plates to the RHS was a major factor affecting the experimentally observed behaviour. The sinusoidally varying imperfections in the ABAQUS model simulated the effects of the localised imperfections in the physical situation. Larger imperfections were required on the more slender sections to simulate the experimental results, since for the same type of welding, larger imperfections are induced in more slender sections. The sensitivity to imperfection size increased as the aspect ratio of the RHS decreased.
The finite element analysis determined similar trends as observed experimentally, namely that both flange slenderness and web slenderness affected the rotation capacity, and that for a given aspect ratio, the relationship between web slenderness and rotation capacity was non-linear, and the slope of the line describing the relationship increased as the web slenderness decreased.

6.5.2 FURTHER STUDY

During the course of finite element study, approximately 3000 individual analyses were performed, investigating the effects of many parameters. However, the cases considered were by no means exhaustive, and finite element analysis provided a relatively cheap and rapid alternative to physical investigations.

There are many areas where the finite element analysis described in this chapter can be extended to consider:

- the behaviour of Class 3 and Class 4 RHS beams,
- the effect of axial compression on rotation capacity,
- the significance of corner radius size,
- the examination of a large range of material properties, and
- the effect of imperfections induced by welding.
Chapter 7

CONCLUSIONS

7.1 GENERAL

This thesis has investigated the plastic behaviour of cold formed rectangular hollow sections. The main aim was to assess the suitability of cold-formed rectangular hollow sections for plastic design, since plastic design of cold-formed RHS is not permitted by many of the current steel design specifications.

The project has been predominately experimental, having involved an extensive range of tests on cold-formed Grade C350 and Grade C450 (DuraGal) RHS beams, joints and frames. A substantial set of finite element analyses was also carried out on models of RHS beams.

- A total of 44 bending tests was performed on 16 RHS and SHS of various dimensions. The main aim of the bending tests was to examine the relationship between web slenderness and rotation capacity.
- Eighteen connection tests were conducted on different types of joints that might be suitable for the column - rafter connection in a portal frame. The aim of the connection tests was to find a connection in cold-formed RHS suitable for plastic hinge formation.
- Tests on three prototype portal frames, 4 m high and 7 m span, were carried out under loading conditions simulating either gravity, or transverse wind load. A major aim was to investigate if a plastic collapse mechanism could form in the portal frame.
- Approximately 3000 individual finite element analyses were performed with the commercial package ABAQUS, to simulate the behaviour of the bending tests on RHS outlined above.

The main conclusion of the thesis is that cold-formed RHS can be used in plastic design, but stricter element slenderness limits, particularly for webs, and consideration of the ductility of welded connections, are required.
7.2 LITERATURE REVIEW

Chapter 2 examined the history of plastic design and the basis of the current element slenderness limits in steel design standards. The most significant findings in the literature review were:

- The current web slenderness limits are based on investigations into the behaviour of I-section webs, yet are applied to both I-sections and RHS.
- Many researchers have recommended slenderness limits that account for flange - web interaction, yet current design standards specify independent web and flange slenderness limits.
- There are inconsistencies in development of the Class 1 web slenderness limit in the limit states standard AISC LRFD (AISC 1994). The Class 1 limit for webs in AISC LRFD is significantly less stringent than the corresponding value in the working stress equivalent, AISC ASD (AISC 1989).

7.3 BENDING TESTS OF COLD-FORMED RHS BEAMS

Plastic bending tests were performed on a range of cold-formed RHS with different web and flange slenderness, aspect ratio and yield stress grade. The aim was to establish a relationship between the web and flange slenderness and the rotation capacity, in order to investigate the Class 1 slenderness limits (suitable for plastic design). The beams were tested under four point bending, providing a central region of constant bending moment and zero shear force. Most specimens failed by local buckling.

The major finding was that the Class 1 web slenderness limits for RHS, which were based on tests of I-section beams, are unconservative for RHS. Some sections which satisfy the current Class 1 slenderness limits of AS 4100, Eurocode 3 and AISC LRFD did not exhibit the rotation capacity suitable for plastic design.
The current design philosophy is to assign separate, independent web and flange slenderness limits for plastic design of RHS. Iso-rotation plots of the test results indicated that the rotation capacity was a function both flange and web slenderness. Hence the Class 1 (plastic) slenderness limits for RHS webs and flanges need to be related to each other. Class 1 limits which incorporate simple bi-linear interaction between flange and web slenderness have been proposed. Alternative forms of limits that account for flange-web interaction, such as multilinear or elliptical functions of flange and web slenderness, may be less conservative than the simple bi-linear limits proposed.

While cold-formed RHS tested did not satisfy the material ductility requirements specified for plastic design in current steel design standards, no RHS beam failed by fracture, suggesting that the restriction on plastic design, on the basis of insufficient material ductility, might be unnecessary.

7.4 PORTAL FRAME KNEE CONNECTION TESTS

The behaviour of various types of knee joints (L-joints) constructed from cold-formed RHS for use in portal frames was examined. In particular, the suitability of the connections to act as plastic hinges was investigated. The types of connections considered were:

- welded stiffened joints,
- welded unstiffened knee joints,
- bolted knee joints with end plates, and
- connections with a fabricated internal sleeve, either bolted or welded.

The connections were tested under both opening and closing moment, simulating the combination of bending moment, axial force, and shear force that would be present under typical loadings of a portal frame structure.

It was found that in tension (opening moment), there was fracture in the heat affected zone of the cold-formed RHS at low rotation values. In compression (closing moment), web local buckling occurred near the connection. The major finding of the experimental study was the unexpected fracture of the welded stiffened joints near or in the heat affected zone.
The stiffened and unstiffened welded connections satisfied the strength interaction requirements of the CIDECT RHS connection design guide (Packer et al 1992), yet did not prove entirely suitable for plastic hinge formation. There was variability in the rotation behaviour of the stiffened welded connection under opening moment, while under closing moments there was insufficient rotation prior to local buckling for some of the sections. It is not suggested that the CIDECT design recommendations for stiffened welded L-joints are incorrect, as they were based on tests of hot-formed RHS, but the recommendations may not be suitable for cold-formed RHS. The stiffened welded joints reached the plastic moment, while the unstiffened welded joints were not able to reach the plastic moment.

The bolted end plate connection was less stiff than the other connections, due to plate separation. It did not demonstrate sufficient rotation capacity to be considered suitable for plastic design.

The internal sleeve connections involved fabricating a thick internal fitting that was inserted into each leg of the connection. For the welded sleeve connection, the two legs of the connection were welded together after the sleeve had been inserted into each leg. For the bolted sleeve connection, four bolts went through each leg and the internal sleeve. The sleeve connection exhibited the most suitable behaviour for plastic design. The bolted sleeve connection separated at the apex as load was increased, and was fairly flexible. By comparison, the welded sleeve did not separate like the bolted sleeve and hence was stiffer. The welded sleeve connection provided sufficient rotation capacity to be considered suitable for a plastic hinge location in a plastically designed frame. The plastic hinge was moved from the apex of the joint to the end of the sleeve, due to the extra strength and stiffness provided by the internal sleeve.

7.5 TESTS OF PORTAL FRAMES

Chapter 5 described tests on three portal frames constructed from cold-formed RHS in Grade C350 and Grade C450 steel. The details of the frame, the loading methods and the results were discussed. The results indicate that a plastic collapse mechanism formed in each frame. The connections of the frame, particularly the welded internal sleeve knee joint, performed adequately.
The behaviour of the frame was modelled by several types of structural analysis. The simple plastic and plastic zone analyses predicted the locations of the plastic hinges which coincided with the places of high curvature and local buckling in the experimental frames. All analyses underestimated the magnitude of the deflections of the frame. Factors such loss of rigidity in the joints contributed to the underestimation of deflections. An advanced plastic zone structural analysis, which included second order effects, gradual yielding, multilinear stress - strain curves varying around the cross section, and the structural imperfections slightly overestimated the strength of the frame. The main cause for the over prediction of strength was the underestimation of the deflections and hence the magnitude of the second order effects. The second order plastic zone analysis ignoring frame imperfections provided the best estimates of the ultimate loads of the three frames, within 2 % of the experimental values.

The distribution of curvature throughout the portal frame was analysed. The points of high curvature required a rotation capacity of $R < 3$ to achieve a load of 99 % of the ultimate load. All but one point required a rotation capacity of $R < 4$ to achieve the ultimate load, indicating that, for the portal frames tested, a rotation capacity of $R = 4$ was adequate.

Although the cold-formed RHS do not satisfy the material ductility requirements of AS 4100 or Eurocode 3, there was no failure associated with material ductility in the three frames tested, demonstrating that the restriction on cold-formed RHS for plastic design is unwarranted provided that the connections are capable of sustaining the required plastic rotations.

7.6  FINITE ELEMENT ANALYSIS

A large number of finite element analyses of RHS beams was conducted, to simulate the bending tests of Chapter 3, using the commercial software ABAQUS. The main aim of the analysis was to establish any trends in the plastic behaviour of cold-formed RHS beams that could supplement the experimental data gathered in Chapter 3.
The following points summarise the significant findings of the analysis:

- The maximum moments predicted were slightly lower than those observed experimentally. The numerical model assumed the same material properties across the whole flange, web or corner of the RHS. In reality, the variation of material properties is gradual, with a smooth increase of yield stress from the centre of a flat face, to a maximum in the corner.

- A perfect specimen without imperfections achieved rotation capacities much higher than those observed experimentally. Introducing a bow-out imperfection, constant along the length of the beam, as was (approximately) measured experimentally, did not affect the numerical results significantly. To simulate the effect of the imperfections induced by welding the loading plates to the beams in the experiments, the amplitude of the bow-out imperfection was varied sinusoidally along the length of the beam. Choosing an imperfection half-wavelength of $d/2$, which approximately matched the size of the local buckle, and an appropriate magnitude of imperfection, predicted the rotation capacities obtained in the test program. The magnitude of the imperfection had an unexpectedly significant impact on rotation capacity, especially for stockier sections.

- When the imperfections were prescribed along the entire length of the beam, the location of local buckling varied, and often two buckles formed. To replicate the experimental observation, that the local buckle formed adjacent to the loading plate welded onto the RHS, the imperfections were limited to a small zone, adjacent to the loading plate.

- It is likely that the imperfection caused by welding the loading plates to the RHS was a major factor affecting the experimentally observed behaviour. The sinusoidally varying imperfections in the ABAQUS model simulated the effects of the localised imperfections in the actual situation. Larger imperfections were required on the more slender sections to reproduce the experimental results, since for the same type of welding, larger imperfections are induced in more slender sections. The sensitivity to imperfection size increased as the aspect ratio of the RHS decreased.

- The finite element analysis determined similar trends as observed experimentally:
  - the rotation capacity was a function of both web and flange slenderness,
  - for a given aspect ratio, the relationship between web slenderness and rotation capacity was non-linear, and the slope of the line describing the relationship increased as the web slenderness decreased.
7.7 SUGGESTIONS FOR FURTHER STUDY

The work conducted in this thesis has identified several areas where further research is required:

**Bending tests to examine Class 2 and Class 3 slenderness limits**
The results have shown that flange - web interaction should be considered in the Class 1 (plastic) slenderness limits for RHS. Further study is required to examine the Class 2 and Class 3 limits of cold-formed RHS in bending, to establish if new slenderness limits, which incorporate flange - web interaction, are required.

**Compression and bending tests**
The web slenderness limits for members in bending and compression are lower than the limits for bending alone, since a greater proportion of the web is in compression for the case of bending and compression. Given that the web limits for bending alone have been shown to be non-conservative, the web slenderness limits for RHS in bending and compression must also be examined.

**Effect of welding on cold-formed connections**
One of the most important findings of this investigation was the fracture of the welded stiffened connections at low rotation values. It is recommended that there be further examination of the ductility of welded joints in cold-formed sections to find alternative connection details, welding procedures or even steel metallurgy, that can improve the performance of such connections, and allow them to form plastic hinges.

**Alternative connections for portal frame knees**
It is suggested that there could be significant refinement of the internal sleeve connections to allow for the possibility of cheaper production of the internal sleeve joints. There is also the possibility of using other connection types which are easy to install, while avoiding problems in the heat affected zone near the weld.
Additional Studies of Portal Frames
The tests have shown that cold-formed RHS portal frames could deform plastically, form a plastic collapse mechanism, and attain the simple plastic collapse load. However, for the pin-based portal frames considered, only two plastic hinges were required to form the collapse mechanism. It would be desirable to consider different or more structurally redundant systems (such as fixed-based frames), wind uplift loading, and a complete structural system, incorporating purlins, sheeting, and bracing.

Deflection Limits for Portal Frames
The deformations, particularly the vertical deflections of the apex, exceeded the recommended deflection limits for beams in AS 4100. However, it was shown that the limits in AS 4100 were more suited for beams supporting floor systems, rather than portal frame rafters. It is recommended that serviceability limits for portal frame structures be reviewed.

Extension of the finite element analysis
While the effects of many parameters were considered, there are many areas where the finite element analysis described in this thesis can be extended to consider:
• the behaviour of Class 3 and Class 4 RHS beams,
• the effect of axial compression,
• the significance of corner radius,
• the examination of a large range of material properties, and
• the effect of imperfections induced by welding.

Finis
Chapter 8

REFERENCES


Baigent, A. H. and Hancock, G. J., (1982), “The Strength of Cold-Formed Portal Frames”, *Proceedings*, Sixth International Conference on Cold-Formed Steel Structures, pp 321 - 347, St Louis, Missouri, USA.


British Standards Institution, (1958), British Standard BS 153: Steel Girder Bridges, British Standards Institution, Great Britain.


CASE, (1992a), “Tests on Rectangular Hollow Sections to Investigate the Effect of Variation of Yield Stress Around a Section”, Investigation Report S885, Centre for Advanced Structural Engineering, School of Civil and Mining Engineering, University of Sydney, Australia.


Eurocode 3 Editorial Group, (1989), “The \( \frac{b}{t} \) Ratios Controlling the Applicability of Analysis Models in Eurocode 3”, Document 5.02, Background Documentation to Chapter 5 of Eurocode 3, Aachen University, Germany.


Kirk, P., (1986), “Design of a Cold-Formed Section Portal Frame Building System”, *Proceedings*, Eighth International Conference on Cold-Formed Steel Structures, pp 295 - 310, St Louis, Missouri, USA.


Rasmussen, K. J. R., Clarke, M. J. and Hancock, G. J., (1997), Structural Analysis 2, Department of Civil Engineering, The University of Sydney, Sydney, Australia.


School of Civil Engineering, (1975), “Test of a Rectangular Hollow Section Joint”, Test Record T242, School of Civil Engineering, The University of Sydney, Sydney, Australia.


Standards Australia, (1975), Australian Standard AS 1250 SAA Steel Structures Code, Standards Australia, Sydney, Australia.


Standards Australia (1981), Australian Standard AS 1252 High Strength Steel Bolts with Associated Steel Nuts and Washers for Structural Engineering, Standards Australia, Sydney, Australia.


Standards Australia, (1990a), Australian Standard AS 4100 Steel Structures, Standards Australia, Sydney, Australia.


Standards Australia, (1991a), Australian Standard AS 1163 Structural Steel Hollow Sections, Standards Australia, Sydney, Australia.


Tubemakers, (1994), Design Capacity Tables for DuraGal Steel Hollow Sections, Tubemakers of Australia Limited, Structural Products Division, (now known as BHP Steel Structural and Pipeline Products), Newcastle, Australia.


(151 references)

As was noted in the Preface, the following papers, based on the work presented in this thesis, have been jointly written with Professor Greg Hancock:

**Journal Papers**


Conference Papers


**Research Reports**


Reports


(20 publications by the author, in addition to the 151 references above)

Some of the above publications of the author are available in electronic format via the World Wide Web page of the Department of Civil Engineering: http://www.civil.usyd.edu.au
APPENDICES
Appendix A

PLASTIC BENDING TESTS: MOMENT - CURVATURE GRAPHS

This section contains the non-dimensional moment - curvature relationships for each plastic bending test. Since there were up to four tests performed on one particular size and stress grade of RHS, all tests for the one size RHS are shown on the one figure.
Figure A.1: Moment - Curvature Graph for $150 \times 50 \times 5.0$ C450 RHS

Figure A.2: Moment - Curvature Graph for $150 \times 50 \times 4.0$ C450 RHS

Figure A.3: Moment - Curvature Graph for $150 \times 50 \times 3.0$ C450 RHS
Figure A.4: Moment - Curvature Graph for 150 × 50 × 2.5 C450 RHS

Figure A.5: Moment - Curvature Graph for 150 × 50 × 2.3 C450 RHS

Figure A.6: Moment - Curvature Graph for 100 × 50 × 2.0 C450 RHS
Figure A.7: Moment - Curvature Graph for $75 \times 50 \times 2.0$ C450 RHS

Figure A.8: Moment - Curvature Graph for $75 \times 25 \times 2.0$ C450 RHS

Test stopped - no local buckling observed.

Figure A.9: Moment - Curvature Graph for $75 \times 25 \times 1.6$ C450 RHS
Figure A.10: Moment - Curvature Graph for 75 × 25 × 1.6 C350 RHS

Figure A.11: Moment - Curvature Graph for 150 × 50 × 3.0 C350 RHS

Figure A.12: Moment - Curvature Graph for 100 × 50 × 2.0 C350 RHS
Moment Curvature
125 x 75 x 3.0 C350 RHS

Figure A.13: Moment - Curvature Graph for 125 × 75 × 3.0 C350 RHS

Figure A.14: Moment - Curvature Graph for 125 × 75 × 2.5 C350 RHS

Figure A.15: Moment - Curvature Graph for 100 × 100 × 3.0 C450 RHS
Figure A.16: Moment - Curvature Graph for 150 × 50 × 4.0 C350 RHS

Figure A.17: Moment - Curvature Graph for 250 × 150 × 6.3 S275 RHS

Figure A.18: Moment - Curvature Graph for 200 × 150 × 6.3 S275 RHS
Appendix B

TESTS OF HOT-FORMED RECTANGULAR HOLLOW SECTIONS

B.1 INTRODUCTION

At the request of Mr Terry Giddings, British Steel Tube and Pipes, some hot-formed RHS were tested as part of the CIDECT project, to provide a comparison with the cold-formed steel. Two samples, 200 × 150 × 6.3 and 250 × 150 × 6.3, in Grade S275J0H, manufactured to EN 10210 (European Committee for Standardisation 1994), were selected. The samples were supplied by Continental Hardware, the British Steel supplier in Singapore.

The hot-formed RHS had the seam weld in one of the larger faces (the web) of the section. The faces of the hot-formed are consequently labelled “web 1”, “web 2”, “flange 1” and “flange 2”.

Grade S275J0H steel has nominal yield stress ($f_{y,n}$) of 275 MPa and nominal minimum tensile strength ($f_{u,n}$) of 410 - 560 MPa, and minimum elongation of 22% based on a gauge length of $5.65 \sqrt{S_o}$.

The typical chemical composition of the hot-formed RHS according to EN 10210 is given in Table B.1:

<table>
<thead>
<tr>
<th>Chemical Composition (Ladle Analysis), % max</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
</tr>
<tr>
<td>0.20</td>
</tr>
</tbody>
</table>

Table B.1: Typical Chemical Composition of Hot-Formed RHS
Coupons were taken from the centre of the flats and the corners of each tube. Tensile coupon tests were carried out in the same manner as for the coupons cut from the cold-formed RHS as detailed in Section 3.2.2 and Appendix C.

The hot-formed material has a distinct yield plateau and there is no significant difference in yield stress between the various faces and the corners of the RHS. The values presented in Table B.2 are the means of the values from each of the four faces on a particular RHS. The stress - strain curves are given in Appendix C.

**B.2 BENDING TEST PROCEDURE**

Bending tests were performed using the “parallel plate” method outlined in Section 3.3 and shown in Figure 3.5. Since the hot-formed sections were larger than the cold-formed sections, the length of the beam tested was also bigger. $L_1$ was 800 mm, and $L_2$ was 2500 mm, as defined in Figure 3.5.

**B.3 RESULTS**

The results of the plastic bending tests are presented in Tables B.2 and B.3. The individual moment - curvature graphs for each test are in Figure B.1. Both specimens failed by local buckling in the web. Each web buckled and compatibility of rotation at the corner caused deformation of the flange. Similarly to the cold-formed sections, the buckle formed adjacent to one of the loading plates.
Table B.2: Summary of Results of Hot-Formed Bending Tests

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$f_y$ (MPa)</th>
<th>$f_u$ (MPa)</th>
<th>$e_f$ (%)</th>
<th>$M_{pn}$ (kNm)</th>
<th>$M_p$ (kNm)</th>
<th>$M_{max}$ (kNm)</th>
<th>$M_{max}$/$M_p$</th>
<th>$R$</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH01</td>
<td>349</td>
<td>495</td>
<td>40.5</td>
<td>107.5</td>
<td>124.1</td>
<td>129.6</td>
<td>1.04</td>
<td>6.3</td>
</tr>
<tr>
<td>BH02</td>
<td>362</td>
<td>486</td>
<td>39.25</td>
<td>77.04</td>
<td>97.25</td>
<td>96.5</td>
<td>0.99</td>
<td>6.9</td>
</tr>
</tbody>
</table>

Table B.3: Summary of Results of Hot-Formed Bending Tests

Figure B.1: Moment - Curvature Behaviour of the Hot-Formed RHS
B.4 DISCUSSION

The hot-formed RHS achieved values of $M_{\text{max}}/M_p$ which were considerable lower compared to the cold-formed RHS. From Table B.2, specimen BH01 reached $M_{\text{max}}/M_p = 1.04$, and specimen BH02 achieved $M_{\text{max}}/M_p = 0.99$ (static values), while, on average, the cold-formed RHS reached $M_{\text{max}}/M_p = 1.09$ (Table 3.2). The different behaviour between the hot and cold-formed RHS can also be seen in the moment - curvature graphs in Figure B.1 and Appendix A. The hot-formed RHS have a long yield plateau in their stress - strain curves (refer to Appendix C), and do not reach strain hardening before local buckling is reached. Theoretically, a specimen with a plastic plateau (and no strain hardening) can only achieve the full plastic moment at infinite curvature. Similar behaviour of hot-formed RHS is shown in results of CIDECT Project 2P (Stranghoner et al 1995).

A slightly different definition of rotation capacity has been employed for the hot-formed sections compared to the cold-formed sections since one of the specimens failed to reach the plastic moment but still displayed a large plastic plateau and rotation capacity. It is unreasonable to reduce the rotation capacity of the hot-formed sections when one specimen have just failed to reach the plastic moment by 1 %, and has clearly demonstrated hinge behaviour. The value of $R$ is calculated at the curvature at which local buckling occurs rather than when the moment falls below $M_p$.

The results of the hot-formed tests are compared with the cold-formed tests in Figure B.2, which plots the web slenderness against the rotation capacity, and Figure B.3, the iso-rotation plot. The rotation capacities of the hot-formed sections were significantly higher than the cold-formed sections of the same slenderness. However, since only two tests were performed on the hot-formed samples, no definitive conclusions can be reached.
Figure B.2: Rotation Capacity - Web Slenderness Relationship: Comparison between Cold-Formed and Hot-Formed RHS

Figure B.3: Iso-Rotation Curve for Cold-Formed RHS: Comparison with Hot-Formed RHS
Bending tests on hot-formed and cold-formed steel were performed by Stranghoner (1995). The four point bending configuration was used a system of tensile struts and stirrups ensured the load was applied through the neutral axis, slightly different to the “parallel plate” method used in this thesis. Tests were performed predominately on SHS, and some of tests on RHS were terminated before local buckling was observed, so the rotation capacity was not measured. The results of Stranghoner are compared to the SHS results of Hasan and Hancock (1988) and Zhao and Hancock (1991) in Figure B.4.

![Figure B.4: Comparison of Hot-Formed and Cold-Formed Results of Hasan and Hancock; Zhao and Hancock; and Stranghoner](image)

For the same flange (or web) slenderness, the rotation capacity achieved in Stranghoner’s tests are lower than those of Hasan and Hancock, and Zhao and Hancock. The results of Stranghoner exhibit no significant difference in the behaviour of the cold-formed and hot-formed sections.
Stranghoner employed a different definition of $M_p$ compared to that used in this thesis. Stranghoner used a hybrid value of the plastic moment, which accounted for the different yield stresses in the flats and corners of the RHS, whereas this thesis applied the yield stress from the adjacent flat face to the entire section when calculating $M_p$. The method of Stranghoner produces a higher value of $M_p$, which in turn, gives a slightly lower value of rotation capacity. However, the aforementioned difference in $M_p$ only creates a small reduction in $R$, and cannot account for the variation in results between Stranghoner and the results in this thesis as indicated in Figure B.4.

The exact shape of the stress - strain relationships in Stranghoner’s tests was unknown. It is possible that the different properties, particularly strain-hardening, are the cause of the lesser rotation capacity. Finite element analysis (Appendix J) illustrated the importance of strain hardening to rotation capacity.
Appendix C

TENSILE COUPON TESTS AND STRESS - STRAIN CURVES

C.1 PROCEDURE

Three coupons were taken from the flats of each RHS. One was cut from the face opposite the weld, and one from each side adjacent to the weld. A coupon was also taken from each of the four corners of most RHS. Figure 2.1 shows the position of the faces with respect to the weld.

The tensile coupons were prepared and tested to Australian Standard AS 1391 (Standards Australia 1991b). Most flat coupons were instrumented with linear strain gauges. An extensometer was used to measure strain for the corner coupons and some flat coupons. The coupons were tested in a 250 kN capacity INSTRON Universal Testing Machine or a 200 kN capacity SINTECH Testing Machine with friction grips to apply the loading. A SPECTRA data acquisition system recorded the load and strain gauge readings. A constant strain rate of approximately $1.3 \times 10^{-4} \text{ s}^{-1}$ for the flats and $1.0 \times 10^{-4} \text{ s}^{-1}$ for the corners was used in the elastic range. Near the yield and ultimate stresses, the machine was halted for one minute to obtain a value of the static stress.
C.2 RESULTS

Values of the yield stress ($f_y$), ultimate stress ($f_u$) and percentage elongation after failure ($e_r$) on a gauge length of $5.65 \sqrt{S_o}$ were obtained from each test ($S_o$ is the original area of the coupon). As the yielding was gradual, the yield stress quoted is the 0.2% proof stress. The stress-strain curve was linear only for small strains. The Young’s modulus of elasticity ($E$) was calculated by averaging the slope of the curve over this initial region. An important parameter in the ductility requirements for plastic design is the ratio $f_u/f_y$, which is also tabulated. For the corner coupons it was difficult to determine a value of $E$ with great precision, so it was assumed that $E = 200$ GPa for the corners.

Results are summarised in Tables C.2, C.3 and C.4. The stresses listed are static values, while the graphs depict both static and dynamic stresses. (The static stresses were those obtained after the machine has been stopped for one minute. The dynamic stresses were recorded while the testing machine is still moving.) For low strain (typically $\leq 30000 \mu$strain) the strain was the average from the gauges (or the extensometer reading for the corner coupons). At higher strain values, the gauges failed and strain was calculated as the change in length divided by the original length (65 mm for the flat coupons and 90 mm for corners).

The “approximate” strain cannot be reliably compared to the strain gauge strain since there was some extension of the sample and the testing machine outside the assumed gauge length. However along the reasonably flat yield plateau the load was approximately constant and most extension would have occurred within the assumed gauge length. On each stress-strain curve, the approximate location of this changeover point is marked by a solid line.

The average of the yield stress values from both of the adjacent faces is used in the calculation of section capacities (plastic moment ($M_p$) and yield load ($N_y$)). The mean of the Young’s modulus of elasticity from both of the adjacent faces is used in stiffness calculations.
The yield stress of the face opposite the weld is on average 10% higher than that of the adjacent faces. The variability of yield stresses has been previously documented (CASE 1992a) and is a result of the forming process. The corners of the RHS have even higher yield stress due to the extra working at those locations but reduced ductility. The yield stress of the corners is, on average, approximately 20% higher than that of the adjacent faces.

One measure of material ductility is the ratio $e_u/e_y$, where $e_u$ is the strain at which the ultimate tensile strength ($f_u$) is reached and $e_y$ is the yield strain. Since the ultimate stress nearly always occurred after the strain gauges broke, it was not possible to obtain a precise value of $e_u$ and hence a range of ratios is reported. Using the approximate strain described above, the ratios obtained are given in Table C.1.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Range of $e_u/e_y$ values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grade C450</td>
<td>10 - 20</td>
</tr>
<tr>
<td>Flats</td>
<td>2 - 5</td>
</tr>
<tr>
<td>Corners</td>
<td></td>
</tr>
<tr>
<td>Grade C350</td>
<td>15 - 30</td>
</tr>
<tr>
<td>Flats</td>
<td>2 - 5</td>
</tr>
<tr>
<td>Corners</td>
<td></td>
</tr>
</tbody>
</table>

Table C.1: Range of $e_u/e_y$ values

Since considerable rotations can occur in plastic design, large strains occur. Certain material ductility requirements are specified for plastic design, given in Section 2.5. None of the sections tested fulfil all the material requirements of either AS 4100 or Eurocode 3. The requirement that $f_u/f_y > 1.2$ was never satisfied, while the strain after fracture, $e_f > 15 \%$, was achieved for approximately 90 \% of the coupons tested (including corners). Eurocode 3 specifies $e_u/e_y > 20$, and as mentioned above, this could not be measured precisely. The range of values obtained for this ratio indicate that this was rarely achieved for the flats of the C450 specimens, and was satisfied considerably more often for the flats of the C350 samples. No corner coupons fulfilled this requirement.
On some occasions, as can be identified from Table C.2, the average yield stress of the adjacent faces of the *DuraGal* (C450) specimens failed to reach the nominal yield stress of $f_{yn} = 450$ MPa. The yield stress from the face opposite the weld always exceeded the nominal value. Tensile coupon tests undertaken by BHP Steel normally show that all faces reach the nominal yield stress of 450 MPa, and the values are approximately 5% higher than those measured at The University of Sydney (CASE 1992b). Reasons for this variance are that BHP Steel use a higher strain rate as opposed to the “static” value used in this thesis, and BHP Steel use the stress at 0.5% total strain, as opposed to the 0.2% proof stress. This different method of calculation is within the limits defined in AS 1391.

Tests (CASE 1992b, 1992c) have indicated that the reliability of DuraGal beams and stub columns, when the strength is calculated using the “static” yield stress of the face opposite the weld, is comparable to the reliability for strength design of hot-rolled sections used in the development of AS 4100.

This thesis uses the “static” yield stress from the adjacent face, which has been shown to have the lower yield stress.

The results for specimen TS07D (cut from beam BS07) are significantly lower than the nominal values ($f_y = 345, 411, 428$ MPa for the flat faces, as opposed to $f_{yn} = 450$ MPa). Analysis of beams using the material properties of specimen TS07D gave a considerably lower bending moment than that observed experimentally. It may be possible that the tensile specimens were mislabeled during the testing process giving incorrect results. It was not feasible to test new coupons cut from that specimen.
<table>
<thead>
<tr>
<th>Specimen</th>
<th>Cut from section</th>
<th>Position</th>
<th>$E$ (GPa)</th>
<th>$f_y$ (MPa)</th>
<th>$f_u$ (MPa)</th>
<th>$f_u/f_y$ (%)</th>
<th>$e_t$</th>
<th>$E^1$ (GPa)</th>
<th>$f_y^1$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TS01D</td>
<td>150 × 50 × 5.0</td>
<td>Adj 1</td>
<td>198</td>
<td>425</td>
<td>492</td>
<td>1.16</td>
<td>18.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C450</td>
<td>Adj 2</td>
<td>198</td>
<td>457</td>
<td>498</td>
<td>1.09</td>
<td>16.2</td>
<td>198</td>
<td>441</td>
</tr>
<tr>
<td></td>
<td>Opp</td>
<td>213</td>
<td>505</td>
<td>543</td>
<td>1.07</td>
<td>15.6</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C1</td>
<td>200</td>
<td>520</td>
<td>556</td>
<td>1.07</td>
<td>20.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C2</td>
<td>200</td>
<td>480</td>
<td>566</td>
<td>1.18</td>
<td>16.7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C3</td>
<td>200</td>
<td>495</td>
<td>566</td>
<td>1.14</td>
<td>16.7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C4</td>
<td>200</td>
<td>500</td>
<td>560</td>
<td>1.12</td>
<td>18.3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TS02D</td>
<td>150 × 50 × 4.0</td>
<td>Adj 1</td>
<td>219</td>
<td>460</td>
<td>537</td>
<td>1.17</td>
<td>20.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C450</td>
<td>Adj 2</td>
<td>205</td>
<td>454</td>
<td>516</td>
<td>1.14</td>
<td>18.3</td>
<td>212</td>
<td>457</td>
</tr>
<tr>
<td></td>
<td>Opp</td>
<td>198</td>
<td>514</td>
<td>580</td>
<td>1.13</td>
<td>15.7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C1</td>
<td>200</td>
<td>563</td>
<td>616</td>
<td>1.09</td>
<td>17.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C2</td>
<td>200</td>
<td>545</td>
<td>601</td>
<td>1.10</td>
<td>16.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C3</td>
<td>200</td>
<td>550</td>
<td>608</td>
<td>1.11</td>
<td>16.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C4</td>
<td>200</td>
<td>570</td>
<td>613</td>
<td>1.08</td>
<td>18.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TS03D</td>
<td>150 × 50 × 3.0</td>
<td>Adj 1</td>
<td>210</td>
<td>445</td>
<td>520</td>
<td>1.17</td>
<td>19.9</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C450</td>
<td>Adj 2</td>
<td>201</td>
<td>442</td>
<td>506</td>
<td>1.14</td>
<td>16.6</td>
<td>206</td>
<td>444</td>
</tr>
<tr>
<td></td>
<td>Opp</td>
<td>218</td>
<td>514</td>
<td>585</td>
<td>1.14</td>
<td>13.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C1</td>
<td>200</td>
<td>540</td>
<td>588</td>
<td>1.09</td>
<td>20.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C2</td>
<td>200</td>
<td>545</td>
<td>594</td>
<td>1.09</td>
<td>20.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C3</td>
<td>200</td>
<td>540</td>
<td>611</td>
<td>1.13</td>
<td>20.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TS04D</td>
<td>150 × 50 × 2.5</td>
<td>Adj 1</td>
<td>202</td>
<td>445</td>
<td>527</td>
<td>1.18</td>
<td>15.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C450</td>
<td>Adj 2</td>
<td>206</td>
<td>446</td>
<td>518</td>
<td>1.16</td>
<td>16.3</td>
<td>204</td>
<td>446</td>
</tr>
<tr>
<td></td>
<td>Opp</td>
<td>204</td>
<td>485</td>
<td>560</td>
<td>1.15</td>
<td>13.1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C1</td>
<td>200</td>
<td>540</td>
<td>600</td>
<td>1.11</td>
<td>25.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C2</td>
<td>200</td>
<td>530</td>
<td>585</td>
<td>1.10</td>
<td>20.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C3</td>
<td>200</td>
<td>545</td>
<td>596</td>
<td>1.09</td>
<td>20.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C4</td>
<td>200</td>
<td>540</td>
<td>588</td>
<td>1.09</td>
<td>17.5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TS05D</td>
<td>150 × 50 × 2.3</td>
<td>Adj 1</td>
<td>207</td>
<td>453</td>
<td>528</td>
<td>1.17</td>
<td>16.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C450</td>
<td>Adj 2</td>
<td>204</td>
<td>434</td>
<td>508</td>
<td>1.17</td>
<td>18.3</td>
<td>205</td>
<td>444</td>
</tr>
<tr>
<td></td>
<td>Opp</td>
<td>191</td>
<td>480</td>
<td>547</td>
<td>1.14</td>
<td>14.5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C1</td>
<td>200</td>
<td>535</td>
<td>585</td>
<td>1.09</td>
<td>17.5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C2</td>
<td>200</td>
<td>470</td>
<td>523</td>
<td>1.11</td>
<td>17.5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C3</td>
<td>200</td>
<td>490</td>
<td>526</td>
<td>1.07</td>
<td>21.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C4</td>
<td>200</td>
<td>480</td>
<td>530</td>
<td>1.10</td>
<td>18.5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TS06D</td>
<td>100 × 50 × 2.0</td>
<td>Adj 1</td>
<td>194</td>
<td>445</td>
<td>492</td>
<td>1.11</td>
<td>12.9</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C450</td>
<td>Adj 2</td>
<td>209</td>
<td>452</td>
<td>505</td>
<td>1.12</td>
<td>10.9</td>
<td>202</td>
<td>449</td>
</tr>
<tr>
<td></td>
<td>Opp</td>
<td>182</td>
<td>480</td>
<td>540</td>
<td>1.13</td>
<td>9.17</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C1</td>
<td>200</td>
<td>490</td>
<td>538</td>
<td>1.10</td>
<td>17.5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C2</td>
<td>200</td>
<td>490</td>
<td>540</td>
<td>1.10</td>
<td>15.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C3</td>
<td>200</td>
<td>510</td>
<td>560</td>
<td>1.10</td>
<td>20.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C4</td>
<td>200</td>
<td>500</td>
<td>547</td>
<td>1.09</td>
<td>20.0</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table C.2: Summary of Tensile Test Results for Bending Test Specimens
<table>
<thead>
<tr>
<th>Specimen</th>
<th>Cut from section</th>
<th>Position</th>
<th>E (GPa)</th>
<th>$f_y$ (MPa)</th>
<th>$f_u$ (MPa)</th>
<th>$f_u/f_y$ (%)</th>
<th>$E^1$ (GPa)</th>
<th>$f_y^1$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TS07D</td>
<td>75 × 50 × 2.0</td>
<td>Adj 1</td>
<td>199</td>
<td>345</td>
<td>430</td>
<td>1.25</td>
<td>28.3</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Adj 2</td>
<td>208</td>
<td>411</td>
<td>484</td>
<td>1.18</td>
<td>12.5</td>
<td>203</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Opp</td>
<td>196</td>
<td>428</td>
<td>482</td>
<td>1.13</td>
<td>23.3</td>
<td>411</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C1</td>
<td>200</td>
<td>475</td>
<td>520</td>
<td>1.09</td>
<td>18.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>C2</td>
<td>200</td>
<td>450</td>
<td>500</td>
<td>1.11</td>
<td>17.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>C3</td>
<td>200</td>
<td>445</td>
<td>480</td>
<td>1.08</td>
<td>19.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>C4</td>
<td>200</td>
<td>465</td>
<td>503</td>
<td>1.08</td>
<td>20.0</td>
<td></td>
</tr>
<tr>
<td>TS08D</td>
<td>75 × 25 × 2.0</td>
<td>Adj 1</td>
<td>202</td>
<td>438</td>
<td>507</td>
<td>1.16</td>
<td>14.2</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Adj 2</td>
<td>204</td>
<td>475</td>
<td>522</td>
<td>1.10</td>
<td>11.0</td>
<td>203</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Opp</td>
<td>204</td>
<td>487</td>
<td>537</td>
<td>1.10</td>
<td>26.0</td>
<td>457</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C1</td>
<td>200</td>
<td>480</td>
<td>538</td>
<td>1.12</td>
<td>19.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>C2</td>
<td>200</td>
<td>550</td>
<td>588</td>
<td>1.07</td>
<td>15.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>C3</td>
<td>200</td>
<td>530</td>
<td>576</td>
<td>1.09</td>
<td>18.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>C4</td>
<td>200</td>
<td>525</td>
<td>585</td>
<td>1.11</td>
<td>15.0</td>
<td></td>
</tr>
<tr>
<td>TS09D</td>
<td>75 × 25 × 1.6</td>
<td>Adj 1</td>
<td>194</td>
<td>450</td>
<td>506</td>
<td>1.12</td>
<td>26.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Adj 2</td>
<td>193</td>
<td>428</td>
<td>515</td>
<td>1.20</td>
<td>12.5</td>
<td>193</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Opp</td>
<td>200</td>
<td>487</td>
<td>545</td>
<td>1.12</td>
<td>14.0</td>
<td>439</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C1</td>
<td>200</td>
<td>560</td>
<td>589</td>
<td>1.05</td>
<td>16.7</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>C2</td>
<td>200</td>
<td>555</td>
<td>576</td>
<td>1.04</td>
<td>14.7</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>C3</td>
<td>200</td>
<td>555</td>
<td>605</td>
<td>1.09</td>
<td>14.7</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>C4</td>
<td>200</td>
<td>565</td>
<td>585</td>
<td>1.04</td>
<td>16.7</td>
<td></td>
</tr>
<tr>
<td>TS10D</td>
<td>75 × 25 × 1.6</td>
<td>Adj 1</td>
<td>205</td>
<td>421</td>
<td>460</td>
<td>1.09</td>
<td>16.1</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Adj 2</td>
<td>205</td>
<td>423</td>
<td>452</td>
<td>1.07</td>
<td>17.2</td>
<td>202</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Opp</td>
<td>206</td>
<td>445</td>
<td>514</td>
<td>1.16</td>
<td>6.25</td>
<td>422</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C1</td>
<td>200</td>
<td>505</td>
<td>540</td>
<td>1.07</td>
<td>16.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>C2</td>
<td>200</td>
<td>500</td>
<td>515</td>
<td>1.03</td>
<td>16.7</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>C3</td>
<td>200</td>
<td>510</td>
<td>532</td>
<td>1.04</td>
<td>17.3</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>C4</td>
<td>200</td>
<td>510</td>
<td>555</td>
<td>1.09</td>
<td>15.3</td>
<td></td>
</tr>
<tr>
<td>TS11D</td>
<td>150 × 50 × 3.0</td>
<td>Adj 1</td>
<td>207</td>
<td>370</td>
<td>433</td>
<td>1.17</td>
<td>32.6</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Adj 2</td>
<td>202</td>
<td>369</td>
<td>425</td>
<td>1.15</td>
<td>27.9</td>
<td>205</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Opp</td>
<td>198</td>
<td>380</td>
<td>436</td>
<td>1.15</td>
<td>22.2</td>
<td>370</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C1</td>
<td>200</td>
<td>465</td>
<td>511</td>
<td>1.10</td>
<td>20.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>C2</td>
<td>200</td>
<td>475</td>
<td>516</td>
<td>1.09</td>
<td>22.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>C3</td>
<td>200</td>
<td>480</td>
<td>534</td>
<td>1.11</td>
<td>21.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>C4</td>
<td>200</td>
<td>495</td>
<td>542</td>
<td>1.09</td>
<td>20.5</td>
<td></td>
</tr>
<tr>
<td>TS12D</td>
<td>100 × 50 × 2.0</td>
<td>Adj 1</td>
<td>205</td>
<td>395</td>
<td>454</td>
<td>1.15</td>
<td>21.1</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Adj 2</td>
<td>200</td>
<td>404</td>
<td>445</td>
<td>1.10</td>
<td>19.0</td>
<td>203</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Opp</td>
<td>199</td>
<td>433</td>
<td>471</td>
<td>1.08</td>
<td>12.0</td>
<td>400</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C1</td>
<td>200</td>
<td>500</td>
<td>536</td>
<td>1.07</td>
<td>21.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>C2</td>
<td>200</td>
<td>485</td>
<td>530</td>
<td>1.09</td>
<td>21.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>C3</td>
<td>200</td>
<td>480</td>
<td>520</td>
<td>1.08</td>
<td>20.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>C4</td>
<td>200</td>
<td>505</td>
<td>545</td>
<td>1.08</td>
<td>22.5</td>
<td></td>
</tr>
</tbody>
</table>

Table C.2 (continued): Summary of Tensile Test Results for Bending Test Specimens
<table>
<thead>
<tr>
<th>Specimen</th>
<th>Cut from section</th>
<th>Position</th>
<th>$E$ (GPa)</th>
<th>$f_y$ (MPa)</th>
<th>$f_u$ (MPa)</th>
<th>$f_{yu}$ (MPa)</th>
<th>$e_t$ (%)</th>
<th>$E^1$ (GPa)</th>
<th>$f_{y^1}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TS13D</td>
<td>125 × 75 × 3.0 C350</td>
<td>Adj 1</td>
<td>211</td>
<td>396</td>
<td>450</td>
<td>1.14</td>
<td>1.25</td>
<td>25.0</td>
<td>211</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Adj 2</td>
<td>210</td>
<td>397</td>
<td>448</td>
<td>1.13</td>
<td>1.28</td>
<td>28.4</td>
<td>211</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Opp</td>
<td>203</td>
<td>403</td>
<td>453</td>
<td>1.12</td>
<td>1.29</td>
<td>20.1</td>
<td>211</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C1</td>
<td>200</td>
<td>470</td>
<td>510</td>
<td>1.09</td>
<td>1.30</td>
<td>23.0</td>
<td>211</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C2</td>
<td>200</td>
<td>475</td>
<td>518</td>
<td>1.09</td>
<td>1.30</td>
<td>22.5</td>
<td>211</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C3</td>
<td>200</td>
<td>475</td>
<td>508</td>
<td>1.07</td>
<td>1.30</td>
<td>24.0</td>
<td>211</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C4</td>
<td>200</td>
<td>470</td>
<td>520</td>
<td>1.11</td>
<td>1.30</td>
<td>22.5</td>
<td>211</td>
</tr>
<tr>
<td>TS16D</td>
<td>150 × 50 × 2.5 C450</td>
<td>Adj 1</td>
<td>204</td>
<td>425</td>
<td>498</td>
<td>1.17</td>
<td>1.25</td>
<td>25.0</td>
<td>207</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Adj 2</td>
<td>209</td>
<td>455</td>
<td>515</td>
<td>1.13</td>
<td>1.30</td>
<td>28.3</td>
<td>207</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Opp</td>
<td>200</td>
<td>507</td>
<td>560</td>
<td>1.10</td>
<td>1.30</td>
<td>20.0</td>
<td>207</td>
</tr>
<tr>
<td>TS17D</td>
<td>100 × 50 × 2.0 C450</td>
<td>Adj 1</td>
<td>203</td>
<td>415</td>
<td>475</td>
<td>1.14</td>
<td>1.25</td>
<td>28.3</td>
<td>203</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Adj 2</td>
<td>203</td>
<td>432</td>
<td>482</td>
<td>1.17</td>
<td>1.30</td>
<td>23.3</td>
<td>203</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Opp</td>
<td>206</td>
<td>462</td>
<td>516</td>
<td>1.12</td>
<td>1.30</td>
<td>18.3</td>
<td>203</td>
</tr>
<tr>
<td>TS19D</td>
<td>100 × 100 × 3.0 C450</td>
<td>Adj 1</td>
<td>212</td>
<td>433</td>
<td>494</td>
<td>1.14</td>
<td>1.25</td>
<td>24.3</td>
<td>207</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Adj 2</td>
<td>202</td>
<td>457</td>
<td>510</td>
<td>1.12</td>
<td>1.30</td>
<td>24.3</td>
<td>207</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Opp</td>
<td>199</td>
<td>482</td>
<td>525</td>
<td>1.09</td>
<td>1.30</td>
<td>21.4</td>
<td>207</td>
</tr>
<tr>
<td>TS20D</td>
<td>150 × 50 × 3.0 C350</td>
<td>Adj 1</td>
<td>200</td>
<td>380</td>
<td>425</td>
<td>1.12</td>
<td>1.25</td>
<td>30.0</td>
<td>200</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Adj 2</td>
<td>200</td>
<td>383</td>
<td>435</td>
<td>1.14</td>
<td>1.30</td>
<td>31.4</td>
<td>200</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Opp</td>
<td>206</td>
<td>403</td>
<td>455</td>
<td>1.13</td>
<td>1.30</td>
<td>27.1</td>
<td>200</td>
</tr>
<tr>
<td>TS21D</td>
<td>125 × 75 × 2.5 C350</td>
<td>Adj 1</td>
<td>204</td>
<td>377</td>
<td>443</td>
<td>1.18</td>
<td>1.25</td>
<td>37.3</td>
<td>201</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Adj 2</td>
<td>199</td>
<td>370</td>
<td>438</td>
<td>1.18</td>
<td>1.25</td>
<td>31.7</td>
<td>201</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Opp</td>
<td>203</td>
<td>420</td>
<td>453</td>
<td>1.08</td>
<td>1.25</td>
<td>30.0</td>
<td>201</td>
</tr>
<tr>
<td>HF01</td>
<td>250 × 150 × 6.3 S275</td>
<td>Web 1</td>
<td>210</td>
<td>355</td>
<td>498</td>
<td>1.40</td>
<td>1.25</td>
<td>37.0</td>
<td>349</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Web 2</td>
<td>209</td>
<td>348</td>
<td>500</td>
<td>1.44</td>
<td>1.25</td>
<td>41.0</td>
<td>349</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Fl 1</td>
<td>201</td>
<td>345</td>
<td>496</td>
<td>1.44</td>
<td>1.25</td>
<td>41.0</td>
<td>349</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Fl 2</td>
<td>209</td>
<td>350</td>
<td>486</td>
<td>1.39</td>
<td>1.25</td>
<td>43.0</td>
<td>349</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C 1</td>
<td>201</td>
<td>380</td>
<td>476</td>
<td>1.25</td>
<td>1.25</td>
<td>43.0</td>
<td>349</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C 2</td>
<td>198</td>
<td>362</td>
<td>466</td>
<td>1.29</td>
<td>1.25</td>
<td>37.5</td>
<td>349</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C 3</td>
<td>204</td>
<td>381</td>
<td>487</td>
<td>1.28</td>
<td>1.25</td>
<td>43.8</td>
<td>349</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C 4</td>
<td>209</td>
<td>378</td>
<td>467</td>
<td>1.24</td>
<td>1.25</td>
<td>38.8</td>
<td>349</td>
</tr>
<tr>
<td>HF02</td>
<td>200 × 150 × 6.3 S275</td>
<td>Web 1</td>
<td>208</td>
<td>369</td>
<td>485</td>
<td>1.31</td>
<td>1.25</td>
<td>41.0</td>
<td>362</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Web 2</td>
<td>199</td>
<td>370</td>
<td>489</td>
<td>1.32</td>
<td>1.25</td>
<td>37.0</td>
<td>362</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Fl 1</td>
<td>205</td>
<td>350</td>
<td>485</td>
<td>1.39</td>
<td>1.25</td>
<td>40.0</td>
<td>362</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Fl 2</td>
<td>201</td>
<td>362</td>
<td>487</td>
<td>1.35</td>
<td>1.25</td>
<td>39.0</td>
<td>362</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C 1</td>
<td>209</td>
<td>387</td>
<td>484</td>
<td>1.25</td>
<td>1.25</td>
<td>33.8</td>
<td>362</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C 2</td>
<td>199</td>
<td>387</td>
<td>486</td>
<td>1.26</td>
<td>1.25</td>
<td>35.0</td>
<td>362</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C 3</td>
<td>210</td>
<td>495</td>
<td>474</td>
<td>0.96</td>
<td>1.25</td>
<td>38.8</td>
<td>362</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C 4</td>
<td>210</td>
<td>370</td>
<td>476</td>
<td>1.29</td>
<td>1.25</td>
<td>37.5</td>
<td>362</td>
</tr>
</tbody>
</table>

**Notes:**
1. The average of $E$ and $f_y$ for the two adjacent sides is used for the whole RHS.
2. For corner coupons it is assumed that $E = 200$ GPa.
3. Specimen TS03DC1 not tested.
4. For TS07D $f_y$ is from face Adjacent 2 only.
5. Nominal strengths: Grade C450 specimens $f_{yn} = 450$ MPa and $f_{un} = 500$ MPa.
   Grade C350 specimens $f_{yn} = 350$ MPa and $f_{un} = 430$ MPa.

Table C.2 (continued): Summary of Tensile Test Results for Bending Test Specimens
<table>
<thead>
<tr>
<th>Specimen</th>
<th>Cut from</th>
<th>Position</th>
<th>$E$ (GPa)</th>
<th>$f_y$ (MPa)</th>
<th>$f_u$ (MPa)</th>
<th>$f_y^1$ (MPa)</th>
<th>$e_t$ (%)</th>
<th>$E^1$ (GPa)</th>
<th>$f_y^1$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>KJ01,KJ02</td>
<td>Web 1</td>
<td>218</td>
<td>449</td>
<td>462</td>
<td>1.03</td>
<td>26.3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>KJ03,KJ04</td>
<td>Web 2</td>
<td>218</td>
<td>422</td>
<td>462</td>
<td>1.09</td>
<td>23.0</td>
<td></td>
<td>218</td>
<td>436</td>
</tr>
<tr>
<td>KJ05,KJ06</td>
<td>Fl 1</td>
<td>180</td>
<td>448</td>
<td>496</td>
<td>1.11</td>
<td>21.6</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>KJ17,KJ18</td>
<td>Fl 2</td>
<td>195</td>
<td>462</td>
<td>508</td>
<td>1.10</td>
<td>18.8</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>KJ11</td>
<td>C1</td>
<td>200²</td>
<td>580</td>
<td>607</td>
<td>1.05</td>
<td>18.8</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SJ01,SJ02</td>
<td>C2</td>
<td>200²</td>
<td>510</td>
<td>565</td>
<td>1.11</td>
<td>18.6</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C3</td>
<td>200²</td>
<td>520</td>
<td>555</td>
<td>1.07</td>
<td>18.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C4</td>
<td>200²</td>
<td>495</td>
<td>545</td>
<td>1.10</td>
<td>22.5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>KJ07,KJ08</td>
<td>Adj 1</td>
<td>204</td>
<td>350</td>
<td>436</td>
<td>1.25</td>
<td>36.3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>KJ12,SJ03</td>
<td>C350</td>
<td>Adj 2</td>
<td>204</td>
<td>348</td>
<td>438</td>
<td>1.26</td>
<td>36.3</td>
<td>204</td>
<td>349</td>
</tr>
<tr>
<td>BJ07</td>
<td>Opp</td>
<td>185</td>
<td>367</td>
<td>433</td>
<td>1.18</td>
<td>43.7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>KJ09,KJ10</td>
<td>Adj 1</td>
<td>199</td>
<td>350</td>
<td>420</td>
<td>1.20</td>
<td>40.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Adj 2</td>
<td>195</td>
<td>345</td>
<td>417</td>
<td>1.21</td>
<td>36.6</td>
<td></td>
<td>199</td>
<td>348</td>
</tr>
<tr>
<td></td>
<td>Opp</td>
<td>199</td>
<td>365</td>
<td>427</td>
<td>1.17</td>
<td>32.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>KJ13,KJ14</td>
<td>Adj 1</td>
<td>209</td>
<td>430</td>
<td>501</td>
<td>1.17</td>
<td>17.5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>KJ15,KJ16</td>
<td>Adj 2</td>
<td>211</td>
<td>428</td>
<td>506</td>
<td>1.18</td>
<td>27.5</td>
<td></td>
<td>210</td>
<td>429</td>
</tr>
<tr>
<td></td>
<td>Opp</td>
<td>212</td>
<td>485</td>
<td>555</td>
<td>1.14</td>
<td>22.5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>KJ17,KJ18</td>
<td>Adj 1</td>
<td>212</td>
<td>443</td>
<td>497</td>
<td>1.11</td>
<td>22.5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Adj 2</td>
<td>206</td>
<td>425</td>
<td>486</td>
<td>1.14</td>
<td>23.7</td>
<td></td>
<td>209</td>
<td>436</td>
</tr>
<tr>
<td></td>
<td>Opp</td>
<td>206</td>
<td>460</td>
<td>523</td>
<td>1.14</td>
<td>22.5</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notes: (1) The average of $E$ and $f_y$ for the two adjacent sides is used for the whole RHS.
(2) For corner coupons it is assumed that $E = 200$ GPa.
(3) Nominal strengths: Grade C450 specimens $f_{yn} = 450$ MPa and $f_{un} = 500$ MPa.
Grade C350 specimens $f_{yn} = 350$ MPa and $f_{un} = 430$ MPa.

Table C.3: Summary of Tensile Test Results for Connection Test Specimens
<table>
<thead>
<tr>
<th>Specimen</th>
<th>Cut from section</th>
<th>Position</th>
<th>$E$ (GPa)</th>
<th>$f_y$ (MPa)</th>
<th>$f_u$ (MPa)</th>
<th>$f_u / f_y$ (%)</th>
<th>$E^1$ (GPa)</th>
<th>$f_{y1}^1$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frame 1</td>
<td>150 × 50 × 4.0</td>
<td>Adj 1</td>
<td>207</td>
<td>409</td>
<td>460</td>
<td>1.12</td>
<td>38.8</td>
<td>207</td>
</tr>
<tr>
<td></td>
<td>C350</td>
<td>Adj 2</td>
<td>208</td>
<td>412</td>
<td>468</td>
<td>1.14</td>
<td>35.0</td>
<td>207</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Opp</td>
<td>207</td>
<td>425</td>
<td>474</td>
<td>1.12</td>
<td>27.5</td>
<td>207</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C 1</td>
<td>201</td>
<td>560</td>
<td>590</td>
<td>1.05</td>
<td>18.3</td>
<td>207</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C 2</td>
<td>200</td>
<td>555</td>
<td>591</td>
<td>1.06</td>
<td>16.7</td>
<td>BF01</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C 3</td>
<td>200</td>
<td>565</td>
<td>600</td>
<td>1.06</td>
<td>13.3</td>
<td>BF01</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C 4</td>
<td>200</td>
<td>560</td>
<td>598</td>
<td>1.07</td>
<td>15.0</td>
<td></td>
</tr>
<tr>
<td>Frame 2</td>
<td>150 × 50 × 4.0</td>
<td>Adj 1</td>
<td>208</td>
<td>455</td>
<td>530</td>
<td>1.16</td>
<td>30.0</td>
<td>207</td>
</tr>
<tr>
<td></td>
<td>C450</td>
<td>Adj 2</td>
<td>205</td>
<td>420</td>
<td>510</td>
<td>1.21</td>
<td>32.9</td>
<td>207</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Opp</td>
<td>204</td>
<td>455</td>
<td>520</td>
<td>1.14</td>
<td>48.8</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>C 1</td>
<td>198</td>
<td>565</td>
<td>612</td>
<td>1.08</td>
<td>13.3</td>
<td>BF02</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C 2</td>
<td>205</td>
<td>585</td>
<td>630</td>
<td>1.08</td>
<td>15.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>C 3</td>
<td>208</td>
<td>585</td>
<td>630</td>
<td>1.08</td>
<td>15.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>C 4</td>
<td>204</td>
<td>565</td>
<td>620</td>
<td>1.10</td>
<td>18.3</td>
<td></td>
</tr>
</tbody>
</table>

**Notes:**
1. The average of $E$ and $f_y$ for the two adjacent sides is used for the whole RHS.
2. Nominal strengths:
   - Grade C450 specimens $f_{yn} = 450$ MPa and $f_{un} = 500$ MPa.
   - Grade C350 specimens $f_{yn} = 350$ MPa and $f_{un} = 430$ MPa.

Table C.3: Summary of Tensile Test Results for Portal Frame Test Specimens
Figure C.1: Stress - Strain Curve for Specimen TS01D

Figure C.2: Stress - Strain Curve for Specimen TS02D

Figure C.3: Stress - Strain Curve for Specimen TS03D
Figure C.4: Stress - Strain Curve for Specimen TS04D

Figure C.5: Stress - Strain Curve for Specimen TS05D

Figure C.6: Stress - Strain Curve for Specimen TS06D
Stress Strain Curve
TS07D 75 x 50 x 2.0 C450

Figure C.7: Stress - Strain Curve for Specimen TS07D

Stress Strain Curve
TS08D 75 x 25 x 2.0 C450

Figure C.8: Stress - Strain Curve for Specimen TS08D

Stress Strain Curve
TS09D 75 x 25 x 1.6 C450

Figure C.9: Stress - Strain Curve for Specimen TS09D
Figure C.10: Stress - Strain Curve for Specimen TS10D

Figure C.11: Stress - Strain Curve for Specimen TS11D

Figure C.12: Stress - Strain Curve for Specimen TS12D
Figure C.13: Stress - Strain Curve for Specimen TS13D

Figure C.14: Stress - Strain Curve for Specimen TS16D

Figure C.15: Stress - Strain Curve for Specimen TS17D
Figure C.16: Stress - Strain Curve for Specimen TS19D

Figure C.17: Stress - Strain Curve for Specimen TS20D

Figure C.18: Stress - Strain Curve for Specimen TS21D
Figure C.19: Stress - Strain Curve for Specimen TH01

Figure C.20: Stress - Strain Curve for Specimen TH02

Figure C.21: Stress - Strain Curve for Specimen KJ01
Figure C.22: Stress - Strain Curve for Specimen KJ07

Figure C.23: Stress - Strain Curve for Specimen KJ09

Figure C.24: Stress - Strain Curve for Specimen KJ13
Figure C.25: Stress - Strain Curve for Specimen KJ17

Figure C.26: Stress - Strain Curve for Steel Plate

Figure C.27: Stress - Strain Curve for Frame 1
Stress Strain Curve
Frame 2 & Frame 3 150 x 50 x 4.0 C450

Figure C.28: Stress - Strain Curve for Frames 2 and 3
Appendix D

FULL SECTION TENSILE TESTS

D.1 INTRODUCTION

The yield stress \( f_y \) of a steel member is a fundamental parameter in strength formulae. A hot-rolled steel section normally has a clearly defined yield stress which can be observed in tensile tests of coupons cut from the section. Cold-formed hollow sections are formed by forming flat steel strip into a circle and welding the edges together. The rectangular profile is then created from the circular shape by pushing in the circle section. The cold work performed on the section increases the yield stress of the section above that of the original strip. Considerable cold-forming work is done in the corners of the RHS. More work is done on the flat face of the RHS opposite the weld, compared to the two flat faces adjacent to the weld. The different amounts of cold work around the section produce considerable variability of yield stress around the section. The variability of yield stress and tensile strength \( f_y \) has been observed in many test series on cold-formed RHS (CASE1992a). There is even variability of mechanical properties across each flat face of an RHS (Stranghonner et al 1995).

In-line galvanising is a yield enhancing technique used by cold-formed steel manufacturers. In-line galvanising not only provides corrosion protection but increases the strength as well. For example, BHP Structural and Pipeline Products produce cold-formed RHS from strip with a nominal yield stress of 300 MPa (Tubemakers 1994). After cold-forming, the RHS have a nominal yield stress of 350 MPa (the Grade C350 specimens). The in-line galvanised RHS, known as DuraGal, have a nominal yield stress of 450 MPa and are Grade C450.

The method of determining the yield stress of cold-formed sections varies amongst various international specifications. AS 1163 permits the use of tension tests on coupons cut from the flats of the RHS for the determination of the yield stress of a section. ASTM A 500 (ASTM 1993) also allows coupon tests on the flat faces of a cold-formed RHS. Eurocode 3 states that the average yield stress \( f_{yu} \) of a cold-formed hollow section can be determined from a full size section tensile test or Equation D.1 below.
\[ f_{ya} = f_{yb} + \frac{knt^2}{A_g} (f_{ub} - f_{yb}) \]  
(D.1: Figure 5.5.2 of Eurocode 3)

where
- \( f_{yb} \) = yield stress of the original strip
- \( f_{ub} \) = ultimate tensile strength of the original strip
- \( t \) = thickness of the section
- \( A_g \) = gross area of the cross section
- \( k \) = 7 for cold rolling
  = 5 for other methods of forming
- \( n \) = number of 90° bends in the section of internal radius \( \leq 5t \)

This section describes full section tension tests on cold-formed RHS. The results are compared to the yield stresses obtained from coupons cut from the flat faces and corners of the same specimens, and Equation D.1 above.

### D.2 TEST PROGRAM

#### D.2.1 TEST SPECIMENS

Two cold-formed RHS, manufactured by Tubemakers of Australia Limited (BHP Structural and Pipeline Products), were chosen for the full section tensile tests. Both sections were 150 \( \times \) 50 \( \times \) 3.0 RHS, in either Grade C350 or Grade C450 DuraGal. Section 3.2 contains information on the properties of these specimens. The sections were labelled as shown in Table D.1.

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Section ( d \times b \times t )</th>
</tr>
</thead>
<tbody>
<tr>
<td>FS03D</td>
<td>150 ( \times ) 50 ( \times ) 3.0 C450</td>
</tr>
<tr>
<td>FS11A</td>
<td>150 ( \times ) 50 ( \times ) 3.0 C350</td>
</tr>
</tbody>
</table>

Table D.1: Section Identification

344
D.2.2 TENSILE COUPON TESTS

Three coupons were taken from the flats of each tube. One was cut from the face opposite the weld, and one from each of the sides adjacent to the weld. The flat coupons were cut from the centre of the relevant face. Corner coupons were cut from all four corners. Appendix C contains information on the procedure of tensile coupon testing. The results are summarised in Table D.2.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Cut from section</th>
<th>Position</th>
<th>$E$ (GPa)</th>
<th>$f_y$ (MPa)</th>
<th>$f_u$ (MPa)</th>
<th>$\frac{f_u}{f_y}$</th>
<th>$e_t$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TS03D</td>
<td>150 × 50 × 3.0 C450</td>
<td>Adj 1</td>
<td>210</td>
<td>445</td>
<td>520</td>
<td>1.17</td>
<td>19.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Adj 2</td>
<td>201</td>
<td>442</td>
<td>506</td>
<td>1.14</td>
<td>16.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Opp</td>
<td>218</td>
<td>514</td>
<td>585</td>
<td>1.14</td>
<td>13.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C1$^2$</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>C2</td>
<td>200$^1$</td>
<td>540</td>
<td>588</td>
<td>1.09</td>
<td>20.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C3</td>
<td>200$^1$</td>
<td>545</td>
<td>594</td>
<td>1.09</td>
<td>20.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C4</td>
<td>200$^1$</td>
<td>540</td>
<td>611</td>
<td>1.13</td>
<td>20.0</td>
</tr>
<tr>
<td>TS11D</td>
<td>150 × 50 × 3.0 C350</td>
<td>Adj 1</td>
<td>207</td>
<td>370</td>
<td>433</td>
<td>1.17</td>
<td>32.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Adj 2</td>
<td>202</td>
<td>369</td>
<td>425</td>
<td>1.15</td>
<td>27.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Opp</td>
<td>198</td>
<td>380</td>
<td>436</td>
<td>1.15</td>
<td>22.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C1</td>
<td>200$^1$</td>
<td>465</td>
<td>511</td>
<td>1.10</td>
<td>20.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C2</td>
<td>200$^1$</td>
<td>475</td>
<td>516</td>
<td>1.09</td>
<td>22.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C3</td>
<td>200$^1$</td>
<td>480</td>
<td>534</td>
<td>1.11</td>
<td>21.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>C4</td>
<td>200$^1$</td>
<td>495</td>
<td>542</td>
<td>1.09</td>
<td>20.5</td>
</tr>
</tbody>
</table>

Notes: (1) For corner coupons it is assumed that $E = 200$ GPa.
(2) Coupon TS03DC1 not tested.
(3) Nominal strengths: Grade C450 specimens $f_{yn} = 450$ MPa and $f_{un} = 500$ MPa.
Grade C350 specimens $f_{yn} = 350$ MPa and $f_{un} = 430$ MPa.

Table D.2: Summary of Tensile Test Results
D.2.3 FULL SECTION TENSILE TEST PROCEDURE

The full section tensile tests were performed to the requirements of ASTM A 370 (ASTM 1989). The tests were carried out in a 2000 kN capacity DARTEC testing machine using a servo controlled hydraulic ram. A diagram of the test set-up is shown in Figure D.1. Snug-fitting solid steel plugs were inserted into each end of the RHS. The plugs ensured that the hydraulic grips of the DARTEC machine could clamp the specimen without crushing. The plugs extended into the RHS for a length greater than that of the grips, but not into the part of the RHS on which elongation was measured. The relevant dimensions are also shown in Figure D.1. The gauge length over which elongation was measured was 200 mm. A strain rate of $1.0 \times 10^{-4} \text{ s}^{-1}$ was used from zero load until yield. A plastic strain rate of $5.0 \times 10^{-4} \text{ s}^{-1}$ was then employed until failure.

Longitudinal strain gauges were placed on each of the wider faces (the webs) of each RHS. Fine scribe marks were made at 10 mm intervals on the specimens to determine the percentage elongation over the gauge length. Load, extension, and strain gauge readings were recorded by a SPECTRA data acquisition system.

![Figure D.1: Schematic Detail of Full Section Tensile Test](image)
D.2.4 FULL SECTION TENSILE TEST RESULTS

The RHS were tested until failure. Specimen FS03D (150 × 50 × 3.0 C450) failed by fracture initiated in both of the adjacent faces (the webs). Fracture was sudden. The line of fracture across the web was at an angle of 45° to the direction of loading. The fracture in specimen FS11A (150 × 50 × 3.0 C350) initiated in a corner and spread across the flange of the RHS. The failure surface moved slowly across the webs until complete separation occurred. There was a “V” shaped line of fracture across the web. Both specimens failed at the approximate mid-length of the sample.

Values of the yield stress, ultimate strength and percentage elongation after failure (ε) on various gauge lengths were obtained from each RHS. As the yielding was gradual, the yield stress quoted is the 0.2% proof stress. The stress-strain curve was linear only for small strains. The Young’s modulus of elasticity (E) was calculated by averaging the slope of the curve over this initial region. The specimen dimensions are listed in Table D.3, and the results are given in Table D.4. Note that all stresses quoted are static values. Static values are obtained by stopping the test machine for about one minute near the yield and ultimate loads. These values are often considerably lower than dynamic values (measured when the test machine is moving).

The stress-strain curve for each full section tensile test is shown in Figures D.2 and D.3. The curves from the individual coupons cut from the faces and the corners are included in Figures D.2 and D.3.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Cut from section</th>
<th>d (mm)</th>
<th>b (mm)</th>
<th>t (mm)</th>
<th>r_e (mm)</th>
<th>A_g (mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FS03D</td>
<td>150 × 50 × 3.0 C450</td>
<td>150.37</td>
<td>49.89</td>
<td>2.95</td>
<td>6.0</td>
<td>1125</td>
</tr>
<tr>
<td>FS11A</td>
<td>150 × 50 × 3.0 C350</td>
<td>150.50</td>
<td>50.19</td>
<td>2.91</td>
<td>6.0</td>
<td>1110</td>
</tr>
</tbody>
</table>

Table D.3: Section Dimensions for Full Section Tensile Tests
<table>
<thead>
<tr>
<th>Specimen</th>
<th>Cut from section</th>
<th>$E$ (GPa)</th>
<th>$f_y$ (MPa)</th>
<th>$f_u$ (MPa)</th>
<th>Percentage elongation over gauge length</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>10 mm$^a$</td>
</tr>
<tr>
<td>FS03D C450</td>
<td>150 × 50 × 3.0</td>
<td>208</td>
<td>476</td>
<td>528</td>
<td>45</td>
</tr>
<tr>
<td>FS11A C350</td>
<td>150 × 50 × 3.0</td>
<td>208</td>
<td>408</td>
<td>441</td>
<td>70</td>
</tr>
</tbody>
</table>

*Notes: (a) Elongation measured across the failure plane. (b) Elongation measured outside necking region.*

Table D.4: Summary of Results for Full Section Tensile Tests

Figure D.2: Stress - Strain Curve for Specimen S03
Figure D.3: Stress - Strain Curve for Specimen S11

**D.3 COMPARISON OF DIFFERENT METHODS**

Table D.5 compares the yield stress and tensile strength (in MPa) determined using various methods. The methods are described below. The value in brackets is the difference between the appropriate value and the value obtained from the full section tensile test expressed as a percentage of the full section tensile test result.
Table D.5: Comparison of Different Methods

<table>
<thead>
<tr>
<th>Section</th>
<th>Full section</th>
<th>Coupon Tests</th>
<th>Eurocode 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Ave. of oppos.</td>
<td>Weighted average</td>
<td>low</td>
</tr>
<tr>
<td>FS03D</td>
<td>476</td>
<td>444</td>
<td>514</td>
</tr>
<tr>
<td>150 × 50 × 3.0</td>
<td>%</td>
<td>(-6.7)</td>
<td>(+8.0)</td>
</tr>
<tr>
<td>C450</td>
<td>528</td>
<td>513</td>
<td>585</td>
</tr>
<tr>
<td></td>
<td>%</td>
<td>(-2.8)</td>
<td>(+11)</td>
</tr>
<tr>
<td>FS11A</td>
<td>408</td>
<td>370</td>
<td>380</td>
</tr>
<tr>
<td>150 × 50 × 3.0</td>
<td>%</td>
<td>(-9.1)</td>
<td>(-6.9)</td>
</tr>
<tr>
<td>C350</td>
<td>441</td>
<td>429</td>
<td>436</td>
</tr>
<tr>
<td></td>
<td>%</td>
<td>(-2.7)</td>
<td>(-1.1)</td>
</tr>
</tbody>
</table>

**D.3.1 METHODS OF DETERMINING YIELD STRESS**

The first column of results in Table D.5 was obtained from the full section tensile tests. These values may be used according to Fig 5.5.2 of Eurocode 3.

The next three columns are derived from the results of tensile coupon tests. The average of the yield stress of both adjacent faces has been used in this thesis. The yield stress of the opposite face is normally about 10% higher than those of the adjacent faces. The opposite face value has been used by CASE (1992b, 1992c). A weighted average is also given to account for the variability of yield stress about the section. The yield stress of each face and corner is weighted with respect to the proportion of area of the entire section the region contains. Since no coupon was cut from the face containing the weld, this face is assumed to have the same properties as the opposite face.

The final set of values are calculated from Figure 5.5.2 of Eurocode (also shown as Equation D.1). This equation is based on the yield stress and ultimate strength of the original strip steel from which the RHS was formed. Unfortunately there was no data available on the virgin steel for these specimens, but a range of values for the mechanical properties was obtained.
from BHP Structural and Pipeline Products over a three month period at the time of production of the tubes. Low, mean and high values are given below. The low value is the minimum allowed by the specification. During the three month period more than 99% of specimens achieved the targets. The median value is the “middle score”. Half the samples were above, and the other half were below, these values. The high reading was exceeded by only 1% of samples. The following values were assumed:

\[
\begin{align*}
    f_{yb} & = 300 \text{ MPa (min)} \\
    & = 340 \text{ MPa (mean)} \\
    & = 390 \text{ MPa (max)} \\
    f_{ub} & = 400 \text{ MPa (min)} \\
    & = 460 \text{ MPa (mean)} \\
    & = 510 \text{ MPa (max)}
\end{align*}
\]

D.3.2 DISCUSSION

No definite conclusions can be made from only two tests but the following observations can be made:

- The average of the adjacents is slightly lower than the weighted average. This is because the higher strength corners comprise a small percentage of the area, and being a high aspect ratio section the flat area of the adjacents (the webs) is larger than the flange. For thicker sections the influence of the corners would be greater, and for lower aspect ratio sections (eg SHS) the influence of the higher strength opposite face increases. The higher strength in the corners would be more significant under bending, since the corners have a higher lever arm to the bending neutral axis, compared to the web.

- The weighted average, adjacent average and full section yield stress are all reasonably close together (10% for C350, 7% for C450). Use of the average of the adjacents would probably underestimate the strength of the section.
Since the properties of the original strip are unknown, it is only possible to calculate a range of values of yield stress from Equation D.1. It is most likely that the properties of the steel are approximately equal to the median values given above. Fig 5.5.2 of Eurocode 3 gives very low values compared to the tests for two reasons. The equation does not include the strength enhancing effects of in-line galvanising, hence it gives the same answer for C350 and C450 steel. The strength enhancement calculation is based merely on bending the corners to 90°. The forming process of the RHS involves forming a circle then further cold working into a RHS. Hence the cold work on the flats is not included. Therefore Figure 5.5.2 of Eurocode 3 will give conservative values of the yield stress.

All results in this thesis have used the average of the adjacents in strength calculations. For in-plane bending, the maximum moment achieved by the compact specimens was, on average, 10 - 20 % higher than the plastic moment based on the average of the adjacents. In stub column tests, the maximum load was, on average, 10 - 20 % higher than the section capacity based on the average of the adjacents. This indicates that the use of the average yield stress of the adjacent faces is slightly conservative to the order of approximately 10 %. Nevertheless, the average yield stress of the adjacent faces provides a useful lower bound for strength design which is not overly conservative.

In plastic bending tests and stub column tests by CASE (1992b, 1992c), the yield stress of the opposite face was used. The conclusion from these tests was that the reliability of strength design, using the yield stress of the opposite face, was comparable to the reliability of hot rolled steel beams used in the development of AS 4100. This indicates that the use of the yield stress of the opposite face is suitable in strength design.
Appendix E

STUB COLUMN TESTS

E.1 PROCEDURE

Stub column tests were performed in a 2000 kN capacity DARTEC testing machine, using a servo-controlled hydraulic ram. The ends of each RHS sample were machined flat and perpendicular prior to testing. The bottom platen was fixed against rotation. The top platen was mounted on a spherical seat which allowed complete contact between each end of the stub column and the platens. The test set up is shown in Figure E.1. Displacement transducers measured the axial shortening of the specimen. A SPECTRA data acquisition system recorded the transducer readings and the load at regular stages during each test. The test was halted for approximately one minute to obtain a value of the static load near the maximum.

The lengths of the stub columns were chosen using the guidelines set out by Johnston (1976). This specifies a minimum stub column length of $3d$ and a maximum length of $20r_y$. It was not always possible to adhere to these limits for the sections with an aspect ratio $d/b$ of 3 since $3d > 20r_y$ for these samples. In these cases the length chosen was approximately $3d$. 
E.2 RESULTS

Each specimen experienced local buckling which initiated in the web. Compatibility of rotation at the corner produced deformation in the flange of the RHS. The load increased until local buckling occurred, after which the load dropped with increased axial shortening.

Tables E.1 and E.2 set out the results for each stub column test. The maximum static load \( N_{\text{max}} \) is compared to the yield load \( N_y = A_g f_y \) and the section capacity \( N_s \) (determined to AS 4100, Eurocode 3 and AISC LRFD), which includes the effect of local buckling by the use of effective widths. The yield stress used in calculations is the measured yield stress (Appendix C) rather than the nominal yield stress. The ratios of these loads and the web slenderness are also tabulated. Table E.3 tabulates the dimensions of each test piece. Figures E.2, E.3 and E.4 shows the relationship between these ratios and the web slenderness, as well as the yield slenderness limit for the three specifications mentioned above.
<table>
<thead>
<tr>
<th>Specimen</th>
<th>Cut from Section</th>
<th>$N_{\text{max}}$ (kN)</th>
<th>$N_y$ (kN)</th>
<th>$\frac{N_{\text{max}}}{N_y}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>SCS01D</td>
<td>$150 \times 50 \times 5.0$ C450</td>
<td>878</td>
<td>790</td>
<td>1.11</td>
</tr>
<tr>
<td>SCS02D</td>
<td>$150 \times 50 \times 4.0$ C450</td>
<td>670</td>
<td>668</td>
<td>1.00</td>
</tr>
<tr>
<td>SCS03D</td>
<td>$150 \times 50 \times 3.0$ C450</td>
<td>422</td>
<td>499</td>
<td>0.845</td>
</tr>
<tr>
<td>SCS04D</td>
<td>$150 \times 50 \times 2.5$ C450</td>
<td>330</td>
<td>438</td>
<td>0.754</td>
</tr>
<tr>
<td>SCS05D</td>
<td>$150 \times 50 \times 2.3$ C450</td>
<td>263</td>
<td>383</td>
<td>0.687</td>
</tr>
<tr>
<td>SCS06D</td>
<td>$100 \times 50 \times 2.0$ C450</td>
<td>235</td>
<td>267</td>
<td>0.881</td>
</tr>
<tr>
<td>SCS07D</td>
<td>$75 \times 50 \times 2.0$ C450</td>
<td>190</td>
<td>188</td>
<td>1.01</td>
</tr>
<tr>
<td>SCS08D</td>
<td>$75 \times 25 \times 2.0$ C450</td>
<td>160</td>
<td>269</td>
<td>0.946</td>
</tr>
<tr>
<td>SCS09D</td>
<td>$75 \times 25 \times 1.6$ C450</td>
<td>114</td>
<td>129</td>
<td>0.904</td>
</tr>
<tr>
<td>SCS10D</td>
<td>$75 \times 25 \times 1.6$ C350</td>
<td>101</td>
<td>124</td>
<td>0.815</td>
</tr>
<tr>
<td>SCS11D</td>
<td>$150 \times 50 \times 3.0$ C350</td>
<td>397</td>
<td>420</td>
<td>0.945</td>
</tr>
<tr>
<td>SCS12D</td>
<td>$100 \times 50 \times 2.0$ C350</td>
<td>207</td>
<td>237</td>
<td>0.874</td>
</tr>
<tr>
<td>SCS13D</td>
<td>$125 \times 75 \times 3.0$ C350</td>
<td>418</td>
<td>446</td>
<td>0.936</td>
</tr>
</tbody>
</table>

Table E.1: Summary of Stub Column Test Results (1)
<table>
<thead>
<tr>
<th>Specimen</th>
<th>AS 4100</th>
<th>Eurocode 3</th>
<th>AISC LRFD</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$w$</td>
<td>$N_s$</td>
<td>$N_{max}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$l_m$</td>
<td>$N_s$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$l_m$</td>
<td>$N_s$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$l_m$</td>
<td>$N_s$</td>
</tr>
<tr>
<td>Limit</td>
<td>40</td>
<td>42</td>
<td>1.40</td>
</tr>
<tr>
<td>SCS01D</td>
<td>38.4</td>
<td>790</td>
<td>1.11</td>
</tr>
<tr>
<td>SCS02D</td>
<td>49.7</td>
<td>569</td>
<td>1.18</td>
</tr>
<tr>
<td>SCS03D</td>
<td>65.5</td>
<td>352</td>
<td>1.20</td>
</tr>
<tr>
<td>SCS04D</td>
<td>76.1</td>
<td>281</td>
<td>1.17</td>
</tr>
<tr>
<td>SCS05D</td>
<td>87.1</td>
<td>227</td>
<td>1.16</td>
</tr>
<tr>
<td>SCS06D</td>
<td>62.3</td>
<td>203</td>
<td>1.16</td>
</tr>
<tr>
<td>SCS07D</td>
<td>47.4</td>
<td>170</td>
<td>1.12</td>
</tr>
<tr>
<td>SCS08D</td>
<td>48.7</td>
<td>146</td>
<td>1.09</td>
</tr>
<tr>
<td>SCS09D</td>
<td>61.6</td>
<td>95</td>
<td>1.22</td>
</tr>
<tr>
<td>SCS10D</td>
<td>60.6</td>
<td>92</td>
<td>1.10</td>
</tr>
<tr>
<td>SCS11D</td>
<td>58.9</td>
<td>318</td>
<td>1.25</td>
</tr>
<tr>
<td>SCS12D</td>
<td>59.6</td>
<td>185</td>
<td>1.12</td>
</tr>
<tr>
<td>SCS13D</td>
<td>51.0</td>
<td>386</td>
<td>1.08</td>
</tr>
</tbody>
</table>

Table E.2: Summary of Stub Column Test Results (2)
Table E.3: Dimensions of Stub Columns

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Cut from Section</th>
<th>d (mm)</th>
<th>b (mm)</th>
<th>t (mm)</th>
<th>r (mm)</th>
<th>$A_g$ (mm$^2$)</th>
<th>$A_g/A_n$</th>
<th>$L_{sc}$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SCS01D</td>
<td>$150 \times 50 \times 5.0$ C450</td>
<td>150.99</td>
<td>50.11</td>
<td>4.89</td>
<td>11.8</td>
<td>1792</td>
<td>0.988</td>
<td>399.9</td>
</tr>
<tr>
<td>SCS02D</td>
<td>$150 \times 50 \times 4.0$ C450</td>
<td>150.38</td>
<td>50.17</td>
<td>3.88</td>
<td>7.1</td>
<td>1462</td>
<td>0.987</td>
<td>400.1</td>
</tr>
<tr>
<td>SCS03D</td>
<td>$150 \times 50 \times 3.0$ C450</td>
<td>150.64</td>
<td>50.09</td>
<td>2.94</td>
<td>5.8</td>
<td>1125</td>
<td>0.986</td>
<td>400.2</td>
</tr>
<tr>
<td>SCS04D</td>
<td>$150 \times 50 \times 2.5$ C450</td>
<td>150.25</td>
<td>50.03</td>
<td>2.55</td>
<td>4.0</td>
<td>982</td>
<td>1.024</td>
<td>400.2</td>
</tr>
<tr>
<td>SCS05D</td>
<td>$150 \times 50 \times 2.3$ C450</td>
<td>150.28</td>
<td>50.57</td>
<td>2.23</td>
<td>4.4</td>
<td>863</td>
<td>0.900</td>
<td>400.1</td>
</tr>
<tr>
<td>SCS06D</td>
<td>$100 \times 50 \times 2.0$ C450</td>
<td>100.38</td>
<td>50.43</td>
<td>2.07</td>
<td>4.4</td>
<td>595</td>
<td>1.036</td>
<td>300.0</td>
</tr>
<tr>
<td>SCS07D</td>
<td>$75 \times 50 \times 2.0$ C450</td>
<td>75.25</td>
<td>49.88</td>
<td>1.93</td>
<td>4.4</td>
<td>457</td>
<td>0.965</td>
<td>225.0</td>
</tr>
<tr>
<td>SCS08D</td>
<td>$75 \times 25 \times 2.0$ C450</td>
<td>75.24</td>
<td>25.06</td>
<td>1.98</td>
<td>4.1</td>
<td>371</td>
<td>0.992</td>
<td>224.9</td>
</tr>
<tr>
<td>SCS09D</td>
<td>$75 \times 25 \times 2.0$ C450</td>
<td>75.03</td>
<td>25.15</td>
<td>1.55</td>
<td>3.2</td>
<td>294</td>
<td>0.970</td>
<td>224.9</td>
</tr>
<tr>
<td>SCS10D</td>
<td>$75 \times 25 \times 2.0$ C350</td>
<td>75.21</td>
<td>25.10</td>
<td>1.55</td>
<td>3.4</td>
<td>294</td>
<td>0.969</td>
<td>224.8</td>
</tr>
<tr>
<td>SCS11D</td>
<td>$150 \times 50 \times 3.0$ C350</td>
<td>150.42</td>
<td>50.07</td>
<td>2.98</td>
<td>5.9</td>
<td>1137</td>
<td>0.997</td>
<td>224.8</td>
</tr>
<tr>
<td>SCS12D</td>
<td>$100 \times 50 \times 2.0$ C350</td>
<td>100.88</td>
<td>50.45</td>
<td>2.05</td>
<td>4.2</td>
<td>593</td>
<td>1.033</td>
<td>300.3</td>
</tr>
<tr>
<td>SCS13D</td>
<td>$125 \times 75 \times 3.0$ C350</td>
<td>125.32</td>
<td>75.60</td>
<td>2.94</td>
<td>6.3</td>
<td>1125</td>
<td>0.986</td>
<td>374.9</td>
</tr>
</tbody>
</table>

**E.3 DISCUSSION**

$N_{max}/N_y$ is less than or equal to unity for all but one section when the web slenderness exceeds yield slenderness limit and $N_{max}/N_y$ is greater than one when the web slenderness is less than the limit. This indicates that the web slenderness limit for pure compression is appropriate.

$N_{max}/N_s$ is greater than 1.0 for all but one section (SCS10D for AISC LRFD only). This fact indicates that the effective width formulation in the determination of $N_s$ is safe. The linear regression relationship for $N_{max}/N_s$ in Figure E.2, which is in the region of 1.1 to 1.2, suggests that AS 4100 is slightly conservative in this formulation.
Figure E.2: Load - Slenderness Relation for Stub Column Tests

Figure E.3: Load - Slenderness Relation for Stub Column Tests
Figure E.4: Load - Slenderness Relation for Stub Column Tests
Figure E.5: Stub Column Test Result for SCS01D

Figure E.6: Stub Column Test Result for SCS02D

Figure E.7: Stub Column Test Result for SCS03D
Figure E.8: Stub Column Test Result for SCS04D

Figure E.9: Stub Column Test Result for SCS05D

Figure E.10: Stub Column Test Result for SCS06D
Figure E.11: Stub Column Test Result for SCS07D

Figure E.12: Stub Column Test Result for SCS08D

Figure E.13: Stub Column Test Result for SCS09D
Figure E.14: Stub Column Test Result for SCS10D

Figure E.15: Stub Column Test Result for SCS11D

Figure E.16: Stub Column Test Result for SCS12D
Figure E.17: Stub Column Test Result for SCS131D
Appendix F

IMPERFECTION MEASUREMENTS

F.1 INTRODUCTION

RHS are not perfect rectangular specimens. The cold-forming process, welding, handling and other factors can introduce geometric imperfections. Local imperfections can have a significant effect on the moment and rotation capacity (Key and Hancock 1993). Hence the local imperfections of the RHS were measured as part of the test program. The nature of the imperfection is important in subsequent finite element modelling of the bending tests.

F.2 METHOD

F.2.1 FINAL METHOD

Two methods of imperfection measurement were used. The first method was found to give unreliable results. In the second method, the sample was placed above an accurately machined and greased beam. A displacement transducer was moved along this machined beam, measuring the distance to the specimen at 25 mm intervals. This is shown diagrammatically in Figure F.1. Measurement was made along fourteen lines down the specimen as shown in Figure F.1(a). By comparing the transducer readings along the edges of each face to those at the centre of each face, the “bow-out” imperfection (refer to Figure F.3) at the middle of each flange and third points of each web was obtained, resulting in a profile of the beam. The readings were made in the centre of each bending specimen, between the loading points (Figure 3.5). It was only practical to measure each specimen to within approximately 100 mm of the centre of the loading points.
F.2.2 INITIAL METHOD

The first method of imperfection measurement involved moving a frame mounted along precision bearings as shown in Figure F.2. This method only obtained web profiles relative to any bow-out imperfections at the end. Vibrations of the frame as it moved along the shafts affected the precision of the measurements, and resulted in an imperfection profile which cannot be assumed to be correct. For completeness the imperfection results obtained using this method are also included in this Appendix. The wave-like imperfections obtained are a result of the vibration of the frame.
Imperfections were measured using an aluminium frame mounted with a series of transducers as shown in Figure F.3. This frame was able to move longitudinally along a set of precision shafts. As the frame travelled along the length of the specimen a position sensor was triggered every 25 mm, and all transducer readings were taken simultaneously.

Each specimen was measured after the loading plates were welded to the webs of the specimen. Hence imperfection measurements were only taken on the middle portion of the specimen between the loading plates. Measurements were taken four times for each specimen and the average imperfection was calculated recorded.

Figure F.2: Schematic Diagram of Original Imperfection Measurement Apparatus
F.3 RESULTS

Most RHS exhibited a “bow-out” (as shown in Figure F.3) or “bow-in” on each flat face of the section that was reasonably constant along the length of the RHS. It was most common for there to be a bow-out on each web, and a bow-in on each flange. The magnitude of the imperfection was approximately constant along the length of the specimen, that is, no significant wave-like imperfections along the length were detected. It was not possible to measure imperfections close to the loading plates that were welded to the specimen (refer to Figure 3.5). Visible inspection of the specimens indicated that there was some imperfection induced into the specimen next to the welds connecting the loading plates to the RHS.

The profiles are shown in Figure F.5 to F.33. It can be observed that the values of the imperfections are small (typically less than ±1% of the specimen dimension). The stiffer faces of the RHS tend to exhibit a negative imperfection (an inwards bow), while the other faces exhibit an outwards bow. For example a “thick” specimen, BS01C, exhibits inwards imperfection on all faces, and a thinner specimen exhibits bow-out on the less stiff webs but inwards bowing on the stiffer flanges.

Figure F.3: Typical “Bow-Out” Imperfection of an RHS (note: bow-out exaggerated)
The imperfection resulted from the cold-forming process. The coil of steel was initially rolled as a circle and then the centre of each face was pushed in by successive sets of rollers to form the rectangular shape. The same set of rollers was used for the same outer size of RHS and hence similar force formed the section into the rectangular shape. The thinner sections tend to “spring back” more after passing a set of rolls causing the “bow-out”.

This imperfection was generally approximately constant along the length of the beam. There was often some extra initial deformation near the loading plates, caused by welding of the loading plates and the clamping of the samples during this welding, but it was difficult to measure such imperfections accurately. There was more variability in the profile of the smaller specimens - it was more difficult to measure these samples accurately, and smaller values of these imperfections approach the precision of the displacement transducer. The profile of the weld face of the RHS was often difficult to measure precisely due to the seam of the join on the outside of the RHS.

The figures give the “bow-out” deformation along the length of the RHS specimen at six positions along the flange and web shown below.

![Diagram of Imperfection Profile Measurement](image)

Figure F.4: Location of Imperfection Profile Measurement
Figure F.5: Section Profile for BS01C

Figure F.6: Section Profile for BS02A

Figure F.7: Section Profile for BS02C
Figure F.8: Section Profile for BS03C

Figure F.9: Section Profile for BS04A

Figure F.10: Section Profile for BS04C
Figure F.11: Section Profile for BS05C

Figure F.12: Section Profile for BS06A

Figure F.13: Section Profile for BS06C
Figure F.14: Section Profile for BS07B

Figure F.15: Section Profile for BS07C

Figure F.16: Section Profile for BS08B
Figure F.17: Section Profile for BS08C

Figure F.18: Section Profile for BS09A

Figure F.19: Section Profile for BS09C
Figure F.20: Section Profile for BS10B

Figure F.21: Section Profile for BS10C

Figure F.22: Section Profile for BS11B
**Figure F.23: Section Profile for BS11C**

**Figure F.24: Section Profile for BS12B**

**Figure F.25: Section Profile for BS12B**

378
Section Profile
BS13A 125 x 75 x 3.0 C350

Figure F.26: Section Profile for BS13A

Section Profile
BS13B 125 x 75 x 3.0 C350

Figure F.27: Section Profile for BS13B

Section Profile
BS13C 125 x 75 x 3.0 C350

Figure F.28: Section Profile for BS13C
Figure F.29: Section Profile for BS16A

Figure F.30: Section Profile for BS17A

Figure F.31: Section Profile for BS20A
The imperfections of remaining specimens were measured using the initial method described in Section F.2.2. For these specimens the imperfection of the centre of each web *relative to any bow-out at the ends* is graphed in Figures F.34 to F.39.

The web imperfections of Specimen BS04B in Figure F.37 are unusually large and are approximately ten times greater than those observed on other specimens. The reason for this is unknown, but it is possible the specimen moved while measurements were taken.
Figure F.34: Web Imperfections for BS01B

Figure F.35: Web Imperfections for BS02B

Figure F.36: Web Imperfections for BS03B
Figure F.37: Web Imperfections for BS04B
(Refer to note at bottom of page 381)

Figure F.38: Web Imperfections for BS05B

Figure F.39: Web Imperfections for BS06B
Appendix G

WELDING PROCEDURES

Welding procedures may be pre-qualified by Clause 4.3 of Australian / New Zealand Standard AS/NZS 1554.1 (Standards Australia 1995). The joint preparation, welding consumables, workmanship and welding techniques need to be pre-qualified to AS/NZS 1554.1 and are summarised in a welding procedure sheet.

A variety of welding procedures was used. These procedures are shown in this Appendix. Some samples were welded by an independent fabricator, R. K. Engineering, (Procedure RKE - 138). This provided a useful benchmark as to the quality of welding that would be found in practice as performed by a typical steel fabricator. All other welding was performed in the Department of Civil Engineering, University of Sydney.

For the welding procedures performed at The University of Sydney (TJW01 - TJW07), the weld run location only shows the weld runs at a given section through a weld, and not the weld sequence around the circumference of the RHS. Figure G.1 shows the circumferential weld sequence around the RHS.
a) Weld run location in test series

Figure G.1: Circumferential Weld Run Location
### Welding Procedure Sheet

**R. K. ENGINEERING PTY LTD**  
WELDING PROCEDURE SHEET  
(rewritten at The University of Sydney)  
St Marys NSW 2760

<table>
<thead>
<tr>
<th>Welding Procedure:</th>
<th>RKE - 138</th>
</tr>
</thead>
<tbody>
<tr>
<td>Date Produced:</td>
<td>31 August 1994</td>
</tr>
<tr>
<td>Material:</td>
<td>Grade 350 Plate - AS 3678 / RHS AS 1163</td>
</tr>
<tr>
<td>Thickness:</td>
<td>Plate: 7.5 - 24 mm</td>
</tr>
<tr>
<td>Joint Type:</td>
<td>H-C 4a (Table 4.4(D) of AS 1554.1)</td>
</tr>
<tr>
<td>Welding Process:</td>
<td>GMAW</td>
</tr>
<tr>
<td>Welding Position:</td>
<td>Horizontal</td>
</tr>
<tr>
<td>Gas Trade Name:</td>
<td>Argoshield 50</td>
</tr>
<tr>
<td>Gas Composition:</td>
<td>5% CO₂, 3% O₂, 92% Ar</td>
</tr>
<tr>
<td>Gas Flow Rate (L/min):</td>
<td>15 - 18</td>
</tr>
<tr>
<td>Electrode Trade Name:</td>
<td>Autocraft LW1</td>
</tr>
<tr>
<td>Electrode Classification:</td>
<td>ES4-GM/C W503H (AS 2717.1 (1984))</td>
</tr>
<tr>
<td>Electrode Size (mm):</td>
<td>0.9</td>
</tr>
<tr>
<td>PreHeat Temp:</td>
<td>50°C</td>
</tr>
<tr>
<td>InterPass Temp:</td>
<td>N/A</td>
</tr>
<tr>
<td>InterPass Procedure:</td>
<td>None</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Pass No</th>
<th>Amps</th>
<th>Volts</th>
<th>Polarity</th>
<th>Stickout (mm)</th>
<th>Wire Speed (mm/min)</th>
<th>Arc Travel (mm/min)</th>
<th>Arc Energy (kJ/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>255</td>
<td>26</td>
<td>+ve</td>
<td>25</td>
<td>8500</td>
<td>395</td>
<td>1.00</td>
</tr>
<tr>
<td>2</td>
<td>255</td>
<td>26</td>
<td>+ve</td>
<td>25</td>
<td>8500</td>
<td>395</td>
<td>1.00</td>
</tr>
</tbody>
</table>

**Sketch/Pass Sequence**

Gap: $G = 2 - 4$ mm, Landing $F_t = 1.5$ mm (max)

**Qualification Requirements**

<table>
<thead>
<tr>
<th>Requirement</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ultrasonic</td>
<td>NDT 1397 (19/6/96)</td>
</tr>
<tr>
<td>Hardness</td>
<td>-</td>
</tr>
<tr>
<td>Bend Test</td>
<td>-</td>
</tr>
<tr>
<td>Macro:</td>
<td>Dye Penetration: -</td>
</tr>
<tr>
<td></td>
<td>Visual: -</td>
</tr>
<tr>
<td></td>
<td>Stress Rel.: -</td>
</tr>
</tbody>
</table>

R.K.E Rep: F. Kriz  
Approved: 6 September 1994
<table>
<thead>
<tr>
<th>Welding Procedure:</th>
<th>TJW01</th>
</tr>
</thead>
<tbody>
<tr>
<td>Date Produced:</td>
<td>23 May 1996</td>
</tr>
<tr>
<td>Material:</td>
<td>DURAGAL C450L0 RHS - AS 1163 Grade 250 or 350 Plate - AS 3678</td>
</tr>
<tr>
<td>Thickness:</td>
<td>RHS: 4 mm, Plate: 10 mm</td>
</tr>
<tr>
<td>Joint Type:</td>
<td>H-C 4a (Table 4.4(D) of AS/NZS 1554.1) H-C 4c (Table 4.4(D) of AS/NZS 1554.1)</td>
</tr>
<tr>
<td>Welding Machine:</td>
<td>TransMIG 250 Powersource and Transmatic 62 wire feeder from CIGWELD</td>
</tr>
<tr>
<td>Welding Process:</td>
<td>GMAW</td>
</tr>
<tr>
<td>Gas Trade Name:</td>
<td>Argoshield 52</td>
</tr>
<tr>
<td>Gas Composition:</td>
<td>23 % CO₂, 77 % Ar</td>
</tr>
<tr>
<td>Gas Flow Rate (L/min):</td>
<td>25</td>
</tr>
<tr>
<td>Electrode Trade Name</td>
<td>Autocraft LW1</td>
</tr>
<tr>
<td>Electrode Classification:</td>
<td>ES4-GM/C W503H (AS 2717.1 (1984))</td>
</tr>
<tr>
<td>Electrode Size (mm):</td>
<td>0.9</td>
</tr>
<tr>
<td>PreHeat Temp:</td>
<td>N/A</td>
</tr>
<tr>
<td>InterPass Temp:</td>
<td>N/A</td>
</tr>
<tr>
<td>InterPass Procedure:</td>
<td>None</td>
</tr>
<tr>
<td>Pass No</td>
<td>Amps</td>
</tr>
<tr>
<td>1</td>
<td>230</td>
</tr>
<tr>
<td>2</td>
<td>230</td>
</tr>
<tr>
<td>Qualification Requirements</td>
<td>Macro:</td>
</tr>
<tr>
<td>Ultrasonic:</td>
<td>Dye Penetration:</td>
</tr>
<tr>
<td>Hardness:</td>
<td>Visual:</td>
</tr>
<tr>
<td>Bend Test:</td>
<td>Stress Rel.:</td>
</tr>
<tr>
<td>Welder:</td>
<td>G. Holgate</td>
</tr>
<tr>
<td>Checked:</td>
<td>G. Towell</td>
</tr>
<tr>
<td>Approved:</td>
<td>T. Wilkinson</td>
</tr>
<tr>
<td>Date:</td>
<td>23 May 1996</td>
</tr>
</tbody>
</table>
Welding Procedure: TJW01

JOINT PREPARATION

Notes: 1) Purge inside of tube with Argoshield 52 to improve penetration.
2) Crotch and heel profile only apply to knee connection.
3) Extra fillet weld on crotch of connection only
Welding Procedure: TJW02


Date Produced: 7 July 1996

Material: DURAGAL C450L0 RHS - AS 1163

Thickness: RHS: 4 mm

Joint Type: H-C 4a (Table 4.4(D) of AS/NZS 1554.1)

Welding Machine: TransMIG 250 Powersource and Transmatic 62 wire feeder from CIGWELD

Welding Process: GMAW

Gas Trade Name: Argoshield 52

Gas Composition: 23 % CO₂, 77 % Ar

Gas Flow Rate (L/min): 25

Electrode Trade Name: Autocraft LW1

Electrode Classification: ES4-GM/C W503H (AS 2717.1 (1984))

Electrode Size (mm): 0.9

PreHeat Temp: N/A InterPass Temp: N/A

InterPass Procedure: None

<table>
<thead>
<tr>
<th>Pass No</th>
<th>Amps</th>
<th>Volts</th>
<th>Polarity</th>
<th>Stickout mm</th>
<th>Wire Speed mm/min</th>
<th>Welding Speed mm/min</th>
<th>Arc Energy kJ/mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>170</td>
<td>21</td>
<td>DCEP</td>
<td>15</td>
<td>6500</td>
<td>405</td>
<td>0.53</td>
</tr>
</tbody>
</table>

Qualification Requirements

Ultrasonic: Macro:

Hardness: Dye Penetration:

Bend Test: Visual:

Stress Rel.:

Welder: G. Holgate Checked: G. Towell

Approved: T. Wilkinson Date: 7 July 1996
Welding Procedure: TJW02

JOINT PREPARATION

Profile at toe

Profile at crotch

Gap, \( G = 0 \) mm
Landing, \( F_r = 2 \) mm

WELD RUN LOCATION & SEQUENCE

Notes:
<table>
<thead>
<tr>
<th>Welding Procedure:</th>
<th>TJW03</th>
</tr>
</thead>
<tbody>
<tr>
<td>Date Produced:</td>
<td>31 July 1996</td>
</tr>
</tbody>
</table>
| Material: | C350L0 RHS - AS 1163  
Grade 250 or 350 Plate - AS 3678 |
| Thickness: | RHS: 5 mm, Plate: 10 mm |
| Joint Type: | H-C 4a (Table 4.4(D) of AS/NZS 1554.1)  
H-C 4c (Table 4.4(D) of AS/NZS 1554.1) |
| Welding Machine: | TransMIG 250 Powersource and Transmatic 62 wire feeder from CIGWELD |
| Welding Process: | GMAW |
| Gas Trade Name: | Argoshield 52 |
| Gas Composition: | 23 % CO₂ , 77 % Ar |
| Gas Flow Rate (L/min): | 25 |
| Electrode Trade Name | Autocraft LW1 |
| Electrode Classification: | ES4-GM/C W503H (AS 2717.1 (1984)) |
| Electrode Size (mm): | 0.9 |
| PreHeat Temp: | N/A |
| InterPass Temp: | N/A |
| InterPass Procedure: | None |

<table>
<thead>
<tr>
<th>Pass No</th>
<th>Amps</th>
<th>Volts</th>
<th>Polarity</th>
<th>Stickout (mm)</th>
<th>Wire Speed (mm/min)</th>
<th>Welding Speed (mm/min)</th>
<th>Arc Energy (kJ/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>200</td>
<td>20</td>
<td>DCEP</td>
<td>15</td>
<td>8000</td>
<td>175</td>
<td>1.37</td>
</tr>
<tr>
<td>2</td>
<td>200</td>
<td>20</td>
<td>DCEP</td>
<td>15</td>
<td>8000</td>
<td>200</td>
<td>1.20</td>
</tr>
</tbody>
</table>

Qualification Requirements

| Ultrasonic: | Dye Penetration: |
| Hardness: | Visual: |
| Bend Test: | Stress Rel.: |

Welder: G. Holgate  
Checked: G. Towell  
Approved: T. Wilkinson  
Date: 31 July 1996
### JOINT PREPARATION

- $\theta = 45^\circ$
- $t = 5 \text{ mm (RHS)}$
- $G = 2 \text{ mm}$
- $F_t = 1 \text{ mm}$

Notes:
1) Purge inside of tube with Argoshield 52 to improve penetration.
2) Crotch and heel profile only apply to knee connection.
3) Extra fillet weld on crotch of connection only

### WELD RUN LOCATION & SEQUENCE

- Profile at toe
- Profile at crotch

Notes:
1) Purge inside of tube with Argoshield 52 to improve penetration.
2) Crotch and heel profile only apply to knee connection.
3) Extra fillet weld on crotch of connection only
Welding Procedure: TJW04
Date Produced: 31 July 1996
Material: C350L0 RHS - AS 1163
Grade 250 or 350 Plate - AS 3678
Thickness: RHS: 4 mm, Plate: 10 mm
Joint Type: H-C 4a (Table 4.4(D) of AS/NZS 1554.1)
H-C 4c (Table 4.4(D) of AS/NZS 1554.1)
Welding Machine: TransMIG 250 Powersource and Transmatic 62 wire feeder from CIGWELD
Welding Process: GMAW
Gas Trade Name: Argoshield 52
Gas Composition: 23 % CO₂, 77 % Ar
Gas Flow Rate (L/min): 25
Electrode Trade Name: Autocraft LW1
Electrode Classification: ES4-GM/C W503H (AS 2717.1 (1984))
Electrode Size (mm): 0.9
PreHeat Temp: N/A
InterPass Temp: N/A
InterPass Procedure: None
<table>
<thead>
<tr>
<th>Pass No</th>
<th>Amps</th>
<th>Volts</th>
<th>Polarity</th>
<th>Stickout</th>
<th>Wire Speed</th>
<th>Welding Speed</th>
<th>Arc Energy</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>mm/min</td>
<td>mm/min</td>
<td>kJ/mm</td>
</tr>
<tr>
<td>1</td>
<td>210</td>
<td>20</td>
<td>DCEP</td>
<td>15</td>
<td>7500</td>
<td>190</td>
<td>1.33</td>
</tr>
<tr>
<td>2</td>
<td>210</td>
<td>20</td>
<td>DCEP</td>
<td>15</td>
<td>7500</td>
<td>200</td>
<td>1.26</td>
</tr>
</tbody>
</table>
Qualification Requirements
Ultrasonic: Dye Penetration: Visual: Stress Rel:
Hardness: Bend Test:
Welder: G. Holgate
Checked: G. Towell
Approved: T. Wilkinson
Date: 31 July 1996
Welding Procedure: TJW04

JOINT PREPARATION

Notes: 1) Purge inside of tube with Argoshield 52 to improve penetration.
2) Crotch and heel profile only apply to knee connection.
3) Extra fillet weld on crotch of connection only
<table>
<thead>
<tr>
<th>Welding Procedure:</th>
<th>TJW05</th>
</tr>
</thead>
<tbody>
<tr>
<td>Date Produced:</td>
<td>31 July 1996</td>
</tr>
<tr>
<td>Material:</td>
<td>DURAGAL C450L0 RHS - AS 1163 Grade 250 or 350 Plate - AS 3678</td>
</tr>
<tr>
<td>Thickness:</td>
<td>RHS: 4 mm, Plate: 10 mm</td>
</tr>
<tr>
<td>Joint Type:</td>
<td>Figure 4.4.5.2 of AS/NZS 1554.1</td>
</tr>
<tr>
<td>Welding Machine:</td>
<td>TransMIG 250 Powersource and Transmatic 62 wire feeder from CIGWELD</td>
</tr>
<tr>
<td>Welding Process:</td>
<td>GMAW</td>
</tr>
<tr>
<td>Gas Trade Name:</td>
<td>Argoshield 52</td>
</tr>
<tr>
<td>Gas Composition:</td>
<td>23 % CO₂, 77 % Ar</td>
</tr>
<tr>
<td>Gas Flow Rate (L/min):</td>
<td>25</td>
</tr>
<tr>
<td>Electrode Trade Name</td>
<td>Autocraft LW1</td>
</tr>
<tr>
<td>Electrode Classification:</td>
<td>ES4-GM/C W503H (AS 2717.1 (1984))</td>
</tr>
<tr>
<td>Electrode Size (mm):</td>
<td>0.9</td>
</tr>
<tr>
<td>PreHeat Temp:</td>
<td>N/A</td>
</tr>
<tr>
<td>InterPass Temp:</td>
<td>N/A</td>
</tr>
<tr>
<td>InterPass Procedure:</td>
<td>None</td>
</tr>
<tr>
<td>Pass No</td>
<td>Amps</td>
</tr>
<tr>
<td>---------</td>
<td>------</td>
</tr>
<tr>
<td>1</td>
<td>190</td>
</tr>
<tr>
<td>Qualification Requirements</td>
<td>Macro:</td>
</tr>
<tr>
<td>Ultrasonic:</td>
<td>Dye Penetration:</td>
</tr>
<tr>
<td>Hardness:</td>
<td>Visual:</td>
</tr>
<tr>
<td>Bend Test:</td>
<td>Stress Rel.:</td>
</tr>
<tr>
<td>Welder:</td>
<td>Checked: G. Towell</td>
</tr>
<tr>
<td>G. Holgate</td>
<td></td>
</tr>
<tr>
<td>Approved:</td>
<td>Date: 31 July 1996</td>
</tr>
<tr>
<td>T. Wilkinson</td>
<td></td>
</tr>
</tbody>
</table>
Welding Procedure: TJW05

JOINT PREPARATION

Profile at toe

Profile at crotch

Gap $G = 0$ mm

WELD RUN LOCATION & SEQUENCE

Thickness RHS $t = 4$ mm
Weld $t_{\text{min}} = 4$ mm

Notes:
<table>
<thead>
<tr>
<th>Welding Procedure:</th>
<th>TJW06</th>
</tr>
</thead>
<tbody>
<tr>
<td>Date Produced:</td>
<td>10 December 1996</td>
</tr>
<tr>
<td>Material:</td>
<td>C350L0 RHS - AS 1163 Grade 250 or 350 Plate - AS 3678</td>
</tr>
<tr>
<td>Thickness:</td>
<td>RHS: 4 mm, Plate: 10 mm</td>
</tr>
<tr>
<td>Joint Type:</td>
<td>Not prequalified to AS/NZS 1554.1 (see figure)</td>
</tr>
<tr>
<td>Welding Machine:</td>
<td>TransMIG 250 Powersource and Transmatic 62 wire feeder from CIGWELD</td>
</tr>
<tr>
<td>Welding Process:</td>
<td>GMAW</td>
</tr>
<tr>
<td>Gas Trade Name:</td>
<td>Argoshield 52</td>
</tr>
<tr>
<td>Gas Composition:</td>
<td>23 % CO₂, 77 % Ar</td>
</tr>
<tr>
<td>Gas Flow Rate (L/min):</td>
<td>27</td>
</tr>
<tr>
<td>Electrode Trade Name</td>
<td>Autocraft LW1</td>
</tr>
<tr>
<td>Electrode Classification:</td>
<td>ES4-GM/C W503H (AS 2717.1 (1984))</td>
</tr>
<tr>
<td>Electrode Size (mm):</td>
<td>0.9</td>
</tr>
<tr>
<td>PreHeat Temp:</td>
<td>N/A</td>
</tr>
<tr>
<td>InterPass Temp:</td>
<td>N/A</td>
</tr>
<tr>
<td>InterPass Procedure:</td>
<td>None</td>
</tr>
<tr>
<td>Pass No</td>
<td>Amps</td>
</tr>
<tr>
<td>--------</td>
<td>------</td>
</tr>
<tr>
<td>1</td>
<td>210</td>
</tr>
<tr>
<td>2</td>
<td>210</td>
</tr>
<tr>
<td>3</td>
<td>210</td>
</tr>
<tr>
<td>Qualification Requirements</td>
<td></td>
</tr>
<tr>
<td>Ultrasonic:</td>
<td></td>
</tr>
<tr>
<td>Hardness:</td>
<td></td>
</tr>
<tr>
<td>Bend Test:</td>
<td></td>
</tr>
<tr>
<td>Welder:</td>
<td>G. Holgate</td>
</tr>
<tr>
<td>Approved:</td>
<td>T. Wilkinson</td>
</tr>
</tbody>
</table>
Welding Procedure: TJW06

JOINT PREPARATION: None

WELD RUN LOCATION & SEQUENCE

Notes:
<table>
<thead>
<tr>
<th>Welding Procedure:</th>
<th>TJW07</th>
</tr>
</thead>
<tbody>
<tr>
<td>Date Produced:</td>
<td>10 December 1996</td>
</tr>
<tr>
<td>Material:</td>
<td>C350L0/C450 L0 RHS - AS 1163</td>
</tr>
<tr>
<td>Thickness:</td>
<td>RHS: 4 mm, Plate: 10 mm</td>
</tr>
<tr>
<td>Joint Type:</td>
<td>Table 4.4(D) Type H-C 2b AS/NZS 1554.1</td>
</tr>
<tr>
<td>Welding Machine:</td>
<td>TransMIG 250 Powersource and Transmatic 62 wire feeder from CIGWELD</td>
</tr>
<tr>
<td>Welding Process:</td>
<td>GMAW</td>
</tr>
<tr>
<td>Gas Trade Name:</td>
<td>Argoshield 52</td>
</tr>
<tr>
<td>Gas Composition:</td>
<td>23 % CO₂ , 77 % Ar</td>
</tr>
<tr>
<td>Gas Flow Rate (L/min):</td>
<td>27</td>
</tr>
<tr>
<td>Electrode Trade Name:</td>
<td>Autocraft LW1</td>
</tr>
<tr>
<td>Electrode Classification:</td>
<td>ES4-GM/C W503H (AS 2717.1 (1984))</td>
</tr>
<tr>
<td>Electrode Size (mm):</td>
<td>0.9</td>
</tr>
<tr>
<td>PreHeat Temp:</td>
<td>N/A</td>
</tr>
<tr>
<td>InterPass Temp:</td>
<td>N/A</td>
</tr>
<tr>
<td>InterPass Procedure:</td>
<td>None</td>
</tr>
<tr>
<td>Pass No</td>
<td>Amps</td>
</tr>
<tr>
<td></td>
<td>mm/min</td>
</tr>
<tr>
<td>1</td>
<td>210</td>
</tr>
<tr>
<td>Qualification Requirements</td>
<td>Macro: MTJW07</td>
</tr>
<tr>
<td>Ultrasonic:</td>
<td>Dye Penetration:</td>
</tr>
<tr>
<td>Hardness:</td>
<td>Visual:</td>
</tr>
<tr>
<td>Bend Test:</td>
<td>Stress Rel.:</td>
</tr>
<tr>
<td>Welder: G. Holgate</td>
<td>Checked: G. Towell</td>
</tr>
<tr>
<td>Approved: T. Wilkinson</td>
<td>Date: 10 December 1996</td>
</tr>
</tbody>
</table>
Welding Procedure: TJW07

JOINT PREPARATION: None

Gap $G = 1 - 3$ mm
Landing $F_r = 2$ mm
Angle $\theta = 90 - 120^\circ$

WELD RUN LOCATION & SEQUENCE

Joining RHS walls

Internal sleeve (acting as backing plate)

Notes:
Appendix H

TESTS OF BOLTED MOMENT END PLATE CONNECTIONS

H.1 INTRODUCTION

As an addition to the test series on knee joints, there were three tests on bolted moment end plate connections between beams. This aim was to investigate the behaviour of prototype joints for the apex connection between the two rafters of the portal frame.

H.2 CONNECTION DETAILS

The end plate connection involved a 10 mm plate butt welded to each end of the RHS. Eight high strength fully tensioned M16 bolts (Category 8.8/TF in AS 4100) join each half of the connection. A minimum bolt tension of 95 kN in the installed bolts was ensured by the use of Coronet Load Indicating Washers. The bolt layout and weld detail are shown in Figure H.1. This arrangement was chosen after the tests of Wheeler, Clarke and Hancock (1995). The RHS used in this arrangement was 150 × 50 × 4.0 in either Grade C350 or C450 DuraGal. One sample was welded by an external fabricator. Table H.1 lists the sections tested and the welding procedures used.
Figure H.1: Connection Details for Bolted Moment End Plate

Table H.1: Summary of Bolted Moment End Plate Tests

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Cut from Section</th>
<th>Welding procedure</th>
</tr>
</thead>
<tbody>
<tr>
<td>SJ01</td>
<td>150 × 50 × 4.0 C450</td>
<td>RKE - 138</td>
</tr>
<tr>
<td>SJ02</td>
<td>150 × 50 × 4.0 C450</td>
<td>TJW01</td>
</tr>
<tr>
<td>SJ03</td>
<td>150 × 50 × 4.0 C350</td>
<td>TJW04</td>
</tr>
</tbody>
</table>

H.3 TEST PROCEDURE

The bolted end plate connections were tested by the “parallel plate” method that was used for the plastic bending tests described in Section 2. The test rig is shown diagrammatically in Figure H.2.
Curvature ($\kappa$) was calculated in two ways. The readings of strain on the top and bottom flange of the RHS can be used to determine curvature as indicated in Figure H.3.

$$\tan \kappa = \frac{e_1 + e_2}{d}$$

Figure H.3: Calculation of Curvature from Strains

Alternatively the average curvature over a finite length of the beam can be calculated from displacement values. Assuming a constant curvature between the transducer locations, $\kappa$ can be determined as shown in Figure H.4.
For the plastic bending tests (Chapter 3), the two methods of curvature calculation gave almost identical curvatures until the point of local buckling (Figure 3.12), as was shown previously by Hasan and Hancock (1988). For the connection tests, there is additional rotation at the connection itself, and therefore the curvature is not constant between the loading points. Hence the average curvature (from the deflection readings) of a finite length of the beam including the joint is larger than the “true” curvature of the plain RHS section obtained from the strain gauges.

### H.4 RESULTS

The results of the bolted moment end plate tests are presented in Table H.2. Table H.2 lists the ratio $M_{\text{max}}/M_p$ and two values of the rotation capacity. The first value, $R_{\text{gauge}}$, is based on the curvature obtained from the strain gauges (Figure H.3). This represents the “true” curvature of the plain RHS member. The second value, $R_{\text{defl}}$, is based on the curvature obtained from the deflection (Figure H.4). This is an average curvature of the RHS and connection between the loading points. There is additional rotation at the joint itself since the joint is not perfectly rigid. As a comparison, the results of a simple beam test (no connection) are included. The moment-curvature graph can be seen in Figure H.6.
Both of the higher strength C450 specimens (SJ01 and SJ02) experienced fracture in the HAZ of the RHS on the tension flange. Fracture began in the corner of the RHS and spread around the section. There was evidence of some weld fracture in specimen SJ01 in the corner of the RHS. The lower strength Grade C350 sample (SJ03) failed by local buckling of the web, the same mode of failure as the beam only tests. The local buckle formed midway between the loading plate and the connection itself. The failure modes are illustrated in Figure H.5. For all the beam only tests, the local buckle formed next to the loading plate.

(a) Failure mode for SJ01 and SJ02

(b) Failure mode for SJ03

Figure H.5: Failure Modes
<table>
<thead>
<tr>
<th>Specimen - Section</th>
<th>Welding procedure</th>
<th>$\frac{M_{\text{max}}}{M_p}$</th>
<th>$R_{\text{gauge}}$</th>
<th>$R_{\text{defl}}$</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>SJ01: 150 × 50 × 4.0 C450</td>
<td>RKE-138</td>
<td>1.14</td>
<td>2.1</td>
<td>5.8</td>
<td>Fracture in HAZ on tension flange</td>
</tr>
<tr>
<td>SJ02: 150 × 50 × 4.0 C450</td>
<td>TJW01</td>
<td>1.06</td>
<td>1.4</td>
<td>5.0</td>
<td>Fracture in HAZ on tension flange Weld Failure</td>
</tr>
<tr>
<td>BS02B: 150 × 50 × 4.0 C450</td>
<td>Beam only</td>
<td>1.27</td>
<td>6.6</td>
<td>n/a</td>
<td>Local buckle</td>
</tr>
<tr>
<td>SJ03: 150 × 50 × 4.0 C350</td>
<td>TJW07</td>
<td>1.31</td>
<td>10.1</td>
<td>13.0</td>
<td>Local buckle</td>
</tr>
<tr>
<td>BJ07: 150 × 50 × 4.0 C350</td>
<td>Beam only</td>
<td>1.28</td>
<td>12.9</td>
<td>n/a</td>
<td>Local buckle</td>
</tr>
</tbody>
</table>

Table H.2: Summary of Results for Bolted Moment End Plate Tests

Figure H.6: Moment - Curvature Relationship for RHS Bolted End Plate Tests
H.5 DISCUSSION

It can be observed that the curvature in SJ01 and SJ02 is less at failure than for the simple beam tests. In addition, the curvature of the RHS (based on strain gauges) was considerably less than the average curvature of the beam (based on deflection). This indicates that the connection itself was not perfectly rigid, with additional rotation being provided at the joint itself. The two curvature lines deviate at a low moment value ($M/M_p = 0.6$) which demonstrates a lack of rigidity in the connection. As for the knee joints, there appears to have been a loss of strength in the HAZ of the RHS. Specimens SJ01 and SJ02 showed insufficient ductility to form a plastic hinge but reached the plastic moment. Since some extra rotation has been provided by the connection itself, there is the possibility that the entire connection would perform similarly to a plastic hinge. Since the end plate connection in the portal frame test is located at the apex of the frame, it will not have to support a plastic hinge. Hence, the bolted moment end plate connection is suitable for the portal frame test program, but may not be appropriate in other portal frames where considerable rotation is required at the joint. An alteration to the welding procedure, which does not affect the HAZ as much, may make this connection more suitable.

Specimen SJ03 behaved in a similar manner to the beam only specimens. The curvatures based on strain and deflection are very similar, which indicates that this connection was much stiffer than SJ01 or SJ02. The increased stiffness may be caused by the bolts being more highly tensioned in specimen SJ03. Curvature induced into the end plate by the heat of welding can affect the stiffness of a connection. There was no failure at the weld and the weld experienced very large strains. The difference in rotation capacity between the C350 and C450 bolted end plate connections was much greater than between the C350 and C450 welded knee connections. The reason for this is unknown, but may be attributable to easier welding for the bolted end plate connection with consequently less heat input. The SJ03 connection behaved most satisfactorily.
Appendix I

PHOTOGRAPHS

This Appendix shows a variety of photographs of failed connections, and the portal frames during construction and testing.
Figure I.1: Failure of Stiffened Welded Connection Under Closing Moment

Figure I.2: Failure of Stiffened Welded Connection Under Opening Moment
Figure I.3: Failure of Unstiffened Welded Connection Under Closing Moment

UNSTIFFENED WELDED COMPRESSION
Web buckle across joint
Figure I.4: Failure of Unstiffened Welded Connection Under Opening Moment
Figure I.5: Failure of Bolted End Plate Connection Under Closing Moment

Figure I.6: Failure of Bolted End Plate Connection Under Opening Moment
Figure I.7: Failure of Welded Sleeve Connection Under Closing Moment

Figure I.8: Failure of Welded Sleeve Connection Under Opening Moment
Figure I.9: Failure of Bolted Sleeve Connection Under Closing Moment

Figure I.10: Failure of Bolted Sleeve Connection Under Closing Moment
Figure I.11: The Internal Sleeve

Figure I.12: Failure of Bolted Moment End Plate Connection
Figure I.13: Sleeve After Being Inserted into Rafter
Figure I.14: The Sledgehammer
Figure I.15: Welding the Column Rafter Connection after Insertion of Sleeve
Figure I.16: Welding the Column Rafter Connection after Insertion of Sleeve
Figure I.17: Gravity Load Simulator
Figure I.18: Lateral Restraint Mechanism on North Column
Figure I.19: South Column of Frame 1 at Various Stages During Testing
Appendix J

PARAMETRIC STUDY IN THE FINITE ELEMENT ANALYSIS OF COLD-FORMED RHS BEAMS

J.1 INTRODUCTION

Chapter 6 described the finite element analysis which simulated the bending tests on cold-formed RHS beams, using the package ABAQUS. Chapter 6 only considered the stages of the development of the finite element model that were required to simulate the experiments. However, also of interest are results of the numerical simulations which examined the effects of various parameters, such as material properties, corner radius, and the effect of welding. This appendix gives the results of those analyses. Only a small number of cases were considered, so any conclusions derived from the results must be considered as preliminary.
J.2 EFFECT OF YIELD STRESS

Figures J.1 to J.4 display the results of various ABAQUS analyses to examine the effect of yield stress. The material properties were bilinear elastic - perfectly plastic, at different levels of yield stress. No geometric imperfections were included in the models. For each sized RHS tested, the results are presented in two separate figures. In the first graph of each pair (Figures J.1 and J.3), the curvature is normalised with respect to the plastic curvature ($\kappa_p$). The second graph of each set (Figures J.2 and J.4) shows the (non-normalised) curvature.

The maximum moment never reached the full plastic moment, $M_p$. The yield criterion in the numerical simulation was a Von Mises criterion. Hence the material yielded when the von Mises stress was $f_v$. The major principal stress in the RHS was the longitudinal stress, caused by the bending moment within the beam. But there were other, smaller, non major principal stresses in the section, and the material yielded due to von Mises stress when the major principal stress was slightly less than $f_v$. Consequently, none of sections had a rotation capacity under the normal definition, as the plastic moment was never reached, however it was obvious the beams in Figures J.1 to J.4 demonstrated plastic hinge formation, be it at a moment slightly less than $M_p$. Therefore, a slightly modified definition of $R$ is defined as $R = (\kappa_{97.5}/\kappa_p) - 1$, where $\kappa_{97.5}$ is the curvature at which the moment drops below 97.5% of the maximum moment.

As the yield stress increased, the rotation capacity decreased. The sections became effectively more slender, as element slenderness is a function of both the yield stress and $d/t$. Rotation capacity is indirectly a function of yield stress, since $R = \kappa/\kappa_p$, and $\kappa_p = M_p/\text{EI}$. Hence, if a section with a higher yield stress buckles at the same curvature, $\kappa$, as a section with a lower yield stress, the former will have a lower value of $R$ than a section with a lower yield stress.

The second graph in each pair (Figures J.2 and J.4) is important. The graphs indicate that the curvature at buckling was (almost) invariant under yield stress. The graphs suggest that for a given sized RHS, there was a critical strain at which the section experienced local buckling, regardless of the yield stress.
Figure J.1: Effect of Yield Stress on 150 × 50 × 3 RHS

Figure J.2: Effect of Yield Stress on 150 × 50 × 3 RHS
Figure J.3: Effect of Yield Stress on 100 × 50 × 2 RHS

Figure J.4: Effect of Yield Stress on 100 × 50 × 2 RHS
The analyses were repeated on specimens with imperfections. Sinusoidally varying imperfections (Section 6.2.9.3) with amplitude $d/1000$ and $-b/1000$ were imposed. Figures J.5 and J.6 show the moment - curvature relationships for $150 \times 50 \times 3$ RHS with varying levels of yield stress.

The imperfections reduced the rotation capacity of the sections, compared to the RHS with perfect geometry in Figures J.1 and J.2. Figure J.6 indicates that the curvature at buckling was not significantly affected by yield stress when the same size imperfections were imposed. The concept of a “critical strain” or “critical curvature” for a given section is worth further examination.

Figure J.5: Effect of Yield Stress on $150 \times 50 \times 3$ RHS (imperfections included)
Many of the graphs in Chapter 6 depict the relationship between web slenderness and rotation capacity. The web slenderness was usually defined in the AS 4100 format as \( \lambda_w = \frac{d-2t}{\sqrt{t} \sqrt{f_y/250}} \). Hence for a given sized RHS (\( d \) and \( b \) fixed) the slenderness can be varied by altering either the thickness, \( t \), or the yield stress, \( f_y \). A range of analyses was performed on 150 \( \times \) 75 RHS, to examine whether the slenderness - rotation capacity relationship differed when either the yield stress, or the thickness varied. Elastic - plastic material properties were used, and imperfections with maximum magnitude \( d/1000 \) and \( -b/1000 \) were included in the model.

Table J.1 summarises the results, which are also shown in Figure J.7. The results indicate that for a given set of outer dimensions for an RHS, the slenderness - rotation capacity relationships were highly similar when either the yield stress, or the thickness was varied.
<table>
<thead>
<tr>
<th>Section size</th>
<th>Yield stress $f_y$ (MPa)</th>
<th>Rotation capacity $R_{97.5}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$150 \times 75 \times 3.0$</td>
<td>200</td>
<td>5.97</td>
</tr>
<tr>
<td>$150 \times 75 \times 3.0$</td>
<td>250</td>
<td>4.10</td>
</tr>
<tr>
<td>$150 \times 75 \times 3.0$</td>
<td>300</td>
<td>2.58</td>
</tr>
<tr>
<td>$150 \times 75 \times 3.0$</td>
<td>350</td>
<td>1.94</td>
</tr>
<tr>
<td>$150 \times 75 \times 3.0$</td>
<td>400</td>
<td>1.64</td>
</tr>
<tr>
<td>$150 \times 75 \times 3.0$</td>
<td>450</td>
<td>1.30</td>
</tr>
<tr>
<td>$150 \times 75 \times 3.0$</td>
<td>500</td>
<td>1.02</td>
</tr>
<tr>
<td>$150 \times 75 \times 3.0$</td>
<td>600</td>
<td>0.66</td>
</tr>
<tr>
<td>$150 \times 75 \times 3.0$</td>
<td>700</td>
<td>0</td>
</tr>
<tr>
<td>$150 \times 75 \times 2.5$</td>
<td>450</td>
<td>0.46</td>
</tr>
<tr>
<td>$150 \times 75 \times 3.0$</td>
<td>450</td>
<td>1.30</td>
</tr>
<tr>
<td>$150 \times 75 \times 3.5$</td>
<td>450</td>
<td>2.20</td>
</tr>
<tr>
<td>$150 \times 75 \times 4.0$</td>
<td>450</td>
<td>4.36</td>
</tr>
<tr>
<td>$150 \times 75 \times 4.5$</td>
<td>450</td>
<td>5.33</td>
</tr>
</tbody>
</table>

Table J.1: Summary of Results for $150 \times 75$ RHS with Varying Slenderness

Figure J.7: Results for $150 \times 75$ RHS with Varying Slenderness
J.3 EFFECT OF STRAIN HARDENING AND PLASTIC PLATEAU

The effect of strain hardening material properties was examined. Figure J.8 shows three typical stress - strain curves: elastic - plastic, elastic - strain hardening, and elastic - plastic - strain hardening. Two parameters were investigated. For elastic - strain hardening material properties, the effect of $\alpha$, where $\alpha$ is the ratio of the strain hardening modulus to the elastic modulus. For the elastic - plastic - strain hardening material, the parameter $\zeta$, which defines the strain at the end of the plastic plateau, was examined. When $\zeta$ was varied, it was assumed that $\alpha = 0.02$. For each case, $f_y = 450$ MPa. Geometric imperfections were not included in any of the analyses. Typical moment - curvature graphs are displayed in Figures J.9 to J.14. An experimental result is also given on each figure for comparison. The code in the legend of each figure refers to the RHS dimensions and either 10000$\alpha$ or $\zeta$. Hence 150_50_3_alpha_001 refers to a $150 \times 50 \times 3$ RHS, with $\alpha = 0.0001$, and 150_50_3_zeta_10 refers to a $150 \times 50 \times 3$ RHS, with $\zeta = 10$.

![Figure J.8: Different Material Properties](image.png)
There were convergence problems after buckling for some in this set of analyses, but buckling did occur in all specimens at approximately the same curvature.

Figure J.9: Effect of Strain Hardening on 150 × 50 × 2 RHS

Figure J.10: Effect of Strain Hardening on 150 × 50 × 3 RHS
There were convergence problems after buckling for some in this set of analyses, but buckling did occur in all specimens at approximately the same curvature.
Figure J.13: Effect of Plastic Plateau on 150 × 50 × 3 RHS

Figure J.14: Effect of Plastic Plateau on 100 × 50 × 2 RHS
Altering the material properties had an insignificant effect on the most slender section analysed, (150 × 50 × 2 RHS: Figures J.9 and J.12). The 150 × 50 × 2 section had a low rotation capacity, and the cross section did not fully yield. Since the section did not experience much yielding before local buckling occurred, changing the post - yield material properties had very little effect on the curvature at buckling.

Changing the material properties had a notable impact on the stockier sections, which exhibited plasticity, as a significant proportion of the section experienced the change in material properties before local buckling was initiated. Increasing the strain hardening modulus (ie increasing $\alpha$) altered the response in two ways as demonstrated in the behaviour of a 150 × 50 × 3 RHS in Figure J.10:

- The maximum moment increased, since the yielded portions of the section experienced a stress greater than $f_y$ due to strain hardening,
- The rotation capacity increased as the higher strain hardening modulus provided greater resistance to local instability. Lay (1966) demonstrated a similar link between strain hardening and rotation capacity when considering flange local buckling of I-sections.

As $\zeta$ increased, so did the length of the yield plateau. Hence large values of $\zeta$ ($\zeta > 5$) have no effect on the performance of sections with low values of rotation capacity (say, $R < 3$), since no strain hardening had occurred before the specimen buckled.

Figures J.13 (150 × 50 × 3 RHS) and J.14 (100 × 50 × 2 RHS) indicate that changing $\zeta$ gave rise to two distinct behaviour patterns. For the specimens with $\zeta > 10$, the pre-buckling behaviour was identical, since the strain hardening had not commenced at the onset of buckling, and hence the material properties were the same before buckling. The post-buckling behaviour was affected by $\zeta$, depending on the curvature at which strain hardening commenced. For the lower values of $\zeta$, strain hardening commenced before buckling occurs. It can seen that strain hardening commenced as the moment started to rise above the plastic plateau. Since strain hardening had commenced, the sections had greater resistance to local buckling, compared to the sections with $\zeta > 10$ which did not experience the benefits of strain hardening before buckling. As $\zeta$ decreased, strain hardening commenced at lower strains, and a greater proportion of the section experienced strain hardening at a given curvature, when compared to a section with a higher $\zeta$. As a consequence, the moment is greater.
Once $\zeta$ was below some critical value, varying $\zeta$ (with a constant value of $\alpha = 0.02$) did not affect the rotation capacity to a great extent. For example, in Figure J.14, the $100 \times 50 \times 2$ specimens with $\zeta = 1, 2, \text{and} 5$, buckled at similar curvature, even though strain hardening had been initiated at differing values of curvature.

Most of the ABAQUS specimens exhibited rotation capacity that was considerably greater than the values observed experimentally for a similarly sized RHS. Geometric imperfection were not considered in this section, and, as described in Chapter 6, geometric imperfections are required in the model to more accurately model the observed rotation capacities.

### J.4 EFFECT OF CORNER RADIUS

A major aim of this thesis is to investigate the relationship between flange and web slenderness, and rotation capacity. Section 2.4.2 and Figure 2.15 outlined the definition of “slenderness” in various design codes. With regards to RHS, two definitions of plate width are common:

- the “flat width” definition ($d-2r_c$ or $b-2r_c$), used in AISC LRFD and CSA-S16.1,
- the “clear width” ($d-2t$ or $b-2t$), used in AS 4100.

Eurocode 3 defines the width as $d-1.5t$ or $b-1.5t$, which is similar to the flat width definition, assuming that $r_c = 1.5t$. For hot-formed RHS, the corners are reasonably tight, and $r_c = 1.5t$ is a suitable approximation, but cold-formed RHS have larger corner radii, typically, $r_c = 2.0t - 2.5t$.

The experimental results did not provide any conclusive information about the influence of the corner radii. Therefore, a series of numerical analyses was performed to examine the effect of the size of the corner radius.

For all cases examined in this section it is assumed that each corner radius is a quarter circle, and that at the end of the corner, the tangent is concurrent with the flat face of the RHS, as indicated in Figure J.15(a). In some cases it has been observed that the corner of the RHS was not a quadrant of a circle, and the tangent was not aligned with the flat face of the RHS as shown in Figure J.15(b). The second case is not considered.
The effect of corner radius size was examined by a set of numerical analyses. Sections considered were either 150 × 150 ($d/b = 1.0$), 150 × 75 ($d/b = 2.0$), or 150 × 50 ($d/b = 3.0$), with a variety of thicknesses, and an imperfection size of: $d/1000$ (for the web), and $b/1000$ (for the flange). The material properties assumed were those for specimen BS02 (refer to Appendix C). For each section, five different sizes of corner radius were considered, from $r = 1.1t$ to $r = 3.0t$. The results are summarised in Table J.2. There is an additional column, which compares the maximum and minimum values of curvature $\kappa / \kappa_0$ for each different radius for a given sized sections, to provide a measure of the relative effect of corner radius for that section.

The results are displayed in Figure J.16. Two sets of web slenderness - rotation capacity relationships are given in Figure J.16: the first shows the clear width definition of slenderness ($\lambda_w = (d-2t)/t\sqrt{f_y/250}$, as defined in AS 4100); the other set uses the flat width definition of slenderness ($\lambda_w = (d-2r_c)/t\sqrt{f_y/250}$ - which is similar to the definition in AISC LRFD, except the term $(f_y/250)$ is used, rather than $(f_y/E)$, so that both sets of results can be compared easily).
### Table J.2: Summary of Results for Different Corner Radius Size

<table>
<thead>
<tr>
<th>Section</th>
<th>( r_e = 1.1t )</th>
<th>( r_e = 1.5t )</th>
<th>( r_e = 2.0t )</th>
<th>( r_e = 2.5t )</th>
<th>( r_e = 3.0t )</th>
<th>( (\kappa_t/\kappa_p)_{\text{min}} )</th>
<th>( (\kappa_t/\kappa_p)_{\text{max}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>150 × 50 × 2.0</td>
<td>0.46</td>
<td>0.45</td>
<td>0.47</td>
<td>0.47</td>
<td>0.49</td>
<td>1.03</td>
<td></td>
</tr>
<tr>
<td>150 × 50 × 2.5</td>
<td>1.43</td>
<td>1.44</td>
<td>1.38</td>
<td>1.41</td>
<td>1.42</td>
<td>1.03</td>
<td></td>
</tr>
<tr>
<td>150 × 50 × 3.0</td>
<td>2.97</td>
<td>2.95</td>
<td>2.89</td>
<td>2.89</td>
<td>2.92</td>
<td>1.02</td>
<td></td>
</tr>
<tr>
<td>150 × 50 × 3.5</td>
<td>5.11</td>
<td>5.05</td>
<td>4.93</td>
<td>4.90</td>
<td>4.94</td>
<td>1.04</td>
<td></td>
</tr>
<tr>
<td>150 × 50 × 4.0</td>
<td>7.37</td>
<td>7.37</td>
<td>7.29</td>
<td>7.21</td>
<td>7.17</td>
<td>1.02</td>
<td></td>
</tr>
<tr>
<td>150 × 50 × 4.5</td>
<td>9.73</td>
<td>9.61</td>
<td>9.30</td>
<td>9.17</td>
<td>8.94</td>
<td>1.08</td>
<td></td>
</tr>
<tr>
<td>150 × 90 × 2.5</td>
<td>0.37</td>
<td>0.36</td>
<td>0.35</td>
<td>0.0</td>
<td>0.30</td>
<td>1.22</td>
<td></td>
</tr>
<tr>
<td>150 × 90 × 3.0</td>
<td>0.97</td>
<td>0.97</td>
<td>0.98</td>
<td>0.97</td>
<td>1.01</td>
<td>1.02</td>
<td></td>
</tr>
<tr>
<td>150 × 90 × 3.5</td>
<td>1.93</td>
<td>1.97</td>
<td>1.99</td>
<td>1.97</td>
<td>1.81</td>
<td>1.06</td>
<td></td>
</tr>
<tr>
<td>150 × 90 × 4.0</td>
<td>3.16</td>
<td>3.19</td>
<td>3.17</td>
<td>3.17</td>
<td>3.23</td>
<td>1.02</td>
<td></td>
</tr>
<tr>
<td>150 × 90 × 4.5</td>
<td>4.67</td>
<td>4.83</td>
<td>4.79</td>
<td>4.84</td>
<td>4.96</td>
<td>1.05</td>
<td></td>
</tr>
<tr>
<td>150 × 90 × 5.0</td>
<td>6.61</td>
<td>6.64</td>
<td>6.33</td>
<td>6.58</td>
<td>6.73</td>
<td>1.05</td>
<td></td>
</tr>
<tr>
<td>150 × 90 × 5.5</td>
<td>8.64</td>
<td>8.58</td>
<td>8.55</td>
<td>8.58</td>
<td>8.61</td>
<td>1.01</td>
<td></td>
</tr>
<tr>
<td>150 × 150 × 4.0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.51</td>
<td>0.56</td>
<td>1.04</td>
<td></td>
</tr>
<tr>
<td>150 × 150 × 4.5</td>
<td>1.09</td>
<td>1.10</td>
<td>1.09</td>
<td>1.11</td>
<td>1.08</td>
<td>1.01</td>
<td></td>
</tr>
<tr>
<td>150 × 150 × 5.0</td>
<td>1.67</td>
<td>1.67</td>
<td>1.69</td>
<td>1.69</td>
<td>1.76</td>
<td>1.03</td>
<td></td>
</tr>
<tr>
<td>150 × 150 × 5.5</td>
<td>2.12</td>
<td>2.11</td>
<td>2.16</td>
<td>2.43</td>
<td>2.29</td>
<td>1.10</td>
<td></td>
</tr>
<tr>
<td>150 × 150 × 6.0</td>
<td>3.02</td>
<td>2.91</td>
<td>2.88</td>
<td>2.92</td>
<td>3.11</td>
<td>1.06</td>
<td></td>
</tr>
<tr>
<td>150 × 150 × 7.0</td>
<td>4.44</td>
<td>4.47</td>
<td>4.66</td>
<td>4.69</td>
<td>4.97</td>
<td>1.10</td>
<td></td>
</tr>
<tr>
<td>150 × 150 × 8.0</td>
<td>7.07</td>
<td>7.08</td>
<td>7.08</td>
<td>7.17</td>
<td>7.59</td>
<td>1.06</td>
<td></td>
</tr>
</tbody>
</table>

- **Average**: 1.05
- **Standard deviation**: 0.048

Table J.2: Summary of Results for Different Corner Radius Size
Several important observations can be made:

Changing the radius size altered the rotation capacity. Hence, the clear width definition of slenderness is not strictly correct, as members with the same clear width slenderness (but different radius size) had varying rotation capacities. However, the changes in rotation capacity for a given section were generally small, with an average of 5 % variation as the radius changed. For 16 out of the 20 different RHS considered, the variation was less than or equal to 6 %.

For a given aspect ratio, the spread of results in the web slenderness - rotation capacity relationship was smaller when using the clear width slenderness, compared to the flat width slenderness. This suggests that the clear width slenderness definition gives more precise results when considering the correlation between rotation capacity and slenderness, particularly if the size of the corner radius is unknown.
Most unexpectedly, for RHS with \( d/b = 3.0 \) and thicknesses \( t \geq 3.5 \text{ mm} \), the rotation capacity increased as the flat width slenderness increased (corner radius decreased), indicating that the flat width slenderness is inappropriate for this case.

No definitive conclusions can be made from the small range of sections considered, but the limited results suggest that the trend between web slenderness and rotation capacity, is more accurately defined when the clear width definition of slenderness is used, rather than the flat width definition.

**J.5 EFFECT OF BEAM LENGTH**

All the analyses up to this stage have used the same longitudinal dimensions, as defined in Figure J.17. A small series of analyses was carried out to examine the effect of changing the length, \( L_1 \). Two sections were considered, 100 × 50 × 2 and 150 × 50 × 4. Sinusoidally varying imperfections were included in the flanges and webs, and the material properties assumed were those of specimen BS02 (refer to Appendix C). Because of symmetry, the ABAQUS model only considers half the length of the beam, so three values of \( L_1/2 \) were considered: \( L_1/2 = 400 \text{ mm} \) (default), 500 mm, and 600 mm. \( L_2 \) was increased, so that the value \( (L_2 - L_1) \) remained constant. The different moment - curvature graphs are given in Figures J.18 and J.19, and the code in the legend refers to the RHS dimensions, and the value of \( L_1/2 \).

![Figure J.17: Simplified Representation the Plastic Bending Tests](image-url)
Figure J.18: Effect of Specimen Length of 100 × 50 × 2 RHS

Figure J.19: Effect of Specimen Length of 150 × 50 × 4 RHS
Figures J.18 and J.19 indicate that there was no significant difference in the rotation capacity as the beam length increased. The post-buckling response was slightly different in each case, since the relative size of the buckle with respect to the beam length changed, and the curvature was an averaged value over the beam length (see Section 6.2.9.5).

It is not unexpected that the behaviour was barely affected by beam length. One symmetry condition imposed on the RHS was that out-of-plane deflections were prevented, so there was no possibility of the interaction between local and flexural-torsional (lateral) buckling.
J.6  EFFECT OF WELDING

J.6.1  INTRODUCTION

In Chapter 6, geometric imperfections were imposed on the RHS beams, and had a significant effect on the rotation capacity. The type of imperfections imposed were bow-out imperfections, the amplitude of which varied sinusoidally along the length of the RHS beam. However, this was not the shape of the imperfections that were measured. The measured imperfection profiles, given in Appendix F, showed that the bow-out imperfection was approximately constant along the length of the beam, but there were imperfections near the loading plate, caused by the welding of the plate to the RHS, that could not be measured. It was thought that the imperfections caused by the welding were a major factor in causing local buckling, and the sinusoidally varying imperfection imposed attempted to simulate the effect of the “real” imperfections. Hence, some preliminary analyses were performed, to investigate how the process of welding induced imperfections into a specimen, and how the rotation capacity was affected. The process of welding was simulated by imposing a heat differential. The weld material was assumed to be at a high temperature, and then it was cooled, which induced imperfections and stresses into the RHS beam, prior to the bending of the beam. Various changes in temperature were considered.

J.6.2  ABAQUS PROCEDURE

A numerical analysis in ABAQUS is broken into steps. In all the previous numerical simulations described, there has only been one step, the enforced deflection of the beam, representing the vertical loading on the beam. To simulate the weld, an extra step was inserted before the bending step. An initial temperature was applied to the nodes that defined the weld, and a final temperature was defined. The final temperature was 298 K (25°C), while the initial temperature was some higher value. There was no data available on the temperature the weld metal may have been when it was deposited, as it was dependent on many factors, so temperature differentials of 400°C, 600°C, 800°C, and 1200°C were considered. A coefficient of thermal expansion of $11.7 \times 10^{-6} \degree\text{C}^{-1}$ was assigned as an additional material property.
J.6.3 IMPERFECTIONS INDUCED

As the weld metal cooled, it contracted, and deformed the shape of the RHS. A typical deformed shape, with the deflections magnified, is shown in Figure J.20. Most noticeable is that the deformations were localised close to the weld, within a length of about $d/3$ or $d/2$ from the weld.

![Diagram: Loading plate not shown. RHS web and flange have been deformed where the plates were welded to the beam. Deformations not to scale.](image)

Figure J.20: Imperfections Induced by Welding (Imperfections Magnified)

Figure J.20 gives a spurious impression of the nature of the imperfections induced by welding, as the deformations caused by the cooling of the weld have been magnified, but the initial imperfections have not been magnified by the same factor. Figure J.21 shows the cross section of the RHS adjacent to the weld. The initial imperfection and the weld induced deformations are magnified by the same amount. The initial imperfection sizes were $+d/500$ and $-b/500$, and the temperature change was $400^\circ$. The deformation caused by the cooling of the weld was much smaller than the amplitude of the sinusoidally varying “bow-out” initial imperfections.
**J.6.4 RESULTS**

The results of the ABAQUS simulations are summarised in Table J.3 and the moment - curvature relationships are shown in Figure J.22. Two types of imperfections were considered; the first where the bow-out imperfection was constant along the length of the beam, and the other in which the amplitude of the bow-out varied sinusoidally.
<table>
<thead>
<tr>
<th>Section</th>
<th>Type of Imperfection</th>
<th>Imperfection Size</th>
<th>Temperature</th>
<th>$R$</th>
</tr>
</thead>
<tbody>
<tr>
<td>150 × 50 × 3.0</td>
<td>None</td>
<td>0</td>
<td>0°C</td>
<td>3.97</td>
</tr>
<tr>
<td>150 × 50 × 3.0</td>
<td>None</td>
<td>0</td>
<td>400°C</td>
<td>3.93</td>
</tr>
<tr>
<td>150 × 50 × 3.0</td>
<td>Constant bow-out</td>
<td>1/500</td>
<td>0°C</td>
<td>3.97</td>
</tr>
<tr>
<td>150 × 50 × 3.0</td>
<td>Constant bow-out</td>
<td>1/500</td>
<td>400°C</td>
<td>3.93</td>
</tr>
<tr>
<td>150 × 50 × 3.0</td>
<td>Constant bow-out</td>
<td>1/500</td>
<td>600°C</td>
<td>3.63</td>
</tr>
<tr>
<td>150 × 50 × 3.0</td>
<td>Constant bow-out</td>
<td>1/500</td>
<td>800°C</td>
<td>3.46</td>
</tr>
<tr>
<td>150 × 50 × 3.0</td>
<td>Constant bow-out</td>
<td>1/500</td>
<td>1200°C</td>
<td>2.83</td>
</tr>
<tr>
<td>150 × 50 × 3.0</td>
<td>Sinusoidal bow-out</td>
<td>1/500</td>
<td>0°C</td>
<td>2.32</td>
</tr>
<tr>
<td>150 × 50 × 3.0</td>
<td>Sinusoidal bow-out</td>
<td>1/250</td>
<td>0°C</td>
<td>1.61</td>
</tr>
<tr>
<td>150 × 50 × 3.0</td>
<td>Sinusoidal bow-out</td>
<td>1/500</td>
<td>400°C</td>
<td>2.35</td>
</tr>
<tr>
<td>150 × 50 × 3.0</td>
<td>Sinusoidal bow-out</td>
<td>1/500</td>
<td>600°C</td>
<td>2.29</td>
</tr>
<tr>
<td>150 × 50 × 3.0</td>
<td>Sinusoidal bow-out</td>
<td>1/500</td>
<td>800°C</td>
<td>2.35</td>
</tr>
<tr>
<td>150 × 50 × 3.0</td>
<td>Sinusoidal bow-out</td>
<td>1/500</td>
<td>1200°C</td>
<td>2.30</td>
</tr>
<tr>
<td>Experimental</td>
<td>BS03A</td>
<td></td>
<td></td>
<td>2.7</td>
</tr>
<tr>
<td>Experimental</td>
<td>BS03B</td>
<td></td>
<td></td>
<td>2.3</td>
</tr>
<tr>
<td>Experimental</td>
<td>BS03C</td>
<td></td>
<td></td>
<td>2.9</td>
</tr>
</tbody>
</table>

Table J.3: Summary of Results Examining Effect of Weld Temperature
Figure J.22: Moment - Curvature Graphs for Imperfection Types and Weld Temperatures

J.6.5 DISCUSSION

It is clear that simulating a temperature change in the weld affects the rotation capacity, but only slightly. For sections with the initial constant bow-out imperfection, only the largest temperature change of 1200°C made a significant impact on rotation capacity, however the rotation capacity still exceeded the experimentally observed value.

The introduction of sinusoidally varying imperfections had a more significant impact on rotation capacity than inclusion of temperature effects. When the temperature changed was enforced on a section which already had sinusoidally varying imperfections, the effect on rotation capacity was insignificant.

An uncertainty in the analysis was the value of temperature change to be prescribed. The temperature of the weld metal is dependent on the welding procedure, consumables etc.
The conclusion to be drawn is that the amplitude of the sinusoidally varying bow-out imperfections is a critical factor in simulating the experimental results. The effect of the weld cooling slightly changes the magnitude of the initial imperfections.

The ABAQUS simulations described above are only preliminary, and there is large scope for further analyses to examine the effect of welding: not only how imperfections and stresses induced can affect the stability of a section, but also how the heat affects material properties such as ductility.

**J.7 CONCLUSION**

This chapter considered various parameters in the finite element analysis of RHS beams.

Different idealised material properties were considered: elastic - plastic, elastic - strain hardening, and elastic - plastic - strain hardening. The most important observation was that increasing strain hardening modulus increased the rotation capacity.

Several RHS sections, which varied only by the size of the corner radius, were analysed, to consider whether the flat width or clear width definition of slenderness was more appropriate to use to describe the slenderness - rotation capacity relationship. The limited results suggest that the trend between web slenderness and rotation capacity, is described better when the clear width definition of slenderness is used, rather than the flat width definition.

Some preliminary analyses were made to examine the effect of the welding of the loading plates to the RHS. The weld was initially hot, and as the weld metal cooled, it induced deformations into the RHS, but the deformations induced were considerably smaller than the typical initial imperfections. The amplitude of the sinusoidally varying bow-out imperfections is a more critical factor in simulating the experimental results.
Appendix K

VITA

The author, TIM WILKINSON, was born in Sydney, Australia, on February 10, 1970, the son of Jeffrey John Wilkinson and Joan Adelia Roche, and descendent of Henry Kable and Susannah Holmes, convicts on the First Fleet, which arrived in Australia in 1788.

After completing secondary education at St Aloysius College, Milsons Point, Sydney, Australia, in 1987, he entered The University of Sydney, Sydney, Australia, obtaining a Bachelor of Science in 1991, and a Bachelor of Engineering in Civil Engineering with First Class Honours in 1993. In 1994, the author worked for the Australian Institute of Steel Construction. In 1995, he returned to The University of Sydney to commence his PhD in the Department of Civil Engineering. The author also received a Master of Arts in Pure Mathematics in 1997 from The University of Sydney.
“Nope. It’s oak.”

Colonel Henry Blake, describing his desk, from the television show *M*A*S*H*.