

Cyclic Behaviour of Carbon Steel and Stainless Steel Tubular Members

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Abstract

Concentrically braced frames are a common form of earthquake resistant structure. Performance of the structure is largely dependent on the ability of the key dissipative components, in this case the diagonal bracing members, to undergo significant inelastic deformations. Whilst many earlier studies have examined the hysteretic response of bracing members, comparatively less attention has been given to the ultimate behaviour and failure conditions. There are also significant uncertainties in existing models for predicting the ductility capacity of braces owing to their semi-empirical nature as well as the scatter of test results. This research project examines the cyclic behaviour of tubular braces made of a familiar structural material, carbon steel, and an increasing popular alternative structural material, stainless steel, which is known for its high tensile ductility.

As part of the current study, laboratory tests were performed on hot-rolled carbon steel, cold-formed carbon steel and cold-formed austenitic stainless steel hollow section members and materials coupon cut from them. A total of 24 tensile coupon tests, 62 cyclic material tests and 16 cyclic member tests were conducted. Strain-life relationships of the materials under low and extremely low cycle fatigue regimes were established from the results of the cyclic material tests. These data were also used for calibrating material cyclic hardening models, which were incorporated in numerical models of hollow section members. These models, verified against the results of the cyclic member tests from this study and other research programmes, were employed in parametric studies to investigate the influence of member geometry and material properties on the behaviour of the bracing members.

Although the three materials exhibit similar strain-life relationships, cold-formed stainless steel members perform better in terms of displacement ductility and energy dissipation, which is due to the cyclic hardening and higher post-yield stiffness of the stainless steel material. Implications of these findings on the design of earthquake resistant concentrically braced frames are discussed and design guidance for stainless steel bracing members is proposed.

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Notation

A	<i>Cross-sectional area</i>
A_g	<i>Gross cross-sectional area</i>
A_u	<i>Minimum cross-sectional area after fracture</i>
A_0	<i>Initial cross-sectional area</i>
b	<i>Fatigue strength exponent</i>
b	<i>Width of the flat area of the longer face of a hollow section</i>
b_{iso}	<i>Rate at size of the yield surface changes</i>
B	<i>Width of a hollow section</i>
c	<i>Fatigue ductility exponent</i>
c_p	<i>Constant</i>
C_{kin}	<i>Coefficient determining the maximum change in backstress</i>
C_m	<i>Ductile damage fraction</i>
C_s	<i>Empirical constant</i>
C'	<i>Material constant</i>
d	<i>Width of the flat area of the shorter face of a hollow section</i>
$d\lambda$	<i>Positive scalar factor</i>
$d\mu$	<i>Proportionality scalar constant determined by the yield criterion</i>
D	<i>Damage index</i>
D	<i>Depth of a hollow section</i>
E	<i>Young's modulus</i>
$E_{0.2}$	<i>Tangent stiffness at 0.2% proof strength</i>

$f_{0.2}$	<i>0.2% proof strength</i>
$f_{0.2,b}$	<i>0.2% proof strength of sheet material</i>
$f_{0.2,c}$	<i>0.2% proof strength of corner material</i>
$f_{1.0}$	<i>1.0% proof strength</i>
f_{cr}	<i>Plate critical buckling stress</i>
f_u	<i>Material ultimate strength</i>
$f_{u,act}$	<i>Actual material ultimate strength</i>
$f_{u,b}$	<i>Nominal ultimate strength before forming</i>
f_y	<i>Material yield strength</i>
$f_{y,a}$	<i>Average yield strength of a cold-formed section</i>
$f_{y,act}$	<i>Actual material yield strength</i>
$f_{y,b}$	<i>Nominal yield strength before cold-forming</i>
$f_{y,max}$	<i>Estimated maximum value of material yield strength</i>
$f_{y,nom}$	<i>Nominal material yield strength</i>
F	<i>Axial load</i>
F_b	<i>Initial buckling resistance</i>
$F_{c,\mu\Delta=2}$	<i>Compressive resistance at displacement ductility of 2</i>
$F_{c,\mu\Delta=4}$	<i>Compressive resistance at displacement ductility of 4</i>
$F_{t,max}$	<i>Maximum tensile resistance of a section</i>
$F_{u,cal}$	<i>Calculated ultimate tensile resistance of a section</i>
F_y	<i>Yield load of a section</i>
$F_{y,cal}$	<i>Calculated yield resistance of a section</i>
$F(\sigma_{ij})$	<i>Yield function</i>
h	<i>Depth of a section</i>
k	<i>Constant for methods of forming</i>

K	<i>Effective length factor</i>
K	<i>Strength coefficient of monotonic stress-strain curve</i>
K'	<i>Strength coefficient of cyclic stress-strain curve</i>
L	<i>Length of a member</i>
L_H	<i>Distance between the plastic hinges that form near the ends of a member</i>
L_{pl}	<i>Length of a plastic hinge</i>
m	<i>Material constant</i>
M_{pl}	<i>Plastic moment capacity of a section</i>
n	<i>Monotonic strain hardening exponent</i>
n	<i>Number of 90° bends in the cross-section with an internal radius $r_i \leq 5t$</i>
n_i	<i>Number of cycles of loading at a particular strain (stress) amplitude</i>
n^{RO}	<i>Strain hardening exponent representing a curve that passes through zero stress and 1.0% proof stress</i>
n'	<i>Cyclic strain hardening exponent</i>
$n_{0.2,1.0}^{RO}$	<i>Strain hardening exponent representing a curve that passes through 0.2% and 1.0% proof stresses</i>
$n_{0.2,u}^{RO}$	<i>Strain hardening exponent representing a curve that passes through 0.2% proof stress and ultimate stress</i>
N^*	<i>Number of cycles to failure adjusted for mean stress/strain effects</i>
N_{cr}	<i>Elastic critical buckling load</i>
N_f	<i>Number of cycles to failure</i>
N_{fi}	<i>Number of cycles to failure at a particular strain (or stress) amplitude</i>
N_t	<i>Transition life</i>
q	<i>Behaviour factor</i>
Q_∞	<i>Maximum change in the size of the yield surface</i>

$Q(\sigma_{ij})$	<i>Plastic potential function</i>
r	<i>Radius of curvature</i>
r	<i>Radius of gyration</i>
r_i	<i>Internal corner radius</i>
R	<i>Force reduction factor</i>
s_{ij}	<i>Deviatoric stress tensor</i>
t	<i>Thickness</i>
u_f	<i>True tensile toughness</i>
u_i	<i>Plastic hysteresis energy of a small element</i>
U_c	<i>Complementary strain energy</i>
V	<i>Base shear</i>
V_d	<i>Design base shear</i>
V_e	<i>Elastic base shear</i>
V_i	<i>Volume of a small element with strain amplitudes large enough to contribute to the fatigue process</i>
V_t	<i>Total volume of material under cyclic loading</i>
V_y	<i>Actual resistance of a structure</i>
w	<i>Width of a section</i>
W	<i>Hysteretic energy dissipation</i>
W_f	<i>Hysteretic energy dissipation to fracture</i>
x	<i>Distance along a member</i>
x	<i>Position in section depth</i>
y	<i>Distance along a member</i>
y	<i>Position in section depth</i>
α	<i>Backstress</i>

α	<i>Imperfection factor in column buckling curve</i>
α_i	<i>Value of backstress corresponding to ϵ_i^p</i>
α_{ij}	<i>Backstress tensor</i>
α'	<i>Material constants</i>
δ	<i>Axial displacement</i>
δ_1	<i>Normalised compressive displacement</i>
δ_2	<i>Normalised tensile displacement</i>
δ_c	<i>Applied compressive displacement</i>
δ_f	<i>Accumulated fracture ductility</i>
δ_t	<i>Applied tensile displacement</i>
δ_y	<i>Yield displacement</i>
Δ	<i>Lateral deflection at mid-length</i>
Δ_u	<i>Maximum top displacement of a structure</i>
Δ_y	<i>Top displacement of a structure which responses elastically under V_y</i>
ΔW	<i>Plastic hysteresis energy per cycle</i>
$\Delta\epsilon/2$	<i>Total strain amplitude</i>
$\Delta\epsilon_e/2$	<i>Elastic strain amplitude</i>
$\Delta\epsilon_{hyp}/2$	<i>Hypothetical uniform strain amplitude</i>
$\Delta\epsilon_{max}$	<i>Maximum plastic strain range</i>
$\Delta\epsilon_p/2$	<i>Plastic strain amplitude</i>
$\Delta\epsilon_s/2$	<i>Surface strain amplitude</i>
$\Delta\sigma/2$	<i>Stress amplitude</i>
ϵ	<i>Coefficient dependant on material yield stress</i>
ϵ	<i>Total strain</i>

ϵ_a	<i>Axial component of strains</i>
ϵ_b	<i>Bending component of strains</i>
ϵ_c	<i>Applied compressive strain</i>
ϵ_e	<i>Elastic strain</i>
ϵ_f	<i>Tensile ductility</i>
ϵ_{ij}	<i>Strain tensor</i>
ϵ_L	<i>Percentage elongation at fracture</i>
ϵ_m	<i>Mean strain</i>
ϵ_p	<i>Plastic strain</i>
ϵ_{pu}	<i>Plastic strain at ultimate strength</i>
$\epsilon_{p,c}$	<i>Maximum compressive plastic strain</i>
$\epsilon_{p,t}$	<i>Maximum tensile plastic strain</i>
ϵ_t	<i>Applied tensile strain</i>
$\epsilon_{t0.2}$	<i>Total strain at 0.2% proof stress</i>
$\epsilon_{t1.0}$	<i>Total strain at 1.0% proof stress</i>
ϵ_y	<i>Yield strain</i>
ϵ^p	<i>Equivalent plastic strain</i>
ϵ_i^p	<i>Equivalent plastic strain corresponding to σ_i^0 or α_i</i>
ϵ_{ij}^p	<i>Plastic strain tensor</i>
ϵ_p^0	<i>Value of plastic strain when the compressive stress reduces to zero</i>
ϵ'_f	<i>Fatigue ductility coefficient</i>

γ	<i>Rate at which the backstress changes</i>
γ_{ov}	<i>Material overstrength factor</i>
κ	<i>Curvature</i>
κ	<i>Material constant representing the size of the yield surface</i>
λ	<i>Biaxiality ratio</i>
$\bar{\lambda}$	<i>Dimensionless global slenderness</i>
$\bar{\lambda}_0$	<i>Plateau length of column buckling curve</i>
ν	<i>Poisson's ratio</i>
μ_{Δ}	<i>Displacement ductility</i>
θ	<i>Angle of rotation</i>
θ_f	<i>Rotation to fracture</i>
σ	<i>Stress</i>
σ_i	<i>Value of stress corresponding to ϵ_i^p</i>
σ_{ij}	<i>Stress tensor</i>
σ_m	<i>Mean stress</i>
σ_{max}	<i>Maximum stress value accounting for mean stress effect</i>
$\sigma _0$	<i>Yield stress at zero equivalent plastic strain</i>
σ^s	<i>Average of the first and the last data points of stress for the non-linear kinematic hardening model</i>
σ^0	<i>Size of yield surface</i>
σ_i^0	<i>Size of yield surface in the i^{th} cycle</i>
σ_i^c	<i>Maximum compressive stress in elastic range in the i^{th} cycle</i>
σ_i^t	<i>Maximum tensile stress in elastic range in the i^{th} cycle</i>

σ'_f	<i>Fatigue strength coefficient</i>
ω_0	<i>Amplitude of initial imperfection</i>

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Chapter 1

Introduction

1.1 Background

Concentrically braced frames are efficient lateral load resisting structures. Besides being applied to resist static loading, they are often used in earthquake resistant structures, due to their high lateral stiffness. However, the buckling and fracture of the diagonal bracing members, which are the dissipative elements and major source of lateral stiffness, often limit their seismic resistance. Buckling can be controlled by restricting the slenderness of the members while fracture can be delayed by using more ductile material or minimising the strains that the members experience. This research project focuses on square and rectangular steel hollow section bracing members. The influence of member dimensions and material properties on the response to cyclic loading is examined experimentally and numerically.

1.2 Earthquake resistant design

Seismic actions are often the most violent forces that structures have to resist. Buildings of high importance are protected from earthquakes in imaginative ways. The 508 m tall Taipei 101 Tower in Taiwan is installed with a 660 T mass damper on the 88th floor, which swings to offset the vibrations caused by earthquakes and strong gusts. The Los Angeles City Hall in the U.S.A. has been retrofitted with base isolation,

decoupling the superstructure from the movements of the substructure in earthquakes. Meanwhile, more commonly used lateral force resisting structural systems can also resist seismic forces by appropriate dimensioning of the members and careful assessment of the inelastic response. Moment resisting frames, concentrically braced frames and eccentrically braced frames are among the most popular types of steel earthquake resistant structures. Recommendations for their design are set out in seismic design codes (AISC, 2002; EN 1998-1, 2004), which allow the use of carbon steel and reinforced concrete members. However, a relatively new construction material – stainless steel – shows potential for application to earthquake resistant structures owing to its high material ductility and energy absorption under monotonic loading. Further research is required to confirm its potential and to develop design guidance accordingly.

1.3 *Structural stainless steel*

Steel structures traditionally refer to those constructed with carbon steel structural components, since the use of other steel alloys is typically confined to architectural installations and building services. Stainless steel is generally considered as an architectural material due to its high corrosion resistance and aesthetic surface finish, while its high material cost hinders its wider use in load-carrying components. The Gateway Arch, St. Louis, built with a stainless steel skin covering the composite core, is an example of architectural application of stainless steel, though with clear structural demands. (Figure 1.1).

There are a number of examples to demonstrate the ability of using stainless steel members as the main structural components while retaining the stunning architectural effects, such as the trussed roof of the Louvre Pyramid in Paris with stainless steel ties and connections (Figures 1.2 and 1.3), and the stainless steel bracing system of the Sanomatalo Building in Helsinki. As well as aesthetic reasons, stainless steel components have been used due to its minimal maintenance required and hence the low life cycle cost. A greenhouse in Bergianska trädgården, north of Stockholm, was constructed with all structural components in EN 1.4401 stainless steel to minimise maintenance that would damage the plants (Johansson and Olsson, 2000). The Fudo temple in Japan, a building with high cultural

value, was designed with stainless steel members in expectation of maintaining durability for more than 500 years (Aoki, 2000).



Figure 1.1: *The Gateway Arch, St. Louis*

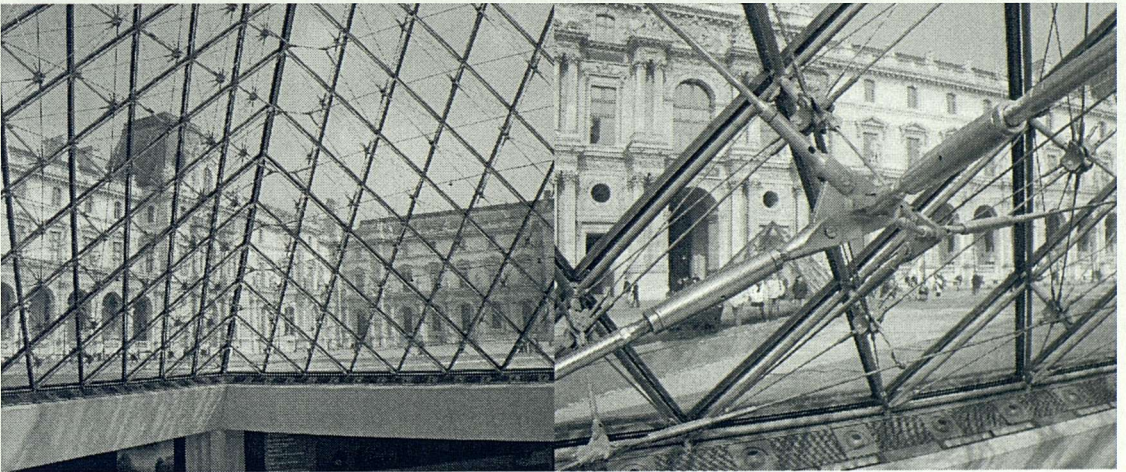


Figure 1.2: *Stainless steel roof truss supporting the glass panel of the roof of the Louvre Pyramid*

Figure 1.3: *Stainless steel connections in the Louvre Pyramid*

Apart from the characteristics mentioned, stainless steel differs from carbon steel in terms of mechanical properties, such as the higher ductility and more pronounced strain hardening of stainless steel. If stainless steel structures are designed with concepts based on the performance of carbon steel members, inefficient use of materials may result (Gardner, 2005). Therefore, the increasing use of stainless steel as a primary structural material justifies more research activities for the development of new approaches to stainless steel design. The implications of high ductility and pronounced

strain hardening of stainless steel on member behaviour under cyclic loading is investigated in this research project.

1.4 Hot-rolled and cold-formed steel sections

Besides the difference in chemical compositions, the means of manufacturing has significant implications on the mechanical properties of structural members. Square and rectangular hollow sections are primarily produced by hot-rolling and cold-forming. Hot-rolled sections are produced above the re-crystallisation temperature of the material (typically around 850°C), resulting in sections with homogeneous material properties, high ductility and relatively low residual stresses. Cold-formed sections are manufactured at ambient temperature with plastic deformation during the forming process. The cold-work enhances the yield strength, reduces the ductility and leaves high residual stresses in the sections. Different levels of plastic deformation across the sections, with corner areas undergoing larger deformation than flat areas, results in a non-homogeneity of material properties. Other differences between hot-rolled and cold-formed sections include geometric imperfections and corner geometry, as well as the general structural behaviour and load-carrying capacity of the produced sections (Gardner et al., submitted). Instead of using the nominal properties, considerations on the manufacturing process has to be made during design, such as following the recommendations in a design code specifically for cold-formed sections (EN 1993-1-3, 2006).

1.5 Outline of thesis

The performance of steel hollow section braces under cyclic loading is investigated. In light of the popularity of carbon steel sections, the increasing demand for research in stainless steel sections, and the significant difference between hot-rolled and cold-formed sections, this research project focuses on braces of three materials – hot-rolled carbon steel, cold-formed carbon steel and cold-formed stainless steel. This introductory chapter provides the background and motivation of the research project.

Chapter 2 gives an overview of important topics, illustrated with key literature that forms the cornerstone of this research project. More specific and detailed reports of relevant literature are given at appropriate stages in the thesis.

The two main parts of this research project are the experimental programmes, reported in Chapters 3 and 4, and the numerical studies, presented in Chapters 5 and 6. Chapter 3 describes the material testing, which consists of tensile coupon tests, cyclic axial material tests and cyclic bending material tests. Strain-life relationships and material cyclic hardening models are calibrated for the three materials. Braces of the same materials are tested under cyclic axial loading. The set-up and the results of 16 cyclic tests on hollow section braces are described in Chapter 4. The development of finite element models and details of a strain-based damage prediction method are presented in Chapter 5, along with the verification against experimental results. In Chapter 6, parametric studies are carried out to investigate the performance of braces outside the tested range. The first part of the studies focuses on the influence of geometric properties while the second part examines the effects of material properties. A predictive equation of the ductility of the braces is proposed for each of the three materials.

Based on the findings of material properties and member behaviour, the implications on seismic design of concentrically braced frames are discussed in Chapter 7. The final chapter summarises the findings in the research project and proposes possible areas of future research.

Chapter 2

Literature Review

2.1 Introduction

In this chapter, the background information and literature relevant to the current research project are reviewed. An introduction to the theory of the material models and fatigue prediction models utilised in this study is given, followed by an overview of previous experimental and numerical work which is relevant to this thesis. The development of current design guidance for stainless steel structures and concentrically braced frames are also described. Further literature is introduced and reviewed in later chapters to provide specific background information on each subject area.

2.2 Materials

This research project focuses on carbon steel and stainless steel hollow section structural members. Carbon steel is the generic name of the most familiar types of structural steel, which are composed of around 0.2% carbon by mass. The term stainless steels refers to a large family of alloys with chromium and nickel as the main alloying elements. In particular, austenitic stainless steel is the most common group of stainless steel for structural applications due to its excellent corrosion resistance and high strength. Austenitic stainless steel has a higher tensile ductility and exhibits more substantial strain hardening than carbon steel, with greater ratios of ultimate tensile

strength to yield strength, greater levels of elongation at fracture and further differences in other physical and thermal properties (Gardner, 2005; Gardner and Ng, 2006).

Structural members can be manufactured by hot-rolling and cold-forming. Hot-rolled sections are rolled into the desired shape at high temperature. The materials are annealed during the forming process so the material properties of hot-rolled members are homogeneous around the section. In contrast, cold-formed sections are formed at room temperature by deforming the material plastically into the section shape. The cold-working changes material properties such as yield strength and tensile ductility. Therefore, hot-rolled and cold-formed sections manufactured from the same virgin material will possess different material properties.

In this study, the materials examined are cut from hot-rolled and cold-formed carbon steel sections (grades S355 J2H (EN 10210-1, 2006) and S235 JRH (EN 10219-1, 2006), respectively) and cold-formed austenitic stainless steel sections (grades EN 1.4301 and 1.4307 (EN 10088-2, 2005)). Their chemical compositions are shown in Table 2.1.

Table 2.1: *Chemical composition of the carbon steel and stainless steel examined in the current study*

Steel name / steel grade	% by mass, maximum or range permitted							
	C	Si	Mn	P	S	N	Cr	Ni
S355 J2H	0.22	0.55	1.60	0.030	0.030	-	-	-
S235 JRH	0.17	-	1.40	0.040	0.040	0.009	-	-
1.4301	0.07	-	-	-	-	-	17.5- 19.5	8.0- 10.5
1.4307	0.03	-	-	-	-	-	17.5- 19.5	8.0- 10.5

The nominal mechanical properties of the materials are given in Table 2.2. Although the nominal yield strengths specified in the design standards are different, the actual yield strengths of the studied materials cut from the structural sections are broadly similar due partly to the increase in yield strength by cold-working. Owing to the

similarity in chemical compositions and specified mechanical properties (EN 10088-2, 2005), the two stainless steel grades are grouped as the same material in this study, and indeed, exhibited similar behaviour.

Table 2.2: *Specified mechanical properties of the carbon steel and stainless steel examined in the current study*

Steel name / steel grade	Yield strength (N/mm ²)	Ultimate strength (N/mm ²)	Elongation at fracture (%)
S355 J2H	355	510-680	22
S235 JRH	235	360-510	24
1.4301	230	540-750	45
1.4307	220	520-700	45

Materials in different parts of a cold-formed section are influenced by cold-working to different degrees, with materials in corner areas most affected. The strength enhancement of materials at corner areas can be significant and should be harnessed for efficient design. EC3 Part 1-3 (EN 1993-1-3, 2006) takes into account the corner strength enhancement of cold-formed sections through an average yield strength, $f_{y,a}$, calculated by the following expression:

$$f_{y,a} = f_{y,b} + \frac{knt^2}{A_g}(f_{u,b} - f_{y,b}) \quad \text{but} \quad f_{y,a} \leq \frac{f_{u,b} + f_{y,b}}{2} \quad (2.1)$$

where $f_{y,b}$ is the nominal yield strength of the sheet before forming, $f_{u,b}$ is the nominal ultimate strength of the sheet before forming, A_g is the gross cross-sectional area, k is a numerical coefficient that depends on the type of forming ($k = 7$ for roll forming and $k = 5$ for other methods of forming), n is the number of 90° bends in the cross-section with an internal radius $r_i \leq 5t$, and t is the thickness of the material before cold-forming.

Strength enhancement of corner areas is even more pronounced in cold-formed stainless steel sections. Ashraf et al. (2005) gathered test results of flat, corner and virgin stainless steel materials and established relationships between them. A relationship predicting the corner 0.2% proof strength, $f_{0.2,c}$, from the 0.2% proof strength of sheet material, $f_{0.2,b}$, is given as:

$$f_{0.2,c} = \frac{1.881f_{0.2,b}}{(r_i / t)^{0.194}} \quad (2.2)$$

where r_i is the internal corner radius and t is the thickness of the material before cold-forming.

The difference of the mechanical properties of the three materials – hot-rolled carbon steel, cold-formed carbon steel and cold-formed stainless steel is not confined to the values in Table 2.2. Their tensile stress-strain curves are known to exhibit distinct behavioural features. Hot-rolled carbon steel displays a sharp yield point, followed by a flat yield plateau before strain hardening occurs, whereas cold-formed carbon steel and stainless steel both exhibit rounded stress-strain characteristics, with the degree of roundedness being significantly more pronounced in the case of stainless steel (Gardner, 2005).

The stress-strain curves of hot-rolled and cold-formed carbon steel are often simplified into a bilinear representation. Due to the roundness of the curve and substantial hardening, using a bilinear model to represent the stress-strain curve of stainless steel results in inefficient use of the material. Models capable of representing the roundness of the stress-strain curves have been proposed. It is based on the observation that a plot of the true stress against true plastic strain in a log-log coordinates results in a straight line and can be represented by a power function:

$$\sigma = K(\varepsilon_p)^n \quad (2.3)$$

where σ is the true stress, ε_p is the true plastic strain, K is the strength coefficient and n is the strain hardening exponent. After including the elastic strain, ε_e , and re-arranging the terms, the total strain, ε , is given by:

$$\varepsilon = \varepsilon_e + \varepsilon_p = \frac{\sigma}{E} + \left(\frac{\sigma}{K} \right)^{1/n} \quad (2.4)$$

where E is the Young's modulus. This relationship has been proposed by Ramberg and Osgood (1943) and is referred to as the Ramberg-Osgood relationship. This expression is suitable for representing stress-strain behaviour of non-linear materials, such as aluminium alloys, stainless steels and some high strength steels (Gardner and Ashraf, 2006). Regarding the application to stainless steel, previous studies have revealed that this expression gives excellent agreement with experimental data up to the 0.2% proof

stress, $f_{0.2}$ (Gardner and Nethercot, 2001). At high strains, however, it overestimates the stress corresponding to a given level of strain. A compound (2-stage) version has been proposed by Mirambell and Real (2000) and developed by Rasmussen (2003):

$$\varepsilon = \frac{\sigma}{E} + 0.002 \left(\frac{\sigma}{f_{0.2}} \right)^{n^{RO}} \quad (\sigma \leq f_{0.2}) \quad (2.5)$$

$$\varepsilon = \frac{(\sigma - f_{0.2})}{E_{0.2}} + \varepsilon_{pu} \left(\frac{\sigma - f_{0.2}}{f_u - f_{0.2}} \right)^{n_{0.2,u}^{RO}} + \varepsilon_{t0.2} \quad (\sigma \geq f_{0.2}) \quad (2.6)$$

where n^{RO} is a strain hardening component, f_u is the ultimate material strength, ε_{pu} is the plastic strain at ultimate strength, $\varepsilon_{t0.2}$ is the total strain at the 0.2% proof stress, $E_{0.2}$ is the stiffness at the 0.2% proof stress, $n_{0.2,u}^{RO}$ is a strain hardening exponent that can be determined from the ultimate strength and another intermediate point. Since the ultimate stress is well beyond the stresses experienced in general structural scenarios and there is no ultimate stress in compression, a modification of Equation 2.6 using the 1.0% proof stress, $f_{1.0}$, and the corresponding strain, $\varepsilon_{t1.0}$, has been devised by Gardner and Ashraf (2006):

$$\varepsilon = \frac{(\sigma - f_{0.2})}{E_{0.2}} + \left(\varepsilon_{t1.0} - \varepsilon_{t0.2} - \frac{f_{1.0} - f_{0.2}}{E_{0.2}} \right) \left(\frac{\sigma - f_{0.2}}{f_{1.0} - f_{0.2}} \right)^{n_{0.2,1.0}^{RO}} + \varepsilon_{t0.2} \quad (\sigma \geq f_{0.2}) \quad (2.7)$$

where $n_{0.2,1.0}^{RO}$ is a strain hardening coefficient representing a curve that passes through $f_{0.2}$ and $f_{1.0}$.

2.3 Cyclic plasticity

When reversals of loading occur, the resulting stress-strain response forms hysteretic loops. A mathematical description of the cyclic stress-strain response beyond the elastic domain is provided by the theory of cyclic plasticity. Since plastic deformation is history dependent, constitutive equations are expressed in incremental or differential forms. However, cyclic plasticity models are also assumed to be rate insensitive, which means the constitutive equations are invariant with respect to the time scale, so the incremental or differential forms are equivalent to each other. A time-independent mathematical model of plasticity consists of:

- (1) a yield criterion which defines the threshold beyond which plastic deformation occurs;
- (2) a loading criterion which differentiates elastic unloading from plastic loading;
- (3) a flow rule which determines the plastic strain increment; and
- (4) a hardening rule which governs the evolution of the yield surface.

2.3.1 Yield criterion

A yield surface in a stress space is analogous to a yield point on a stress-strain curve, which is a yield criterion defining the elastic limit. The general form of a yield criterion is $F(\sigma_{ij}) = 0$, where σ_{ij} is a stress tensor. When the stress point is inside the yield surface, i.e. $F(\sigma_{ij}) < 0$, the material is within the elastic range. On the other hand, when the stress point is on the yield surface, i.e. $F(\sigma_{ij}) = 0$, the material yields. A form of yield criterion often used for ductile materials is the von Mises criterion (Von Mises, 1913). It states that yielding of a material occurs when the octahedral shear stress at a point reaches a critical value. The yield criterion can be expressed as

$$F(\sigma_{ij}) = \frac{3}{2} s_{ij} s_{ij} - f_y^2 = 0 \quad (2.8)$$

where s_{ij} is the deviatoric stress tensor and f_y is the yield stress.

2.3.2 Loading criterion

A loading criterion determines if a stress increment causes a plastic loading or an elastic unloading. In order to cause plastic deformation, the stress point must be on the yield surface, i.e. $F=0$. For a plastic loading to occur in a hardening material, the stress increment must move the stress point beyond the current yield surface. Otherwise, it is a neutral loading or an elastic unloading. In mathematical terms, the loading criteria are

$$(1) \text{ Loading: } F = 0 \quad \text{and} \quad \frac{\partial F}{\partial \sigma_{ij}} d\sigma_{ij} > 0 \quad (2.9)$$

The stress point moves beyond the loading surface and plastic deformation occurs, i.e. $d\epsilon_{ij}^p \neq 0$, where ϵ_{ij}^p is the plastic strain tensor.

$$(2) \text{ Neutral loading: } F = 0 \quad \text{and} \quad \frac{\partial F}{\partial \sigma_{ij}} d\sigma_{ij} = 0 \quad (2.10)$$

The stress point moves on the loading surface and no further plastic deformation occurs, i.e. $d\varepsilon_{ij}^p = 0$.

$$(3) \text{ Unloading: } F = 0 \quad \text{and} \quad \frac{\partial F}{\partial \sigma_{ij}} d\sigma_{ij} < 0 \quad (2.11)$$

The stress point moves inwards and no further plastic deformation occurs, i.e. $d\varepsilon_{ij}^p = 0$.

2.3.3 Flow rule

A flow rule specifies the stress-strain relationship after yielding. In elasticity theory, stress and strain tensors are related through an elastic potential function, which is the complementary strain energy U_c , to give

$$\varepsilon_{ij} = \frac{\partial U_c}{\partial \sigma_{ij}} \quad (2.12)$$

Von Mises (1928) proposed a similar relationship for plasticity theory by adopting a plastic potential function $Q(\sigma_{ij})$:

$$d\varepsilon_{ij}^p = d\lambda \frac{\partial Q(\sigma_{ij})}{\partial \sigma_{ij}} \quad (2.13)$$

where $d\lambda$ is a positive scalar factor which can be determined by a yield criterion and $Q(\sigma_{ij}) = C$, with C as a constant, represents a surface in the stress space perpendicular to $d\varepsilon_{ij}^p$. So Equation 2.13 is also referred to as the normality rule. A common approach to determine the plastic potential Q is to assume that it is the same as the yield function $F(\sigma_{ij})$. Equation 2.13 can then be rewritten as

$$d\varepsilon_{ij}^p = d\lambda \frac{\partial F(\sigma_{ij})}{\partial \sigma_{ij}} \quad (2.14)$$

If a flow rule indicates that the plastic strain increment is normal to the yield surface, as in the case of Equation 2.14, it is called an associated flow rule. An associated flow rule describes the plastic deformation well for metals but non-associated flow rules, i.e. $Q \neq F$, provide a better representation of the plastic deformation of porous materials such as rocks, concrete and soils.

2.3.4 Hardening rule

A hardening rule defines how a yield surface evolves according to the change in plastic deformation. Isotropic hardening and kinematic hardening are the two main types of hardening rules. An isotropic hardening rule assumes that the yield surface expands or contracts uniformly in all directions while a kinematic hardening rule fixes the size of a yield surface but allows it to translate in the stress space.

2.3.4.1 Isotropic hardening model

The general form of yield criterion of an isotropic hardening model is expressed as

$$F(\sigma_{ij}) = f(\sigma_{ij}) - f_y^2(\alpha) = 0 \quad (2.15)$$

The size of the yield surface is controlled by the hardening parameter α through the function $f_y(\alpha)$. $f(\alpha)$ is a monotonically increasing function of α for an isotropically hardening material. If the von Mises (1913) yield criterion is applied, Equation 2.15 becomes

$$F(\sigma_{ij}) = \frac{3}{2} s_{ij} s_{ij} - f_y^2(\alpha) = 0 \quad (2.16)$$

The parameter α is usually defined as equivalent plastic strain, $\epsilon^p = \sqrt{\frac{2}{3} \epsilon_{ij}^p \epsilon_{ij}^p}$, or total plastic work. Its evolution equation can be derived by applying the normality rule (Equation 2.13).

2.3.4.2 Kinematic hardening model

The motivation for formulating a kinematic hardening model is to address the inability to describe the Bauschinger effect by an isotropic hardening rule. The general form of the yield surface becomes

$$F = f(\sigma_{ij} - \alpha_{ij}) - \kappa = 0 \quad (2.17)$$

where α_{ij} is a hardening parameter called the backstress and κ is a material constant representing the size of the yield surface. Prager (1949) proposed a linear relationship between the backstress and plastic strain:

$$d\alpha_{ij} = c_p d\varepsilon_{ij}^p \quad (2.18)$$

where c_p is a constant. By substituting in Equation 2.14, Equation 2.18 can be written as

$$d\alpha_{ij} = c_p d\lambda \frac{\partial F}{\partial \sigma_{ij}} \quad (2.19)$$

It can be seen that the backstress moves in the direction perpendicular to the yield surface at the loading point.

Ziegler (1959) proposed a modification to this:

$$d\alpha_{ij} = (\sigma_{ij} - \alpha_{ij}) d\mu \quad (2.20)$$

where $d\mu$ is a proportionality scalar constant determined by the yield criterion. The backstress moves along the direction of $(\sigma_{ij} - \alpha_{ij})$. Prager's and Ziegler's models are schematically shown in Figure 2.1.

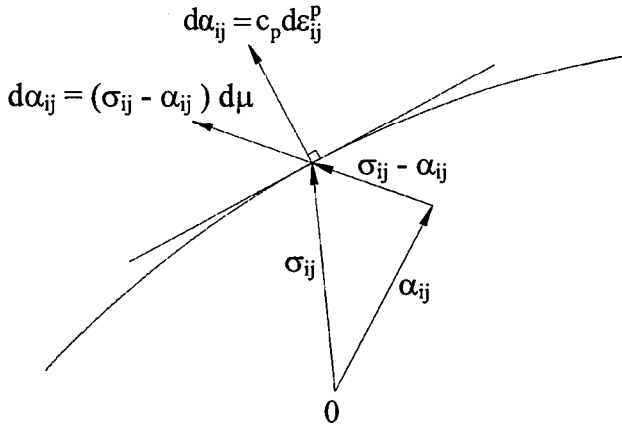


Figure 2.1: Schematic representation of Prager's (1949) and Ziegler's (1959) kinematic hardening models

Since materials often harden in a nonlinear manner, a multilayer model has been proposed by Mroz (1967) to improve the linear kinematic rule. The model consists of a certain number of surfaces in the stress space, containing regions of constant hardening moduli. Due to the large number of surfaces required to describe a material accurately, multilayer models are computationally expensive. Several two-surface models have been developed to obtain the same specific properties but by using only two surfaces,

the yielding and bounding surfaces (Dafalias and Popov, 1976), between which the hardening modulus continuously varies.

2.3.4.3 ABAQUS model

Due to rising computational power and developments in the theory of plasticity, cyclic plasticity models have been incorporated in many numerical modelling packages. The nonlinear combined isotropic/kinematic hardening model in ABAQUS (2007) is based on a model proposed by Chaboche (1986). It consists of a nonlinear kinematic hardening component and an isotropic hardening component.

The kinematic component is a purely kinematic term (Ziegler's (1959) rule) with an additional relaxation term which introduces nonlinearity. The hardening rule is

$$d\alpha_{ij} = C_{kin} \frac{1}{\sigma_0} (\sigma_{ij} - \alpha_{ij}) d\varepsilon^p - \gamma \alpha_{ij} d\varepsilon^p \quad (2.21)$$

where C_{kin} and γ are material constants that can be calibrated from cyclic test data.

The isotropic component defines the change of the yield surface size, σ_0 , through a function of equivalent plastic strain ε^p . It is written as

$$\sigma_0 = \sigma|_0 + Q_\infty (1 - e^{-b_{iso}\varepsilon^p}) \quad (2.22)$$

where Q_∞ and b_{iso} are material constants and $\sigma|_0$ is the yield stress at zero equivalent plastic strain.

2.4 Prediction of fatigue life

Components of machines and structures subjected to cyclic loading suffer microscopic damage due to the cyclic stresses. The microscopic damage accumulates and results in macroscopic damage such as cracks. The process of damage and failure of a material subjected to cyclic loading is called fatigue.

There are three major approaches to analyse fatigue and fracture of metals. The first one is a stress-based approach, which is based on the analysis of the nominal (average) stress in the region of the component being analysed. The second one is a strain-based approach, which considers the localised plastic deformation where a fatigue crack begins. The third one is a fracture mechanics approach, which deals with crack growth by using fracture mechanics. A strain-based approach is most suitable for situations where local yielding is involved and the fatigue life is short. Therefore, it is most relevant to predicting fracture in the tests specimens in this study, which are subjected to large plastic deformation.

2.4.1 Strain-based approach

Basquin (1910) has shown that when the logarithm of the stress amplitude is plotted against the logarithm of the number of reversals to failure, a linear relationship resulted

$$\frac{\Delta\sigma}{2} = \sigma'_f (2N_f)^b \quad (2.23)$$

where $\Delta\sigma/2$ is the stress amplitude, σ'_f is the fatigue strength coefficient, b is the fatigue strength exponent and $2N_f$ is the number of reversals to failure. Similarly, based on data of constant strain amplitude fatigue tests, Coffin (1954) and Manson (1954) independently proposed a linear relationship between plastic strain and number of reversals to failure, which is now commonly referred to as the Coffin-Manson relationship.

$$\frac{\Delta\varepsilon_p}{2} = \varepsilon'_f (2N_f)^c \quad (2.24)$$

where $\Delta\varepsilon_p/2$ is the plastic strain amplitude, ε'_f is the fatigue ductility coefficient and c is the fatigue ductility exponent. Noting that total strain amplitude is the sum of the elastic and plastic strain amplitudes, Equations 2.23 and 2.24 can be combined to obtain a total strain-life relationship:

$$\frac{\Delta\varepsilon}{2} = \frac{\Delta\varepsilon_e}{2} + \frac{\Delta\varepsilon_p}{2} = \frac{\Delta\sigma}{2E} + \frac{\Delta\varepsilon_p}{2} = \frac{\sigma'_f}{E} (2N_f)^b + \varepsilon'_f (2N_f)^c \quad (2.25)$$

where $\Delta\varepsilon/2$ is the total strain amplitude, $\Delta\varepsilon_e/2$ is the elastic strain amplitude and E is the Young's modulus. The fatigue life at which elastic strain and plastic strain are equal

is called the transition life, N_t . It is referred to as low cycle fatigue when fatigue life is lower than N_t and high cycle fatigue when it is higher than N_t (Figure 2.2).

2.4.2 Effect of mean stress/strain

A non-zero mean stress/strain can influence the fatigue behaviour of materials. Experimental data show that the presence of compressive mean normal stresses/strains delays fatigue failure while tensile mean stresses/strains reduce fatigue life (Topper et al., 1969; Gong & Norton, 1999). A number of methods have been proposed to account for mean stress effects. Smith et al (1970) proposed a method that is based on the assumption that the fatigue life for any value of mean stress depends on the product

$\sigma_{\max} \frac{\Delta \varepsilon}{2}$, where $\sigma_{\max} = \sigma_m + \frac{\Delta \sigma}{2}$, σ_m is the mean stress and $\frac{\Delta \sigma}{2}$ is the stress amplitude.

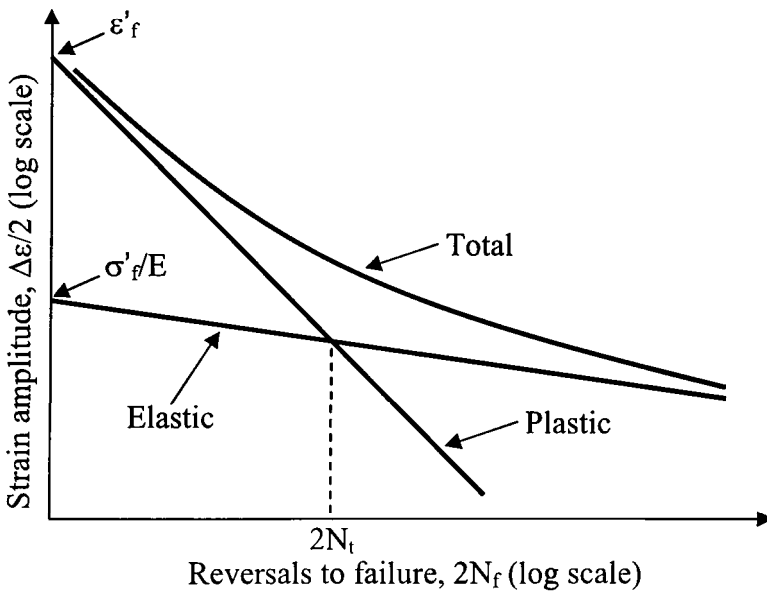


Figure 2.2: Elastic, plastic and total strain-life curves

Gong and Norton (1999) proposed three approaches to account for mean stress/strain effects. One of these methods has the advantage that explicit values of the mean strain can be used directly so that there is no need to obtain data of the mean stress. The modified strain-life equation is as follow:

$$\frac{\Delta \varepsilon}{2} = \frac{\sigma'_f}{E} (2N^*)^b + \varepsilon'_f (2N^*)^c \quad (2.26)$$

where $N^* = N_f \left(1 + \frac{100\varepsilon_m}{\varepsilon'_f} \right)^{1/c}$ and ε_m is the mean strain.

2.4.3 Multiaxial loading

Many engineering structures and components are subjected to random multiaxial loading. Multiaxial fatigue assessment is important to predict the failure of elements under repeated loading and unloading. One approach is to account for multiaxial loading by modifying the Coffin-Manson equation. Mowbray (1980) introduced a correction method for biaxial stress using the biaxiality ratio $\lambda = \sigma_2 / \sigma_1$

where $\Delta\sigma_1 > \Delta\sigma_2$. The multiaxial relationship is:

$$\frac{\Delta\varepsilon}{2} = \frac{\sigma'_f}{E} f(\lambda, \nu) (2N_f)^b + \left(\frac{3}{3-m} \right) \varepsilon'_f g(\lambda, m) (2N_f)^c \quad (2.27)$$

where $f(\lambda, \nu) = \frac{1 - \nu\lambda}{(1 - \lambda + \lambda^2)^{1/2}}$, $g(\lambda, m) = \frac{(2 - \lambda)[3(1 - \lambda + \lambda^2)^{1/2} - m(1 + \lambda)]}{6(1 - \lambda + \lambda^2)}$, ν is the

Poisson's ratio and m is a material constant.

2.4.4 Variable amplitude loading

The strain-life relationship mentioned above is based on constant-amplitude fatigue test data. For cases of variable-amplitude loading, a cumulative damage rule is required. The simplest form is the linear Palmgren-Miner rule, or sometimes referred to as the Miner's rule (Miner, 1945):

$$D = \sum_{i=1}^n \frac{n_i}{N_{fi}} \quad (2.28)$$

where N_{fi} is the number of cycles to failure at a strain (or stress) level of ε_i (or σ_i) and n_i are the number of cycles for which that particular strain (or stress) level is applied. Fatigue occurs when the damage index, D , reaches unity.

Experimental data has showed that this prediction is not always reliable (Topper et al., 1969; Bernard-Connolly et al., 1983) since the loading sequence is not considered in

the Miner's rule. The damage of low amplitude cycles followed by high amplitude cycles is usually overestimated and vice versa. Non-linear damage rules have been proposed to replace the Miner's rule. However, there is no consensus about which damage rule is the most accurate and general. Some rules perform better for certain kinds of loading history while most others give more accurate results for other loading histories. In the current study, the damage is mainly caused by the localised deformation due to local buckling during the last few cycles, when the strain amplitudes are largely similar. Therefore, the effect of loading sequence is not of great importance and satisfactory results can be obtained from the Miner's rule.

2.5 Member testing

The cyclic behaviour of carbon steel structural sections has been studied for many years. Early studies (Popov et al., 1979; Jain et al., 1980) focused on the load-displacement hysteretic behaviour of the members. The influence of member and cross-section slenderness on compressive strength and lateral displacement at mid-length was studied, and hysteretic models were proposed to simulate their behaviour. Since many of the experiments were terminated before the specimens fractured, the results cannot be used for calibrating expressions of member ductility and verifying damage prediction methods. However, in more recent studies, the failure features and fracture life of the specimens have generally been reported as the attention shifted to the ductility of the bracing members, which is expressed in terms of number of cycles to failure or displacement ductility, μ_Δ :

$$\mu_\Delta = \delta / \delta_y \quad (2.29)$$

where δ is the largest displacement applied and δ_y is the yield displacement of the member. Relationships between ductility of the members and member geometry have been established (Tremblay, 2002; Shaback & Brown, 2003; Goggins et al., 2006) but opinion is divided on whether member or cross-section slenderness is more influential on fracture life. A summary of cyclic axial loading tests conducted on carbon steel sections is shown in Table 2.3. In particular, the results from Walpole (1996) and Goggins et al. (2006) are used in the verification of the numerical models and damage prediction procedure in the current study.

The number of research activities on structural stainless steel has been growing in recent years. Gardner and Nethercot (2004a; 2004b) carried out stub column, pinned end column and three-point bending tests on stainless steel hollow sections. Similarly, tests on compression and flexural members were carried out by Ellobody and Young (2005), and Zhou and Young (2005). Local geometric imperfection profiles and membrane residual stress distributions were also measured. The tests showed that the design strengths predicted by design codes are generally conservative.

Table 2.3: *Number of cyclic axial loading tests conducted on carbon steel sections*

References	SHS	RHS	CHS	I-section	T-section	Single angle section	Double angle section	Double channel section
Popov et al. (1979)	-	-	6	-	-	-	-	-
Black et al. (1980)	3	-	5	8	2	-	4	1
Jain et al. (1980)	12	5	-	-	-	8	-	-
Popov and Black (1981)	3	-	5	9	2	-	4	1
Gugerli (1982)		4	-	5	-	-	-	
Lee and Goel (1987)	7	-	-	-	-	-	-	-
Liu (1987)	-	3	-	-	-	-	-	-
Archambault et al. (1995)	12	2	-	-	-	-	-	-
Walpole (1996)	-	3	-	-	-	-	-	-
Elchalakani et al. (2003)	-	-	20	-	-	-	-	-
Shaback and Brown (2003)	9	-	-	-	-	-	-	-
Goggins et al. (2006)	10	5	-	-	-	-	-	-
Han et al. (2007)	11	-	-	-	-	-	-	-
Total	67	22	36	22	4	8	8	2

Cyclic tests on structural stainless steel members have not been reported in the literature but test data on the cyclic behaviour of stainless steel hollow sections for application in automotive industry are available. Yoon et al. (2003) studied 429EM

stainless steel pipes subjected to cyclic axial loading. Cyclic softening was observed in all tests and a strain-life relation was obtained. Chang et al. (2005) investigated the collapse and ovalisation of 316L stainless steel tubes under cyclic bending. Cyclic hardening was found in these symmetric curvature-controlled cyclic bending tests.

2.6 Numerical modelling

Simulation of carbon steel and stainless steel structural members by finite element models has been performed successfully (Gardner and Nethercot, 2004c; Young, 2008). In general, reliability of a numerical model depends on the choice of material model, mesh size, boundary conditions, imperfection amplitudes, and the accuracy of replicating the member geometry. Simulation of stainless steel members requires that extra attention be paid to the modelling of strain hardening and cold-working in the corner areas. Gardner and Nethercot (2004c) highlighted the important aspects in developing a consistent approach to the modelling of structural stainless steel members. Material models given by Equations 2.6 and 2.7 were found to give excellent agreement with experimental stress-strain data. Cold-working on corner materials increases the 0.2% proof stress to approximately 85% of the ultimate strength of the flat material, with the effect extending beyond the corner to a distance equal to approximately two times the material thickness. In the absence of measured global imperfection amplitudes, an amplitude of $L/2000$ was recommended. Dawson and Walker's (1972) model for the local imperfection amplitudes was calibrated for stainless steel hollow sections and is given in Equation 2.30.

$$\omega_0/t = 0.023(f_y/f_{cr}) \quad (2.30)$$

where ω_0 is the initial imperfection amplitude, t is the plate thickness, f_y is the material yield stress and f_{cr} is the plate critical buckling stress.

2.7 Centrally braced frames

A centrally braced frame is an efficient configuration to resist lateral load because the frame acts effectively as a truss, which means individual members are only

subjected to axial loading. Diagonal bracing members provide high stiffness to frames and prevent excessive drift under extreme lateral loading conditions. I- sections, angle sections, circular or rectangular hollow sections are often used as bracing members. Common bracing configurations are shown in Figure 2.3. All of these configurations are suitable for resisting static load; however, K-braced frames are not recommended for dissipative earthquake design due to the increased likelihood of causing premature failure of the columns.

Economical design of concentrically braced frames requires the use of dissipative behaviour through significant inelastic deformations of bracing members. In order to achieve that, careful member selection and detailing to prevent undesirable failure mechanism at other members is needed. The seismic design procedures for concentrically braced frames were discussed in Elghazouli (2003).

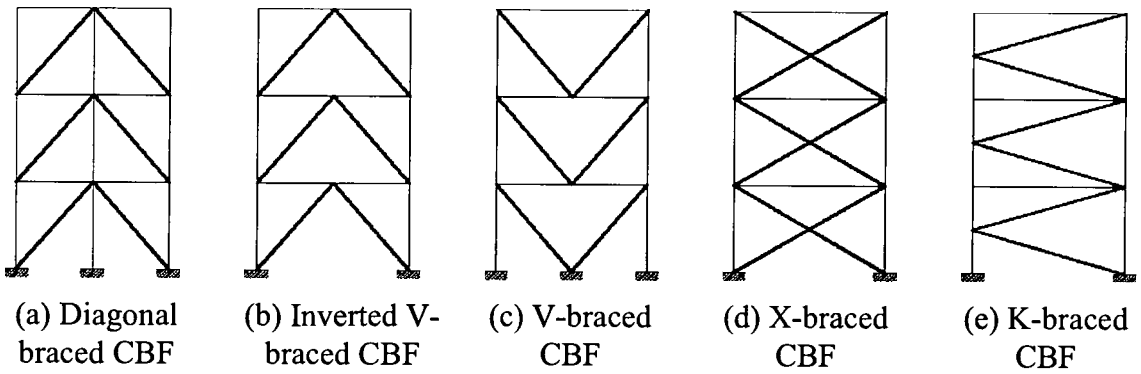


Figure 2.3: Common concentrically braced frame configurations

2.8 Design guidance

In Europe, design guidance for steel buildings is specified in Eurocode 3. General design rules for structural steel members are given in EC3 Part 1-1 (EN 1993-1-1, 2005), supplemented by design rules for cold-formed members in EC3 Part 1-3 (EN 1993-1-3, 2006) and stainless steel members in EC3 Part 1-4 (EN 1993-1-4, 2006).

The design specifications for stainless steel are based on existing design principles for carbon steel and modified to account for the differences between the two materials. Although they generally yield conservative results and may be readily used by

designers who are familiar with the design of carbon steel structures, they do not lead to the efficient use of stainless steel. Nethercot and Gardner (2004) observed that use of elastic, perfectly plastic material model is overly simplistic and strain hardening must be utilised for more efficient design. Based on this premise, a new design approach was proposed by Ashraf et al. (2008) to replace the traditional discrete section classification by using a continuous parameter to reflect the section slenderness. The new method predicts the resistance of stainless steel cross-sections more accurately and offers material savings of approximately 25% for hollow sections. The approach, termed the continuous strength method, has also been extended to cover carbon steel (Gardner, 2008).

General design rules for earthquake resistant structures, including a section for steel structures, are specified in EN 1998-1 (2004). However, there is no specific guideline for using stainless steel in earthquake design. The feasibility of stainless steel earthquake resistance structures has not been investigated until recently (Di Sarno et al., 2006; Di Sarno et al., 2008). The potential for using stainless steel in earthquake resistant structures is indicated by its excellent energy absorbing capacity and delayed local buckling due to the augmented material strain hardening.

Chapter 3

Material Testing

3.1 Introduction

Under extreme seismic conditions, structural members, particularly those acting as dissipative elements, experience small numbers of very large displacement cycles. The response of structural members to this form of loading is controlled by their geometry and by the hysteretic behaviour of the constituent material, which can be studied through fatigue testing at high strain amplitudes.

Fatigue testing configurations include axial tension-compression, axial tension-tension, torsion and bending. In axial testing arrangements, specimens are subjected to uniform stress and strain amplitudes across the section while maximum stress and strain amplitudes only occur on the surface of specimens in torsion and bending configurations. Axially-loaded fatigue testing has been carried out on many materials, including carbon steel (Gong & Norton, 1999; Yang, 2005) and stainless steel (grade EN 1.4301) (Bhanu Sankara Rao et al., 1993; Bergengren et al., 1995). Stocky cylindrical test specimens are often employed to prevent buckling of specimens under high compressive strains (Smith et al., 1963), although thin sheet specimens have also been employed when the applied strain amplitudes are small (Bergengren et al., 1995). While axial testing configurations allow easy interpretation of test results owing to the uniform distribution of stress and strain, bending configurations have the advantage of avoiding the issue of buckling and therefore enabling testing of thin plates. Three-point

bending (Omerspahic et al, 2006), four-point bending (Bayoumi & ABD El Latif, 1995) and uniform bending (Yoshida et al., 1998) set-ups are among the possible variations of bending test configurations.

Another way of categorising fatigue tests is to consider the applied strain amplitude, which can be decomposed into elastic and plastic components. Testing regimes in which the elastic strain amplitude is higher than the plastic strain amplitude are generally referred to as high cycle fatigue (HCF); such tests are usually stress-controlled and typically involve more than 10^6 cycles to cause failure. Conversely, the term low cycle fatigue (LCF) is applied when plastic strain dominates, and the number of cycles to failure generally ranges from 10^2 to 10^4 . Low cycle fatigue testing is generally strain-controlled (either by total or plastic strain), since hardening or softening of the material complicates load- or stress-controlled testing.

It has been observed that plastic strain-life data from LCF tests fall approximately on a straight line when plotted on a log-log scale. This observation forms the basis of the Coffin-Manson relationship (Coffin, 1954, Manson, 1954), which is expressed as:

$$\frac{\Delta\epsilon_p}{2} = \epsilon'_f (2N_f)^c \quad (3.1)$$

where $\Delta\epsilon_p/2$ is the plastic strain amplitude, ϵ'_f is the fatigue ductility coefficient, c is the fatigue ductility exponent and $2N_f$ is the number of reversals to failure.

In the very low cycle regime, often referred to as extremely low cycle fatigue (ELCF), an over-estimation of fatigue life by the Coffin-Manson relationship has been observed (Kuroda, 2001; Xue, 2008). Models accounting for the combined effects of ductile and fatigue damage have been proposed to establish strain-life relationships that cover both the LCF and ELCF regimes. This has been achieved, for example, by combining an exponential function with the power law of the Coffin-Manson relationship (Xue, 2008), partitioning the damage into ductile and cyclic damage components (Kuroda, 2001; Tateishi et al., 2007) or by employing plastic void growth and coalescence models (Kanvinde & Deierlein, 2007).

In addition to stress- and strain-life relationships, a further means to predict fatigue failure is by consideration of energy dissipation (Morrow, 1965). The relationship between total energy to fracture and fatigue life can be derived from the area under the hysteresis loops of Masing-type materials and is given as:

$$W_f = 4\sigma'_f \epsilon'_f \left(\frac{c-b}{c+b} \right) (2N_f)^{1+b+c} \quad (3.2)$$

where W_f is the total energy to fracture, σ'_f is fatigue strength coefficient, ϵ'_f is the fatigue ductility coefficient, b is the fatigue strength exponent, c is the fatigue ductility exponent and $2N_f$ is the number of reversals to failure. The relationship shows that the total energy to fracture increases as the fatigue life increases. Assessment of energy dissipation is also useful in relating axial and bending fatigue test results. Esin (1980) developed a method to correlate fatigue data from axial, bending and rotating bending tests by considering energy dissipation of the cross-section of the specimens. This approach has been applied in the present study.

Existing strain-life relationships, stress-strain hysteretic behaviour and energy criteria reported in the literature have been derived principally from LCF tests at cyclic strain amplitudes of less than $\pm 2.0\%$. However, strain amplitudes encountered by structural members under extreme loading, such as earthquakes, may significantly exceed this value and belong to the ELCF regime. This chapter presents a programme of high strain amplitude fatigue testing of structural carbon steel and stainless steel materials. The results are used to develop strain-life relationships and cyclic hardening parameters, suitable for incorporation into numerical models to predict fracture of structural members subjected to large amplitude cyclic loading.

3.2 *Materials and testing arrangements*

3.2.1 *Materials and test coupons*

LCF and ELCF fatigue tests were carried out to contribute to the limited data currently available on structural materials. Test coupons were machined from the flat faces of square and rectangular hollow sections to reflect the actual material properties of

complete structural members. The location of the extracted coupons was consistently opposite to the weld, as shown in Figure 3.1. The materials examined were from hot-rolled and cold-formed carbon steel sections (grades S355 J2H (EN 10210-1, 2006) and S235 JRH (EN 10219-1, 2006), respectively) and cold-formed stainless steel sections (grades EN 1.4301 and 1.4307). Owing to the similarity in chemical compositions and specified mechanical properties (10088-2, 2005), the two stainless steel grades were grouped as the same material in this study, and indeed, exhibited similar behaviour.

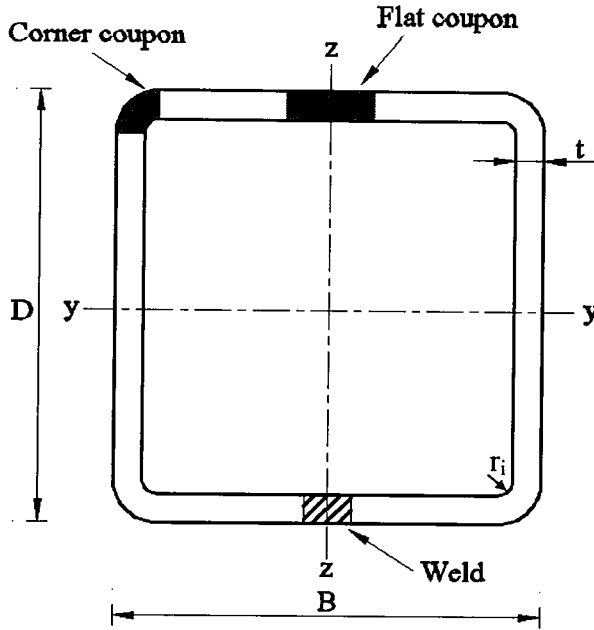


Figure 3.1: *Location of flat coupons from within structural hollow sections*

The monotonic tensile stress-strain curves of the three materials – hot-rolled carbon steel, cold-formed carbon steel and cold-formed stainless steel – abbreviated herein as ‘hot CS’, ‘cold CS’ and ‘cold SS’, respectively, are known to exhibit distinct behavioural features. Hot-rolled carbon steel displays a sharp yield point, followed by a flat yield plateau before strain hardening occurs, whereas cold-formed carbon steel and stainless steel both exhibit rounded stress-strain characteristics, with the degree of roundedness being significantly more pronounced in the case of stainless steel (Gardner, 2005). Stainless steel also exhibits more substantial strain hardening than carbon steel, with greater ratios of ultimate tensile strength to yield strength, greater levels of elongation at fracture and differences in other physical and thermal properties (Gardner & Ng, 2006). Typical stress-strain curves for the three materials (up to 2% strain) are shown in Figure 3.2, while a summary of the results of the monotonic tensile coupon

tests (performed in accordance with (EN 10002-1, 2001)) is given in Table 3.1. In the table, and throughout the thesis, specimens are identified by section size ($B \times D \times t$), material (either carbon steel ‘CS’ or stainless steel ‘SS’), forming process (either hot-rolled ‘HR’ or cold-formed ‘CF’) and finally by either test details for material test coupons (either ‘T’ for tensile tests on flat materials, ‘C’ for tensile tests on corner materials or the strain amplitude for cyclic tests) or length for member test specimens. For both the cold-formed carbon steel and stainless steel, the 0.2% proof strength has been calculated as the equivalent yield strength. The percentage elongation at fracture ε_L is defined as the percentage elongation over the standard gauge length, which is a function of cross-sectional area, as defined in EN 10002-1 (2001). The tensile ductility ε_f is defined by Equation 3.3 and is determined by measuring the reduction in cross-sectional area after fracture.

$$\varepsilon_f = \ln\left(\frac{A_0}{A_u}\right) \times 100\% \quad (3.3)$$

where A_0 is the initial cross-sectional area of the tensile coupon and A_u is the minimum cross-sectional area after fracture. However, tensile ductility has not been measured on corner tensile coupons.

The geometry and dimensions of the cyclic material test coupons, which included a central region of reduced cross-sectional area to constrain the location of fracture, are shown in Figure 3.3. Drilled and reamed holes 20 mm from each end of the coupons were provided to accommodate the pins in the jaws of the testing machine. As recommended in (BS 7270, 2006), the dimensions have been chosen to ensure that buckling does not occur under the highest compressive forces anticipated in the test programme, and that premature failure does not occur outside the reduced cross-section. The length and the width of the region of full cross-sectional area are slightly larger than those in the standard in order to provide greater surface area for gripping.

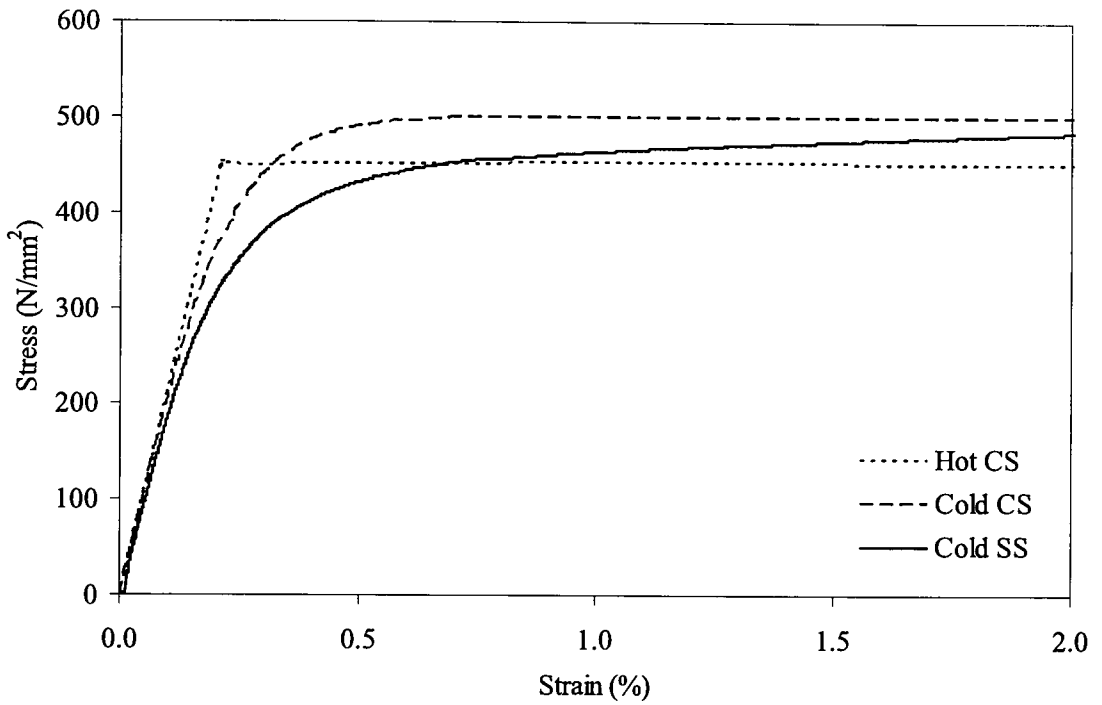


Figure 3.2: Monotonic tensile stress-strain curves of hot-rolled carbon steel, cold-formed carbon steel and cold-formed stainless steel

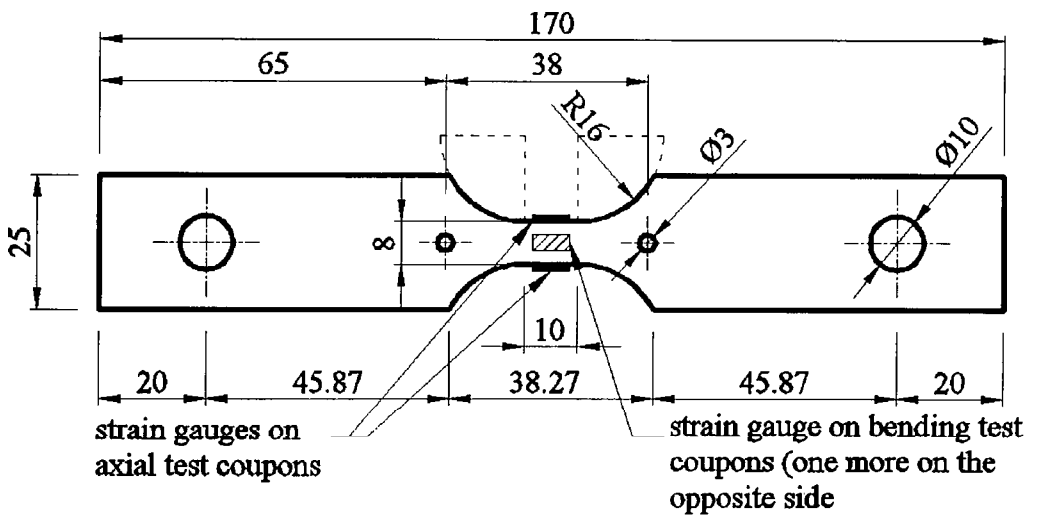


Figure 3.3: Dimensions of cyclic material test coupons (in mm)

Table 3.1: Monotonic tensile material properties

Specimen	Young's modulus, E (N/mm ²)	Yield strength or 0.2% proof stress, f_y or $f_{0.2}$ (N/mm ²)	Ultimate tensile strength, f_u (N/mm ²)	Percentage elongation at fracture, ϵ_L (%)	Tensile ductility, ϵ_f (%)
60×60×3-CS-HR-T	215150	449	555	37	114
40×40×4-CS-HR-T	212310	443	531	41	123
40×40×3-CS-HR-T	219610	478	555	31	111
40×40×4-CS-HR-C	215520	420	520	37	-
60×60×3-CS-CF-T	207430	361	402	49	97
40×40×4-CS-CF-T	201640	410	430	38	110
40×40×3-CS-CF-T	212910	451	502	24	106
60×40×4-CS-CF-T	211950	400	452	21	65
60×60×3-CS-CF-C	215470	442	472	21	-
40×40×4-CS-CF-C	210830	479	507	17	-
40×40×3-CS-CF-C	196680	533	589	16	-
60×40×4-CS-CF-C	202380	480	570	15	-
100×100×3-SS-CF-T	201300	419	725	66	96
60×60×3-SS-CF-T	197730	483	745	58	88
50×50×3-SS-CF-T	197960	552	798	59	97
60×40×3-SS-CF-T	191690	538	753	58	92
100×100×3-SS-CF-C	181440	694	839	56	-
60×60×3-SS-CF-C	193440	614	855	35	-
50×50×3-SS-CF-C	173530	723	927	30	-
60×40×3-SS-CF-C	198530	741	984	23	-

3.2.2 Axial tests

Axially loaded tests were performed to establish strain-life relationships and to calibrate a hardening model for cyclic plasticity. Coupons were subjected to fully-reversed constant total strain amplitude cycles. Test coupons were gripped by friction in the jaws of the testing apparatus as well as positively held by pins at each end. Relative movement between the top and bottom jaws of the testing apparatus was prevented by guide plates (Figure 3.4). Deformation of the coupons was concentrated in the central 3 mm of exposed length of the test coupons; two strain gauges (one on each side of the test coupon, as shown in Figure 3.3) covered this exposed region.

Tests were carried out at $\pm 1\%$, $\pm 3\%$, $\pm 5\%$ and $\pm 7\%$ total strain amplitudes, which caused significant plastic deformation and covered both the LCF and ELCF regimes.

Generally, three specimens were tested at each strain amplitude. Cyclic loading was applied by means of a 250 kN Instron actuator at strain rates of about $5 \times 10^{-3} \text{ s}^{-1}$, which complied with BS 7270 (2006), to ensure that the heat generated during testing did not significantly affect the results. Initially, amplitudes of applied displacements were slightly adjusted in each cycle according to the readings given by strain gauges in order that the target strain amplitudes were achieved, as in some cases minor slippage was observed. Upon stabilisation, or loss of adhesion of the strain gauges to the specimen, testing proceeded under displacement control. Testing continued until the occurrence of fracture, and failure was defined at the point of a 10% drop in load.

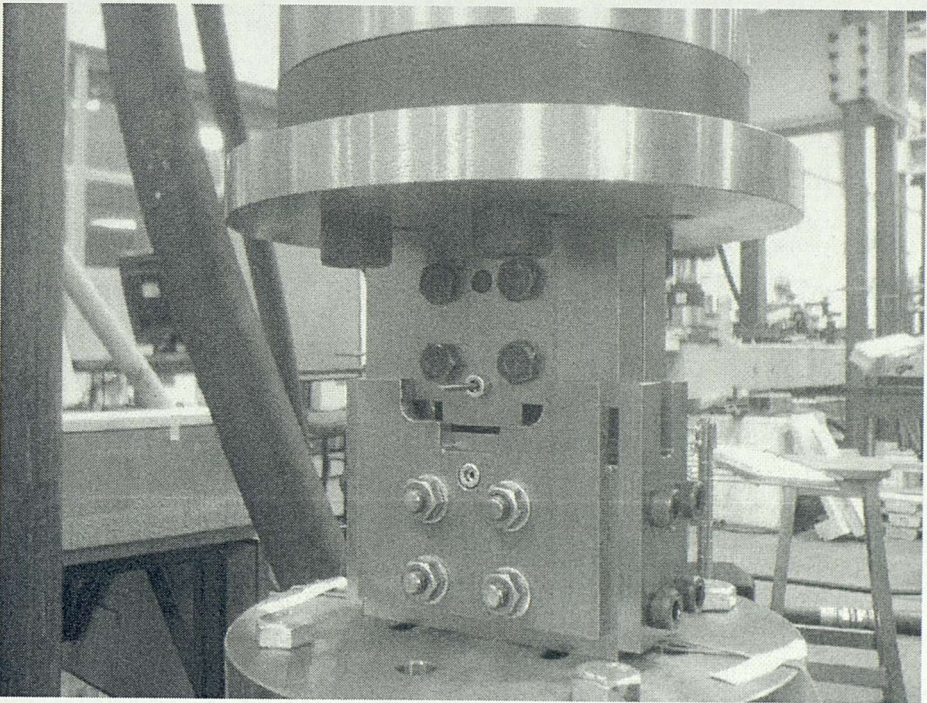


Figure 3.4: *Cyclic axial test setup*

3.2.3 *Bending tests*

In axial fatigue tests, specimens are prone to buckling under high strain amplitudes. Preventive measures such as using stockier specimens are not always feasible. For example, in the context of the present study, the thickness of the specimens was restricted to that of the thin-walled tubular members from which they were extracted. Bending fatigue test specimens are not prone to buckling and thus higher strain amplitudes can be applied.

A four-point minor-axis bending set-up (Figure 3.5) was adopted in the current study to convert the axial displacement of the Instron 250 kN actuator into a constant bending moment along the specimen gauge region. This enabled fully reversed constant strain amplitude testing of the coupons under uniform bending. Curvature of the specimens was concentrated in the region of the reduced cross-section and hence the location of fracture was constrained. The end supports of the testing apparatus were free to slide on rollers, minimising any axial force in the coupons.

Under a given applied displacement, a thick coupon will experience higher strain on the surface than a thin coupon. Utilising the full stroke of the actuator, 75 mm, tests were carried out at $\pm 2\%$, $\pm 6\%$, $\pm 10\%$ and $\pm 15\%$ strain amplitudes on the 4 mm thick specimens but at lower strain amplitudes of $\pm 2\%$, $\pm 6\%$, $\pm 8\%$ and $\pm 10\%$ on the 3 mm thick specimens. Tests were conducted at strain rates of about $5 \times 10^{-3} \text{ s}^{-1}$. Strain gauges were attached to both the upper and lower faces of the coupons (Figure 3.3). Similar to the axial tests, the amplitudes of applied displacement were initially adjusted according to the readings of the strain gauges to achieve the desired strain amplitudes, after which constant displacement amplitudes were maintained. Failure was defined as in the axial tests.

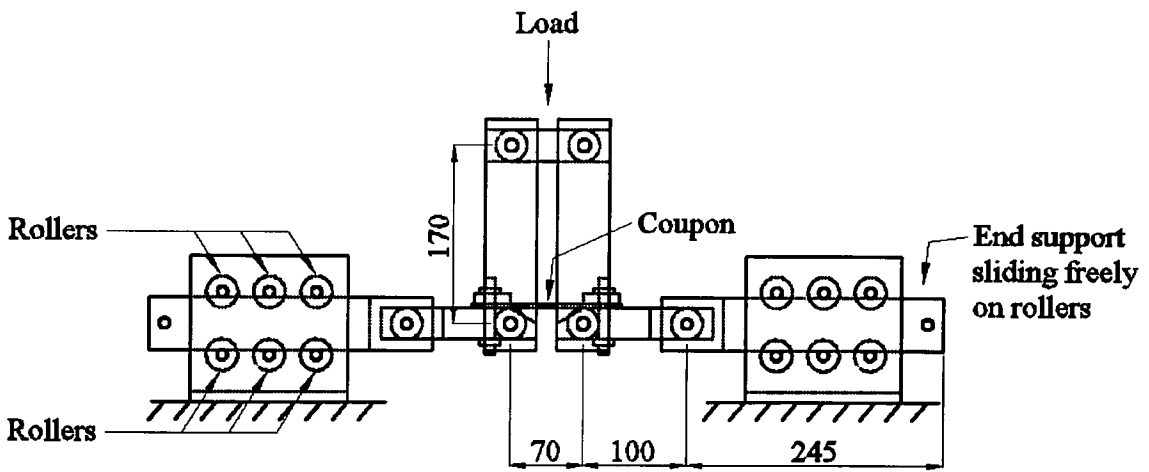


Figure 3.5: Cyclic bending test setup (dimensions in mm)

3.3 Axial test results

In this section, the results of the constant strain amplitude fully-reversed cyclic axially loaded test specimens are presented and analysed. Cyclic stress-strain curves are provided, strain-life relationships are developed and calibration of a cyclic material hardening model is performed.

3.3.1 Cyclic stress-strain curves

When a family of stabilised hysteresis loops with various strain amplitudes is plotted on the same graph, a cyclic stress-strain curve can be defined by tracing the locus of the tips of the loops (Figure 3.6). The resulting curve may be represented by a form of Ramberg-Osgood relationship, written in terms of stress and strain amplitudes, as given by:

$$\frac{\Delta \epsilon}{2} = \frac{\Delta \epsilon_e}{2} + \frac{\Delta \epsilon_p}{2} = \frac{\Delta \sigma}{2E} + \left(\frac{\Delta \sigma}{2K'} \right)^{1/n'} \quad (3.4)$$

where $\Delta \epsilon/2$ is the total strain amplitude, $\Delta \epsilon_e/2$ is the elastic strain amplitude, $\Delta \epsilon_p/2$ is the plastic strain amplitude, $\Delta \sigma$ is the stabilised stress amplitude at half of the number of cycles to failure, E is the Young's modulus, K' is the cyclic strength coefficient and n' is the cyclic strain hardening exponent. The material constants K' and n' can be obtained from a power law regression fit to the stress amplitude versus plastic strain amplitude data.

Three approaches for establishing cyclic stress-strain curves have been employed in previous studies, referred to herein as the multiple-step, incremental-step and companion methods. In the multiple step method, a specimen is cycled at constant strain amplitude. Once the material behaviour has stabilised, the strain amplitude is increased and again held constant until the response re-stabilises. This process continues until the full cyclic stress-strain curve has been established from a single specimen. In the incremental step method, a single specimen is subjected to repeated blocks of incrementally increasing and decreasing strain amplitudes. Generally, after a few blocks of loading cycles, the response stabilises and the cyclic stress-strain curve

can be obtained from half of a stable block. These two methods have the convenience of using a single specimen to determine the cyclic stress-strain curve, but their use becomes restricted at large strain amplitudes because specimens are likely to fail prematurely before stabilisation is achieved. In the companion method, a series of specimens are required and each specimen is subjected only to a single strain amplitude. The response at half of the number of cycles to failure is used to obtain cyclic stress-strain curves. This method is suitable for establishing cyclic stress-strain curves at high strain amplitudes and has therefore been used in this study.

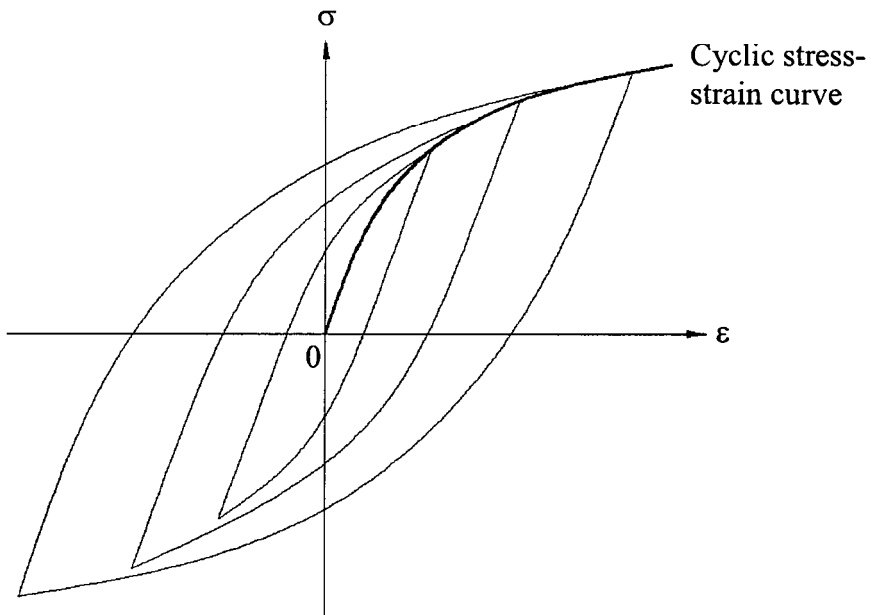


Figure 3.6: *Stabilised cyclic stress-strain hysteresis loops*

The extent of cyclic hardening or softening is revealed by cyclic stress-strain curves, and may be measured relative to the monotonic curve. The cyclic and monotonic stress-strain curves from two sections of each material are plotted in Figures 3.7-3.12. Stainless steel, shown in Figures 3.11 and 3.12, exhibits strong cyclic hardening; as the strain amplitude increases, the stabilised stress amplitude rises substantially to values much higher than the ultimate stress reaches in tensile tests. The hot-rolled and cold-formed carbon steel, shown in Figures 3.7-3.8 and 3.9-3.10, respectively, exhibit less pronounced cyclic hardening than stainless steel.

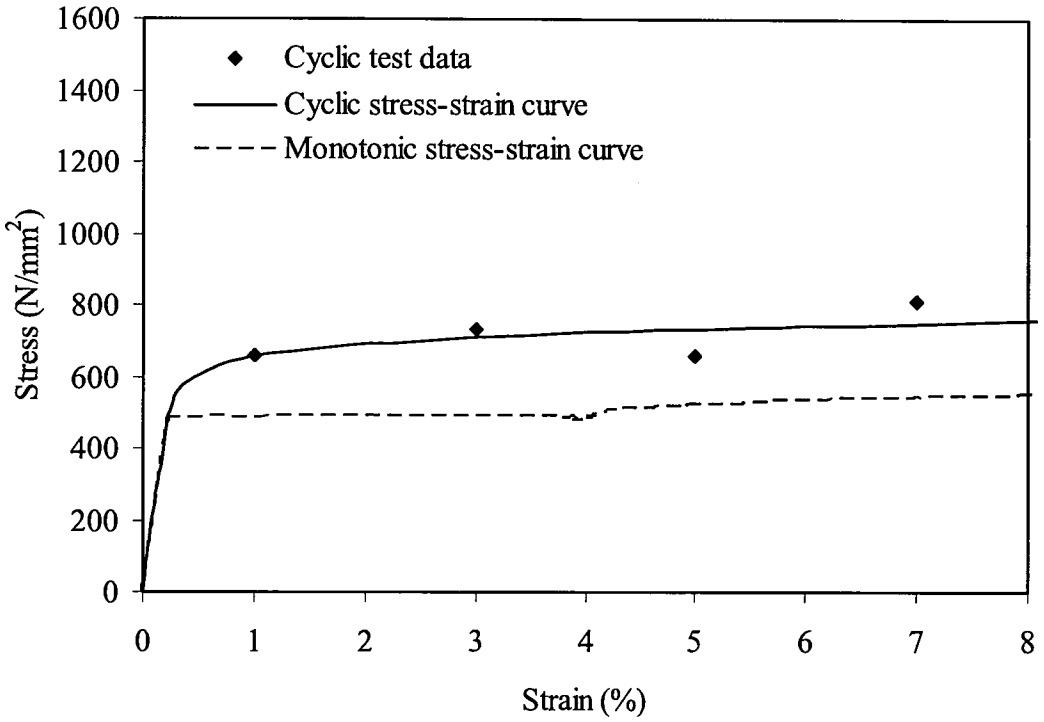


Figure 3.7: Comparison of cyclic and monotonic stress-strain curves for hot-rolled carbon steel 40×40×3 specimens

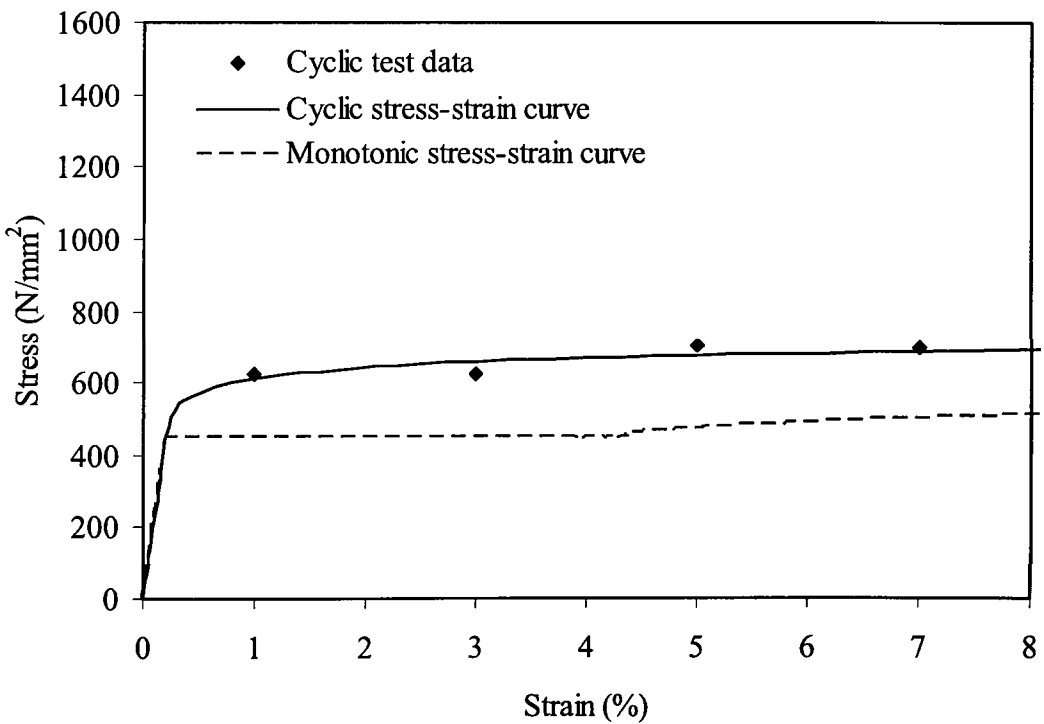


Figure 3.8: Comparison of cyclic and monotonic stress-strain curves for hot-rolled carbon steel 40×40×4 specimens

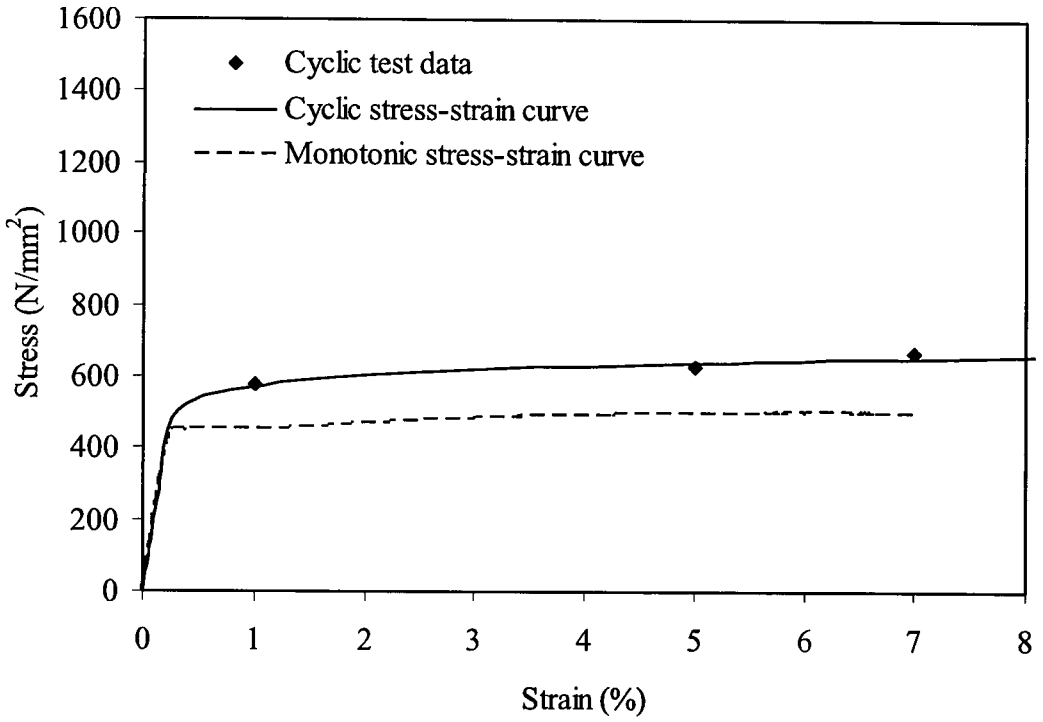


Figure 3.9: Comparison of cyclic and monotonic stress-strain curves for cold-formed carbon steel 40×40×3 specimens

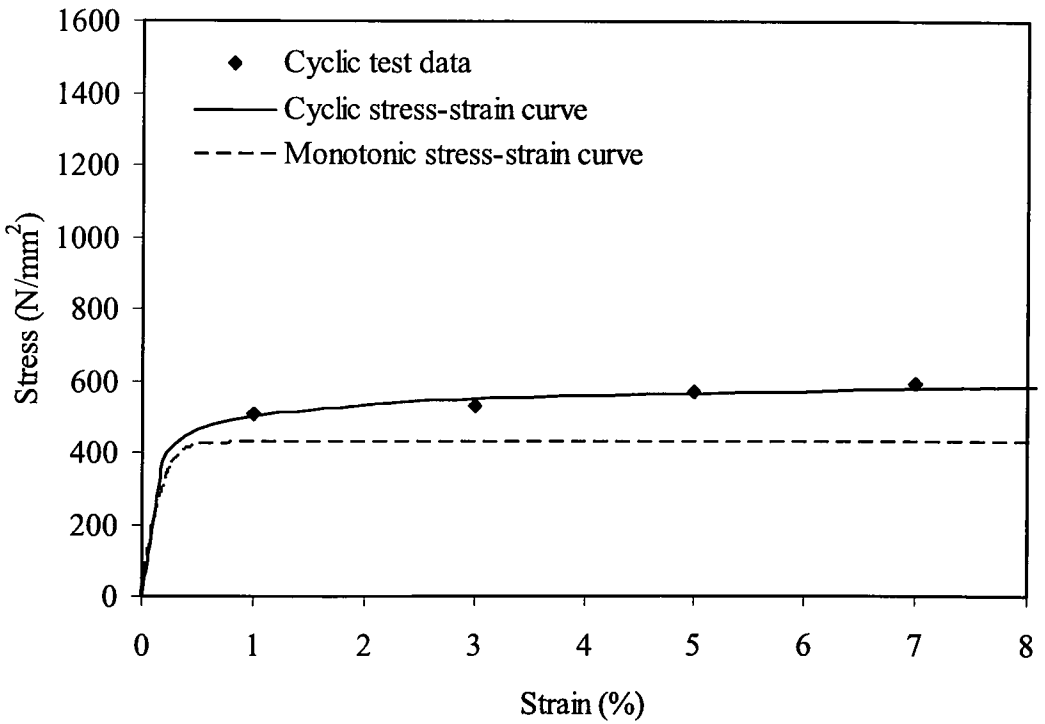


Figure 3.10: Comparison of cyclic and monotonic stress-strain curves for cold-formed carbon steel 40×40×4 specimens

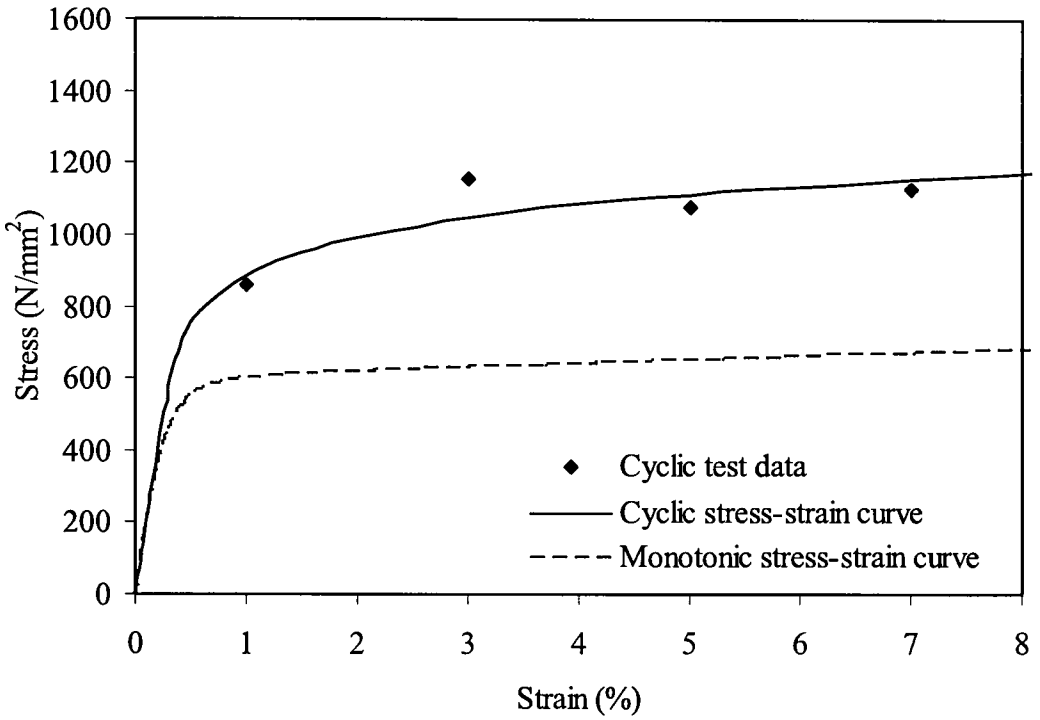


Figure 3.11: Comparison of cyclic and monotonic stress-strain curves for cold-formed stainless steel 50×50×3 specimens

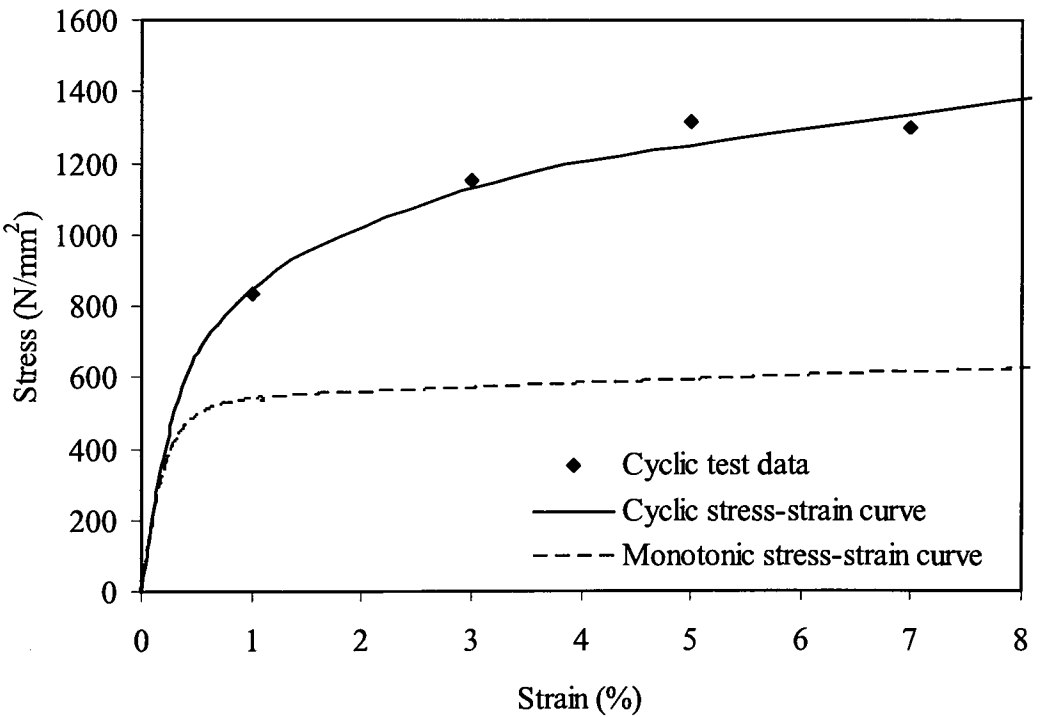


Figure 3.12: Comparison of cyclic and monotonic stress-strain curves for cold-formed stainless steel 60×60×3 specimens

From the test results, the four material parameters in Equations 3.1 and 3.4 have been derived and are listed in Table 3.2. Results derived from selected individual sections, together with average values for each material, are presented. It is known that fatigue tests often show considerable scatter even in carefully controlled environments. Some scatter is evident, particularly for the stainless steel specimens, in the present study.

The rate and extent of cyclic hardening can also be shown by examining the progression of stress amplitudes at each cycle throughout the tests. This is shown at the four considered strain amplitudes ($\pm 1\%$, $\pm 3\%$, $\pm 5\%$, $\pm 7\%$) for hot-rolled carbon steel in Figures 3.13-3.14, cold-formed carbon steel in Figures 3.15-3.16 and cold-formed stainless steel in Figures 3.17-3.18. The tests on hot-rolled and cold-formed carbon steel show a small degree of cyclic hardening, and the material response stabilises in the first few cycles. For stainless steel, the degree of cyclic hardening is relatively low in the $\pm 1\%$ strain amplitude tests, and the material response stabilises at about half the number of cycles to failure. This accords with the findings of Bernard-Connolly et al. (1983) who studied the cyclic properties of stainless steel (grade 1.4301) and observed low levels of cyclic hardening at low strain amplitudes and stated that below $\pm 0.5\%$ strain amplitudes cyclic softening occurs. At the higher strain amplitudes employed in the current study, stainless steel shows strong cyclic hardening (Figures 3.17 and 3.18). At $\pm 7\%$ strain amplitude, the stainless steel specimens showed the largest degree of cyclic hardening but failure occurred before the material response had fully stabilised.

Table 3.2: *Parameters of Coffin-Manson law and cyclic stress-strain curve*

Specimen	K'	n'	ϵ'_f	c
40×40×3-CS-HR	890	0.0622	0.6115	-0.6658
40×40×4-CS-HR	807	0.0564	0.6823	-0.6496
All hot CS	847	0.0591	0.6406	-0.6581
40×40×4-CS-CF	702	0.0688	0.4143	-0.6532
40×40×3-CS-CF	764	0.0580	0.3937	-0.525
All cold CS	726	0.0634	0.4027	-0.6392
60×60×3-SS-CF	2348	0.1975	0.5343	-0.6742
50×50×3-SS-CF	1562	0.1081	0.3875	-0.6534
All cold SS	1768	0.1333	0.4352	-0.6766

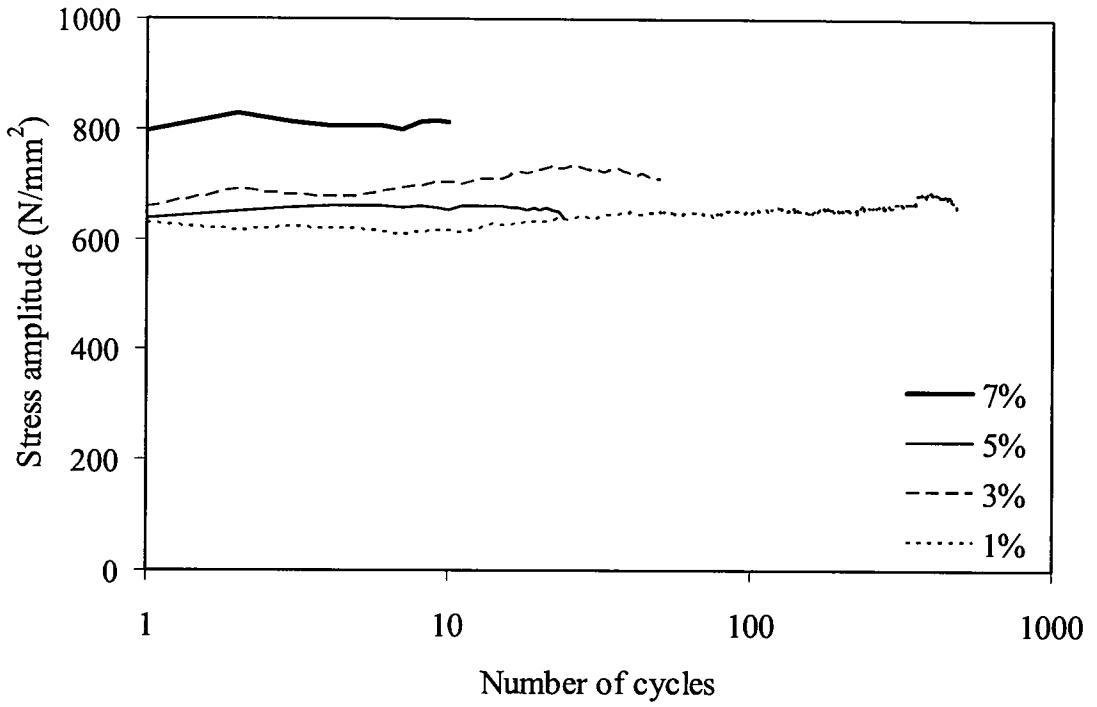


Figure 3.13: Variation of stress amplitudes during tests at different strain amplitudes of hot-rolled carbon steel 40×40×3 specimens

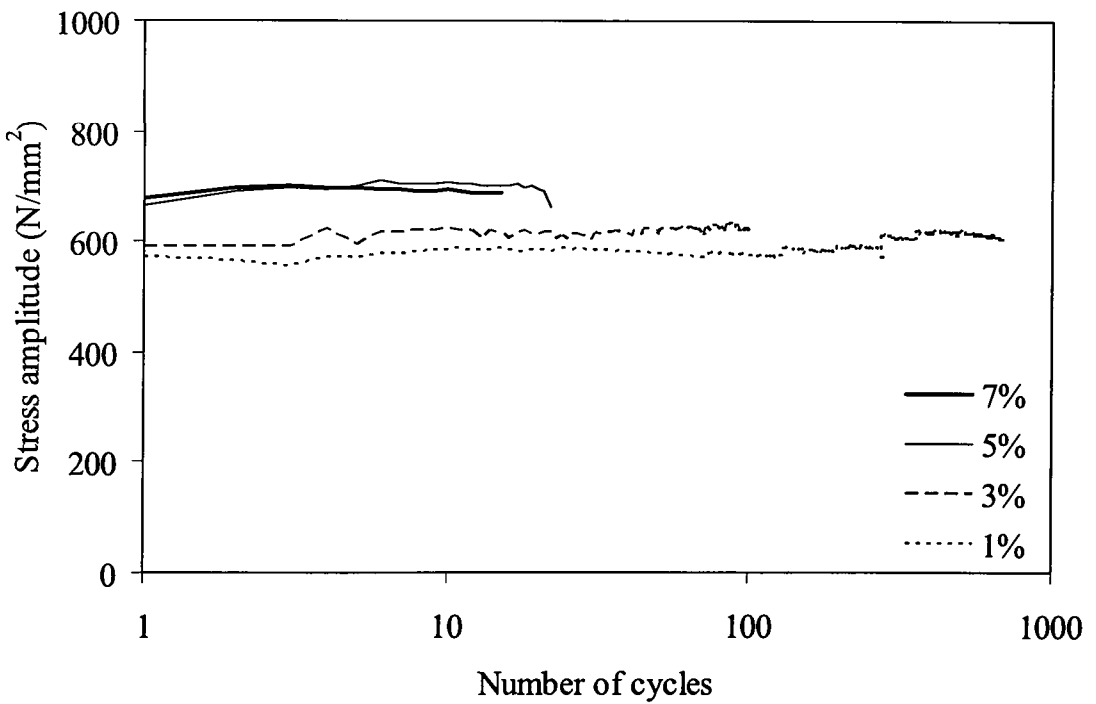


Figure 3.14: Variation of stress amplitudes during tests at different strain amplitudes of hot-rolled carbon steel 40×40×4 specimens

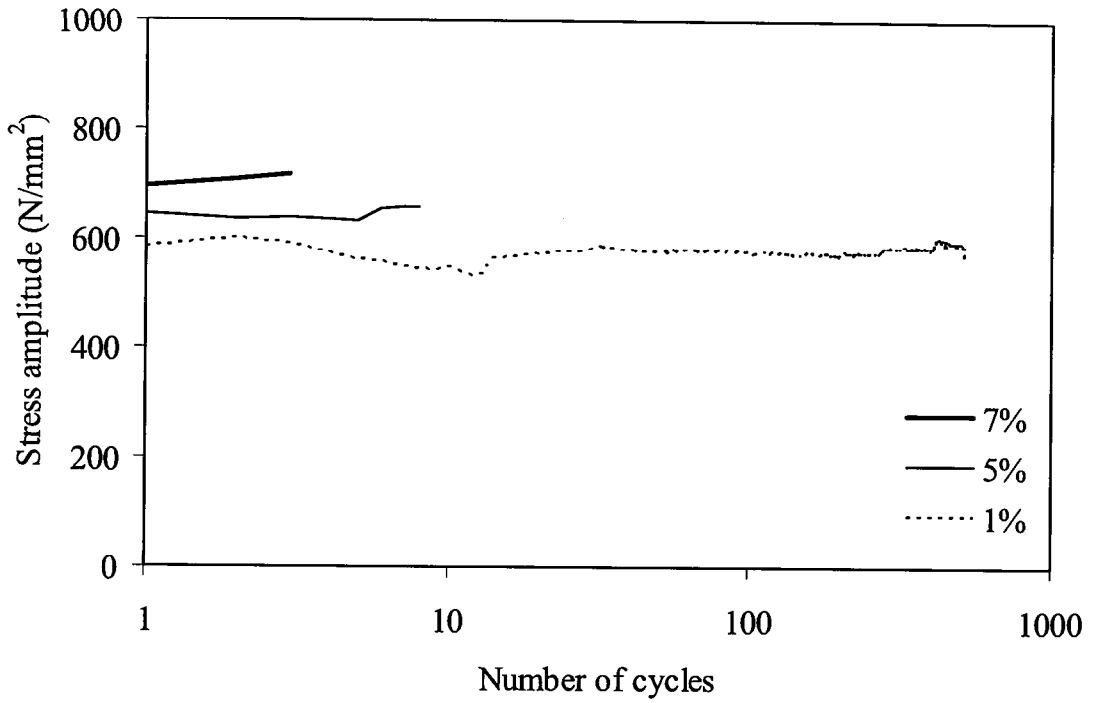


Figure 3.15: Variation of stress amplitudes during tests at different strain amplitudes of cold-formed carbon steel 40×40×3 specimens

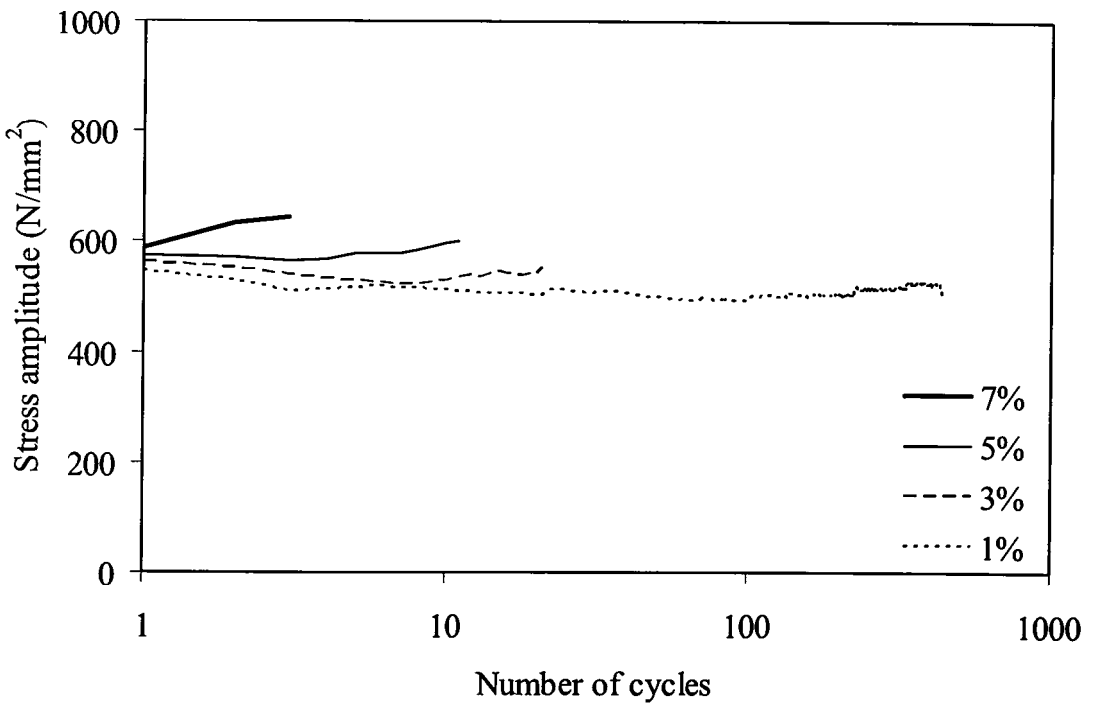


Figure 3.16: Variation of stress amplitudes during tests at different strain amplitudes of cold-formed carbon steel 40×40×4 specimens

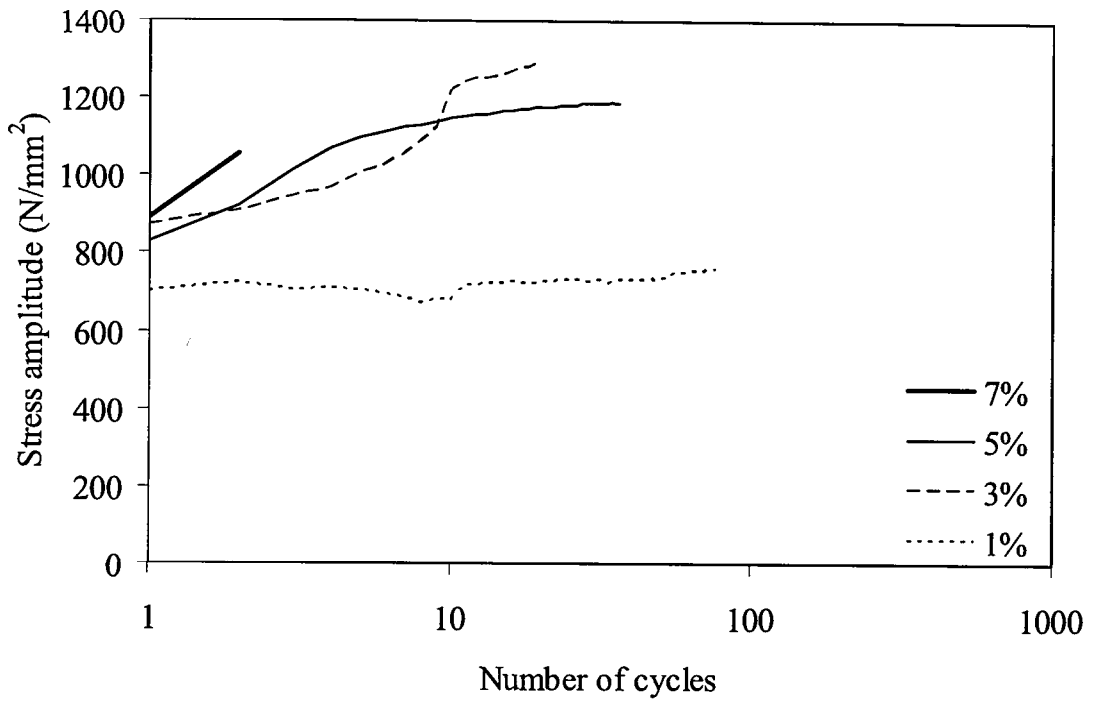


Figure 3.17: Variation of stress amplitudes during tests at different strain amplitudes of cold-formed stainless steel 50×50×3 specimens

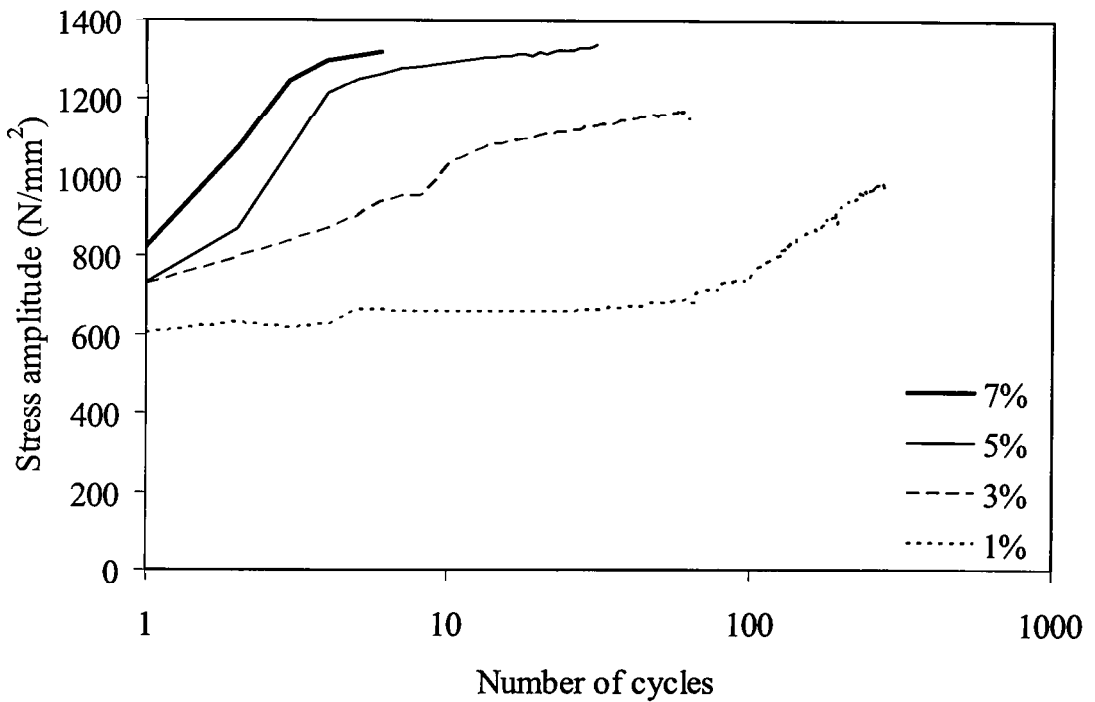


Figure 3.18: Variation of stress amplitudes during tests at different strain amplitudes of cold-formed stainless steel 60×60×3 specimens

3.3.2 Strain-life relationship

In order to calibrate the material parameters of the Coffin-Manson relationship (Equation 3.1), the total strain amplitudes were resolved into elastic and plastic components. Although the total strain amplitudes were kept constant throughout the tests, the amplitudes of the elastic and plastic components changed due to cyclic hardening of the materials. However, cyclic hardening was transient and the material behaviour generally stabilised quite rapidly, as shown by the strain and stress measurements (Figures 3.19 and 3.20, respectively) from the first ten cycles of a typical $\pm 5\%$ strain amplitude test of a stainless steel specimen. The initial increase in stress amplitude stabilised after 5 cycles of loading. The stabilised stress amplitude $\Delta\sigma$ at half the number of cycles to failure was used to calculate the elastic strain amplitude from the following expression:

$$\frac{\Delta\varepsilon_e}{2} = \frac{\Delta\sigma}{2E} \quad (3.5)$$

where the terms are defined as in Equation 3.4. The plastic strain amplitude was then obtained from the difference between the total and elastic strain amplitudes.

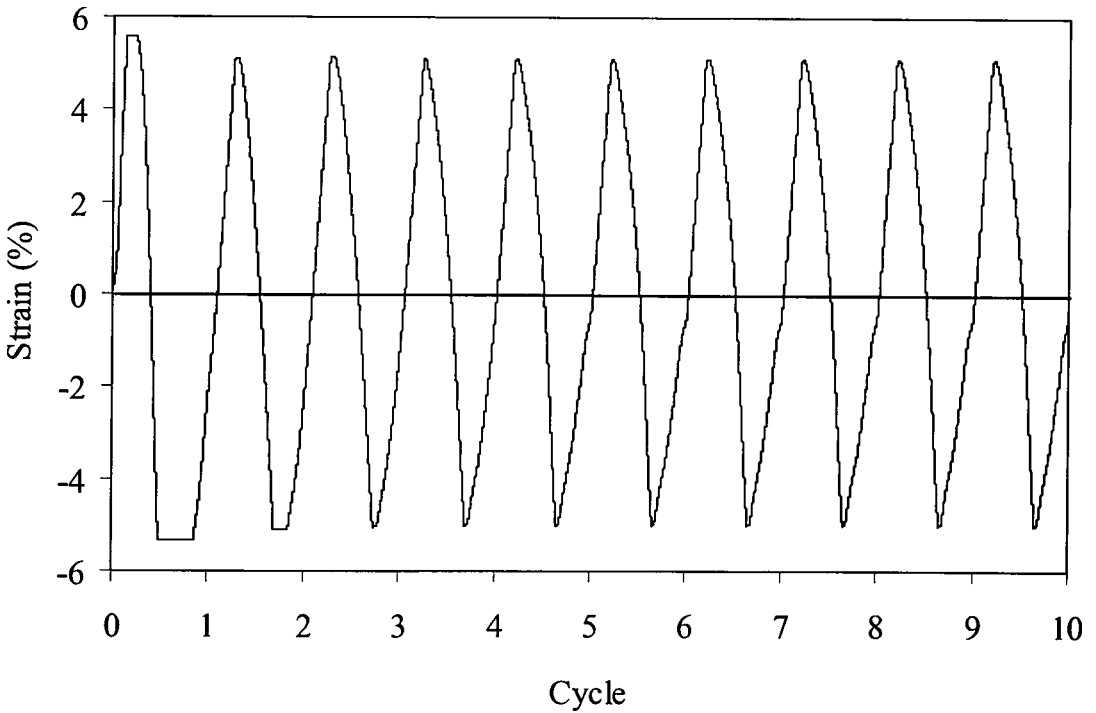


Figure 3.19: Measurement of strain during the first 10 cycles of a typical test at $\pm 5\%$ amplitude (60 $\times$ 40 $\times$ 3-SS-CF-5% (a))

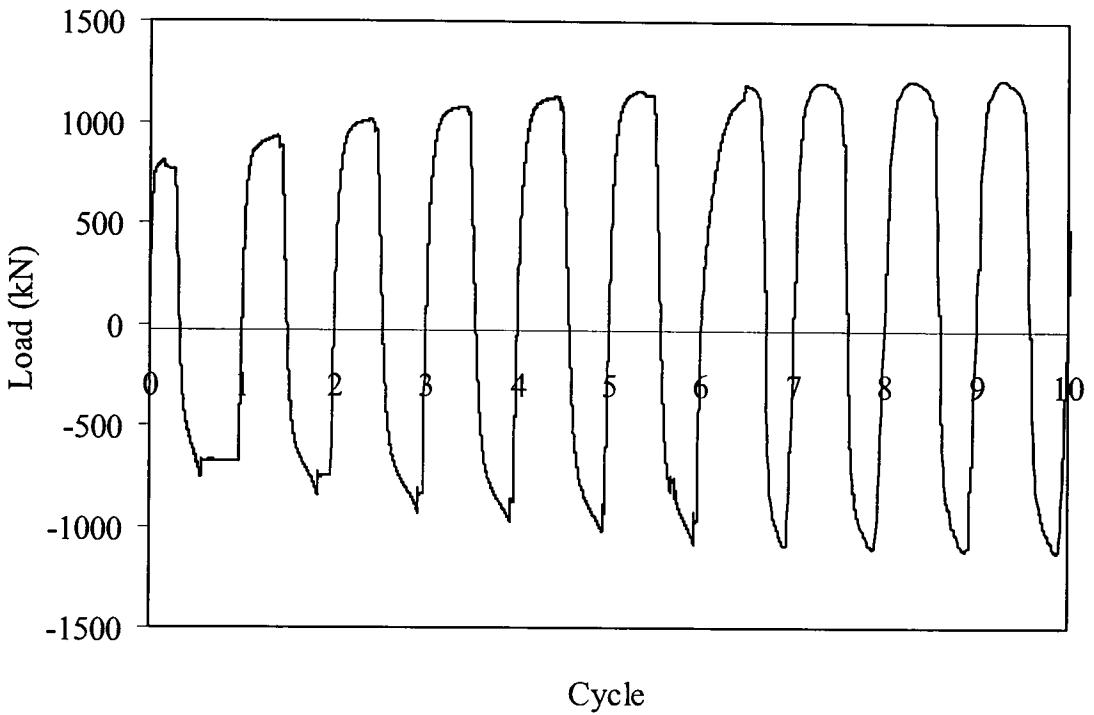


Figure 3.20: *Measurement of load during the first 10 cycles of a typical test at $\pm 5\%$ amplitude ($60 \times 40 \times 3$ -SS-CF-5% (a))*

The results of all specimens tested in the axial configuration are summarised in Table 3.3. Up to four specimens were cut from each section and were tested at different strain amplitudes. The total strain amplitudes and stress amplitudes at half of the number of cycles to failure were measured in the tests while the elastic and plastic strain amplitudes were calculated as described above. The number of reversals to failure is plotted against elastic and plastic strain amplitudes on a log-log scale for each material and shown in Figures 3.21-3.23. Tensile test data are included as 0.5 reversal tests at strain amplitudes equal to the tensile ductility (calculated on the basis of area reduction through Equation 3.3). As a characteristic of LCF tests, the plastic strain amplitudes account for the majority of the total strain amplitudes. Variation in fatigue life in LCF largely depends on changes in plastic strain amplitudes. In contrast, elastic strain amplitudes are small and their variation is minor so their influence on fatigue life in LCF is relatively insignificant.

Table 3.3: Results of cyclic axial material tests

Specimen	Total strain amplitude $\Delta\varepsilon/2$ (%)	Elastic strain amplitude $\Delta\varepsilon_e/2$ (%)	Plastic strain amplitude $\Delta\varepsilon_p/2$ (%)	Stress amplitude at half-life $\Delta\sigma/2$ (N/mm ²)	Number of reversals to failure $2N_f$
40×40×4-CS-HR-1%	1	0.3	0.7	621	1465
40×40×4-CS-HR-3% *	3	0.3	2.7	623	215
40×40×4-CS-HR-5%	5	0.3	4.7	705	45
40×40×4-CS-HR-7%	7	0.3	6.7	695	31
40×40×3-CS-HR-1%	1	0.3	0.7	657	991
40×40×3-CS-HR-3%	3	0.3	2.7	732	107
40×40×3-CS-HR-5%	5	0.3	4.7	660	49
40×40×3-CS-HR-7%	7	0.4	6.6	810	19
40×40×4-CS-CF-1%	1	0.3	0.7	508	879
40×40×4-CS-CF-3%	3	0.3	2.7	531	43
40×40×4-CS-CF-5%	5	0.3	4.7	572	23
40×40×4-CS-CF-7%	7	0.3	6.7	592	7
40×40×3-CS-CF-1%	1	0.3	0.7	576	1043
40×40×3-CS-CF-3% *	3	0.2	2.8	333	33
40×40×3-CS-CF-5%	5	0.3	4.7	627	17
40×40×3-CS-CF-7%	7	0.3	6.7	664	7
60×40×4-CS-CF-5% *	5	0.5	4.5	1160	41
100×100×3-SS-CF-1%	1	0.2	0.8	402	533
100×100×3-SS-CF-3%	3	0.6	2.4	1251	89
100×100×3-SS-CF-5%	5	0.5	4.5	1062	15
60×60×3-SS-CF-1%	1	0.4	0.6	835	557
60×60×3-SS-CF-3%	3	0.6	2.4	1152	157
60×60×3-SS-CF-5%	5	0.7	4.3	1315	63
60×60×3-SS-CF-7%	7	0.7	6.3	1299	13
50×50×3-SS-CF-1%	1	0.4	0.6	861	671
50×50×3-SS-CF-3%	3	0.6	2.4	1156	39
50×50×3-SS-CF-5%	5	0.5	4.5	1076	73
50×50×3-SS-CF-7%	7	0.6	6.4	1126	5
60×40×3-SS-CF-3%	3	0.6	2.4	1067	55
60×40×3-SS-CF-5% (a)	5	0.6	4.4	1127	25
60×40×3-SS-CF-5% (b)	5	0.6	4.4	1083	9

* Data point rejected due to abnormal load values.

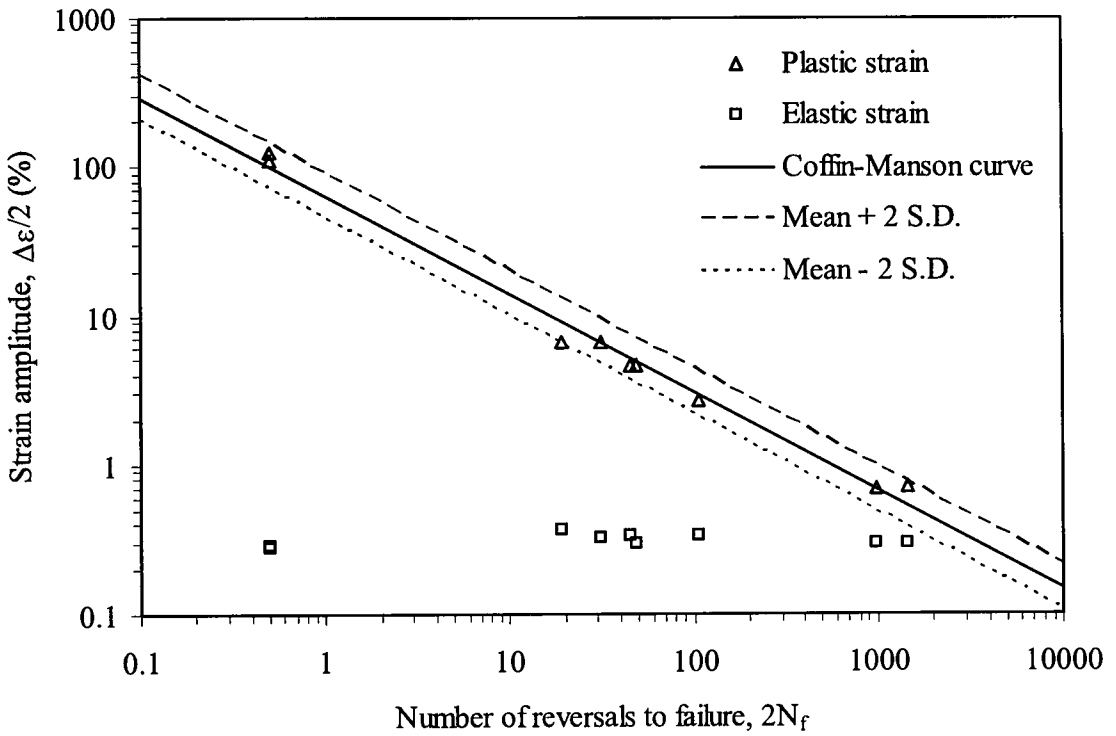


Figure 3.21: Strain-life relationship in ELCF and LCF regimes for hot-rolled carbon steel specimens

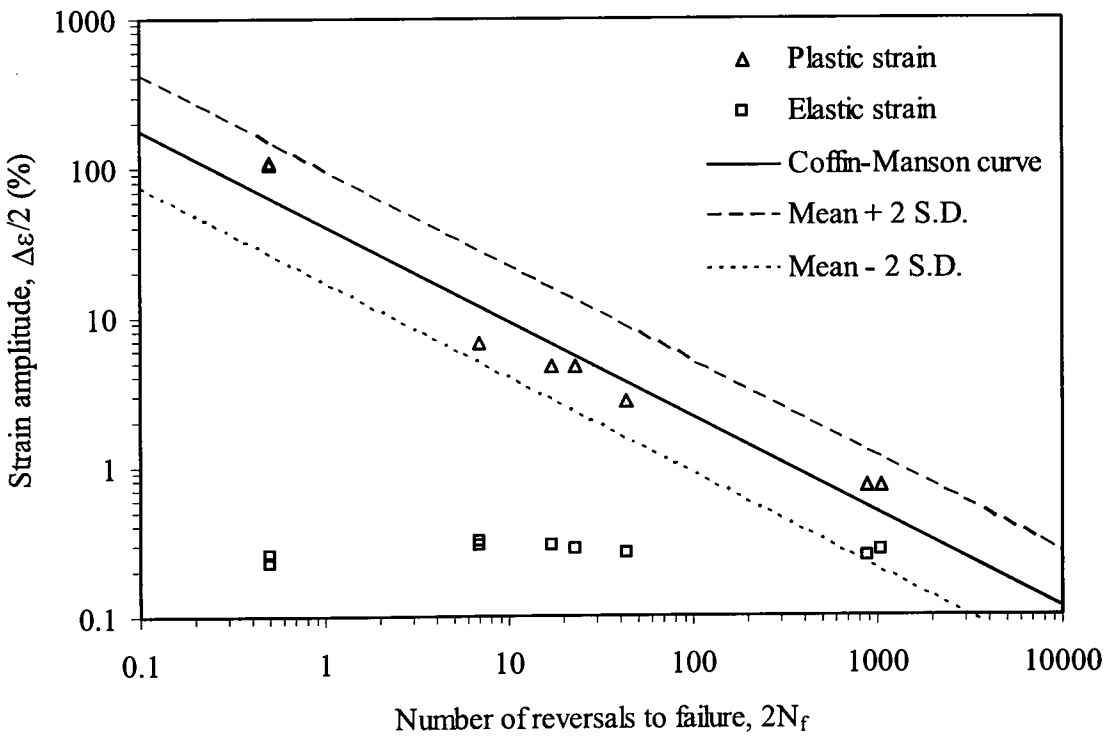


Figure 3.22: Strain-life relationship in ELCF and LCF regimes for cold-formed carbon steel specimens

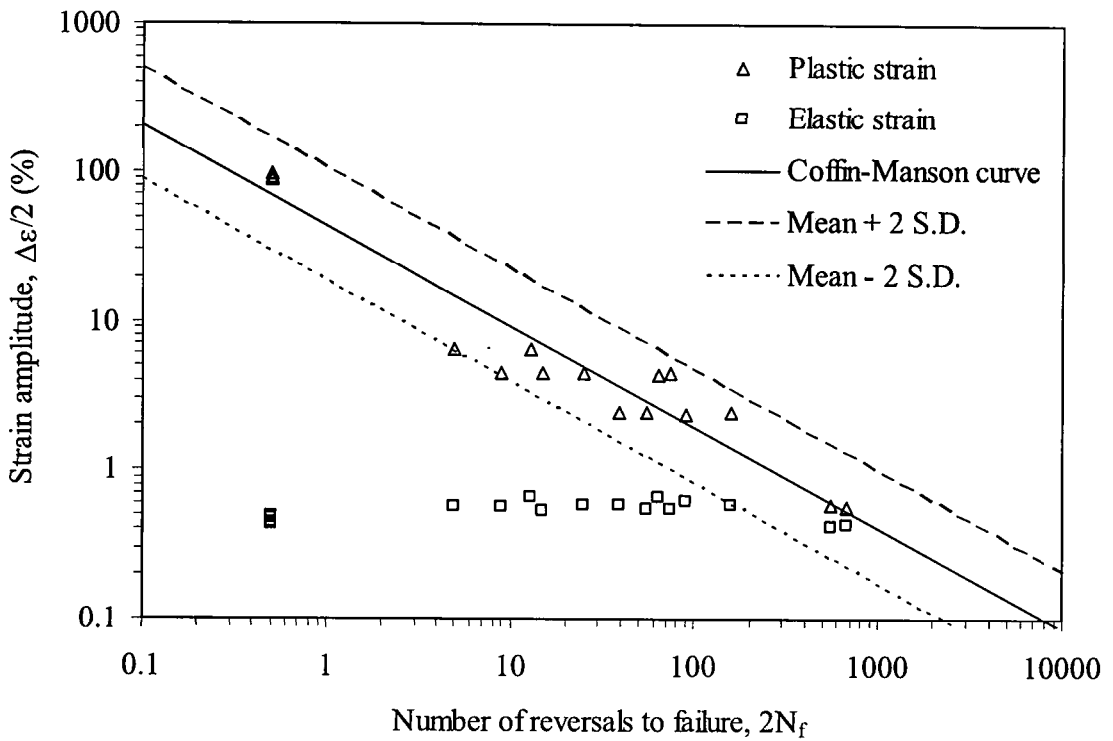


Figure 3.23: *Strain-life relationship in ELCF and LCF regimes for cold-formed stainless steel specimens*

The Coffin-Manson relationship (Equation 3.1) has been fitted to the plastic strain-life data and presented as solid lines in Figures 3.21-3.23, while scatter bands at two times the standard deviation are shown as dashed lines. The plastic strain-life data may be seen to be well predicted by the Coffin-Manson relationship though the scatter is larger for the cold-formed material than for the hot-rolled material. This may reflect the greater inhomogeneity in material properties associated with cold-formed material due to variable strain history experienced during sheet and section forming. The test data for the three materials and their respective Coffin-Manson curves are compared in Figure 3.24, where the hot-rolled carbon steel displays superior fatigue life than the cold-formed carbon steel and cold-formed stainless steel. The difference is partly attributed to the increased number of inclusions present in the cold-formed material which act as sites for crack initiation (Suresh, 1991). The reduced ductility due to cold-work will also reduce its resistance to failure through the ductility exhaustion mechanism.

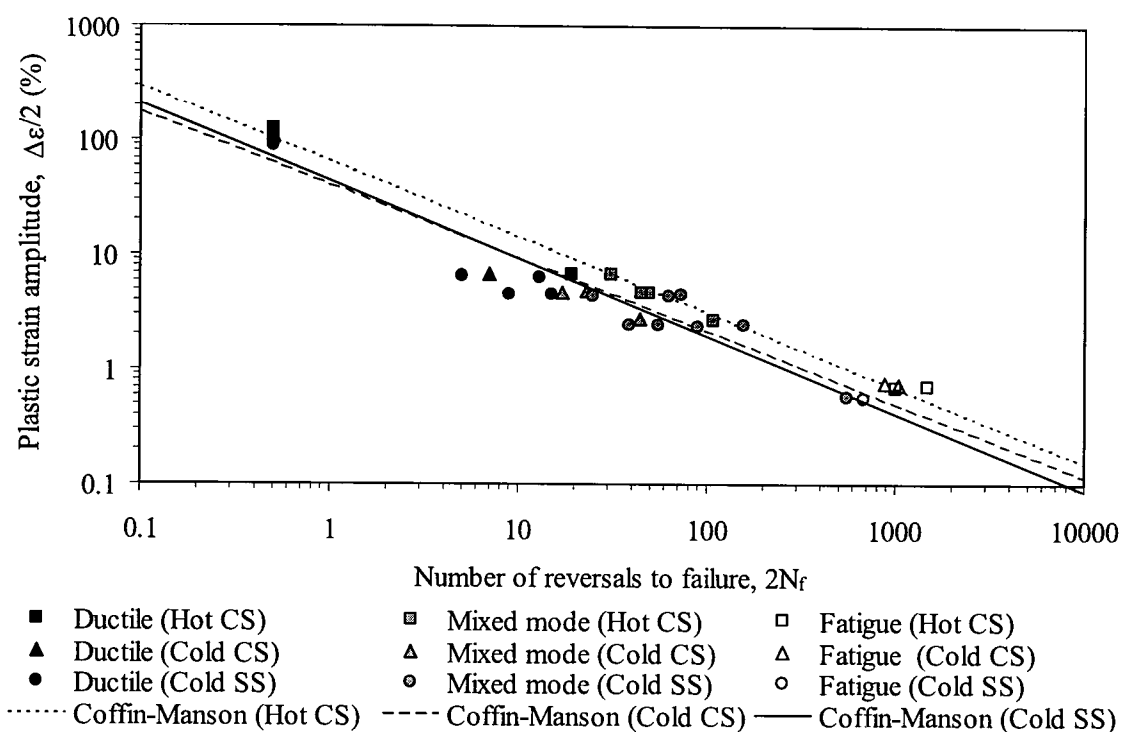


Figure 3.24: Comparison of the strain-life relationships for hot-rolled carbon steel, cold-formed carbon steel and cold-formed stainless steel, and their failure modes

The importance of the level of cold-work on strain-life response can be observed by comparing the data from the current study to existing LCF data from the literature (Gong & Norton, 1999; Yang, 2005; Bhanu Sankara Rao et al., 1993; Bergengre et al., 1995; Smith et al., 1963; Plumbridge et al., 1980; Lefebvre & Ellyin, 1984; Wong et al., 2001; Chiou & Yip, 2005; Ye et al., 2006), as shown in Figures 3.25 and 3.26. Although scattered, the results generally indicate the anticipated trend that fatigue performance is degraded in cold-worked material. Furthermore, materials in cold-formed structural members, particularly in the corner regions, will typically undergo higher levels of cold-worked than the cold-worked steel sheets tested in the literature.

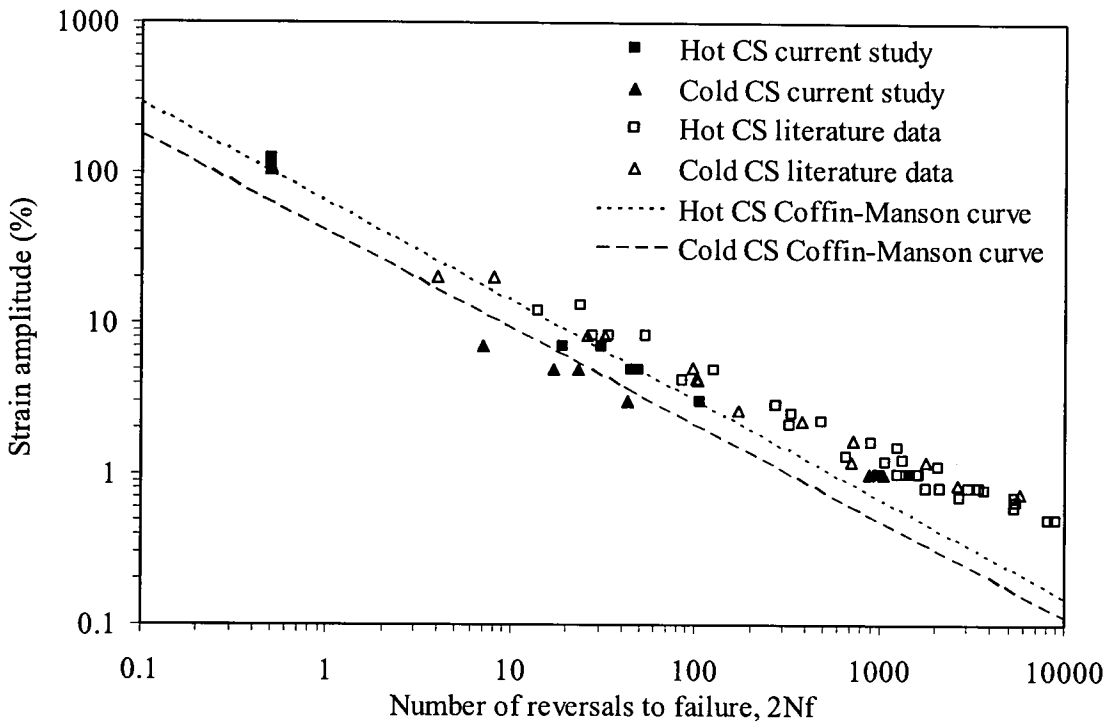


Figure 3.25: Strain-life relationships of low and high cycle fatigue data, and monotonic tensile data, of carbon steel (CS) material from tests performed in the current study and in the literature

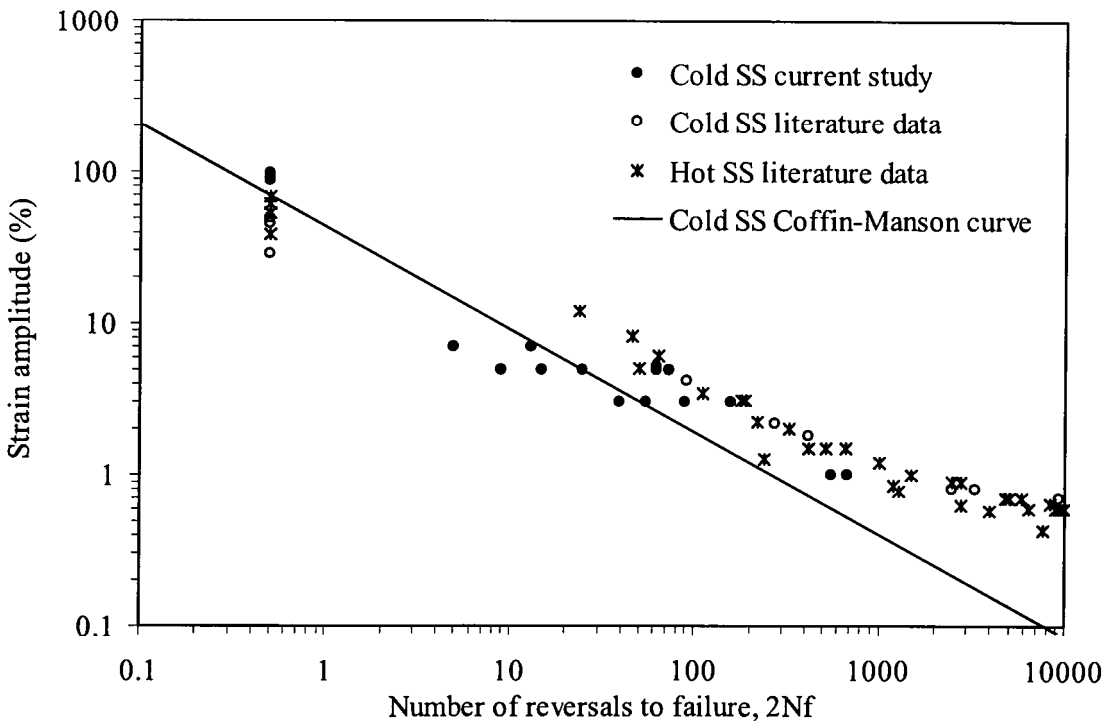


Figure 3.26: Strain-life relationships of low and high cycle fatigue data, and monotonic tensile data, of stainless steel (SS) material from tests performed in the current study and in the literature

Since the range of strain amplitudes applied in the current study cover LCF, ELCF and tensile tests, specimens have failed by various mechanisms. The two failure modes at the highest and lowest strain amplitudes are ductile failure, as observed in the tensile tests, and fatigue failure, as generally seen in the LCF tests, respectively. The ductile damage failure mechanism typically occurs when the number of cycles to failure is below 100. Failure through ductile damage leads to a relatively uneven surface that is dull in appearance, and often exhibits a cup-and-cone profile. Fatigue failure is characterized by a series of 'beach' marks that indicate the progressive advancement of a fatigue crack. Crack initiation takes place at the specimen surface where dislocation slip may most easily occur, and propagates inwards creating a relatively flat fracture surface.

The fracture surface of all specimens in the current study have been examined carefully and categorised into different failure modes. Ductile fracture surfaces such as the one shown in Figure 3.27 and fatigue fracture surface with clear beach marks as shown in Figure 3.28 have been classed as ductile and fatigue modes respectively. Specimens which do not show distinct ductile or fatigue failure features are classified as mixed mode failure (Figure 3.29).

The failure modes of each specimen are also indicated in Figure 3.24 – all tensile coupons have failed in a ductile manner while most of the tests which were carried out in the LCF regime, i.e. $\pm 1\%$ strain amplitude, showed fatigue fracture. Specimens that failed in ELCF regime exhibited either a ductile damage or mixed mode failure. This change in damage mode may partly explain the scatter in the test results.

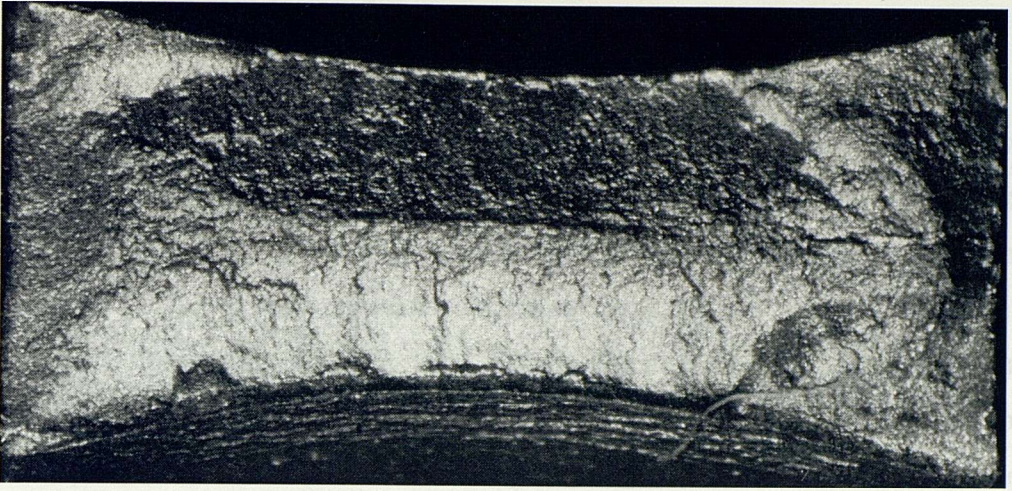


Figure 3.27: Ductile fracture surface shown on an axially loaded specimen tested at $\pm 7\%$ strain amplitudes

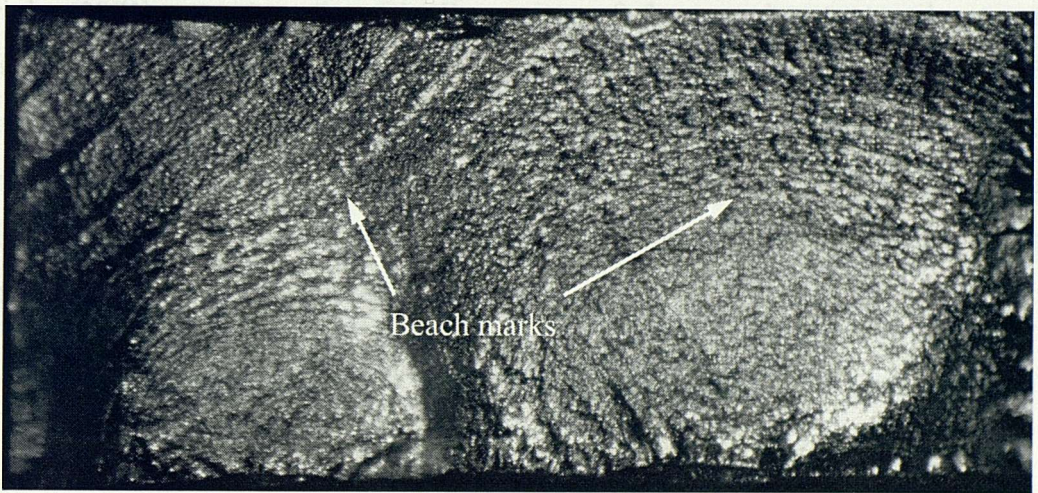


Figure 3.28: Fatigue fracture surface shown on an axially loaded specimen tested at $\pm 1\%$ strain amplitudes



Figure 3.29: Mixed mode fracture surface shown on an axially loaded specimen tested at $\pm 3\%$ strain amplitudes

3.3.3 Low and extremely low cycle fatigue relationships

It has been shown in previous studies (Kuroda, 2001; Tateishi et al., 2007) that when the number of reversals to failure falls below approximately 200, the strain-life relationship may deviate from the Coffin-Manson model, due to changes in the damage mechanisms. As the strain amplitudes increases from the LCF regime to the ELCF regime, the failure mode changes from fatigue fracture to accumulation of ductile damage. Various models have been proposed to establish a unified strain-life relationship for both the LCF and ELCF regimes. Two of these models are discussed below.

Kuroda (2001) considers that ELCF consists of damage due to tensile straining, ductility exhaustion during cyclic straining and crack propagation. The combined effect of these three damage modes determines the fatigue life of a material and the total damage fraction during a cycle is given by the sum of the damage fractions accumulated due to each of these mechanisms. Failure is conceded when the total damage fraction attains unity, i.e.:

$$\frac{\varepsilon_{p,t}}{\varepsilon_f} + 4N_f \left(\frac{\Delta\varepsilon_p}{2\varepsilon_f} \right)^{1/\alpha'} + \frac{\Delta\varepsilon_p N_f^{\alpha'}}{C'} = 1 \quad (3.6)$$

where $\varepsilon_{p,t}$ is the maximum tensile plastic strain applied to the specimen, ε_f is the uniaxial tensile ductility (calculated on the basis of reduction in area at the fracture surface), N_f is the number of cycles to failure, $\Delta\varepsilon_p/2$ is the plastic strain amplitude, and α' and C' are material constants derived from the test data (Kuroda, 2001).

Tateishi et al. (2007) modified the Coffin-Manson relationship to account for ductile damage. The total damage in the material is taken to be the sum of the ductile damage fraction and fatigue damage fraction and again failure is conceded when the total damage attains unity. When the maximum plastic strain range $\Delta\varepsilon_{\max}$ exceeds a damage threshold value in pure tension, taken to be the plastic strain corresponding to the ultimate tensile stress from monotonic tensile tests ε_{pu} then the number of cycles to failure N_f is reduced by a factor linked to the ductile damage fraction C_m , as described by Equation 3.7:

$$\frac{\Delta \varepsilon_p}{2} = \varepsilon'_f (2N_f)^c C_m$$

$$C_m = \begin{cases} \left(\frac{\varepsilon_f - \Delta \varepsilon_{\max}}{\varepsilon_f - \varepsilon_{pu}} \right)^{-c} & \text{if } \Delta \varepsilon_{\max} > \varepsilon_{pu} \\ 1.0 & \text{if } \Delta \varepsilon_{\max} \leq \varepsilon_{pu} \end{cases} \quad (3.7)$$

where ε'_f and c are the Coffin-Manson constants.

These two ELCF life prediction models have been applied to the data generated in this study. Tensile test data are not applicable to the models and thus have not been used in the calibration process, but have been used to determine the fundamental material parameters (ε_{pu} and ε_f). The strain-life curves predicted by these models are compared with the Coffin-Manson curve in Figures 3.30-3.32. The two models provide improved agreement with the cyclic test data in the ELCF regime, but do not accurately predict the results of the tensile tests or the results of cyclic tests with higher cycles to failure. In contrast, the tensile test data can be included in the calibration of the unmodified Coffin-Manson relationship. The resulting curve is capable of representing the strain-life relationship of the materials ranging from the tensile tests up to 1000 cycles to failure with satisfactory accuracy. This enables the interpolation of the strain-life relationship between the tensile tests and the cyclic tests at the highest strain amplitude, which is useful for predicting the fatigue life of the materials in the region of local buckling of the bracing members discussed in the following chapters.

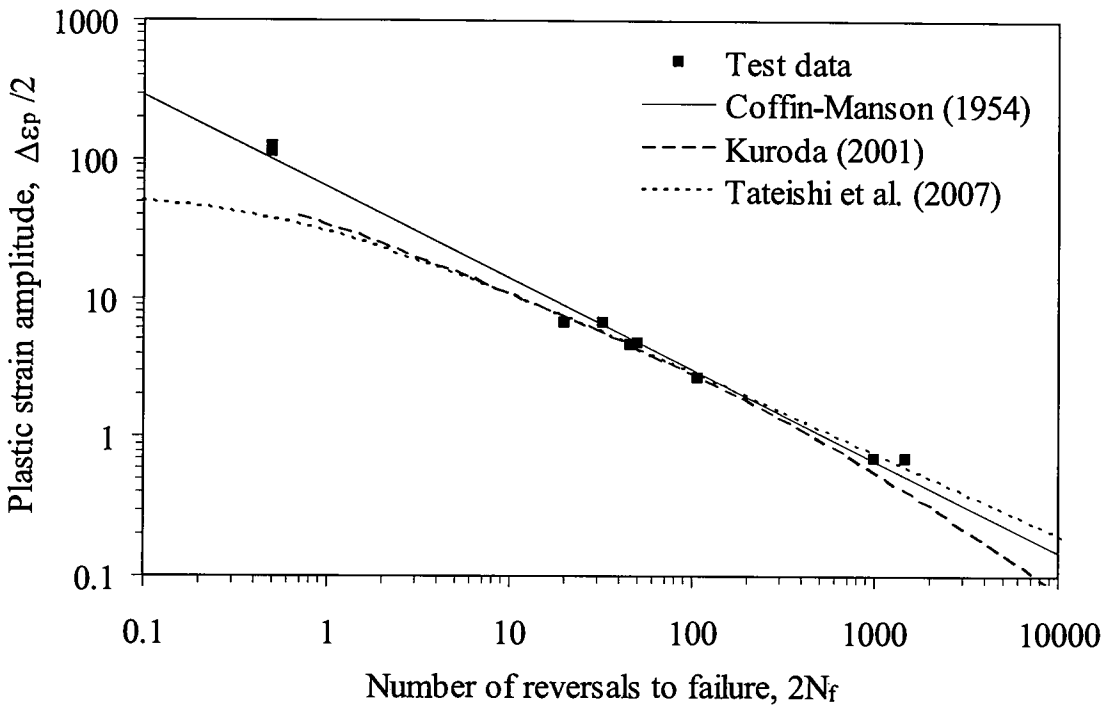


Figure 3.30: Comparison between strain-life models and hot-rolled carbon steel data

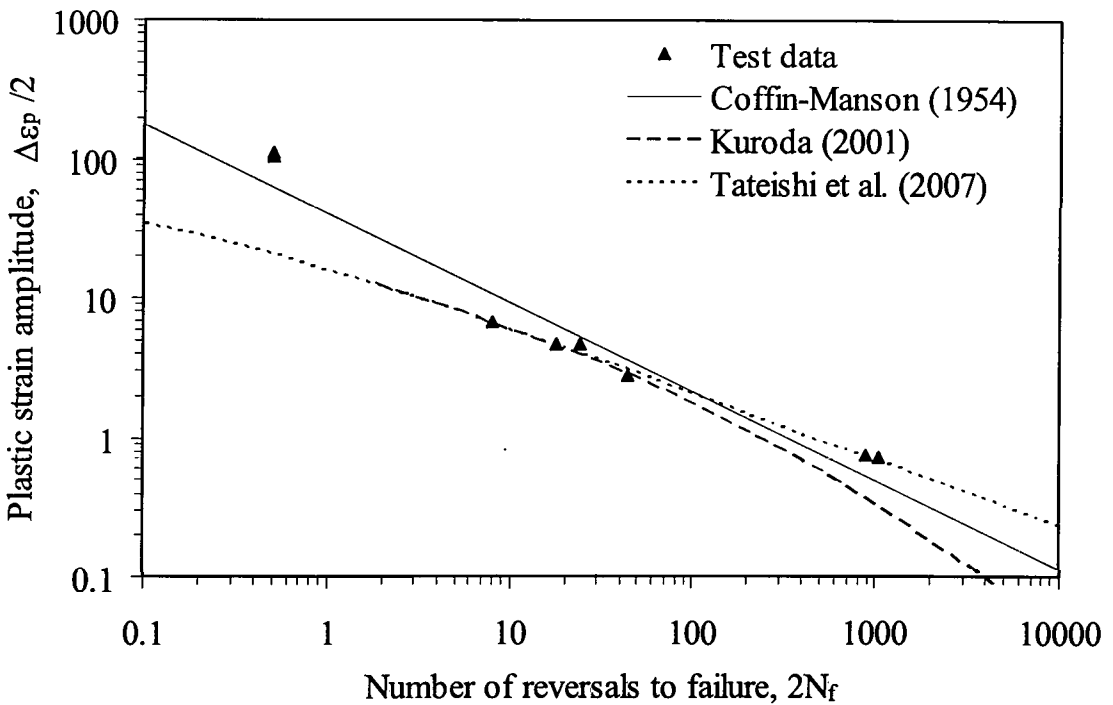


Figure 3.31: Comparison between strain-life models and cold-formed carbon steel data

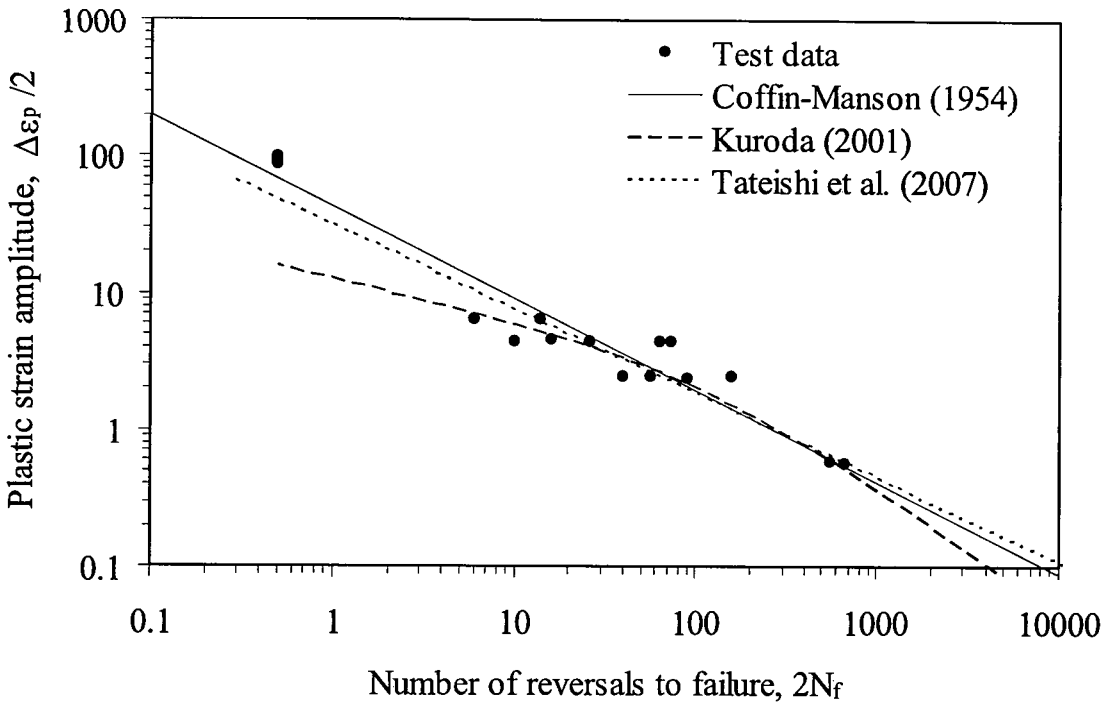


Figure 3.32: Comparison between strain-life models and cold-formed stainless steel data

3.3.4 Calibration of a cyclic material hardening model

Although Equation 3.4 provides an overview of how a material behaves under cyclic loading at different stress or strain amplitudes, it only characterises the stabilised hysteretic behaviour. If the behaviour before reaching the stabilised state is required, such as when specifying the material model in a numerical simulation, a constitutive model of cyclic plasticity is needed. The theory of cyclic plasticity provides a mathematical description of stress and strain in plastically deformed solids under cyclic loading. The current study focuses on the constitutive model for cyclic plasticity of metals employed in the non-linear finite element software ABAQUS (2007), which is based on the work by Chaboche (1986). In this model, the von Mises (1913) yield criterion and an associative flow rule are assumed. The characteristics of a material are defined by a combined isotropic/kinematic hardening model.

The model consists of a non-linear kinematic hardening component and an isotropic hardening component. The isotropic component defines the change of the size of the yield surface σ^0 as a function of equivalent plastic strain ϵ^p , and is given by:

$$\sigma^0 = \sigma|_0 + Q_\infty (1 - e^{-b_{iso}\epsilon^p}) \quad (3.8)$$

where $\sigma|_0$ is the yield stress at zero equivalent plastic strain (defined in this study as the 0.01% proof stress), Q_∞ is the maximum change in the size of the yield surface and b_{iso} is the rate at which the size of the yield surface changes as plastic strain increases. The size of the yield surface in the i^{th} cycle, σ_i^0 , can be obtained from:

$$\sigma_i^0 = \frac{\sigma_i^t - \sigma_i^c}{2} \quad (3.9)$$

where σ_i^t is the maximum tensile stress and σ_i^c is the maximum compressive stress in the elastic range as shown in Figure 3.33.

The equivalent plastic strain corresponding to σ_i^0 is:

$$\epsilon_i^p = \frac{1}{2}(4i - 3)\Delta\epsilon_p \quad (3.10)$$

where $\Delta\epsilon_p$ is the plastic strain range and can be approximated as:

$$\Delta\epsilon_p = \epsilon_{p,t} - \epsilon_{p,c} \approx \Delta\epsilon - 2\sigma_1^t/E \quad (3.11)$$

where $\epsilon_{p,t}$ is the maximum tensile plastic strain and $\epsilon_{p,c}$ is the maximum compressive plastic strain Equation 3.8 has been fitted through each data pair $(\sigma_i^0, \epsilon_i^p)$, including the value $\sigma|_0$ at zero equivalent plastic strain.

The kinematic component of the model defines the change of backstress α , which is expressed as:

$$\alpha = \frac{C_{kin}}{\gamma}(1 - e^{-\gamma\epsilon^p}) + \alpha_1 e^{-\gamma\epsilon^p} \quad (3.12)$$

where C_{kin} and γ are constants that can be calibrated by test data from a stabilised cycle. The ratio C_{kin}/γ is the maximum change in backstress and γ determines the rate at which the backstress varies as the plastic strain increases. Data pairs (σ_i, ϵ_i^p) are shifted as follows

$$\epsilon_i^p = \epsilon_i - \frac{\sigma_i}{E} - \epsilon_p^0 \quad (3.13)$$

such that $\varepsilon_1^p = 0$, where ε_p^0 is the value of plastic strain when the curve crosses the strain axis. For each data pair $(\sigma_i, \varepsilon_i^p)$, values of α_i are obtained as

$$\alpha_i = \sigma_i - \sigma^s \quad (3.14)$$

where σ^s is the average of the first and the last data points, σ_1 and σ_n , respectively, in a stabilised cycle, which are shown in Figure 3.34. Equation 3.12 can then be fitted to the pairs of data points $(\alpha_i, \varepsilon_i^p)$.

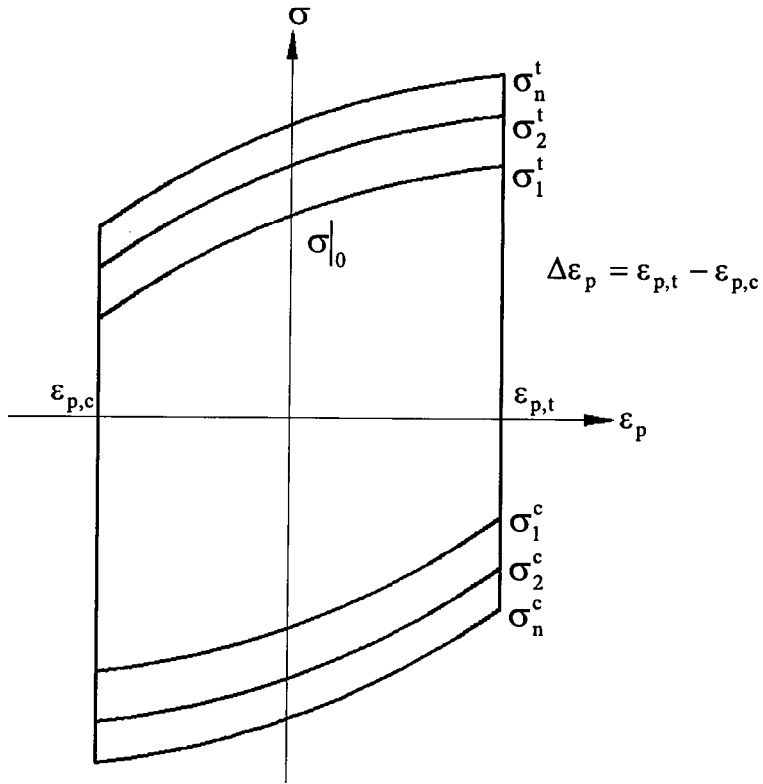


Figure 3.33: Calibration of isotropic hardening component

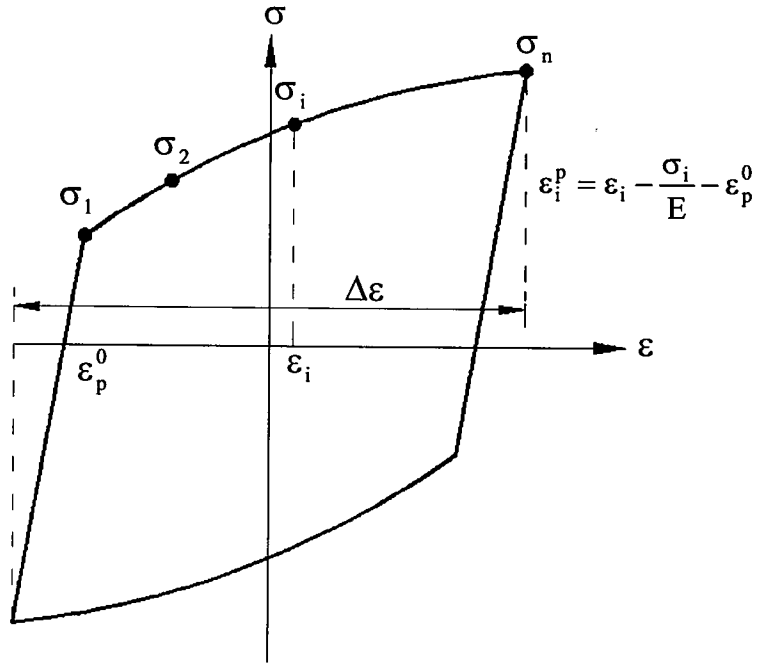


Figure 3.34: Calibration of kinematic hardening component

The method used for fitting Equations 3.8 and 3.12 requires careful consideration. The commonly used least squares regression fits a curve to data points by minimising the sum of the squares of residuals of the dependent variables (i.e. only the vertical distance between data points and the fitted curve is minimised). The general form of the curves resulting from Equations 3.8 and 3.12 comprises an initial steep slope followed by a rounded portion leading onto a more gently sloped region. Any changes in the shape of the curve in this steeply sloped section influence the residuals more significantly than in the flatter region. Effectively, less weighting is given to the flatter region of the curve in the fitting process. Furthermore, if the data points are not evenly spaced along the x-axis, more weighting will be given to the part of the curve where data points are concentrated. Therefore, in order to generate an accurate and representative stress-strain hysteresis curve, the total least squares regression method which can minimise errors on both axes, has been adopted.

The objective of total least squares regression is to minimise the normal distance from the data points to the curve. This method remains unbiased towards any axis if the quantities are measured in the same units or are normalised. A variance of this method is to minimise the area between the data points and the curve instead of the distance

between them. This method is units-invariant and is not biased towards any variables but fits the vertical and horizontal residuals with equal weighting. Also, results do not depend on the distribution of the data points. Figure 3.35 shows a typical set of data points ($\sigma_i^0, \varepsilon_i^p$) and the corresponding best fit curve, obtained on the basis of area minimisation.

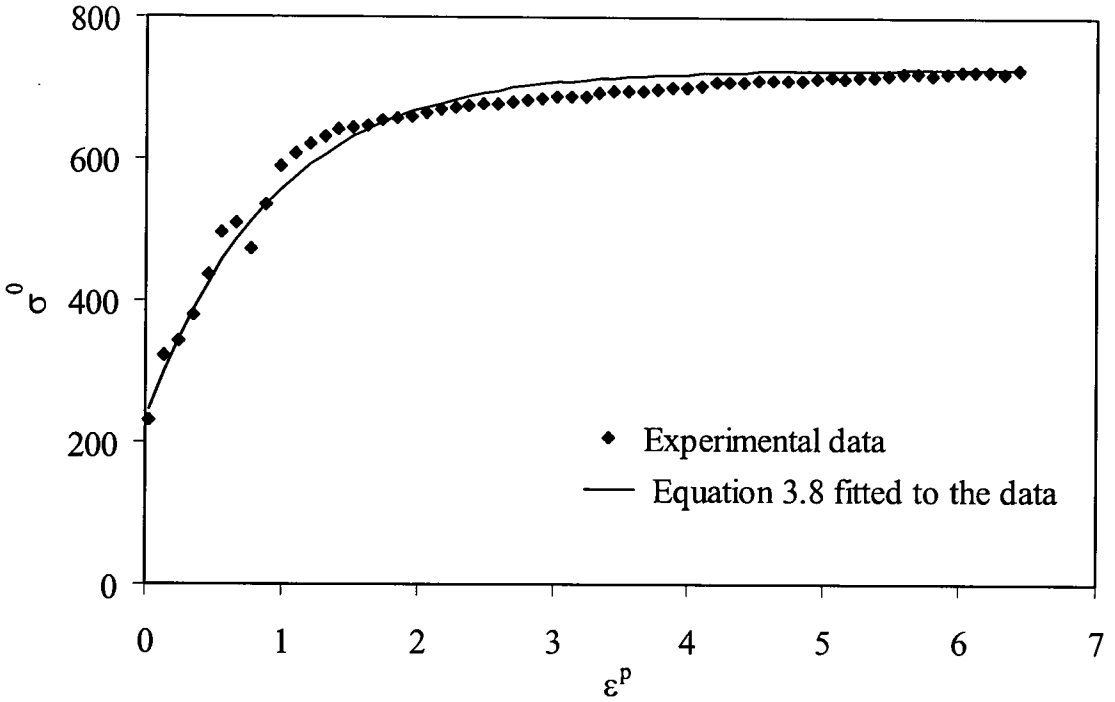


Figure 3.35: Best fit curve of Equation 3.8 to experimental data

The values of the parameters calculated for each test are shown in Table 3.4. The results reveal that stainless steel exhibits a larger degree of cyclic hardening than carbon steel as indicated by the larger values of Q_∞ . The stress-strain curve of stainless steel deviates from linearity at lower strains than carbon steel and maintains a higher post-yield modulus; this is reflected in higher values of the ratio C_{kin}/γ for stainless steel than for the two carbon steels. Although there is a degree of variability of some of the individual material parameters, it should be noted that the shape of a complete stress-strain hysteresis curves depends on the combination of the isotropic and kinematic components. A more representative assessment of the variability of the parameters is to consider the maximum change in size of a hysteresis loop, which is given by $2(\sigma|_0 + Q_\infty) + C_{\text{kin}}/\gamma$. These values are given in the last column of Table 3.4 where it can be seen that the coefficients of variance are relatively low. Experimental and FE simulated stress-strain hysteresis curves of cold-formed carbon steel and cold-

formed stainless steel in the first 2 cycles at $\pm 1\%$ strain amplitude tests are shown in Figures 3.36 and 3.37, respectively, indicating that the cyclic hardening model is capable of capturing the key observed behavioural features.

Table 3.4: *Parameters of non-linear combined isotropic/kinematic hardening model of cyclic plasticity*

Specimen	Total strain amplitude, $\Delta\varepsilon/2$ (%)	$\sigma _0$ (N/mm ²)	Q_∞ (N/mm ²)	b_{iso}	C_{kin} (N/mm ²)	γ	C_{kin}/γ (N/mm ²)	$2(\sigma _0 + Q_\infty) + C_{kin}/\gamma$
40×40×4-CS-HR-1%	1	448	33	0.17	29700	229	130	1092
40×40×3-CS-HR-1%	1	478	87	1.50	30200	188	161	1290
40×40×3-CS-HR-3%	3	478	97	1.53	14300	120	119	1269
40×40×4-CS-HR-5%	5	448	30	6.63	11600	68	171	1126
40×40×3-CS-HR-5%	5	478	51	3.21	4278	40	107	1166
40×40×4-CS-HR-7%	7	448	46	3.46	16800	129	130	1119
40×40×3-CS-HR-7%	7	478	38	0.18	58000	202	287	1319
Mean CS-HR		465	55	2.38	23554	139	169	1197
COV CS-HR		0.03	0.45	0.89	0.70	0.47	0.36	0.07
40×40×4-CS-CF-1%	1	229	5.08	0.131	221000	798	277	745
40×40×3-CS-CF-1%	1	353	32.9	0.133	87100	394	221	993
40×40×4-CS-CF-3%	3	229	34.8	0.900	127000	413	308	835
40×40×4-CS-CF-5%	5	229	43.4	0.707	202000	635	318	863
40×40×3-CS-CF-5%	5	353	37.7	0.289	47000	150	313	1095
40×40×4-CS-CF-7%	7	229	58	4.23	90200	318	284	858
40×40×3-CS-CF-7%	7	353	35.9	1.47	82500	303	272	1050
Mean CS-CF		282	35	1.12	122400	430	285	920
COV CS-CF		0.22	0.41	1.20	0.49	0.47	0.11	0.13

Table 3.4 (continued): Parameters of non-linear combined isotropic/kinematic hardening model of cyclic plasticity

Specimen	Total strain amplitude, $\Delta\varepsilon/2$ (%)	σ_0 (N/mm ²)	Q_∞ (N/mm ²)	b_{iso}	C_{kin} (N/mm ²)	γ	C_{kin}/γ (N/mm ²)	$2(\sigma_0 + Q_\infty) + C_{kin}/\gamma$
60×60×3-SS-CF-1%	1	232	544	0.155	96900	283	342	1894
50×50×3-SS-CF-1%	1	217	319	0.170	254000	558	455	1527
100×100×3-SS-CF-3%	3	215	648	1.16	59200	156	379	2105
60×60×3-SS-CF-3%	3	232	495	1.08	73900	181	408	1862
50×50×3-SS-CF-3%	3	217	529	1.21	127000	228	557	2049
60×40×3-SS-CF-3%	3	283	395	1.21	70600	177	399	1755
100×100×3-SS-CF-5%	5	215	422	2.33	69700	146	477	1751
60×60×3-SS-CF-5%	5	232	550	1.92	121000	236	513	2077
50×50×3-SS-CF-5%	5	217	410	1.50	95000	187	508	1762
60×40×3-SS-CF-5% (a)	5	283	428	1.51	66000	157	420	1842
60×40×3-SS-CF-5% (b)	5	283	282	3.56	90900	172	528	1658
60×60×3-SS-CF-7%	7	232	470	2.71	152000	262	580	1984
50×50×3-SS-CF-7%	7	217	717	0.815	99200	169	587	2455
Mean SS-CF		237	478	1.49	105800	224	472	1902
COV SS-CF		0.11	0.25	0.62	0.47	0.47	0.16	0.12

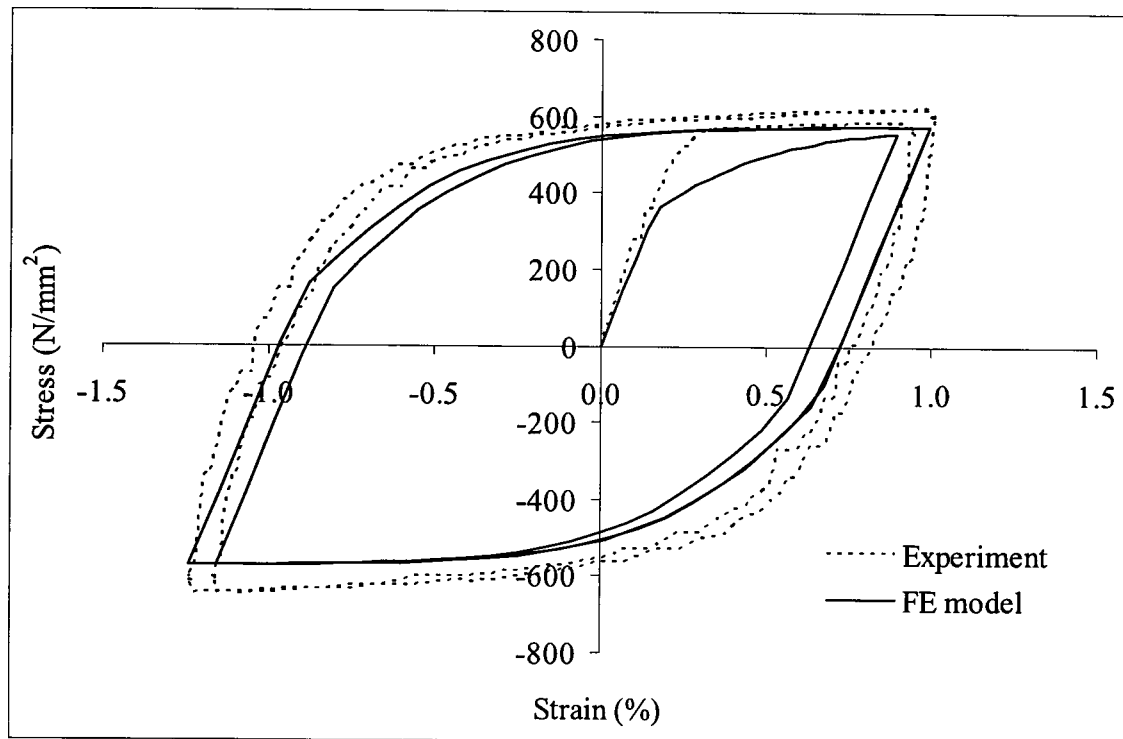


Figure 3.36: Experimental and FE simulated stress-strain hysteresis curves of a cold-formed carbon steel specimen in the first 2 cycles of $\pm 1\%$ strain amplitude tests

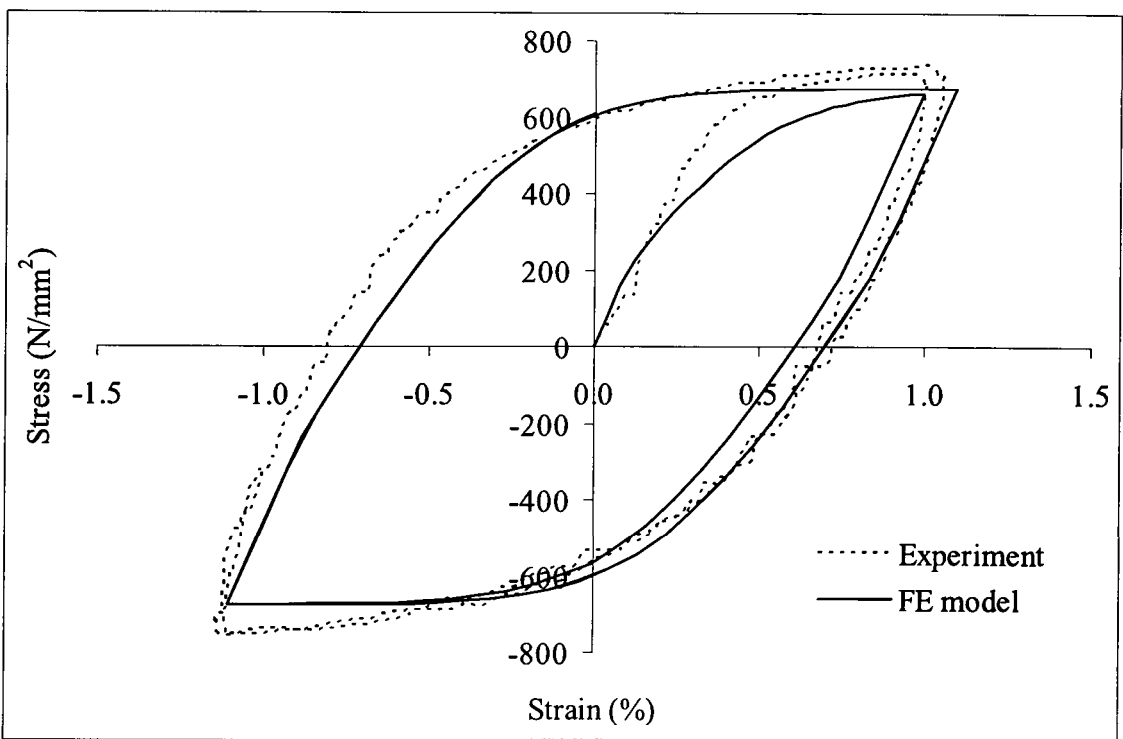


Figure 3.37: Experimental and FE simulated stress strain hysteresis curves of a cold-formed stainless steel specimen in the first 2 cycles of $\pm 1\%$ strain amplitude tests

3.4 *Bending test results*

The same materials that were examined in the axial arrangement (Section 3.3) have also been tested in the bending configuration, to enable higher strain amplitudes, achievable in the absence of instabilities, to be explored. In the bending specimens, the strain distribution is non-linear through the thickness, thus failure is likely to initiate at the specimen surface and develop inwards. The bending test data and surface strain-life relationships are shown in Figure 3.38. The results display less scatter than those from the axial tests – this may reflect the benefits of the bending configuration which essentially avoids any issues of buckling or slippage. Similar to the results from the axial tests, the three materials follow a similar trend but the hot-rolled carbon steel generally displays a superior fatigue life for a given strain range.

As for the axial specimens, the fracture surfaces of each of the bending specimens have also been examined to categorise the failure into ductile, mixed and fatigue modes. Specimens that failed in less than 20 reversals generally showed a ductile failure mode, while those that failed after more than 300 reversals exhibited features of fatigue fracture. The progressive change from a ductile failure mode at $\pm 15\%$ strain amplitude to a mixed mode at $\pm 10\%$ and $\pm 6\%$ strain amplitudes and finally to a fatigue failure mode at $\pm 2\%$ strain amplitude is depicted in Figures 3.39-3.42. The observed failure modes from the bending tests are similar to those from the axial tests, other than the region of final failure, which consistently appears towards the mid-thickness of the specimens owing to the imposed strain distribution. Despite the different failure modes, the strain-life data from the bending tests is well represented by a simple power-law, indicated by the straight lines in the log-log plot of Figure 3.38.

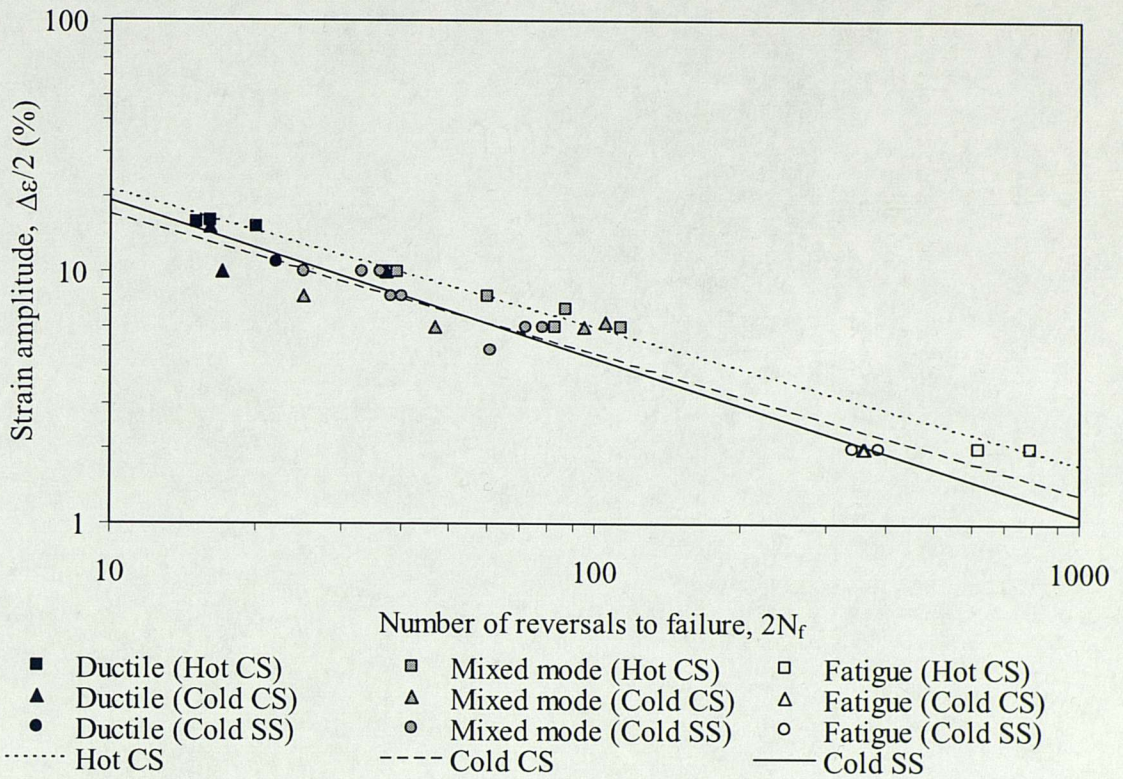


Figure 3.38: Strain-life relationships and fracture modes of bending test specimens

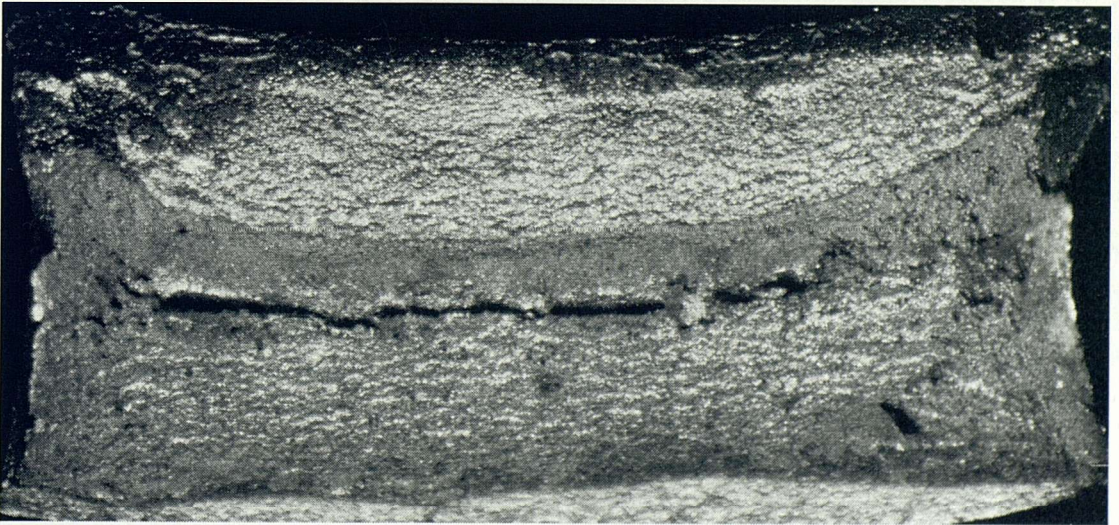


Figure 3.39: Fracture surfaces of a bending specimen tested at $\pm 15\%$

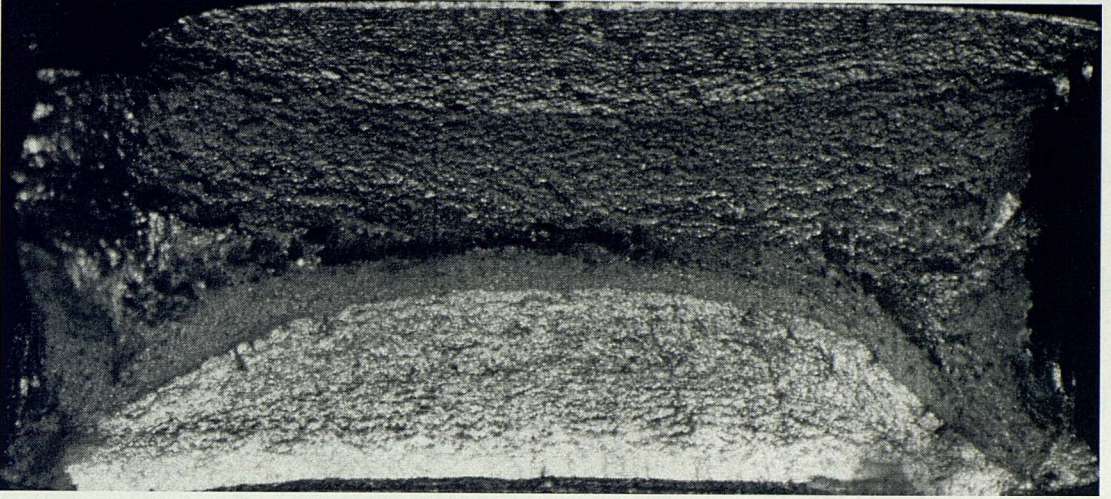


Figure 3.40: *Fracture surfaces of a bending specimen tested at $\pm 10\%$*

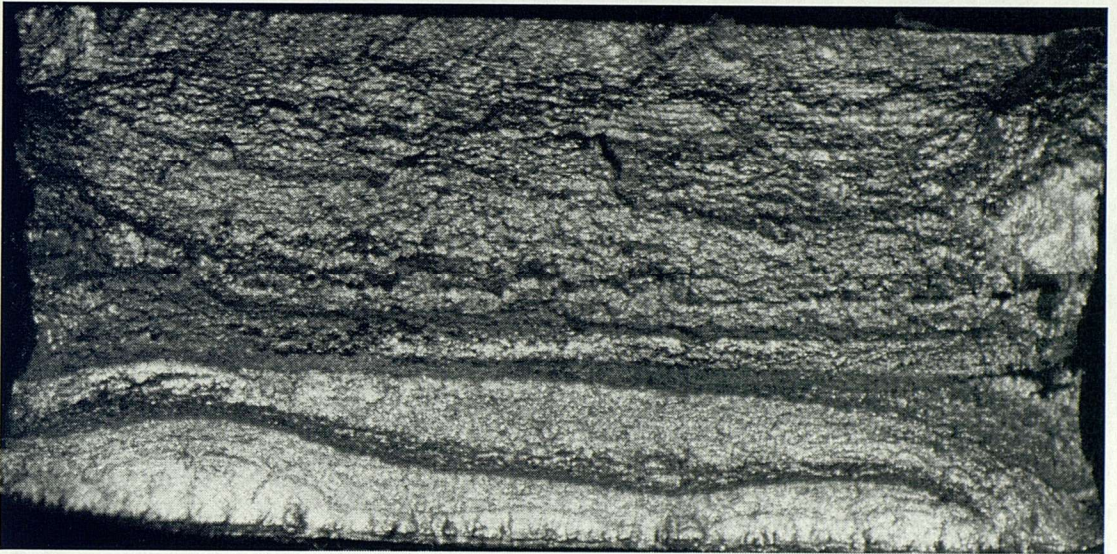


Figure 3.41: *Fracture surfaces of a bending specimen tested at $\pm 6\%$*

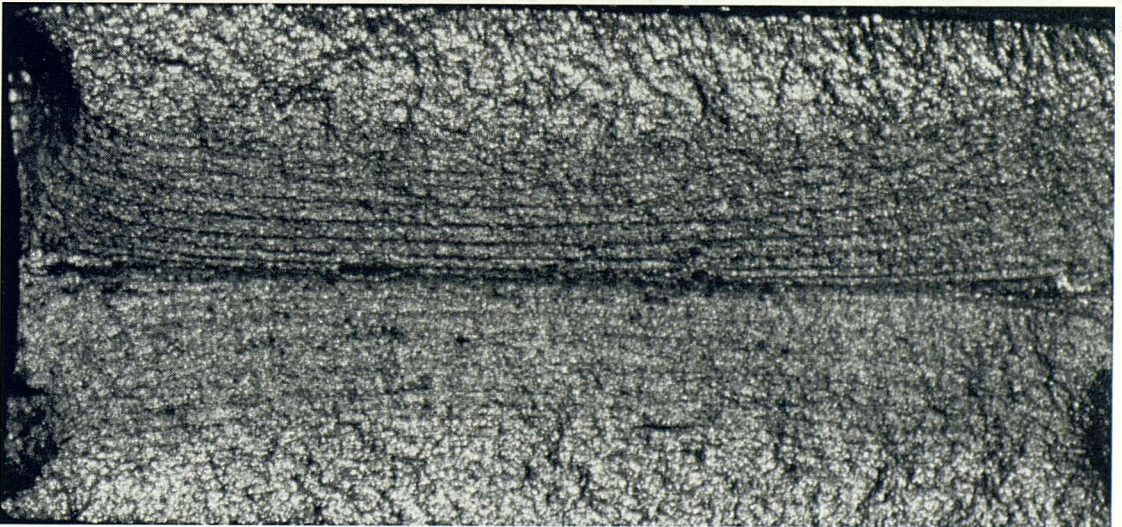


Figure 3.42: *Fracture surfaces of a bending specimen tested at $\pm 2\%$*

A drawback to the bending configuration is that the results cannot be directly used for obtaining the material parameters in Equation 3.1 since the strain amplitude is not uniform through the thickness. For a given surface strain, the number of reversals to failure in a bending test is higher than in an axial test. A method to correlate results from these two testing configurations has been developed (Esin 1968, 1980) and is explored further herein.

Esin (1968) related fatigue damage to plastic hysteresis energy and considered the probability of localised plastic flow in a nominally elastic material. On this basis, Esin (1980) proposed a method to correlate fatigue data from axial, bending and rotating bending tests. The approach was shown to be applicable to elastic strain dominant high cycle fatigue. Esin (1980) postulated that material in some parts of a cross-section under smaller stress amplitudes, such as the material towards the mid-thickness of a bending specimen, would not contribute to the fatigue process. Esin's (1980) approach, adapted to the strain-based scenario of the current study, is introduced below.

Assuming that fatigue fracture occurs when the plastic hysteresis energy reaches the true tensile toughness, which is measured by the area under the true stress-true strain curve, the number of cycles to failure N_f of a volume V_t of material with a true tensile toughness u_f in a cross-section of unit depth can be expressed as:

$$N_f = \frac{V_t u_f}{\Delta W} \quad (3.15)$$

where ΔW is the plastic hysteresis energy per cycle of the material. According to Esin's (1980) proposal, a small element in a cross-section has a certain probability to deviate from the theoretically calculated nominal values of stress and strain. Therefore, ΔW can be expressed as

$$\Delta W = \sum V_i u_i \quad (3.16)$$

where u_i is the plastic hysteresis energy of a small element loaded at a certain strain amplitude and V_i is the volume of that small element with strain amplitudes large enough to contribute to the fatigue process. Let $\Delta \epsilon_{hyp}/2$ be a hypothetical uniform strain amplitude across the section that gives the same plastic hysteresis energy as using the actual strain amplitude at each point. This leads to:

$$\frac{\Delta \varepsilon_{\text{hyp}}}{2} = \frac{\sum \frac{\Delta \varepsilon_i}{2} A_i}{\sum A_i} \quad (3.17)$$

where A_i is the cross-sectional area of a small element at a constant strain amplitude $\Delta \varepsilon_i/2$. Under uniform axial loading, the plastic strain energy dissipated by all elements is equal, and by utilising the concept of equivalent strain amplitude, the number of cycles to failure, from Equation 3.15, becomes:

$$N_f = \frac{V_t u_f}{u_i \sum V_i} = \left(\frac{u_f}{u_i} \right) \left(\frac{V_t}{\sum V_i} \right) \quad (3.18)$$

It should be noted that in bending fatigue tests, $V_t \neq \sum V_i$ because elements at lower stress amplitudes do not contribute to the fatigue process; the ratio of $V_t/\sum V_i$ can be determined experimentally. In the current study, the strain amplitudes applied in the bending tests are much higher than those studied by Esin (1980), and it has been assumed that the volume of material not involved in the fatigue process is negligible. Therefore, the ratio $V_t/\sum V_i$ has been taken as unity.

In order to apply the above approach, the total energy to fracture from the axial and bending tests has been calculated. The energy to fracture per unit volume of material in the axial tests was obtained by numerically integrating the areas under the stress-strain hysteresis loops, while in the bending tests the same was calculated by integrating the areas under the moment-curvature hysteresis loops divided by the width and thickness of the specimen. The moment was derived from the load measurements and curvature was calculated from the readings of strain gauges and an assumed length of constant curvature of the specimen.

The total energy to fracture for the test specimens from the axial and bending configurations has been plotted for the three materials – hot-rolled carbon steel, cold-formed carbon steel and cold-formed stainless steel – in Figures 3.43-3.45. It may be seen that the trends of the test data are largely independent of the test configuration, indicating that consideration of total energy is a suitable means of predicting fracture, regardless of the applied strain distribution. Following Esin's approach, and considering the rectangular cross-section shown in Figure 3.46 subjected to a linearly

varying bending strain distribution, the hypothetical uniform strain (Equation 3.18) becomes:

$$\frac{\Delta\epsilon_{\text{hyp}}}{2} = \frac{\int_{h-x}^h \frac{\Delta\epsilon_s}{2} \frac{y}{h} w dy}{wx} = \frac{\Delta\epsilon_s}{2} \left(1 - \frac{x}{2h}\right) \quad (3.19)$$

where the symbols are defined in Figure 3.46. Assuming all material in the cross-section is subjected to fatigue (i.e. $x = h$), Equation 3.19 can be simplified to:

$$\frac{\Delta\epsilon_{\text{hyp}}}{2} = 0.5 \left(\frac{\Delta\epsilon_s}{2} \right) \quad (3.20)$$

This implies that the bending surface-strain life relationship can be determined simply by multiplying the strain amplitudes of axial test data by a factor of two. On this basis, the (surface) strain-life test data from all tests have been plotted in Figures 3.47-3.49; the dashed lines represent the prediction of the bending test data (through Equation 3.20) from the axial test data – good agreement may be observed. This finding suggests that calibration of the parameters of the strain-life relationship (Equation 3.1) may be performed on the basis of data obtained from either axial or bending tests.

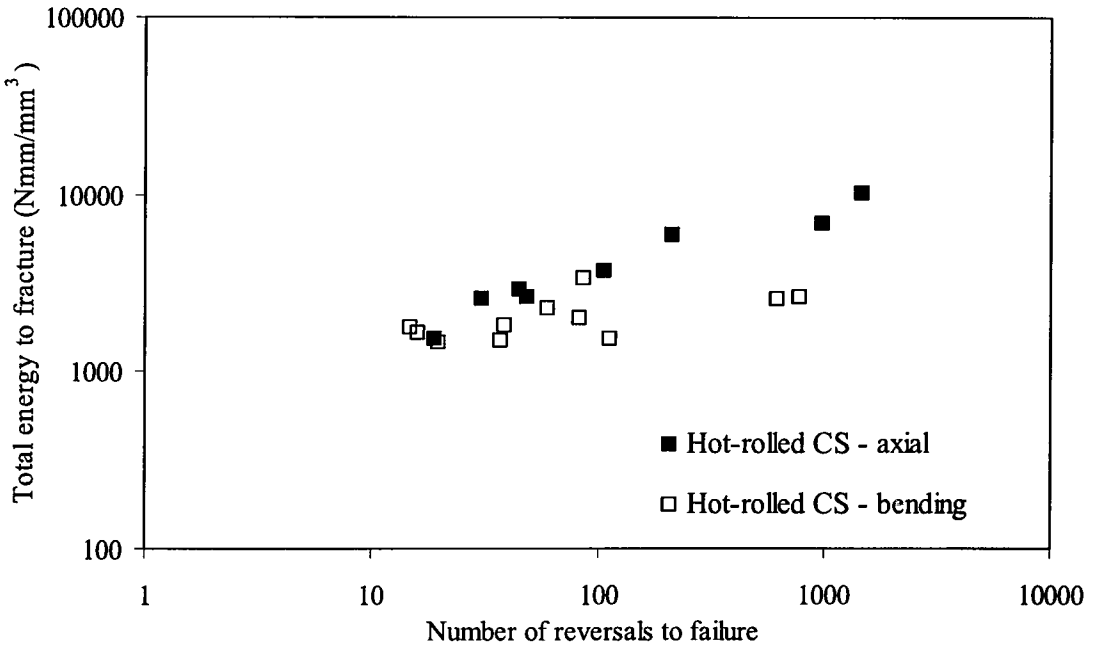


Figure 3.43: Relationship between total energy to fracture and number of reversals to failure of hot-rolled carbon steel specimens

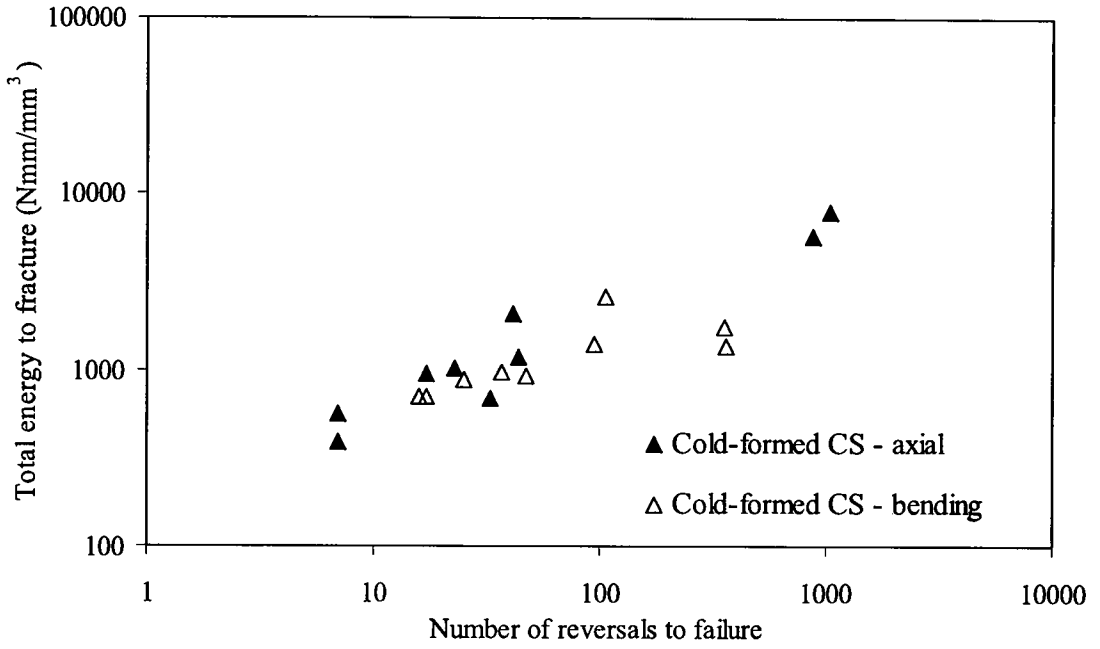


Figure 3.44: Relationship between total energy to fracture and number of reversals to failure of cold-formed carbon steel specimens

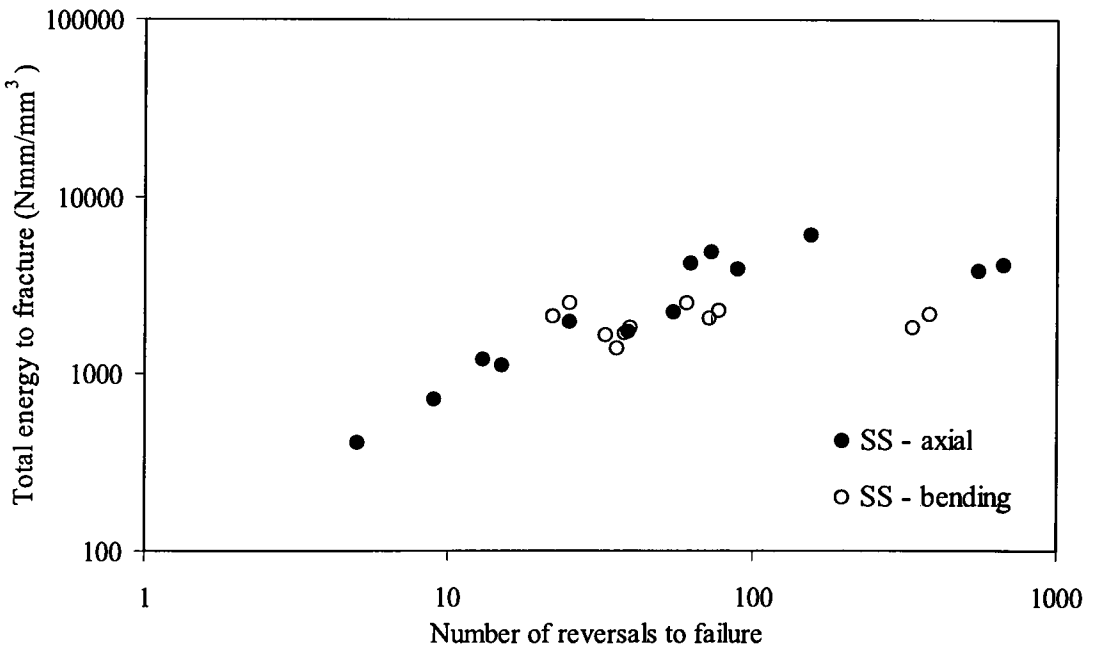


Figure 3.45: Relationship between total energy to fracture and number of reversals to failure of cold-formed stainless steel specimens

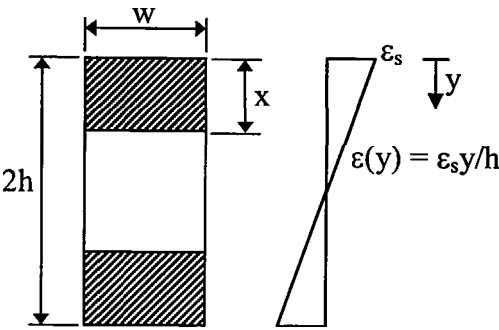


Figure 3.46: *Distribution of strain through a rectangular cross-section subjected to bending*

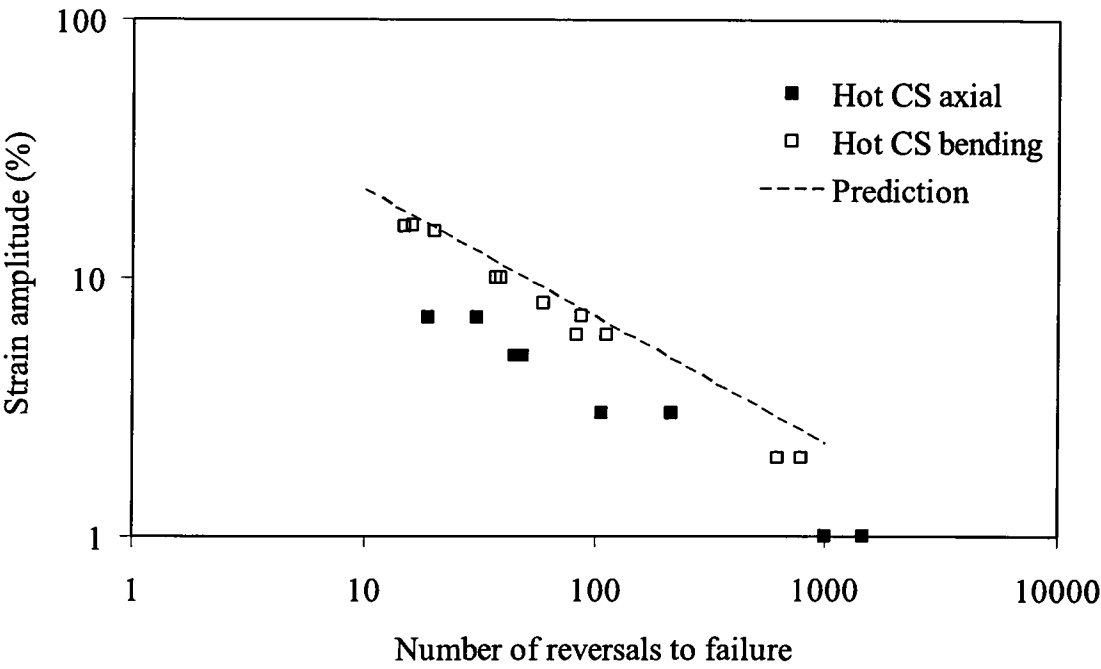


Figure 3.47: *Prediction of strain-life relationship of bending tests from axial test data of hot-rolled carbon steel specimens*

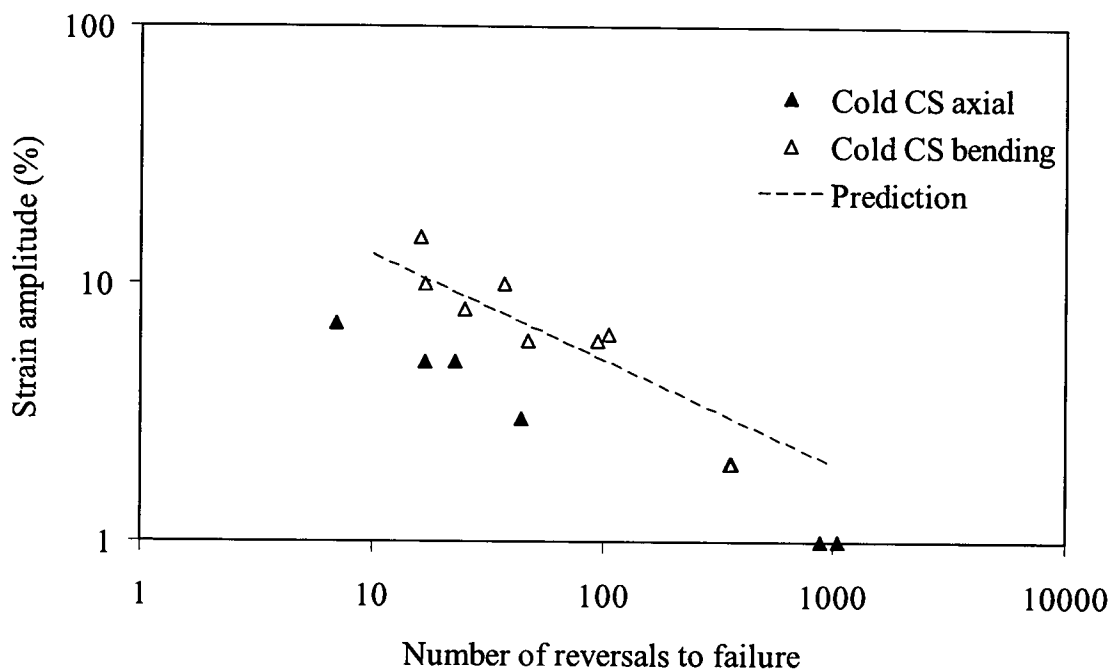


Figure 3.48: *Prediction of strain-life relationship of bending tests from axial test data of cold-formed carbon steel specimens*

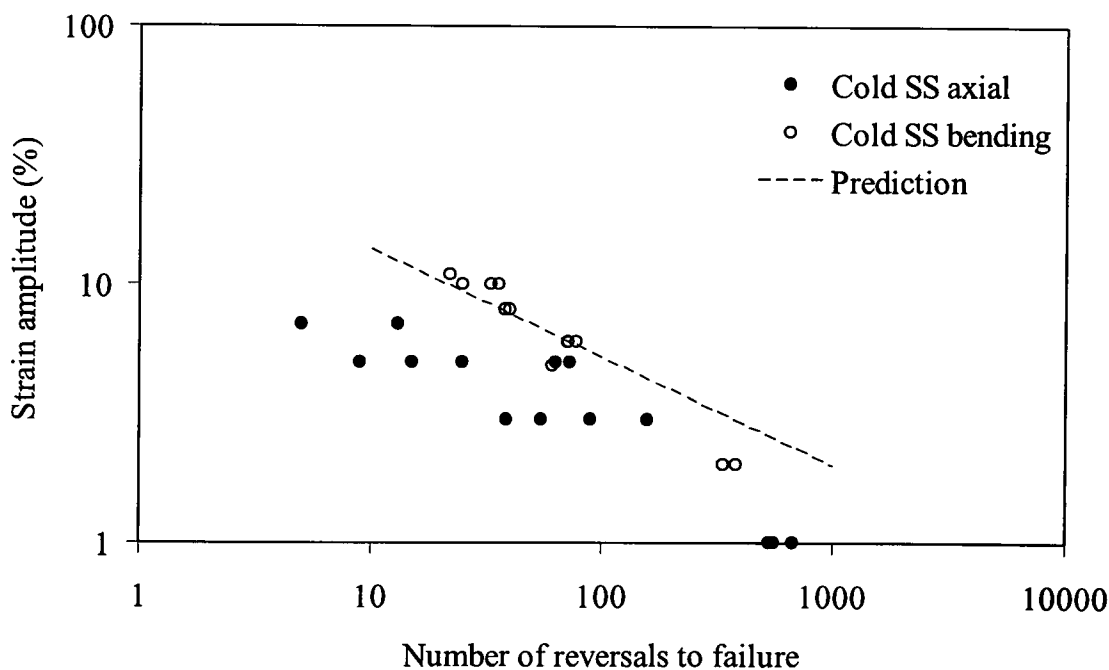


Figure 3.49: *Prediction of strain-life relationship of bending tests from axial test data of cold-formed stainless steel specimens*

3.5 *Concluding remarks*

Cyclic material tests have been carried out to study the ELCF and LCF properties of material from hot-rolled carbon steel (S355J2H), cold-formed carbon steel (S235JRH) and cold-formed stainless steel (EN 1.4301 and 1.4307) structural sections. A total of 62 tests were carried out: 31 constant amplitude axial tests were conducted at strain amplitudes ranging from $\pm 1\%$ to $\pm 7\%$, and an additional 31 corresponding tests at surface strain amplitudes ranging from $\pm 2\%$ to $\pm 15\%$ were performed in a bending configuration. The test results revealed that the three materials exhibit similar strain-life relationships despite significantly different elongations at fracture measured in monotonic tensile tests. The hot-rolled (annealed) material generally displayed marginally better fatigue performance than the cold-formed material, attributed to the increased level of inclusions and reduced ductility caused by cold-work. Stainless steel, which displays substantial strain hardening in monotonic tensile tests, also demonstrated a much larger degree of cyclic hardening than the two carbon steel materials. Material parameters for strain-life relationships and cyclic stress-strain curves for the three materials were obtained from the test results.

In order to model stress-strain hysteresis behaviour prior to the attainment of a stabilised state, the non-linear combined isotropic/kinematic hardening model in ABAQUS (2007) has been calibrated on the basis of total least squares fits to the generated test data. However, ABAQUS only allows one set of material parameters to be used for each element, which does not address the issue of different hysteretic response at various strain amplitudes. In the current study, the strain amplitudes experienced by the elements in the numerical models described in the following chapters change throughout the analysis. Therefore, the mean values of the material parameters derived from material tests at each strain amplitude are applied in the material model. Also, due to the limited number of parameters in the cyclic material hardening model, the hysteretic behaviour cannot always be simulated accurately at maximum and minimum strains. However, although there is some variability in the individual model parameters between different tests, the FE simulated hysteresis loops generated by the combined isotropic/kinematic hardening model (on the basis of average parameters for each material) are able to capture the key observed behavioural features. The material model can be improved if the material behaviour can be defined

by sets of material parameters at more than one strain amplitude and more parameters are used to define the changes in the backstress.

Bending tests were carried out without the concerns of slippage and buckling that are possible in axially loaded tests; the scatter of the generated test data was lower in the bending configuration. A method to correlate axial and bending test results through an energy criterion has been established, demonstrating that the strain-life relationship of Equation 3.1 can be derived from testing in either configuration. It is preferable to conduct high strain amplitude tests in bending configurations because they avoid the issue of buckling and provide better grip for the specimens. However, axial test results are more readily used for calibrating cyclic material models due to the uniform strain distribution through the thickness of the specimens.

Chapter 4

Member Testing

4.1 Introduction

This chapter presents a series of cyclic axial tests on carbon steel and stainless steel hollow section members which have been carried out up to complete fracture, in order to simulate the behaviour of bracing members in concentrically braced frames. The cyclic behaviour of the bracing members in terms of hysteretic loops, displacement ductility and energy dissipation were evaluated. The results of these tests were used to verify the finite element models described in Chapter 5.

4.2 Experimental programme

The primary objective of the experimental programme was to investigate the cyclic behaviour of bracing members in concentrically braced frames by means of cyclic axial tests. It involved 16 square and rectangular hollow members of various lengths and section sizes in the three materials, some of which were tested under static loading in compression and bending as part of a comparative study between hot-rolled and cold-formed steel tubular members (Gardner et al., submitted). The measured geometric properties of the tested sections are given in Table 4.1, for which the associated symbols are defined in Figure 3.1.

Table 4.1: Measured geometry of test specimens

Specimen identification	Depth, D (mm)	Width, B (mm)	Thickness, t (mm)	Internal corner radius, r_i (mm)
60×60×3×2050-CS-HR	60.42	60.26	3.25	2.75
40×40×3×2050-CS-HR	40.12	39.92	2.98	1.75
40×40×3×1250-CS-HR	40.12	39.92	2.98	1.75
60×60×3×2050-CS-CF	59.87	59.75	2.76	4.00
40×40×4×2050-CS-CF	40.42	40.32	3.75	3.50
40×40×3×2050-CS-CF	40.13	40.01	2.79	3.00
40×40×3×1250-CS-CF	40.13	40.01	2.79	3.00
60×60×3×2850-SS-CF	60.62	60.27	2.79	2.75
50×50×3×2850-SS-CF	50.19	50.03	2.76	1.00
60×40×3×2850-SS-CF	60.13	39.98	2.75	1.50
60×60×3×2050-SS-CF	60.62	60.27	2.79	2.75
50×50×3×2050-SS-CF	50.19	50.07	2.76	1.00
60×40×3×2050-SS-CF	60.13	39.98	2.75	1.50
60×60×3×1250-SS-CF	60.62	60.27	2.79	2.75
50×50×3×1250-SS-CF	50.19	50.03	2.76	1.00
60×40×3×1250-SS-CF	60.13	39.98	2.75	1.50

Codes typically impose an upper limit on the global slenderness $\bar{\lambda}$ of bracing members in concentrically braced frames which is specified as 2.0 in EC8 (EN 1998-1, 2004).

The global slenderness $\bar{\lambda}$ is defined as

$$\bar{\lambda} = \sqrt{f_y A / N_{cr}} \quad (4.1)$$

where f_y is the material yield strength, A is the cross-sectional area, and N_{cr} is the elastic critical buckling load. If high dissipative structural behaviour is assumed in the design, cross-sections of bracing members are required to be Class 1. The local slenderness of a cross-section, upon which the cross-section class is determined, is defined as $b/t\epsilon$ where b is the flat width of the wider face of the section, t is the thickness of the section and $\epsilon = \sqrt{235/f_y}$. In the current experimental and parametric studies, all specimens conform to the $\bar{\lambda} < 2.0$ requirement. All cross-sections are deemed as Class 1 according to EC3 Part 1-3 (EN 1993-1-1, 2005) and the new slenderness limits recently proposed for stainless steel sections by Gardner and Theofanous (2008), though the most slender stainless steel sections in the experimental and parametric studies, respectively, are Class 3 according to EC3 Part 1-4 (EN 1993-1-4, 2006). The revised slenderness limits for stainless steel sections (Gardner &

Theofanous, 2008) are based on a considerably larger pool of experimental data than was available at the time of preparation of EC3 Part 1-4 (EN 1993-1-4, 2006), and are thus considered more representative of observed physical behaviour.

Specimens were tested under cyclic axial loading in an internal reaction frame as shown in Figure 4.1. Displacements and rotations were restrained at both ends of the members, other than axial displacement at the loaded end, which was controlled by a 100 T Instron hydraulic actuator. A transducer was used to measure any displacement at the unloaded end of the member so that the flexibility of the test rig could be eliminated when the test data were processed. Strain gauges were placed at the centre of each flat face and at each corner at the mid-length of the specimens in order to measure the distribution of strain across the section during buckling. Two draw-wire transducers were attached to the specimens to measure the lateral deflection at mid-length in the vertical and horizontal directions (Figure 4.2). In order to prevent premature fracture at the welded end plates, a stiffener plate was inserted at each end of the specimens to provide an adequate length of weld between the specimen and the end plates. The end plates were bolted to the actuator and to the reaction frame. Details of the end plate connections are shown in Figure 4.3.

The applied axial displacement history followed the recommendations of ECCS (1986), i.e. one cycle at each level of 0.25, 0.5, 0.75 and 1.0 δ_y , followed by three cycles at each level of 2, 4, 6, 8 δ_y , etc., where δ_y is the estimated axial yield displacement. The yield displacement of each specimen was determined by the length of the member and the weighted yield strain of the cross-section. In the current study, tensile coupons of the materials at the flat faces and corner areas were tested. For the cold-formed carbon steel and stainless steel sections, the strain at the 0.2% proof stress was taken as the yield strain. The displacement was applied at a strain rate of $3 \times 10^{-4} \text{ s}^{-1}$ which is within the recommended range for quasi-static testing (BS 7270, 2006).

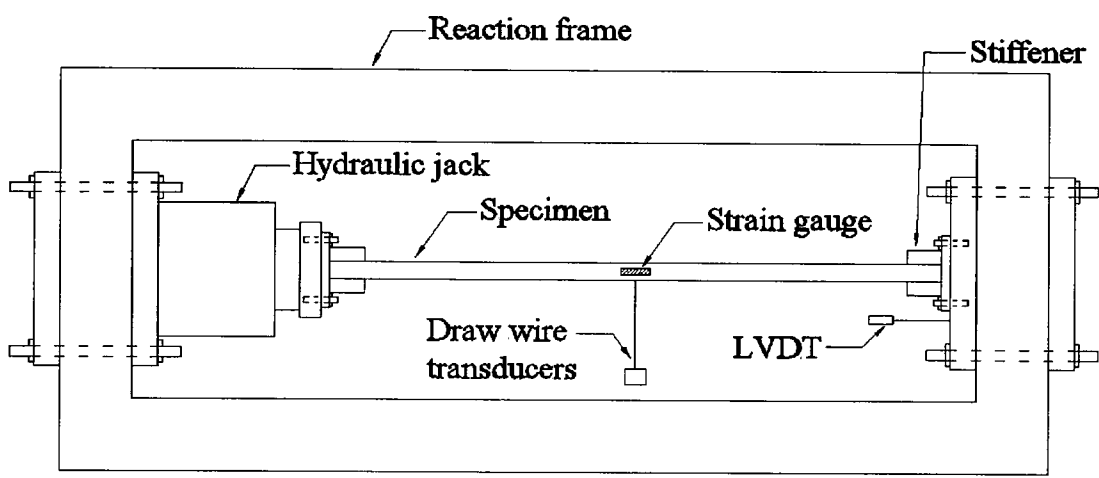


Figure 4.1: Schematic diagram of the test set-up (plan view)

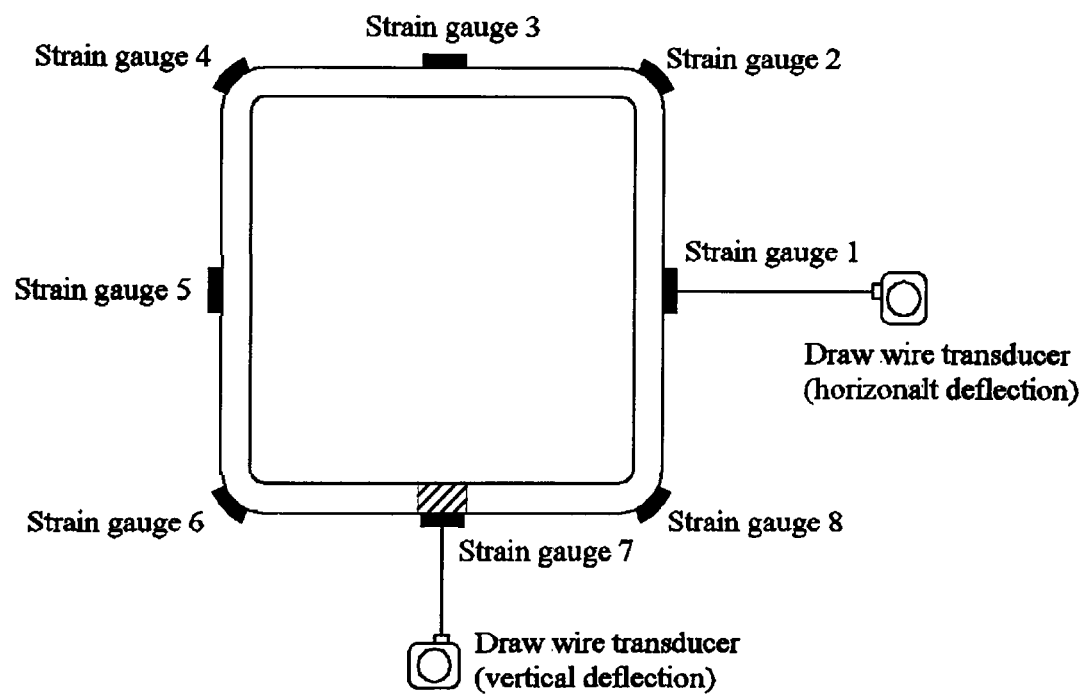


Figure 4.2: Instrumentation on the cross-section at mid-length of the members

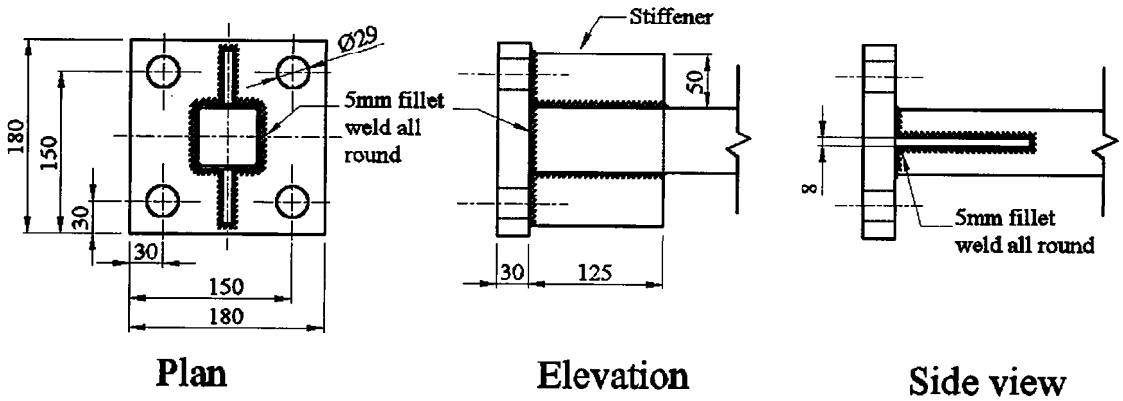


Figure 4.3: End connection details of test specimens (dimensions in mm)

4.3 Test results

A summary of the test specimens and results, including applied yield displacement, local and global slenderness, number of cycles before the occurrence of global buckling, local buckling and corner fracture, together with the displacement ductility, is given in Table 4.2. A more detailed description of test results is provided below.

4.3.1 Hysteretic response

Specimens typically underwent elastic deformation in the first few cycles. Global buckling was generally initiated when the applied displacement was in the proximity of the yield displacement. At this stage in the cyclic loading regime, the deformed shape of the specimens resembled that of elastic buckling (Figure 4.4). As expected, the stockier members buckled one or two cycles after the relatively slender specimens. Upon reloading in tension, the specimens were straightened but there always remained a small residual lateral deflection. The three RHS specimens exhibited uniaxial buckling about the weaker axis though the longer RHS specimens also showed small

lateral displacements about the stronger axis. All short specimens experienced uniaxial buckling though specimen 60×60×3×1250-SS-CF showed some lateral deflection in the second direction. Some degree of biaxial buckling occurred in all medium-length and long specimens. As the applied displacement increased, local buckling occurred at mid-length and, in some specimens, close to the end stiffeners. Plastic hinges formed at those locations, which resulted in the mechanism shown in Figure 4.5. More pronounced local deformation was observed at the mid-length of the member than at the member ends due to the larger rotation.

Table 4.2: Summary of member test results

Specimen identification	Yield displacement (mm)	b/te	$\bar{\lambda}$	Number of cycles to			Displacement ductility
				global buckling	local buckling	corner opening	
60×60×3×2050-CS-HR	4.31	20.6	0.57	5	7	10	4
40×40×3×2050-CS-HR	4.92	14.7	0.89	4	15	18	10
40×40×3×1250-CS-HR	3.00	14.7	0.50	5	8	11	6
60×60×3×2050-CS-CF	4.51	20.8	0.53	5	7	10	4
40×40×4×2050-CS-CF	4.51	9.1	0.89	4	-	11	6
40×40×3×2050-CS-CF	5.95	14.2	0.90	3	5	10	4
40×40×3×1250-CS-CF	3.63	14.2	0.50	4	8	10	4
60×60×3×2850-SS-CF	7.41	25.5	0.89	3	8	10	4
50×50×3×2850-SS-CF	8.27	23.7	1.16	3	12	14	8
60×40×3×2850-SS-CF	8.55	28.4	1.40	3	-	10	4
60×60×3×2050-SS-CF	5.33	25.5	0.62	5	8	9	4
50×50×3×2050-SS-CF	5.95	23.7	0.80	4	8	10	4
60×40×3×2050-SS-CF	6.15	28.4	0.97	4	8	11	6
60×60×3×1250-SS-CF	3.25	25.5	0.34	5	8	10	4
50×50×3×1250-SS-CF	3.63	23.7	0.45	5	8	10	4
60×40×3×1250-SS-CF	3.75	28.4	0.54	5	6	10	4



Figure 4.4: *Buckling of specimen $60 \times 60 \times 3 \times 2850$ -SS-CF*

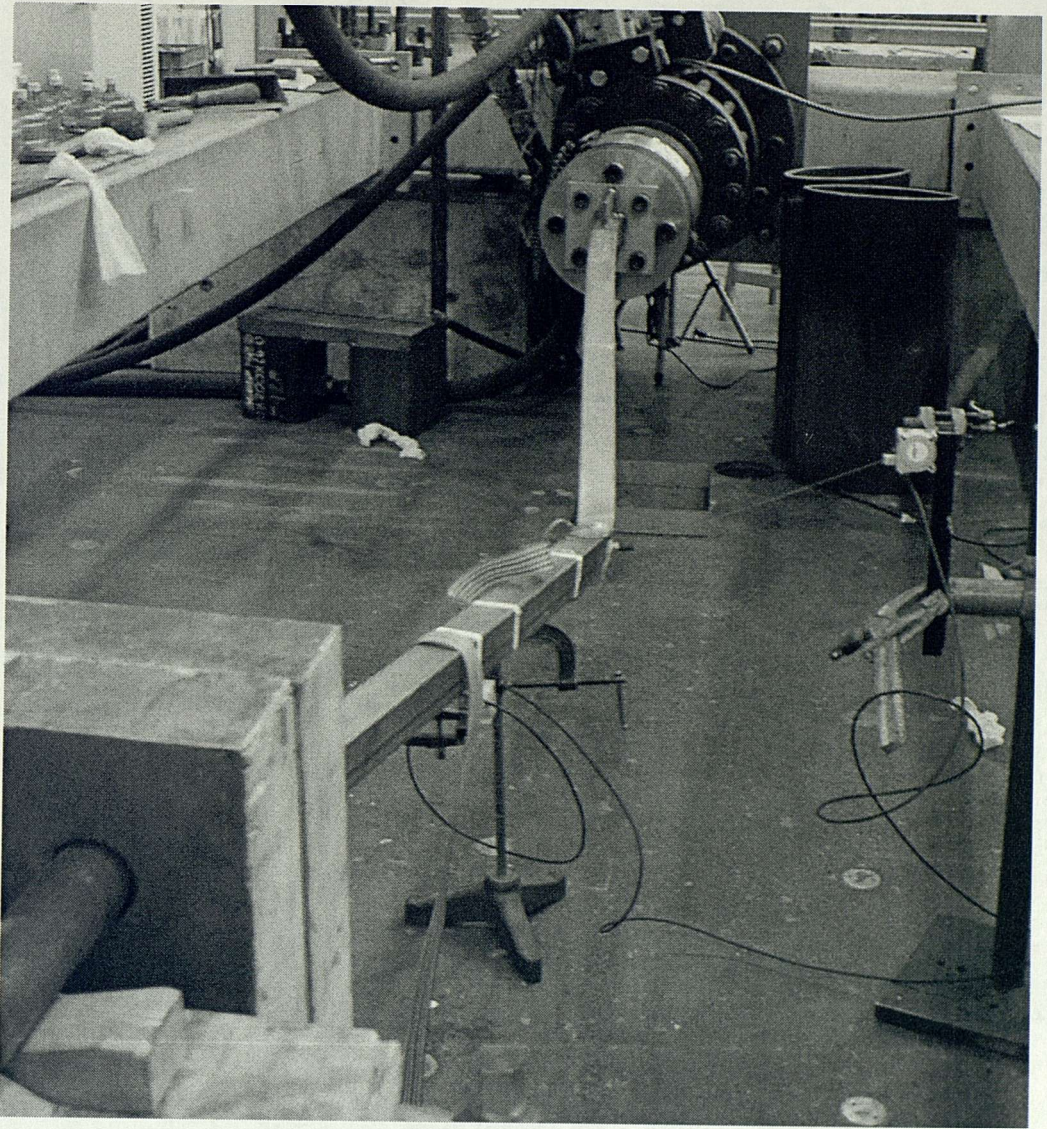


Figure 4.5: *Formation of a plastic mechanism in specimen 50×50×3×2850-SS-CF*

Measurements of mid-length lateral deflections for the short, medium and long cold-formed stainless steel $60 \times 40 \times 3$ specimens are shown in Figure 4.6, along with the predictions of lateral deflection according to Tremblay (2002). The predictions are based on the deformed geometry of the specimens after the formation of a plastic mechanism, which leads to longer specimens having larger lateral deflections at a given applied axial displacement. The measured deflections gradually increased as larger axial displacements were applied and as the plastic mechanism formed but only increased modestly during cycles at the same applied axial displacements. The permanent elongation due to yielding, but not axial shortening, is considered so the predictions provide a conservative estimation of the lateral deflections. In Figure 4.7, the measured mid-length lateral deflections for three specimens with similar global slenderness but different local slenderness and materials are compared. The predicted values given by Tremblay (2002) for the three specimens are very close and have therefore been represented by a single curve. It may be seen that, as far as lateral deflection are concerned, neither local slenderness nor material properties are as influential as the length of the specimens and the deformed shape, i.e. 'elastic buckling' or 'plastic hinge mechanism'. Lateral deflections and their implications on the compressive strains of the members are discussed in detail in Section 6.2.2.

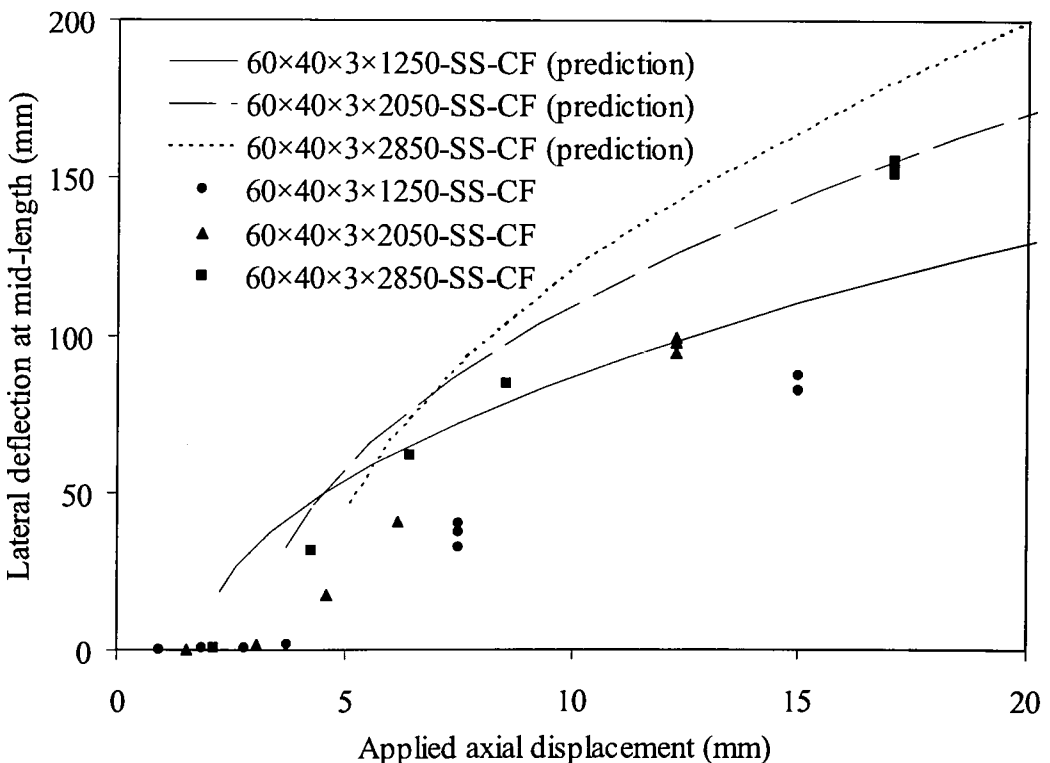


Figure 4.6: Measured and predicted (Tremblay, 2002) deflections at mid-length for three specimens with different global slenderness

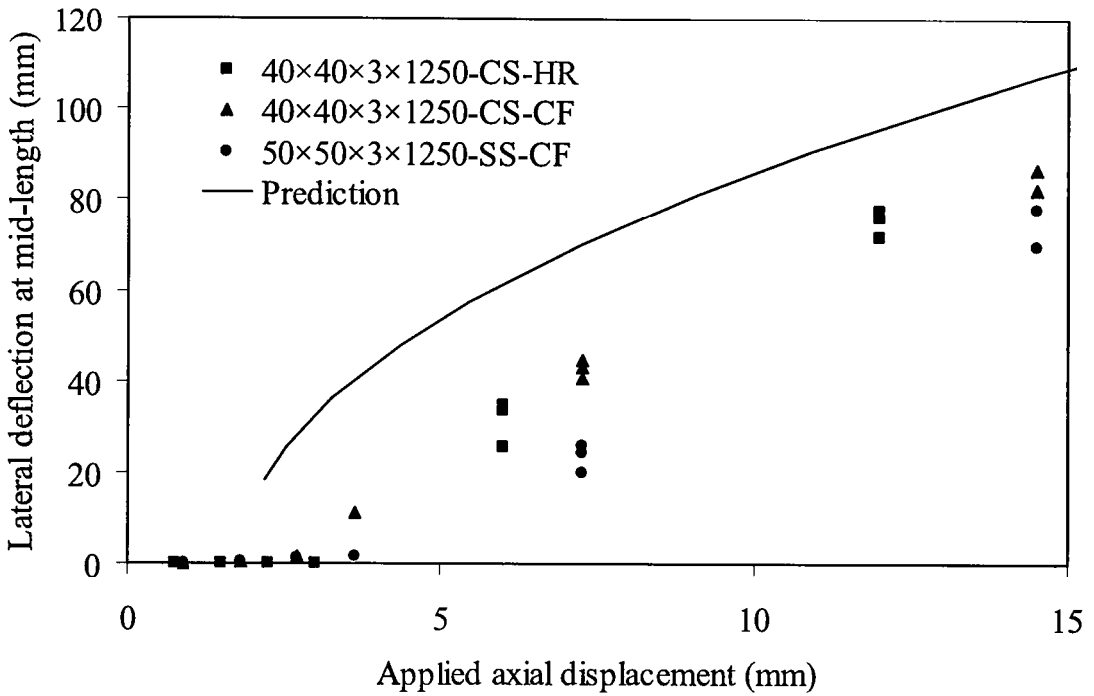


Figure 4.7: *Observed and predicted (Tremblay, 2002) deflections at mid-length for three specimens with similar global slenderness*

Typical load-displacement hysteretic curves are given for all test specimens in Figures 4.8-4.23. In the following sections, the terms tensile loading half cycle and compressive loading half cycle are used as indicated in Figure 4.8. Comparison between typical normalised load-displacement hysteretic loops for bracing members of the three materials with similar global slenderness and local slenderness can be made by considering the graphs of 60×60×3×2050-CS-HR, 60×60×3×2050-CS-CF, 50×50×3×1250-SS-CF in Figures 4.8, 4.11 and 4.22. Cyclic hardening is most pronounced for the cold-formed stainless steel specimen, which is manifested by high maximum tensile resistance in Figure 4.22. Furthermore, during the first cycle at each displacement amplitude, a non-zero tangent stiffness is maintained throughout almost the entire tensile half-cycle. In contrast, the tensile resistance of the carbon steel specimens reaches a plateau earlier in the corresponding cycles. In the second and third tensile half-cycles at $\pm 4\delta_y$, for example, the stainless steel specimen maintains a high tangent stiffness while the carbon steel specimens show a concave curve in the tensile loading half cycle and low tangent stiffness at zero applied displacement. A rise in tensile resistance with increasing displacement amplitude is a common feature for stainless steel specimens due to the substantial hardening of the material, while the tensile resistance of cold-formed carbon steel specimens remains largely unchanged

throughout the tests and that of the hot-rolled carbon steel specimens drops slightly as displacement increases.

During the elastic stage of every compressive half-cycle, the load-displacement response follows a similar slope to that of the initial elastic loading curve. The response of stainless steel members typically deviates from linearity sooner than the carbon steel specimens. Upon initial buckling of the carbon steel specimens, a sharp drop of compressive resistance was observed, while a more gradual loss of compressive resistance was observed for the stainless steel specimens. Discussions of the shape of the hysteretic loops are revisited in Section 6.3.2 with the additional results from the parametric studies.

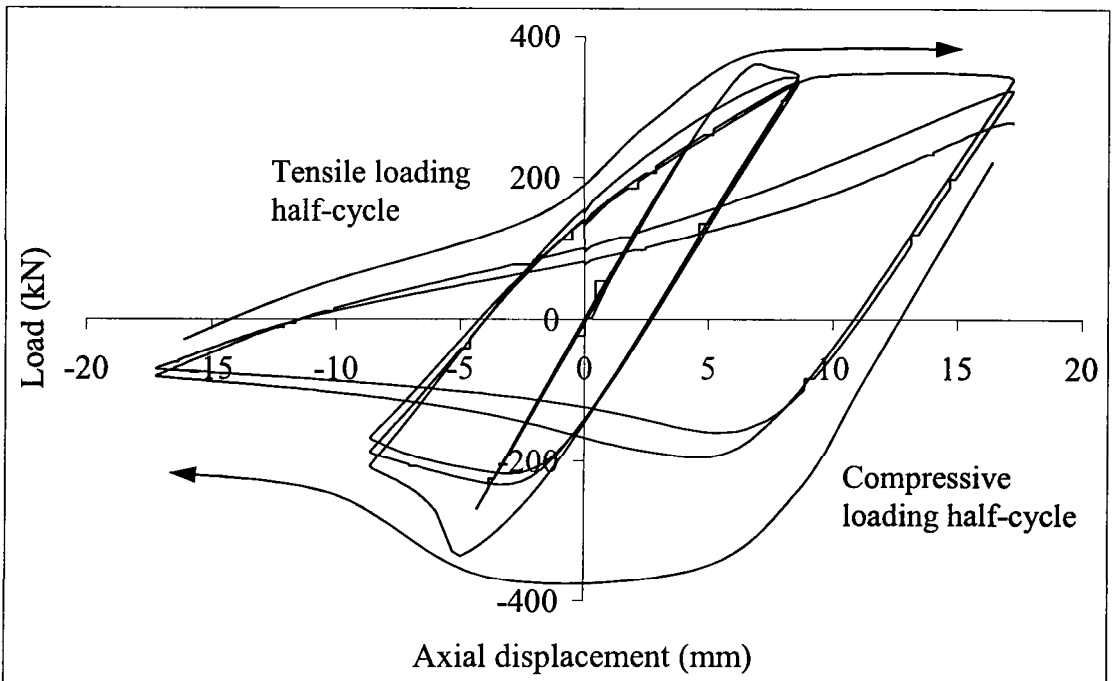


Figure 4.8: *Load-displacement hysteretic loops for specimen 60×60×3×2050-CS-HR*

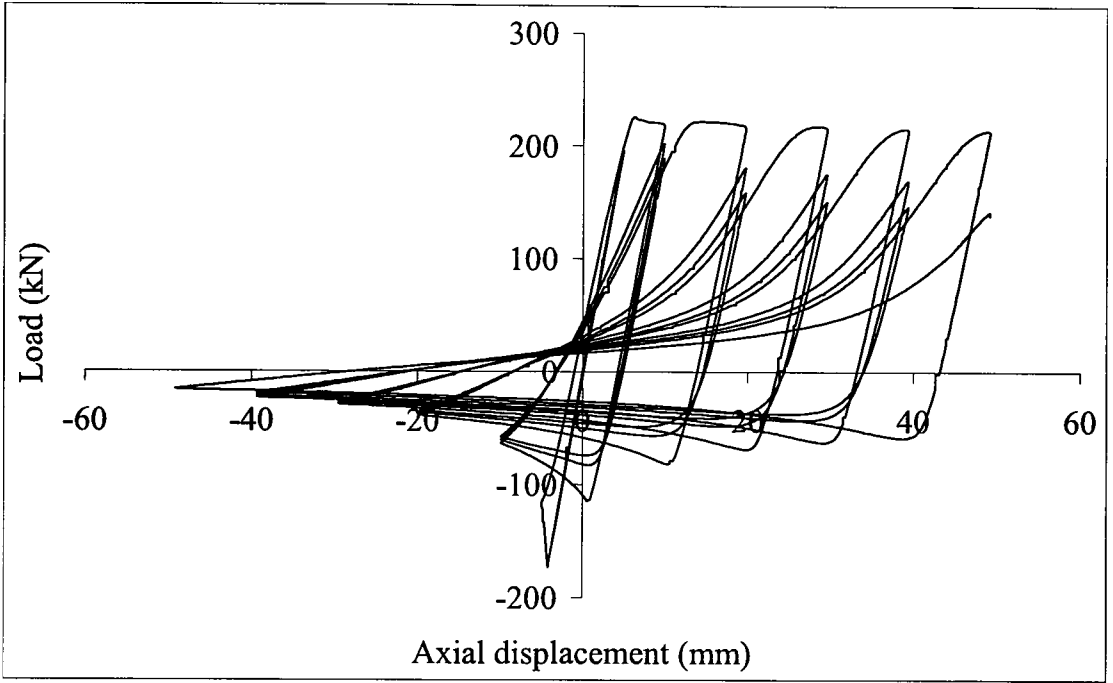


Figure 4.9: *Load-displacement hysteretic loops for specimen 40×40×3×2050-CS-HR*

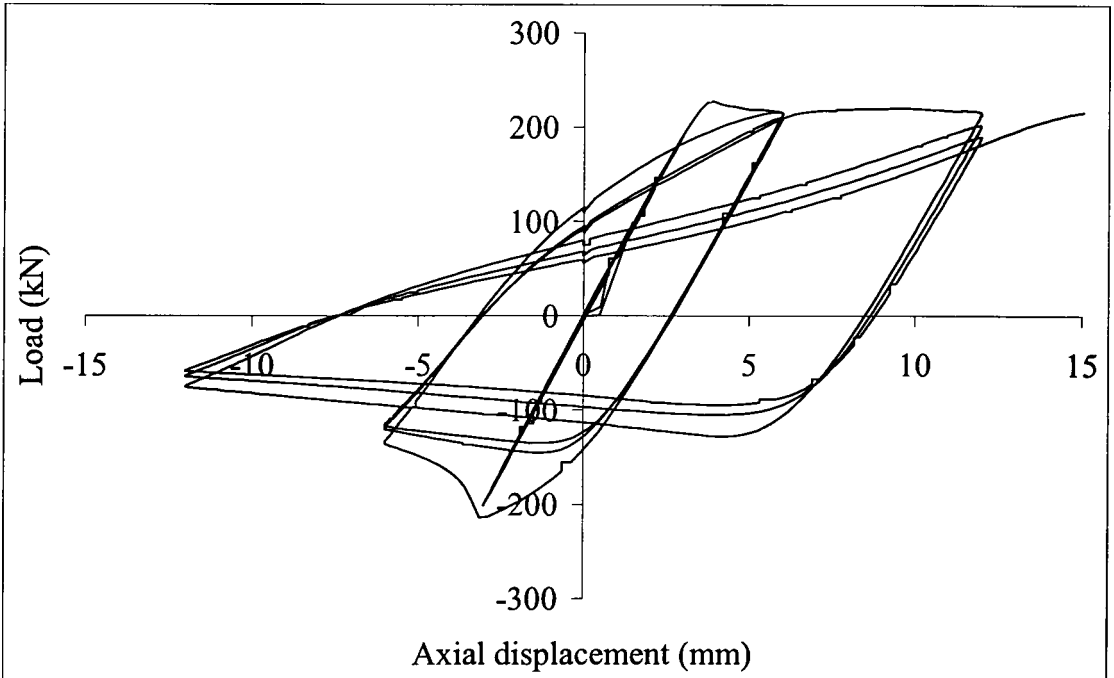


Figure 4.10: *Load-displacement hysteretic loops for specimen 40×40×3×1250-CS-HR*

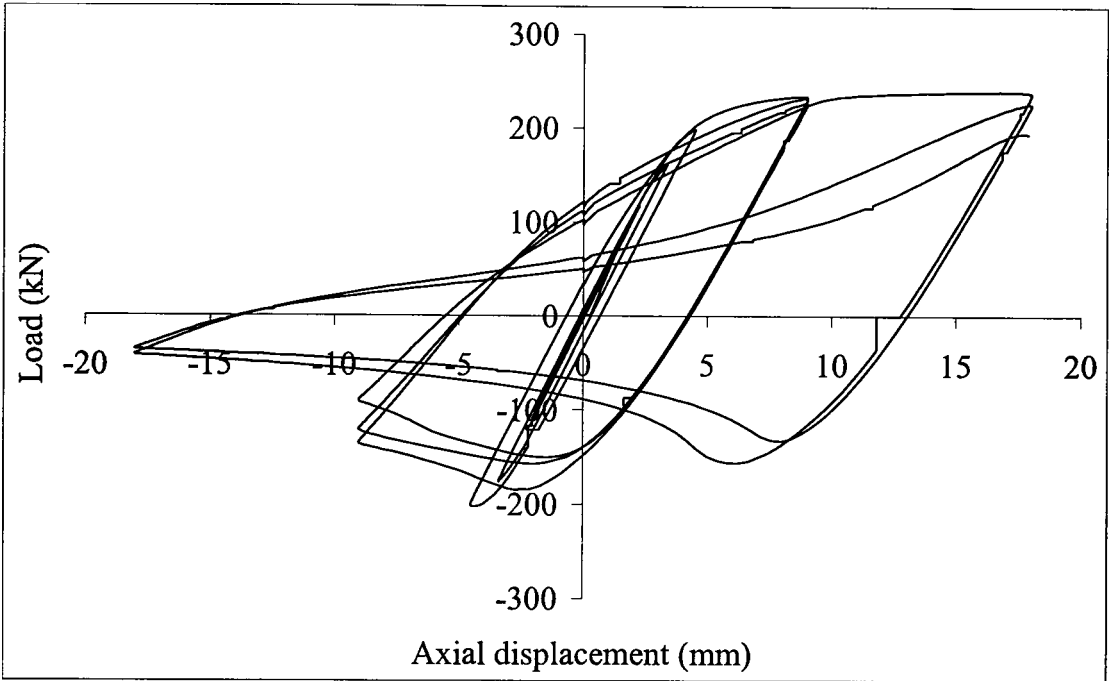


Figure 4.11: Load-displacement hysteretic loops for specimen 60×60×3×2050-CS-CF

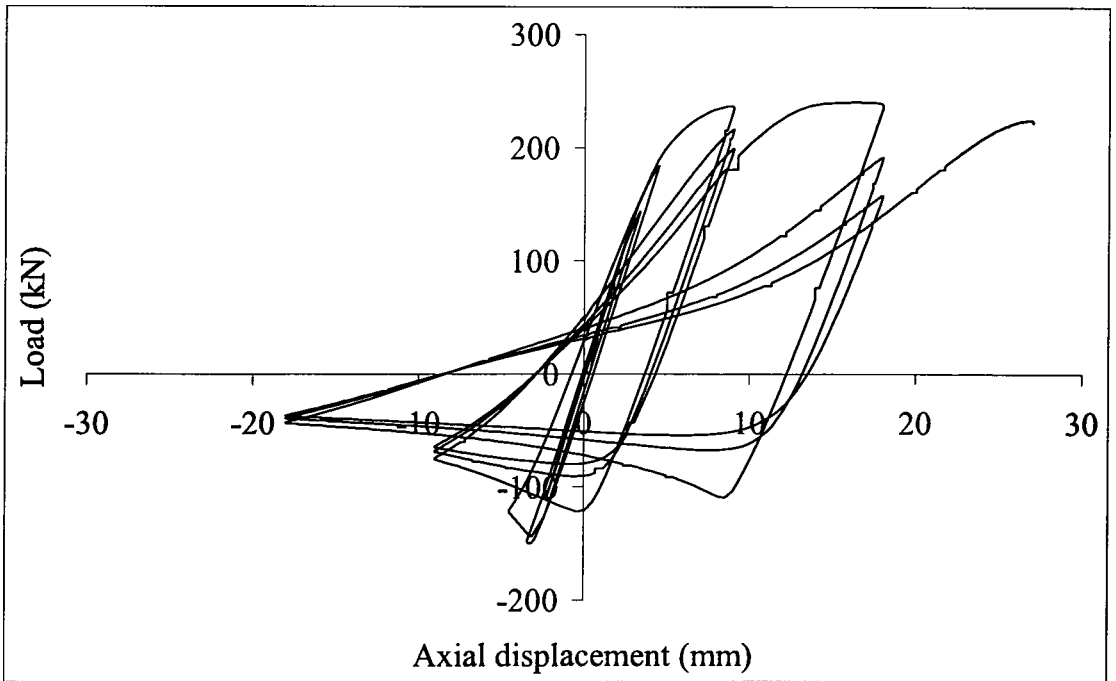


Figure 4.12: Load-displacement hysteretic loops for specimen 40×40×4×2050-CS-CF

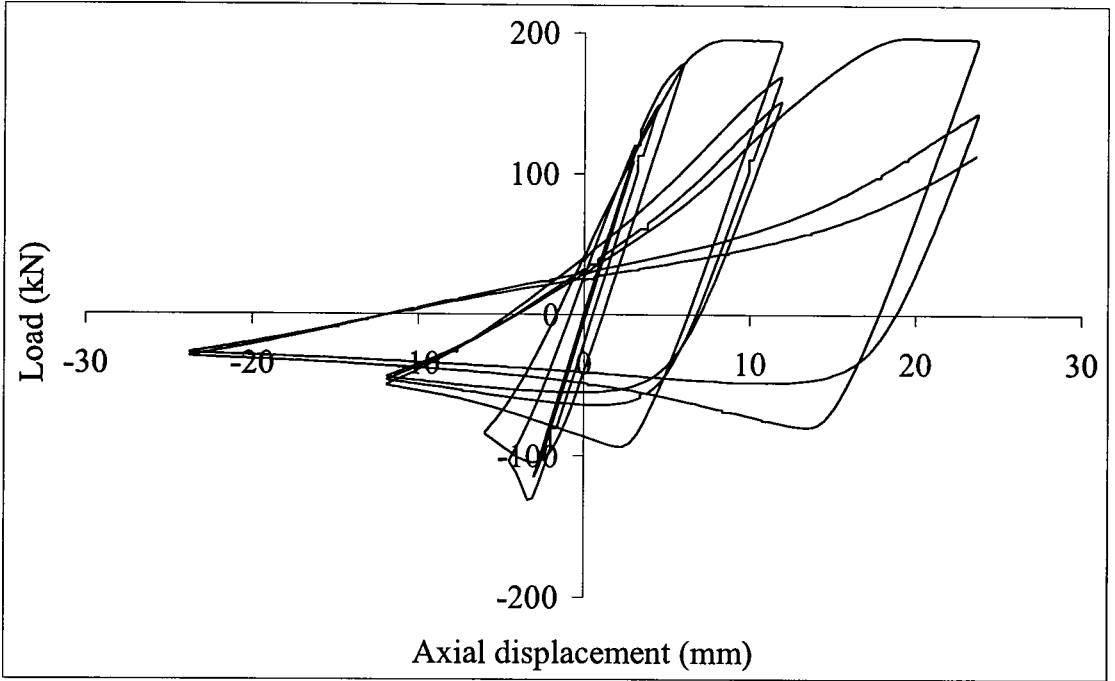


Figure 4.13: *Load-displacement hysteretic loops for specimen 40×40×3×2050-CS-CF*

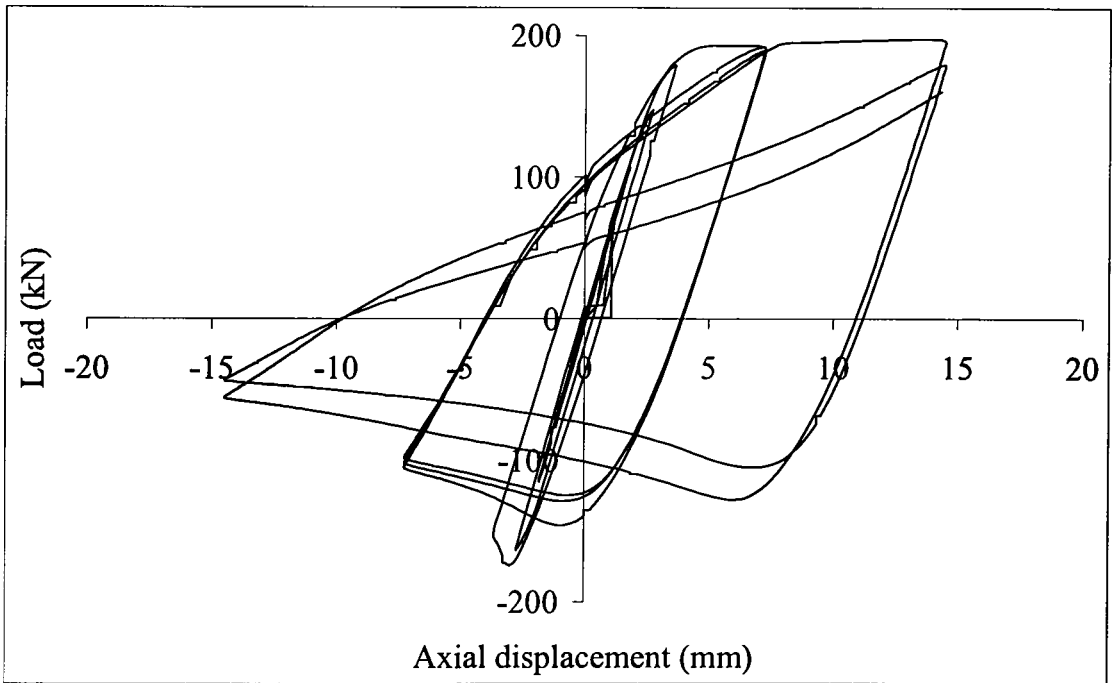


Figure 4.14: *Load-displacement hysteretic loops for specimen 40×40×3×1250-CS-CF*

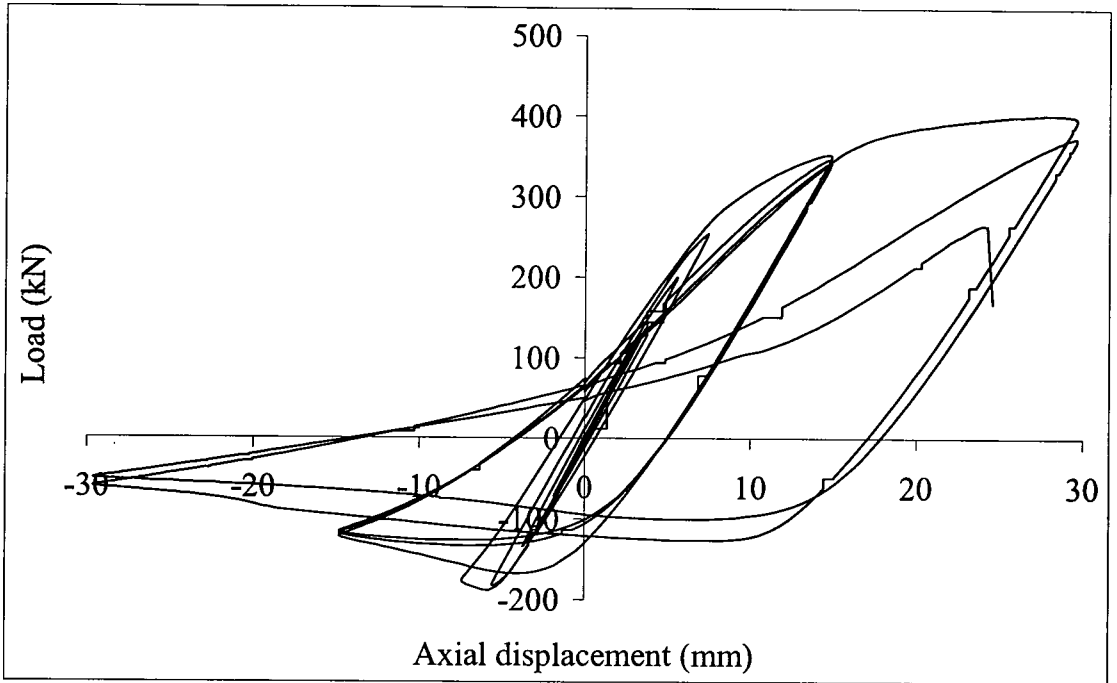


Figure 4.15: *Load-displacement hysteretic loops for specimen 60×60×3×2850-SS-CF*

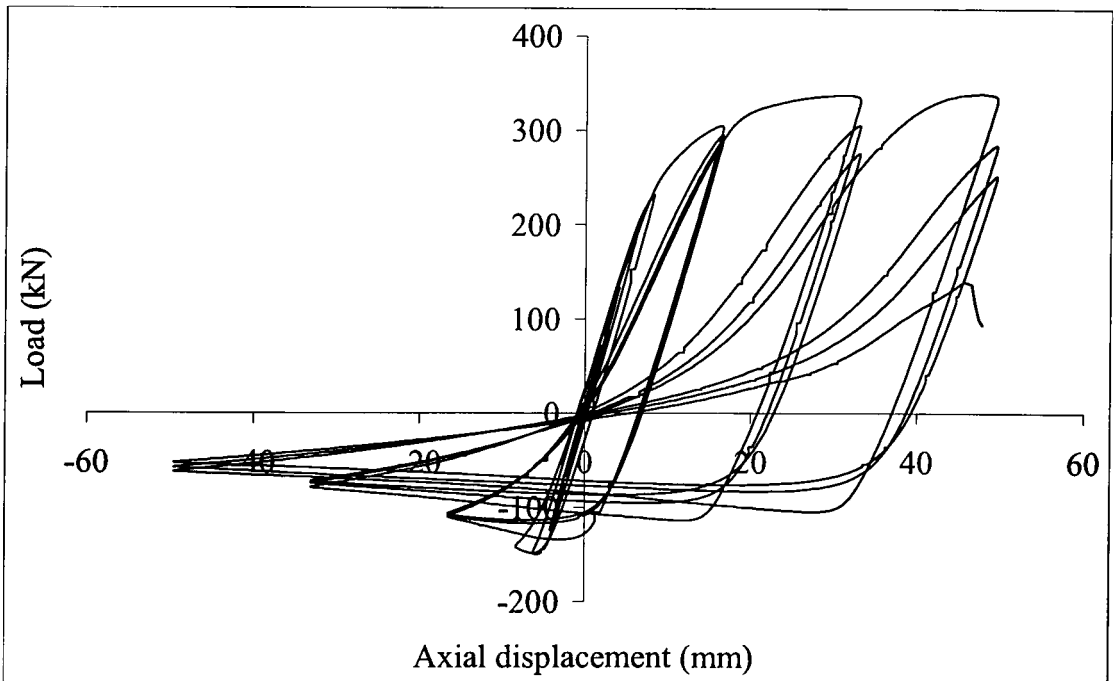


Figure 4.16: *Load-displacement hysteretic loops for specimen 50×50×3×2850-SS-CF*

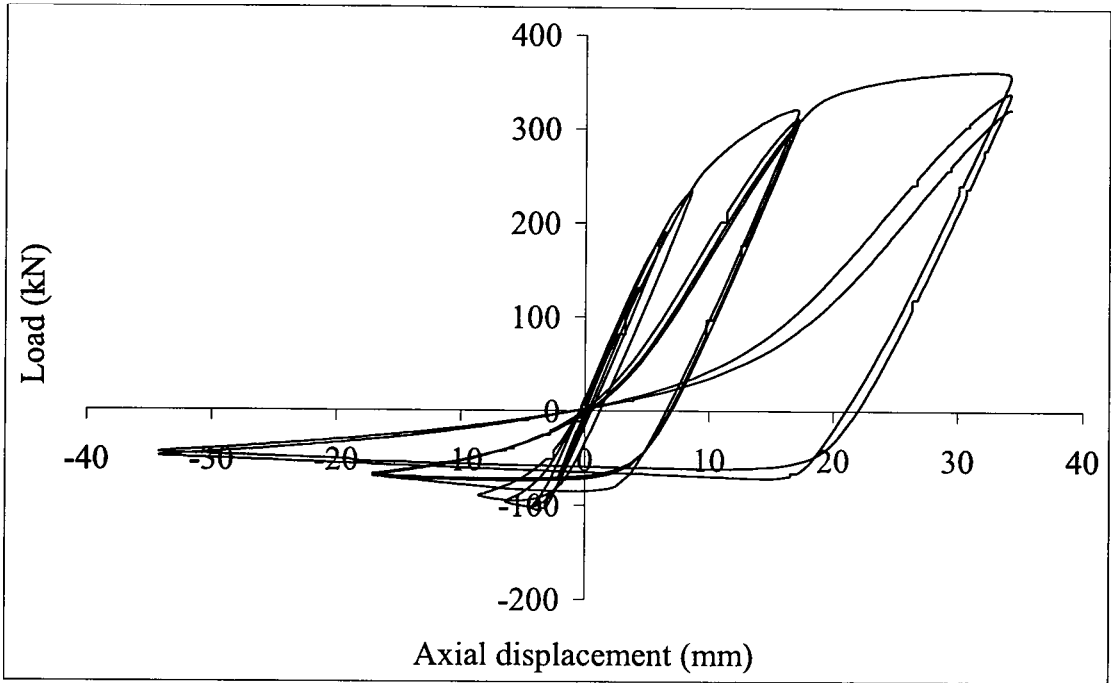


Figure 4.17: *Load-displacement hysteretic loops for specimen 60×40×3×2850-SS-CF*

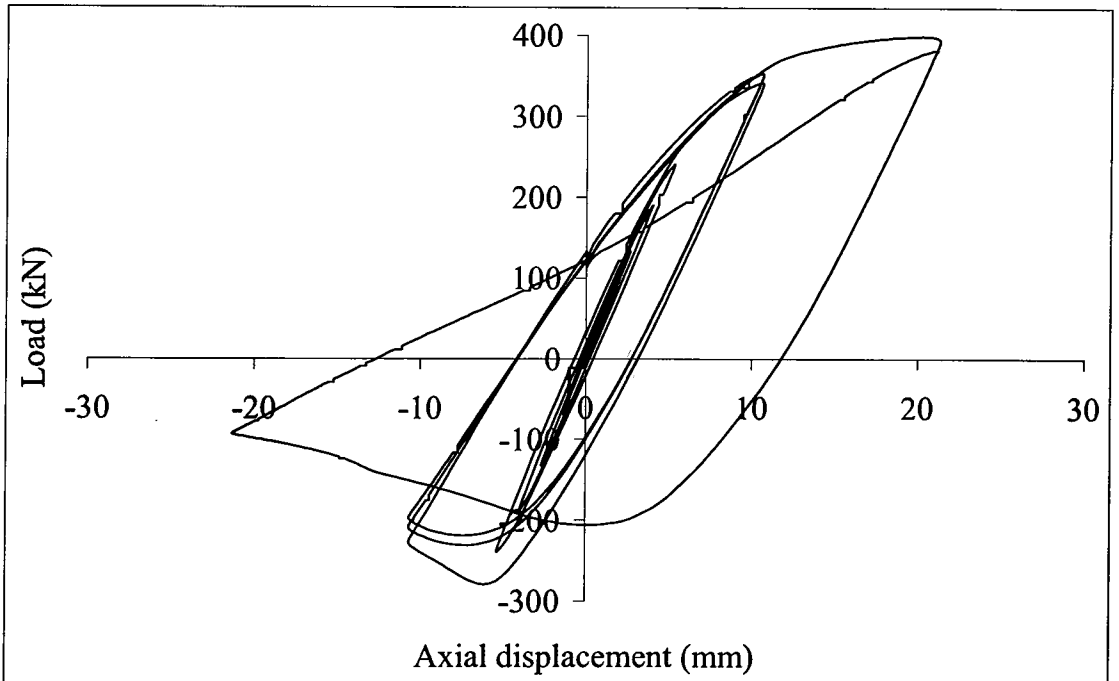


Figure 4.18: *Load-displacement hysteretic loops for specimen 60×60×3×2050-SS-CF*

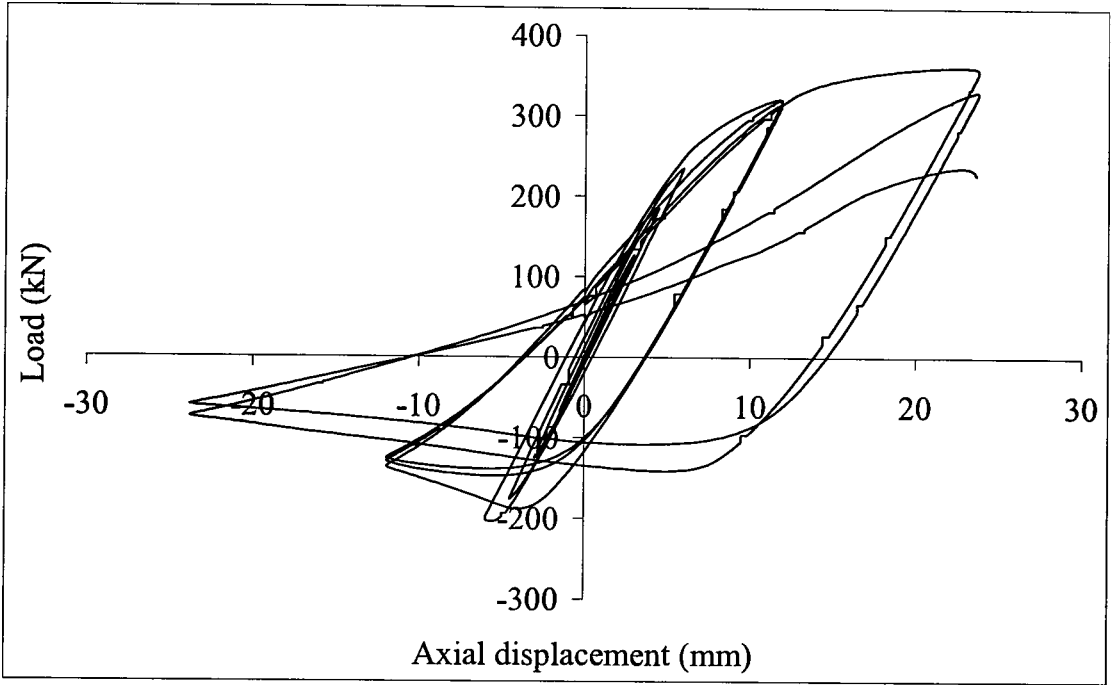


Figure 4.19: Load-displacement hysteretic loops for specimen $50 \times 50 \times 3 \times 2050$ -SS-CF

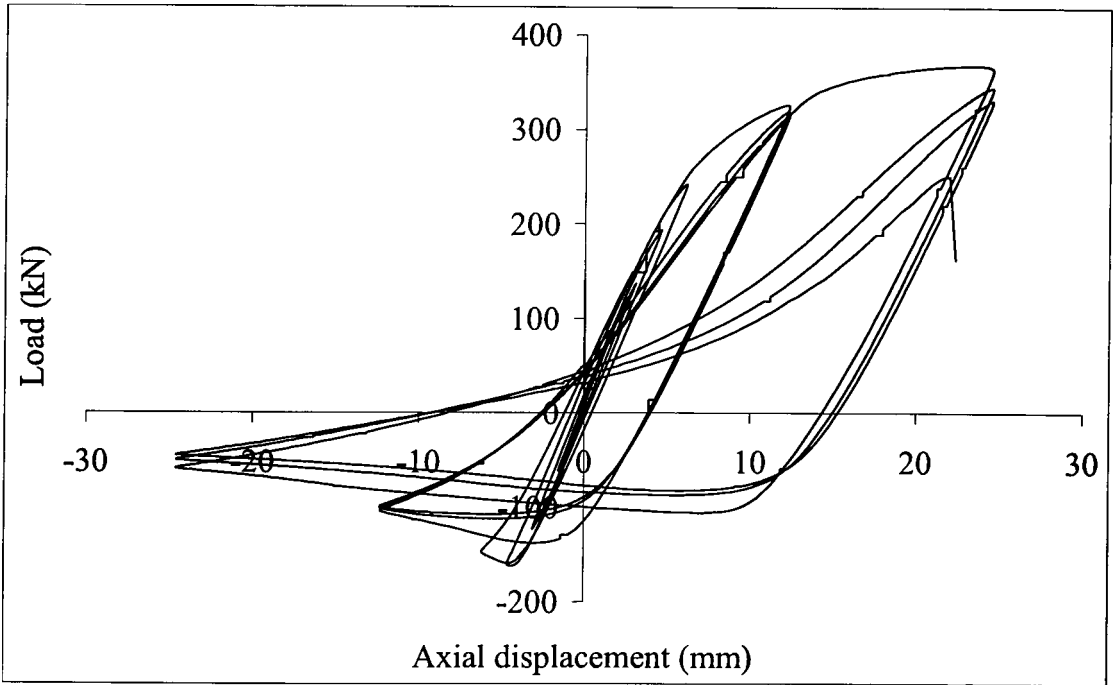


Figure 4.20: Load-displacement hysteretic loops for specimen $60 \times 40 \times 3 \times 2050$ -SS-CF

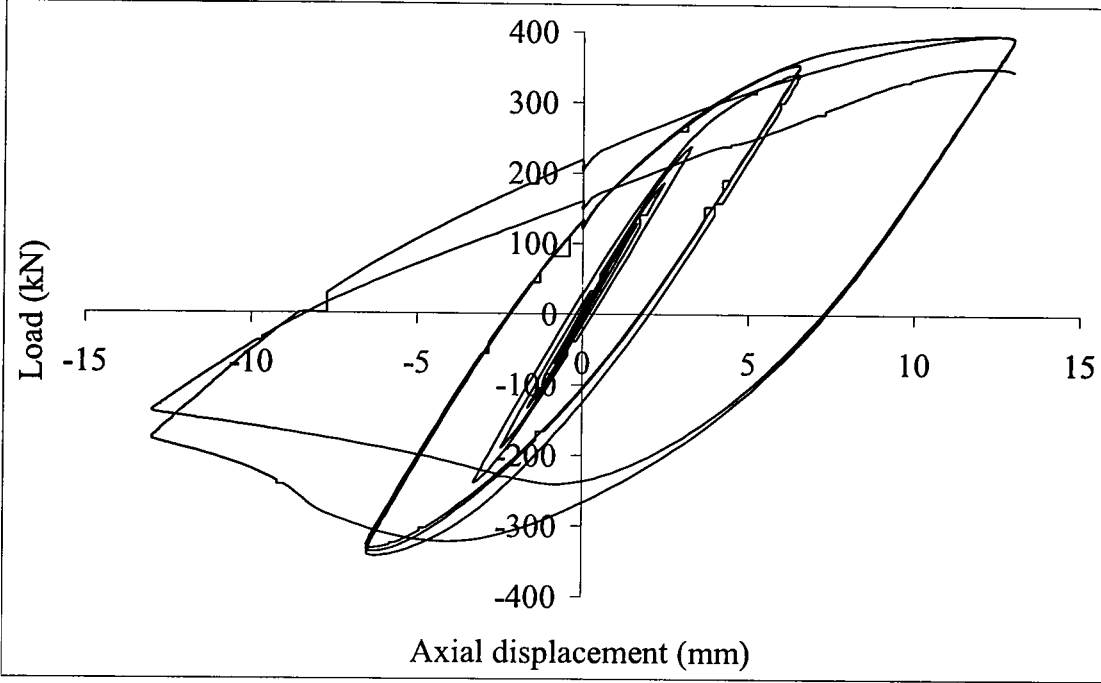


Figure 4.21: *Load-displacement hysteretic loops for specimen 60×60×3×1250-SS-CF*

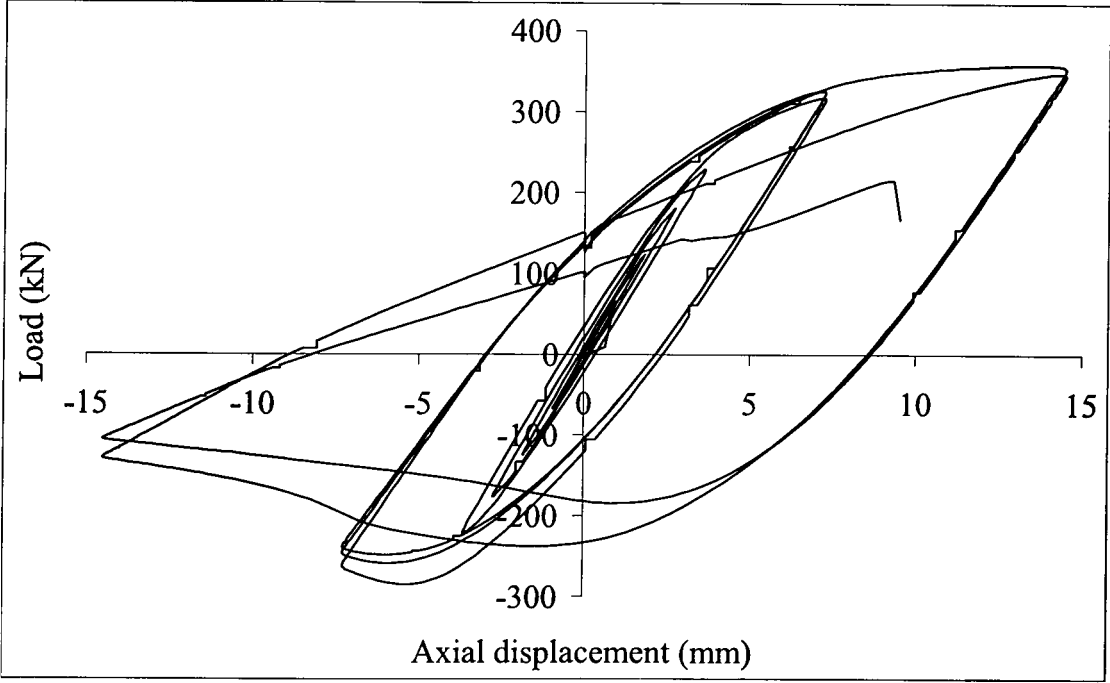


Figure 4.22: *Load-displacement hysteretic loops for specimen 50×50×3×1250-SS-CF*

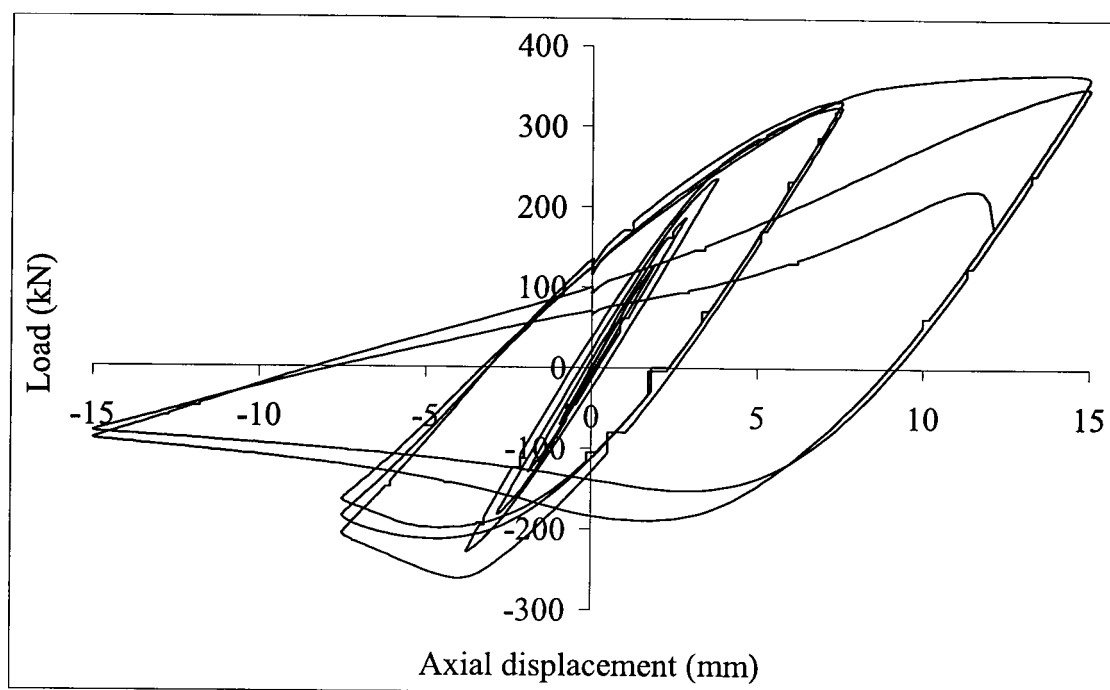


Figure 4.23: Load-displacement hysteretic loops for specimen 60×40×3×1250-SS-CF

During the compressive loading half cycles, nonlinearity initiates earlier for the stainless steel member but the deterioration of compressive resistance is slower after buckling has occurred. This is reflected in Figure 4.24 which shows the change in maximum compressive resistance at various applied displacements, where the values are normalised by the yield load, F_y , and the yield displacement, δ_y , respectively. The stainless steel specimen generally develops a higher compressive resistance in comparison to the specimens of the other two materials as the test progresses due to the more substantial cyclic hardening of the stainless steel material.

The normalised initial buckling loads observed in the experiments are plotted against global slenderness and are compared with the design resistance given in Eurocode 3 (Figure 4.25). Buckling ‘curve a’ and ‘curve c’ in EC3 Part 1-1 (EN 1993-1-1, 2005) are prescribed for hot-rolled and cold-formed carbon steel hollow sections respectively. The buckling curve for stainless steel hollow sections provided in EC3 Part 1-4 (EN 1993-1-4, 2006) has a plateau length $\bar{\lambda}_0 = 0.4$ and an imperfection factor $\alpha = 0.49$. Although the design curves have been derived from quasi-static test results, they offer satisfactory prediction of the buckling loads of the test specimens which have been

preceded by a number of small amplitude load cycles. The stainless steel curve gives the best agreement with the tests while the hot-rolled and cold-formed carbon steel data points are all above their corresponding curves.

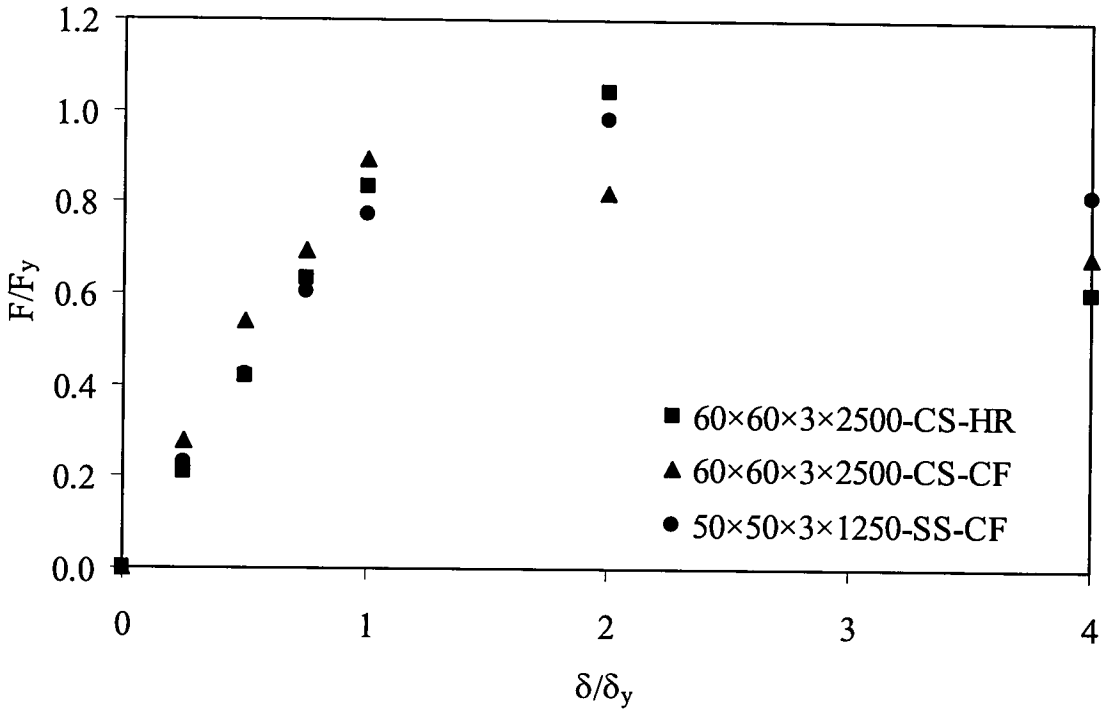


Figure 4.24: Maximum compressive resistance at various displacement amplitudes

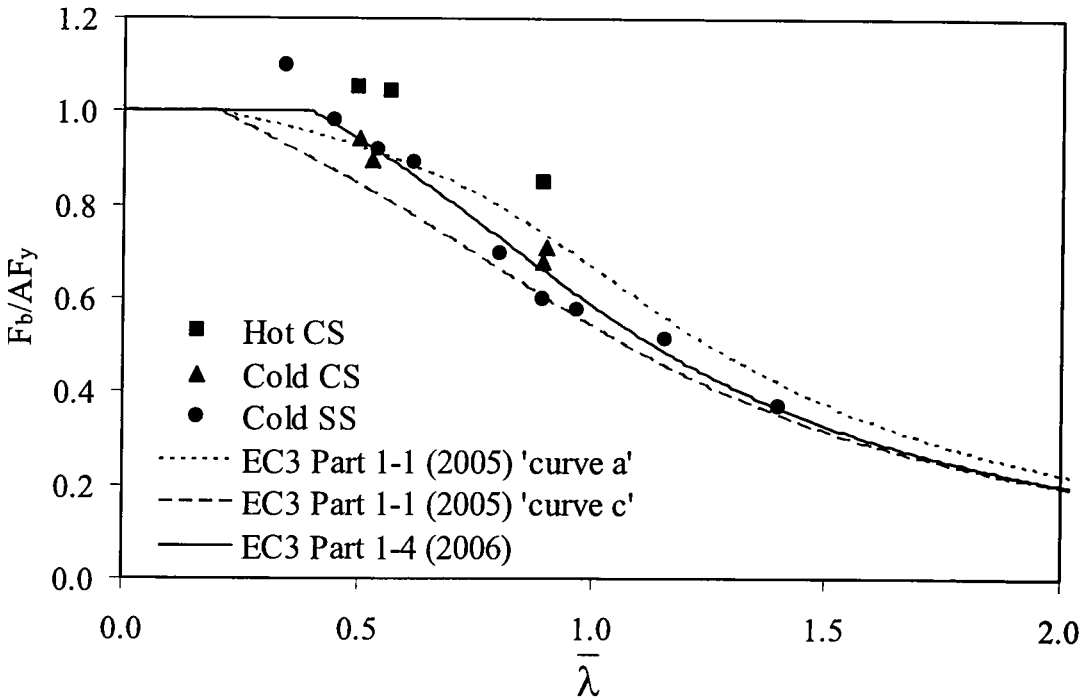


Figure 4.25: Normalised initial buckling load of cyclic specimens compared with static buckling load given by Eurocode 3

The compressive resistance in subsequent cycles reduces due to the accumulated elongation and residual lateral deformation of the specimens. In order to evaluate the minimum post-buckling resistance of bracing members at certain ductility level, which is a critical loading scenario for determining the forces developed in other frame members (Elghazouli, 2003), various predictive equations have been proposed. According to AISC (2002), the design post-buckling compressive strength is 30% of the initial buckling strength regardless of global slenderness and applied displacement. Tremblay (2002) has collected test results with symmetrical loading histories and derived expressions for minimum post-buckling resistance at compressive displacements of $2\delta_y$, $3\delta_y$ and $5\delta_y$ respectively in terms of global slenderness. Similarly, Remennikov and Walpole (1998) have proposed an empirical relationship for post-buckling resistance at a compressive displacement of $5\delta_y$ for braces with global slenderness less than 1.27.

The symmetric loading regime employed in the present study simulates braces in frames with similar level of expected peak ductility in tension and compression. In light of this, the minimum post-buckling resistance is taken as the load in the test member when the target compression ductility level is reached for the first time after the specimen has experienced the same ductility level in tension. Minimum post-buckling resistances of the test specimens at compression displacements equal to $2\delta_y$ (denoted as $F_{c,\mu\Delta=2}$) and $4\delta_y$ (denoted as $F_{c,\mu\Delta=4}$) are shown in Figures 4.26 and 4.27 respectively, along with predictive curves (Remennikov & Walpole, 1998; Tremblay, 2002; AISC, 2002). As suggested in a previous study (Tremblay, 2002), the predictions of resistance at compressive displacements equal to $4\delta_y$ have been obtained by linear interpolation of the predictions at displacements equal to $3\delta_y$ and $5\delta_y$. The behaviour of the specimens of the three materials follows a similar trend, with stainless steel specimens retaining slightly higher post-buckling resistance. Tremblay's (2002) expressions provide good estimations of the post-buckling resistance for stainless steel specimens at displacements equal to $2\delta_y$ and $4\delta_y$, despite the expressions having been derived from results of carbon steel specimens. The curve from AISC (2002) also gives satisfactory predictions for members with intermediate and high global slenderness at displacements equal to $4\delta_y$. However, Remennikov and Walpole (1998) overestimate

the post-buckling resistance of the specimens over much of the range of global slenderness for which the expression is intended.

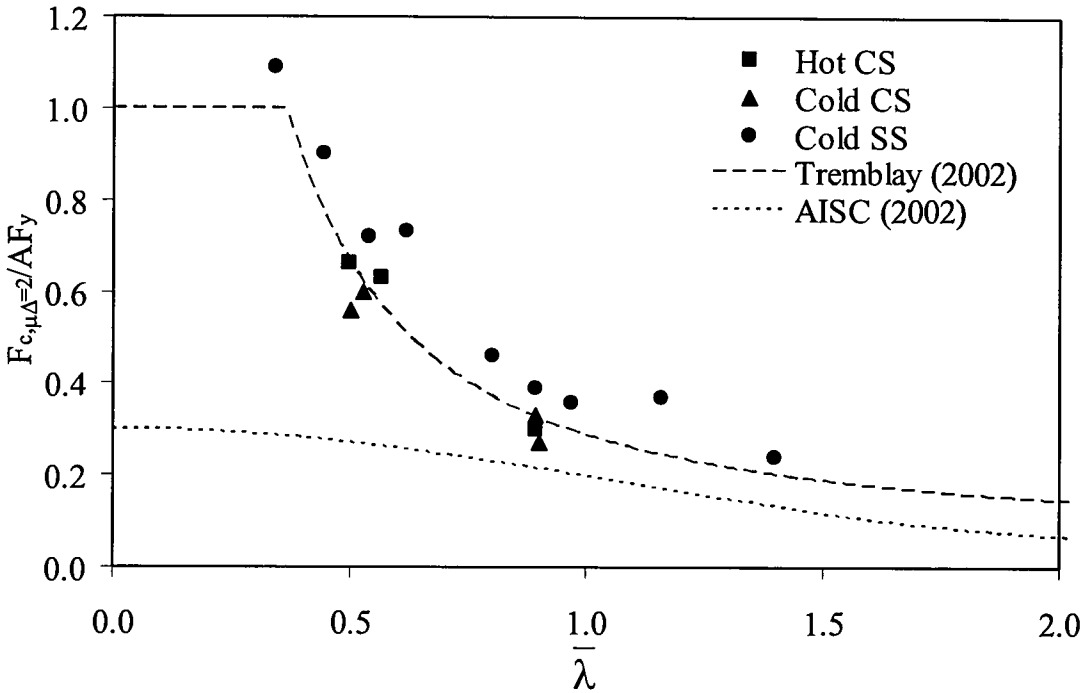


Figure 4.26: Normalised compressive resistance at compression displacement of $2\delta_y$ for cyclic specimens compared with prediction by Tremblay (2002) and AISC (2002)

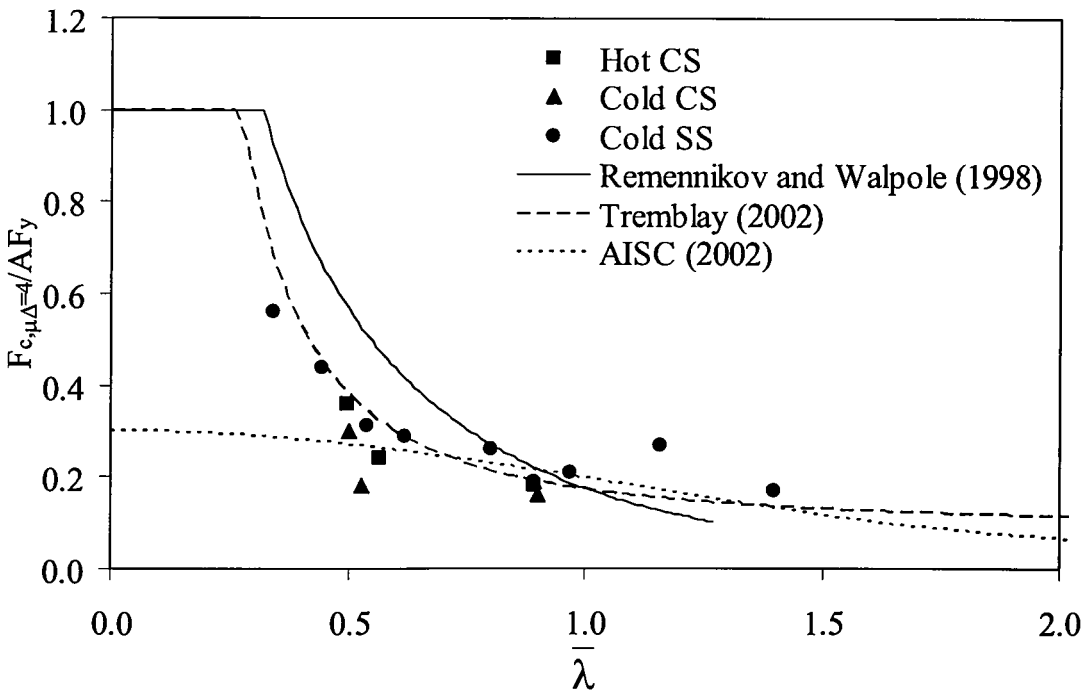


Figure 4.27: Normalised compressive resistance at compression displacement of $4\delta_y$ for cyclic specimens compared with prediction by Remennikov and Walpole (1998), Tremblay (2002) and AISC (2002)

The shape of the hysteretic loops also affects an important indicator of the performance of a bracing member – energy dissipation. In order to provide a normalised comparison of energy dissipation, hysteretic energy, W , can be normalised by yield load and yield displacement of the specimens. Although the stainless steel specimens exhibit higher tensile and compressive resistances than the carbon steel specimens, the areas under the hysteretic loops of specimens of the three materials are similar. Figure 4.28 shows the accumulation of hysteretic energy dissipation of specimens with similar slenderness in the three materials. The sharper increase in energy dissipation of the stainless steel specimen in the last two cycles relates to its ability to maintain high stiffness at $\pm 4\delta_y$, which leads to larger hysteretic loops. Figure 4.29 shows the normalised energy dissipation during the 8th cycle, which is the cycle with the largest amount of energy dissipation for most specimens. It reveals a trend of decreasing energy dissipation with increasing global slenderness but there is no obvious distinction between the three materials.

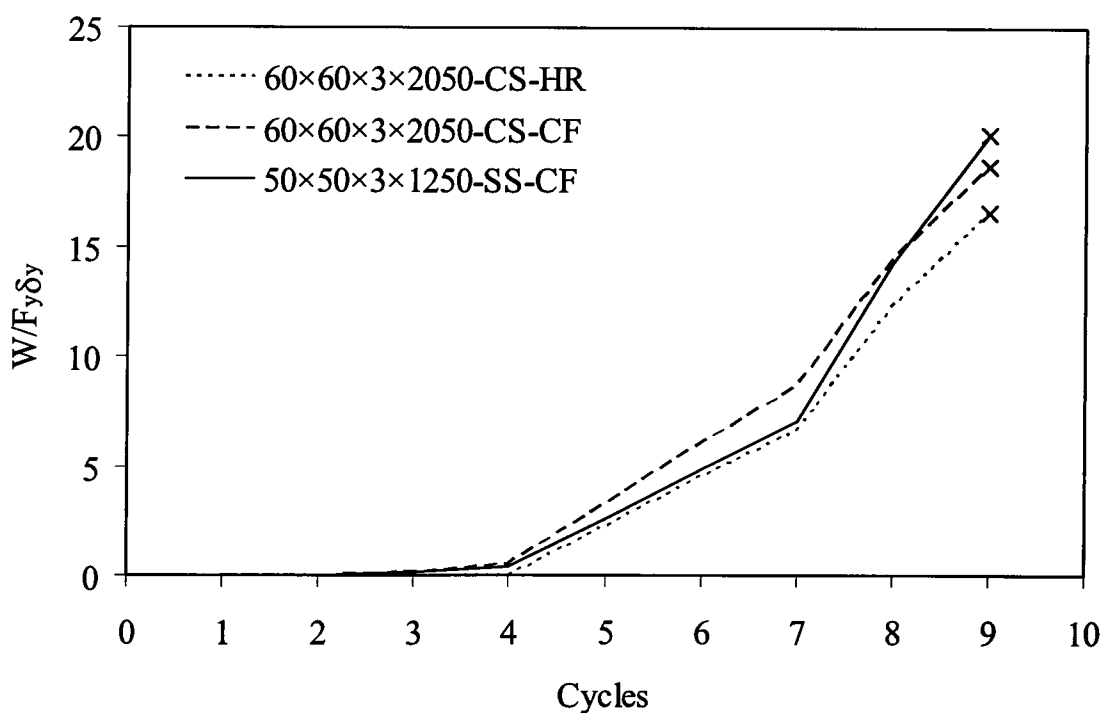


Figure 4.28: *Accumulated normalised energy dissipation*

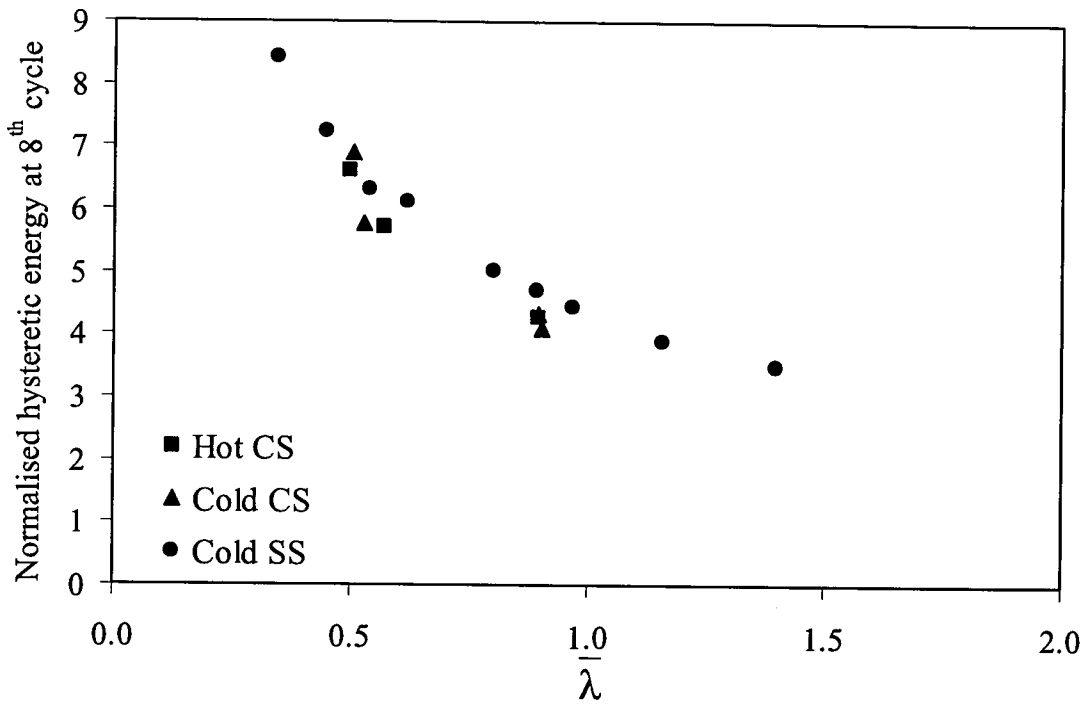


Figure 4.29: Normalised energy dissipation at the 8th cycle

4.3.2 Fracture life

Once local buckling occurs, high stresses and strains develop in the corner areas which offer higher axial stiffness than the flat faces. Strain localisation in the corner areas leads to crack initiation after a few cycles of successive inelastic buckling and tensile yielding. These cracks grow into larger opening and propagate quickly around the cross-section under tensile loading. This sequence of events was observed in all test specimens. Tests were continued until complete fracture of the specimens. Figures 4.30-4.32 show the onset of local buckling, formation of small openings at the corners and complete fracture of specimen 60×40×3×1250-SS-CF, respectively.

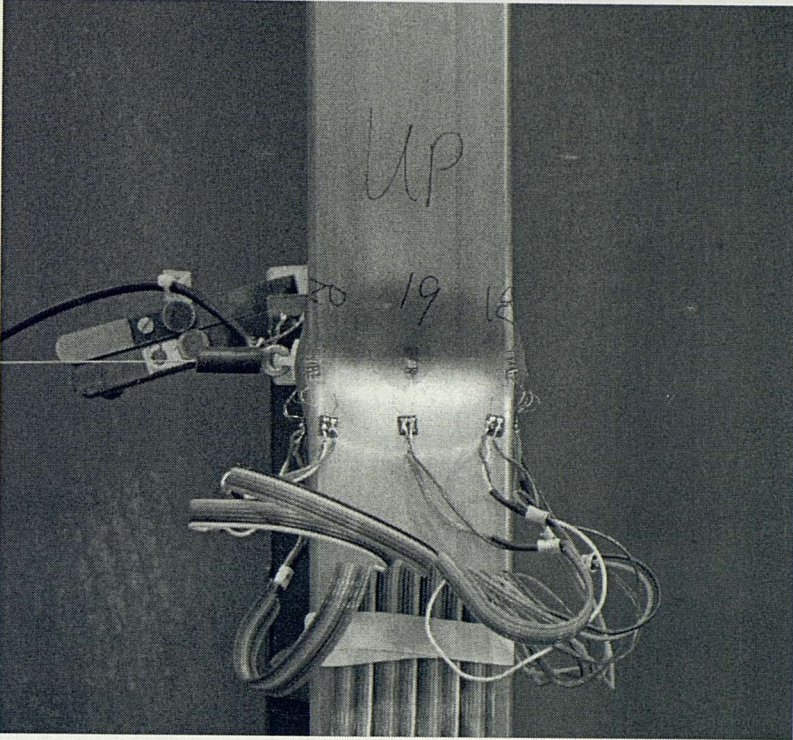


Figure 4.30: *Local buckling of Specimen 60×40×3×1250-SS-CF*

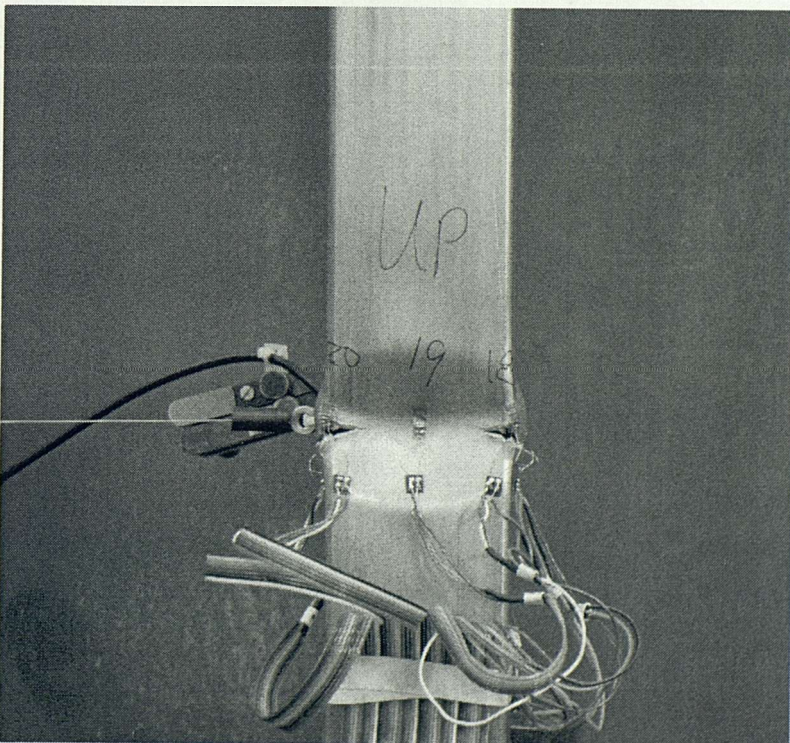


Figure 4.31: *Small opening at corners of Specimen 60×40×3×1250-SS-CF*

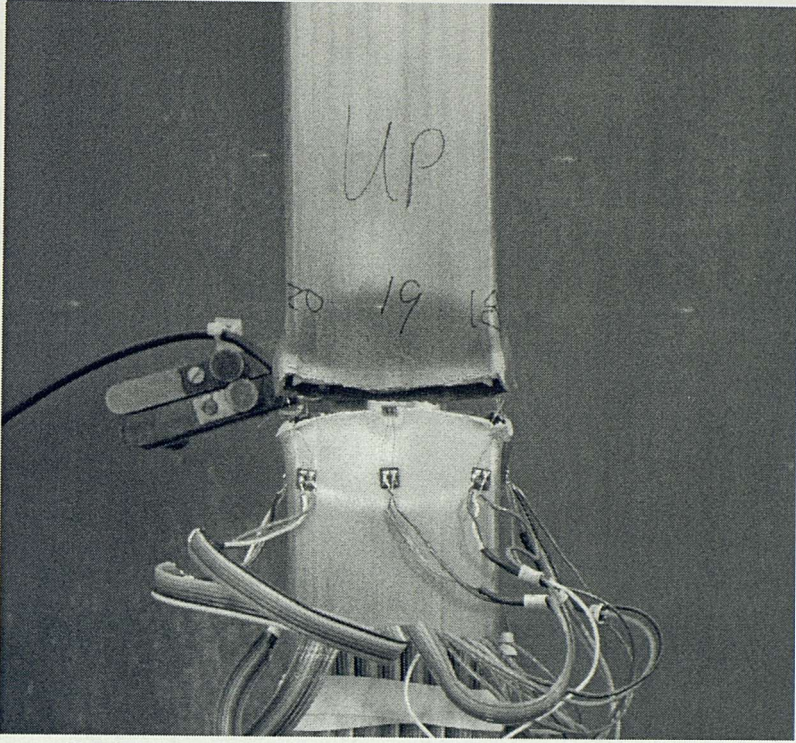


Figure 4.32: Complete fracture of Specimen $60 \times 40 \times 3 \times 1250$ -SS-CF

13 out of 16 test specimens fractured at the mid-length where the rotation was the largest. Specimens $40 \times 40 \times 4 \times 2050$ -CS-CF and $40 \times 40 \times 3 \times 2050$ -CS-CF fractured in the heat affected zone adjacent to the end stiffeners while specimen $60 \times 40 \times 3 \times 2850$ -SS-CF failed at the weld. Although small cracks appeared in the weld of specimens $40 \times 40 \times 3 \times 2050$ -CS-HR, $40 \times 40 \times 3 \times 1250$ -CS-CF and $60 \times 60 \times 3 \times 2050$ -SS-CF, final fracture occurred at mid-length. These weaknesses at the weld, by altering the end conditions of the members, may have led to the unexpectedly high numbers of cycles to local buckling and corner opening of specimens $40 \times 40 \times 3 \times 2050$ -CS-HR and $50 \times 50 \times 3 \times 2850$ -SS-CF. Although the weld and the heat affected zones at the ends became weak spots in some specimens, no unfavourable influence of the seam weld along the length of the specimen was observed.

Although theoretically the shorter faces of a structural cross-section provide larger restraint to the longer faces, thereby increasing their resistance to local buckling, this effect is not clearly evidenced by the test results on sections with different aspect ratio in the present study. Since a bracing member may buckle in any direction, using a

square hollow section instead of a rectangular hollow section avoids the issue of buckling about the minor axis. Further investigations into the cyclic performance of bracing members carried out in Chapter 6 are on square hollow sections only, since these are most commonly employed in practice owing to their equal flexural properties about the principal axes of the section.

Complete fracture of a specimen is a common definition of failure but is not always appropriate for signalling the significant loss of resistance of a member. Once an opening is formed, it grows quickly and the reduction in cross-sectional area weakens the specimen. Some specimens survived a number of additional cycles following corner opening by relying on a small fraction of the cross-section remaining intact; however, in practice, such a bracing member will no longer be capable of providing effective lateral stiffness to the frame. Therefore, in this study, the number of cycles to failure has been defined as the cycle when small cracks at the corner grow into an opening, which can often be indentified in a load-displacement hysteresis curve by a sudden drop in load.

Table 4.2 summarises the numbers of cycles at which global buckling, local buckling and initiation of an opening occurred. An alternative measurement of deformation capacity, the displacement ductility, is also presented in the table. This is defined as the maximum displacement to which a specimen has been subjected before failure, divided by its yield displacement. The majority of the specimens showed local buckling at around the eighth cycle when the maximum strain in the flat faces reached around 4% due to combined axial and bending deformations. These specimens failed soon afterwards at a displacement ductility of 4.

4.4 *Concluding remarks*

A total of 16 square and rectangular hollow section members of three materials – hot-rolled carbon steel, cold-formed carbon steel and cold-formed stainless steel – were tested under cyclic axial loading. The full hysteretic response and key test results from all tests have been reported. Despite a significant difference in monotonic material properties, in particular tensile ductility and pronounced strain hardening, member

behaviour was found to differ to a lesser extent. Nevertheless, stainless steel specimens showed higher tensile and compressive resistance and maintained higher tensile stiffness, when compared to carbon steel specimens with similar characteristics and slenderness. All of the test results were used in Chapter 5 for verification of the finite element models and fatigue damage prediction method.

Chapter 5

Numerical Modelling

5.1 Introduction

In addition to the experimental investigation presented in Chapter 4, numerical studies were required to examine the cyclic behaviour of bracing members over the full slenderness range. With the emphasis on assessing the ultimate behaviour and failure conditions, detailed three-dimensional finite element models of typical bracing members have been developed. They have been used instead of phenomenological models because they are able to simulate not only global behaviour, but also local buckling and cross-section deformation. Results from the numerical models have been used in conjunction with a failure prediction approach to quantify the fracture life of bracing members. The models have been validated against experimental results on braces of hot-rolled carbon steel, cold-formed carbon steel and cold-formed stainless steel, and subsequently used to conduct a number of parametric investigations to assess the influence of key parameters including the relative importance of local and global slenderness.

5.2 *Development of FE models*

5.2.1 *General*

The general purpose finite element (FE) package ABAQUS (2007) was used to develop models to replicate cyclic loading tests on SHS and RHS braces. The models were subjected to cyclic axial displacements with large plastic deformation within non-linear analyses. The numerical results were initially validated against tests, after which, a series of parametric studies was performed.

5.2.2 *Element type and mesh size*

An element type which is suitable for modelling sections with both high and low local slendernesses is required for the current investigation. The 4-node doubly curved general-purpose shell element with reduced integration, designated S4R (ABAQUS, 2007), was employed throughout the study. This element allows transverse shear deformation and can be used for thick or thin shell applications. It does not suffer from transverse shear locking, nor does it have any unconstrained hourglass modes. It has been used in previous studies for modelling hollow section columns and has produced accurate results for compressive resistance and failure modes (Ellobody & Young, 2005; Chan & Gardner, in press).

Strain outputs from the models were found to be sensitive to mesh size. In order to improve computational efficiency, varying mesh densities were employed in different areas in the models. Refined meshes were used only in areas where local buckling was anticipated, i.e. at the mid-height and ends of the members, since all braces were fixed-ended. The fine mesh at the end of the braces was required to capture the location and shape of local buckling accurately but no strain outputs from that part were used for predicting fracture. The demand of the fineness of mesh at each end of the braces was lower than at mid-length and therefore a less refined mesh was used. The extent of the refined mesh in the longitudinal direction was set to 1.5 times the dimension of the larger face of the section. This ensured that the refined area was sufficiently large to accommodate the half wavelength of the locally buckled shape, which was

approximately equal to the dimension of the larger face of the section. Two lines of symmetry were also utilised in the current models such that only one quarter of the members were modelled, as depicted in Figure 5.1.

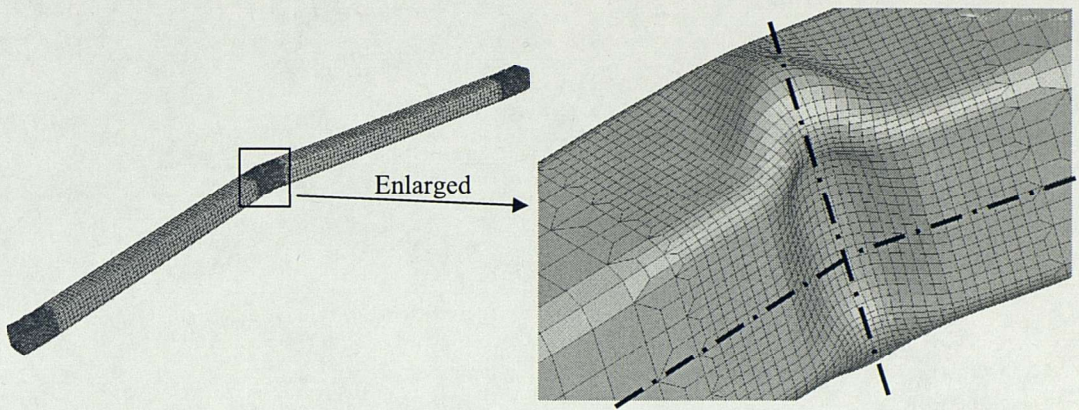


Figure 5.1: *Refined mesh at the middle and ends of member (left), and the profile of local buckling and lines of symmetry (right)*

The size of the elements in the refined mesh was chosen by means of a convergence study. The convergence study was carried out on a $40 \times 40 \times 2.5$ member (1100 mm in length) under monotonically increasing compressive load. For the reasons outlined in Section 5.2.6, both the peak strains of a single element and the average strains over a fixed area at the corner were recorded and compared at various levels of lateral displacement. It was observed that the local buckling resistance and shape were both influenced by mesh size. A finer mesh, which more closely resembles a continuum, is less stiff and therefore is more prone to buckling than a coarse mesh. Moreover, a finer mesh allows high localised peak strains to be recorded, in contrast to a coarser mesh that would provide lower, more uniform, strains over a larger area. As anticipated, the strain outputs were found to increase as the mesh size decreased (Figures 5.2 and 5.3) and since the variation in strain output was small beyond a mesh size of 32 elements per face, this mesh size was adopted for the remainder of the study.

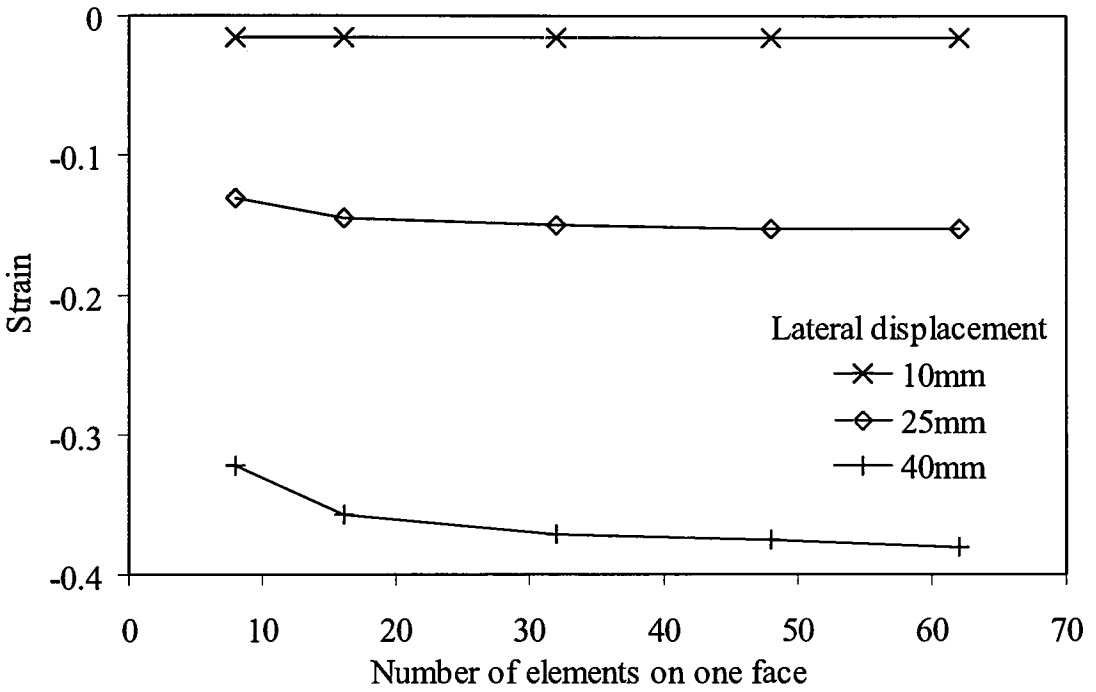


Figure 5.2: Output of average strains from different mesh sizes

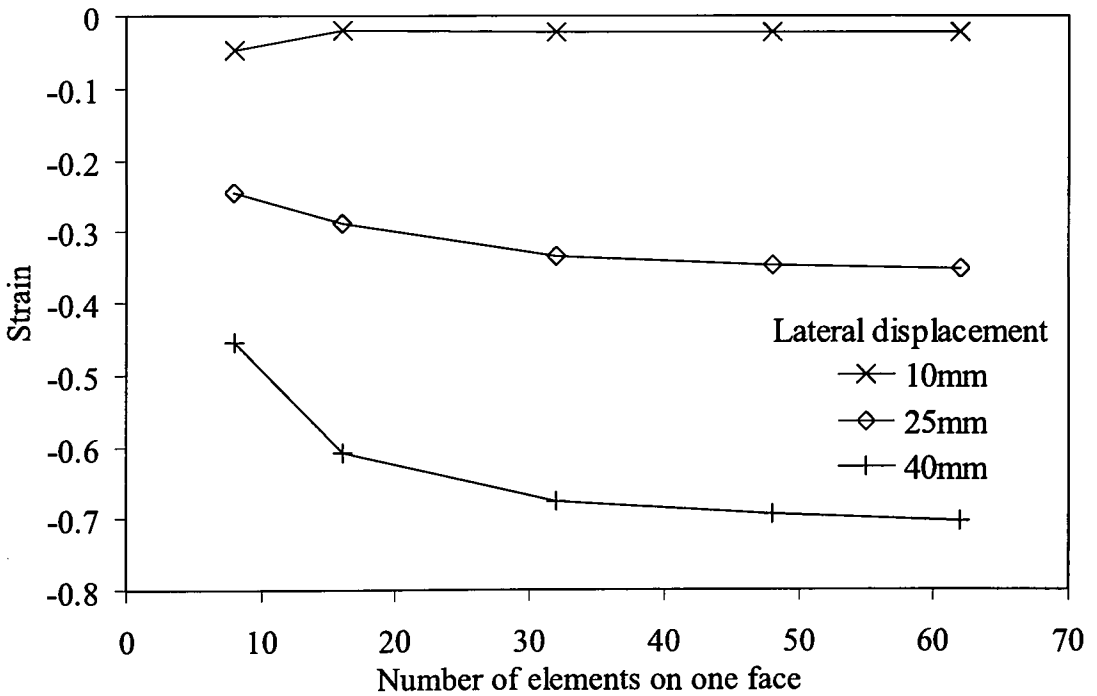


Figure 5.3: Output of peak element strains from different mesh sizes

5.2.3 Material modelling

The investigated specimens and hence the constituent material were subjected to large plastic strains and load reversals. A material model capable of simulating cyclic material properties such as the Baushinger effect and expansion of the elastic material range was needed. The nonlinear isotropic/kinematic hardening model in ABAQUS (2007) was adopted in the FE models in the current study. A series of large amplitude strain-controlled fully-reversed cyclic axial material tests was carried out on coupons cut from the flat faces of square and rectangular hollow section members to calibrate the material cyclic hardening model. Brace specimens from the same batch of members were also tested under cyclic loading, providing the data against which the FE models were validated. Details of the calibration process and the cyclic material parameters have been reported in Chapter 3.

The strength enhancement in the corner regions of the cold-formed sections was also considered. In the nonlinear isotropic/kinematic hardening model, the yield strength of the corner materials was increased by the same proportion as it was observed in the tensile coupon tests on the flat and corner materials of the hollow section members. The strength enhancement was assumed to extend beyond the corner to a distance equal to one times the material thickness for the cold-formed carbon steel models (Karren, 1967) and two times the material thickness for the cold-formed stainless steel models (Gardner & Nethercot, 2004c; Cruise & Gardner, 2008). Homogeneous material properties around the section were assumed for the hot-rolled carbon steel models.

5.2.4 Boundary conditions and displacement history

Boundary conditions were applied to the numerical models to represent the fixed end conditions of the test specimens. All degrees of freedom of nodes at both ends of the members were restrained, except for the axial displacement at the loaded end. The braces were analysed under displacement control. The applied axial displacement history followed the recommendations of ECCS (1986), i.e. one cycle at each level of 0.25, 0.5, 0.75 and 1.0 δ_y , followed by three cycles at each level of 2, 4, 6, 8 δ_y , etc. where δ_y is the estimated axial yield displacement.

5.2.5 Initial imperfections

All structural members contain initial geometric imperfections. These imperfections trigger out-of-plane deflections at stresses below the theoretical critical buckling values. In this study, both global and local initial geometric imperfections were incorporated into the numerical models to assess their influence.

The effect of global imperfection amplitude on the buckling of tubular columns under ambient and fire scenarios has been examined numerically in previous studies (Gardner & Nethercot, 2004; Ng & Gardner, 2007). The results of these studies have indicated that buckling loads were relatively insensitive to variation in imperfection amplitudes below about $L/1000$, where L is the column length, with an amplitude of $L/2000$ providing the best prediction of test behaviour.

For local imperfections, Gardner and Nethercot (2004c) calibrated the predictive formula proposed by Dawson and Walker (1972) for stainless steel specimens and gave the following expression:

$$\omega_0/t = 0.023(f_y/f_{cr}) \quad (5.1)$$

where ω_0 is the initial imperfection amplitude, t is the plate thickness, f_y is the material yield stress and f_{cr} is the plate critical buckling stress. Gardner et al. (submitted) also measured the local imperfections in the carbon steel specimens used in this study and other similar specimens. A least-squares regression analysis of the measured imperfection data for cold-formed and hot-rolled hollow sections resulted in the proposal of the predictive expressions given by Equations (5.2) and (5.3) respectively. These are based on a second Dawson and Walker (1972) model in which the ratio of f_y/f_{cr} is raised to power of 0.5.

$$\omega_0/t = 0.034(f_y/f_{cr})^{0.5} \quad (5.2)$$

$$\omega_0/t = 0.028(f_y/f_{cr})^{0.5} \quad (5.3)$$

A sensitivity study was conducted to assess the response of the models to variation in global and local imperfections. The amplitude of the local imperfection was varied from $t/10$ to $t/1000$ while that of the global imperfection was varied from $L/100$ to

L/10000. As in previous research (Gardner & Nethercot, 2004c; Ng & Gardner, 2007), the maximum compressive resistance was found to be relatively insensitive to the amplitude of the global imperfection below about L/1000. The study also showed that larger amplitudes of local imperfection (and to a lesser extent global imperfection) caused earlier local buckling but the effect on fracture was small. It was found that varying the amplitude of either global or local imperfection by a factor of 100 generally only resulted in a difference of 2 or less cycles to fracture, which was predicted by a strain-based damage prediction method (Table 5.1).

Table 5.1: Sensitivity of numbers of cycles to local buckling and fracture to variation in amplitudes of local and global imperfections

Specimen identification	Amplitude of local imperfection						Amplitude of global imperfection					
	t/10	D&W**	t/1000	t/10	D&W**	t/1000	L/100	L/1000	L/10000	L/100	L/1000	L/10000
	Number of cycles to						Number of cycles to					
	Local buckling			Fracture			Local buckling			Fracture		
60×60×3×2050-CS-HR	5	8	8	8	9	10	5	7	8	9	9	10
40×40×3×2050-CS-HR	8	9	10	10	12	12	8	9	8	11	11	12
40×40×3×1250-CS-HR	6	8	8	9	10	10	5	8	8	9	10	10
60×60×3×2050-CS-CF	5	5	8	7	8	9	5	5	5	7	8	8
40×40×4×2050-CS-CF	9	11	11	12	13	13	9	11	11	13	13	13
40×40×3×2050-CS-CF	5	8	8	8	10	10	6	8	8	9	10	10
40×40×3×1250-CS-CF	5	6	7	7	8	9	5	5	6	7	8	8
60×60×3×2850-SS-CF	6	8	8	9	10	10	6	8	8	9	9	10
50×50×3×2850-SS-CF	7	10	*	9	12	*	8	10	*	10	12	*
60×40×3×2850-SS-CF	8	9	11	10	11	13	8	8	*	11	11	*
60×60×3×2050-SS-CF	5	8	8	9	9	10	5	7	8	9	9	9
50×50×3×2050-SS-CF	7	8	9	9	10	11	7	8	8	9	10	10
60×40×3×2050-SS-CF	6	8	9	9	10	12	6	8	8	9	10	10
60×60×3×1250-SS-CF	7	8	8	10	10	11	4	6	8	9	10	11
50×50×3×1250-SS-CF	6	8	8	9	9	10	5	6	8	8	9	9
60×40×3×1250-SS-CF	5	5	6	9	9	9	5	5	5	9	9	9

* Analysis has not been completed due to numerical difficulty.

** Predicted amplitudes of local imperfection by Dawson and Walker (1972) models (Equations 5.1-5.3).

Global imperfection with an amplitude of $L/1500$ was employed for the remainder of the study. Local geometric imperfections comprising longitudinal sinusoidal waves with half-wavelengths equal to the width of the section and compatible sinusoidal half-waves across the section faces were introduced into the numerical models (Figure 5.4). The amplitudes of the local imperfections followed the values given by Equations 5.1-5.3. The global imperfections were incorporated into the models by applying an equivalent load at mid-length while the local imperfections were included by defining nodes directly in their imperfect configuration.

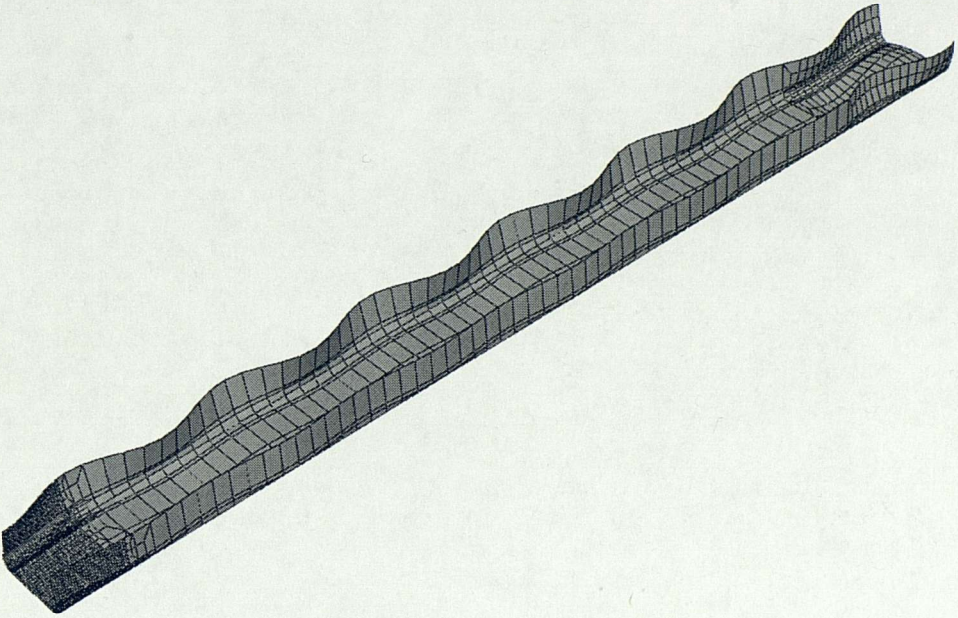


Figure 5.4: *Exaggerated initial local imperfections*

5.2.6 Output of strains

Assessment of strains was central to the parametric studies, enabling prediction of fatigue fracture from a strain-life relationship. There were factors to consider when extracting strains from the numerical models, including the measure of strain, the location of strain output and through-thickness effects. These factors are discussed below.

Unlike true stress, there is no clearly preferred true strain. The discrepancy between different strain measures becomes significant when the strain is large. ABAQUS (2007)

offers different measures and components of strain output. The total strain designated E , which is the sum of the elastic and plastic strains, was adopted and examined in this study, since this is the strain measure upon which the utilised strain-life relationships are based (see Section 5.3.2).

In general, strains may be extracted either from a single element or from a group of adjacent elements. Output from a single element can pick up the peak strain from the most critical location but is sensitive to changes in mesh size. In contrast, strains averaged over a constant area are less sensitive to the mesh size, but the extent of the chosen measurement area requires careful consideration. In this study, the measurement area was selected such that it was sufficiently small to avoid diluting the peak strains by the results of elements but large enough to be insensitive to mesh size. From the models, it was observed that the highest strains occurred at the corners and around the middle eighth of the buckling wave in the longitudinal direction (Figures 5.5 and 5.6). Furthermore, experimental evidence has shown that the ductility of the corner material of cold-formed sections is lower than that of the flat material due to work-hardening and that fracture generally initiates at the corners. Average strains over an area covering the corner and one-eighth of the buckling wavelength were therefore used to predict damage and calculate fatigue life. For comparison, the results from both the peak strain values from a single element and the average values of the elements over a constant area are presented and examined later in this chapter.

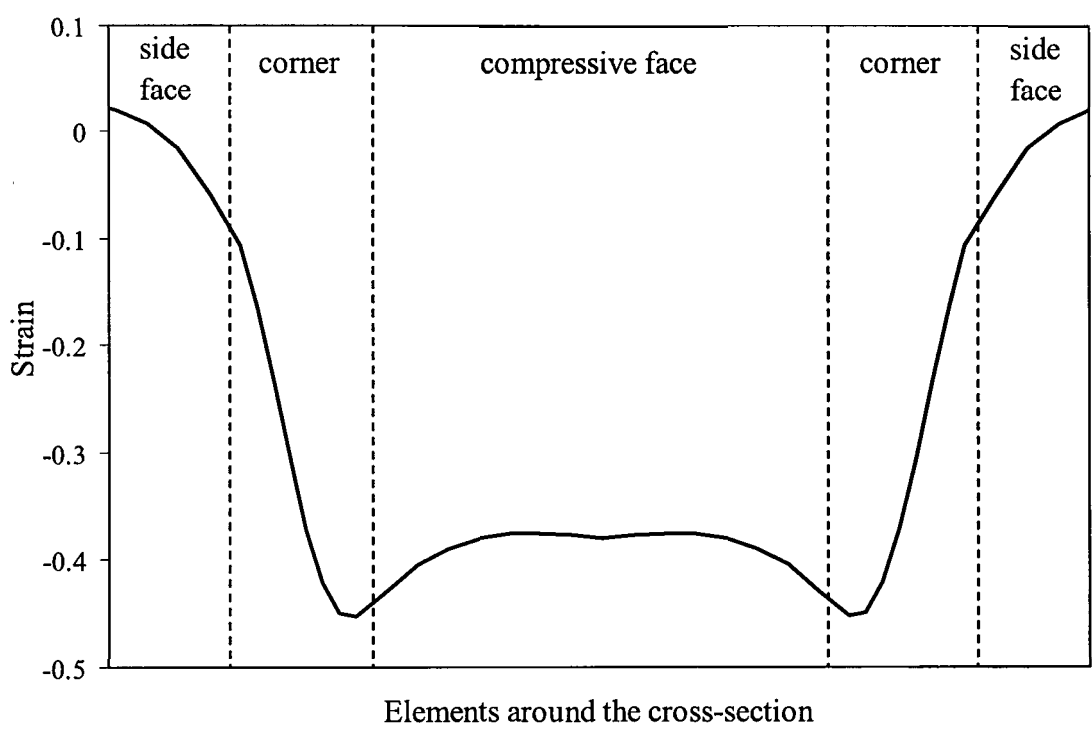


Figure 5.5: *Strain distribution around the compressive face of the section at the region of local buckling*

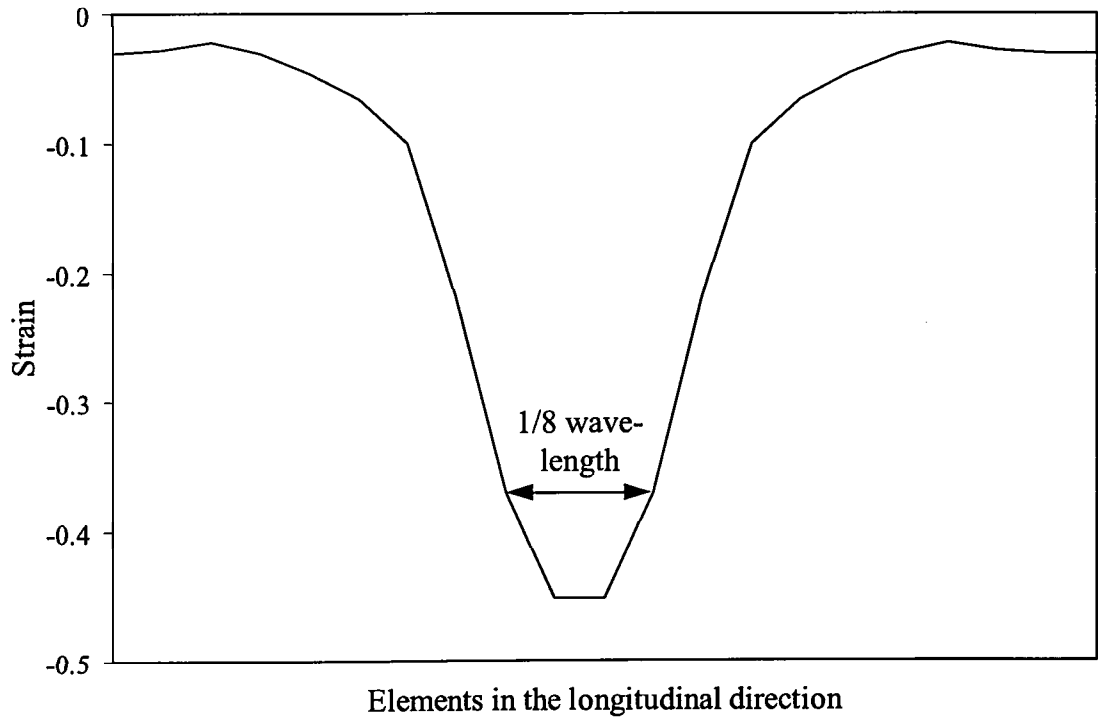


Figure 5.6: *Strain distribution in the longitudinal direction of an element at the corner*

The S4R element has five integration points (by default) through the thickness, from which strain readings may be taken. Strains will vary through the thickness due to local bending. In this study, the points on the outer and inner surfaces of a section are referred to as SPOS and SNEG respectively. Output of strains based on SPOS consistently yields the largest strain amplitudes. However, prediction of fatigue lives from these strain amplitudes alone would overestimate the damage since this maximum value would be assumed constant through the thickness. Conversely, using the average of SPOS and SNEG would underestimate the damage since the effect of local bending would be ignored. Therefore, fatigue lives calculated on the basis of the SPOS strains and on the average of the two surfaces can be assumed to yield lower and upper bounds.

From the combination of areas considered and surfaces from which strain output may be taken, four cases arise: (1) SP-avg (average strains over the corner area based on SPOS), (2) SPN-avg (average strains over the corner area based on $(\text{SPOS} + \text{SNEG})/2$), (3) SP-peak (peak strains from element based on SPOS), and (4) SPN-peak (peak strains from a single element based on $(\text{SPOS} + \text{SNEG})/2$).

5.3 Damage prediction

5.3.1 Overview

The bracing members examined in the current study were subjected to large cyclic displacements, resulting in high localised plastic deformation which would, in reality, lead to fatigue failure. Fatigue due to such a high plastic deformation is considered as low cycle fatigue, or sometimes called ultra low cycle fatigue for failure in less than 20 cycles. However, since fracture was not explicitly modelled within the numerical models, an indirect approach to assess the damage and predict the occurrence of fracture was needed. A strain-based approach was deemed most suitable for this purpose.

The adopted procedure for predicting damage first requires the monitoring of strains during each cycle of loading. A fatigue life is then obtained through the Coffin-Manson (Coffin, 1954; Manson, 1954) relationship, calibrated for the material being studied.

Finally, the damage caused by each cycle is accumulated by means of Miner's rule and fracture is predicted from this damage value. Details of these steps are discussed in the following sub-sections.

5.3.2 Strain-life relationship

It has been observed that plastic strain-life data from low cycle fatigue tests fall approximately on a straight line when plotted on a log-log scale. This observation forms the basis of the Coffin-Manson (Coffin, 1954; Manson, 1954) relationship (Equation 3.1).

Based on constant strain amplitude cyclic material tests at $\pm 1\%$, $\pm 3\%$, $\pm 5\%$ and $\pm 7\%$ total strain amplitudes, the Coffin-Manson relationships for each of hot-rolled carbon steel, cold-formed carbon steel and cold-formed stainless steel have been calibrated and reported in Chapter 3 (Figure 3.24). Strain amplitudes from the critical areas of the models (i.e. the corner regions adjacent to the location of local buckling) can be input into the calibrated Coffin-Manson relationship to give the corresponding fatigue lives.

5.3.3 Variable amplitude loading

The strain-life relationship outlined above applies to constant-amplitude fatigue test data. In the present study, however, strain amplitudes experienced by the members varied from cycle to cycle due to the variable displacement history and changes in the buckling shape. Therefore, a fatigue life corresponding to the strain amplitude in each cycle was calculated and the combined effects of these cycles were assessed through a cumulative damage rule. A conventional form of cumulative damage model is the linear Miner's rule (Miner, 1945):

$$D = \sum_{i=1}^n \frac{n_i}{N_{fi}} \quad (5.4)$$

where N_{fi} is the number of cycles to failure at a strain (or stress) level of ϵ_i (or σ_i) and n_i is the number of cycles for which that particular strain (or stress) level is applied.

Fatigue occurs when the damage index, D , reaches unity. Equation 5.4 was adopted in this study.

5.4 *Verification of FE models*

Prior to performing parametric studies described in Chapter 6, the FE models were verified against cyclic axial loading tests on hot-rolled carbon steel, cold-formed carbon steel and cold-formed stainless steel hollow sections reported in Chapter 4. A number of other available experiments performed on members in a similar configuration were also considered (Walpole, 1996; Goggins et al., 2006). Among the tests of Walpole (1996), only the one carried out with fixed end conditions was employed for verification (150×100×6×2704-CS-CF). Just two tests reported by Goggins et al. (2006) were used in the verification since in other tests either failure was not reached or fracture occurred close to the weld, examination of which is outside the scope of this study.

Complete fracture of a specimen is a common definition of failure but is not always appropriate for signalling significant loss of resistance of a member. Once an opening is formed, typically in the corner region of a section, it grows quickly and the reduction in cross-sectional area weakens the specimen. Some specimens survived a number of additional cycles following corner opening by relying on a small fraction of the cross-section remaining intact; however, in practice, such a bracing member will no longer be capable of providing effective lateral stiffness to the frame. Therefore, in this study, the number of cycles to failure has been defined as the cycle when small cracks at the corner grow into an opening, which can often be indentified in a load-displacement hysteresis curve by a sudden drop in load. Hysteresis curves from selected FE simulations and corresponding tests on specimens of the three materials are shown in Figures 5.7-5.22. The comparison shows that the FE models are capable of simulating the hysteretic behaviour well for all three materials. However, the FE models occasionally underestimate the compressive resistance and overestimate the tensile tangent stiffness of the test specimens. Also, since the initial elastic stiffness is assumed in the FE models for elastic unloading, the models give steeper unloading curves than test specimens in all cases. This discrepancy is particularly large for unloading after

tensile displacement. In the simulation of some tests, such as the one shown in Figure 5.8, the damage assessment procedure has predicted fewer of cycles to failure than the test results, which leads to fewer hysteretic loops shown in the figures. If the failure criteria are changed, such as using a different strain-life relationship, the FE models have demonstrated the capability to simulate the remaining cycles of hysteretic behaviour accurately.

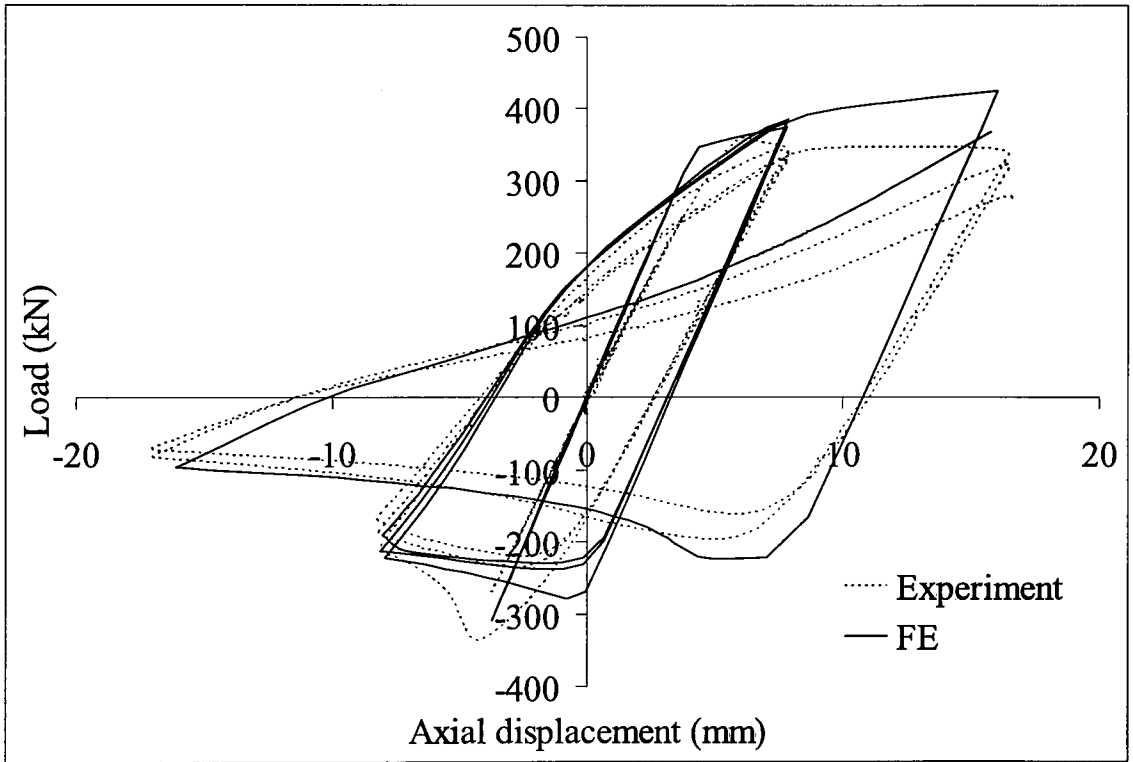


Figure 5.7: Comparison of hysteretic curves for specimen $60 \times 60 \times 3 \times 2050$ -CS-HR from FE simulation and experiment

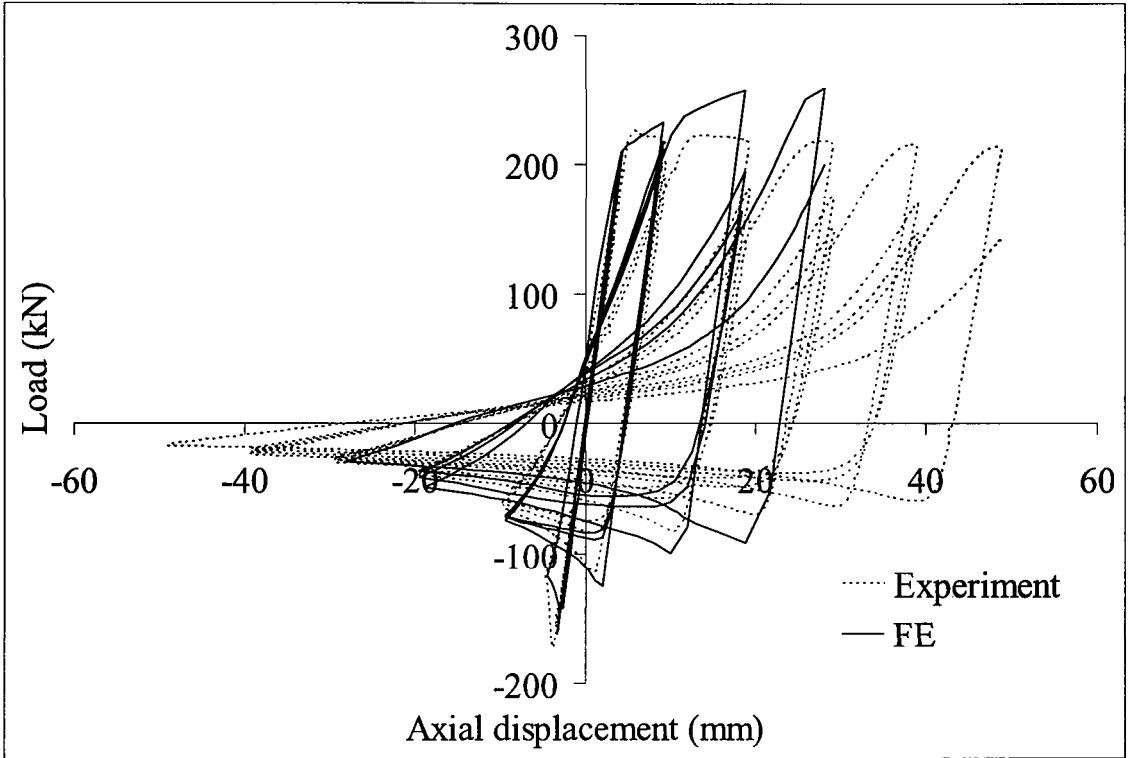


Figure 5.8: Comparison of hysteretic curves for specimen 40×40×3×2050-CS-HR from FE simulation and experiment

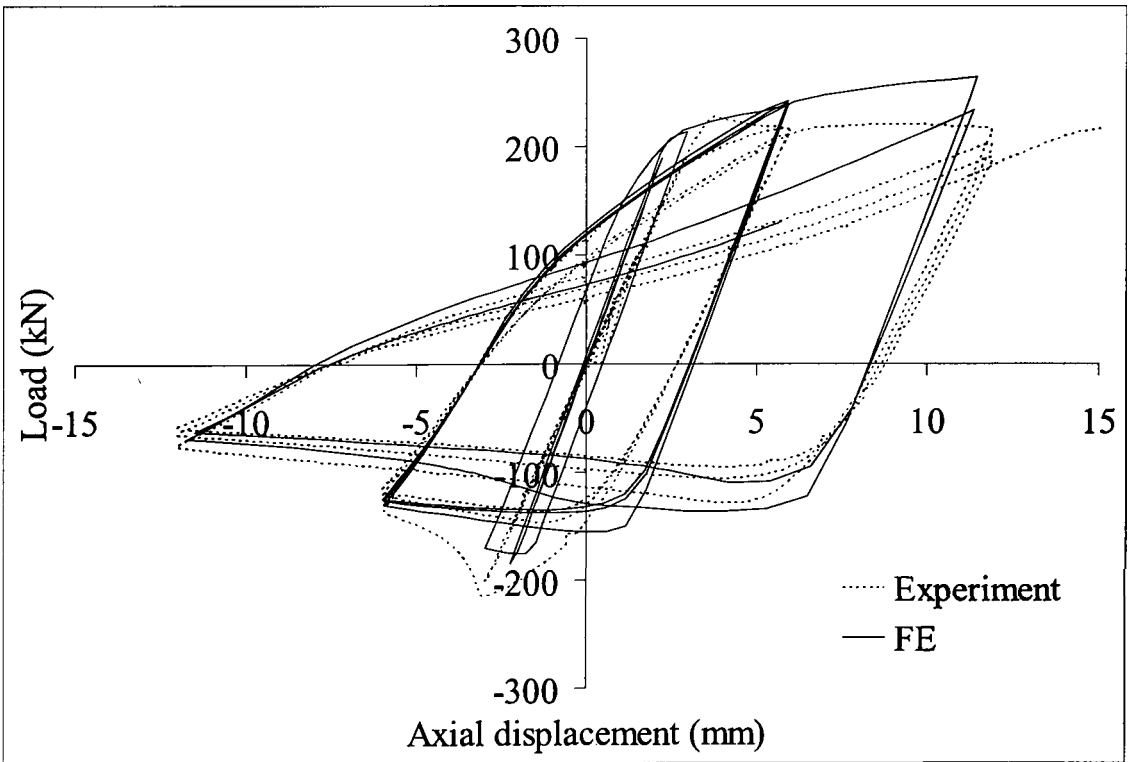


Figure 5.9: Comparison of hysteretic curves for specimen 40×40×3×1250-CS-HR from FE simulation and experiment

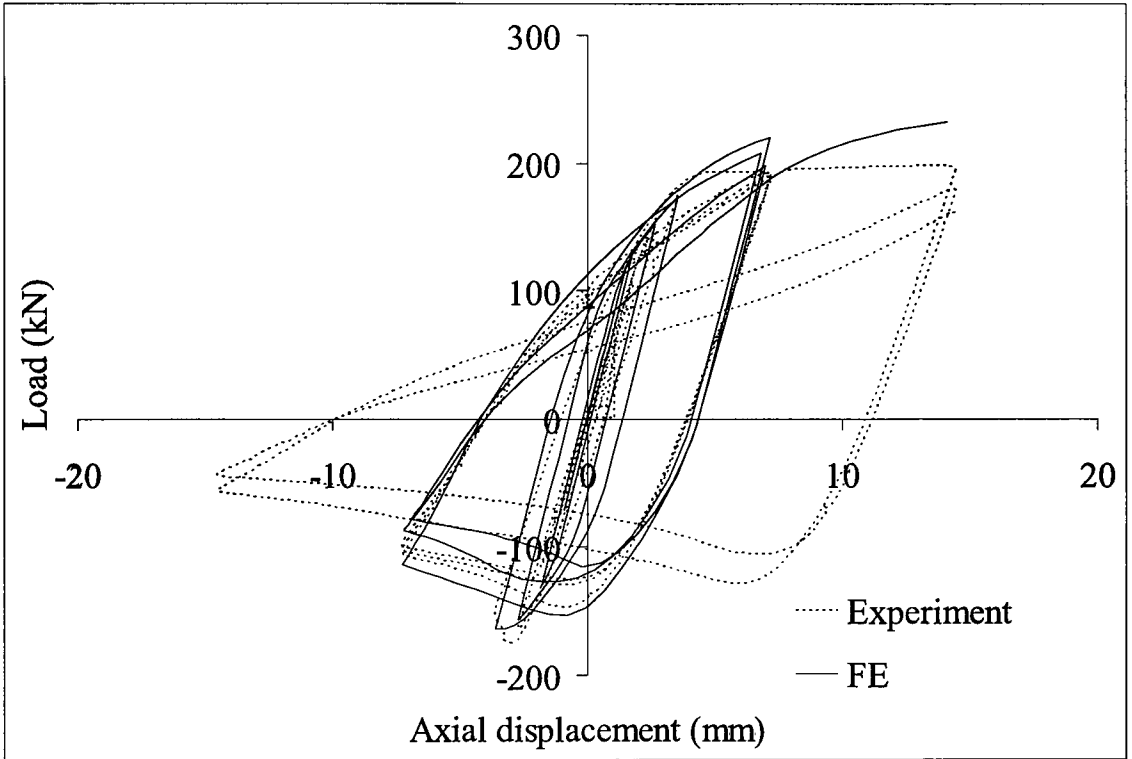


Figure 5.10: Comparison of hysteretic curves for specimen $60 \times 60 \times 3 \times 2050$ -CS-CF from FE simulation and experiment

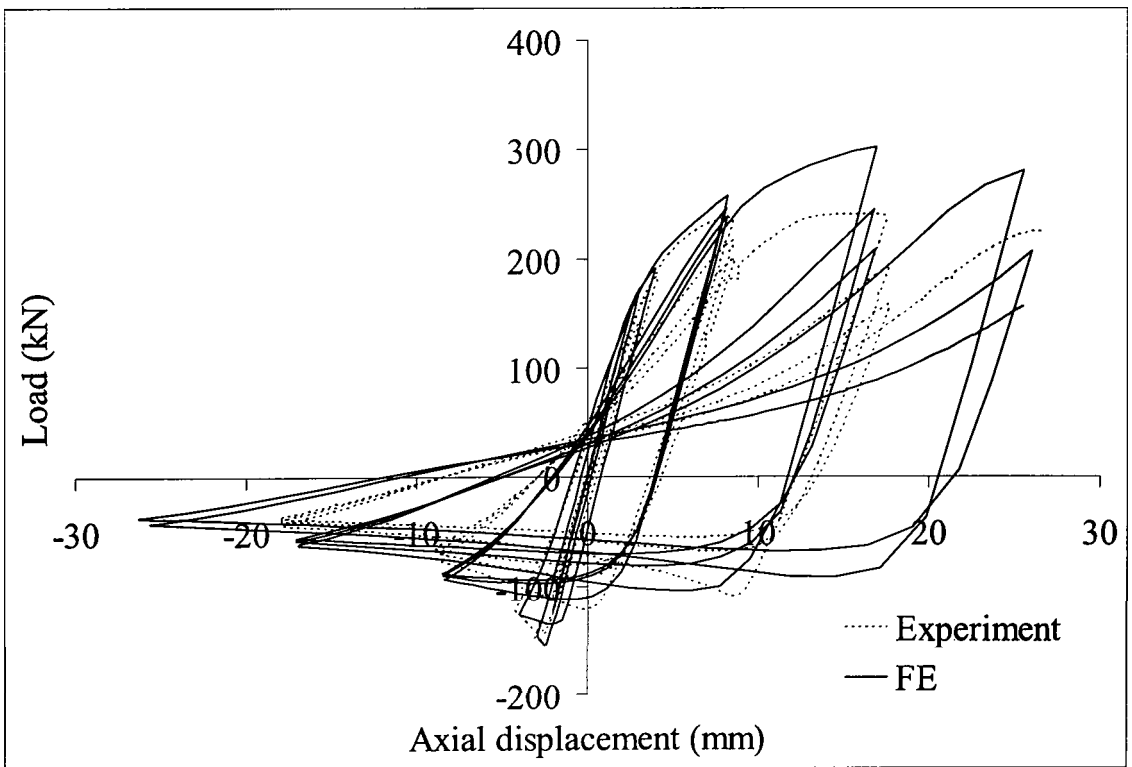


Figure 5.11: Comparison of hysteretic curves for specimen $40 \times 40 \times 4 \times 2050$ -CS-CF from FE simulation and experiment

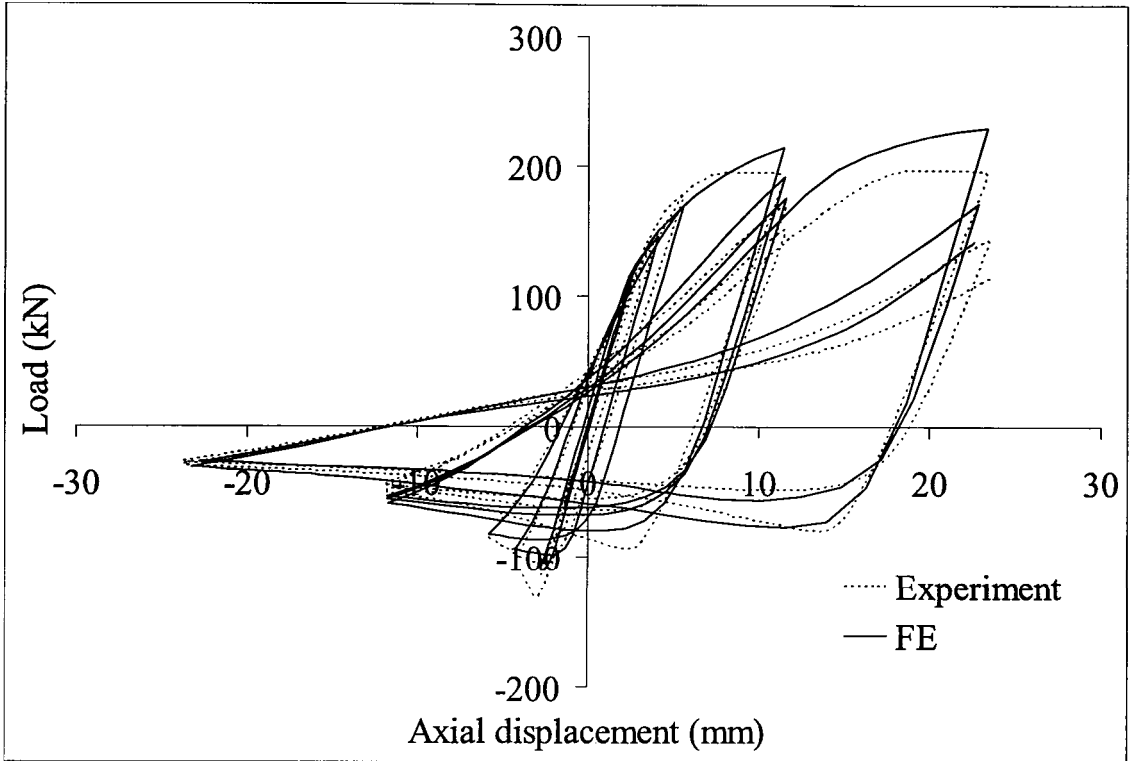


Figure 5.12: Comparison of hysteretic curves for specimen $40 \times 40 \times 3 \times 2050$ -CS-CF from FE simulation and experiment

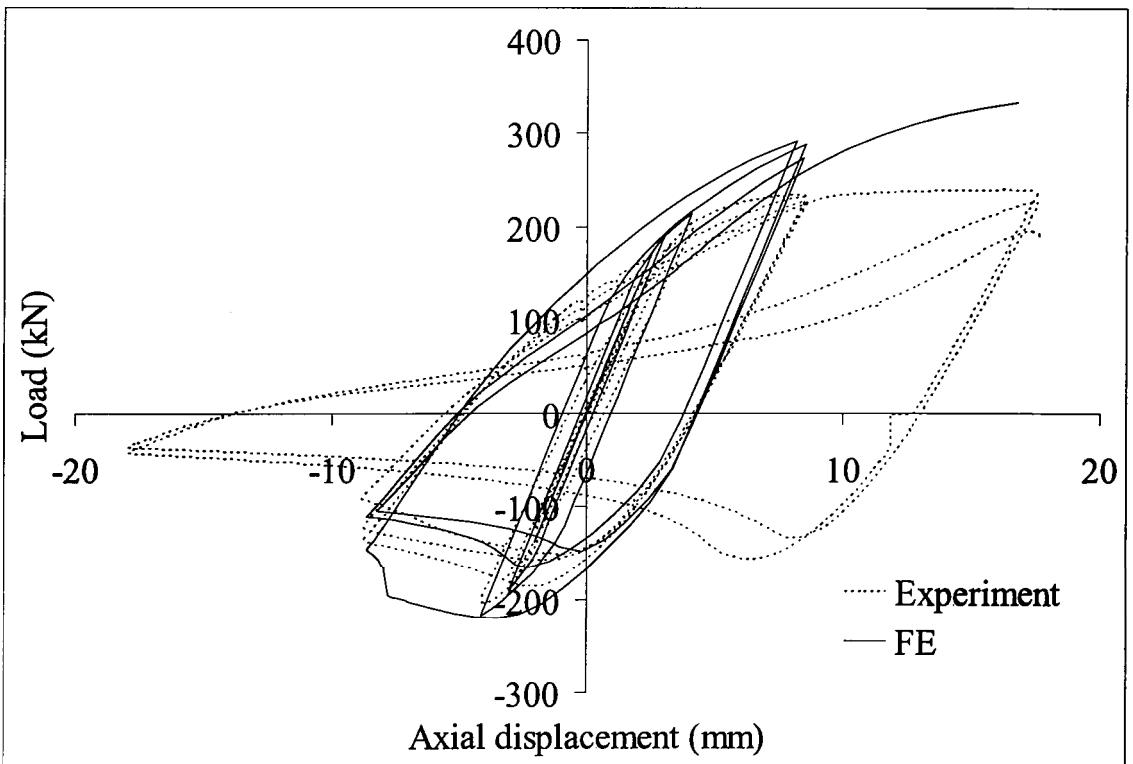


Figure 5.13: Comparison of hysteretic curves for specimen $40 \times 40 \times 3 \times 1250$ -CS-CF from FE simulation and experiment

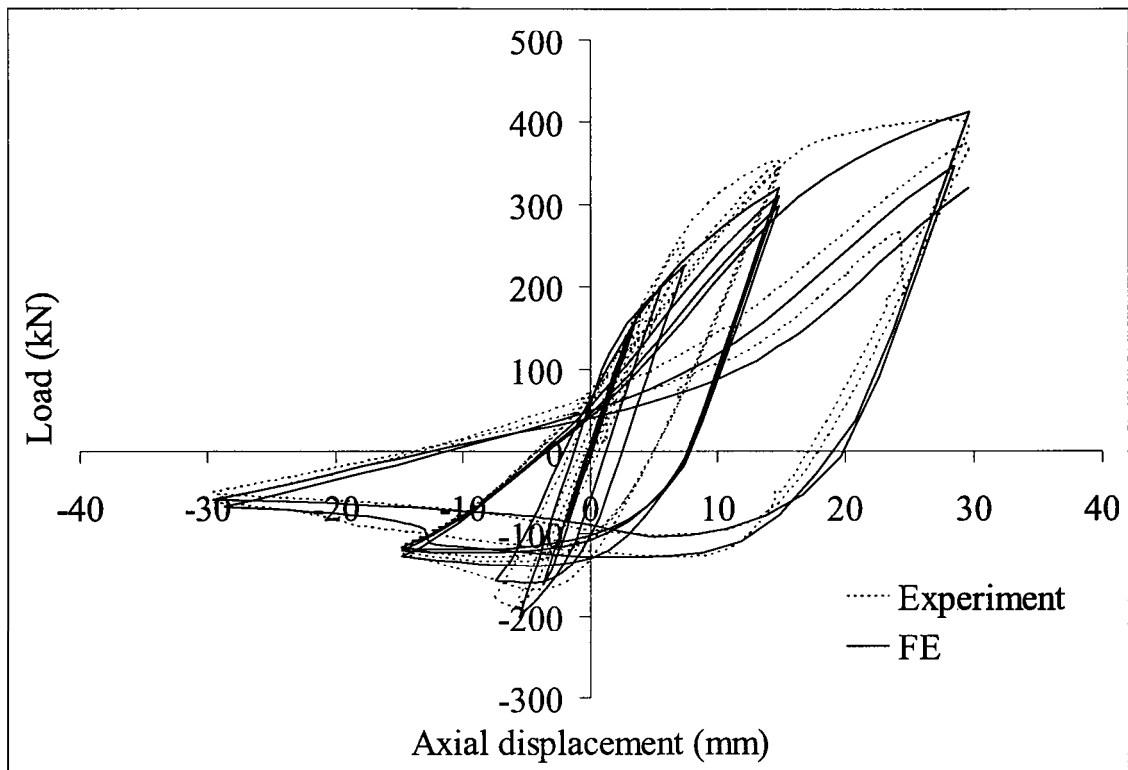


Figure 5.14: Comparison of hysteretic curves for specimen $60 \times 60 \times 3 \times 2850$ -SS-CF from FE simulation and experiment

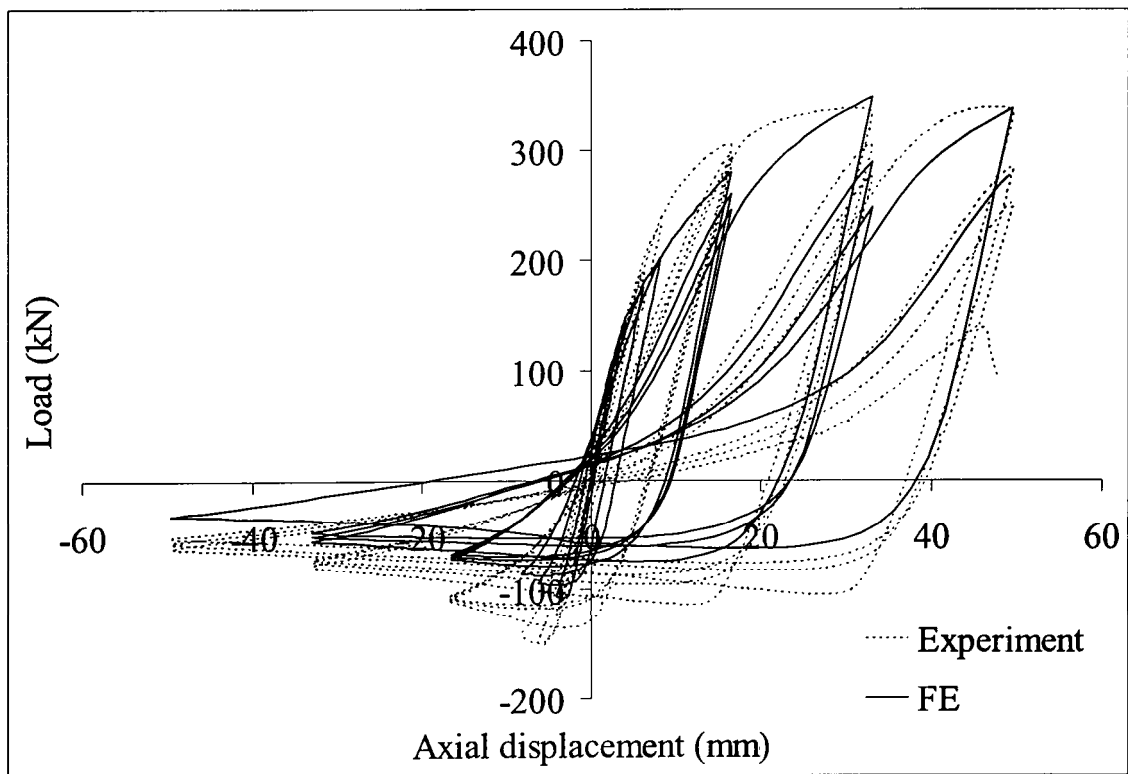


Figure 5.15: Comparison of hysteretic curves for specimen $50 \times 50 \times 3 \times 2850$ -SS-CF from FE simulation and experiment

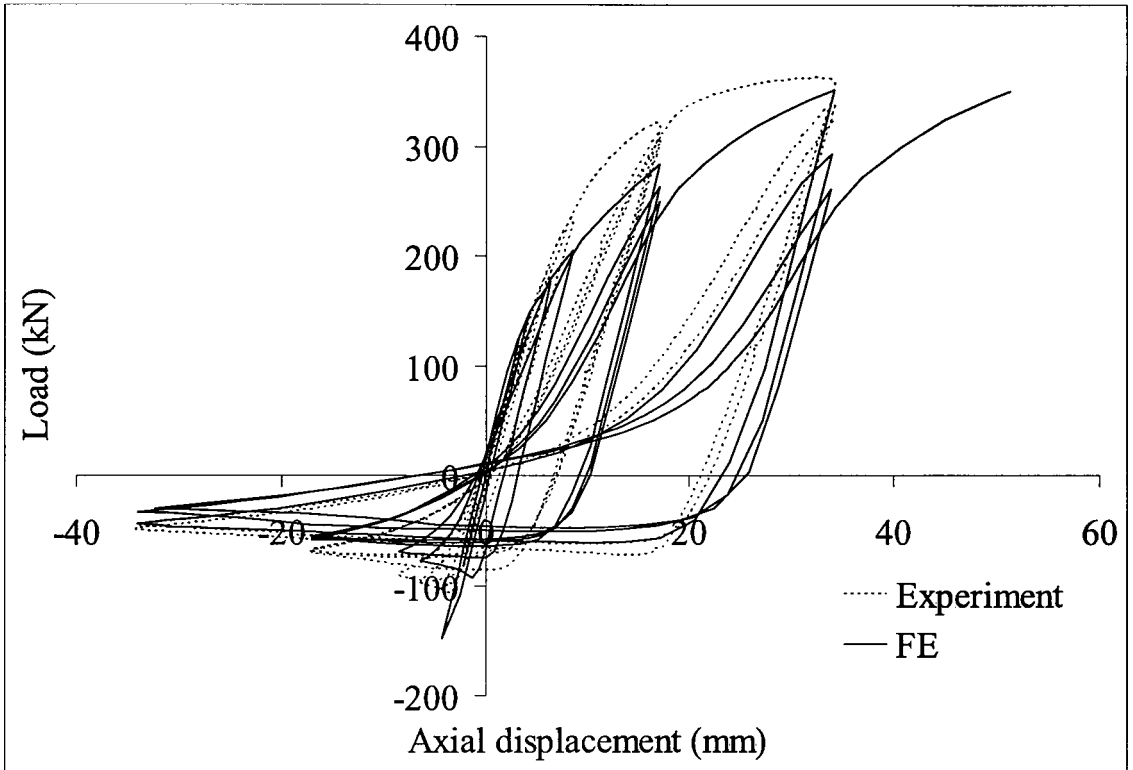


Figure 5.16: Comparison of hysteretic curves for specimen $60 \times 40 \times 3 \times 2850$ -SS-CF from FE simulation and experiment

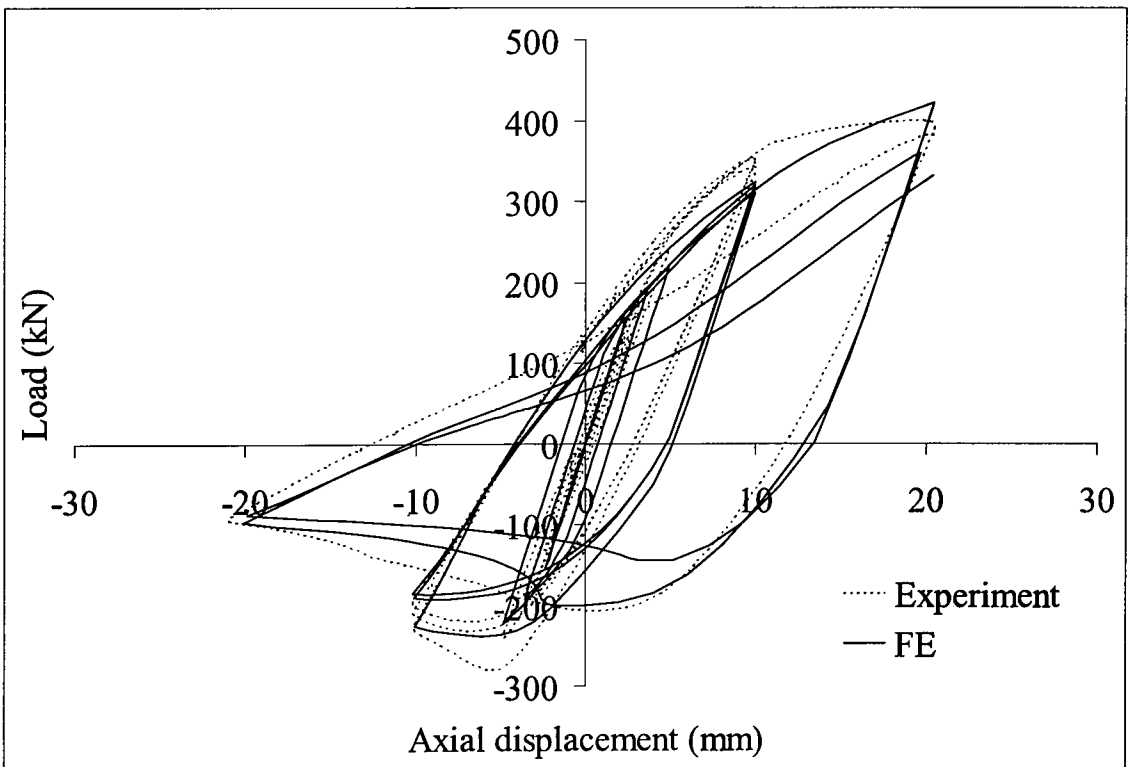


Figure 5.17: Comparison of hysteretic curves for specimen $60 \times 60 \times 3 \times 2050$ -SS-CF from FE simulation and experiment

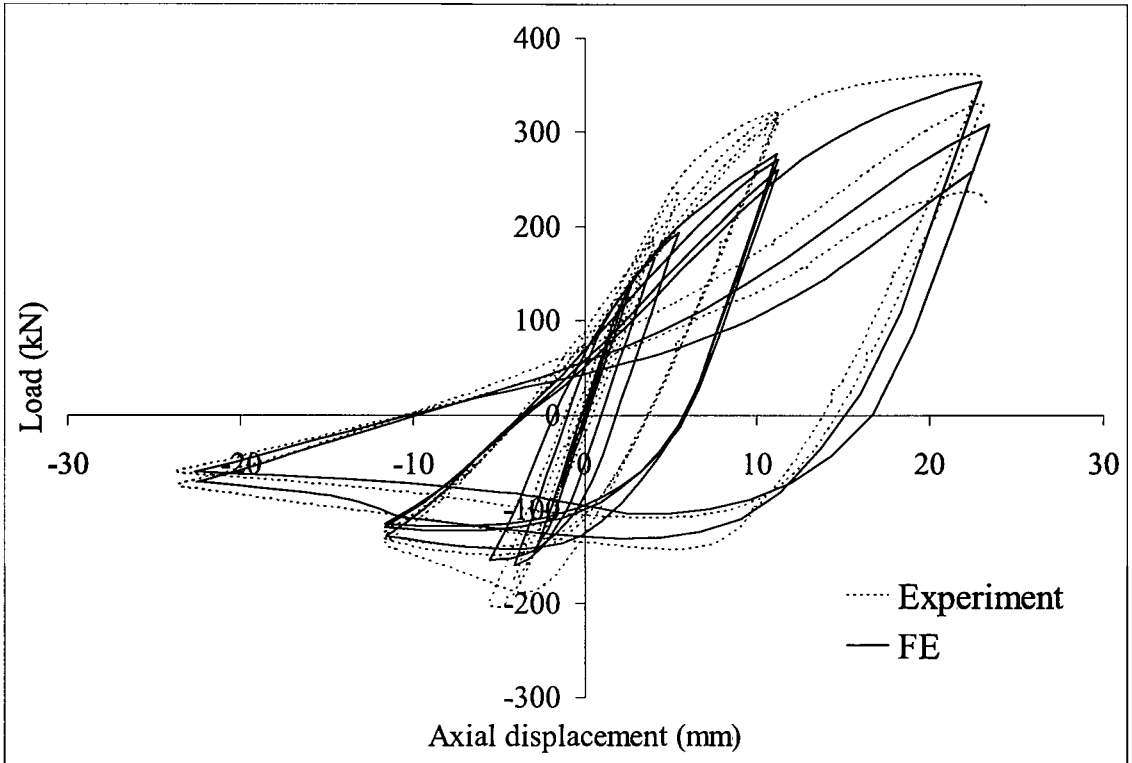


Figure 5.18: Comparison of hysteretic curves for specimen $50 \times 50 \times 3 \times 2050$ -SS-CF from FE simulation and experiment

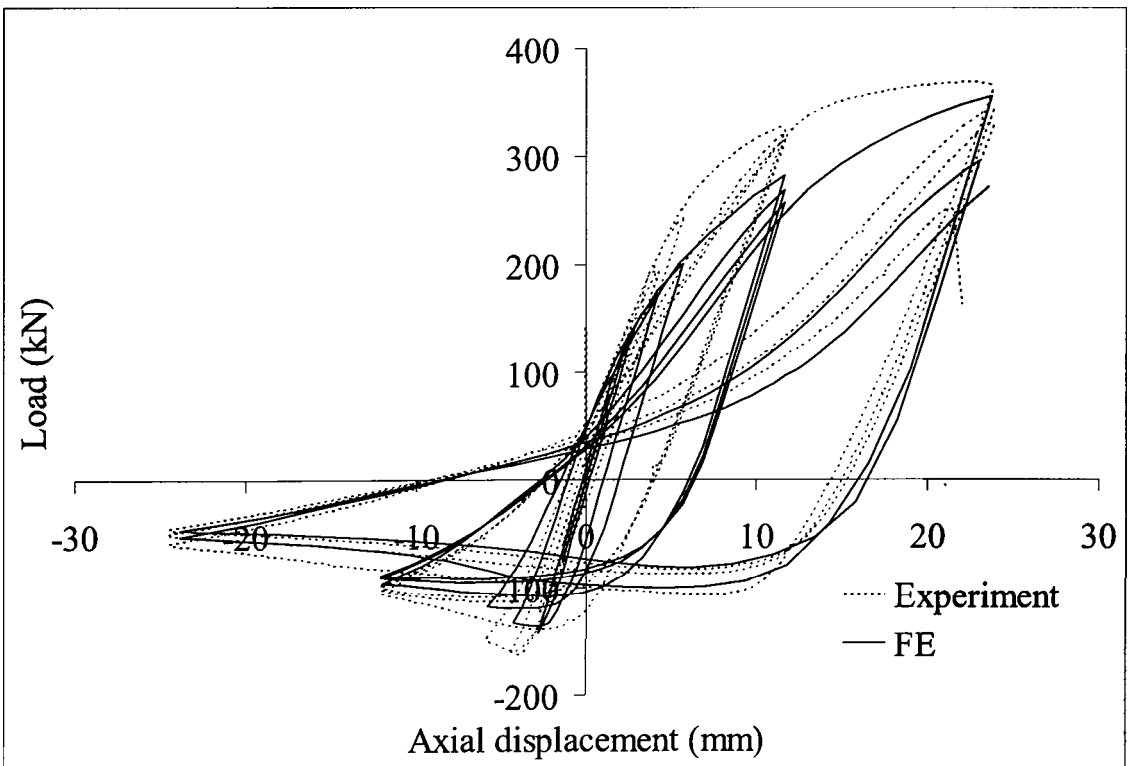


Figure 5.19: Comparison of hysteretic curves for specimen $60 \times 40 \times 3 \times 2050$ -SS-CF from FE simulation and experiment

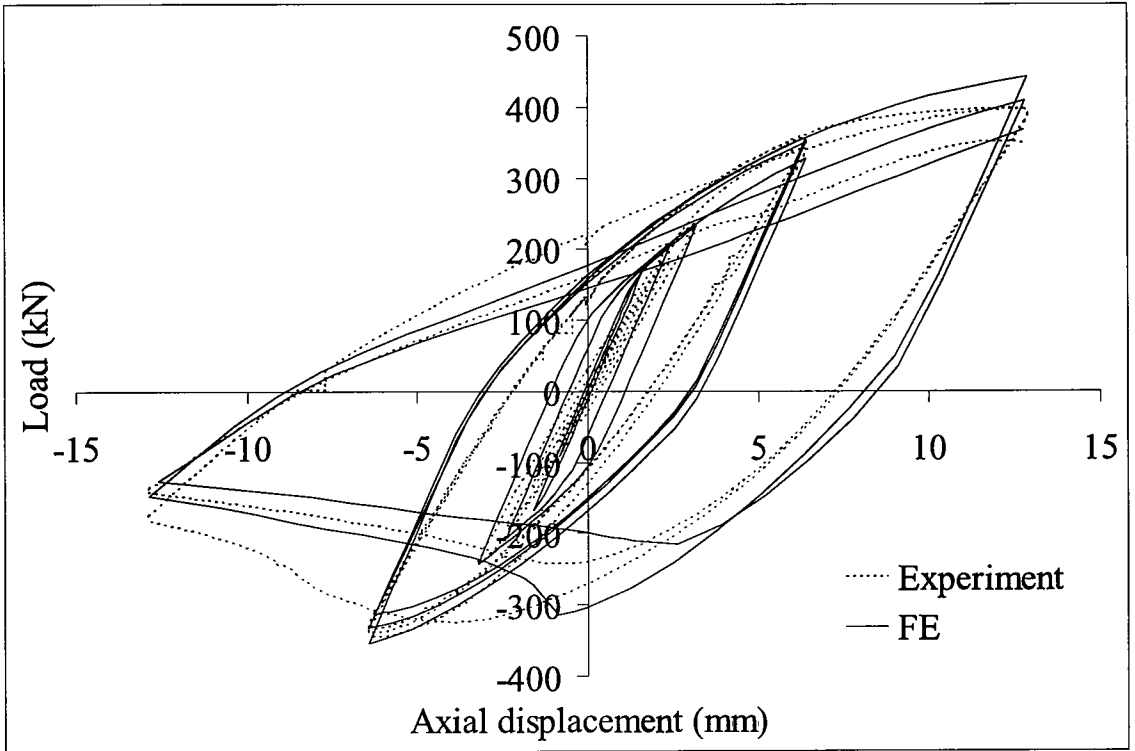


Figure 5.20: Comparison of hysteretic curves for specimen $50 \times 50 \times 3 \times 1250$ -SS-CF from FE simulation and experiment

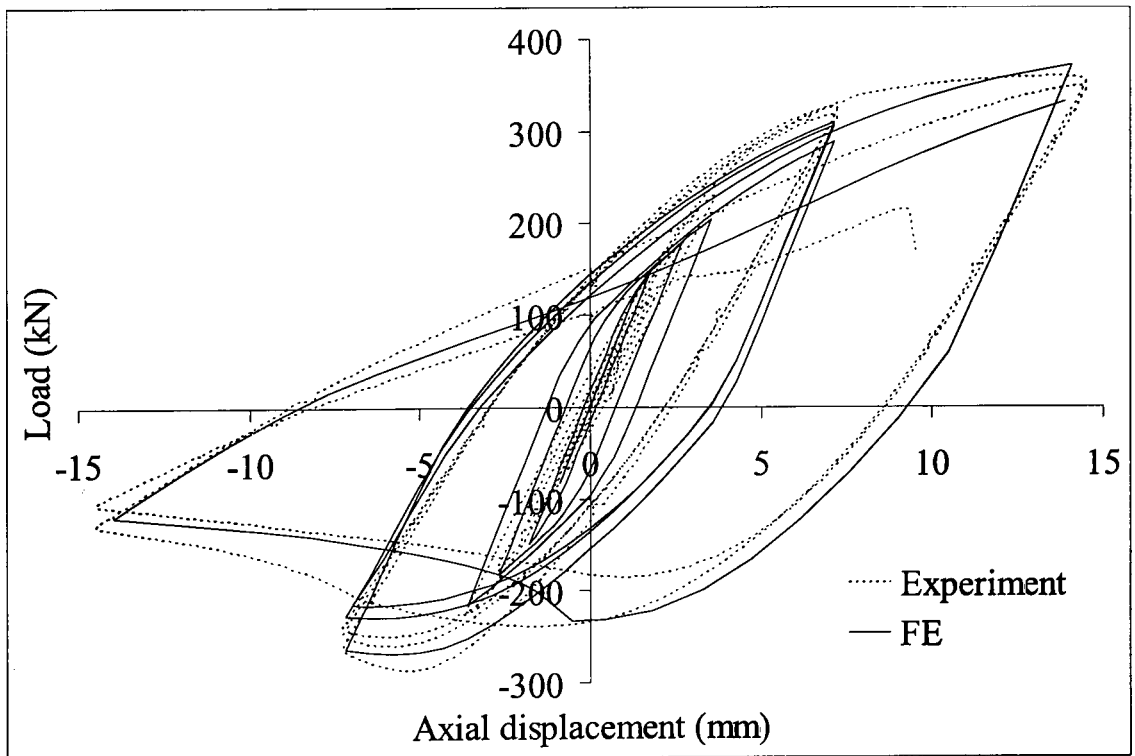


Figure 5.21: Comparison of hysteretic curves for specimen $50 \times 50 \times 3 \times 1250$ -SS-CF from FE simulation and experiment

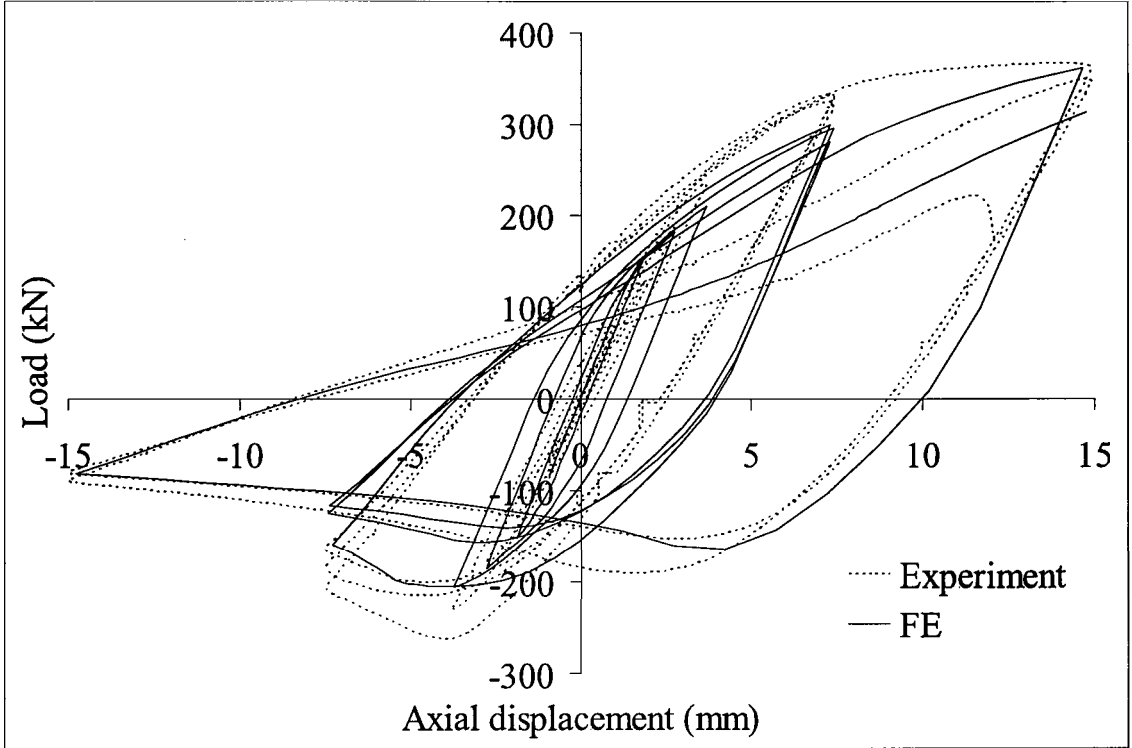


Figure 5.22: Comparison of hysteretic curves for specimen $60 \times 40 \times 3 \times 1250$ -SS-CF from FE simulation and experiment

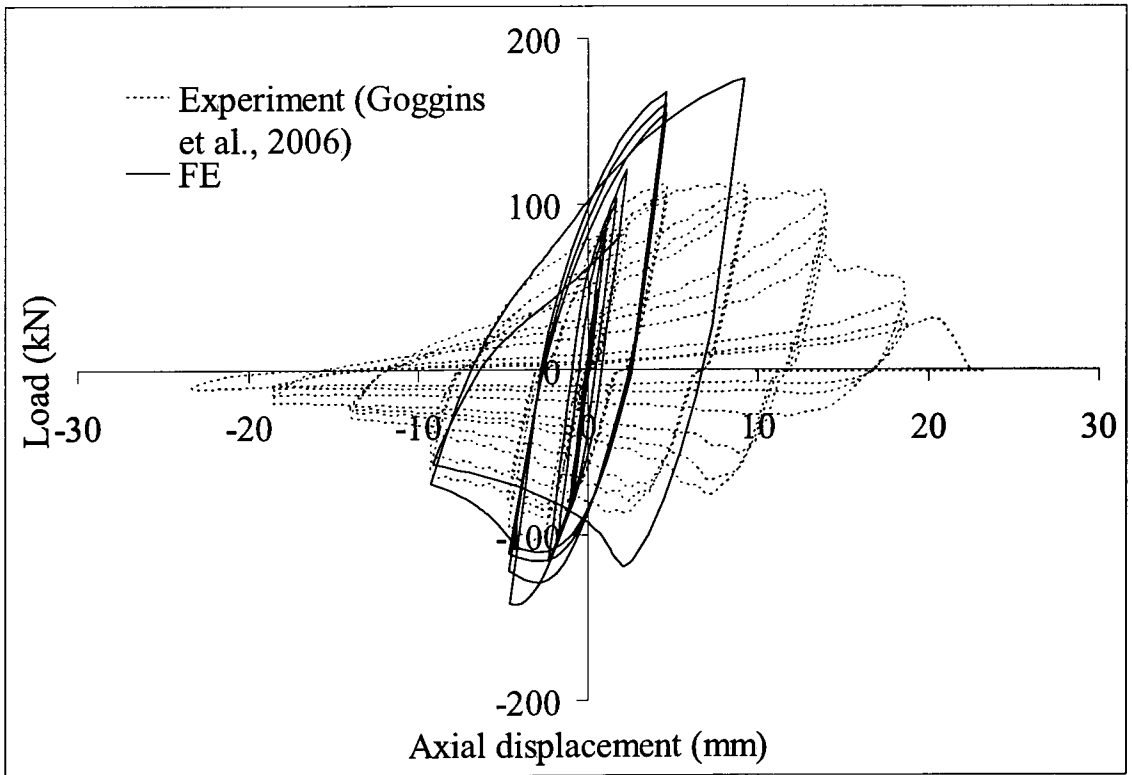


Figure 5.23: Comparison of hysteretic curves for specimen $40 \times 40 \times 2.5 \times 1150$ -CS-CF from FE simulation and experiment (Goggins et al., 2006)

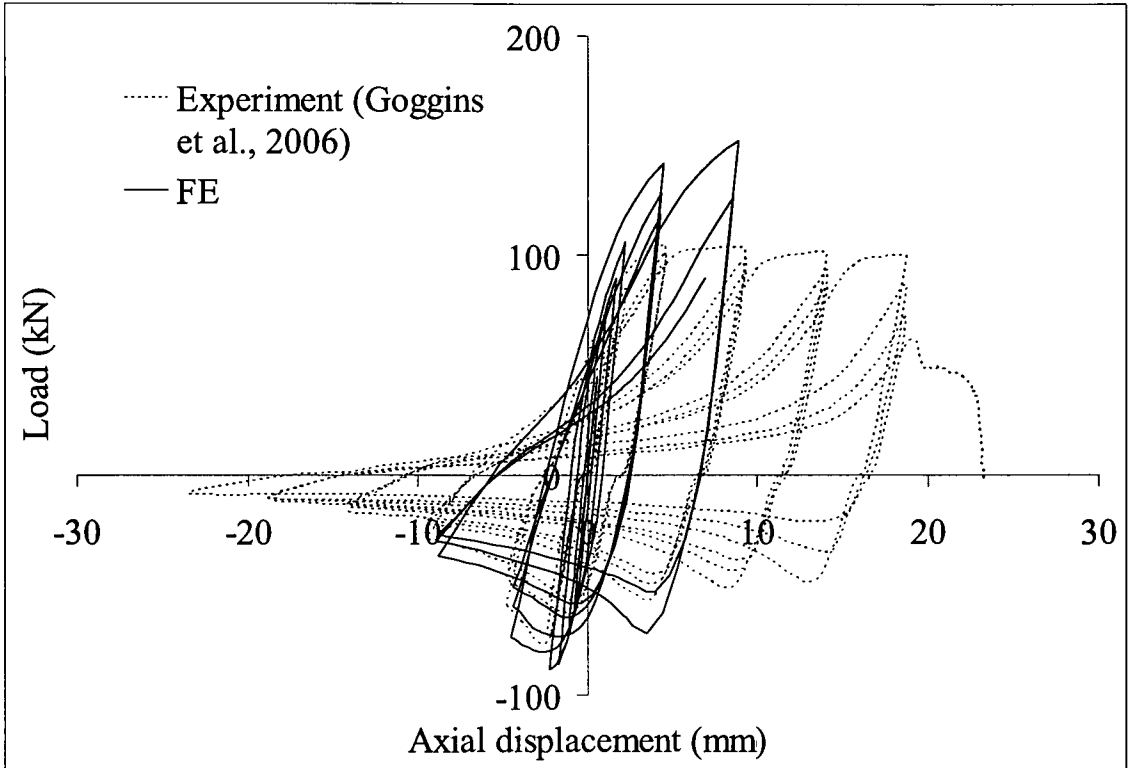


Figure 5.24: Comparison of hysteretic curves for specimen $50 \times 25 \times 2.5 \times 1150$ -CS-CF from FE simulation and experiment (Goggins et al., 2006)

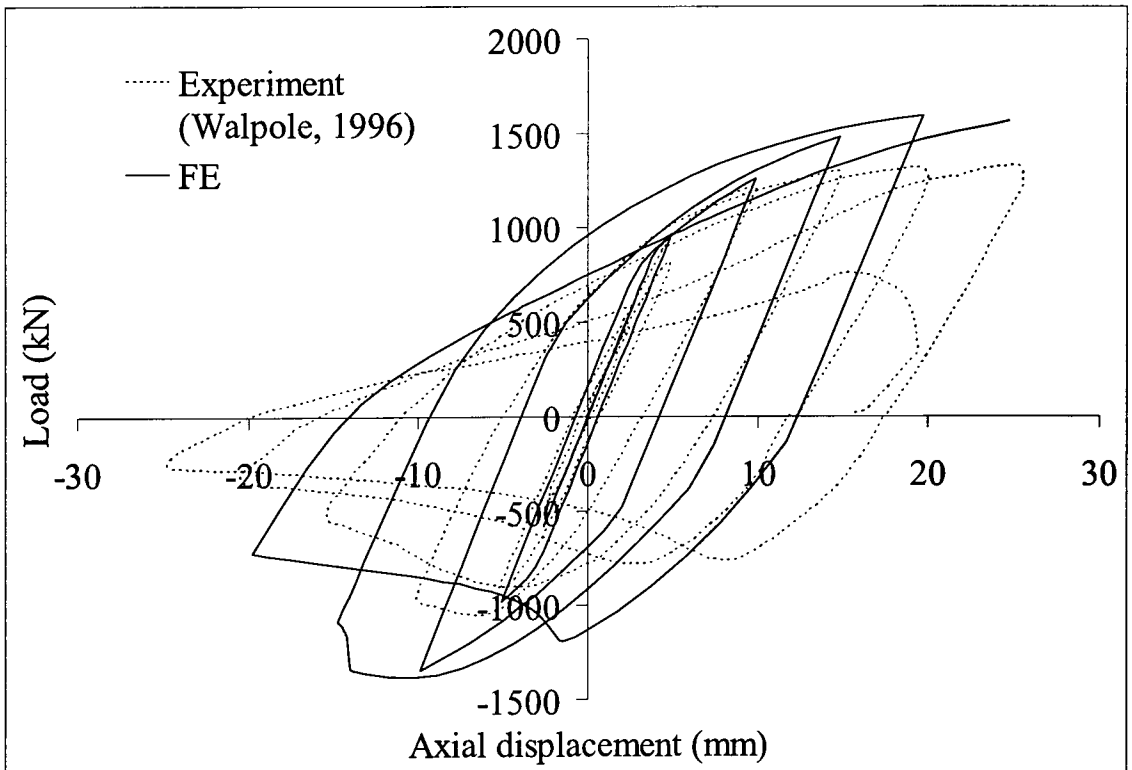


Figure 5.25: Comparison of hysteretic curves for specimen $150 \times 100 \times 6 \times 2704$ -CS-CF from FE simulation and experiment (Walpole, 1996)

A crucial criterion for accurate prediction of fracture is reproducing the strain distribution correctly in the FE models. The FE and experimental strain measurements at various stages of the cyclic axial test on specimen $60 \times 60 \times 3 \times 2850$ -SS-CF are presented in Figures 5.26-5.28. The FE strain output has been extracted from the same locations as the strain gauges (Figure 4.2). The FE models capture the variation of strain across the section accurately and are capable of predicting the high strains at the corner areas which eventually lead to fracture. The only significant discrepancy between test measurements and FE arises at the corner areas after local buckling has occurred (Figure 5.28). This discrepancy is believed to be due to the fact that the actual deformations in the tests have exceeded the working range of the strain gauges and they are thus unreliable.

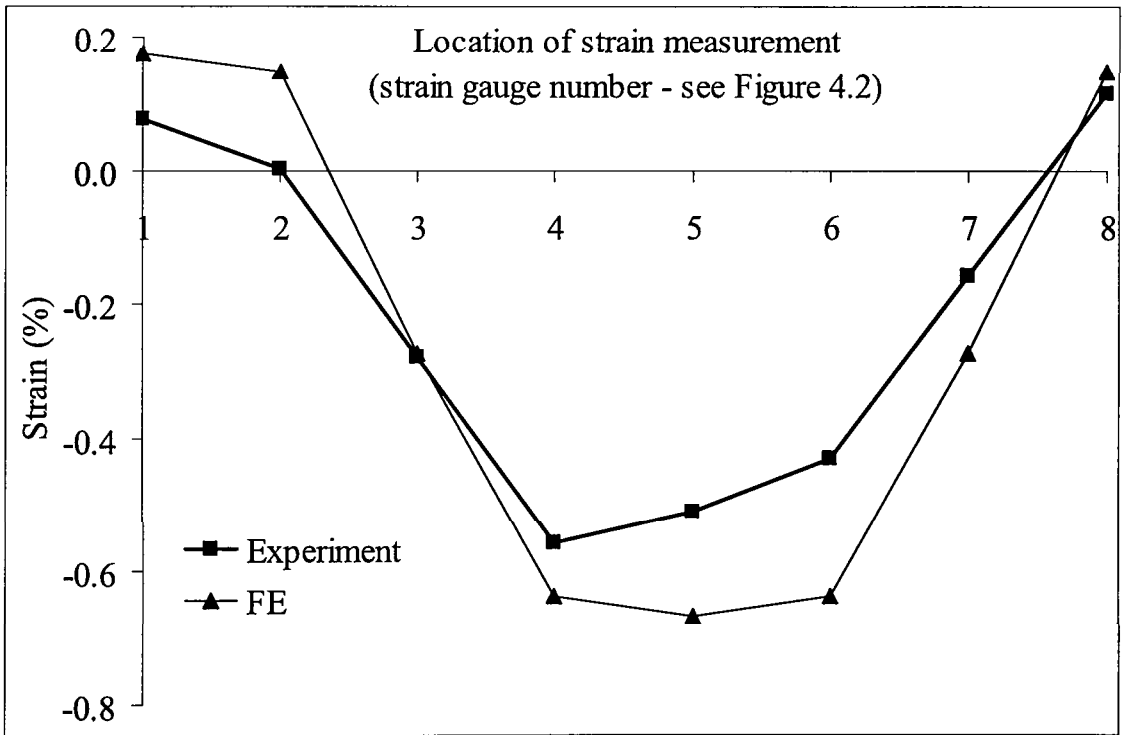


Figure 5.26: Comparison of strain measurements at mid-length at the maximum compressive displacement in the 4th cycle for specimen $60 \times 60 \times 3 \times 2850$ -SS-CF from FE simulation and experiment

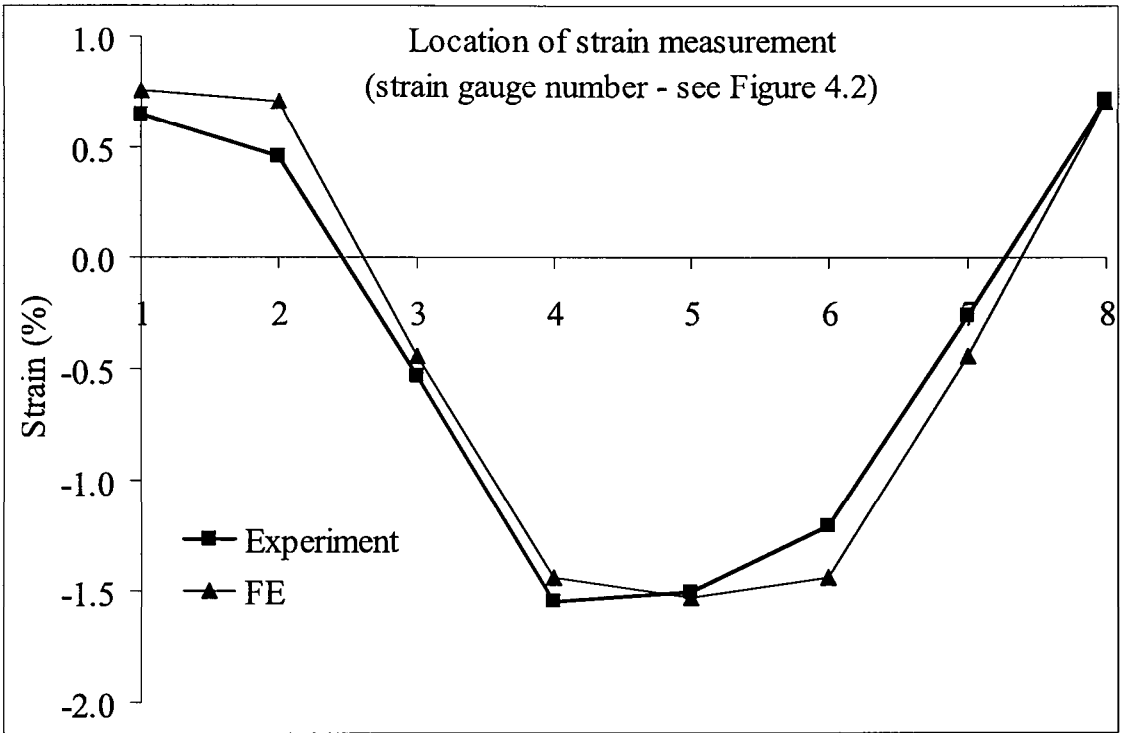


Figure 5.27: Comparison of strain measurements at mid-length at the maximum compressive displacement in the 6th cycle for specimen 60×60×3×2850-SS-CF from FE simulation and experiment

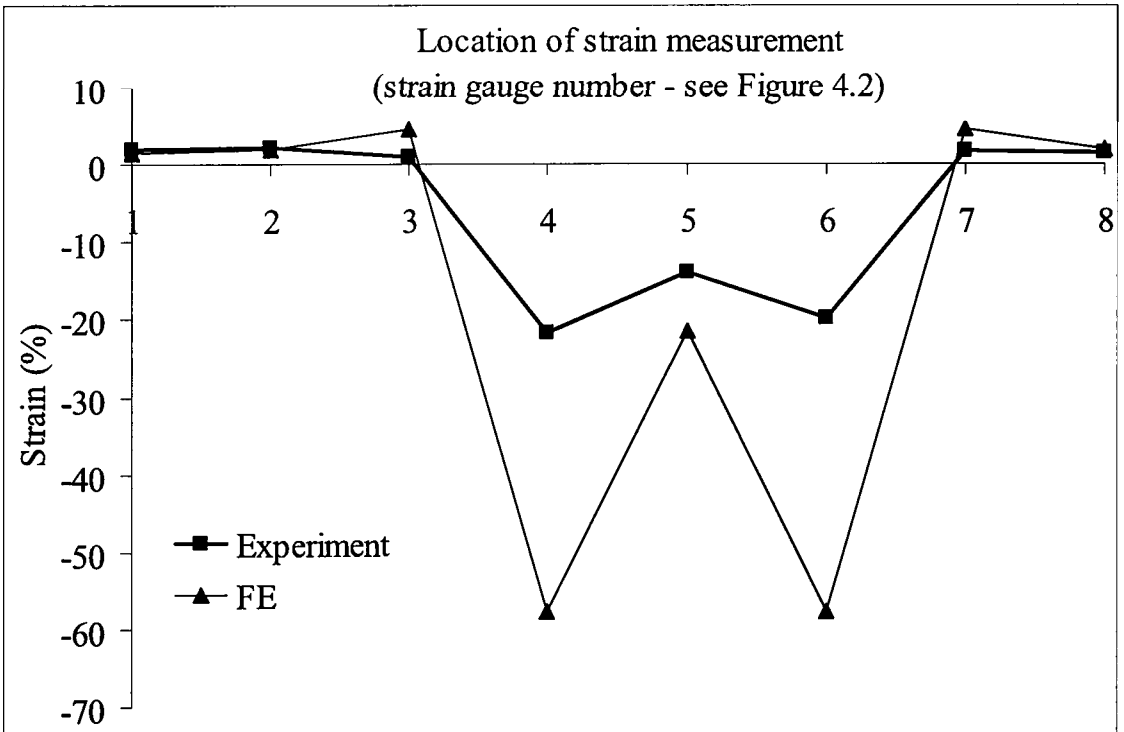


Figure 5.28: Comparison of strain measurements at mid-length at the maximum compressive displacement in the 8th cycle for specimen 60×60×3×2850-SS-CF from FE simulation and experiment

A comparison between the number of cycles to failure and predictions from the FE models is presented in Table 5.2. The agreement between the test and FE results is good with only a few exceptions which may be due to premature cracking in the weld regions, which was evident for 6 specimens (see Table 5.2). These cracks reduce the rotational stiffness at the ends of the specimens and increase their effective length, which lessens the rotation at the mid-length. The discrepancy between the results of Goggins et al. (2006) and the described numerical models may be due to the limited information on the material properties.

Table 5.2: *Verification of FE models*

Specimen identification	Number of cycles to			
	local buckling		fracture	
	Test	FE	Test	FE
60×60×3×2050-CS-HR	7	8	10	9
40×40×3×2050-CS-HR**	15	9	18	12
40×40×3×1250-CS-HR	8	8	11	10
60×60×3×2050-CS-CF	7	5	10	8
40×40×4×2050-CS-CF*	-	11	11	13
40×40×3×2050-CS-CF*	5	8	10	10
40×40×3×1250-CS-CF**	8	6	10	8
60×60×3×2850-SS-CF	8	8	10	10
50×50×3×2850-SS-CF	12	10	14	12
60×40×3×2850-SS-CF*	-	9	10	11
60×60×3×2050-SS-CF**	8	8	9	9
50×50×3×2050-SS-CF	8	8	10	10
60×40×3×2050-SS-CF	8	8	11	10
60×60×3×1250-SS-CF	8	8	10	10
50×50×3×1250-SS-CF	8	8	10	9
60×40×3×1250-SS-CF	6	5	10	9
40×40×2.5×1100-CS-CF***	-	8	14	9
50×25×2.5×1100-CS-CF***	-	7	17	9
150×100×6×2704-CS-CF***	-	5	7	6

* Fracture at annealed material close to the weld.

** Small cracks at annealed material close to the weld but fracture at mid-length.

*** Tests results from Goggins et al. (2006) and Walpole (1996); number of cycles at which local buckling occurred was not reported.

5.5 Concluding remarks

In this chapter, FE models were developed to investigate the cyclic response of hollow section braces and a strain-based damage assessment method was adopted to predict the fracture lives of the members, both of which were verified against experimental results. The numerical predictions of the load-displacement hysteretic response, strain distribution across the section, and number of cycles to local buckling and fracture were found to be satisfactory.

Chapter 6

Parametric Studies

6.1 Introduction

Satisfactory behaviour of structures under severe seismic loading is usually largely dependent on the ability of key components to undergo significant inelastic deformations. In the case of concentrically braced frames, the critical elements are the diagonal bracing members which are expected to experience repeated cycles involving yielding in tension and member buckling in compression (Elghazouli, 2003). The performance of bracing members depends on various factors, including local slenderness, global slenderness, material yield strength, section shape and end restraint. Due to the difficulty in modelling the non-linearity and cyclic plasticity accurately, numerous experimental studies have been carried out to study the cyclic inelastic behaviour of bracing members.

The interest of researchers in the early days was primarily in the load-displacement hysteretic response of the braces. Models were proposed to predict residual elongation at zero load, loss of compressive strength, the area under the hysteresis loops which represents the amount of energy dissipation, and other key characteristics of the hysteresis loops (Kahn & Hanson, 1976; Popov et al., 1979; Jain et al., 1980; Popov & Black, 1981). It was generally concluded (Jain et al., 1980; Popov & Black, 1981) that global slenderness was the most important parameter influencing the hysteretic behaviour of braces. Slender members lost compressive resistance more rapidly than

stocky members, resulting in fewer inelastic response cycles and lower amount of energy dissipation.

More recently, attention has shifted to examination of the factors influencing the fracture life of bracing members. Through experimental testing, both global and local slenderness were found to be important factors in determining fracture life. Tang and Goel (1989) proposed one of the first empirical equations for predicting the fracture life of bracing members, which suggests that fracture life is proportional to both the aspect ratio of the cross-section and the global member slenderness but inversely proportional to the square of the local slenderness. However, the validity of this prediction method is limited to bracing members in inverted V braced frames. Further developments (Tremblay, 2002; Shaback & Brown, 2003; Goggins et al., 2006) in the prediction of fracture life of brace members have utilised this basic proposed equation and generalised the applicability to bracing members in other concentrically braced frame configurations.

A more general relationship was established following a comprehensive survey of the experimental cyclic behaviour of steel bracing members conducted by Tremblay (2002), in which buckling resistance, post-buckling resistance in compression, tensile resistance, fracture life and a number of other properties from about 50 members were assessed. Shaback and Brown (2003) carried out tests on square hollow section bracing members and calibrated a more sophisticated expression of fracture life, defined as the weighted sum of normalised compressive and tensile deformation, in terms of global slenderness, local slenderness, aspect ratio of the cross-section and material yield strength. A similar experiment programme was also conducted by Goggins et al. (2006) and examined the influence of several of these parameters for square and rectangular tubular members. Another measure of the ductility of a member – the displacement ductility capacity (μ_{Δ}) – defined as the maximum displacement normalised by yield displacement, was expressed in terms of material yield strength, global slenderness and local slenderness respectively in three empirical relationships.

Previous experimental studies and the semi-empirical rules arising from these investigations suggest that fracture lives of bracing members depend to a varying

degree on global slenderness, local slenderness, cross-section proportions and material strength. Despite the availability of a number of models for predicting the ductility capacity of braces, there are significant uncertainties in these predictions owing to their semi-empirical nature as well as the scatter of test results. More importantly, due to physical limitations in laboratory testing, experimental studies cover a limited range of member sizes and lengths, and the influences of local and global slenderness are often difficult to quantify in isolation.

In addition, the introduction of new structural stainless steel design guidance and extensive research on structural members of the material cast light on the possibility of application to seismic resistance structures. Therefore, utilising the FE models and damage assessment method developed in Chapter 5, a number of parametric investigations was conducted to assess the influence of key parameters including the relative importance of local and global slenderness, and compare the performance of the bracing members of carbon steel and stainless steel.

6.2 *Investigation on the influence of slenderness parameters*

The primary aim of the first part of the parametric studies was to investigate the influence of global and local slendernesses on the structural behaviour of hollow section bracing members, examining, in particular, the number of cycles to failure. The global slenderness is defined as $\bar{\lambda} = \sqrt{f_y A / N_{cr}}$ where f_y is the yield strength, A is the cross-sectional area, and N_{cr} is the elastic critical buckling load. The local slenderness is defined as $b/t\epsilon$ where b is the flat width of the wider face of the section, t is the thickness of the section and $\epsilon = \sqrt{235/f_y}$.

There are two series of models in the parametric studies, which were all based on the cold-formed carbon steel material model (yield strengths of flat and corner materials are 440 N/mm² and 491 N/mm² respectively, which results in an average yield strength of around 460 N/mm² for the cross-section). Series 1 consisted of models with the same local slenderness but differing global slendernesses. Their results show the influence of global slenderness on the fracture lives of the members. By changing both the

thicknesses of the sections and the lengths of the members but keeping the section sizes constant, models in Series 2 show the effects of both the global and local slendernesses. All models in Series 1 and 2 satisfy the local and global slenderness requirements in EN 1998-1 (2004) for ductile behaviour. In this and the following section, a global imperfection of $L/1500$ and local imperfection with amplitudes obtained from Equations 5.1-5.3 were incorporated in the models. In some cases, slightly larger local imperfections were applied to overcome numerical difficulties, but the sensitivity study showed that the effect of such changes was negligible.

All members were subjected to the loading history described in Section 5.2.4. The predicted numbers of cycles to the initiation of global and local buckling, and to eventual fatigue fracture, based on the four strain measures, are given in Tables 6.1 and 6.2, alongside displacement ductility of the members, μ_{Δ} . As explained in Sections 5.2.6, the case of SP-peak, using the peak strains from the most critical location and assuming maximum values through the thickness, gave the lowest fracture lives. In contrast, the case of SPN-avg gave the lowest strain amplitudes and predicted the longest fracture lives. For the remainder of the study, fracture lives were predicted on the basis of SP-avg due to the low sensitivity to mesh size and ability to reflect the influence of local bending.

Table 6.1: Models in Series 1

Specimen identification	$b/t\epsilon$	$\bar{\lambda}$	Cycle at which buckling begins		Number of cycles to failure				μ_{Δ}
			Global	Local	SP avg	SPN avg	SP peak	SPN peak	
120×120×6×1500-CS-CF	21.9	0.25	6	7	9	9	9	9	4
60×60×3×1500-CS-CF	21.9	0.49	5	6	9	9	8	9	4
30×30×1.5×1500-CS-CF	21.9	0.99	3	8	9	9	9	9	4
20×20×1×1500-CS-CF	21.9	1.48	3	8	10	10	10	10	4

Table 6.2: Models in Series 2

Specimen identification	b/tε	$\bar{\lambda}$	Cycle at which buckling begins		Number of cycles to failure				μ_{Δ}
			Global	Local	SP avg	SPN avg	SP peak	SPN peak	
60×60×6×5000-CS-CF *	8.2	1.78	-	-	-	-	-	-	-
60×60×4×5000-CS-CF	15.1	1.69	2	14	16	16	15	16	8
60×60×3×5000-CS-CF	21.9	1.63	2	11	12	13	12	12	6
60×60×2.5×5000-CS-CF	27.4	1.63	2	8	12	12	10	11	6
60×60×6×3500-CS-CF	8.2	1.25	3	14	16	17	15	16	8
60×60×4×3500-CS-CF	15.1	1.18	3	11	12	13	12	12	6
60×60×3×3500-CS-CF	21.9	1.14	3	8	10	10	9	10	4
60×60×2.5×3500-CS-CF	27.4	1.14	3	7	9	9	9	9	4
60×60×6×2500-CS-CF	8.2	0.89	4	11	15	15	14	15	8
60×60×4×2500-CS-CF	15.1	0.84	4	8	10	11	10	11	4
60×60×3×2500-CS-CF	21.9	0.82	4	7	9	9	9	9	4
60×60×2.5×2500-CS-CF	27.4	0.81	4	5	7	8	7	7	2
60×60×6×1500-CS-CF	8.2	0.53	5	10	13	13	12	13	6
60×60×4×1500-CS-CF	15.1	0.51	5	8	10	11	9	11	4
60×60×3×1500-CS-CF	21.9	0.49	5	6	9	9	8	9	4
60×60×2.5×1500-CS-CF	27.4	0.49	5	5	8	9	7	8	4

* Analysis has not been completed due to numerical difficulty.

6.2.1 Overview of the results

The numbers of cycles at which global buckling, local buckling and fracture occurred for members in Series 1 are presented in Figure 6.1. In Series 2, both global and local slendernesses were varied. Their individual influences were isolated by analysing the results in groups of members with constant section thickness or member length. The numbers of cycles at which global buckling, local buckling and fracture occurred for members with constant section thicknesses but varying member length are shown in Figures 6.2-6.5. The same set of data was reorganised and plotted in Figures 6.6-6.9 to show the influence of local slenderness on the behaviour of the members.

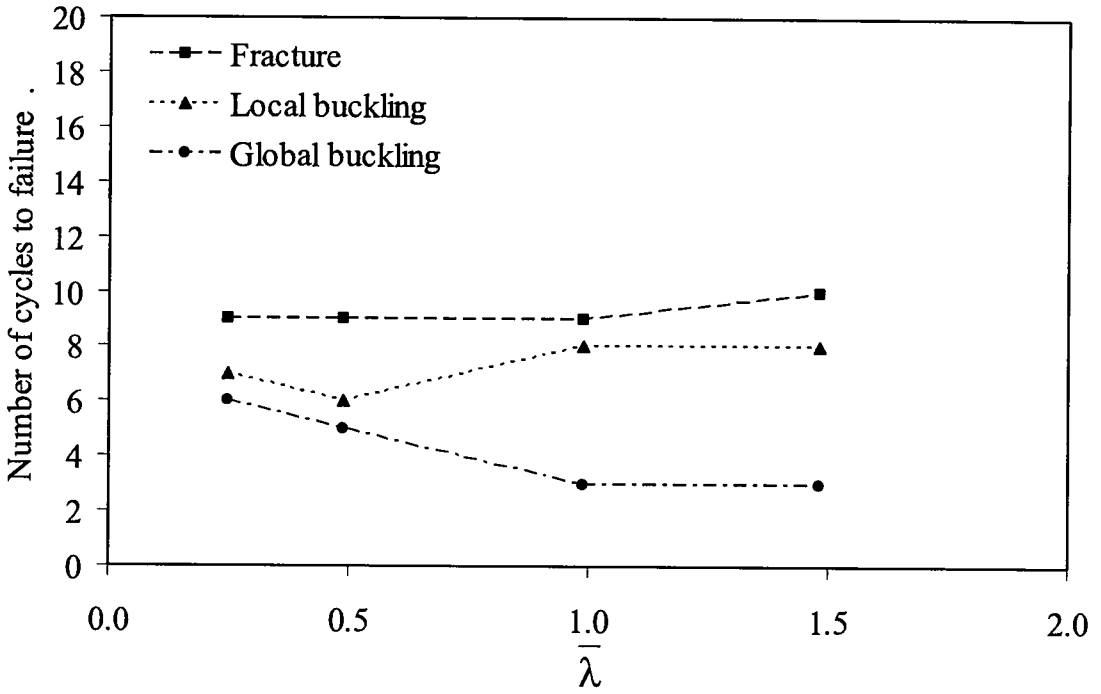


Figure 6.1: Global slenderness against numbers of cycles when global buckling, local buckling and fracture occurred for members in Series 1

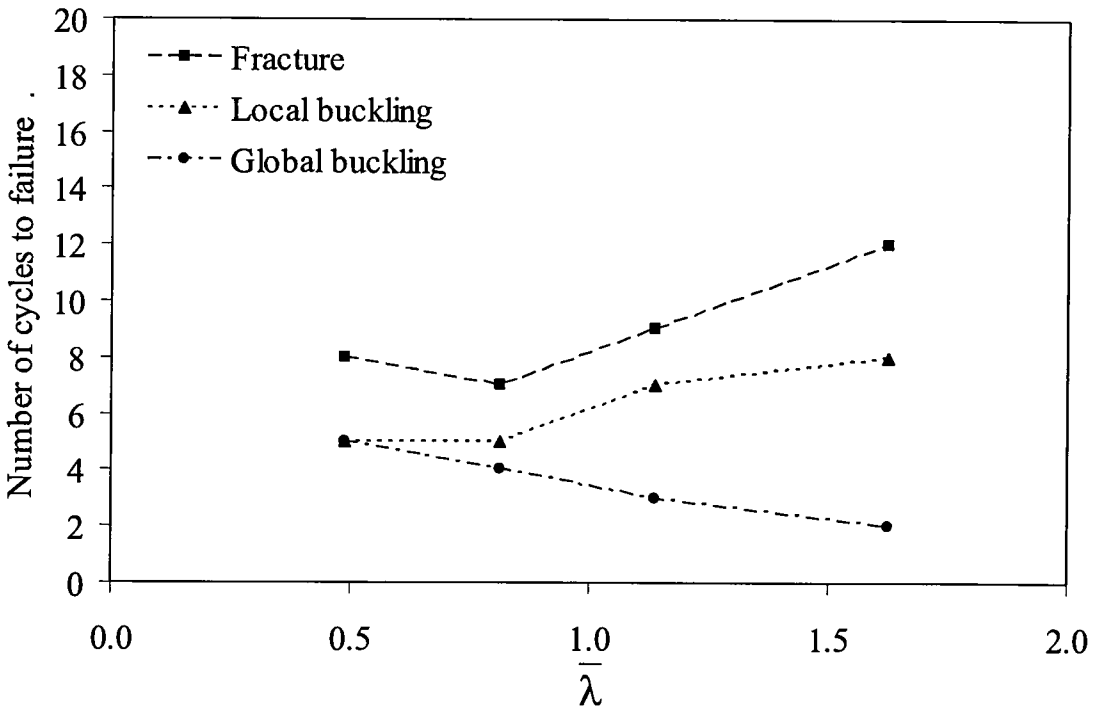


Figure 6.2: Global slenderness against numbers of cycles when global buckling, local buckling and fracture occurred for members with sections of 2.5 mm thick in Series 2

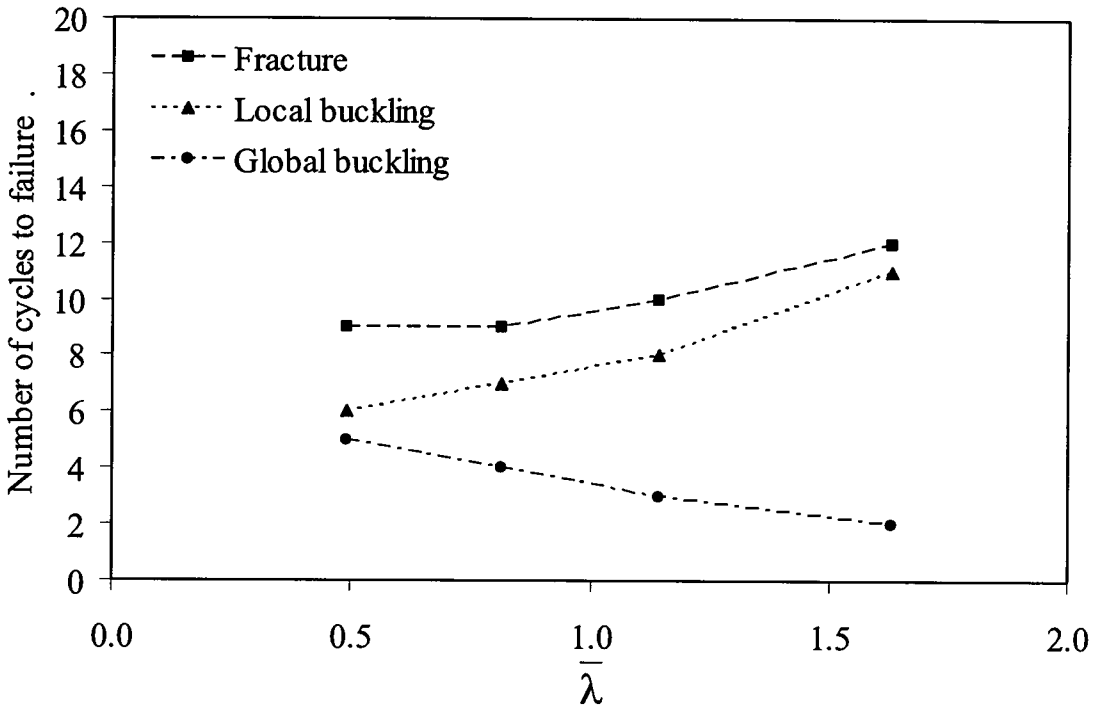


Figure 6.3: Global slenderness against numbers of cycles when global buckling, local buckling and fracture occurred for members with sections of 3.0 mm thick in Series 2

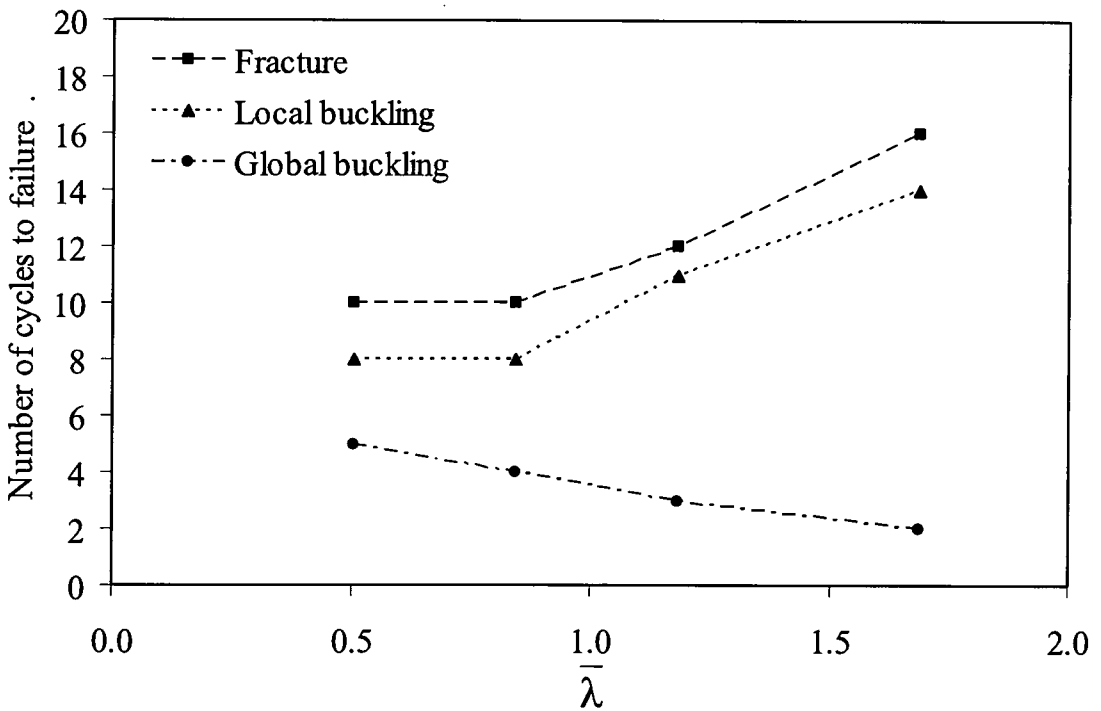


Figure 6.4: Global slenderness against numbers of cycles when global buckling, local buckling and fracture occurred for members with sections of 4.0 mm thick in Series 2

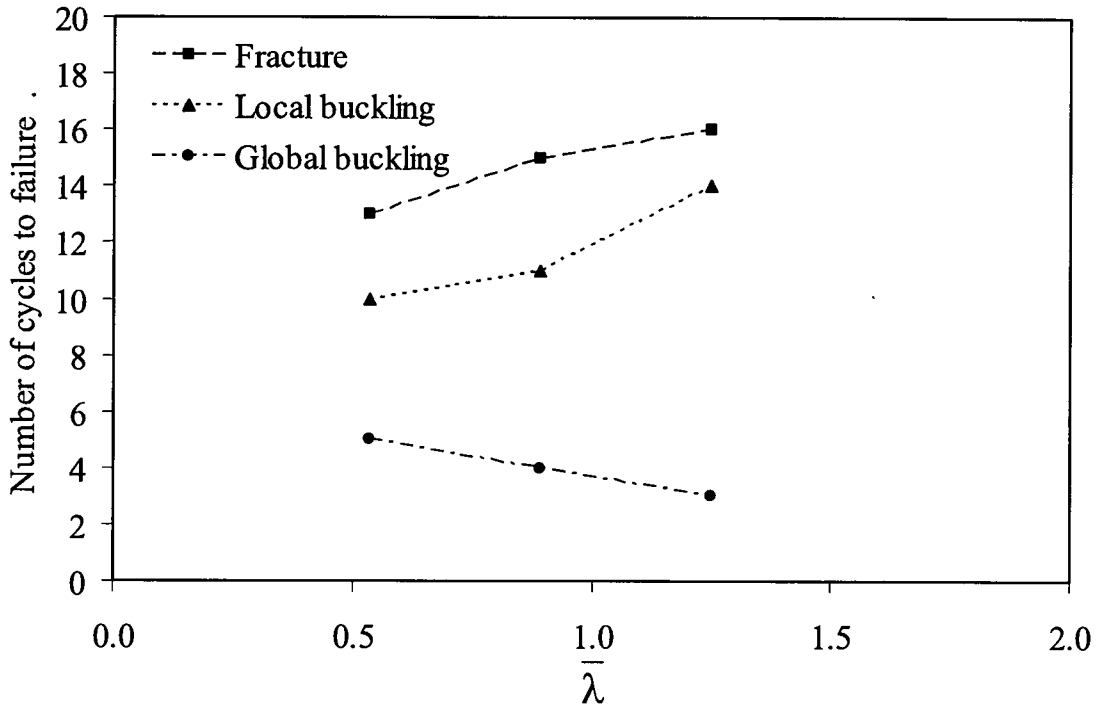


Figure 6.5: Global slenderness against numbers of cycles when global buckling, local buckling and fracture occurred for members with sections of 6.0 mm thick in Series 2

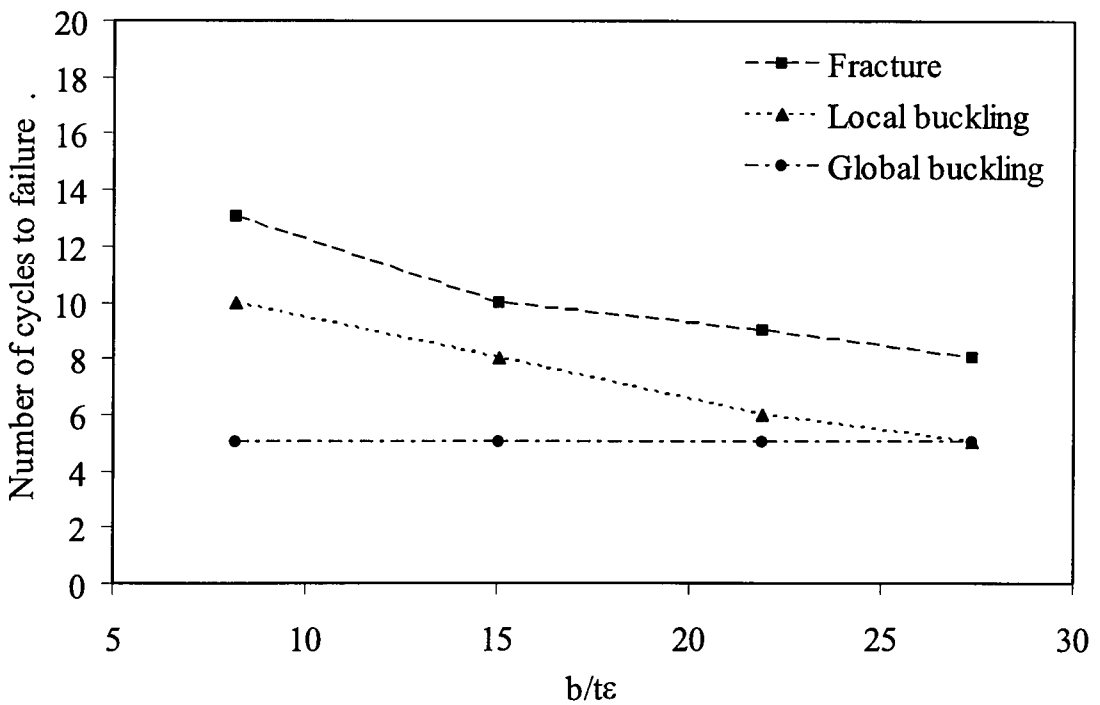


Figure 6.6: Local slenderness against numbers of cycles when global buckling, local buckling and fracture occurred for 1500 mm long members in Series 2

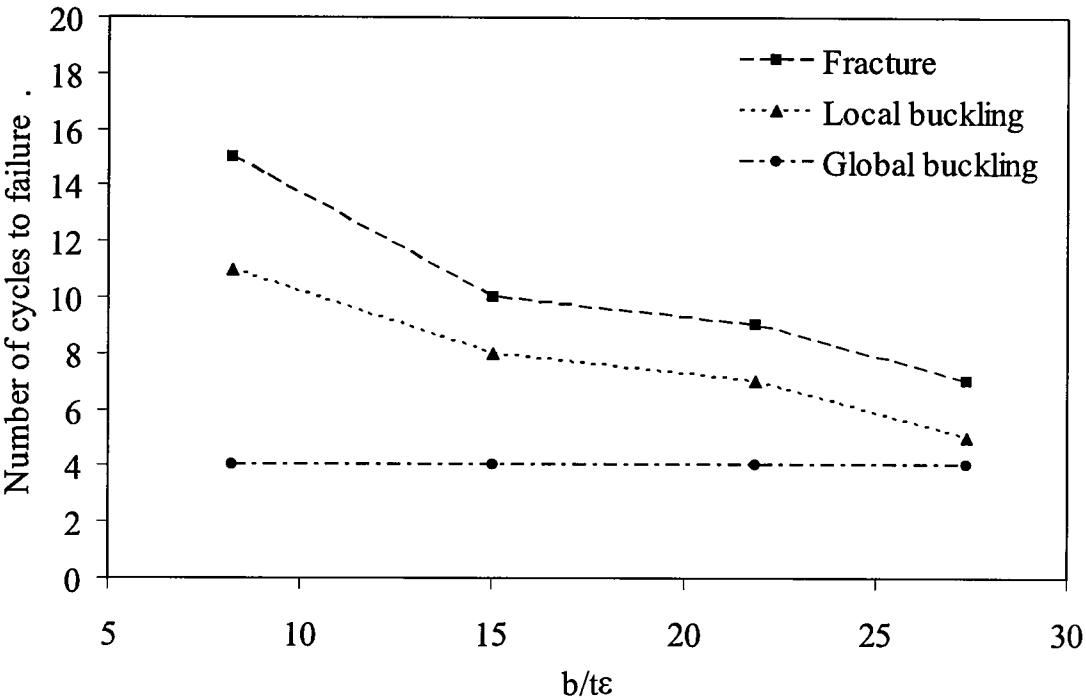


Figure 6.7: Local slenderness against numbers of cycles when global buckling, local buckling and fracture occurred for 2500 mm long members in Series 2

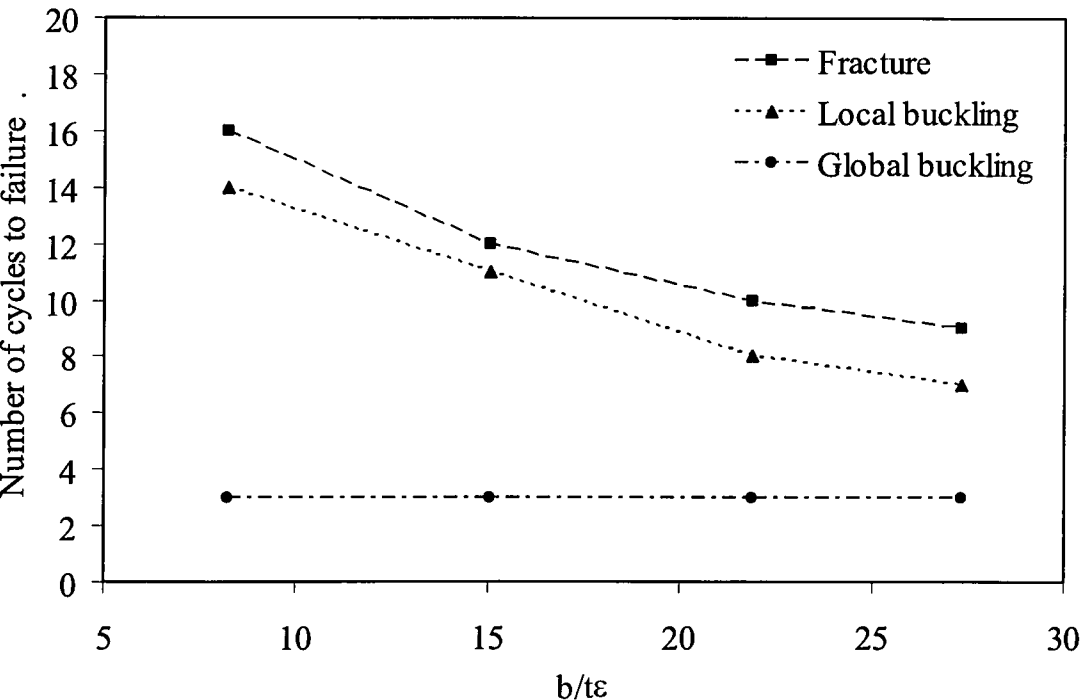


Figure 6.8: Local slenderness against numbers of cycles when global buckling, local buckling and fracture occurred for 3500 mm long members in Series 2

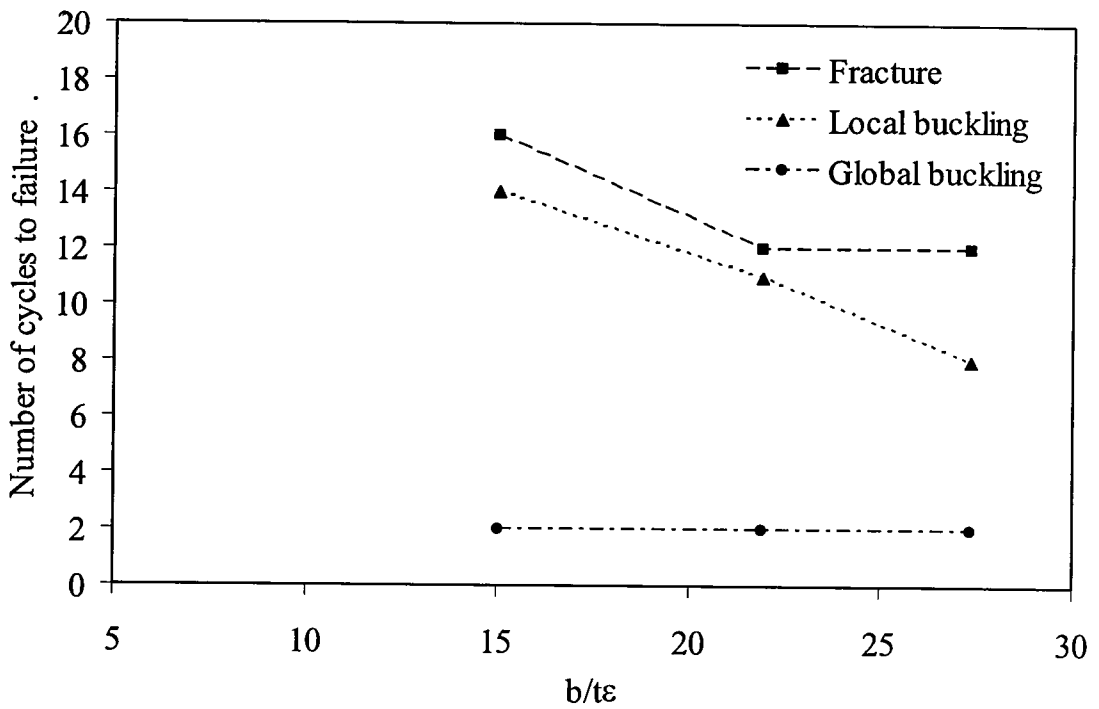


Figure 6.9: Local slenderness against numbers of cycles when global buckling, local buckling and fracture occurred for 5000 mm long members in Series 2

As observed in the experiments in this research project and other studies (Tremblay, 2002; Goggins et al., 2006), the fatigue fracture of hollow section braces occurs at the corner areas, where material ductility is lower and high localised strains develop due to the redistribution of load following local buckling (Figure 5.5). The resistance to local buckling depends on the elastic local buckling stress of a plate element, which decreases as its width-to-thickness ratio increases. As a result, a section with high local slenderness is more prone to local buckling. The influence of local slenderness is clearly shown by the trends of numbers of cycles to local buckling and fracture in Figures 6.6-6.9 and has been observed in experiments (Han et al., 2007).

However, it has been observed that local buckling is more severe in braces with low global slenderness, even if local slenderness is kept low (Tremblay, 2002). This is generally thought to be due to the higher compressive strains induced in the plastic hinges that form in braces with lower global slenderness. Moreover, in Figures 6.1-6.5, it is shown that the braces with lower global slenderness have lower numbers of cycles to the initiation of local buckling, even though the local slenderness is kept constant. This implies that less slender braces experience higher compressive strains which induce earlier local buckling. These observations suggest that global slenderness

influences certain aspects of the local buckling of the braces, such as the amplitude of compressive strain on the braces before local buckling and the severity of local buckling once it has initiated, which are discussed below.

6.2.2 Effect of member geometry on strains

If a brace with fixed ends buckles elastically under a compressive load, its deformed shape can be represented by the following expression:

$$y = \frac{\Delta}{2} \left(1 - \cos \frac{2\pi x}{L} \right) \quad (6.1)$$

where y is the lateral deflection at distance x unit from a fixed end, Δ is the lateral deflection at the mid-length and L is the total length of the brace (Figure 6.10). By assuming small angles and incompressibility, the applied compressive displacement at the end, δ_c , can be expressed as:

$$\delta_c = \int_0^L \frac{1}{2} \left(\frac{dy}{dx} \right)^2 dx \quad (6.2)$$

which, after integration and rearrangement, gives:

$$\Delta = \frac{2}{\pi} \sqrt{\delta_c L} \quad (6.3)$$

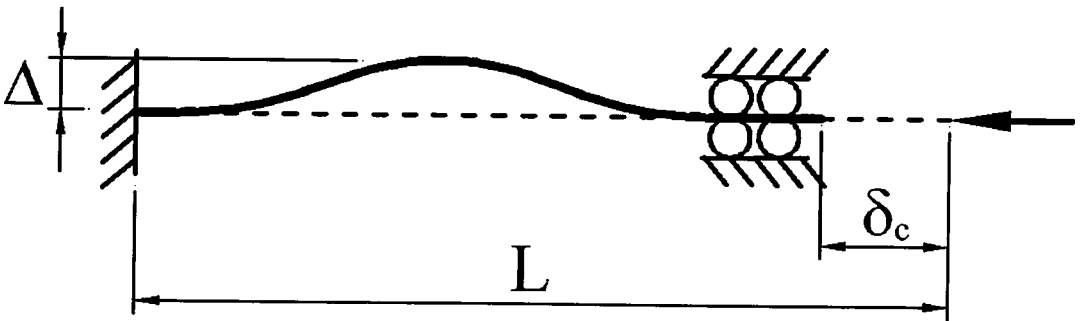


Figure 6.10: Elastic buckling of bracing member

As the bending moments at the ends and the mid-length of the members increase, plastic hinges may develop at these locations and the brace would form a mechanism. By considering the geometry of this mechanism, which provides an upper bound value of lateral deflection (Figure 6.11), a relationship between the applied compressive displacement at the end and lateral deflection at the mid-length can be given by:

$$\Delta = \sqrt{\left(\frac{L_H}{2}\right)^2 - \left(\frac{L_H - \delta_c}{2}\right)^2} = \sqrt{\frac{2L_H\delta_c - \delta_c^2}{4}} \quad (6.4)$$

where L_H is the distance between the hinges that form near the ends. These hinges have been assumed to be located at a distance from the brace ends equal to the depth of the member on the plane of buckling (Tremblay, 2002). If δ_c is small relative to L_H , Equation 6.4 can be simplified into:

$$\Delta = \frac{1}{\sqrt{2}} \sqrt{\delta_c L_H} \quad (6.5)$$

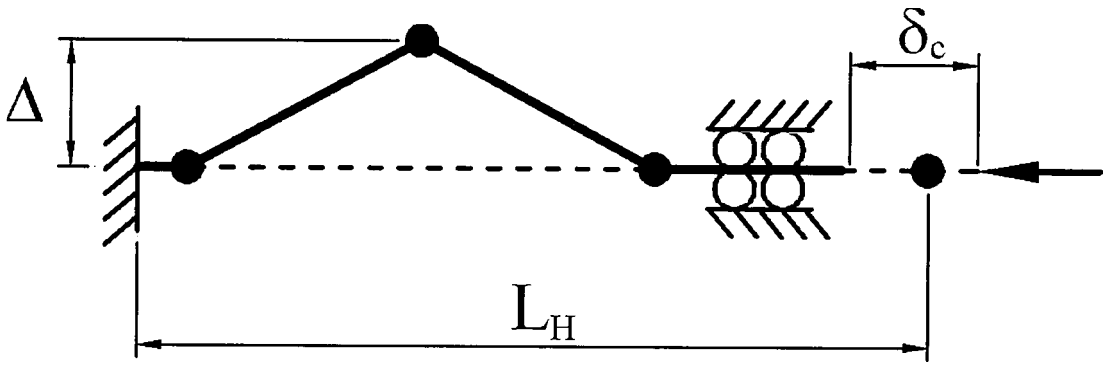


Figure 6.11: Plastic mechanism in bracing member

Figure 6.12 shows the relationship between the lateral deflection and the applied axial displacement for braces in Series 1 under monotonic compressive loading alongside the curves for the ideal cases given by Equations 6.3 and 6.5. The more slender braces exhibit large lateral deflection at a given axial displacement. They generally follow the elastic buckling shape and approach the upper bound mid-length deflection of the mechanism. Since the less slender braces have higher resistance to global buckling, they undergo axial shortening at small applied axial displacement. Under increasing compressive strain, the less slender braces buckle but the lateral deflection does not reach the values predicted by Equations 6.3 and 6.5, which ignore the compressibility of the braces.

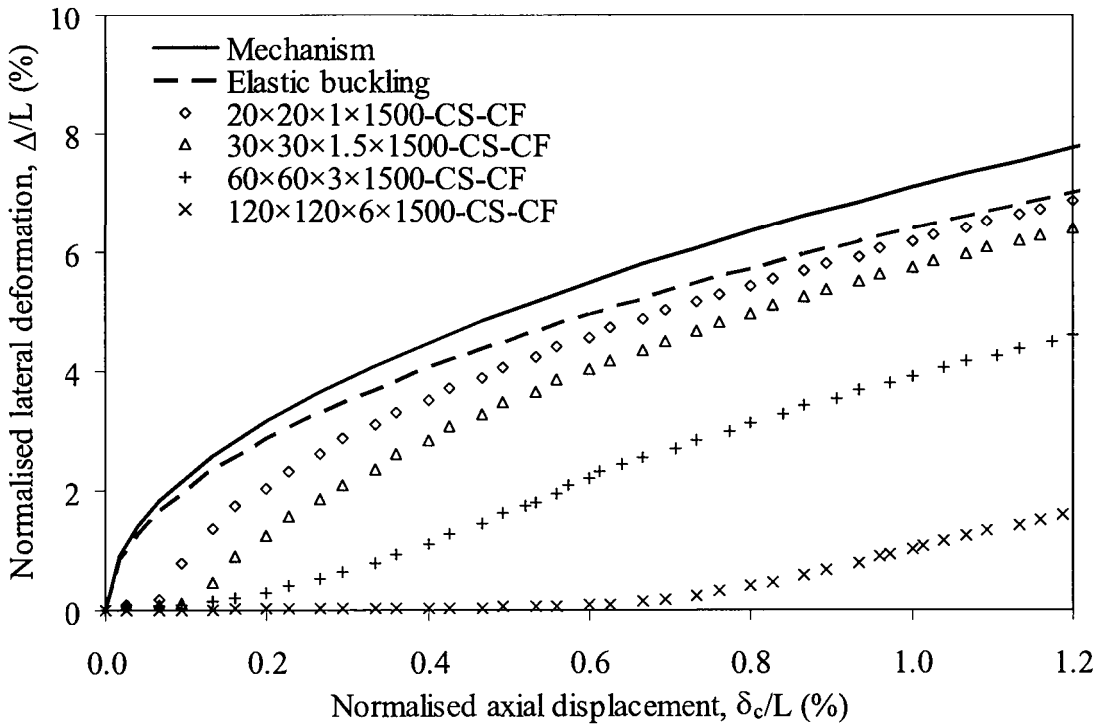


Figure 6.12: *The relationship between lateral deflection and monotonic axial displacement*

In cases of cyclic loading, the permanent elongation of the braces after attaining the maximum tensile displacement, δ_t , can be taken into account. Assuming the braces still buckle elastically, the lateral deflection is given by:

$$\Delta = \frac{2}{\pi} \sqrt{(\delta_c + \delta_t - \delta_y)L} \quad (6.6)$$

where δ_y is the yield displacement. Similarly, the lateral deflection for braces for the mechanism case has been shown by Tremblay (2002) to be:

$$\Delta = \frac{1}{\sqrt{2}} \sqrt{(\delta_c + \delta_t - \delta_y)L_H} \quad (6.7)$$

Figures 6.13 and 6.14 show the mid-length lateral deflection for selected braces from Series 1 and Series 2 compared with the predictions from Equations (6.6) and (6.7). Similar to the results under monotonic loading, the less slender braces undergo large axial deformation and deflect laterally to a lesser extent.

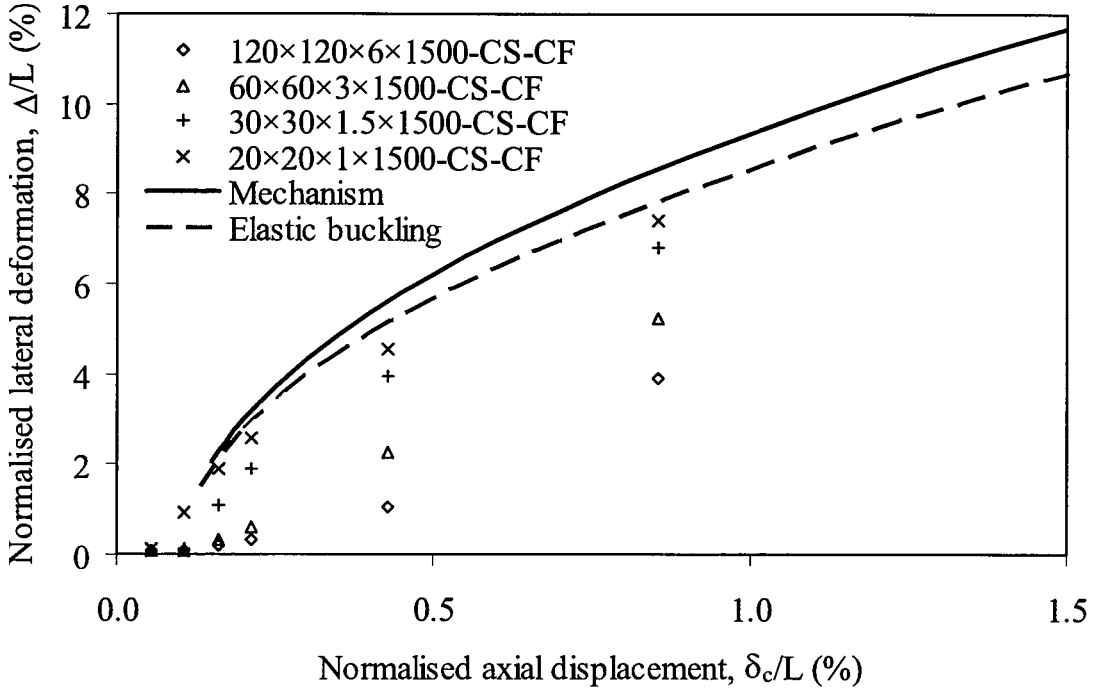


Figure 6.13: The relationship between lateral deflection and axial displacement under cyclic loading for models in Series 1

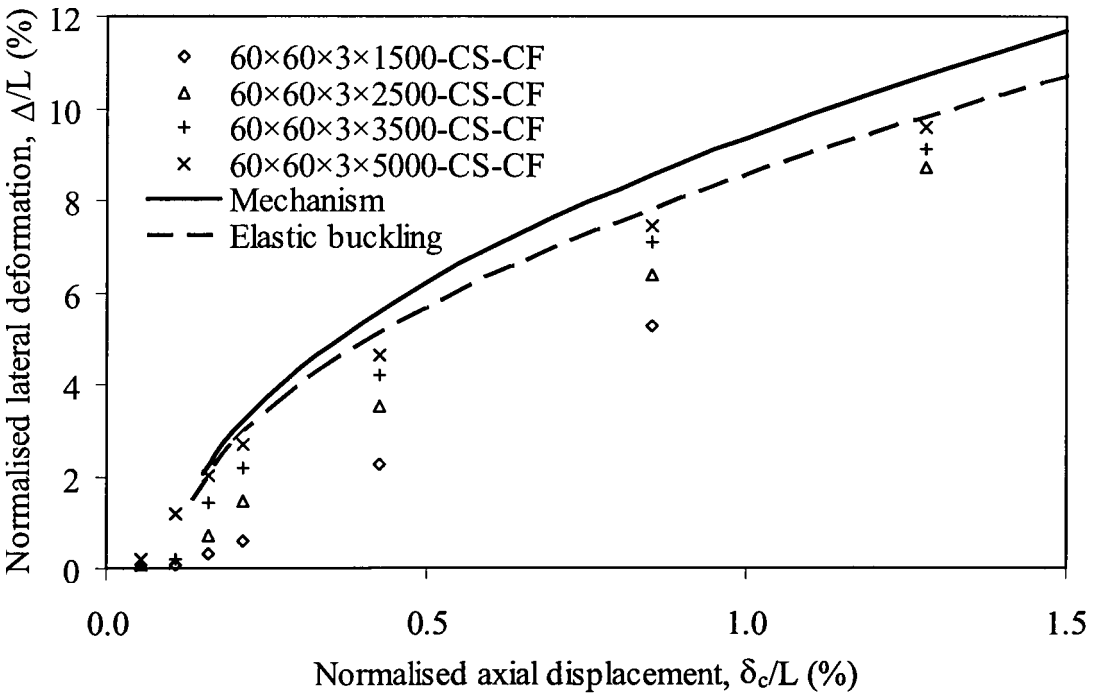


Figure 6.14: The relationship between lateral deflection and axial displacement under cyclic loading for selected models in Series 2

Based on the lateral deflections in the above scenarios, the compressive strains at the section at the mid-length of the braces before and after buckling can be obtained. By considering a section being bent to a radius of curvature, r , over an angle, θ (Figure 6.15), and assuming plane sections remain plane, the strain at the outermost fibre, ε_b , is given by:

$$\varepsilon_b = \frac{(r + \frac{h}{2} - r)\theta}{r\theta} = \frac{h}{2r} \quad (6.8)$$

where h is the depth of the section.

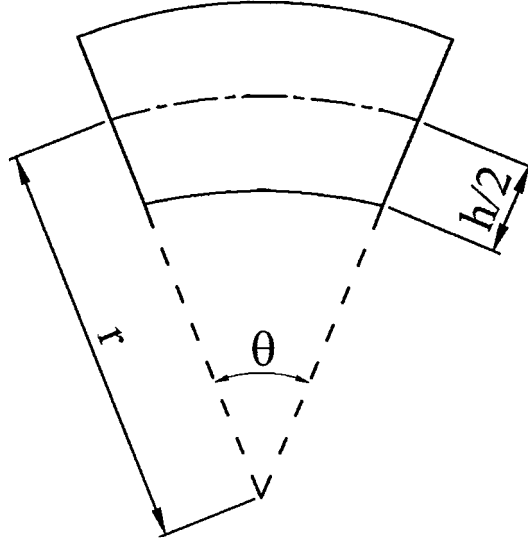


Figure 6.15: A section under bending

If a brace buckles elastically, the curvature, κ , at the mid-length can be obtained from the second derivative of the deformed shape (Equation 6.1):

$$\kappa = \frac{d^2y}{dx^2} \Big|_{x=\frac{L}{2}} = 2\Delta \left(\frac{\pi}{L}\right)^2 \cos \frac{2\pi x}{L} \Big|_{x=\frac{L}{2}} = -2\Delta \left(\frac{\pi}{L}\right)^2 \quad (6.9)$$

Noting that $\kappa = 1/r$ and ignoring the sign of the curvature, the strain at the outermost fibre at mid-length due to elastic buckling can be obtained from Equations 6.6, 6.8 and 6.9:

$$\varepsilon_b = \frac{2\pi h}{L^2} \sqrt{(\delta_c + \delta_t - \delta_y)L} = \frac{2\pi h}{L} \sqrt{\varepsilon_c - \varepsilon_t - \varepsilon_y} \quad (6.10)$$

where ε_c is the applied compressive strain if the brace remains straight, ε_t is the maximum tensile strain the brace has attained and ε_y is the yield strain. This

relationship corresponds to the strain at the outmost fibre at the section at the mid-length for an incompressible, elastically buckled brace.

If plastic hinges form, the radius of curvature is given by the rotation at the plastic hinge, θ , over the length of the plastic hinge, L_{pl} :

$$r = 1/\kappa = L_{pl}/\theta \quad (6.11)$$

If the length of the plastic hinge is assumed to be equal to the depth of the section, which has been observed following analysis of the deformed specimens in the present study, substitution of Equation 6.11 into Equation 6.8 gives:

$$\varepsilon_b = \frac{h\theta}{2L_{pl}} = \frac{\theta}{2} \quad (6.12)$$

By considering the geometry of the mechanism and Equation 6.12,

$$\varepsilon_b = \frac{1}{2}[\pi - 2\sin^{-1}(\frac{1 - \varepsilon_c}{1 + \varepsilon_t - \varepsilon_y})] \quad (6.13)$$

Figures 6.16 and 6.17 show a comparison between the strains at the outmost fibre at the mid-length of the member for the deformed shapes associated with elastic buckling and the mechanism for selected models of Series 1 and 2 respectively. It is demonstrated that strains associated with the elastic buckling case depends on the depth of the section and the length of the member, or the global slenderness (Equation 6.10). The lower compressive strains experienced by less slender members partly explains the later occurrence of local buckling. In contrast, after the formation of plastic hinges, the strain at the plastic hinges depend only on the rotation, which is in turn related to the applied axial displacement, but not the dimensions of the braces (Equation 6.12). In the parametric studies, the observation that braces fracture soon after local buckling regardless of the slenderness supports this deduction.

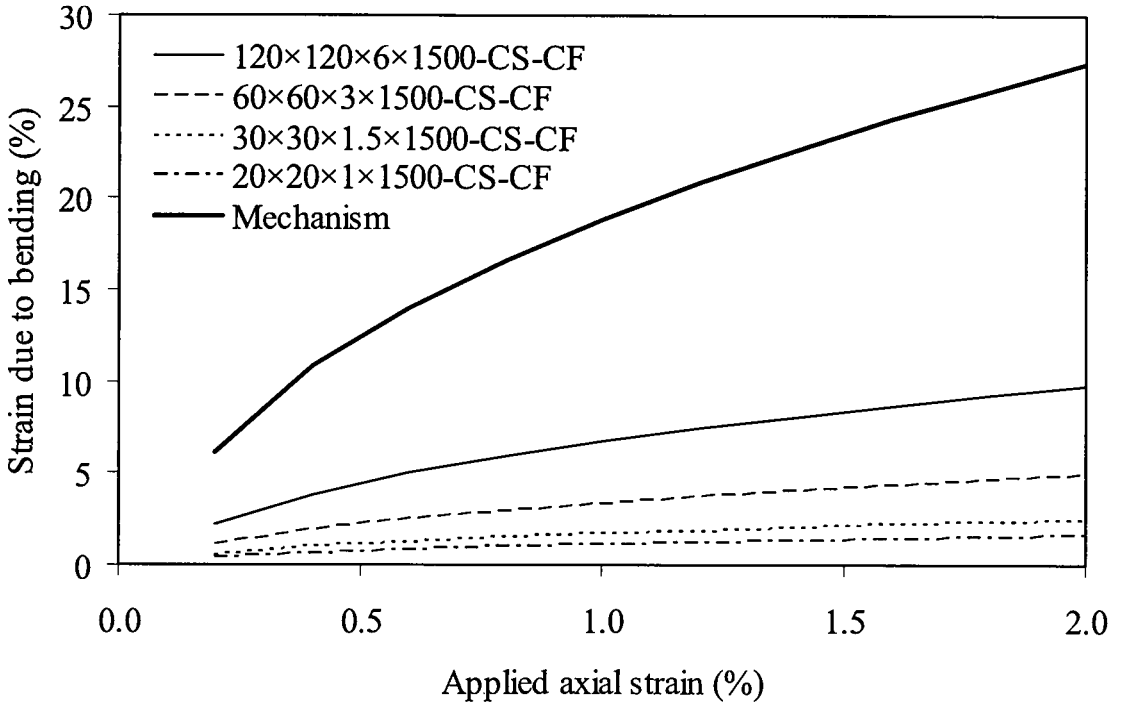


Figure 6.16: Variation of strain due to bending with applied axial strain under cyclic loading for models in Series 1

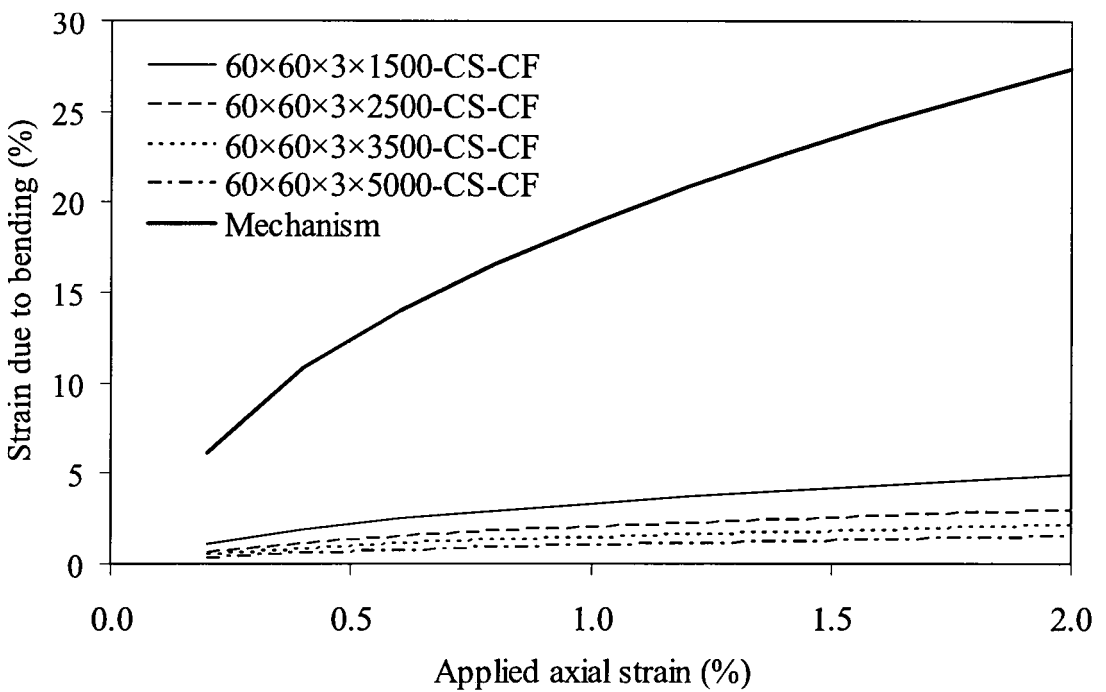


Figure 6.17: Variation of strain due to bending with applied axial strain under cyclic loading for selected models in Series 2

However, in reality the braces are compressible and strain due to axial deformation may be induced. As shown in Figures 6.12-6.14, the proportion of the strain due to axial deformation is particularly large for less slender braces. Since the total strain can be resolved into axial and bending components (Figure 6.18a), the larger axial component for the less slender braces implies a more uniform strain distribution across the section (Figure 6.18b), resulting in less restraint being provided to the compression flange by the two webs. This could lead to earlier and more severe local buckling for less slender bracing members.

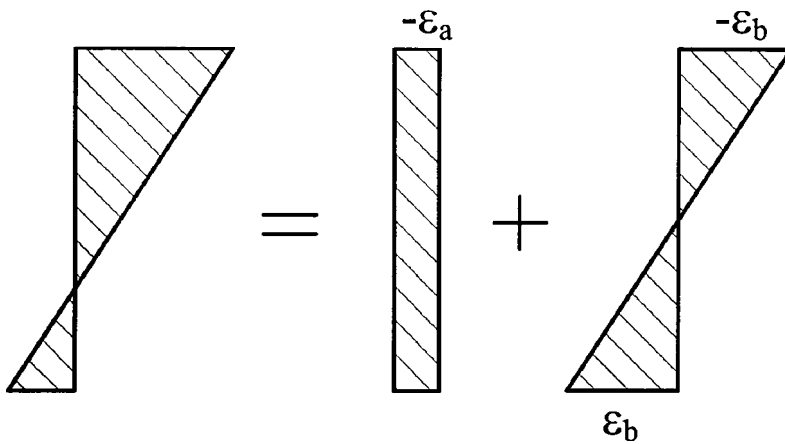


Figure 6.18a: *Resolving strains into axial and bending components*

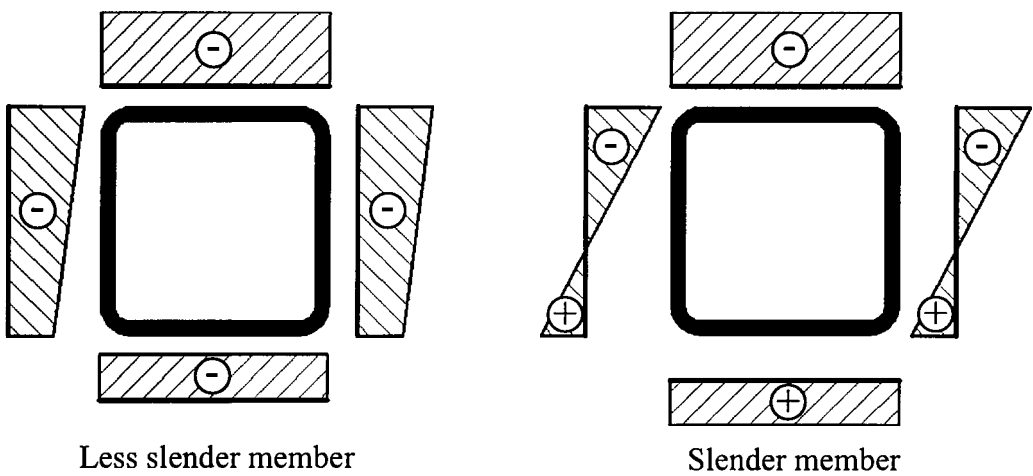


Figure 6.18b: *Distributions of strain across the section for members with different slenderness (compressive strain indicated by negative sign)*

Another factor that may affect the severity of local buckling is the moment gradient along the length of the braces. The moment gradient in a bracing member for the mechanism case (Figure 6.11) is given by $2M_{pl}/0.5L_H$, where M_{pl} is the plastic moment capacity of the section. Less slender braces have either larger M_{pl} (as in Series 1) or smaller L_H (as in Series 2), resulting in a steeper moment gradient.

The factors outlined above influence the behaviour of bracing members under cyclic loading which, in turn, controls the number of cycles to failure of the brace members. Depending on the slenderness of the members, axial shortening causes the actual behaviour deviating, to variable extent, from the ideal cases discussed above. In addition, the combined action of the factors is difficult to quantify. Therefore, the influence of the global slenderness on the number of cycles to failure is less definitive, which may result in less obvious trend as shown by the models in Series 1.

6.2.3 *Comparison with previous investigations*

Figures 6.19 and 6.20 summarise the relationship between slenderness and the number of cycles to failure for all models in Series 1 and 2. Although the current study shows that both global slenderness and local slenderness have influence on the fracture life of a member, the effects of these slenderness parameters have not always been observed in experiments, leading to mixed comments on the relative importance of these geometric parameters. To discuss this aspect of behaviour, observations from previous studies are compared with the results from the current numerical investigation.

It was indicated in a previous comprehensive survey of cyclic tests on braces (Tremblay, 2002) that fracture of bracing members depended strongly upon global slenderness and, to a lesser extent, on local slenderness, and a linear regression analysis was performed to show that ductility of members increased with global slenderness. Importantly, however, it was pointed out that in the experimental database examined (Tremblay, 2002), braces with low global slenderness usually had high local slenderness, i.e. large sections with thin walls, suggesting that global slenderness may not be the sole factor affecting the fracture lives. This indicates that the experimental results may well be interpreted in a different manner. To illustrate this, Figures 6.21-6.22 depict the global and local slendernesses of all SHS and RHS members from the

database (Walpole, 1996; Black et al., 1980; Gugerli, 1982; Lee & Goel, 1987; Archambault et al., 1995; Liu, 1987; Shaback, 2001) and similar experimental programmes (Han et al., 2007; Goggins et al., 2006). Figure 6.21 demonstrates the inter-relationship between global slenderness and local slenderness in these tested specimens. From the results of these experimental programmes, it is unclear that the changes of fracture lives are caused by varying global slenderness or local slenderness. This suggests that additional tests on slender braces with thin walls, as well as stockier braces with thick walls, are needed to ascertain the influence of local and global slenderness in isolation.

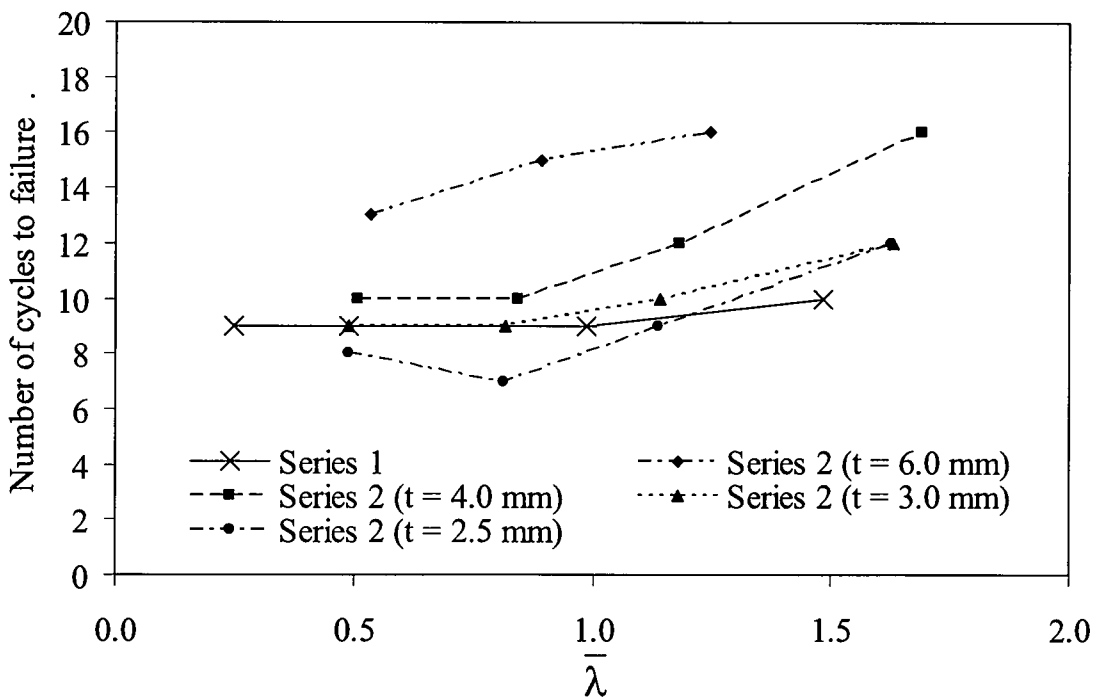


Figure 6.19: Relationship between global slenderness and number of cycles to failure

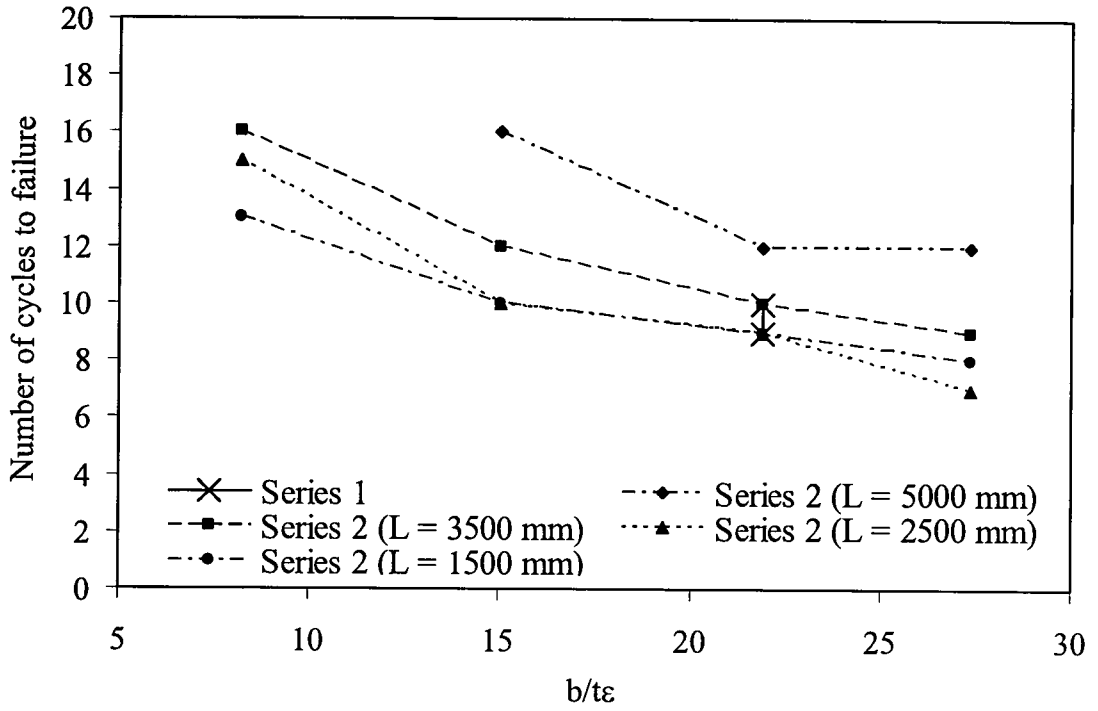


Figure 6.20: Relationship between local slenderness and number of cycles to failure

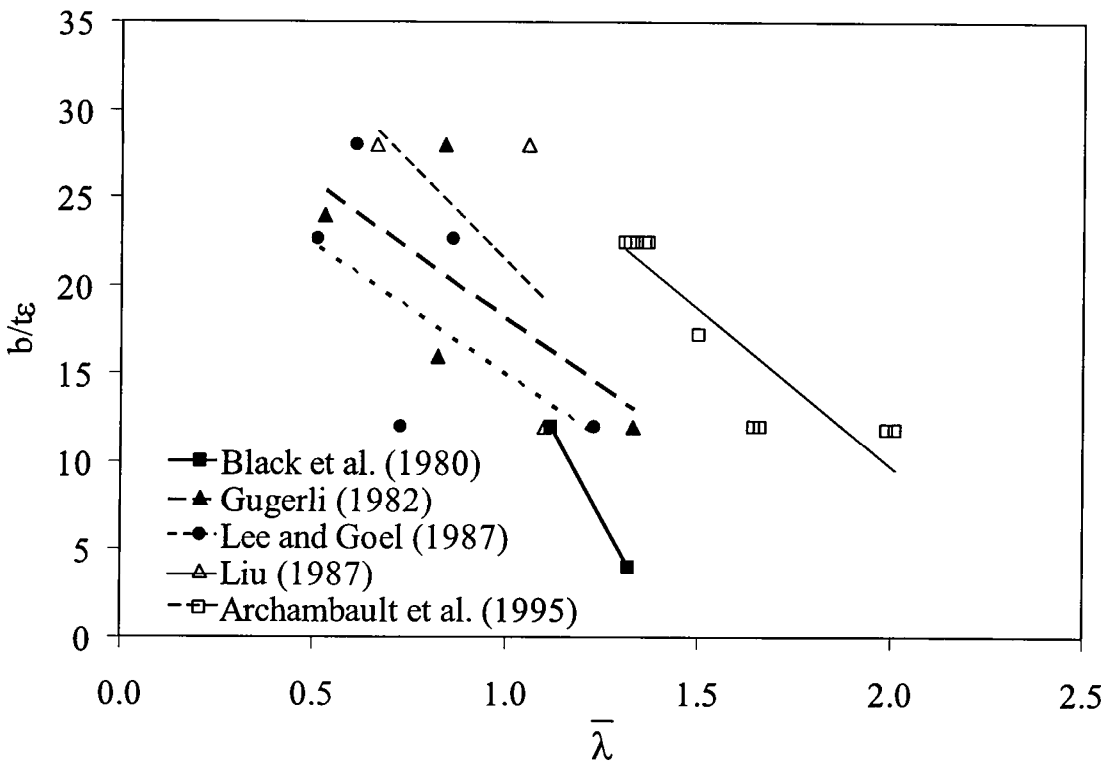


Figure 6.21: Test programmes in which both local and global slenderness were varied simultaneously

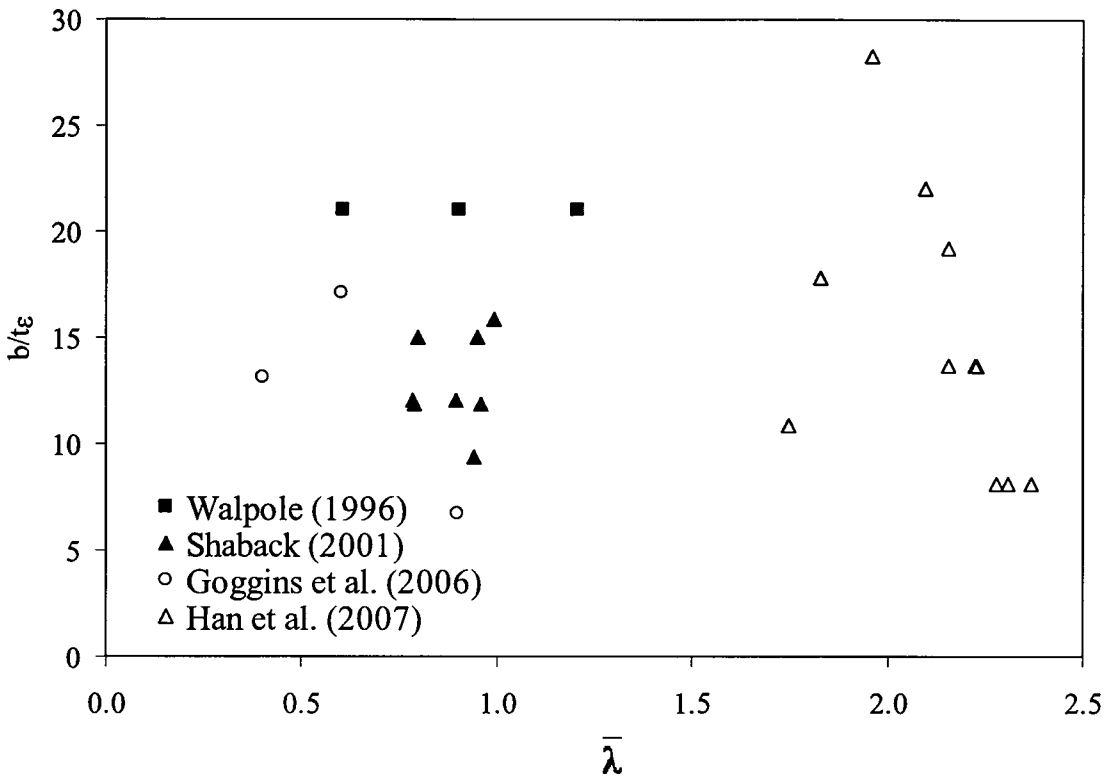


Figure 6.22: Test programmes in which variation was confined to one slenderness parameter or the co-existing influence of the two slenderness parameters were identified

It is important to note that in the two experimental programmes in which variation was essentially confined to one slenderness parameter, the importance of global slenderness and local slenderness, respectively (Walpole, 1996; Han et al., 2007), can be observed (Figure 6.22). Walpole (1996) demonstrated that, in the absence of the influence of local slenderness, the number of cycles to failure dropped from 8 to 7 only as the global slenderness decreased from 1.8 to 0.4. Han et al. (2007) selected specimens with a very large range of local slenderness and limited variation in global slenderness to show that the number of cycles to failure indeed depended significantly on the local slenderness.

Furthermore, the co-existing influence of the global slenderness and local slenderness has been given attention by some researchers (Shaback & Brown, 2003; Goggins et al., 2006). In the test programme conducted by Goggins et al. (2006), although the specimens with high global slenderness had low local slenderness, and vice versa (Figure 6.22), the influence of the global slenderness and local slenderness was observed. Two linear regression analyses were performed to establish the relationships between the number of cycles to failure and these two slenderness parameters

respectively. Similarly, based on the test data of Shaback (2001), Shaback and Brown (2003) concluded that both the global slenderness and local slenderness were influential for different aspects of the hysteretic response of braces. A cumulative measure of tensile and compressive deformation was expressed in terms of global slenderness and local slenderness, among other parameters. However, this cumulative measure is sensitive to the shape of the hysteretic loops, which has a large variability when different loading histories are applied. In contrast, displacement ductility is more easily related to design scenarios when only the maximum ductility demand is known. In order to address the co-existing influence of global slenderness and local slenderness observed in the current and some other studies, an expression of displacement ductility will be proposed based on all available experimental and FE results in Section 6.3.1.

6.3 *Comparison between braces of the three materials*

The second part of the parametric studies was conducted to compare the performance of members of the three materials, particularly in the aspects of number of cycles to failure, energy dissipation and tangent stiffness. In this section, 16 models were considered for each material. These were 60×60 mm square hollow sections with thicknesses of 2.5 mm, 3.0 mm, 4.0 mm and 6.0 mm. The member lengths were 1500 mm, 2500 mm, 3500 mm and 5000 mm. In order to ensure a consistent comparison between the materials, the average yield strengths of the sections of the three materials were set equal to one another at around 460 N/mm², while the cyclic material parameters reflected their particular characteristics in terms of cyclic hardening.

Similar to the experimental specimens, all models remained elastic in the first few cycles, followed by global buckling and inelastic deformation. Local buckling was observed at the mid-length and, to a lesser extent, at the ends of the members. Fracture was predicted to occur at the mid-length where the deformation was the largest, although a few exceptional cases showed that fracture occurred at the area of local buckling at the ends.

6.3.1 *Fracture life*

The number of cycles to failure for each model is plotted against global slenderness in Figures 6.23-6.26. All stainless steel models exhibit a larger number of cycles to fracture than their carbon steel counterparts. Except for the most slender cross-sections, the number of cycles to fracture increases with global slenderness, as does the difference in fracture life between the stainless steel and carbon steel members. The plots also show a smaller but consistent difference between hot-rolled and cold-formed carbon steel, with the former exhibiting slightly large number of cycles for most of the slenderness ranges. The same set of data was reorganised and plotted against local slenderness in Figures 6.27-6.30. A similar trend can be observed upon variation of local slenderness. The number of cycles to fracture increases with decreasing local slenderness, and at low slenderness the difference in fracture life between the stainless steel and carbon steel members is greatest, apart from the most stocky members.

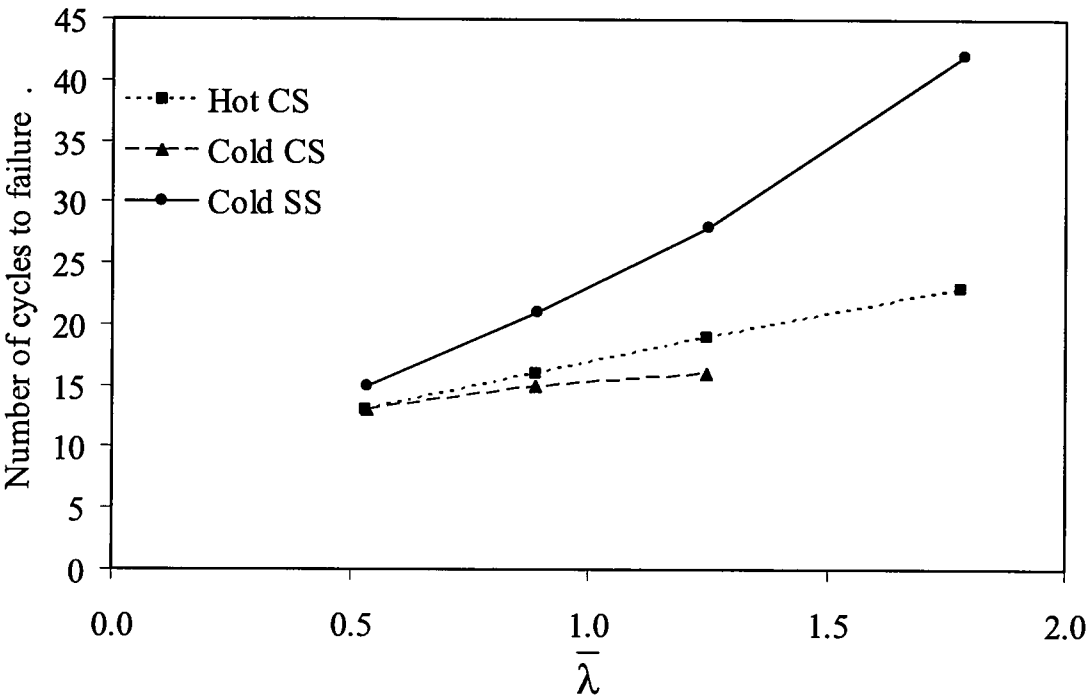


Figure 6.23: Number of cycles to failure for FE specimens with 6.0 mm thick sections

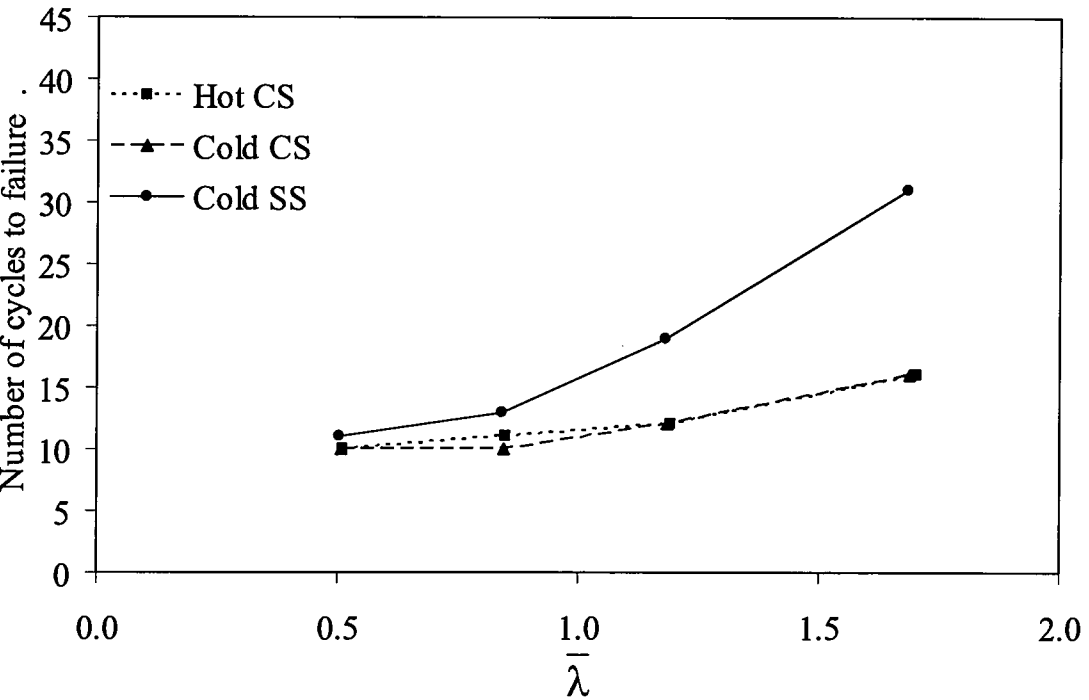


Figure 6.24: Number of cycles to failure for FE specimens with 4.0 mm thick sections

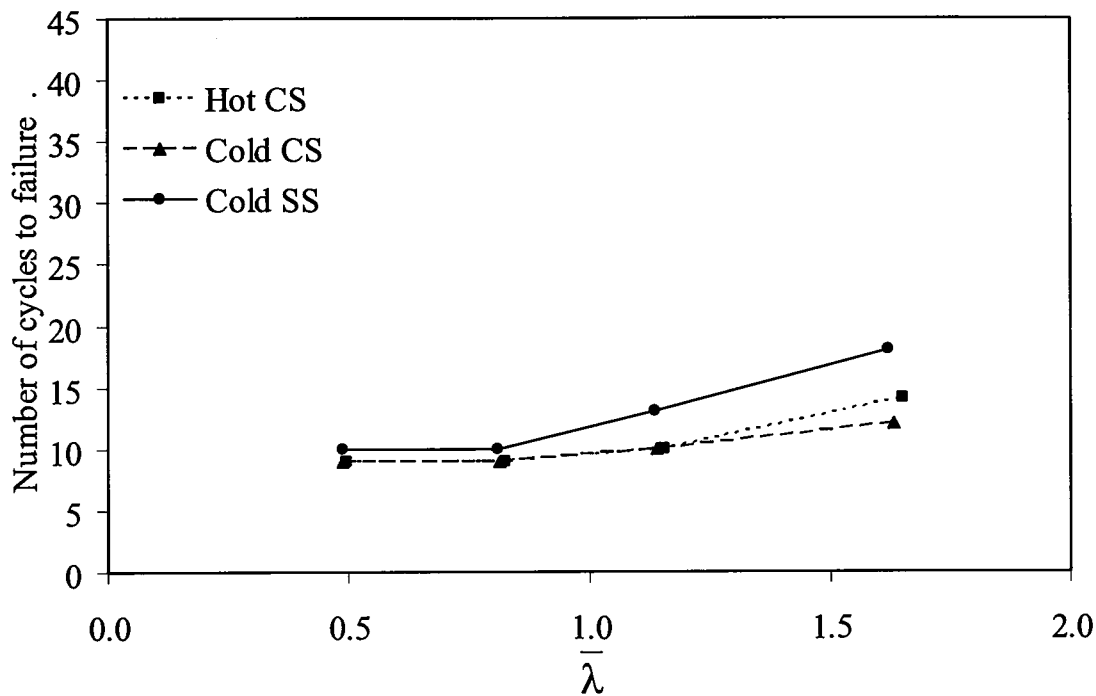


Figure 6.25: Number of cycles to failure for FE specimens with 3.0 mm thick sections

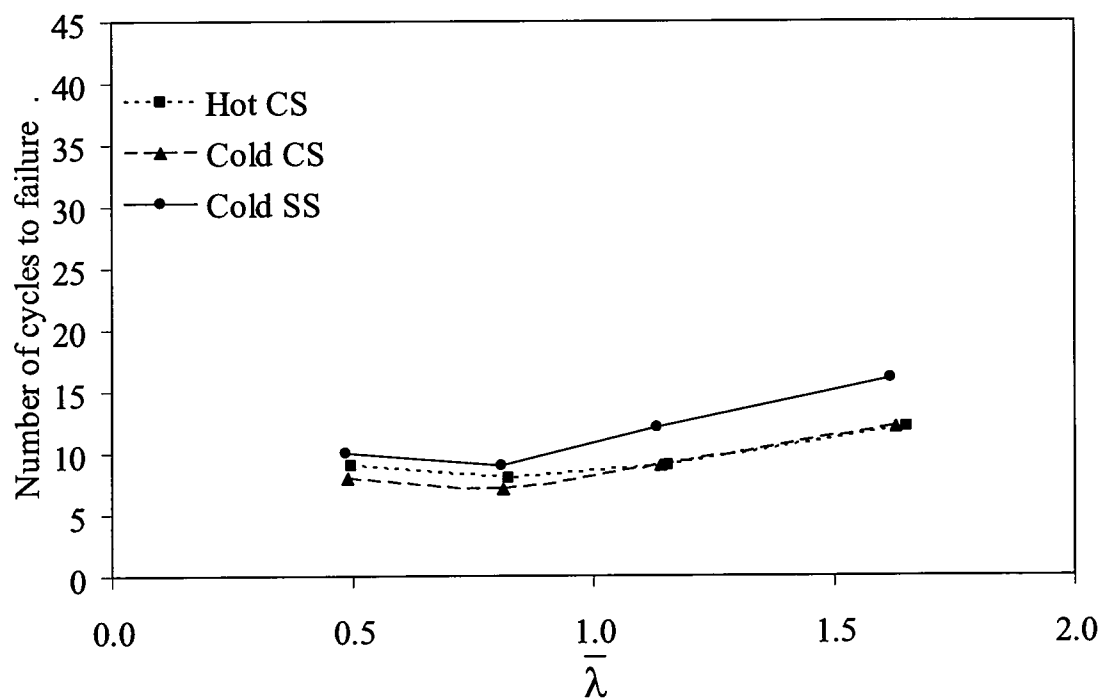


Figure 6.26: Number of cycles to failure for FE specimens with 2.5 mm thick sections

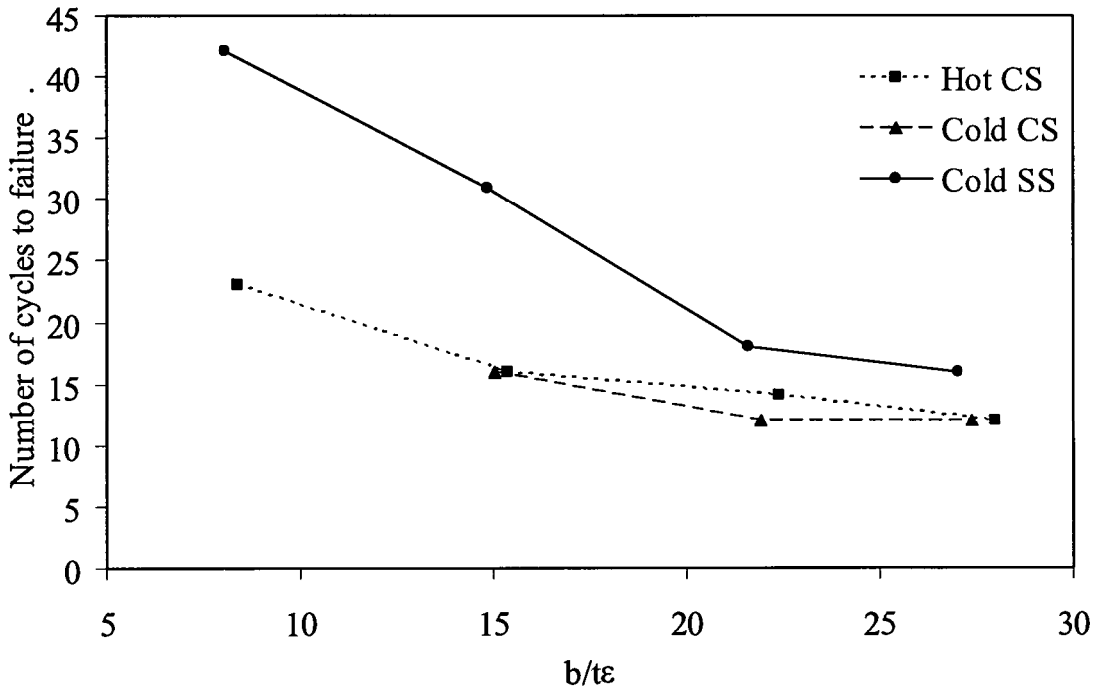


Figure 6.27: Number of cycles to failure for 5000 mm long FE specimens

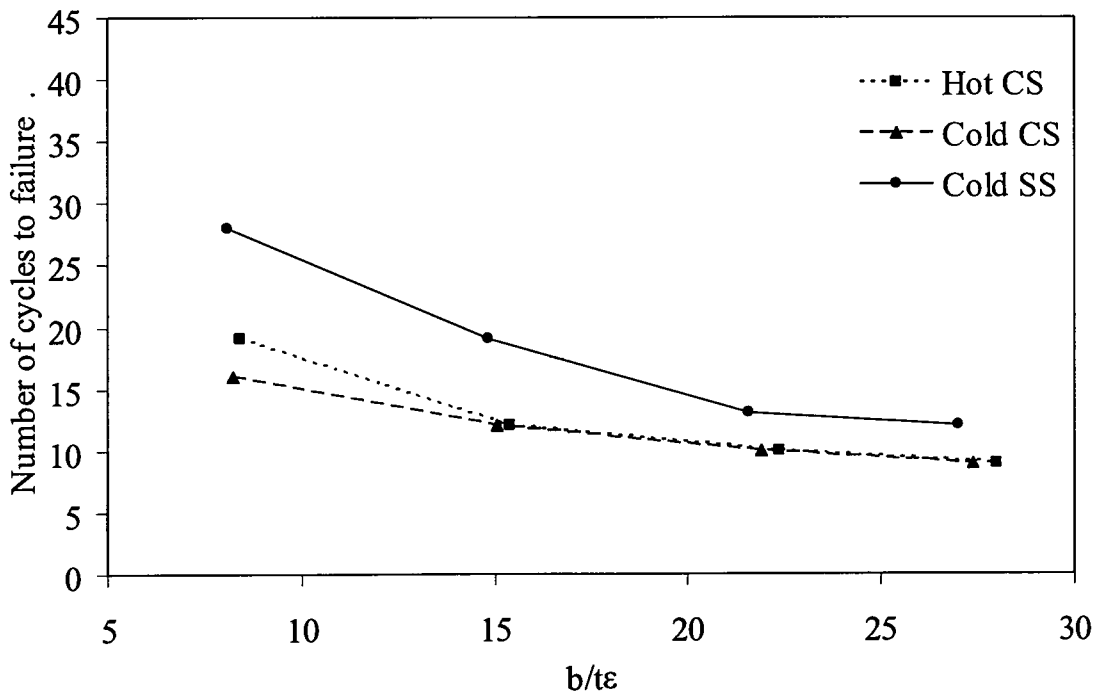


Figure 6.28: Number of cycles to failure for 3500 mm long FE specimens

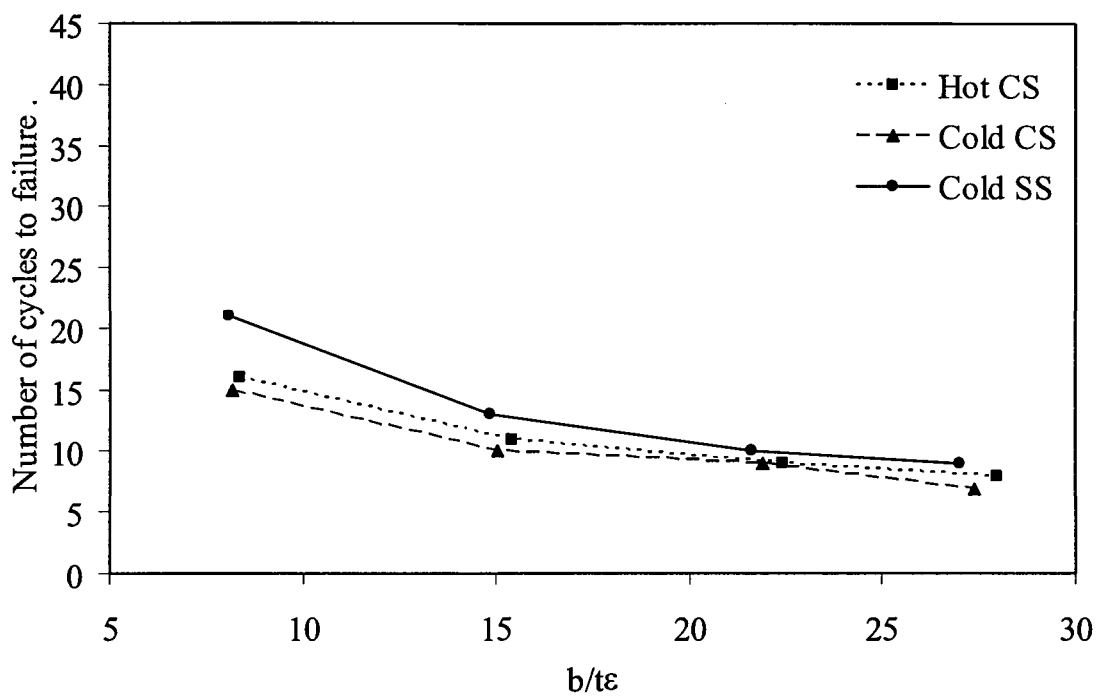


Figure 6.29: Number of cycles to failure for 2500 mm long FE specimens

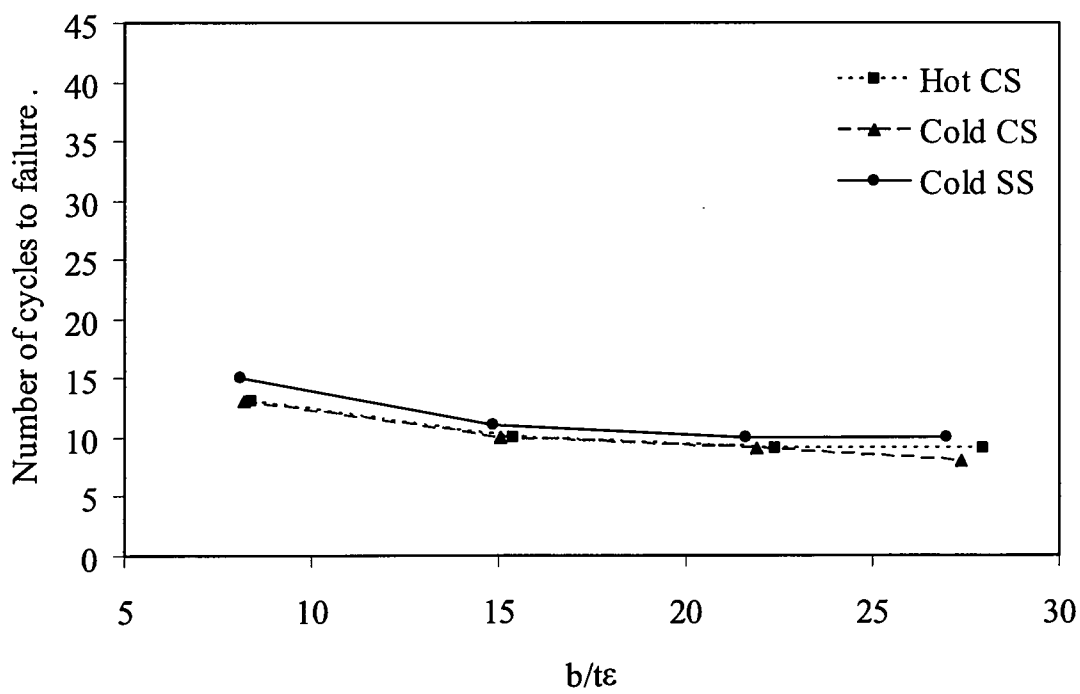


Figure 6.30: Number of cycles to failure for 1500 mm long FE specimens

Empirical equations have been proposed to predict the fracture life of braces from test results on cold-formed carbon steel bracing members and occasionally together with hot-rolled carbon steel bracing members. Although many of these equations have not been specifically developed for hot-rolled carbon steel and cold-formed stainless steel members, these equations have been compared with the experimental and FE results of specimens of the three materials from the current study, in addition to the data available in the literature (Walpole, 1996; Goggins et al., 2006). Hence, the applicability of the equations to more materials and over a larger range of slenderness can be assessed.

A cumulative measure of ductility takes into account the characteristics of the loading history prior to failure. Shaback and Brown (2003) define the accumulated fracture ductility (δ_f) as the weighted sum of normalised compressive (δ_1) and tensile deformation (δ_2) (Figure 6.31):

$$\delta_f = \sum (0.1\delta_1 + \delta_2) \quad (6.14)$$

Predictive equations have been calibrated by tests on cold-formed carbon steel square hollow section bracing members (Shaback & Brown, 2003; Archambault et al., 1995) and are given as:

$$\delta_f = C_s \frac{(350/f_y)^{-3.5}}{[(b-2t)/t]^{1.2}} \left(\frac{4(b/d)-0.5}{5} \right)^{0.55} (70)^2 \quad \text{for } \frac{KL}{r} < 70 \quad (6.15)$$

$$\delta_f = C_s \frac{(350/f_y)^{-3.5}}{[(b-2t)/t]^{1.2}} \left(\frac{4(b/d)-0.5}{5} \right)^{0.55} \left(\frac{KL}{r} \right)^2 \quad \text{for } \frac{KL}{r} > 70 \quad (6.16)$$

where C_s is an empirical constant taken as 0.065, K is the effective length factor, b is the longer face of the cross-section, d is the shorter face of the cross-section, t is the thickness of cross-section, r is the radius of gyration and f_y is the material yield strength in N/mm^2 . The predicted values of accumulated fracture ductility are generally similar to the experimental and FE results only with some exceptions for specimens exhibiting large accumulated fracture ductility. However, this prediction method tends to overestimate the ductility of cold-formed stainless steel experimental specimens and underestimates the corresponding FE specimens (Figure 6.32). This contradiction indicates a high sensitivity of accumulated fracture ductility to the shape of the hysteretic loops.

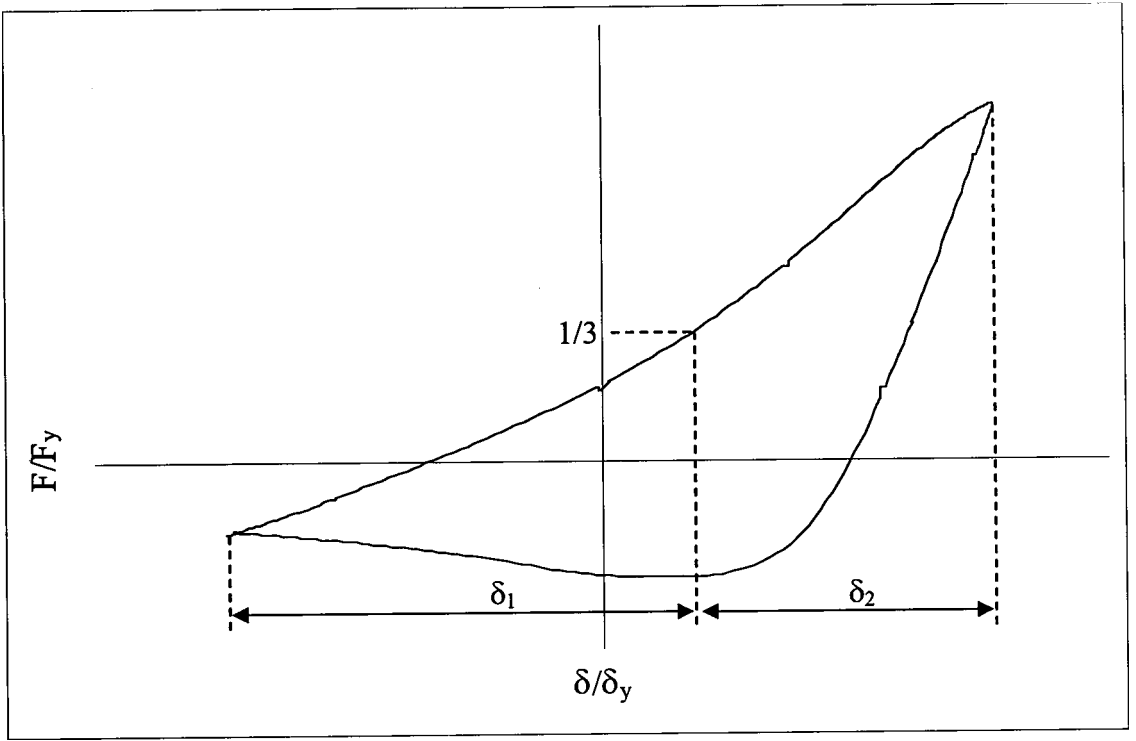


Figure 6.31: Determination of accumulated fracture ductility

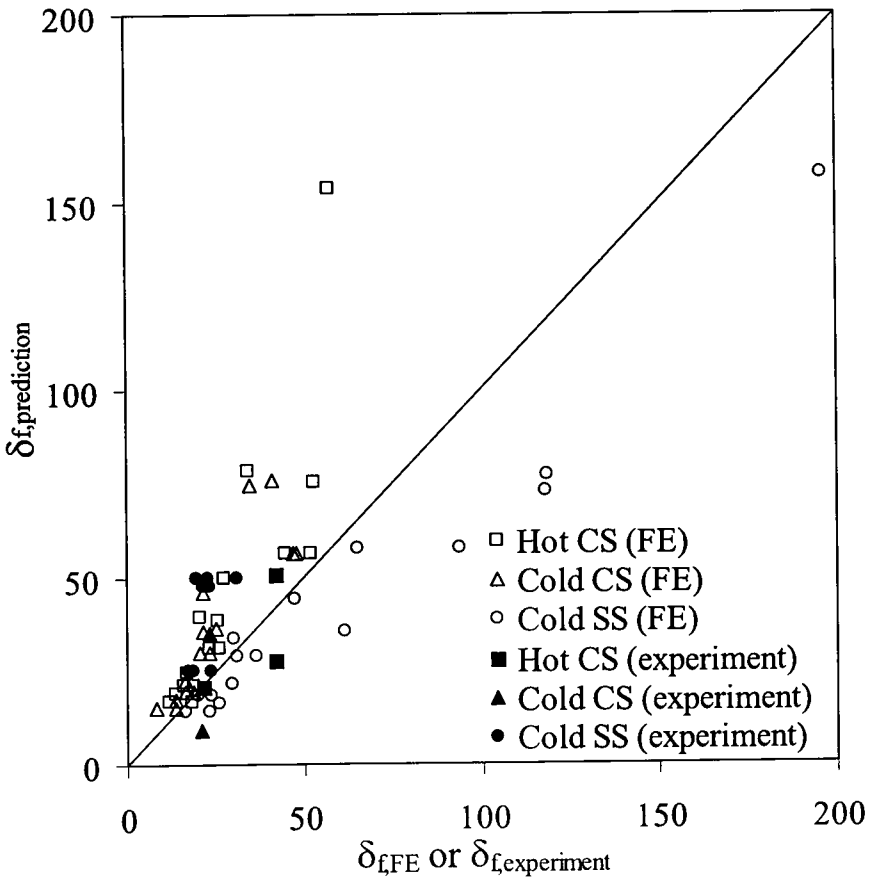


Figure 6.32: Comparison between the predicted values of accumulated fracture ductility by Shaback and Brown (2003) and the values obtained from FE models and experiments

A prediction method independent of the shape of hysteretic loops is more desirable because it gives the ductility of bracing members without requiring detailed knowledge of the load-displacement response. Such a relationship between displacement ductility (μ_Δ) and global slenderness has been established from tests conducted with a number of loading histories (Tremblay, 2002):

$$\mu_\Delta = 2.3 + 8.3\bar{\lambda} \quad (6.17)$$

Goggins et al. (2006) noted that both global slenderness and local slenderness affected the displacement ductility of bracing members. Equation 6.18 was re-calibrated with their test data and a new relationship expressing displacement ductility in terms of local slenderness was proposed (Equation 6.19):

$$\mu_\Delta = -0.7 + 26.2\bar{\lambda} \quad (6.18)$$

$$\mu_\Delta = 29.1 - 1.07(b/t) \quad (6.19)$$

A comparison between these equations and results from the current study shows that these relationships generally overestimate the displacement ductility and do not fit the derived test data over the full range of global and local slenderness (Table 6.3).

In light of the co-existing influence of global slenderness and local slenderness, Tremblay et al. (2003) proposed an expression for rotation at fracture, θ_f , in terms of both slenderness parameters:

$$\theta_f = 0.091\left(\frac{b}{t}\frac{d}{t}\right)^{-0.1}\left(\frac{KL}{r}\right)^{0.3} \quad (6.20)$$

where b/t is the section width-to-thickness ratio, d/t is the section depth-to-thickness ratio, KL is the effective length and r is the radius of gyration. In cases where rotation at fracture has not been measured, it has to be obtained from the plastic hinge model (Tremblay, 2002), which has been shown to overestimate the lateral deflections of the braces in Section 6.2.2. Consequently, although Equation 6.20 generally provides better predictions than Equations 6.17-6.19, it tends to underestimate the rotation at fracture implied from the experiments. (Figure 6.33)

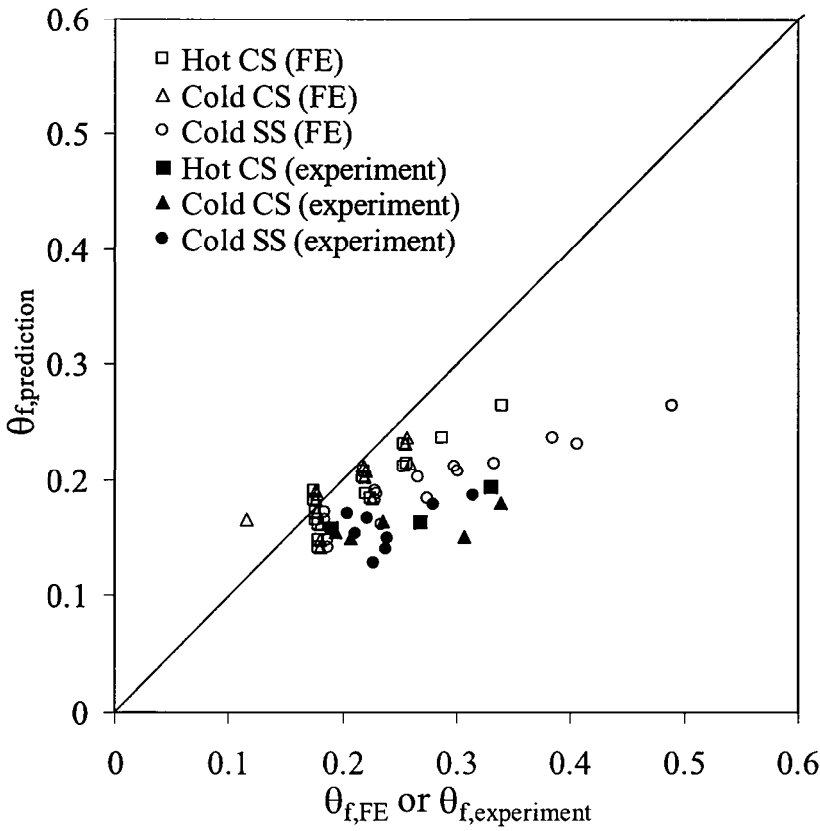


Figure 6.33: Comparison between the predicted values of rotation at fracture by Tremblay et al. (2003) and the values obtained from FE models and experiments

In contrast, since displacement ductility is easily measured and has been reported in most experimental programmes, it is a more convenient evaluation of member ductility. Also, an equation of a plane is a suitable form of expression because it represents the co-existing linear variation between displacement ductility and the two slenderness parameters, as shown by experimental and numerical results. Predictive expressions for displacement ductility which address the co-existing influence of global slenderness and local slenderness for braces of each material are therefore proposed based on the results of this study as follows:

$$\text{Hot-rolled carbon steel:} \quad \mu_{\Delta} = 3.69 + 6.97\bar{\lambda} - 0.05(b/t\epsilon) - 0.19\bar{\lambda}(b/t\epsilon) \quad (6.21)$$

$$\text{Cold-formed carbon steel:} \quad \mu_{\Delta} = 6.45 + 2.28\bar{\lambda} - 0.11(b/t\epsilon) - 0.06\bar{\lambda}(b/t\epsilon) \quad (6.22)$$

$$\text{Cold-formed stainless steel:} \quad \mu_{\Delta} = -3.42 + 19.86\bar{\lambda} + 0.21(b/t\epsilon) - 0.64\bar{\lambda}(b/t\epsilon) \quad (6.23)$$

Table 6.3 shows the comparison between the predicted values and the FE or experimental results for Equations 6.17-6.19 and 6.21-6.23, which indicates that the

new expressions improve the predictions of displacement ductility and that separate expressions for each material are necessary. The new relationships predict the displacement ductility accurately for members with a range of ductilities (Figure 6.34). Different coefficients for Equations 6.21-6.23 may arise if only experimental or numerical results are used for calibration, which suggests that the applicability of these relationships is limited by the data used for calibration. Therefore, all valid experimental and numerical results in the current studies, together with the available data in the literature (Walpole, 1996; Goggins et al., 2006), have been used to calibrate these relationships, such that they are applicable to members with wide ranges of global slenderness and local slenderness. Although a number of factors have been identified as influential to the fracture behaviour of bracing members, the proposed predictive equation (Equations 6.21-6.23) provides a simple means of determining displacement ductility on the basis of global and local slenderness.

Table 6.3: Comparison between predicted displacement ductility by various equations and displacement ductility obtained from FE models or experiments

References	Equation	Prediction / FE or Prediction / test					
		Hot CS		Cold CS		Cold SS	
		Mean	COV	Mean	COV	Mean	COV
Tremblay (2002)	$\mu_{\Delta} = 2.3 + 8.3\bar{\lambda}$	1.80	0.34	1.95	0.45	(1.55)	(0.33)
Goggins et al. (2006)	$\mu_{\Delta} = -0.7 + 26.2\bar{\lambda}$	(3.08)	(0.53)	3.15	0.65	(2.42)	(0.47)
	$\mu_{\Delta} = 29.1 - 1.07(b/t)$	(2.55)	(0.31)	2.76	0.36	(2.08)	(0.35)
Current study	$\mu_{\Delta} = 3.69 + 6.97\bar{\lambda} - 0.05(b/t\epsilon) - 0.19(\bar{\lambda})(b/t\epsilon)$	1.03	0.18	-	-	-	-
	$\mu_{\Delta} = 6.45 + 2.28\bar{\lambda} - 0.11(b/t\epsilon) - 0.06(\bar{\lambda})(b/t\epsilon)$	-	-	1.10	0.30	-	-
	$\mu_{\Delta} = -3.42 + 19.86\bar{\lambda} + 0.21(b/t\epsilon) - 0.64(\bar{\lambda})(b/t\epsilon)$	-	-	-	-	1.02	0.21

* Values in brackets indicate that the equation has not been derived from a database consisting of specimens of that material.

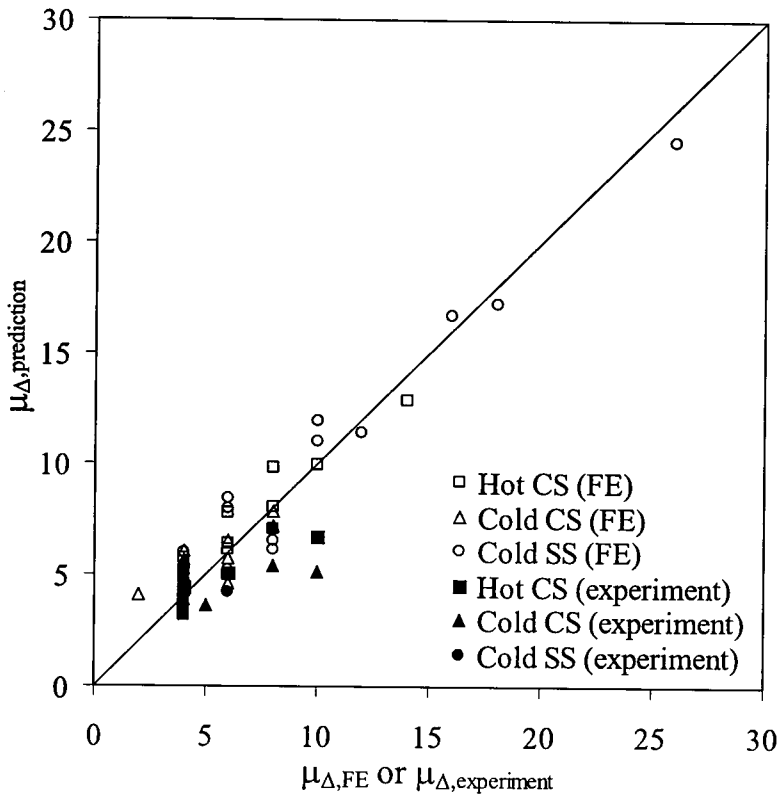


Figure 6.34: Comparison between the predicted values of displacement ductility from the proposals in the current study and the values obtained from FE models and experiments

The trends of displacement ductility predicted by Equations 6.21-6.23 are given in Figure 6.35. Members with a cross-section slenderness $b/t\epsilon = 30$, which is close to the upper limit of Class 1 cross-sections, exhibit a relatively low level of ductility regardless of the global slenderness or the constituent material. Stainless steel members with stocky cross-sections and high global slenderness show higher ductility than their carbon steel counterparts. The hot-rolled carbon steel bracing members show higher ductility in comparison with the cold-formed carbon steel members, particularly for relatively high $\bar{\lambda}$ and low $b/t\epsilon$. This difference diminishes however at low $\bar{\lambda}$ values and high $b/t\epsilon$ proportions. These trends are shown clearly in Figures 6.36-6.37.

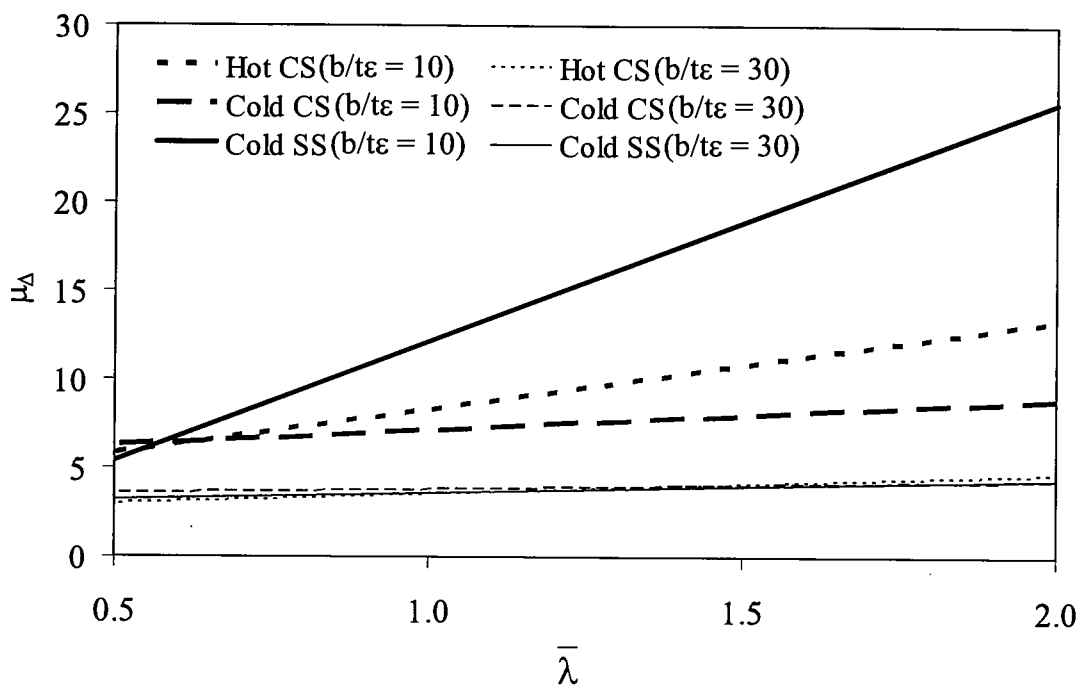


Figure 6.35: Variation of displacement ductility against member slenderness of bracing members with cross-section of $b/t\epsilon = 10$ and $b/t\epsilon = 30$ for the three materials

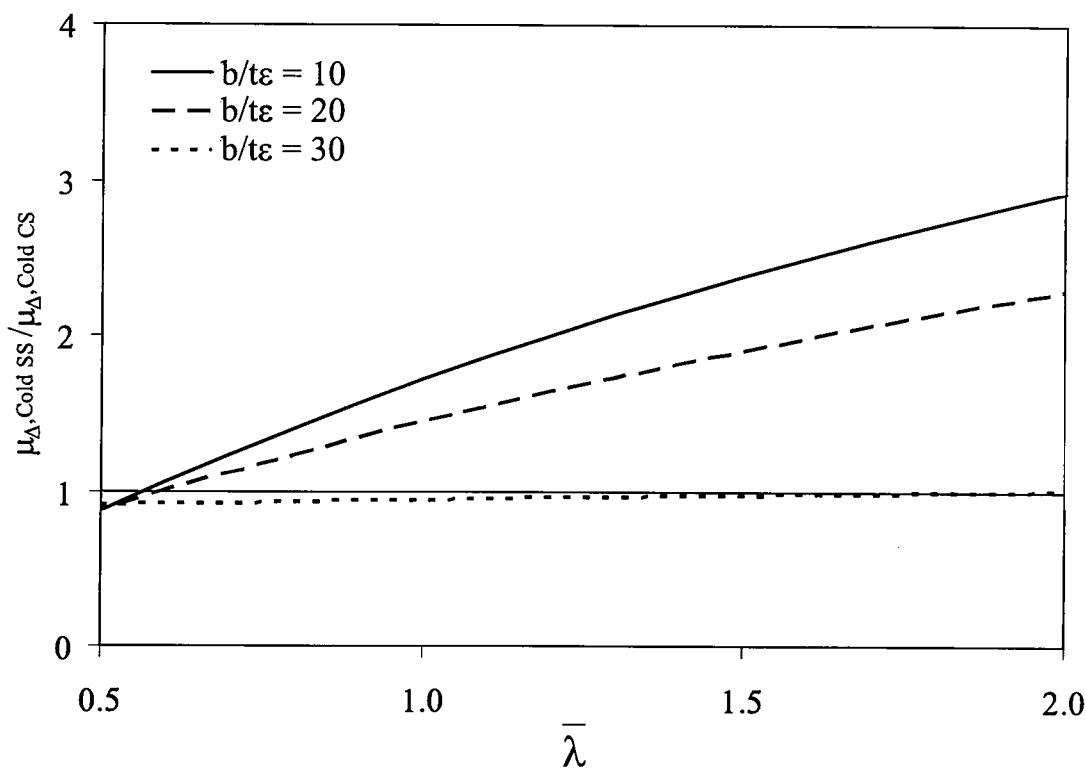


Figure 6.36: Comparison between the variation of displacement ductility of bracing members of the three materials

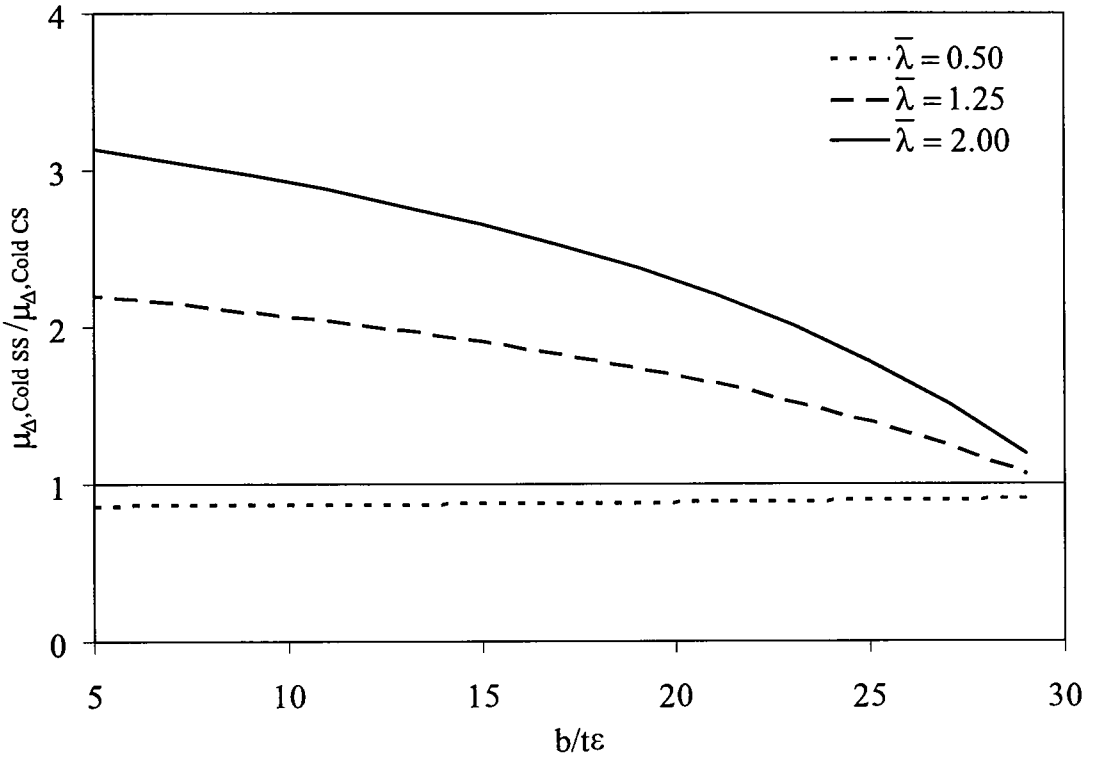


Figure 6.37: Comparison between the variation of displacement ductility of bracing members of the three materials

Since strain localisation due to local buckling is a significant contributor to the initiation of fracture, the difference in number of cycles to failure between different members can be explained by examining the factors that influence local buckling. The occurrence of local buckling of a plated element of a hollow section member depends on the tangent stiffness of the material, which changes at different stages of loading. Figures 6.38-6.39 show a comparison of the stress-strain hysteretic behaviour of the three materials subjected to cyclic axial loading at strain amplitudes of $\pm 1\%$ and $\pm 3\%$ respectively. The details of these tests have been reported in Chapter 3. In Figure 6.40, cold-formed stainless steel shows higher compressive stiffness than hot-rolled and cold-formed carbon steel except for some points on the elastic unloading branch after load reversal. In Figure 6.41, cold-formed stainless steel has similar compressive stiffness to hot-rolled carbon steel while cold-formed carbon steel has lower stiffness.

For slender members and all members with stocky cross-sections, stainless steel exhibited a higher number of cycles to fracture than carbon steel. Strains experienced at the critical cross-sections of these members following global buckling were in the

region of $\pm 1\%$. As shown in Figure 6.40, the tangent compressive stiffness at zero strain of cold-formed stainless steel is approximately 54000 N/mm^2 , which is twice of the value for cold-formed carbon steel. Stainless steel material maintains higher compressive stiffness at this strain amplitude and therefore delays local buckling and hence fracture. The difference between stainless steel and carbon steel is greatest for members with high global slenderness or low local slenderness. Strains in the critical regions of members with low global slenderness or high local slenderness are higher, at around $\pm 3\%$. At this strain amplitude, the stiffness of the three materials is similarly low, as shown in Figure 6.41 where the tangent compressive stiffness at zero strain of the three materials is below 1% of the elastic stiffness, and hence local buckling is more readily triggered, resulting in a similar number of cycles to failure for all three materials.

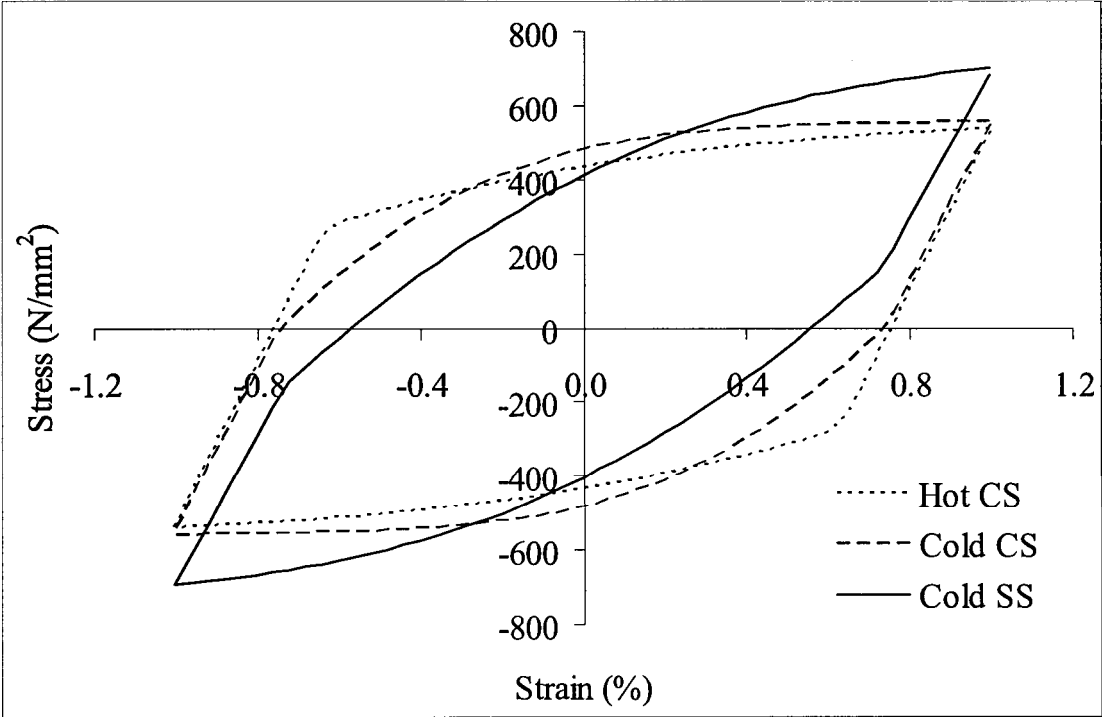


Figure 6.38: Stress-strain hysteresis behaviour of the three materials subjected to cyclic axial loading at $\pm 1\%$ strain amplitude

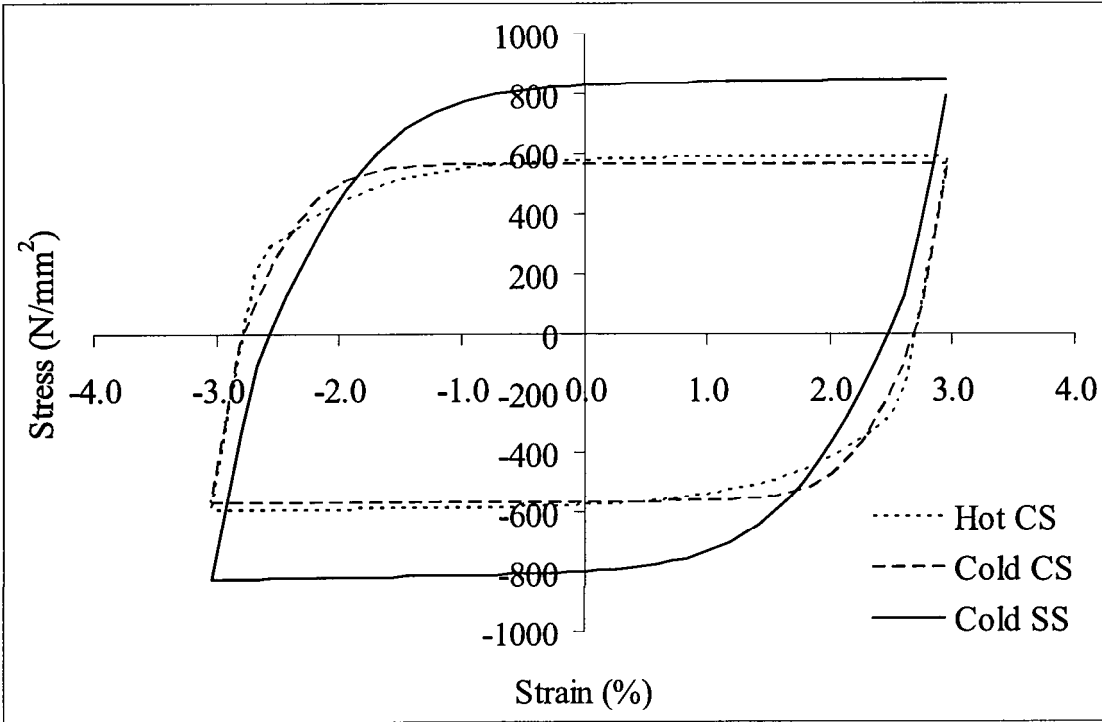


Figure 6.39: Stress-strain hysteresis behaviour of the three materials subjected to cyclic axial loading at $\pm 3\%$ strain amplitude

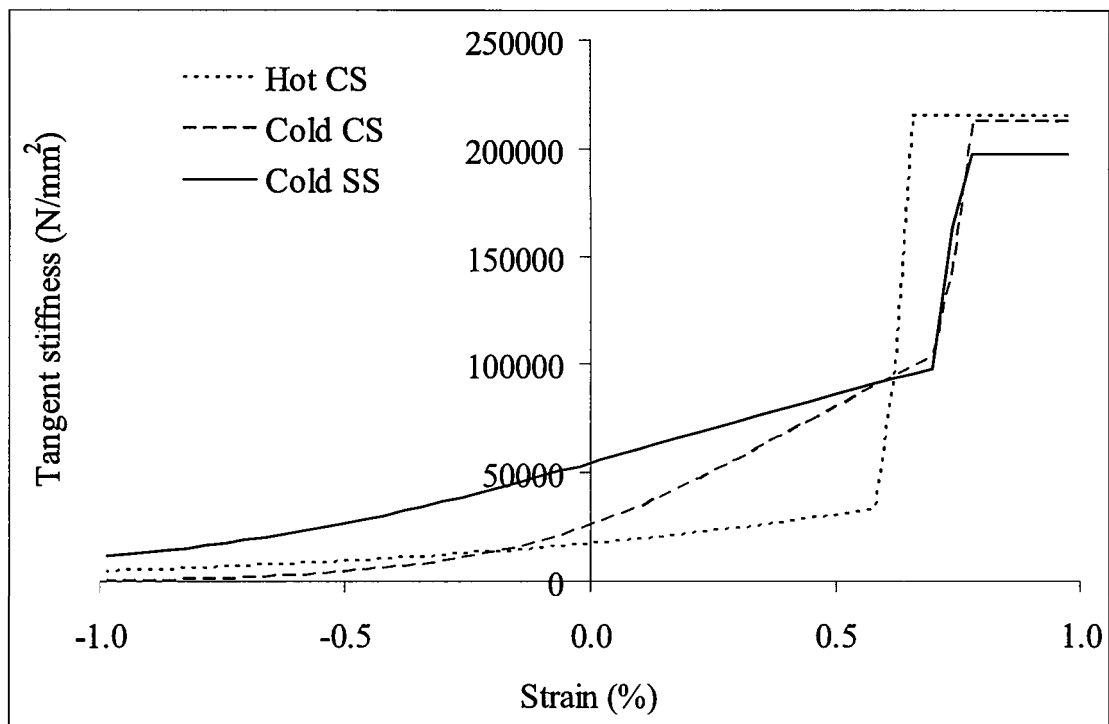


Figure 6.40: Tangent stiffness of the three materials subjected to cyclic axial loading at $\pm 1\%$ strain amplitude

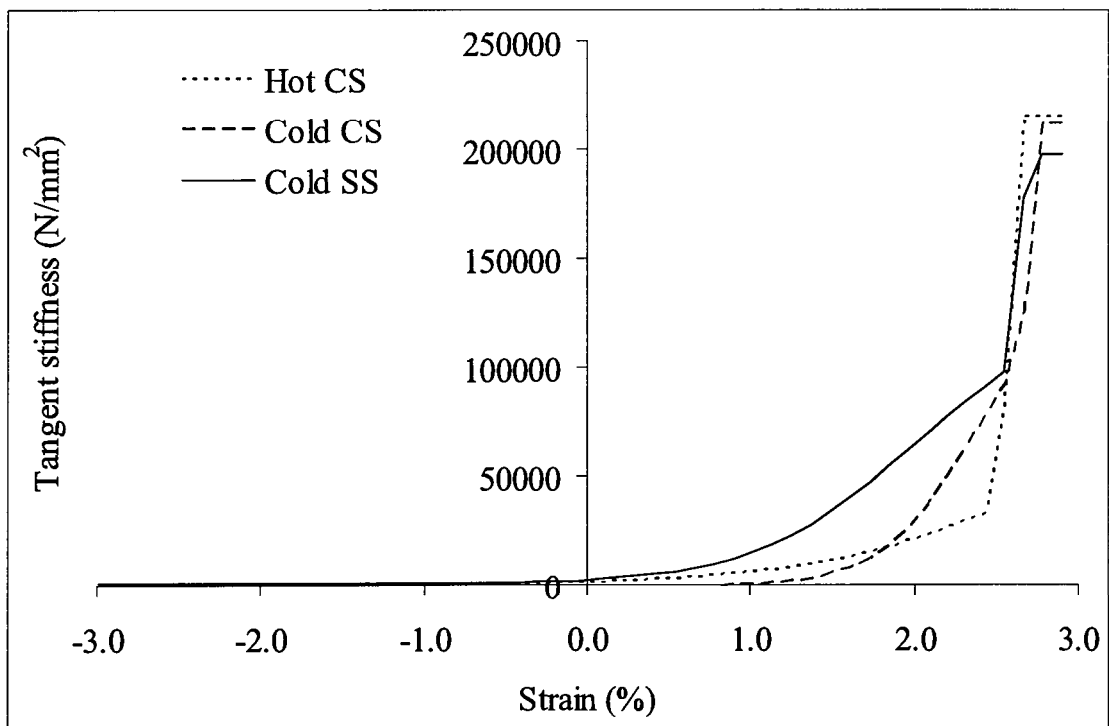


Figure 6.41: Tangent stiffness of the three materials subjected to cyclic axial loading at $\pm 3\%$ strain amplitude

6.3.2 *Energy dissipation and hysteretic loops*

Normalised hysteretic energy is obtained by calculating the area under load-displacement hysteretic loops with normalisation by yield load and yield displacement, respectively. It was found that the areas under the hysteretic loops of models of the whole ranges of global and local slendernesses of the three materials were similar for all three materials and over the investigated ranges of both local and global slenderness; hence the accumulated energy dissipation of the three materials follows a similar trend (Figure 6.42). However, the stainless steel members with stocky cross-sections or high global slenderness withstand significantly more cycles than their carbon steel counterparts and thus the total energy dissipation to fracture is higher. The same also applies, but to a lesser extent, to the higher energy dissipation in the hot-rolled carbon steel members in comparison with the cold-formed carbon steel specimens. This is attributed to the reduced ductility at the corners caused by the cold-forming process.

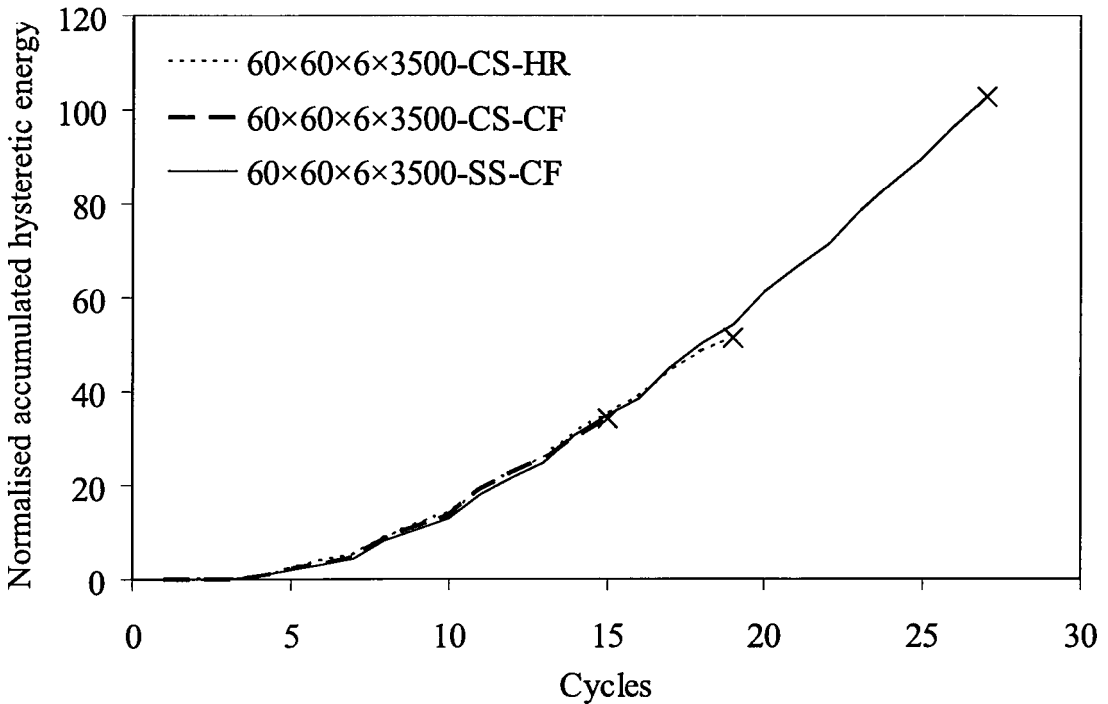


Figure 6.42: *Normalised accumulated hysteretic energy for 60×60×6×3500 FE specimens of the three materials*

The shapes of the hysteretic loops of the models of the three materials are different although their areas are approximately the same. Figures 6.43 and 6.44 show the hysteretic loops of the 8th and 9th cycle for the 60×60×6×3500 models of the three materials respectively. These represent the shapes of typical hysteretic loops during the first and subsequent cycles of loading at a certain applied displacement. The stainless steel specimen develops higher tensile resistance in both cycles due to cyclic hardening of the material. Figures 6.45 and 6.46 show the tangent tensile stiffness of the braces during these two cycles respectively. In the 8th cycle, the peak tangent tensile stiffness reached by the three specimens is similar but the stainless specimen maintains a higher tangent stiffness while the two carbon steel specimens exhibit a load plateau at large tensile displacement. During the next cycle, the peak tangent stiffness of the stainless steel specimen does not deteriorate as much as that of the carbon steel specimens. Also, the peak tangent stiffness develops at a smaller tensile displacement than the carbon steel specimens. These are crucial characteristics that influence the design of concentrically braced frames, since bracing members acting in tension are the main source of stiffness in concentrically braced frames.

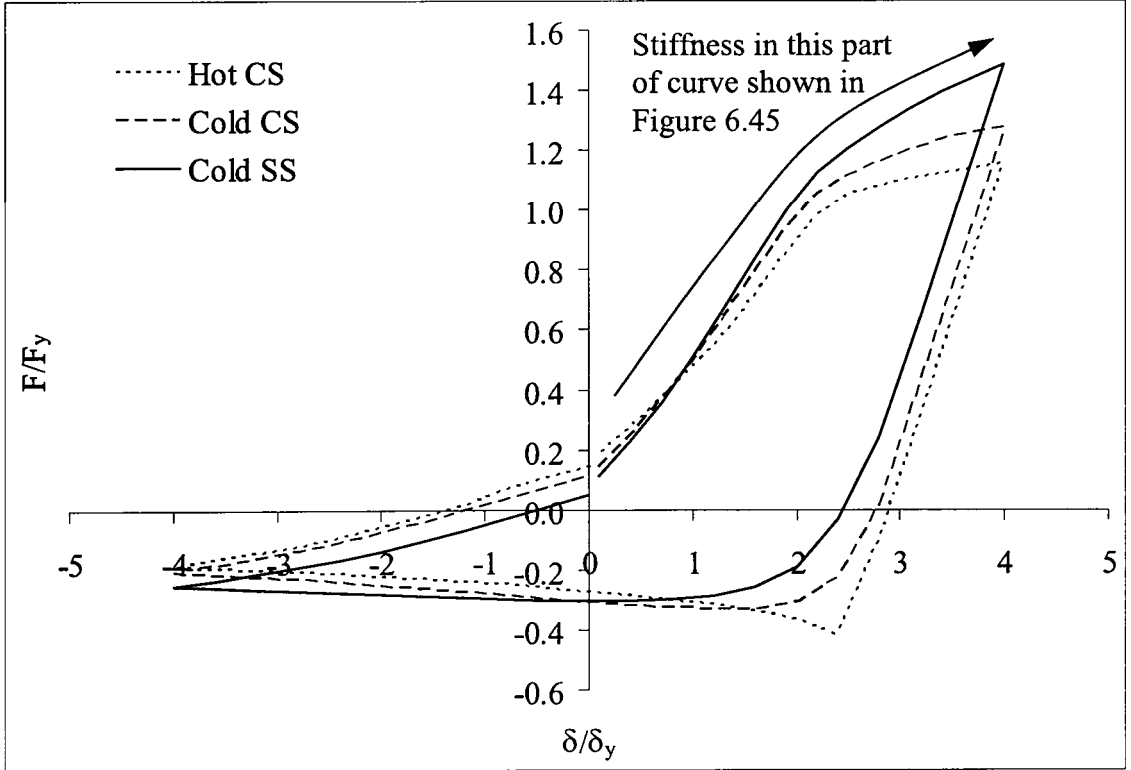


Figure 6.43: Normalised load-displacement hysteretic loops of the 8th cycle of 60×60×6×3500 models of the three materials

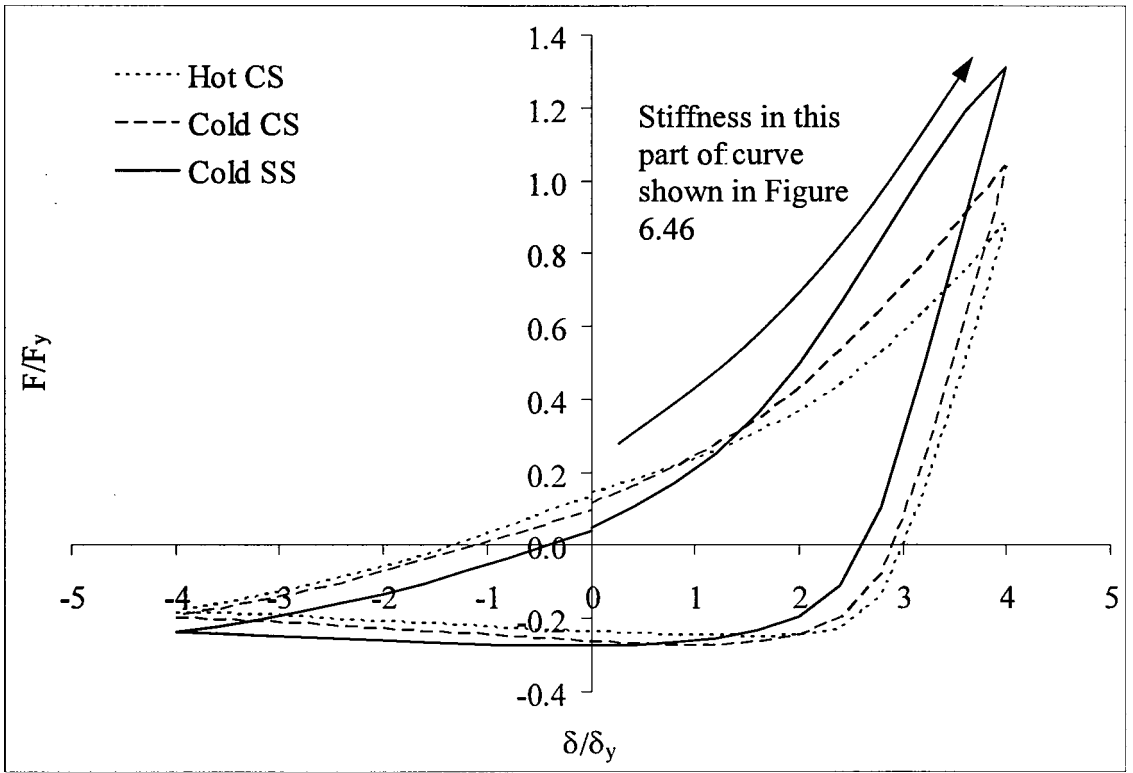


Figure 6.44: Normalised load-displacement hysteretic loops of the 9th cycle of $60 \times 60 \times 6 \times 3500$ models of the three materials

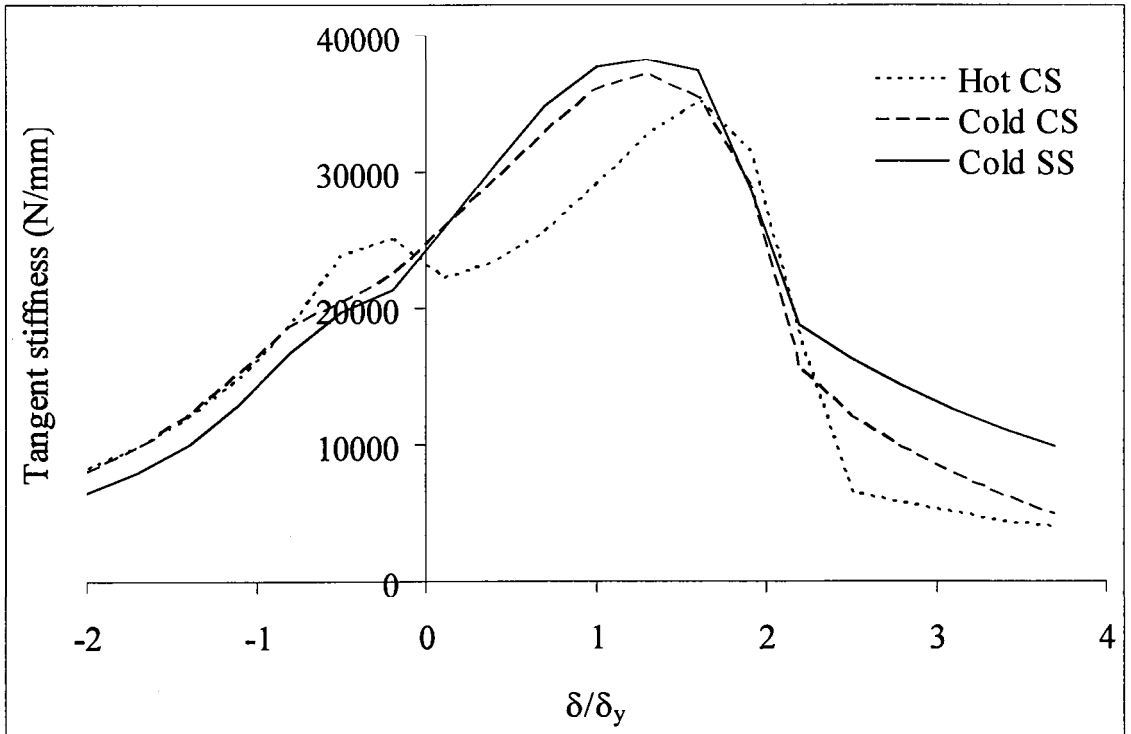


Figure 6.45: Tangent stiffness during the 8th tensile loading half cycle for $60 \times 60 \times 6 \times 3500$ FE specimens of the three materials

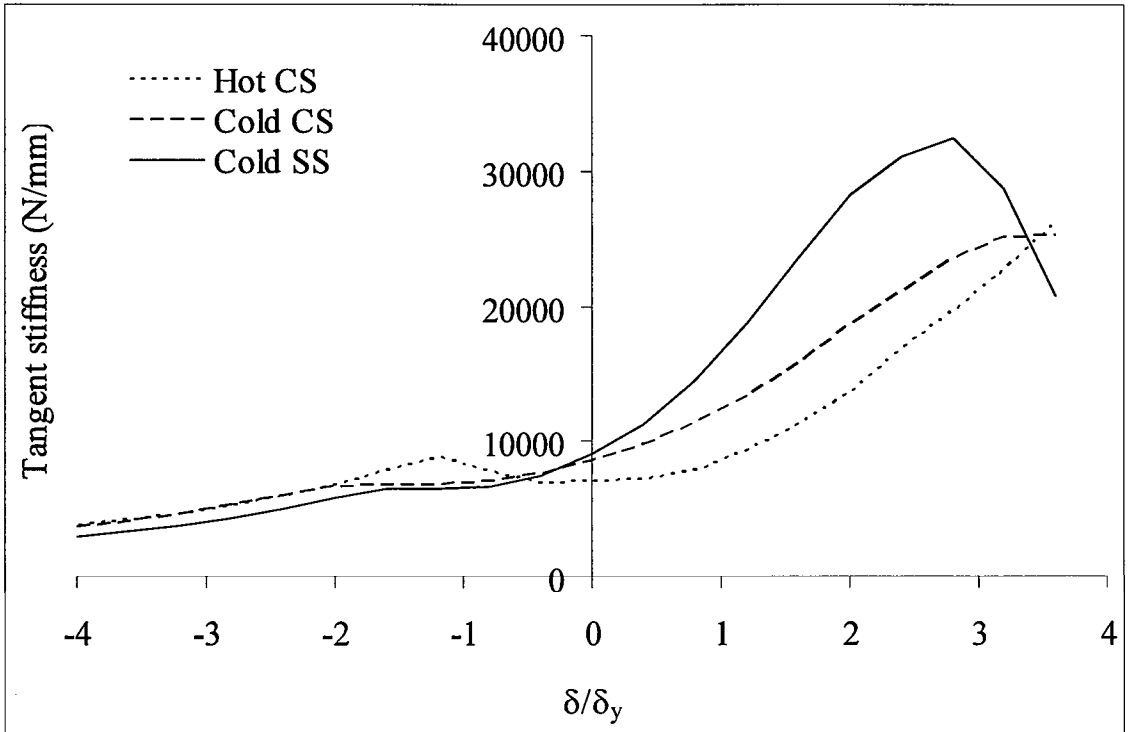


Figure 6.46: *Instantaneous stiffness during the 9th tensile loading half cycle for 60×60×6×3500 FE specimens of the three materials*

6.4 Concluding remarks

Two series of parametric studies were presented in this chapter. The first series of studies were conducted to investigate the influence of global and local slendernesses on the number of cycles to failure. Local buckling was found to be crucial to the promotion of fracture since it induces high localised strains at the corner areas of the sections, which exhibit lower ductility material. Braces with low local slenderness have high resistance to local buckling and hence delay fracture. Global slenderness of the braces through various influencing factors, affects the compressive strains before the occurrence of local buckling and the severity of the buckling once it has initiated. Due to the combined action of the factors and axial shortening of the braces leading to deviation from the ideal cases considered, the trend of the variation of the number of cycles to fracture with global slenderness may not be obvious in certain cases. However, in general, both global and local slendernesses were influential to the number of cycles to failure of the braces.

Observations from the current study have highlighted some inconsistencies with conclusions found in the literature, where fracture life has been deemed to increase with global slenderness but was often found not to vary significantly with local slenderness. The apparent contradiction with the findings of the current study is due to the difficulty in isolating the effects of global and local slendernesses in some studies. In some studies. An alternative selection of specimens to confine changes to only one variable at one time in other programmes (Walpoe, 1996; Han et al., 2007) gives a clearer view on the effects of each of these factors. A relationship which addresses the co-existing influence of global slenderness and local slenderness has been proposed in this study.

In the second series of parametric studies, the performance of braces of the three materials was compared. Stainless steel members were found to be capable of sustaining more cycles of loading than carbon steel members prior to fracture, especially for stocky sections and members with high global slenderness. In comparison with cold-formed carbon steel, hot-rolled specimens were relatively more ductile particularly for high global slenderness and low local slenderness. The empirical expressions, derived from carbon steel test data, for predicting the buckling resistance, compressive post-buckling resistance and mid-length lateral deflection have been found to be generally applicable to cold-formed stainless steel members. However, due to the difference in member ductility and inclusion of data of specimens of a larger range of slenderness, previous relationships between member ductility and slenderness do not give satisfactory predictions of the experimental and FE results obtained in the current study. Expressions of displacement ductility in terms of both global slenderness and local slenderness have been calibrated for braces of each material. These relationships accurately reflect the influence of slenderness and material properties on the ductility of the braces.

The level of energy dissipation of members of the three materials at the same amplitude of loading were shown to be similar, although higher total energy dissipation to failure of stainless steel members was found owing to the higher number of cycles to failure. For carbon steel members, the energy was relatively higher in hot-rolled than cold-formed specimens. The most significant and crucial difference between the materials were the tangent tensile stiffness of the members. Stainless steel specimens and FE

models exhibited higher stiffness than those of the two other materials. These conclusions can have implications on the overall structural response of framed structures.

Chapter 7

Design Implications

7.1 Introduction

Diagonal bracing members are the main dissipative elements and primary source of lateral stiffness of seismic resistant concentrically braced frames. Their stiffness, strength and ductility, among other characteristics, are crucial to the performance of the frames in seismic events. Sufficient strength of bracing members ensures elastic response to avoid structural damage under relatively small, frequent events. Other frame members are dimensioned accordingly to satisfy capacity design requirements. The stiffness of the bracing members controls the extent of lateral deflection to minimise non-structural damage. It also determines the natural period of the frames which in turn is directly related to level of seismic forces induced in the structure under extreme earthquakes, bracing members are expected to yield and provide adequate ductility. Other frame members and components are dimensioned accordingly to satisfy capacity design requirements. All of these properties have direct implications on the seismic design for concentrically braced frames (Elghazouli, 2003).

Similar to the current study, most experimental studies reported in the literature are based on quasi-static cyclic tests on individual bracing members under predefined displacement histories. These tests provide a well-controlled means for assessing the hysteretic behaviour of bracing members of practical size and proportions. This behaviour typically needs to be verified against realistic dynamic loading conditions

representative of seismic events, particularly with respect to the interaction between braces and other frame members. In order to address this issue, rectangular hollow bracing members in diagonal (Figure 2.3 (a)) and X-bracing (Figure 2.3 (d)) configurations have been tested by Tremblay et al. (2003). These tests validated factors to use in determining the member slenderness for the design of X-bracing members. The actual conditions, in terms of interaction between tension and compression braces, were more closely simulated by shake table testing with appropriately scaled acceleration histories from real earthquakes (Elghazouli et al., 2005). These tests highlighted the implications of asymmetric response and actual dynamic effects on the strength and ductility of the braces.

While braced frames incorporating carbon steel members have been extensively studied, no experimental data is available on the cyclic behaviour of stainless steel braces and frames. Di Sarno et al. (2008) carried out numerical studies on braced frames to assess the benefits of using stainless steel members in various parts of the structure. Inelastic static (pushover) and dynamic (response history) analyses were performed on models using monotonic material properties of stainless steel and cyclic material models. Application of stainless steel braces and columns was found to improve the performance of the frames but there was no benefit in using stainless steel beams. The potential for using stainless steel in earthquake resistant structures was indicated by its excellent energy absorbing capacity and delayed local buckling due to the enhanced material strain hardening.

Based on the results of previous experimental and numerical studies, coupled with the cyclic test data on stainless steel obtained in the current study, the main implications of the characteristics of bracing members on the performance and design of concentrically braced frames are discussed in this chapter.

7.2 *Seismic design concepts*

Current seismic codes, such as EC 8 (EN 1998-1, 2004), set a number of performance targets for earthquake resistant design. Structures have to withstand the design seismic action without partial or total collapse. They also have to resist lower intensity

earthquakes without notable structural damage. Non-structural damage in frequent, small earthquakes should also be within limits determined by use and repair costs.

EC 8 (EN 1998-1, 2004) allows steel structures to be designed according to non-dissipative or dissipative structural behaviour. Non-dissipative structural behaviour requires the structures to respond mainly in the elastic range. In this case, large design forces on structural members would result in uneconomical design except for areas of low seismicity or structures of high importance. A more efficient approach is to allow significant inelastic deformations, or dissipative behaviour, under extreme seismic events. Design codes, such as EC 8 (EN 1998-1, 2004), specify seismic actions on structures in the form of an elastic response spectrum, which depends on ground conditions, probability of occurrence of earthquakes in the site location, natural period of the structure and their importance. Dissipative behaviour of the structure allows the reduction of elastic forces (V_e) obtained from the elastic response spectrum by a behaviour factor (q) in European practice, or force reduction/modification factor (R) in other codes, to arrive at design forces (V_d) (Figure 7.1). The behaviour factor depends on the expected ductility capacity of the structures (Δ_u/Δ_y) (Elghazouli, 2003), which is mainly provided by inelastic deformation of dissipative elements. However, the actual strength of the structures (V_y) may be higher than the design strength, resulting in overstrength (V_y/V_d). The overstrength changes the ductility demand on the dissipative elements and the required strength of other frame members. In the case of concentrically braced frames, bracing members provide the ductility capacity to enable reduction in design force by yielding in tension and buckling in compression. Other frame members are protected from yielding by dimensioning to carry the gravity loads together with the forces corresponding to the maximum strength of the braces.

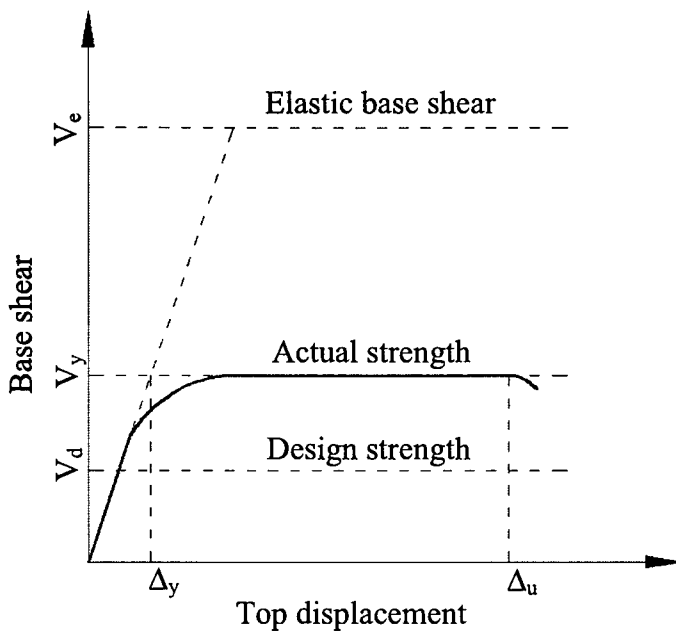


Figure 7.1: Relationship between overstrength, behaviour factor and ductility

7.3 Response of concentrically braced frames

The inelastic response of concentrically braced frames with diagonal bracing members is discussed in this section. To aid the discussion, Figure 7.2 illustrates two idealised single-storey concentrically braced frames (Elghazouli, 2003). Loading cases of other types of concentrically braced frames can be derived from the same principles discussed below.

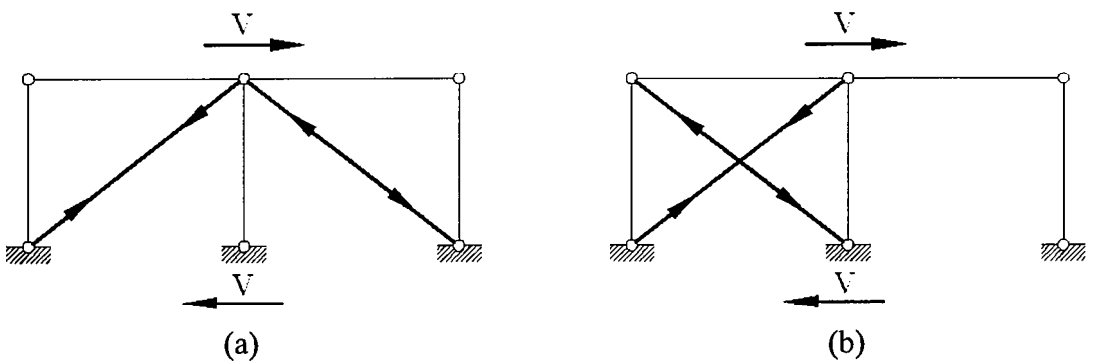


Figure 7.2: Idealised single-storey concentrically braced frames with (a) diagonal bracings; and (b) X-bracings

As the base shear, V , increases, the brace in compression buckles and this is subsequently followed by yielding of the tensile brace. Unless the resistance of the

brace in compression deteriorates rapidly, the base shear and over turning moment of the frame reaches the maximum values at this point, assuming that strain hardening is less significant than the influence of brace buckling. The reactions of the columns at the braced bay of the frame with X-bracings (Figure. 7.2 (b)) are also at the peak values at this moment. EC 8 (EN 1998-1, 2004), which ignores the compressive strength of braces in the calculation of the lateral resistance of frames, recommends a lower limit of slenderness of the braces to avoid failure of the columns caused by the overloaded compressive forces in this critical loading scenario (Elghazouli, 2008).

As the deflection becomes larger, the tensile force in the brace increases due to strain hardening and reaches the maximum tensile strength. This is the maximum force that brace connections and foundation have to resist. Meanwhile, the post-buckling strength of the compression brace reduces significantly. The reaction of the internal column of the diagonally braced frame (Figure. 7.2 (a)) is at the maximum possible value, under the combined force of its tributary gravity load, maximum force in the tension brace and minimum post-buckling force in the compression brace.

By considering these loading scenarios, it can be concluded that the tensile yield strength, maximum tensile strength, initial buckling strength and minimum post-buckling strength of the brace are critical parameters that need to be realistically evaluated.

7.4 Tensile strength

In current design codes, the assessment of tensile strength of a member is based on nominal strength of the material, with appropriate factors applied. In reality, the actual yield strength of a material can be substantially higher than the nominal value due to cold-working, variability of production process and strain rate effects. Overstrength, even though not accurately quantified in most cases, is considered conservative in design for static loads. However, it can cause undesirable effects in seismic design as it may change the failure mode of structures which may result in brittle failure. Therefore, in capacity design checks, the estimated maximum value of material yield strength ($f_{y,max}$) should be used instead of the nominal value ($f_{y,nom}$) unless the actual material

yield strength ($f_{y,act}$) is determined from tests. In EC 8 (EN 1998-1, 2004), the maximum material yield strength is given by:

$$f_{y,max} = \gamma_{ov} f_{y,nom} \quad (7.1)$$

where γ_{ov} is the overstrength factor. If cold-formed members are used, procedures in EC3 Part 1-3 (EN 1993-1-3, 2006) for estimating the effect of cold-forming on the nominal yield strength should be followed if test data on the section tensile strength are not available.

To assess the above-mentioned effect, the average values of nominal yield strength and the actual yield strength of tensile coupons cut from the flat and corner areas of square and rectangular hollow sections tested in the current study are compared in Table 7.1. The actual yield strength is higher than the nominal value for all coupons tested. The discrepancies between the actual and nominal values in cold-worked materials are larger than those in hot-rolled materials, with corner areas in cold-worked sections showing the largest difference. It is also important to note that stainless steel exhibits significantly high material overstrength in comparison with cold-worked carbon steel. The variation in material overstrength, which may lead to unforeseen failure modes, is especially large in cold-formed sections due to the variability in levels of cold-working in different sections.

Table 7.1: Comparison between actual yield and ultimate strengths with nominal yield strengths of the materials

Material and type of coupon		$f_{y,act} / f_{y,nom}$		$f_{u,act} / f_{y,nom}$	
		Mean	C.O.V.	Mean	C.O.V.
Hot CS	Flat	1.31	0.04	1.56	0.02
Cold CS	Flat	1.79	0.22	1.94	0.26
	Corner	2.07	0.07	2.30	0.08
Cold SS	Flat	2.21	0.09	3.36	0.02
	Corner	3.08	0.05	4.00	0.04

Tensile resistance increases beyond yielding of a material because of strain hardening. Stainless steel shows pronounced strain hardening which results in actual ultimate strength ($f_{u,act}$) more than 3 times the nominal yield strength, as shown in Table 7.1. On

the other hand, the ultimate strength of carbon steel material is between 1.56 to 2.30 times higher than the nominal yield strength (Table 7.1).

As discussed before, the actual yield strength is not the only important parameter. The maximum tensile strength ($F_{t,max}$) of a section is also critical. The tensile resistances of the brace specimens (calculated from the actual yield strength and ultimate strength of the materials) are compared with the actual maximum tensile strength observed in the tests conducted in this investigation (Table 7.2). Properties of flat and corner coupons have been weighted according to the proportion of the corresponding areas in the sections. Under cyclic axial loading, the maximum tensile strength of hot-rolled and cold-formed carbon steel sections is on average 12% and 7%, respectively, higher than resistance calculated from the actual yield strength of the material. Cold-formed stainless steel sections exhibit more pronounced strain hardening resulting in maximum tensile strength which is 16% to 30% higher than the calculated values. These maximum values of tensile forces have been observed at applied displacement between $2\delta_y$ and $4\delta_y$, which is the typical range of ductility demand in braced frames.

The ultimate strength of the materials can alternatively be used to calculate the maximum tensile strength of the sections. The results ($F_{t,max} / F_{u,cal}$ in Table 7.2) are within 10% of the actual values for hot-rolled and cold-formed carbon steel sections. However, this method underestimates the resistance of cold-formed stainless steel sections considerably. This is mostly due to the lower strain levels that the brace specimens have experienced in tension compared to the strain at ultimate load in tensile coupon tests.

Due to the significant levels of material overstrength, coupled with large variability in some cases, and the importance of this effect in capacity design, tensile tests on the whole cross-section are recommended to obtain the maximum tensile strength for capacity design. If only the actual yield strength of the flat and corner materials are known, a further strength enhancement factor ranging between 10% and 30% is deemed appropriate for calculating the maximum tensile force in the braces. Otherwise, calculating the maximum tensile strength of braces from the ultimate strength of the materials provides conservative yet reasonable estimation. The only other major source

of material overstrength which has not been considered herein is the effect of strain rate during dynamic loading, which has been shown to increase the yield strength by slightly over 10% (Elghazouli et al., 2005). In light of this, an appropriate strength enhancement factor can be applied accordingly.

Table 7.2: *Comparison between actual tensile resistances and calculated yield and ultimate tensile resistances of the sections*

Specimen identification	$F_{t,max} / F_{y,cal}$	$F_{t,max} / F_{u,cal}$
60x60x3x2050-CS-HR	1.12	0.91
40x40x3x2050-CS-HR	1.11	0.96
40x40x3x1250-CS-HR	1.12	0.96
60x60x3x2050-CS-CF	1.06	0.96
40x40x4x2050-CS-CF	1.09	1.04
40x40x3x2050-CS-CF	1.06	0.95
40x40x3x1250-CS-CF	1.07	0.96
60x60x3x2850-SS-CF	1.29	0.85
50x50x3x2850-SS-CF	1.16	0.81
60x40x3x2850-SS-CF	1.28	0.92
60x60x3x2050-SS-CF	1.28	0.84
50x50x3x2050-SS-CF	1.25	0.87
60x40x3x2050-SS-CF	1.30	0.94
60x60x3x1250-SS-CF	1.28	0.84
50x50x3x1250-SS-CF	1.24	0.87
60x40x3x1250-SS-CF	1.29	0.93

7.5 Compressive strength

Initial buckling strength and post-buckling compressive strength of braces play an important role in the behaviour. Both of them depend on the slenderness of the members and the capacity of the cross-sections. Design codes (e.g. AISC, 2005; EN 1993-1-1, 2006) represent the buckling strength of a column as the plastic section capacity reduced by a factor dependent on global slenderness, type of section and axis

of buckling. The buckling curves in Figure 4.24 present the reduction factors of the three types of hollow sections, which give conservative assessment of buckling strength under monotonic loading. Most initial buckling loads of specimens tested are above the buckling curves, due to material overstrength and strain hardening during the initial small cycles of loading. The suggestion by Elghazouli (2003) of applying a buckling strength enhancement factor with a fixed value between 10% and 20% in capacity design checks is in line with these test results.

Under repeated buckling and tensile yielding, there is a reduction in the compressive strength of a bracing member. The minimum post-buckling strength at a specific ductility demand level represents a critical loading case in some frame configurations, such as that shown in Figure 7.2(a). Based on experimental results, reduction factors to the initial buckling loads have been proposed in order to estimate the residual compressive strength at specific ductility demand levels (Remennikov and Walpole, 1998; Tremblay, 2002; AISC, 2005). Tremblay's (2002) expressions for minimum post-buckling strength at displacements of $2\delta_y$ and $5\delta_y$, which can also be interpolated to obtain values at $4\delta_y$ (Figures 4.26 and 4.27). AISC (2005) reduction factor of 70% provides similar results to Tremblay's (2002) prediction at $4\delta_y$ for intermediate values of $\bar{\lambda}$. Although there is no data at displacement equal to $5\delta_y$ to provide a direct comparison for the curve of Remennikov and Walpole (1998), results show that it is reasonable to use that as a lower bound of compressive strength at large applied displacements.

In general, the minimum post-buckling strength of stainless steel specimens is slightly higher than those of carbon steel specimens but there is no significant difference in trends for the braces made from the three materials. The prediction by Tremblay (2002) at displacements equal to $2\delta_y$ and AISC (2005) prediction at $5\delta_y$ provide conservative estimates of the minimum post-buckling strength for stainless steel specimens but overestimate the strength of some of the carbon steel specimens.

7.6 Ductility considerations

Since providing the strength to resist extreme seismic action within the elastic range is uneconomic in most cases, inelastic response made use of in design codes, provided that there is sufficient ductility capacity (μ_Δ). Detailed analysis of single degree of freedom systems indicated that there is an approximate relationship between force reduction factor (R or q) and the displacement ductility. For long-period structures, the equal displacement approach seems to apply which means that the ductility is comparable in magnitude to the force reduction factor (Figure 7.3(a)), i.e.

$$\mu_\Delta = \frac{\Delta_u}{\Delta_y} = R \quad (7.2)$$

For short period structures, the displacement ductility demand is greater and follows more closely the equal energy approach (Figure 7.3(b)), i.e.

$$\mu_\Delta = \frac{\Delta_u}{\Delta_y} = \frac{R^2 + 1}{2} \quad (7.3)$$

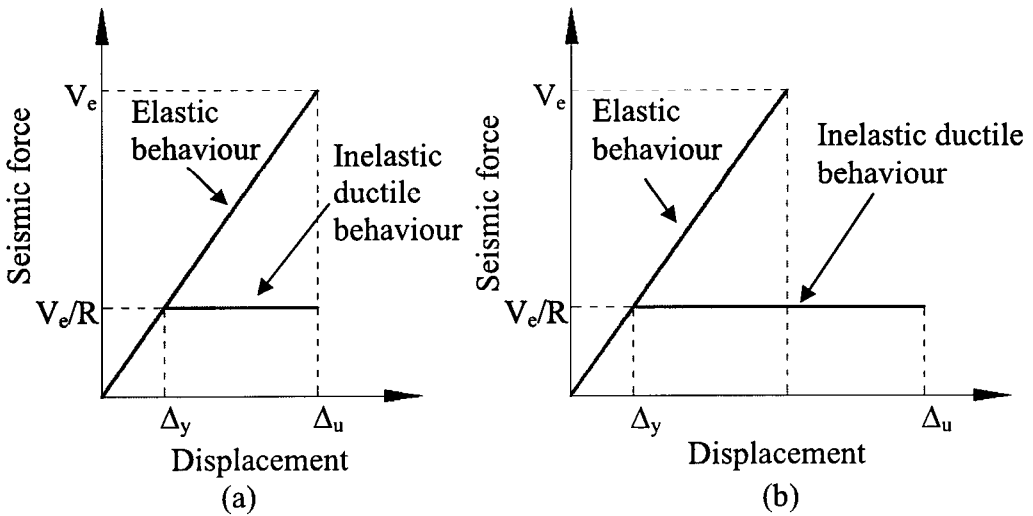


Figure 7.3: Relationship between force reduction factor and ductility

Most seismic codes however adopt a single behaviour factor, or force reduction factor, which is dependent on structural configuration and ductility capacity, regardless of the natural period range of the structure. For example, EC8 (EN 1998-1, 2004) classifies concentrically braced frames into three structural ductility categories and assigns recommended behaviour factors to each of them. In order to provide sufficient local

ductility for inelastic deformation, the choice of cross-sectional class is restricted according to the behaviour factor employed in the design (Table 7.3).

Table 7.3: *Requirements on cross-sectional class*

Reference values of behaviour factor	Required cross-sectional class
$1.5 < q \leq 2.0$	Class 1, 2 or 3
$2.0 < q \leq 4.0$	Class 1 or 2
$q > 4.0$	Class 1

In the case of square and rectangular hollow sections, the ductility is typically limited by the high strains that develop in the corner regions of the cross-sections during local buckling. As demonstrated in the tests, cracks initiate at these areas and propagate through the cross-sections when the braces are loaded in tension after buckling in compression, leading eventually to complete fracture of the members. Therefore, it is rational to set limits on the local slenderness, such as the cross-section class limits in EC8 (EN 1998-1, 2004), to control the occurrence of local buckling and subsequently fracture.

The tests and analysis performed in this study as well as previous investigations on the cyclic response of bracing members indicate that not only local slenderness but also global slenderness has a significant influence on the ductility of bracing members (Tremblay, 2002; Shaback & Brown, 2003; Goggins et al., 2006; Han et al., 2007). Based on Equations 6.21-6.23, constant displacement ductility capacity curves of braces of the three materials have been derived in this thesis and are shown in Figure 7.4. The global slenderness on the x-axis of the graph is the practical range for bracing members. The limit of $\bar{\lambda} \leq 2.0$ is recommended by EC8 (EN 1998-1, 2004) to prevent early elastic buckling, low energy dissipation and sudden loading. An additional limit of $\bar{\lambda} > 1.3$ for X diagonal bracing members is placed to avoid the development of excessive compressive load (Elghazouli, 2008). The limit of local slenderness of Class 1 carbon steel sections (EN 1993-1-1, 2005), which can be used in design for the highest structural ductility class, and the limit for Class 1 stainless steel sections (EN 1993-1-4, 2006) are also shown in the figure.

It is clear from Figure 7.4 that for Class 1 carbon steel hollow section members, braces can provide displacement ductility capacity close to 4 for intermediate and high global slenderness. However, the displacement ductility capacity drops sharply as the global slenderness decreases below 1.0. If higher displacement ductility is required, for example $\mu_{\Delta} = 8$ (although this is beyond that implied in EC8), the influence of global slenderness becomes more significant. For low global slenderness, a ductility capacity of 8 cannot even be provided within practical range of local slenderness. Therefore, it is recommended that both global and local slenderness of braces should be taken into consideration in the assessment of ductility capacity. Besides, categorising braces in classes overlooks the continuous variation of ductility with slenderness, resulting in less efficient use of some sections. In contrast, a continuous measure of ductility capacity in terms of global slenderness and local slenderness, such as Equations 6.21-6.23, provides a more accurate representation.

Figure 7.4 shows that within the range of practical slenderness, stainless steel braces have ductility capacity similar to, if not higher than, carbon steel braces. It has also been found that the energy dissipation per cycle of bracing members varies with global slenderness with similar trends for braces of the three materials (Figure 4.28). Consequently, the total energy to fracture of the braces is larger for stainless steel braces as they have larger number of cycles to failure. Therefore, in the current practice of limiting the cross-sectional class for various ductility categories, the Class 1 limit for carbon steel can be applied to stainless steel members for displacement ductility of approximately 4. An additional class limit is needed to classify cross-sections capable of providing higher displacement ductility. At high displacement ductility, such as 8, a more relaxed limit can be applied to stainless steel cross-sections than the carbon steel ones; further research is ongoing on this matter. In addition to the original lower limit for global slenderness of X-diagonal bracing, another limit is recommended for all types of bracing members in order to avoid the drop of displacement ductility at low global slenderness. For example, a lower limit of 1.0 can be applied to members providing displacement ductility of approximately 4 since it is shown that displacement ductility decreases significantly at lower global slenderness.

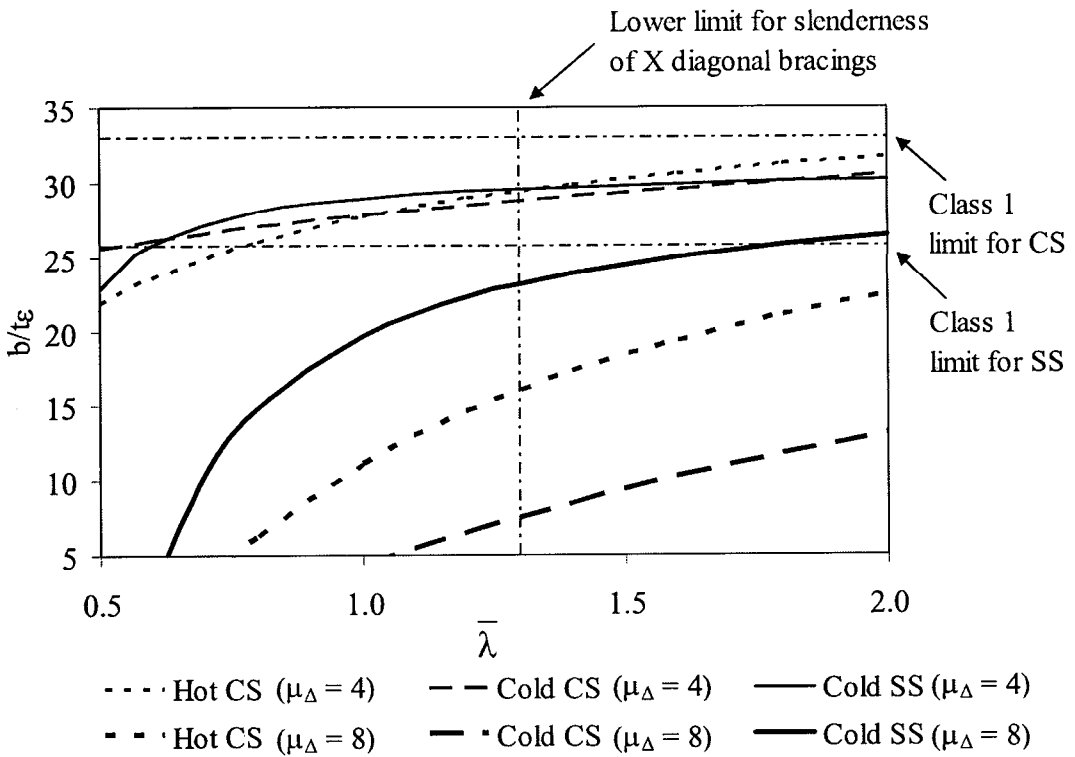


Figure 7.4: Ductility capacity provided by braces compared with limits of global and local slenderness

7.7 Stiffness considerations

Seismic design guides impose limits on the maximum storey drifts as criteria for serviceability and ultimate limit states (SEAOC, 1999; EN 1998-1, 2004; AISC, 2005). The initial stiffness, as well as its variation with loading, has a direct influence on the performance. In concentrically braced frames, the lateral stiffness is mainly provided by the bracing members. Typically, the bracing members in compression show high tangent stiffness at the initial loading stages before it drops quickly, possibly to negative values, due to buckling. On the other hand, the tangent stiffness of braces in tension increases gradually during straightening and reaches the peak values when they are fully stretched.

While the initial elastic stiffness of hot-rolled carbon steel, cold-formed carbon steel and cold-formed stainless steel material is similar, the tangent stiffness of braces made from three materials is different after cycles of inelastic deformation. As demonstrated in the numerical studies, when the applied displacements were increased to a new level,

the stainless steel specimens maintained a higher stiffness than the carbon steel members owing to the more pronounced strain hardening of stainless steel material (Figure 6.45). During subsequent loading cycles at the same displacement amplitude, the tangent stiffness of the stainless steel members was similar to that of the carbon steel specimens up to zero displacement but increased more significantly and attained higher values thereafter (Figure 6.46).

Although the relatively high tangent stiffness of stainless steel braces reduces the storey drift of frames at a given loading, it also has a direct influence on the effective natural period. Therefore, depending on the range of effective periods with respect to the characteristic spectral shapes, the advantage of lower storey drift at a given loading stage may be offset by the higher seismic forces implied. The actual influence of the stiffness of bracing members on the performance of the overall frame has to be investigated through nonlinear dynamic time-history analysis.

7.8 *Concluding remarks*

The implications of the results of the cyclic loading tests on bracing members made with hot-rolled carbon steel, cold-formed carbon steel and cold-formed stainless steel have been discussed in this chapter, with reference to current seismic design procedures (SEAOC, 1999; EN 1998-1, 2004; AISC, 2005). The material overstrength of stainless steel braces was found to be higher than that of carbon steel braces due to the more pronounced strain hardening and cyclic hardening effects. This points to the need for conducting tensile tests on the cross-section in order to determine the actual yield and maximum tensile strength of braces, which are necessary for capacity design checks. Otherwise, satisfactory estimates of the section tensile capacity can be made based on the ultimate tensile strength of the flat and corner material coupons. Similar to carbon steel braces, the buckling curves can be used for assessing the initial buckling strength of stainless steel members by applying a buckling strength enhancement factor. The current predictive expressions for post-buckling compressive strength were shown to be suitable for braces from the three materials. It was also shown that the ductility capacity of stainless steel braces is similar to carbon steel braces with some evidence indicating that the ductility and energy dissipation capabilities of stainless steel braces for specific

slenderness ranges is better than their carbon steel counterparts. Therefore, unless code provisions are revised to account for the favourable properties specific to stainless steel members, it is conservative to apply the local slenderness limits for carbon steel to stainless steel braces for classifying sections into different ductility categories. Finally, the higher tangent stiffness of stainless steel braces under tensile loading can have direct effects on the performance of braced frames, which needs to be investigated through further dynamic testing and analysis.

Chapter 8

Conclusions and Suggestions for Further Work

8.1 Project summary

This section summarises the important findings from the project and presents an overall conclusion. More detailed conclusions can also be found at the end of each relevant chapter.

The seismic resistance of concentrically braced frames is largely dependent on the response of the braces, which yield in tension and buckle in compression under large amplitude cyclic loading. Cyclic loading tests on carbon steel hollow section braces have led to mixed opinions on the relative importance of global slenderness and local slenderness on the ductility of the braces, owing to the difficulty in isolating the effects of these slenderness parameters in experiments. Meanwhile, the introduction of new stainless steel design provisions and extensive research on stainless steel structural members casts light on the possibility of utilising the high ductility and pronounced strain hardening in resisting seismic loading. The primary objective of the research project has therefore been to investigate the cyclic behaviour of hot-rolled carbon steel (S355J2H), cold-formed carbon steel (S235JRH) and cold-formed stainless steel (EN 1.4301 and 1.4307) hollow section braces through experimental testing and numerical modelling.

Tensile coupon tests on flat and corner materials of the bracing members were firstly conducted to assess the basic tensile mechanical properties. In an extreme seismic event, bracing members in concentrically braced frames and hence the constituent materials are subjected to large plastic strains and load reversals. High amplitude cyclic material tests were performed to obtain the material properties required for numerical simulation of the cyclic response of the bracing members. A total of 62 cyclic material tests were carried out on test coupons cut from the flat faces of the members: 31 axial tests at strain amplitudes ranging from $\pm 1\%$ to $\pm 7\%$, and an additional 31 corresponding tests at surface strain amplitudes ranging from $\pm 2\%$ to $\pm 15\%$ in a bending configuration.

The results of the axial cyclic material tests and monotonic tensile coupon tests were employed to calibrate the Coffin-Manson model (Coffin, 1954; Manson, 1954) for the three materials. These relationships were similar for the three materials despite the significant difference in elongations to fracture measured in monotonic tensile tests. The hot-rolled (annealed) material generally displayed marginally better fatigue performance than the cold-formed material, attributed to the increased level of inclusions and reduced ductility caused by cold-work. Examination of the fracture surface revealed a change from a brittle fatigue failure mode to a ductile tensile failure mode as the applied strain amplitude increased, which are features of material failing under low cycle fatigue (LCF) and extremely low cycle fatigue (ELCF), respectively. In addition to the Coffin-Manson relationship, two ELCF life prediction models (Kuroda, 2001; Tateishi, 2007) were applied to the data generated in this study. The two models provided improved agreement with the cyclic test data in the ELCF regime, but did not accurately predict the results of the monotonic tensile tests or the results of cyclic tests with higher cycles to failure.

Bending tests were carried out without the concerns of slippage and buckling that were possible in axial loading tests; the scatter of the generated test data was lower in the bending configuration. The total energy to fracture for the test specimens was found to follow similar trends, which were largely independent of the tests configuration. A method to correlate axial and bending test results through an energy criterion was established, demonstrating that the Coffin-Manson relationship can be derived from testing in either configuration.

The non-linear combined isotropic/kinematic hardening model employed in ABAQUS (2007) was calibrated with the generated cyclic stress-strain data. Stainless steel, which displayed substantial strain hardening in the monotonic tensile tests, also demonstrated a much larger degree of cyclic hardening than the two carbon steel materials. Although there was some variability in the individual model parameters between different tests, the FE simulated hysteresis loops generated by the combined isotropic/kinematic hardening model (on the basis of average parameters for each material) were able to capture the key observed behavioural features. The calibrated material cyclic hardening models and the strain-life relationships provided detailed input for numerical modelling of the cyclic response of the bracing members of the three materials. A drawback of the cyclic material hardening model is that it does not reflect the varying hysteretic behaviour at different strain amplitudes. This can be improved if different sets of parameters are allowed to define the hysteretic response at various strain amplitudes. The simulation of the shape of the stress-strain hysteretic loops can be further improved by incorporating more parameters in the material model.

Chapter 4 presented a series of cyclic axial loading tests on square and rectangular hollow section members – 3 hot-rolled carbon steel specimens, 4 cold-formed carbon steel specimens and 9 cold-formed stainless steel specimens were examined. After initial loading cycles at small displacements, the specimens showed global buckling when the applied displacement was in the proximity of the yield displacement. As the applied displacement increased, local buckling occurred at mid-length and, in some specimens, close to the end stiffeners. High localised strains developed in the plastic hinges formed in these areas, leading to eventual fatigue fracture. Despite a significant difference in monotonic material properties, in particular tensile ductility and pronounced strain hardening, member behaviour was found to differ to a lesser extent. Existing empirical expressions, derived from carbon steel test data, for predicting initial buckling resistance, compressive post-buckling resistance and mid-length lateral deflection were found to be also generally applicable to cold-formed stainless steel members. Nevertheless, stainless steel specimens showed higher tensile and compressive resistance and maintained higher tensile stiffness, when compared to carbon steel specimens with similar slenderness.

In Chapter 5, finite element models, incorporating the calibrated material cyclic hardening models, were developed to investigate the cyclic behaviour of bracing members over the full range of global and local slenderness. Since fracture was not explicitly simulated, a strain-based damage prediction method utilising the calibrated strain-life relationships was adopted. The finite element models and damage prediction method were verified against experimental results from the current study and two other research programmes (Walpole, 1996; Goggins et al., 2006).

The parametric studies were divided into two parts – investigation into the influence of slenderness parameters on ductility of the members and comparison between the members of the three studied materials. In the first part, local buckling was found to be crucial to the fracture of the members since it induced high localised strains at the corner areas. Braces with lower local slenderness exhibited higher resistance to local buckling and hence delayed fracture. Global slenderness of the braces, through various influencing factors, was shown to affect the compressive strains before the occurrence of local buckling and the severity of the buckling once it had initiated. In general, braces with higher global slenderness experienced lower compressive strains and were found to have higher ductility.

Observations from the current study highlighted some inconsistencies with conclusions found in the literature, where fracture life was deemed to increase with global slenderness but was often found not to vary significantly with local slenderness. The apparent contradiction with the findings of the current study was due to the difficulty in isolating the effects of global and local slenderness in some studies. An alternative selection of specimens to confine changes to only one variable at one time in other programmes (Walpole, 1996; Han et al., 2007) gave a clearer view on the effects of each of these factors.

The second part of the parametric studies presented a comparison between the performance of the braces of the three materials. Stainless steel members were found to be capable of sustaining more cycles of loading than carbon steel members prior to fracture, especially for stocky sections and members with high global slenderness. In comparison with cold-formed carbon steel, hot-rolled specimens were generally more ductile, particularly for high global slenderness and low local slenderness. Due to the

difference in member ductility and inclusion of data of specimens of a larger range of slenderness, previous relationships between member ductility and slenderness did not give satisfactory predictions of the experimental and FE results obtained in the current study. Expressions of displacement ductility in terms of both global slenderness and local slenderness were calibrated for braces of each material. These relationships accurately reflected the influence of slenderness and material properties on the ductility of the braces.

The level of energy dissipation of the members of the three materials at the same amplitude of loading were shown to be similar, although higher total energy dissipation to failure of stainless steel members was found owing to the higher number of cycles to failure. For carbon steel members, the energy to failure was higher in the hot-rolled specimens than the cold-formed specimens. The most significant and crucial difference between the materials was the tangent tensile stiffness of the members. Stainless steel test specimens and FE models exhibited higher stiffness than those of the two other materials.

The comparison between the three materials has shown that despite the significantly different response under monotonic tensile loading, their behaviour under cyclic loading are largely similar in terms of strain-life relationships of the materials, lateral deflections and loss of compressive resistance of the bracing members. However, stainless steel exhibited higher post-yield tangent stiffness at both material and member levels, as well as higher displacement ductility and energy dissipation to fracture for bracing members in certain slenderness ranges.

The implications of the above findings on the behaviour of concentrically braced frames have been discussed in Chapter 7. The significant material overstrength points to the need for conducting tensile tests on the cross-section in order to determine the actual yield and maximum tensile resistance of braces. Buckling curves in design codes and post-buckling compressive strengths are suitable for assessing the initial buckling strength and degradation of compressive strength respectively for bracing members of the three materials. It has also been shown that unless code provisions are revised to account for the favourable properties specific to stainless steel members, it is conservative to apply the local slenderness limits for carbon steel to stainless steel

braces for classifying sections into different ductility categories. Finally, the higher tangent stiffness of stainless steel braces under tensile loading can have direct effects on the performance of braced frames, which need to be examined through dynamic testing and analysis. Nevertheless, the cyclic material models and predictive expressions for member behaviour calibrated in this research project provide a firm basis for further investigation.

8.2 *Suggestions for further work*

The findings in this research project provide a good starting point for further exploration in several related areas. This final section identifies potential areas of further work.

Test results from cyclic axial material tests on flat material have been used in the calibration of strain-life relationships and cyclic material hardening models. However, initiation of cracks and opening has been observed at corner areas and the material properties derived from axial coupons tests on flat material may not accurately represent that of the corner material. Cyclic material tests on corner materials are therefore recommended in future studies since they will provide valuable information on their strain-life relationships and hysteretic behaviour of the materials at the critical locations. This may improve the simulation of the bracing members and the prediction of their fracture lives.

A reliable numerical method to predict the fatigue fracture of bracing members under cyclic loading has been developed and fully validated in the current study. The application of this method can be also extended to other structural elements and different cyclic loading regimes. The FE models can be easily adapted to simulate the cyclic response of other types of structural elements by modifications to the boundary conditions, application of load, lines of symmetry and locations of refined mesh. Since the damage prediction method is based on the strain induced in a localised area, it is applicable to other member types and locations of fracture, provided suitable mesh sizes and average areas of strain output are carefully chosen.

The implications of the findings presented herein on the behaviour and design of concentrically braced frames have been deduced mainly from the cyclic response of bracing members under quasi-static loading. A more complete understanding of the performance of braced frames in earthquakes, however, requires testing and analyses on frame models under dynamic loading. In dynamic experimental testing, strain rate effects of the materials can be observed. When tests are no longer displacement controlled, asymmetric response may also be discovered (Elghazouli, 2005). By utilising the calibrated cyclic material models and the predictive expressions on member ductility, the interaction of frame members and the benefits of applying of stainless steel elements to various parts of the structures can be assessed with greater accuracy.

Current seismic design codes limit the cross-section class, and hence the local slenderness, of the braces to ensure sufficient local ductility is provided. However, this research project has shown that member ductility depends on the co-existing influence of global slenderness and local slenderness. With further study on frames with braces of a wide range of slenderness, new limits that take into consideration both global slenderness and local slenderness can be investigated. Alternatively, the three ductility classes in EN 1998-1 (2004) may be replaced by a continuous relationship between ductility capacity and slenderness, which may lead to more efficient design.

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