CHAPTER 1

Introduction

1.1 BACKGROUND

The applications of composite members consisting of concrete-filled steel hollow sections have become increasingly popular in civil engineering structures in recent years. This is due to their advantages over the conventional structural sections in terms of strength, ductility, energy absorption capacity (Ge and Usami, 1992), easy construction procedure (Dean, 1992) and overall economy (Webb and Peyton, 1990). The steel and the concrete element in a composite member complement each other ideally. Whilst the steel confines the concrete laterally allowing it to develop the optimum compressive strength and ductility, the concrete in turn supports the steel shell
laterally to prevent elastic local buckling. The applications of these composite sections include the frames of multistorey buildings (Bode, 1976), the bridge decks of motorways and railways (Hanshin Expressway, 1986) and the support for oil storage tanks (Ghosh, 1977).

The research on concrete-filled steel tubular sections gained interest in the late sixties and in the early seventies when a number of researchers (Furlong, 1967; Neogi et al, 1969; Knowles and Park, 1969; Chen and Rentschler, 1974) studied columns and beam-columns experimentally and also proposed analytical procedures for the derivation of ultimate load and moment capacity. More recently, studies published on concrete-filled steel hollow columns and beam-columns (Rangan, 1991; Usami et al, 1992; Shakir-Khalil, 1988) indicated a renewed interest on the topic which was encouraged mainly by the fast and easy constructability of this composite system (Webb and Peyton, 1990). Casselden Place, Melbourne (Dean, 1992) is one of the examples which demonstrated the flexibility and versatility of the system. Concrete-filled steel hollow sections were used in the columns and steel decks were adopted for the permanent formwork for reinforced or prestressed concrete slabs. The construction work of Casselden place was completed in a time which was much faster (Dean, 1992) than the more conventional reinforced concrete buildings. The renewed research interest in concrete-filled steel tubular sections was also driven by the widespread use of high to very high strength concrete combined with high strength steel of upto 600 MPa yield stress. The recent research findings have been incorporated into the relevant design codes (Eurocode 4, 1994, ACI 318, 1989 and AS 3600, 1988).
Recent research interest in concrete-filled RHS include the flexural members and the beam-column connections. The concrete-filled RHS beams have a much superior ductility and post-failure behaviour (Lu and Kennedy, 1994) when compared to the hollow members. Beam-column connections where both beam and column are constructed from concrete-filled steel hollow section is another area of recent research due to their large shock absorption capacity (Ge and Usami, 1992, Hayashi et al, 1995). This characteristic is particularly critical in the event of an earthquake.

One of the important parameters which can affect the ultimate strength of the composite section is the bond at the interface of the steel and the concrete. This bond is relied upon the shear forces between the steel shell and the concrete core where no mechanical connectors are provided (Oehlerls and Sved, 1994). The effect of bond forces at the interface may influence the capacity of the composite section significantly, particularly for the flexural members with high moment gradients and connections under large rotations. However, at this stage design recommendations available in most of the codes (Eurocode 4, 1994) ignore the effect of slippage at the interface.

1.2 STATEMENT OF THE PROBLEM

Although design formulae are available in the codes of practices (Eurocode 4, 1994, ACI 318, 1989) there is a significant lack of understanding in the assumptions for the calculations of the ultimate strength of steel tubular sections filled with high strength concrete. Current design theories (Eurocode 4, 1994) assumed a full bond interaction
between the steel shell and the concrete core for the simplicity of the calculation for the ultimate moment of resistance. However, in reality slippage at the steel-concrete interface is inevitable after the tensile cracking of concrete. For a better understanding of this complex interface behaviour finite element study is a more effective method than the experimental investigation.

Pushout resistance tests where concrete core is used to push through the steel shell (Virdi and Dowling, 1980, Shakir-Khalil, 1991) is a convenient way to determine the bond characteristic. The load versus slip response from the pushout test can be used in a finite element study to examine the influence of interface bond into the overall performance of the concrete-filled RHS sections.

Design guidelines are also lacking for the optimisation of the shell thickness of the steel RHS when used in conjunction with concrete-filled composite section. The shell thickness is specified by limit plate width to thickness ratio according to the code (AS 4100, 1990; AISC, 1991) to avoid elastic local buckling. Once filled with concrete the buckling strength of the steel shell may increase significantly and a thinner option of the thickness can be considered. The other determining factor for the wall thickness is the strength contribution provided by the concrete. Design guidelines are needed to optimise this thickness for the economy of the overall composite section.

When higher strength concretes in the vicinity of 100 MPa are used as filler material for the concrete-filled RHS flexural members questions may arise about the ductility and post-failure behaviour. This is due to the brittle behaviour of high to very high strength
concretes (Attard and Mendis, 1993). Codes of practices (Eurocode 4, 1994) assumed normal strength concrete is used as the filler in the concrete-filled steel tubular composite sections. However, in many design situation, the same code is also used in cases where high strength concrete is used. As the material properties of high strength concrete is significantly different from the normal strength concretes there is a need to examine the behaviour of composite sections with higher strength concretes.

All the abovementioned problems are applicable for the study of rigid beam-column connections where both beam and column portions are constructed from steel tubular sections filled with high strength concrete. A better understanding is needed in force transfer mechanism between steel and concrete when large rotations occur. Due to large rotations slippage may occur at the interface in the cracked and /or crushed locations of concrete which may lead to the connection failure with heavy steel yielding around these locations.

The objectives of this study are to examine experimentally and analytically the behaviour of flexural members and beam-column connections consisting of RHS filled with high strength concrete with emphses on the ultimate strength, ductility and post failure strength reserve. Aims were also made to develop a simple design model to analyse the ultimate moment of the composite section with various bond conditions. Objective was also focussed on the bond between steel and concrete without mechanical connectors. Finally the effort was made to compare the experimental results with that of the analyses.
1.3 TOPICS TREATED IN THIS THESIS

1.3.1 **General**

This thesis studies experimentally and analytically the behaviour of the flexural members, beam-column connections and the pushout resistance of RHS filled with high strength concrete. Studies emphasize on the ultimate flexural strength, ductility and post-failure strength reserve. Study also aims to provide a better understanding of the bond behaviour between the steel shell and the concrete core and the influence of bond on the overall behaviour.

Investigations were carried out in two different methods, experimental and analytical studies. Analytical studies were also conducted in two separate methods. The derivations of the ultimate flexural strength and deflections for the composite section were performed by using the principle of structural mechanics. Finite element analysis was used to model more realistically the physical behaviour. Finite element models were also used to compare the experimental results and also to study the parameters which can significantly influence the overall behaviour of the concrete-filled RHS including shell thickness and cross-sectional geometry of the RHS, compressive strength of concrete and the bond strength between steel and concrete.

The work presented in this thesis may be divided into two main parts (Chapters 3, 4 and 5; and Chapters 6, 7, 8 and 9). In Chapters 3, 4 and 5, experimental investigations on the behaviour of concrete-filled RHS flexural members, beam-column connections and the
pushout resistance between the steel and concrete are presented respectively. Chapter 6 presents the derivation of design formulas for the ultimate moment of resistance and also the deflection of the concrete-filled RHS flexural member. Chapter 7, 8 and 9 present finite element studies on the flexural member, beam-column connection and pushout behaviour of the concrete-filled RHS.

In Chapter 2 an extensive literature review related to concrete-filled RHS beams, beam-columns and connections are presented in which the achievement of other researchers are discussed and the discrepancies are also notified. Attention was focussed on ultimate moment capacity, ductility and post-failure strength, slip at the interface and the local buckling of steel shell. Discussions are presented on the design formulae and guidelines proposed by different researchers. Design procedures suggested in the current codes of practices were also discussed. The conclusions of this research is provided in Chapter 10.

1.3.2 Experimental Studies

In Chapter 3 test results are presented on the flexural behaviour of steel RHS filled with high strength concrete. Spreadsheet graphs were used to present the results such as load versus deflections, load versus strains, moment versus curvature etc. Study emphasizes ductility and post-failure behaviour of the flexural member due to the reason that high strength concrete is a relatively brittle material than normal strength concretes.
In Chapter 4 study is carried on rigid beam-column connections where both beam and column portions are constructed from steel RHS filled with high strength concrete. Two different types of geometry was adopted for these rigid connections in two series of experiments. In the first series beam was placed on top of the short column leaving the cantilever end on one side and in the second series the cantilever beam was connected at the side of the short column. Study emphasizes the rotational capacity and the post failure strength reserve of these rigid composite connections. Among the other important investigation was the interface bond between steel and concrete.

The test results on pushout resistance of the concrete core through the steel shell of the RHS is provided in Chapter 5. Three types of inside steel surface treatment of the steel RHS were adopted, they were acid-washed, greased and untreated. No mechanical connectors were used. Load versus slip graphs were plotted to examine the bond behaviour.

1.3.3 Analytical Studies

Analytical studies presented in this thesis may be divided into two broad sections. In the first section design formulas were derived using the principle of structural mechanics. In the second finite element studies are presented.

In Chapter 6 analyses are presented to calculate the ultimate moment capacity of the concrete-filled RHS flexural member. The main assumptions for the derivation include
the steel portion is elastic-perfectly plastic and the concrete carries no strength in the tensile zone. Different bond conditions between the steel shell and the concrete core were assumed for comparison. Simple analytical expressions were derived so that they can be coded into spreadsheet programs or in a programmable calculator. Efforts were also made to calculate the deflections of the composite flexural members.

Finite element studies of concrete-filled RHS were carried out by using the software ANSYS (ANSYS 5.1 user manual, 1995). Three types of elements were used in all the analyses. For the steel shell of the RHS a plastic shell element with large strain nonlinear capability was used, for concrete core a solid element with cracking and crushing capabilities, and to model the interface a nonlinear spring contact element was used.

The finite element studies on flexural members are presented in Chapter 7. Models were developed following the test specimens as mentioned in Chapter 3 for comparative study. Finite element models were also analysed to study the bond behaviour at the interface by using different flexural member models with different bond stress between steel and concrete. Results are presented in the form of spreadsheet graphs and coloured contours.

In Chapter 8 the beam-column connections of concrete-filled RHS were studied. Connections in two different geometrical configurations were analysed following the test specimens used in the experiments and mentioned in Chapter 4. Coloured contours and graphs were used to present the results.
Chapter 9 presents the finite element study on the pushout behaviour of concrete-filled RHS by developing the models according to the specimens used in the experiments (Chapter 5). Models were also developed to study the influence of different parameters such as the shell thickness and the shape of the cross-section of the RHS, and the compressive strength of concrete.
CHAPTER 2

Literature Review

2.1 INTRODUCTION

This chapter presents a review of studies performed on concrete-filled steel tubular members. The discussion is mainly focussed on flexural members, connections and pushout resistance between steel and concrete. Emphasis is given on ultimate strength, initial stiffness, ductility and post failure strength reserve of the composite members. Interface bond between steel and concrete is also reviewed.

The studies discussed in this chapter are divided mainly in two parts. The first part reviews the experimental studies conducted by different researchers on concrete-filled
steel tubular sections which includes beams, rigid connections and pushout behaviour between the steel shell and the concrete core. In the other part the studies on analysis and design of the composite sections are reviewed and discussed. The design recommendations for the analysis of the steel-concrete composite sections provided in the different codes of practices are also discussed.

2.2 EXPERIMENTAL STUDIES

2.2.1 Concrete-filled steel hollow flexural members

2.2.1.1 General

This section reviews the experimental studies on flexural members of concrete-filled steel hollow sections. This review also includes the studies on concrete-filled steel hollow beam-columns which is considered to be the related topic. Earlier experimental studies (Kloppel and Goder, 1957; Gardner and Jacobsen, 1967; Furlong, 1967; Chapman and Neogi, 1966; Knowles and Park, 1969) include tests on beam-columns of concrete-filled steel tubes. Ordinary strength concretes were used in the experiments. All the above-mentioned studies concluded that the presence of concrete provided much higher buckling strength in the steel shell and steel portions were yielded before failure. Studies also concluded that concrete-filled sections had a much superior overall performance than hollow section counterparts. Renewed interests in studies on concrete-filled steel tube sections in the early nineties was encouraged by the availability of
higher strength concretes of upto 100 MPa and high yield strength steel. Studies on beam-columns (Shakir-Khalil, 1992; Kitada, 1993; Rangan and Joyce, 1992; Ge and Usami, 1992) and beams (Lu and Kennedy, 1994; Prion and Boehme, 1994) indicate concrete-filled steel hollow members have significantly higher ultimate strength, initial stiffness, ductility and post-failure strength reserve than hollow members. Studies also shown due to the prevention of inward buckling of the steel shell, local buckling largely delayed and yield stress of the steel was effectively utilised. In the following sections, studies are reviewed for composite beams and beam-columns on different characteristics of the composite member.

2.2.1.2 Initial stiffness

Studies on beam-columns (Furlong, 1967; Knowles and Park, 1969; Neogi, Sen and Chapman, 1969) demonstrated concrete-filled steel tubular section has significantly higher initial stiffness than hollow section members. Study on concrete-filled RHS long columns (Shakir-Khalil and Al-Rawadan, 1996) with different eccentricities of loads showed increased stiffness for those members with less eccentricities. Study on concrete-filled RHS flexural members (Lu and Kennedy, 1994) demonstrated that concrete-filled sections have much superior initial stiffness than hollow section counterparts. In another study (Kilpatrick and Rangan, 1995) the influence of interface bond in the overall behaviour of the concrete-filled steel tubular beams and beam-columns was investigated. It was shown that the stiffness does not increase significantly with the increase of bond between steel and concrete at the interface. A summarised study (Kitada, 1993) showed superior stiffness of beams with slip restriction at the ends.
This study also indicated the difference in initial stiffness between hollow and concrete-filled steel tube beams is more significant in the rectangular sections when compared to the circular sections.

2.2.1.3 Local buckling of the steel shell

Experiments (Furlong, 1967; Knowles and Park, 1969) on concrete-filled steel tubular beam-columns showed local buckling of the steel shell was largely delayed. This is due to the presence of concrete core which prevents inward buckling and thus buckling mode was changed to cosine wave from sine wave mode in the hollow sections (Wright, 1993). However, this local buckling is dependant on width-to-thickness ratio of the steel hollow sections. Study (Tsuji et al, 1992) on concrete-filled steel RHS columns demonstrated that for columns with larger width-to-thickness(b/t) ratio, local buckling appeared first, then failure occured whilst for the columns with smaller width-to-thickness ratio failure occured first, followed by local buckling. In a recent research (Hayashi et al, 1995) the concrete-filled steel tubular beam-columns were cyclically loaded laterally. Results showed in the case where width-to-thickness ratio was large, the hysteresis characteristics reached maximum load directly after local buckling occured and resistance force decreased drastically. On the contrary, when the width-thickness ratio was small, the resistance force did not decrease even after local buckling had occured. Other composite beam-column studies (Neogi et al,1969; Prion and Boehme, 1994) indicated local buckling either prevented or delayed and the yield stress of steel effectively utilised. Study (Lu and Kennedy, 1994) on concrete-filled RHS flexural members showed the failure was initiated by upward inelastic local buckling of
the top flange of the steel shell. Finally the failure occurred by steel yielding and compression crushing of concrete where the steel had buckled. The RHS used in this study (Lu and Kennedy, 1994) were all compact sections, so in all the concrete-filled beams steel portion heavily yielded and elastic local buckling totally prevented.

2.2.1.4 Ductility and post-failure strength reserve

One of the reasons for great popularity of concrete-filled steel hollow sections in recent construction industry is due to its ductility and post-failure strength reserve (Usame et al, 1993; Kitada, 1993). Tests on concrete-filled steel tube beam-columns (Neogi et al, 1969; Rangan and Joyce, 1992; Prion and Boehme, 1994) demonstrated that at failure large amount of strains were obtained and the overall behaviour was extremely ductile. Tests on flexural members (Barber et al, 1987; Lu and Kennedy, 1994) showed that concrete-filled sections have much superior ductility and post-failure strength than hollow section members. Studies on composite long columns under cyclic loading (Ge and Usami, 1992; Hayashi et al, 1995) demonstrated such composite system has large energy absorption capacity. One of the concerns of these composite members is that when the filler is high strength concrete of compressive strengths around 100 MPa. Studies (Ahmad and Shah, 1980; Carrasquillo et al, 1981; Attard and Mendis, 1993) showed high strength concrete is extremely brittle with no post-failure strength reserve once the peak load is reached. In recent construction practices where high to very high strength concrete are readily available, it is beneficial to use higher strength concretes in the composite sections to achieve higher ultimate strength. This thesis studies the
composite members where the filler material is high strength concrete of compressive strength around 100 Mpa.

2.2.1.5 Ultimate strength

A comparison of different column construction options (Webb and Peyton, 1990) indicated the concrete-filled steel tubes are the most economical solution in the multistoried buildings of more than 30 levels. Earlier studies (Kloppel and Goder, 1957; Gardner and Jacobsen, 1967; Furlong 1967; Knowles and Park, 1969; Neogi et al, 1969) on concrete-filled steel tube beam-columns demonstrated that significantly higher ultimate strength was obtained. In some of the composite members where a thinner steel tube thickness was opted often fail due to premature elastic local buckling before yield stress of steel is achieved. Results (Knowles and Park, 1969) showed the concrete strength increased due to confinement effect only in short composite columns whilst in long columns increase in concrete strength was not observed. The reason for this is the failure which was initiated by local buckling of the steel shell. In a more recent study on composite beam-columns, Rangan and Joyce (1992) used high strength concrete of 67 MPa compressive strength as filler material. Test results of ultimate strength of this study was always higher than prediction although marginally. A summarised study (Kitada, 1992) showed concrete-filled circular sections are more effective in terms of ultimate strength than concrete-filled rectangular sections which is due to superior confinement of concrete in the circular sections. Cyclic loading on concrete-filled steel tubular beam-columns (Hayashi et al, 1995; Prion and Boehme, 1994) indicate a sharp fall in load carrying capacity after ultimate load for thinner wall thicknesses of the steel
tubes whilst for thicker shell thickness the ultimate load occurred late in the loading period after significant curvature of the member was obtained. Flexural member studies (Barber et al, 1987; Lu and Kennedy, 1994) of concrete-filled RHS showed the yield stress of steel effectively utilised on the overall strength of the section. However, in these studies the steel RHS used were all compact sections and generally used for hollow structural members. It is therefore necessary to optimise the wall thickness of the steel for concrete-filled hollow sections in structural use.

2.2.1.6 Slip at the interface

Test results of most of the studies on concrete-filled steel tubular columns and beam-columns indicated negligible or no significance of bond between the steel shell and the concrete core on the overall behaviour. Design Codes (Eurocode 4, 1994; ACI 318, 1989; AS 3600, 1992) also assumed full compatibility in strains of steel and concrete at the interface for the derivation of ultimate strength of the composite section. However, this interface bond may affect significantly on composite flexural members with high moment gradients when no mechanical connectors are provided. Test results (Kilpatrick and Rangan, 1995) showed that by using different bond conditions at the interface did not change the overall behaviour of concrete-filled RHS columns, beam-columns and beams significantly. In another study (Kitada, 1992) results showed by using slip restriction at the ends of the composite beams, the overall behaviour improved noticeably. Efforts were also made to measure the interface slip in some of the experimental studies on flexural members (Lu and Kennedy, 1994; Prion and Boehme, 1994) and negligible amount of slip was recorded. However, it is very difficult to assess
the slippage between steel and concrete in the experiments because the slip will be
different in different locations of the composite beams. This slippage may concentrate
around the cracked locations of concrete whilst strain compatibility may prevail between
steel and concrete within the portions where the cracks occur.

2.2.2 Concrete-filled RHS beam-column connections

2.2.2.1 General

One of the major advantages of using concrete-filled steel hollow sections in the
multistoreyed building constructions is the easy constructability (Webb and Peyton,
1990). In 43-storey Casselden Place prefect in Melbourne, Australia (Dean, 1992) steel
I-beams as floor beams were connected to concrete-filled steel circular columns by a
combination of filled and butt welds. Studies on steel I-beam to concrete-filled steel
tube columns (Shakir-Khalil, 1994; Kang et al, 1995) showed a significant increase in
ultimate strength and rotational capacity of the connections with concrete-filled columns
when compared to the beam-column connections with hollow section columns. In the
following sections review on composite beam-column connections are presented.

2.2.2.2 Rotational capacity

Research (Usami et al, 1992; Ge and Usami, 1992; Hayashi et al, 1995; Prion and
Boehme, 1994) demonstrated large amount of ductility and energy absorption capacity
when concrete-filled steel tube columns were cyclically loaded. Most commonly used composite beam-column connection is a simple shear type connection where steel I-beams are connected to concrete-filled steel hollow section columns (Shakir-Khalil, 1993; Yamamoto et al, 1994). Research (Shakir-Khalil, 1993) carried out on concrete-filled RHS column to steel I-beam connections where beams were connected to the columns by using finplates and allround welds. Beams with large load eccentricities showed significantly large rotations occur at the connections before failure (Shakir-Khalil, 1993; Yamamoto et al 1994). Studies also conducted on steel I-beam to concrete-filled steel tubular columns where stiffeners were used internally (Kang et al, 1995) and externally (Choi et al, 1995) on concrete-filled columns to increase the stiffness of the joint. Results showed stiffened composite joints have large amount of ductility in the post-failure range.

2.2.2.3 Ultimate strength

Tests on simple shear type connections of cantilever structural I-beams to concrete-filled steel tube columns (Shakir-Khalil and Mahmoud, 1995; Yamamoto et al, 1994) indicated the failure occurred in the column portions with inelastic local buckling and heavy steel yielding thereafter. Experimental results (Shakir-Khalil, 1993) also showed earlier elastic local buckling largely prevented due to the presence of concrete core. One of the important characteristic which may influence the overall behaviour of these composite connections is the shear transfer mechanism between steel and concrete. Studies (Shakir-Khalil, 1993) showed once large out-of-plane deformations of the steel
shell of concrete-filled RHS columns took place, the failure mechanism changed to that of a membrane mechanism.

2.2.2.4 Local buckling

One of the most influential parameter to ultimate strength of the concrete-filled steel hollow sections is the buckling strength. This buckling strength is dependant on the width-to-thickness ratio of the cross-section of the steel hollow section (Furlong, 1967; Knowles and Park, 1969; Hayashi et al, 1995).

Experimental studies on concrete-filled RHS truss connections (Packer and Fear, 1991; Packer and Kenedi, 1993) demonstrated a much improved overall performance when compared to unfilled RHS connections. In the hollow section connections failure occurred by elastic local buckling and crushing of the flange of the chord member of the truss by the tensile web members. Whilst in the concrete-filled connections failure was due to bearing failure of concrete in the compressive web members (Packer and Kenedi, 1993).

Experiments on connections of cantilever steel I-beams to concrete-filled RHS columns (Shakir-Khalil, 1994) showed composite connections failed due to inelastic local buckling and then heavy yielding of the steel shell of the RHS. This inelastic local buckling may be dependant on the shear transfer and the interface bond between steel and concrete which is an area of further investigation.
2.2.3 Pushout resistance of concrete-filled RHS

2.2.3.1 General

One of the most important characteristic in the concrete-filled steel hollow sections is the bond between steel and concrete. Studies (Kitada, 1992; Kilpatrick and Rangan, 1995) on composite flexural members showed that restricting the slip by capping the ends the overall stiffness improved noticeably. This interface bond depends mainly on the cross-section geometry (Virdi and Dowling, 1980; Shakir-Khalil, 1991) and the surface roughness of the steel RHS (Virdi and Dowling, 1980) and also on the properties of concrete (Hunaiti, 1992; Shakir-Khalil and Hassan, 1995) when no mechanical connectors are used. A review on pushout tests conducted on concrete-filled steel hollow sections to determine the interface bond behaviour is presented in the following sections.

2.2.3.2 Size and shape of the cross-section

Test results (Virdi and Dowling, 1980) showed that the interlocking between the steel and the concrete at the interface play a major role to determine the pushout behaviour in the concrete-filled steel tubes. Study (Roik and Breit, 1981; Shakir-Khalil, 1991) demonstrated circular steel sections can generate a superior bond strength compared to rectangular counterparts which is due to the better confinement effect. Results (Shakir-Khalil, 1993) also showed bond strength increased significantly when using RHS with smaller cross-sections when compared to RHS with larger cross-section geometry.
Pushout test with shear connectors (Shakir-Khalil, 1991) indicated the function of the shear connectors seems to depend on the type of section used and it was demonstrated that mechanical shear connectors are more effective when used with circular hollow sections rather than rectangular hollow sections.

2.2.3.3 Wall thickness of the RHS

The study of bond resistance in the concrete-filled RHS (Shakir-Khalil, 1991) seems to indicate that pushout force may depend on the wall thickness of the steel hollow section. Study (Virdi and Dowling, 1980) also indicated that the bond strength may be related to width-to-thickness ratio of steel RHS. However, it is difficult to establish the relationship between the wall thickness of the steel hollow section and the bond strength at the steel-concrete interface by only experimental studies because the pushout force is mainly dependant on the interlocking of concrete to the steel shell. In this situation, finite element studies may be more effective than experimental studies.

2.2.3.4 Properties of concrete

Test results (Shakir-Khalil and Hassan, 1995) showed bond strength of concrete-filled steel hollow sections relates significantly with the cement content of the concrete. Higher cement contents in concrete-mixes, such as higher strength concretes causes much higher shrinkages (Swamy and Anand, 1973) than normal strength concretes. In concrete-filled steel hollow sections higher shrinkage diminishes the keying action at the steel-concrete interface. Study (Shakir-Khalil and Hassan, 1995) showed bond strength
in concrete-filled RHS decreases significantly when high strength concrete was used as the filler material instead of normal strength concrete. Research (Hunaiti, 1990) also demonstrated the bond strength reduced noticeably with the increase of age in the concrete-filled composite columns.

2.3 ANALYSIS AND DESIGN

2.3.1 General

Mathematical formulations and design methods were proposed by a number of researchers on concrete-filled steel tube members. In the following paragraphs the analytical and design procedures on concrete-filled steel hollow section members which was recommended by different researchers are presented. Review is mainly related to the studies on composite beams and beam-columns and the discussion is emphasized on the ultimate moment capacity, flexural stiffness and the interface bond.

2.3.2 Ultimate moment capacity

For the derivation of the ultimate moment capacity of the concrete-filled steel hollow sections, the conventional reinforced concrete theory was considered by most of the researchers. In some of the studies (Knowles and Park, 1969; Chen and Rentchler, 1974; Redwood, 1983; Barber et al, 1987), the ultimate stress of concrete was assumed as
0.85 $f'_c$. However, considering the confinement effect of the steel shell on the concrete core, the maximum concrete stress was considered as a more realistic value 1.0 $f'_c$ in a number of studies (Furlong, 1967; Tomii and Sakino, 1979; Lu and Kennedy, 1994).

For the derivation of ultimate bending moment most of the researchers considered concrete carries no strength in the tensile zone and a limiting concrete strain of 0.003 was assumed at failure as suggested by a number of codes of practices (ACI 318, 1989; AS 3600, 1994). For the steel hollow sections, most of the studies assumed the steel section fully plastic at the time of failure for the simplification of the analysis. Except in some of the studies (Barber et al, 1987; Lu and Kennedy, 1994) the stresses in steel were derived from corresponding strain values obtained during experiments to compare test results with the theory. The ultimate moment capacity derived from the ultimate strength concepts of reinforced concrete is quite tedious to compute particularly for nonrectangular shapes of the hollow sections. A rigorous computation procedure (Neogi et al, 1969) was demonstrated by dividing the concrete-filled hollow circular sections into a number of strips. In another analysis of concrete-filled circular tubes (Rangan, 1991), the areas of steel and the effective areas of concrete were assumed to be concentrated at their centroids and the stresses for the corresponding areas were calculated from the strain values. Stress-strain relation of concrete in the circular sections was defined by Hognestad’s parabola (Neogi et al, 1969; Rangan and Joyce, 1992). In the analysis of the composite section most studies assumed strain compatibility exists at the steel-concrete interface which also simplifies the derivation. But in the composite members with transverse loading where large amount of
longitudinal shear stresses are present slippage may occur between the steel shell and the concrete core.

2.3.3 Interface bond

In the analysis of the ultimate moment capacity of concrete-filled steel tube sections an important consideration is the bond condition at the steel-concrete interface when no mechanical connectors are used (Oehlers and Sved, 1994). Design equations provided in the studies on composite beam-columns (Kloppel and Goder, 1957; Gardner and Jacobsen, 1967; Furlong, 1967; Neogi et al, 1969; Chen and Chen, 1973; Shakir-Khalil, 1991; Rangan, 1991) and composite beams (Barber et al, 1987; Lu and Kennedy, 1994) assumed a full bond interaction between the steel shell and the concrete core. It was shown that the analysis compared favourably with the experimental results. This assumption of full bond interaction at the interface may be questioned in the case of composite flexural members with high moment gradients where slippage between steel and concrete may inevitably occur. Superposition models (Chen and Atsuta, 1976; Prion and Boehme, 1994) were proposed where no strain compatibility was assumed as the stress resultants of steel and concrete portion were calculated separately and then added up to obtain the total moment capacity of the concrete-filled steel tube section. This approach although has considerable appeal, can be questioned because even after loss of bond at the interface during the loading procedure, frictional interactions between steel and concrete may always present (Virdi and Dowling, 1980). This interface bond is studied later in this thesis by finite element analysis.
2.4 REVIEW OF THE DESIGN CODES

2.4.1 General

Design guidance are provided for concrete-filled steel tubes in some codes of practices, particularly in Eurocode 4-1994 and ACI 318-1989. The design methods described in these two codes are different in concept. The Eurocode 4 method is similar to that for steel section members whilst the concept of ACI-318 is the traditional reinforced concrete approach. Australian code, AS 3600-1994 also uses the concept of reinforced concrete design. A comparative study which reviewed the design codes with the experimental studies (O'Shea and Bridge, 1994) showed the methods for deriving ultimate strength in Eurocode 4 provide more accurate predictions than other procedures. In the following sections a review of design methods provided in different codes of practices on steel-concrete composite sections is presented. Review mainly concentrates on three design codes. They are Eurocode 4 (1994), ACI 318 (1989) and AS 3600 (1994).

2.4.2 Eurocode 4-1994

In this section the design recommendations in Eurocode 4 on concrete-filled steel tubes are discussed. The discussion is focussed on the related issues to the concrete-filled steel hollow flexural members.
Ultimate strength

Among the major assumptions in the analysis of the ultimate moment of resistance of the composite beams, the steel portion is considered as perfectly plastic. Analysis also assumes full strain compatibility exists between steel and concrete at the interface. The design concrete strength is considered as $0.85 f_{ck}$ where $f_{ck}$ is the characteristic compressive strength of concrete. The concrete portion in the tensile zone is assumed to contribute no strength in the tensile zone.

Local buckling

The width-to-thickness ratio for the rectangular hollow section in the composite members is kept as $h/t \leq 52 \epsilon$,

where $h$ is the greater overall dimension of the section parallel to a principal axis,

t is the thickness of the wall of a concrete-filled hollow section,

$$\epsilon = \sqrt{\frac{235}{f_y}}$$

and $f_y$ is the yield strength of the steel in MPa.

Bond strength

It is assumed that the shear resistance shall be provided by bond stresses and friction at the interface, such that no significant slip occurs. The design shear strength in bond and friction for concrete-filled hollow steel sections are given as 0.4 MPa. However, this assumption of bond strength is only valid for normal strength concrete with compressive strengths up to 50 MPa. Research (Shakir-Khalil and Hassan, 1995) demonstrated that due to high cement content in the higher strength concretes (100 MPa) shrinkage...
became higher which leads to diminishing keying action at the steel-concrete interface and the bond strength decreases significantly.

**Properties of concrete**

One of the major limitations of the current Eurocode (1994) is the limit on the concrete strength which is not more than 50 MPa. The secant modulus of elasticity for 50 MPa concrete is 37000 MPa as mentioned in the Eurocode. This value is about 36000 MPa according to that of the Australian standard (AS 3600, 1994). The nominal value of poisson’s ratio for elastic strains is assumed as 0.2. The poisson’s ratio may be assumed to be zero when concrete in tension is assumed to be cracked.

**Elastic flexural stiffness**

The elastic flexural stiffness of the composite section is given in the Eurocode as,

\[ EI = E_s I_s + 0.8 E_{cd} I_c \]

where \( I_s \) and \( I_c \) are the second moment of areas of structural steel and the concrete respectively. \( E_s \) is the elastic modulus for the structural steel and \( E_{cd} = \frac{E_{cm}}{\gamma_c} \) where \( E_{cm} \) is the secant modulus of elasticity of concrete and \( \gamma_c = 1.35 \) is the safety factor for stiffness.

**2.4.3 ACI 318-1989**

The ACI 318 design recommendations used the reinforced concrete design approach. The review in this section discuss specifically the related area of composite flexural members.
Ultimate strength

For the analysis of the ultimate moment of the composite section a full bond interaction is considered between steel and concrete. For the concrete stress distribution in the overall section, a uniformly distributed concrete stress of $0.85 f_{cc}$ is considered over an equivalent compression zone bounded by edges of the cross-section and a straight line parallel to the neutral axis at a distance $\beta$ from the extreme fibre concrete. $f_{cc}$ is the characteristic compressive cylinder strength of concrete. The factor $\beta$ depends on the concrete strength. It is 0.85 for concrete strength $f_{cc} \leq 30$ MPa and is reduced continuously at a rate of 0.05 for each 7 MPa of strength in excess of 30 MPa, but must not be taken to be less than 0.65. It is also assumed that tensile concrete carries no strength.

The stress in the steel below the yield strength $f_y$ is taken as $E_s$ (elastic modulus of steel) times the steel strain whilst the steel stress is considered as $f_y$ when the strains are greater than yield strain.

Properties of concrete

In the ACI design method no considerations are given for high strength concrete. The secant modulus of elasticity of concrete is calculated from the formula:

$$E_c = \rho^{1.5} 0.043 \sqrt{f_{cc}}$$

This formula is not adjusted for high strength concrete, and overestimates the modulus of elasticity for concrete strength above 40 MPa. Also for shrinkage and creep, no special considerations are included for high strength concrete.
Elastic flexural stiffness

The elastic flexural stiffness of the steel-concrete composite section in the ACI 318 is given as,

$$EI = 0.75(0.2E_c I_g) + E_s I_s$$

where $I_g$ is the second moment of area of the gross composite cross-section ignoring steel, $I_s$ is the second moment of area of the steel, $E_s$ is the modulus of elasticity of the steel and $E_c$ is the elastic modulus of concrete.

2.4.4 AS 3600-1994

The design method proposed in the Australian Standard AS 3600 uses the traditional reinforced concrete theory. The procedure described in AS 3600-1994 is mostly similar to that of ACI 318-89 discussed already in the previous section except in the consideration of elastic flexural stiffness of the composite section. In the assumption of elastic flexural stiffness the corresponding maximum moment capacity $M_{ub}$ of the cross-section is used. The secant flexural stiffness in the AS 3600 is presented as,

$$EI = 167 M_{ub} D$$

where $D$ is the depth of the gross cross-section of the composite beam.
CHAPTER 3

Experimental Study on The Flexural Behaviour of Rectangular Hollow Sections Filled with High Strength Concrete

3.1 INTRODUCTION

In this chapter, results of an experimental study are presented on the flexural behaviour of steel hollow sections filled with high strength concrete. Recently there has been a resurgence in the application of concrete-filled tubes as beam-columns in the construction industry. This has lead to the increased research studies and subsequently codification on the design of such members. Most research studies were concentrated on
the axial compressive behaviour of the element, with an applied bending moment derived from the eccentric load.

Recent studies (Lu and Kennedy, 1994) indicated concrete-filled RHS flexural members have much superior overall performance to the conventional hollow flexural members. The possible use of these composite flexural members are in the floor beams of industrial buildings and of offshore platforms, in the bridge girders and as other flexural members where high stiffness and particularly high ductility are critical. Furthermore, flexural behaviour of such members are also important in the development of design guidelines for concrete-filled RHS beam columns.

Thin-walled steel hollow members experience premature failure due to elastic local buckling under compression. This leads to the drastic loss of ductility in the post-failure range. In concrete-filled members the concrete core changes the buckling mode of steel in the compression flange from sine wave to cosine wave and thus elastic local buckling is either prevented or delayed. Concrete core also increases the ultimate moment of resistance of the overall section significantly by contributing in terms of sectional strength in the compressive zone. The application of high strength concrete (HSC) has increased in recent years in the construction industry in spite of great concerns about the ductility of the members of HSC (Attard and Mendis, 1993). However, because of the full steel confinement on the concrete core the ductility of the overall section has been improved. Another important factor which can influence the overall performance of the composite section is bond between the steel shell and the concrete core. This interface bond is depended upon the transfer of shear forces between steel and concrete when no
mechanical connectors are provided. In the concrete-filled RHS flexural members with high moment gradients where large amount of longitudinal shear stresses may present, interface bond can influence the overall performance.

The objective of this investigation is to study experimentally the flexural behaviour of the HSC-filled RHS composite sections with an emphasis on post-failure strength reserve and ductility. The aim is to investigate a composite section with high stiffness, strength and ductility so that it can be used in the heavy structural applications especially in the earthquake prone areas.

In this chapter test results are presented on HSC-filled RHS flexural members with different configurations. In some of the test specimens one Y16 bar was provided in the tensile zone to examine its effect on the overall strength of the composite section. Steel fibres were mixed with HSC to form the filler material in some of the specimens and two of the specimens were tested to failure without any filling for a comparison with the concrete-filled specimens.

3.2 PREPARATION OF SPECIMENS AND TEST SET UP

3.2.1 Preparation of specimens

In this series of experiments cold-formed RHS of 150 mm depth, 100 mm width and 4.0 mm thickness was used. The yield stress and ultimate stress of the RHS sections were
determined by a series of tensile coupon tests which are described later in this chapter. The total length and the span between supports of all the beam specimens were 3000 mm and 2750 mm, respectively. One reinforcing bar, Y16 of grade 400 was provided in some of the specimens to examine its effect. All Y16 bars were tack-welded to the bottom flange of the RHS beams with steel cover blocks of 25 mm near the ends to prevent any unwanted movement of the Y bar during concrete casting. At one end of the hollow RHS beams an end plate was tack-welded to enhance concrete pouring from the other end. The end plates were removed off before tests.

Care was taken to prepare the mix for high strength concrete. Type A Portland cement was used with a target compressive strength of 100 MPa. Densified silica-fume was mixed with cement in a ratio of 0.1 to 1.0. Coarse sand and fine sand were mixed together in equal proportions and 10 mm aggregate was used. In some of the beam specimens fibre-steel was mixed at 75 kg/m³ by replacing the 10 mm aggregate by equal weight which is considered as the most economical proportion for cost and performance (Fibresteel, 1992). The sizes of the steel fibres were 18 mm in length and 2 mm in width with ends rounded. A laboratory mixer was used and all materials were placed layer by layer. The water/cementitious ratio was taken as 0.27 which led to a dry mix initially but later with the addition of superplasticiser a workable mix of slump 100 mm was produced.

All concrete-filled specimens were cured in water for 28 days. The detail of the mix of fibrous high strength concrete is presented in Table 3.1.
Table 3.1 Mix detail of fibrous high strength concrete

<table>
<thead>
<tr>
<th>Material</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement Type (Ordinary Portland)</td>
<td>632 kg/m³</td>
</tr>
<tr>
<td>Silica-Fume (Densified)</td>
<td>63 kg/m³</td>
</tr>
<tr>
<td>Coarse Sand</td>
<td>253 kg/m³</td>
</tr>
<tr>
<td>Fine Sand</td>
<td>253 kg/m³</td>
</tr>
<tr>
<td>10 mm Aggregate</td>
<td>935 kg/m³</td>
</tr>
<tr>
<td>Fibre-steel</td>
<td>75 kg/m³</td>
</tr>
<tr>
<td>Water</td>
<td>189 kg/m³</td>
</tr>
<tr>
<td>Superplasticizer</td>
<td>15.0 ml./kg of the Cementitious</td>
</tr>
<tr>
<td>Slump</td>
<td>100 mm</td>
</tr>
<tr>
<td>Mean compressive strength</td>
<td>94.5 MPa</td>
</tr>
</tbody>
</table>

Material tests

To determine the mechanical strength of the steel RHS, tensile coupon tests were carried according to the specifications and guidelines mentioned in the code of practices (AS 1391, 1990). Four tensile coupons were cut from the four flat surfaces whilst four other specimens were taken from four corners. Finally the yield stress and the ultimate stress were determined by averaging the results obtained from all the tensile coupons. An averaged value of 434 MPa and 460 MPa was considered for the yield stress and the ultimate stress respectively. The averaged stress-strain diagram obtained from tensile coupon tests of the steel RHS is presented in Fig. 7.2 (chapter 7).

Concrete cylinders of 100 mm diameter and 200 mm length were tested to obtain the mean compressive strength of high strength concrete. Cylinders were taken for every batch of concrete during casting and were tested to failure according to the guidelines specified in the code of practice (AS 1012.9, 1992).
3.2.2 Test set up

A simply supported beam set up was adopted with a hinge and a roller at each end. The supports were constructed from steel plates and mild steel rods. Electrical resistance strain gauges were applied on the top and the bottom flanges of the RHS to record the extreme fibre strains. Load was applied on the top flange of the beam specimen at the midspan by a hydraulic jack driven by a stepped motor. The loads were recorded by a matching load cell which was placed in series with the loading cylinder. Vertical deflections at loading point were recorded by linear voltage displacement transducers. During the entire loading period data were recorded by a computerised data acquisition system. A detail of the test frame is presented in Fig. 3.1.

![Test set up](image)

**Fig. 3.1 Test set up**

3.3 TEST RESULTS AND DISCUSSION

Fourteen flexural members in five different configurations were tested in this series of experiments. The detail of test specimens under different configurations with the ultimate test loads are presented in Table 3.2.
<table>
<thead>
<tr>
<th>Test Specimens</th>
<th>Detail of the test specimens</th>
<th>Ultimate Load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HOL1</td>
<td>Specimens of RHS sections without any filling</td>
<td>42.3</td>
</tr>
<tr>
<td>HOL2</td>
<td></td>
<td>43.7</td>
</tr>
<tr>
<td>RH1</td>
<td>Specimens of RHS sections filled with high strength concrete(HSC)</td>
<td>88.5</td>
</tr>
<tr>
<td>RH2</td>
<td></td>
<td>83.1</td>
</tr>
<tr>
<td>RH3</td>
<td></td>
<td>90.4</td>
</tr>
<tr>
<td>RHR1</td>
<td>Specimens of RHS sections filled with HSC and with a reinforcing Y16 bar in the tensile zone</td>
<td>89.3</td>
</tr>
<tr>
<td>RHR2</td>
<td></td>
<td>90.2</td>
</tr>
<tr>
<td>RHR3</td>
<td></td>
<td>91.2</td>
</tr>
<tr>
<td>FRH1</td>
<td>Specimens of RHS sections filled with fibrous HSC</td>
<td>88.7</td>
</tr>
<tr>
<td>FRH2</td>
<td></td>
<td>87.8</td>
</tr>
<tr>
<td>FRH3</td>
<td></td>
<td>86.3</td>
</tr>
<tr>
<td>RFRH1</td>
<td>Specimens of RHS sections filled with fibrous HSC and with a reinforcing Y16 bar in the tensile zone</td>
<td>93.0</td>
</tr>
<tr>
<td>RFRH2</td>
<td></td>
<td>96.3</td>
</tr>
<tr>
<td>RFRH3</td>
<td></td>
<td>92.3</td>
</tr>
</tbody>
</table>

During the entire loading procedure a large amount of data were recorded by the computerised data acquisition system including the vertical deflections at the midspan (also the loading point), strains at top and bottom flanges of the composite and hollow beams which are plotted against the load levels in this chapter. In Fig. 3.2 vertical deflections are compared against the applied load for beam specimens of different configurations. Results showed that the hollow specimen failed much earlier than the concrete-filled specimens. Despite the RHS sections used in these experiments are classified as compact sections according to the codes of practices (AISC, 1991; AS 4100, 1992), the hollow flexural member (HOL2) failed rapidly after achieving maximum load with post-failure strength reduced drastically with the loss of ductility. The failure in the hollow member is due to severe elastic local buckling of the top flange under compression. In the concrete-filled specimens the failure load was significantly
increased due to the delayed local buckling of the steel shell in the compression flange and also due to the contribution of concrete in the compressive zone.

![Graph showing Load versus deflection](image)

**Fig. 3.2 Load versus deflection**

Significant increase in the post-failure strength and ductility, and some increase in the initial stiffness is also noticeable in the concrete-filled specimens. It can be noticed from Fig. 3.2 that mixing fibre-steel with concrete did not make any significant difference in ultimate load because the fibre-steel can contribute only in the tensile zone of concrete which is negligible when compared to overall strength of the section. The results indicate that the presence of one Y16 bar in the tensile zone in some of the specimens (e.g. RHR3 and RFRH2) did not increase the strength significantly, although a small amount of strength increase is noticeable. This is possible due to following reasons, one is the length of lever-arm of the internal couple due to Y16 bar is significantly less because of the small depth (150 mm) of the overall section, and another reason is the
area of a Y16 bar (200 mm$^2$) is much less than the sectional area of the steel RHS section (1936 mm$^2$).

In Fig. 3.3 top flange strains are plotted against the load level. The negative values indicate compression. Because of high ductility in the post-failure range large magnitude of strains were recorded. Due to large deformations most of the gauges peeled off before the failure load was reached. In the hollow member (HOL2) first strain was compressive then after premature failure strain became tensile due to elastic local buckling (Fig. 3.3). Results showed superior stiffness of load-strain curve for composite beams RHR2 and RFRH3 when compared to composite beams RH3 and FRH1 which is due to the presence of Y16 bar in the tensile zone for the composite beams RHR2 and RFRH3. Fig. 3.4 presents bottom flange strains against the applied load for different flexural members under each configurations. Once again, for concrete-filled specimens high
ductility in the post-failure range is noticeable although full curve is not available due to early peeling of the strain gauges.

![Graph showing load versus strain for different specimens.](image)

**Fig. 3.4 Bottom flange strains**

The strain reading of hollow specimen (HOL2) shows premature failure which is due to local buckling. The load versus strain plots in Fig. 3.4 shows stiffer curves are obtained for the composite beams with steel fibres (FRH1 and RFRH3) in comparison with RH1 and RHR3 respectively. The reason for this can be the improved performance of the tensile concrete due to the addition of steel fibres. Stiffer curves are also achieved for the composite beams with Y bars (RHR3 and RFRH3).

The bending curvature of the flexural members can be calculated from the strain readings as

\[ \kappa = \frac{-\varepsilon_{\text{top}} + \varepsilon_{\text{bot}}}{D} \]  

(3.1)
where $\varepsilon_{\text{top}}$ and $\varepsilon_{\text{bot}}$ are the strains recorded on the top and bottom flanges at the mid span, $\kappa$ is the bending curvature, and $D$ is the overall depth of the RHS section. The resulting test moment versus curvature relationships are plotted in Fig. 3.5.

![Graph showing moment-curvature relationship](image)

**Fig. 3.5 The moment curvature relationship**

The results showed once again the highly ductile post-failure behaviour of the concrete-filled members. Due to the peeling off the strain gauges the bending moment-curvature curves are incomplete. The negative curvature obtained in the moment-curvature curve for hollow member HOL2 is due to local buckling of the top flange.

In this study effort has been made to relate the depth of neutral axis against the increasing moment. The depths of neutral axis are calculated from the recorded strains on the top and bottom flanges under the assumptions of plane remains plane after
bending and strain compatibility exists between the steel shell and the concrete core
throughout the loading history. The expression for the depth of neutral axis is given by

\[ d_{NA} = \frac{-D\varepsilon_{up}}{-\varepsilon_{up} + \varepsilon_{bot}} \quad \text{(3.2)} \]

where \( d_{NA} \) is the distance between the neutral axis and the extreme fibre of the top
flange of steel. The relation between \( d_{NA} \) and the bending moment at the mid span is
presented in Fig. 3.6. Although Eq. 3.1 and Eq. 3.2 used to define curvature and depth
of neutral axes respectively it should be noted that these equations (Eq. 3.1 and Eq. 3.2)
are not valid once the local buckling occurs on the compression flange.

\[ \text{Fig. 3.6 The position of the neutral axis} \]
Fig. 3.6 shows that $d_{N4}$ is more stable in the medium range of loading. It is approximately 75 mm for the specimen without concrete filling and in the range of 65 mm for the concrete-filled specimens. It is noticeable that depth of neutral axis increases for the specimens with Y bars (RHR2 & RFRH1) which indicates increased moment capacity due to more concrete contribution. The depth of neutral axis also increases for the concrete-filled specimens with fibre-steel (FRH3 & RFRH1). For hollow specimens (HOL2) neutral axis depth reached a value of 75 mm which is also the centroidal position of the overall section.

In this series of experiments two of the concrete-filled beam specimens RH2 and FRH2 were subjected to one cycle loading and unloading. The resulting load-deflection curves are plotted in the Fig. 3.7. It can be noticed that due to the early unloading for the specimen RH2 load carrying capacity does not change in the post-failure range but in case of specimen FRH2 there is some drop in the load carrying capacity. This is due to the reason that unloading occurs at close to the failure load. For both of the specimens post failure behaviour is enormously ductile.
After all the beams were loaded to failure, one concrete-filled specimen was cut open by a diamond saw to examine the cracking and crushing of concrete and also if there is any debonding of concrete core against the steel shell. A portion was cut from the composite beam while keeping the centre of the portion as the loading point (which is also the midspan of the beam). An enlarged picture of the concrete core is presented in Fig. 3.8.
It can be noticed from the figure that local buckling of the steel shell occurred at the loading point and just beneath the local buckling area of steel local crushing of concrete is also visible. Large cracks are noticeable in the tensile zone of concrete around the loading point. However, no debonding of the concrete core against the steel shell is visible in the naked eyes. The reason for this is due to mid-span loading on the flexural members which can hardly expect any interface slip. Slippages can occur away from the mid-span.

3.4 SUMMARY

Results are reported of a series of experiments on flexural members constructed from rectangular hollow steel sections filled with high strength concrete. The compressive strength of concrete is 94.5 MPa. Fibre steel, and also a Y16 reinforcing bar were used in some of the specimens to investigate if there is any change on the flexural behaviour. Large amount of data consisting of strains and displacements in different load levels were recorded by an automatic data acquisition system.

From this study, it can be concluded that concrete-filled flexural specimens have much superior overall performance to hollow section members in terms of sectional strength, ductility and initial stiffness. Hollow section members after achieving the premature peak load lost strength rapidly with drastic loss of ductility in the post failure range and the yield strength of steel not fully utilised. Once filled with concrete, the flexural
member behaved in a highly ductile manner. In general, high strength concrete is a brittle material but the test results showed dramatic improvement of ductility of the overall composite section when high strength concrete is within the steel confinement. The inward local buckling of steel shell was also prevented by the concrete core, and thus having much higher buckling strength. High magnitude of strain readings obtained in the tests indicated the stress in the extreme fibre steel both in compression and in tension are well beyond yield stress.

Results shown by providing one Y16 reinforcing bar in the tensile zone of the composite section improved the sectional strength although marginally. The addition of fibre steel to the high strength concrete mix did not influence the sectional strength although some improvement in ductility is visible from the plotted results. Portion of one concrete-filled specimen was cut open after the completion of experiments to examine the behaviour of concrete core within the steel confinement. At the point of loading compression crushing and tensile cracking of the concrete core were noticed but no visible debonding of the concrete against the steel was noted.
CHAPTER 4

Experimental Study on The Beam-Column Connections of Steel Rectangular Hollow Sections Filled with High Strength Concrete

4.1 INTRODUCTION

Study has been carried out on the behaviour of concrete-filled tube members over many years, resulting in the provision of design methods for concrete-filled steel members in different design codes and guidelines (Eurocode 4, 1994; ACI 318, 1989). Research (Ge and Usami, 1992) demonstrated that concrete-filled RHS system has large energy absorption capacity which is critical in the event of an earthquake. In earthquake prone areas, this form of construction has already provided excellent performance in building and highway construction, particularly in Japan (Hanshin Expressway, 1986).
Recent studies (Lu and Kennedy, 1994; Prion and Boehme, 1994) indicated that rectangular hollow sections filled with high strength concrete can be used effectively as flexural members providing a rigid system with significant increase in bending moment capacity, ductility and post-failure strength reserve compared to the hollow section members. It is therefore advantageous to achieve a rigid moment connection between the composite column and the composite beam, leading to an excellent system not only for static loading but also for seismic loading and for situation where a stiffer structural system with reduced vibration effect is needed. This type of rigid system is particularly applicable in highway bridge constructions and offshore oil platforms to reduce the need for lateral bracings. Studies (Packer and Kenedi, 1993) on the rigid connection of concrete-filled steel hollow sections appeared to give much superior performance in space trusses and frame structures when compared to hollow member connections.

In this chapter experimental studies were conducted on beam-column connections where both beam and column members were constructed from steel RHS filled with high strength concrete. Experiments were carried out in two series. In each series different beam-column connection configuration was adopted. In the first series, beams were placed on top of the short columns forming a rigid knee connection while beam remains cantilever. Nine rigid knee connections including one hollow connection were tested. In some of the specimens one Y20 reinforcing bar was placed in the tensile zone of the beam to examine its effect. In the second series cantilever composite beams were connected to the side of the short composite columns. In this series connection specimens were tested under two types of configurations of the inside steel surface of the RHS. In some of the specimens inside steel surface was greased thoroughly
throughout the lengths of beam and column portions of the connections to examine the
effect of the steel-concrete interface into the overall performance of the connection.
Inside steel surfaces of the other specimens were kept untreated. All round welds were
used to connect the beam and column elements.

The objective of this research is to study the behaviour of rigid beam-column
connections between composite columns and composite beams. Emphasis was given to
examine the ultimate strength, rotational capacity, post failure strength and ductility of
the connected elements and welds of the connection assembly. Efforts were also made
to measure the slippage between the steel shell and the concrete core.

4.2 PREPARATION OF SPECIMENS

The experiments on composite beam-column connections presented in this chapter were
conducted in two series. In both series of connection study cold-formed RHS of grade
350 were used. The yield stress and ultimate stress were determined by a series of
tensile coupon tests which has been presented in chapter 3. The section geometry of the
RHS were 150 mm depth, 100 mm in width and 4 mm wall thickness. In the first series
of experiments lengths of 500 mm and 300 mm were cut for the beam and column
portion, respectively. The beam portion was then positioned on top of the column
portion so as to leave a 300 mm cantilever beam on one side of the column. At the other
end one end plate was welded to the beam portion to assist the pouring of concrete. The
connected portion was then welded by a combination of fillet and butt welds. Altogether
nine specimens were prepared, including one without concrete filling. Special care was taken to weld the connections. A large size weld was used to make sure connections should not fail in the weld. Smaller size weld was used in the hollow connection specimen because the expected failure load is very low due to local crushing. In one concrete-filled connection specimen thinner weld was used to examine the effect and to compare with the behaviour of the other connections. In four of the specimens one reinforcing bar of Y20 (Grade 400 MPa) was provided in the tensile zone of the section. The reinforcing bar was connected to the end plate of the beam portion by penetrating weld to provide enough bond for the bar at the connection. The other end of the bar was tack welded to a spacer bar which was welded to the top flange of the RHS beam portion. A 25 mm cover was kept for the Y bar from the inside steel surface of the top flange. An electronic resistance strain gauge was attached to the Y bar for two of the test specimens to examine the strain distribution through the composite section. Effort was also made to record the strain in concrete face in four of the test specimens and for this purpose a small slot was kept in the compressive flange of the beam portion which was covered during concrete casting by using small wooden chips of size equal to the slot and the thickness of RHS so that the concrete surface can be revealed after concrete casting. The details of the strain gauges including their positions in the connection test specimens has given in Fig. 4.1.

In the second series of experiments lengths of 500 mm and 300 mm were cut from RHS as column and beam segment, respectively. Cantilevered beam portion was connected to the narrower side of the short column while the top of the beam was placed in a position which is 50 mm lower than the top of the column. The connection was achieved by a
combination of butt and fillet weld. Large size weld was used to ensure connection must not fail in the welds. On top of the short column a thin end plate was tack-welded to enhance concrete pouring which was removed before the experiments.

In this series of experiments special emphasis was given to bond behaviour at the interface. Of five connection specimens two were greased thoroughly at the inside steel surface to the full length of both beam and column segment. In the other three specimens inside steel surface were kept untouched. In two of the specimens a slot was kept in the bottom flange of the beam portion to measure the compressive strain in concrete.

Care was taken for the mix preparation of high strength concrete. Densified silicafume was added in ratio of one-tenth of the Type A Portland cement. The mixed ratio of cement : sand : coarse aggregate was 1 : 0.8 : 1.6. 10 mm aggregate was used in the mix. The water/binder ratio was 0.27. Initially the mix was very dry due to very low water/cement ratio but later by adding superplasticiser and water reducer a workable mix was achieved with high slump values. A target compressive strength of 100 MPa was considered in the begining. The compressive strength of the concrete was later determined by a series of standard cylinder tests. All concrete-filled connection test specimens were cured in water for 28 days. A detail of the concrete-mix is presented in the Table 4.1.
### Table 4.1: Detail of high strength concrete mix

<table>
<thead>
<tr>
<th>Component</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement Type (Type A Portland)</td>
<td>632 kg/m³</td>
</tr>
<tr>
<td>Silica Fume (Densified)</td>
<td>63 kg/m³</td>
</tr>
<tr>
<td>Fine Sand</td>
<td>253 kg/m³</td>
</tr>
<tr>
<td>Coarse Sand</td>
<td>253 kg/m³</td>
</tr>
<tr>
<td>10 mm Aggregate</td>
<td>1010 kg/m³</td>
</tr>
<tr>
<td>Water</td>
<td>189 kg/m³</td>
</tr>
<tr>
<td>Superplasticiser</td>
<td>8.0 ml/kg of cementitious</td>
</tr>
<tr>
<td>Water Reducer</td>
<td>4.0 ml/kg of cementitious</td>
</tr>
<tr>
<td>Slump</td>
<td>150 mm / 140mm*</td>
</tr>
<tr>
<td>Average Compressive Strength</td>
<td>87 MPa / 85 MPa*</td>
</tr>
</tbody>
</table>

* second series of experiments

### 4.3 TEST SET UP

All connection specimens were welded with a 25 mm thick base plate which was bolted to the main loading frame with four M36 bolts to ensure a fixed support. Base plate and bolts were designed for maximum load and moment prediction at the base of the short column of the concrete-filled connection specimens. Load was applied vertically 25 mm from the free end of the 300 mm cantilever beam portion by 500 kN hydraulic jack driven by a stepped motor. The load levels were recorded by a matching load cell.
Transducers were placed vertically to record displacements at the load application point by attaching the transducer with the loading cylinder, and at the connection. Relative slip between steel shell and the concrete core was also recorded at the free end of the cantilever beam portion in one of the specimens. The test data were recorded by a computerised data acquisition system. A schematic drawing of the test set up of the first series of experiments is shown in Fig. 4.1. In Fig. 4.2 a photograph of the test set up of the second series of tests is presented.

Fig. 4.1 Test Set Up (First series)

Fig. 4.2 Test Set Up (Second series)
4.4 RESULTS AND DISCUSSION

Nine rigid-knee connection specimens were tested to failure in the first series of experiments including one hollow (CHOL) and eight concrete filled specimens. Amongst filled-up specimens four specimens had one Y20 reinforcing bar in the tensile zone of the beam portion (CNR1 to 4). The bar was placed 25 mm from the inner surface of the top flange. In the specimens CN1 to 4 no reinforcement was provided. Specimens CN3, CN4, CNR3 and CNR4 had the narrow slots in the steel section of the beam portion to enable measurement of concrete surface strain. These results were used to check strain compatibility between steel and concrete surfaces.

Five beam-column connection specimens marked as GCN1, GCN2, SCN1, SCN2 and SCN3 were tested in the second series of experiments. In the first two specimens a thin layer of grease was applied on the inside steel surface while the last three specimens were kept untouched. In specimens SCN2 and GCN1 narrow slots in the steel were kept on the compression flange of the beam portion during concrete casting to examine strain compatibility at the steel-concrete interface. Rotational deformations of the beam portions were calculated from deflection readings at the point of loading and at the connection. The ultimate moment capacities and maximum deflections at the loading point for all the specimens are presented in Table. 4.2.

Table 4.2 demonstrates the ultimate moment for the hollow section CHOL is only 17.71 kNm which is almost one-third of the moment capacities of the concrete-filled sections. This is due to the local crushing and buckling failure at the load application point in the
top flange. The failure occurred much early before the yield stress had been achieved. In concrete-filled specimens failure occurred after yield stress had been reached in most of the specimens. Specimen CN1 failed early due to inadequate weld in the baseplate at the moment of 34.13 kNm. CN2 also experienced welding failure but at the connection and at a later stage when compared to CN1.

Table 4.2 Ultimate moment and Maximum deflection at loading point

<table>
<thead>
<tr>
<th>Series No.</th>
<th>Test Specimens</th>
<th>Ultimate Moment kNm</th>
<th>Maximum Deflection mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st Series</td>
<td>CHOL</td>
<td>17.71</td>
<td>26.60</td>
</tr>
<tr>
<td></td>
<td>CN1</td>
<td>34.13</td>
<td>18.27</td>
</tr>
<tr>
<td></td>
<td>CN2</td>
<td>42.65</td>
<td>29.02</td>
</tr>
<tr>
<td></td>
<td>CN3</td>
<td>51.56</td>
<td>51.58</td>
</tr>
<tr>
<td></td>
<td>CN4</td>
<td>50.13</td>
<td>41.15</td>
</tr>
<tr>
<td></td>
<td>CNR1</td>
<td>49.69</td>
<td>61.20</td>
</tr>
<tr>
<td></td>
<td>CNR2</td>
<td>49.80</td>
<td>44.30</td>
</tr>
<tr>
<td></td>
<td>CNR3</td>
<td>52.75</td>
<td>54.89</td>
</tr>
<tr>
<td></td>
<td>CNR4</td>
<td>53.02</td>
<td>55.21</td>
</tr>
<tr>
<td>2nd Series</td>
<td>SCN1</td>
<td>48.07</td>
<td>53.44</td>
</tr>
<tr>
<td></td>
<td>SCN2</td>
<td>50.29</td>
<td>52.87</td>
</tr>
<tr>
<td></td>
<td>SCN3</td>
<td>49.75</td>
<td>54.97</td>
</tr>
<tr>
<td></td>
<td>GCN1</td>
<td>48.65</td>
<td>73.73</td>
</tr>
<tr>
<td></td>
<td>GCN2</td>
<td>47.74</td>
<td>73.16</td>
</tr>
</tbody>
</table>

Table 4.2 shows that in greased specimens deflections are significantly larger than that of specimens with normal steel surfaces. This dramatic reduction in stiffness is due to the fact that there is no frictional bond between the steel shell and the concrete core in the greased specimens. Small reduction in the ultimate moment capacity is noticeable in the greased specimens. The slight reduction in strength attributes to the fact that although no frictional force exists between the steel surface and the concrete core, there is still a significant restraint in the normal direction of the steel surface provided by the concrete core.
Failure was initiated in the column portions due to large combined bending and compressive stress. Local buckling eventually occurred near the column base in the compressive zone when applied load approached the failure load. In the beam portions the top flange steel remained elastic and the bottom flange steel was just yielded for normal surfaced specimens whereas for greased specimens the bottom flange steel of the beam portion yielded heavily after acquiring a strain of 7000 microstrain.

![Graph showing moment vs rotation](image)

**Fig. 4.3** Moment versus rotation

In Fig. 4.3 rotations of the connection joints are plotted against moments for five of the specimens with different configurations. Results show connection specimens with different configurations failed in different ways. Hollow specimen (CHOL) failed by local buckling with sudden loss of strength and ductility. CN2 experienced failure due to inadequate weld at the connection and due to rust formed during curing of the specimen in the curing tank, resulting the moment-rotation curve fall sharply. Another knee connection CNR4 failed in a ductile manner. It is clearly noticeable that the rotational
stiffness for the greased specimen (GCN1) is significantly less than that of the specimen with untouched surface (SCN3), yet the reduction in strength is not significant. Results also indicated despite a Y20 bar in the tensile zone of the beam portion, CNR4 has lesser stiffness than the specimen SCN3. The reason for this is the stiffer beam in the second series of experiments than the knee connection specimens in the first series. The tests were finally stopped due to excessive inclination of the loading jack.

![Graph showing strain vs. moment for different specimens](image)

**Fig. 4.4** Top flange strains in the beam portions

In Fig. 4.4 strains on the top flange of the cantilever beam portions are plotted against the moment. It can be noticed that there are differences in the results of connection specimens of the first series (CN3 and CNR3) whilst in the second series connection specimens (SCN1 and GCN1), variations are not significant. Strain reached a value 8000 microstrain in the specimen CN3 which is without the Y bar whilst in CNR3 (specimen with a Y bar) extreme steel fibre just yielded and a strain reading of 3200 microstrain is achieved. This difference is due to the shift of neutral axis position.
towards the tensile flange in the presence of Y bar. In the specimens GCN1 and SCN1, the maximum strain reached at about 1750 microstrain, which is slightly below the yield strain of the steel. The noticeable feature is that when the strain reaches the maximum value, it slightly decrease. This can be explained by the fact that both specimens failed at the column portion and when column portion failed, some recovery of elastic deformation is observed in the beam portion. After the column portion failed, the beam portion deflected as a rigid body without increase in strain.

![Graph showing bottom flange strains on steel and concrete in the beam portions](image)

**Fig. 4.5 Bottom flange strains on steel and concrete in the beam portions**

Fig. 4.5 presents bottom flange strains of the cantilever beams for the connection specimens with three different configurations. Both strains of steel and concrete were measured to examine the strain compatibility at the interface. Results show strain compatibility does not exist from the beginning. It can be seen from the results that the strain differences between steel and concrete is much higher in greased specimen GCN1 than other specimens. The reason for this is the greased layer which leads to large
amount of debonding at the steel-concrete interface for the specimen GCN1. Results also indicated that the concrete strain is stiffer in the specimen SCN2 when compared to the rigid-knee specimen CN4 which is due to the stiffer beam of SCN2 than the beam of CN4.

![Graph showing tensile strains in the columns](image)

**Fig. 4.6** Tensile strains in the columns

Tensile strains in short columns of three connection specimens are plotted against the moment in Fig. 4.6. Test results demonstrated CNR1 has the highest stiffness than the specimens SCN3 and GCN2 which is due to the shorter and stiffer column portion of the knee connection. A significant reduction of stiffness was observed in the greased specimen GCN2 after the moment reached a value of 25 kNm, when compared to the untreated specimen SCN3. The compressive strains in the column portion of the connections in three different configurations are presented in Fig. 4.7. The specimen CNR1 again shows a stiffer moment versus strain curve than other specimens which is due to the shorter length (300mm) of the short column member of the knee connection.
when compared to 500 mm long short column member of the connections of the second series. The stiffness of GCN2 is reduced after moment reaches 25 kNm. Results in Fig. 4.6 and Fig. 4.7 demonstrated large amount of ductility and post-failure strength reserve.

![Graph showing compressive strains in the column portions](image)

Fig. 4.7 Compressive strains in the column portions

Slip of the concrete core with respect to the steel shell was measured in two of the specimens with different configurations (SCN3 and GCN2). The results are presented in Fig. 4.8. To measure the relative slip, the transducer was placed on the concrete core on the top of the column portion and was fixed to a steel bracket which was welded to the steel shell. Slip in the normal surfaced specimen SCN3 is not noticeable even at relatively higher load level, indicating significant bond exists between the steel shell and the concrete core. It became noticeable at the later stage of the loading as a result of large rotational deflection. The overall slip is not significant with a maximum value of less than 1 mm. In the specimen GCN2, the noticeable separation started at a lower load
level. The slip increased continuously throughout the loading history with an ultimate slip of about 5 mm.

Fig. 4.8 Slip at the steel-concrete interface

Fig. 4.9 Concrete core in the beam portion of the rigid knee connection (after failure)

After failure, one beam portion of the rigid knee connection was cut open to reveal the concrete core (Fig. 4.9). Fig. 4.9 showed large cracks on the left side and on the top face (tensile zone) of the concrete core which was the connected end. The concrete was also
crushed at the connection but at the bottom face which is the compressive zone. The large angular cracks near the right hand side was due to applied loading.

4.5 SUMMARY

The experiments were carried out on connections between composite beam and composite column in two series. In the first series, nine connections were tested including one hollow specimen which failed by local buckling at the point of loading. The cylinder compressive strength of concrete was 87 MPa. Two specimens experienced weld failure; one at the connection joint and the other at the base plate due to inadequate welding. The rest of the specimens developed yielding failure either in the beam portion or the column portion of the connection depending upon the specimen configuration.

In the second series of experiments, concrete-filled RHS beam-column connections were tested to failure. The compressive strength of concrete was 85 MPa. A thin layer of grease was applied inside the steel surface before casting in two of the specimens to study the effect of bonding between the steel surface and the concrete core. Strains in both steel and concrete were measured to examine the strain compatibility. Relative slips between the steel shell and the concrete core were also measured.

From this study it can be concluded concrete-filled RHS connections behaved in extremely ductile manner. It has a significant post-failure strength reserve even in greased specimens. It was observed that in the specimens with totally debonded concrete
core the stiffness is reduced dramatically whilst the reduction in ultimate strength is insignificant. The relative slip between the steel shell and the concrete core for the greased specimens is significantly larger when compared to that in the normal surfaced specimens. Strain incompatibility was observed throughout the loading period between steel and concrete at the interface. It can also be concluded that a reliable rigid connection can be achieved if the weld is designed properly to transfer the load.
CHAPTER 5

Experimental Study on The Pushout Behaviour of Steel Rectangular Hollow Sections Filled with High Strength Concrete

5.1 INTRODUCTION

In the analysis and design of concrete-filled steel tubular columns and beam-columns a complete compatibility of strains between steel and concrete is assumed in the codes of practices (Eurocode 4, 1994; ACI 318, 1989). A number of studies (Furlong, 1967; Knowles and Park, 1969; Rangan and Joyce, 1992; Prion and Boehme, 1994) indicated that the bond between steel and concrete has little or no significance in relation to the performance of columns and beam-columns. However, when longitudinal shear stresses are predominant as in the case of flexural member, bond strength may have a significant
role when no shear connectors are provided. To investigate the bond behaviour between steel and concrete, pushout tests on RHS filled with high strength concrete (HSC) were carried out and the results are presented in this chapter.

Previous studies (Virdi and Dowling, 1980; Shakir-Khalil, 1991) indicated the pushout behaviour primarily depends on the imperfections of the steel. These imperfections are the roughness of the inside surface, the shape and the size of the cross-section of the steel hollow section. In the codes of practices (Eurocode 4, 1994) the value of bond strength in concrete-filled steel tubular sections is recommended as 0.4 N/mm². However, this bond strength is based on normal strength concrete as a filler material. Study (Shakir-Khalil and Hassan, 1994) demonstrated that when steel tubes were filled with higher strength concretes bond strength decreases significantly and became almost half of the bond stress values compared to test specimens with normal strength concrete. The reason for this may be the presence of higher cement content present in HSC which causes higher shrinkage (Swamy and Anand, 1973) and thus diminishing the keying action between steel and concrete. However, more study is needed to define the bond behaviour of HSC-filled steel tubes.

The objective of this study is to examine experimentally the bond behaviour between steel and concrete of RHS filled with HSC without mechanical connectors. Study emphasizes the effect of different inner surface treatment and the cross-section geometry of the RHS.
In the scope of this study twelve concrete-filled RHS specimens were used. Two types of cross-sections were adopted, being $100 \times 150 \times 4.0$ RHS and $75 \times 75 \times 2.5$ RHS. Three different inside steel surface treatments were adopted for the cross-sections. They are acid-washed, greased and untouched surface. In all the composite specimens, the concrete core was pushed through the steel shell. The load versus slip curves were then plotted to analyse test results.

**5.2 PREPARATION OF TEST SPECIMENS**

In this series of experiments steel rectangular hollow sections of two different cross-sectional dimensions were used; they are RHS $100 \times 150 \times 4.0$ (100 mm width, 150 mm depth and 4 mm wall thickness) and RHS $75 \times 75 \times 2.5$ (75 mm width and depth and 2.5 mm wall thickness). In total twelve hollow section specimens were cut into 450 mm lengths being six in each cross-sectional dimension category. The ends of all specimens were then machined up to ensure an even end support. Three types of surface configurations were adopted for the inside steel surface in each cross-section category, they are greased, acid-washed and untouched normal surface. All specimens were washed by water hose before inside surface treatment so that no impurities and dirt were present. In two of the six specimens in each type of cross-sections inside surface was washed by diluted hydrochloric acid to remove the paint and reveal the actual steel surface. The inside surface of two other specimens in each hollow section size was greased by a thin layer of oil with a mop throughout the length of the specimens. In the remaining four specimens, the inside steel surface was untouched. A thin plate was tack-
welded at the end of every specimen to aid concrete pouring, and also to achieve flushed surface of steel and concrete. These plates were later removed by cutting carefully with handsaw before the tests. Care was taken to prepare the mix of high strength concrete. Initially a target strength of 100 MPa was considered. Portland Type A cement was used which was then mixed with densified silica fume. Water to cementation ratio was kept at 0.27. The addition of superplasticiser and water reducer produced a very workable mix. All specimens were filled up to a length of 400 mm leaving a 50 mm gap from one end of the specimen. Cylinders were taken for every batch of concrete and later tested to determine the compressive strength of high strength concrete. All specimens were cured in water for 28 days. A mix-detail of high strength concrete with slump value and average compressive strength is presented in Table 5.1.

<table>
<thead>
<tr>
<th>Table 5.1 Mix-detail of High Strength Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement Type (Type A Portland)</td>
</tr>
<tr>
<td>Silica Fume (Densified)</td>
</tr>
<tr>
<td>Fine Sand</td>
</tr>
<tr>
<td>Coarse Sand</td>
</tr>
<tr>
<td>10 mm Aggregate</td>
</tr>
<tr>
<td>Water</td>
</tr>
<tr>
<td>Superplasticiser</td>
</tr>
<tr>
<td>Water Reducer</td>
</tr>
<tr>
<td>Slump</td>
</tr>
<tr>
<td>Average Compressive Strength</td>
</tr>
</tbody>
</table>
5.3 TEST SET UP AND INSTRUMENTATION

Concrete-filled stub-column specimens were supported on a 24 mm thick plate which was fixed to the loading frame. Special care was taken to ensure no horizontal movement would occur at the time of loading. The bottom of the stub-column specimens were without concrete filling for 50 mm length from the bottom end which was kept for the travel of the 400 mm length concrete core. Steel blocks of 50 mm thickness were made in the size of the cross-section of the concrete portion and placed at the top of the specimen to push the concrete core through the steel shell. On top of these steel blocks another steel block with a recess for the high yield stress steel ball was welded. This steel ball also fits the recess of the loading cylinder. The concrete core was then pushed through the steel shell by the hydraulic jack driven by a stepped motor and a matching load cell was used to record the level of loads. A displacement transducer was attached with the cylinder to record the travel of the concrete core. A photograph showing all details of the test set up is presented in Fig. 5.1.

![Test set up](image_url)

Fig. 5.1 Test set up
5.4 TEST RESULTS AND DISCUSSION

All the specimens were loaded until the concrete core of each specimen hit the base plate after travelling 50 mm through the steel shell. In Table 5.2 the details of test specimens with the peak pushout loads are presented.

Table. 5.2

<table>
<thead>
<tr>
<th>Specimen Nos.</th>
<th>Size of the cross-section of the steel hollow section (mm x mm)</th>
<th>Treatment of the inside steel surface</th>
<th>Peak push-out load (kN)</th>
<th>Travel of the concrete core at the peak load (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ST1</td>
<td>100 x 150</td>
<td>normal</td>
<td>14.9</td>
<td>35.26</td>
</tr>
<tr>
<td>ST2</td>
<td>100 x 150</td>
<td>normal</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>AC1</td>
<td>100 x 150</td>
<td>acid-washed</td>
<td>24.0</td>
<td>*</td>
</tr>
<tr>
<td>AC2</td>
<td>100 x 150</td>
<td>acid-washed</td>
<td>22.7</td>
<td>3.64</td>
</tr>
<tr>
<td>GS1</td>
<td>100 x 150</td>
<td>greased</td>
<td>5.1</td>
<td>33.71</td>
</tr>
<tr>
<td>GS2</td>
<td>100 x 150</td>
<td>greased</td>
<td>5.2</td>
<td>32.17</td>
</tr>
<tr>
<td>SST1</td>
<td>75 x 75</td>
<td>normal</td>
<td>31.3</td>
<td>5.9</td>
</tr>
<tr>
<td>SST2</td>
<td>75 x 75</td>
<td>normal</td>
<td>29.9</td>
<td>6.47</td>
</tr>
<tr>
<td>SAC1</td>
<td>75 x 75</td>
<td>acid-washed</td>
<td>28.5</td>
<td>8.25</td>
</tr>
<tr>
<td>SAC2</td>
<td>75 x 75</td>
<td>acid-washed</td>
<td>28.0</td>
<td>21.18</td>
</tr>
<tr>
<td>GS1</td>
<td>75 x 75</td>
<td>greased</td>
<td>3.25</td>
<td>11.0</td>
</tr>
<tr>
<td>SGS2</td>
<td>75 x 75</td>
<td>greased</td>
<td>7.5</td>
<td>22.31</td>
</tr>
</tbody>
</table>

* Problem in the data acquisition system

From the results it can be noticed that in some of the specimens the peak load was achieved when concrete core has travelled a significant distance. The possible reason for this can be the roughness of the inside steel surface with more interlocking at the
interface. The results indicated that the short-column specimens with smaller cross-sections offered more bond. The plots of load levels against the travel of the concrete core are presented in Fig. 5.2 for the concrete-filled square $75 \times 75 \times 2.5$ RHS sections.

![Graph showing load versus the travel of concrete in concrete-filled 75 x 75 x 2.5 RHS sections.](image)

Fig. 5.2 Load versus the travel of concrete in concrete-filled 75 x 75 x 2.5 RHS sections

Fig. 5.2 showed that in the pushout specimen with normal inside steel surface (SST2) peak pushout load reached at an early stage and then debonding occurred with sudden loss of bond and then concrete core travelled almost freely after the debonding. In the specimen SAC2 which has acid-washed surface, peak load occurred at a later stage. After an earlier debonding and the fall of bond stress the pushout load increased again due to the better interlocking of steel and concrete than the specimen SST2. However, for both acid-washed and normal specimens the initial stiffness for bond is almost identical. For the greased specimen (SGS2) the initial bond stiffness was significantly less than other specimens and peak load occurred at much later stage.
Fig. 5.3 Load versus the travel of concrete in concrete-filled $75 \times 75 \times 2.5$ RHS

Fig. 5.3 indicated a very low bond strength in the greased specimen SGS1. The reason for this may be a thicker layer of grease applied during specimen preparation when comparing to the specimen SGS2 (Fig. 5.2). Results (Fig. 5.3) also showed that a larger bond stress occurred in the untreated specimen SST1 than the acid-washed specimen SAC1. The load-slip curve of SST1 demonstrated the fluctuation of load before the bond finally lost. This up and down of bond stress may occur due to the undulation in the inside surface of steel. It can be noticed from Fig. 5.3 that the initial bond stiffness is higher in the acid-washed specimen (SAC1) when compared to untouched specimen (SST1). This is possibly due to the removal of paint by acid-wash which may lead to a better interlocking between steel and concrete. Results indicate that in case of the normal and acid-washed specimens, peak load was achieved early in some of the specimens which is within 10 mm of the travel of the concrete core where in others peak
load reached at a later stage. In greased specimens peak load occurred when concrete core travelled 30 mm through the steel hollow sections.

![Graph](image)

**Fig. 5.4** Load versus concrete travel in concrete-filled 100 × 150 × 4 RHS

In Fig. 5.4 acid-washed specimen AC1 had more bond strength in the initial loading but after the loss of bond the concrete core travels at the same load level as in the normal specimen ST1. ST1 had less bond stress in the early stages of the loading which may be due to the smoothness caused by paint, but after possible removal of paint by friction at the interface the bond strength increased and reached at the same level as in the acid-washed specimen. The greased specimen GS1 showed significantly less bond stiffness compared to other specimens which is due to almost no bond situation at the interface.

In Fig. 5.5 curves are plotted for bond stress versus travel of concrete core per mm interface length for a comparison of test results between the pushout specimens. Results are presented for six of the total eight acid-washed and untreated specimens. In the other

72
two specimens (AC1 and ST2) results are incomplete due to a fault in the data acquisition system.

![Graph showing bond strength in concrete-filled RHS](image)

**Fig. 5.5 Bond strength in concrete-filled RHS**

Greased specimens are not plotted due to very low bond stress obtained during the experiments. Results demonstrated that higher bond stress was achieved in the narrower cross-sections (75 x 75 x 2.5) when compared to composite specimens AC2 and ST1 (150 x 100 x 4.0). The reason for this is the greater confinement effect at the corner of the narrower sections. The bond stress achieved in the acid-washed and untreated composite sections with 75 x 75 x 2.5 RHS is about 0.25 MPa which is significantly less than the bond strength of 0.4 MPa specified in the code of practices (Eurocode 4, 1994). However, the Eurocode considered the filler material in the composite section is normal strength concrete. Higher strength concretes with much higher cement contents are different in material with a higher shrinkage possibility (Swamy and Anand, 1973)
which can cause inferior keying action at the steel-concrete interface. Recent pushout tests (Shakir-Khalil and Hassan, 1995) on concrete-filled RHS indicated bond strength reduced significantly (0.169 MPa) when high strength concrete was used.

5.5 SUMMARY

Concrete-filled RHS specimens were tested to examine the pushout behaviour between steel and concrete. Three types of inside steel surface configurations were adopted. No mechanical connectors were used. Concrete core was pushed through the steel shell of the RHS and the travel of the concrete core was recorded against the load level for all the specimens.

The results indicated that both face conditions and section configurations play an important role for the bond strength. It was found that for the greased specimens bond stresses are significantly lower than the specimens with other configurations. Results also showed that acid wash at the interface did not change the pushout behaviour significantly compared to the specimens with untouched surfaces. The reason for this can be the roughness of the inside steel surface which contributes significantly to the bond strength. Pushout tests also demonstrated that the specimens with smaller cross-sections had superior bond when compared to the specimens with larger cross-sections. This is mainly due to the greater interlocking effect at the corners of the cross-section.
CHAPTER 6

Analytical Study on The Flexural Behaviour of Concrete-Filled Steel Rectangular Hollow Sections

6.1 INTRODUCTION

The design formulae are proposed for concrete-filled steel tube sections in a number of codes of practices (Eurocode 4, 1994; ACI 318, 1989). In the analysis of ultimate moment capacity of the composite beam a full compatibility of strains between steel and concrete is assumed. This assumption was made after a number of experimental studies on columns and beam-columns (Knowles and Park, 1969; Neogi, Sen and Chapman, 1969; Rangan and Joyce, 1991; Prion and Boehme, 1994) indicate negligible
or no significance of interface bond to the overall performance of the composite section. Another advantage of assuming the full bond interaction between steel and concrete is the simplification of the analysis. However, this assumption can be questioned particularly in the analysis of flexural members where slippage may occur at the interface after cracking and/or crushing of concrete when no mechanical connectors are used.

In the ultimate strength calculation of the composite section, a number of studies (Furlong, 1967; Lu and Kennedy, 1994) assumed the maximum concrete stress as $1.0 f'_c$ in equivalent rectangular stress block instead of $0.85 f'_c$ as mentioned in the reinforced concrete theory considering the confinement effect of the steel shell. Most of the researchers and codes of practices assumed the steel section fully yielded at the ultimate moment for the simplification of the derivation although the steel portion close to the neutral axis position may remain elastic.

The design guidelines for effective flexural stiffness of the steel concrete composite section is provided in the Eurocode (Eurocode 4, 1994) which assumed the concrete portion uncracked. This contradicts with a number of studies (Lu and Kennedy, 1994; Rangan, 1991) which considered tensile concrete is ineffective in the derivation of the ultimate moment of resistance of the concrete-filled tubular sections in flexure.

In this chapter ultimate moment of resistance were derived for concrete-filled RHS flexural members by assuming different bond conditions at the interface. Mathematical expressions are also presented on the deflections under flexure. Deflections were
calculated at the load where steel starts to yield (serviceability limit state) and also at the failure load. The results of this analysis were finally compared with the experimental results (chapter 3 and 4).

6.2 THE ULTIMATE MOMENT OF RESISTANCE

In this section the analytical expressions for the ultimate moment of resistance of the concrete-filled RHS flexural members are presented. The analyses are provided in two series. In the first, full strain compatibility is assumed between steel and concrete whilst in the second series of analysis slippage is considered at the interface. The analyses were then compared to the experimental results presented in chapter 3 and 4.

6.2.1 Analysis based on strain compatibility at the interface

This analysis is based on consideration that the tubular steel section is either elasto-plastic or fully plastic at the time of failure. For analysing the strength of concrete the rectangular stress block concept according to the reinforced concrete design is adopted. Analysis is also given for concrete-filled RHS beam with a reinforcing Y bar in the tensile zone and when fibrous concrete is the filler to compare the test results of composite beams (chapter 3). The followings are the basic assumptions considered for the derivations. At the time of failure,
- Extreme fibre concrete strain in compression reaches a value of 0.003 ($\varepsilon_{cu}$) and because of the confinement due to steel shell the concrete strength $f'_c$ is taken as $1.0f'_c$ instead of $0.85f'_c$.

- For elastic-perfectly plastic consideration portions of structural steel in tension and in compression zone yielded far from neutral axis and the portions close to the neutral axis remained elastic.

- Concrete carries no strength in the tensile zone. Extra reinforcing bar in the tensile zone of the composite section yielded and steel fibres used for fibrous concrete contributes only in the tensile zone of the concrete.

- Throughout the loading history strain compatibility exists between the steel shell and the concrete core.

The last assumption is questionable because during loading procedure as concrete starts cracking some slip between steel and concrete may occur. However, as the slip deformations are heavily concentrated around cracked locations it is assumed that there is no loss in full composite action due to lack of shear transfer in friction and in bond because the portions between cracks can be assumed to deform together.

![Composite Section, Strain Diagram, Forces in the Concrete portion, Forces in the Steel portion]

Fig. 6.1 Strain and force distribution
The distribution of forces in steel and concrete with the strain diagram of the composite section with fibrous concrete and without the reinforcing bar is presented in Fig. 6.1.

The neutral axis position \( k_u \) and the ultimate moment of resistance \( M_u \) for different configurations of the composite section are presented in the following paragraphs.

**Composite section without reinforcing bar and without fibrous concrete**

From Fig. 6.1, the forces acting on the section and their corresponding locations from the neutral axis can be written as in the following,

\[
T_1 = C_1 = B f_{xy}
\]
\[
T_2 = 2 f_{xy} (d - k_u d - d_1)
\]
\[
T_3 = C_3 = f_{xy} d_i
\]
\[
T_4 = \frac{1}{2} \frac{f_y^2}{E_c} k_u db
\]
\[
C_2 = 2 f_{xy} (k_u d - d_i)
\]
\[
C_c = f_{xy} k_u db
\]

and

\[
z_1 = d - k_u d + \frac{t}{2}
\]
\[
z_2 = \frac{1}{2} (d - k_u d + \alpha k_u d)
\]
\[
z_3 = z_6 = \frac{2}{3} \alpha k_u d
\]
\[
z_4 = k_u d + \frac{t}{2}
\]
\[
z_5 = \frac{1}{2} (k_u d + \alpha k_u d)
\]
\[
z_c = k_u d - \frac{\gamma}{2} k_u d
\]
From the equation of equilibrium of forces,

$$\sum T = \sum C$$

In this case, (Fig. 6.1)

Thus neutral axis can be calculated as,

$$k_u = \frac{1}{2 + \frac{\gamma b f_y}{2 t f_y}}$$

Where $f_y$ is the yield stress of the tubular steel section; $f_c$ is the concrete core strength taken as $1.0 f'_c$; $\gamma$ is the reduction factor for the depth of the neutral axis and taken as 0.85 as in the reinforced concrete design. Others are as defined in Fig. 6.1.

The ultimate moment of resistance can be calculated from the summation of moments of all forces of the section to its neutral axis. It can be written as,

$$M_u = \sum T_i z_i + \sum C_i z_i$$

Now by substituting all in the above equation we have,

$$M_u = B t f_y (d + t) + t f_y d^2 \left[ 1 - 2 k_u + 2 \left( 1 - \frac{1}{3} \alpha^2 \right) k_u^2 \right]$$

$$+ f_c \gamma d^2 b \left( 1 - \frac{\gamma}{2} \right) k_u^2$$

The above expression is valid when the steel tubular section behaves elasto-plastically at the time of failure. For full plastic solution $2 \left( 1 - \frac{1}{3} \alpha^2 \right) k_u^2$ shall be substituted by $2 k_u^2$ in
the second expression. The value of $\alpha$ can be calculated from $\alpha = \frac{e_{sy}}{\varepsilon_{cu}} = \frac{E_s}{E_c}$ where $\varepsilon_{cu}$ is the extreme fibre compressive strain in concrete.

(ii) Composite section with a reinforcing bar in the tensile zone and without fibrous concrete

Under this configuration the calculated neutral axis position and ultimate moment of resistance of the section are as follows,

$$k_u = \frac{1 + \frac{A_s}{2} \frac{f_{sy1}}{f_{sy}}}{2 + \frac{\gamma b}{2} \frac{f_c}{f_{sy}}}$$

(6.3)

and

$$M_u = Btf_{sy}(d + t) + tf_{sy}d^2 \left[ 1 - 2k_u + 2 \left( 1 - \frac{1}{3} \alpha^2 \right) k_u^2 \right]$$

$$+ f_s \gamma d^2 b \left( 1 - \frac{\gamma}{2} \right) k_u^2 + A_s f_{sy1}(d_s - k_u d)$$

(6.4)

Where $f_{sy1}$ is the yield stress of the Y bar and $d_s$ is the position of the Y bar from the neutral axis.

(iii) Composite section with fibrous concrete and without reinforcing bar

The basic assumption for analysis of the composite section under the abovementioned configuration is that the steel fibres contribute in terms of strength to the overall
moment capacity only in the tensile zone of concrete. The behaviour of the fibre-steel concrete in the tensile zone is considered elastic with the maximum tensile stress defined by $f_f$. The value of $f_f$ is taken as $1.3 \left(0.6 \sqrt{f_c} \right)$ considering the increase in flexural strength as 30% by adding fibre-steel in concrete matrix. Based on the assumption of linear strain distribution, one can write,

$$\frac{\varepsilon_{cu}}{k_u d} = \frac{\varepsilon_f}{z_f}$$

where $\varepsilon_f = \frac{f_f}{E_c}$

The neutral axis position of the section can be written as,

$$k_u = \frac{1}{\frac{\gamma b}{2 t f_y} + \frac{1}{4 t f_y} \frac{1}{\varepsilon_{cu} E_c} - \frac{b f_f^2}{2 t f_y} - \frac{1}{2 t f_y} - \frac{b}{2} \frac{f_f}{E_c}}$$  \hspace{1cm} (6.5)$$

$E_c$ is the modulus of elasticity of concrete. The derivation for ultimate moment of resistance is as follows,

$$M_u = B t f_y (d + t) + tf_y d^2 \left[1 - 2k_u + 2 \left(1 - \frac{1}{3} \alpha^2\right) k_u^2 \right]$$

$$+ f_f \gamma d^2 b \left(1 - \frac{\gamma}{2}\right) k_u^2 + \frac{1}{3} b \frac{f_f^3}{(\varepsilon_{cu} E_c)^2} k_u^2 d^2$$  \hspace{1cm} (6.6)$$
(iv) Composite section with fibrous concrete and with a reinforcing bar in the tensile zone

The derivations for $k_u$ and $M_u$ for the composite section under abovementioned configuration are provided in the following expressions,

$$k_u = \frac{1 + \frac{1}{2} \frac{A_k}{dt} f_{sy1}}{2 + \frac{b}{2} \frac{f_c}{f_{sy}} - \frac{b}{4} \frac{f_{fy}^2}{f_{sy}^2} \frac{1}{\varepsilon_{ec} E_c} - \frac{1}{4} \frac{\gamma}{f_{sy}}}$$  \hspace{1cm} (6.7)

and

$$M_u = B f_{sy} (d + t) + tf_{sy} d^2 \left[ 1 - 2k_u + \left( 1 - \frac{1}{3} \alpha^2 \right) k_u^2 \right]$$

$$+ f_c \gamma d^2 b \left( 1 - \frac{\gamma}{2} \right) k_u^2 + \frac{b}{3} \left( \frac{f_{fy}}{E_c} \right)^2 k_u d^2 + A_s f_{sy1} (d - k_u d)$$  \hspace{1cm} (6.8)

6.2.2 Analysis by considering slippage at the interface

Flexural strength theories are derived in the following by considering slippage between the steel shell and the concrete core. Two strain diagrams (Fig. 6.2) are drawn for each steel and concrete portion which are assumed parallel to each other by a distance $e$ which is the slip strain at the interface. The basic assumptions for this analysis of flexural moment of resistance of concrete-filled RHS are as follows. At the time of failure,
- The steel portion yields fully.
- Extreme fibre concrete strain in compression reaches a value of 0.003 and due to the confinement of the steel shell the concrete strength $f'_c$ is taken as $1.0f'_c$ instead of $0.85f'_c$.
- Strain compatibility does not exist. The strain difference between steel and concrete at the interface remains identical throughout the section and is mentioned as slip strain (e). Concrete strain is assumed less in the compression zone.
- Concrete carries no strength in the tensile zone.

A constant slip strain assumption is contradictory in reality because the concrete strains in tension and in compression will be always less than the adjacent steel stress in magnitude. However, it can be considered because the concrete strain in the tensile zone cannot be accurately predicted. The following diagram illustrates concrete-filled RHS section with force and strain diagrams for the analysis of ultimate flexural moment.

![Diagram](image)

Composite Section  Strain Diagram  Forces on the Concrete Portion  Forces on the Steel Portion

Fig. 6.2: Force and strain diagram (slip consideration at the interface)
From the diagram, \( \frac{\varepsilon_{sc}}{k_c d} = \frac{\varepsilon_{cu}}{k_c d} \), where \( \varepsilon_{cu} \) and \( \varepsilon_{sc} \) are the extreme fibre compressive strain of concrete and the strain in the inside fibre of steel in the compression flange.

Again, \( \varepsilon_{sc} = \varepsilon_{cu} + e \)

From two abovementioned expressions we have,

\[
e k_c = 0.003(k_s - k_c) \quad (6.9)
\]

In the above expression the value of \( \varepsilon_{cu} \) at the ultimate load is assumed as 0.003.

Now from Fig. 6.2

\[
C_c = \gamma f_c' b k_c d \\
C_1 = B f_{sy} \\
C_2 = 2 t k_c d f_{sy} \\
T_1 = B f_{sy} \\
T_2 = 2 t (d - k_c d) f_{sy}
\]

**The Neutral Axis Position**

From the equation of the equilibrium, one can write

\[
\sum T = \sum C
\]

From the above expression we have,

\[
2 k_s + \frac{\gamma f_c'}{2 t f_{sy}} k_c = 1 \quad (6.10)
\]

Now when all values are known by solving out (6.9) and (6.10) \( k_s \) and \( k_c \) can be determined.

**The Ultimate Moment of Resistance**

The ultimate moment of resistance of the composite section can be derived from the following expression,
\[ M_u = T_1 z_1 + C_1 z_2 + T_2 z_3 + C_2 z_4 + C_3 z_c \]
\[ = B tf_{sy} (d + t) + tf_{sy} d^2 \left(1 - 2k_s + 2k_i^2\right) + \gamma \left(1 - \frac{\nu}{2}\right) f_c^b k_c^2 d^2 \]  
(6.11)

The neutral axis position for composite section with a reinforcing bar in the tensile zone is given by,
\[ 2k_s + \frac{\nu f_c^i}{2tf_{sy}} k_c = 1 + \frac{1}{A_x} \frac{f_{sy}}{dt} \]  
(6.12)

and the corresponding ultimate moment of resistance,
\[ M_u = B tf_{sy} (d + t) + tf_{sy} d^2 \left(1 - 2k_s + 2k_i^2\right) + \gamma \left(1 - \frac{\nu}{2}\right) f_c^b k_c^2 d^2 + A_x f_{sy} (d - k_c d) \]  
(6.13)

The analysis for the composite section when the filler is fibrous concrete is not presented in this section. This is due to the negligible contribution of the steel fibres on the overall moment capacity of the composite section. The slip strain value \(e\) is used to relate with the depth of neutral axes and the ultimate moment of resistance in Fig. 6.3 and Fig. 6.4.

![Graph](image)

**Fig. 6.3 : Slip strain versus Depth of N.A. (steel and concrete)**
Fig. 6.3 demonstrates that at zero slip (full bond) situation the neutral axes positions are the same, then as the slip strain increases the N.A. position for concrete decreased whilst the N.A. position for steel increased. So as the slip increases the more area of concrete is becoming ineffective and thus gradual decrease in the ultimate moment capacity is observed (Fig. 6.4). The cross-sectional geometry of the RHS is assumed as 100 mm width, 150 mm depth and 4 mm wall thickness with an yield stress of 460 MPa, the assumed concrete compressive strength is 87 MPa.

![Graph showing the relationship between slip strain and ultimate moment of resistance.](image)

**Fig. 6.4:** Slip strain versus ultimate moment of resistance

In Fig. 6.4 values of moments are plotted in the close range so that the nonlinearity in the ultimate flexural moment of resistance is visible with the increase of the slip strain. Results indicate that although not drastic, noticeable loss of moment capacity has occurred as the slip increases.
6.2.3 Comparison with the test results

The analysis results are compared with the test results of experimental studies on concrete-filled RHS flexural members and beam-column connections. The connection specimens were chosen for the comparison where the beam portion failed during the experiments. Studies (Lu and Kennedy, 1994) suggested for accurate prediction of ultimate moment capacity of the concrete-filled RHS beams, the stress in steel should be considered corresponding to the strain values obtained during the experiments. The strain values of the flexural member tests (Chapter 3) exceeded 10,000 microstrain which is well beyond yielding of the steel shell. In this situation a more realistic value of ultimate stress (although upper bound) of steel may replace the yield stress to achieve more accurate prediction. In the tensile coupon tests, the yield stress and the ultimate stress of the steel RHS were obtained as 434 MPa and 460 MPa respectively. The concrete compressive strength for concrete-filled RHS beams and connections were 94.5 MPa and 87.0 MPa respectively. A slip strain of 1000 microstrain between steel and concrete was assumed in the prediction of ultimate moment which consider slip at the interface. This slip strain value was taken from the connection test specimen CN4 (Chapter 4) where the beam portion failed. A comparison between the results of the experiments and the analyses is presented in Table 6.1 where twelve test specimens of concrete-filled RHS beams and two connection specimens were compared.
### Table 6.1 Comparison between analyses and test results

<table>
<thead>
<tr>
<th>Test specimens</th>
<th>$M_{u \text{ test}}$ kNm</th>
<th>$M_{u \text{ analysis 1}}$ kNm</th>
<th>$M_{u \text{ analysis 2}}$ kNm</th>
<th>$\frac{M_{u \text{ test}}}{M_{u \text{ analysis 1}}}$</th>
<th>$\frac{M_{u \text{ test}}}{M_{u \text{ analysis 2}}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>RH1</td>
<td>60.8</td>
<td>55.4</td>
<td>52.77</td>
<td>1.09</td>
<td>1.15</td>
</tr>
<tr>
<td>RH2</td>
<td>57.1</td>
<td>55.4</td>
<td>52.77</td>
<td>1.03</td>
<td>1.08</td>
</tr>
<tr>
<td>RH3</td>
<td>62.2</td>
<td>55.4</td>
<td>52.77</td>
<td>1.12</td>
<td>1.18</td>
</tr>
<tr>
<td>RHR1</td>
<td>61.4</td>
<td>61.6</td>
<td>58.81</td>
<td>0.99</td>
<td>1.04</td>
</tr>
<tr>
<td>RHR2</td>
<td>62.0</td>
<td>61.6</td>
<td>58.81</td>
<td>1.01</td>
<td>1.05</td>
</tr>
<tr>
<td>RHR3</td>
<td>62.7</td>
<td>61.6</td>
<td>58.81</td>
<td>1.02</td>
<td>1.07</td>
</tr>
<tr>
<td>FRH1</td>
<td>61.0</td>
<td>55.4</td>
<td>52.77</td>
<td>1.10</td>
<td>1.15</td>
</tr>
<tr>
<td>FRH2</td>
<td>60.4</td>
<td>55.4</td>
<td>52.77</td>
<td>1.09</td>
<td>1.14</td>
</tr>
<tr>
<td>FRH3</td>
<td>59.3</td>
<td>55.4</td>
<td>52.77</td>
<td>1.07</td>
<td>1.12</td>
</tr>
<tr>
<td>RFRH1</td>
<td>63.9</td>
<td>61.6</td>
<td>58.81</td>
<td>1.04</td>
<td>1.09</td>
</tr>
<tr>
<td>RFRH2</td>
<td>66.2</td>
<td>61.6</td>
<td>58.81</td>
<td>1.07</td>
<td>1.13</td>
</tr>
<tr>
<td>RFRH3</td>
<td>63.5</td>
<td>61.6</td>
<td>58.81</td>
<td>1.03</td>
<td>1.08</td>
</tr>
<tr>
<td>CN3 *</td>
<td>51.56</td>
<td>53.1</td>
<td>51.25</td>
<td>0.97</td>
<td>1.01</td>
</tr>
<tr>
<td>CN4 *</td>
<td>50.13</td>
<td>53.1</td>
<td>51.25</td>
<td>0.94</td>
<td>0.98</td>
</tr>
</tbody>
</table>

* The rigid knee connections which failed due to flexural member failure. The strain difference at the interface in the compressive zone of the beam was recorded as 0.001.

The comparison shows a good agreement is achieved between test results and analysis 1 which assumed full bond at the steel-concrete interface. This is because of the tests on flexural members were done under a central point load at the mid-span and lies on the axis of symmetry where no significant slip can be expected. When compared with analysis 2 which assumed a slip strain of 0.001 between steel and concrete, shows more conservative results when compared to the results of analysis 1. However, an excellent agreement was reached between the results of the connection specimens and the analysis 2 which is due to the accurate use of slip strain value obtained from the experiments. Results demonstrated that there was no increase in ultimate moment capacity of the
concrete-filled RHS beams when fibrous concrete was used. This is due to negligible contribution of the tensile concrete on the overall sectional strength.

In this section the analysis results are also compared with the test results of other studies (Lu and Kennedy, 1994) on concrete-filled RHS flexural members. The analysis which assumes full bond at the steel-concrete interface is only used to compare the experimental results (Lu and Kennedy, 1994) due to the nonavailability of interface slip-strain values. Results were compared against the available yield stress of steel instead of ultimate stress. Details of test specimens (Lu and Kennedy, 1994) are given in the Table 6.2.

<table>
<thead>
<tr>
<th>Specimen nos.</th>
<th>Rectangular steel tubular section details</th>
<th>Yield stress of steel $f_{sy}$ (MPa)</th>
<th>Strength of concrete $f'_c$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Width (mm)</td>
<td>Depth (mm)</td>
<td>Thickness (mm)</td>
</tr>
<tr>
<td>CB12</td>
<td>152</td>
<td>152</td>
<td>4.8</td>
</tr>
<tr>
<td>CB13</td>
<td>152</td>
<td>152</td>
<td>4.8</td>
</tr>
<tr>
<td>CB15</td>
<td>152</td>
<td>152</td>
<td>4.8</td>
</tr>
<tr>
<td>CB22</td>
<td>152</td>
<td>152</td>
<td>9.5</td>
</tr>
<tr>
<td>CB31</td>
<td>152</td>
<td>254</td>
<td>6.4</td>
</tr>
<tr>
<td>CB33</td>
<td>152</td>
<td>254</td>
<td>6.4</td>
</tr>
<tr>
<td>CB35</td>
<td>152</td>
<td>254</td>
<td>6.4</td>
</tr>
<tr>
<td>CB41</td>
<td>152</td>
<td>254</td>
<td>9.5</td>
</tr>
<tr>
<td>CB45</td>
<td>152</td>
<td>254</td>
<td>9.5</td>
</tr>
<tr>
<td>CB52</td>
<td>254</td>
<td>152</td>
<td>6.4</td>
</tr>
<tr>
<td>CB53</td>
<td>254</td>
<td>152</td>
<td>6.4</td>
</tr>
<tr>
<td>CB55</td>
<td>254</td>
<td>152</td>
<td>6.4</td>
</tr>
</tbody>
</table>

The concrete in the Lu and Kennedy tests is of normal strength concrete of around 45 MPa compressive strength. Different cross-section geometry of the RHS were adopted.
In this section the test results (chapter 3) are compared with the analysis 1. This time the yield strength of steel is considered instead of ultimate strength to compare with the test/analysis results of Lu and Kennedy tests. The comparisons are presented in Table 6.3 and Table 6.4.

**Table 6.3 Comparison with test results (Lu and Kennedy, 1994)**

<table>
<thead>
<tr>
<th>Specimen Nos.</th>
<th>Ultimate Moment (Test) (kNm)</th>
<th>Ultimate Moment analysis (kNm)</th>
<th>Test /analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td>CB12</td>
<td>73.6</td>
<td>68.7</td>
<td>1.07</td>
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<tr>
<td>CB13</td>
<td>75.1</td>
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</tr>
<tr>
<td>CB15</td>
<td>71.3</td>
<td>68.1</td>
<td>1.05</td>
</tr>
<tr>
<td>CB22</td>
<td>146.5</td>
<td>131.5</td>
<td>1.11</td>
</tr>
<tr>
<td>CB31</td>
<td>210.7</td>
<td>185.2</td>
<td>1.14</td>
</tr>
<tr>
<td>CB33</td>
<td>210.7</td>
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<tr>
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<td>142.9</td>
<td>123.4</td>
<td>1.16</td>
</tr>
</tbody>
</table>

**Table 6.4 Comparison with test results of this study (chapter 3)**

<table>
<thead>
<tr>
<th>Specimen Nos.</th>
<th>Ultimate Moment Test kNm</th>
<th>Ultimate Moment analysis kNm</th>
<th>Test / analysis</th>
</tr>
</thead>
<tbody>
<tr>
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<td>1.17</td>
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<td>1.20</td>
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<td>61.4</td>
<td>57.9</td>
<td>1.06</td>
</tr>
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<td>RHR2</td>
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<td>57.9</td>
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</tr>
<tr>
<td>RHR3</td>
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<td>RFRH2</td>
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<tr>
<td>RFRH3</td>
<td>63.5</td>
<td>57.9</td>
<td>1.09</td>
</tr>
</tbody>
</table>
The Test/analysis values of Table 6.3 and Table 6.4 showed the test results of Lu and Kennedy compares well with the test results of this study (chapter 3). Test/analysis ratios also indicate that the prediction is always lower than the experiments. The reason for this is due to the assumption of yield stress as the steel strength. This assumption is conservative because in reality the extreme fibre stresses in steel may exceed yield stress at the time of failure.

6.3 DEFLECTIONS AT THE LIMIT OF SERVICEABILITY

In this section, analytical studies are presented on the deflections of concrete-filled RHS beam at the serviceability limit. This serviceability limit can be defined as the limit when the extreme fibre steel in the tensile zone is just at the yield point. In the concrete-filled RHS composite beams steel portion in the tensile zone yields earlier than the steel in the compressive zone due to the concrete contribution in the compressive zone.

Derivations are presented in two series based on two assumptions. In the first series it was assumed that strain compatibility exists at the steel-concrete interface whilst in the second series slippage was considered between the steel shell and the concrete core. The analysis presented here is based on the modular ratio theory of the reinforced concrete. In the derivation, first the location of neutral axis (axes for second series) was calculated and then moment of inertia of the cracked section was derived from the transformed section. Finally, the analysis was compared to the experimental results.
6.3.1 Analysis based on strain compatibility at the interface

![Diagram of a composite section, strain diagram, and forces on concrete and steel portions.]

Fig. 6.5: Force and strain diagram

This analysis is based on the modular ratio theory of reinforced concrete.

\[ f_{se} = E_s \varepsilon_{se}, \]
\[ f_{sc} = E_s \varepsilon_{sc}, \]
\[ f_c = E_c \varepsilon_c, \]

where \( \varepsilon_{se}, \varepsilon_{sc} \) and \( \varepsilon_c \) are the extreme fibre strains in tensile steel, in compressive steel and in concrete respectively.

Now considering the strain compatibility,

\[ \varepsilon_{sc} = \frac{k_y d + t}{d - k_y d + t} \varepsilon_{se} \quad \text{and} \quad \varepsilon_c = \frac{k_y d}{d - k_y d + t} \varepsilon_{se} \]

Now the forces acting on the composite section,
\[ C_i = tBf_{wc} = tBE_s \left( \frac{k_y d + t}{d - k_y d + t} \right) \varepsilon_{st} \]

\[ C_2 = 2t \frac{1}{2} f_{wc} k_y d = tf_{wc} k_y d = tE_s \left( \frac{k_y d + t}{d - k_y d + t} \right) \varepsilon_{st} k_d \]

\[ T_i = tBE_s \varepsilon_{st} \]

\[ T_2 = 2t \frac{1}{2} f_{s} (d - k_y d) = tE_s \varepsilon_{st} (d - k_y d) \]

\[ C_c = \frac{1}{2} f_{c} k_y db = \frac{1}{2} E_c \left( \frac{k_y d}{d - k_y d + t} \right) \varepsilon_{st} k_y db \]

Now from the equation of equilibrium,

\[ \sum T = \sum C \quad \text{we have,} \]

\[ \frac{1}{2} \frac{E_c}{E_s} \frac{b}{t} dk_y^2 + 2(B + d + t) k_y - (B + d + t) = 0 \quad (6.14) \]

From the above expression the position of the neutral axis \((k_y)\) can be calculated.

After the neutral axis position is known the transformed section for concrete can be calculated from the compressive concrete area by the modular ratio, \(m\) (Fig. 6.6). The moment of inertia of the section can be calculated against the neutral axis. The transformed section is presented in Fig. 6.6.

Fig. 6.6 : Transformed section
In this section effort has been made to calculate $M_y$ which is moment capacity of the composite section when the extreme fibre tensile steel is just at yield i.e. $\varepsilon_y = \text{yield strain}$. $M_y$ can be derived from forces in the composite section as,

$$M_y = \frac{tE_s e_y d^2}{d - k_y d + t} \left[ \frac{1}{3} E_s b dk_y - \frac{2}{3} \left( B + \frac{2}{3} t + d \right) k_y^2 - 2 \left( B + \frac{2}{3} t + d \right) k_y + B \left( 1 + \frac{t}{d} \right)^2 \right]$$

(6.15)

### 6.3.2 Analysis by considering slippage at the interface

![Diagram showing composite section, strain diagram, forces on concrete portion, forces on steel portion.]

**Fig. 6.7** : Strain and Force distribution

From modular ratio theory and Fig. 6.7,

$$\frac{\varepsilon_c}{k_c d} = \frac{\varepsilon_s + e}{k_s d}$$

(6.16)

From the above expression we have,

$$\varepsilon_c = \frac{k_c d}{d - k_s d + t} \varepsilon_s \quad \text{and} \quad \varepsilon_s = \frac{k_s d + t}{d - k_s d + t} \varepsilon_s$$

Now again from the force equilibrium in the composite section,
\[ \frac{1}{2} \frac{E_s}{E_c} \frac{b}{t} d k_c^2 + 2 \left( B + \frac{d}{3} + t \right) k_s - \left( B + \frac{d}{3} + t \right) = 0 \quad (6.17) \]

Now by substituting (6.16) and (6.17) we can have the neutral axes positions for steel \((k_s)\) and concrete \((k_c)\).

From these neutral axis positions one can analyse the moment of inertia of the transformed section where the moment of areas of steel and concrete are to be calculated against their respective neutral axis positions.

\[ M_y \] for this analysis can be derived as,

\[ M_y = \frac{t E_s e_o d^2}{d - k_c d + t} \left[ \frac{1}{3} \frac{E_s}{E_c} \frac{b}{t} d k_c^3 + 2 \left( B + \frac{2}{3} t + d \right) k_s^2 - 2 \left( B + \frac{2}{3} t + d \right) k_s + B \left( 1 + \frac{t}{d} \right)^2 \right] \quad (6.18) \]

### 6.3.3 Comparison with test results

The analysis of deflections at the serviceable limit of concrete-filled RHS flexural members are compared with an experimental study already presented in chapter 3. Results were compared at the load level of 40 kN which was applied at the mid span of the composite beam of span 2750 mm. It can be noticed from the test results that after 40 kN load level tensile steel started to deform permanently. The test results of six beam specimens were compared with analysis of both series and is presented in Table 6.5. In Table 6.5 analysis 1 represents the analysis with the assumption of strain compatible interface whilst in analysis 2 slip was assumed between steel and concrete.
Table 6.5 Comparison between analyses and test results

<table>
<thead>
<tr>
<th>Test specimens</th>
<th>$\Delta u_{Axial}$</th>
<th>$\Delta u_{analysis 1}$</th>
<th>$\Delta u_{analysis 2}$</th>
<th>$\frac{\Delta u_{test}}{\Delta u_{analysis 1}}$</th>
<th>$\frac{\Delta u_{test}}{\Delta u_{analysis 2}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>RH1</td>
<td>16.5</td>
<td>11.22</td>
<td>13.65</td>
<td>1.47</td>
<td>1.21</td>
</tr>
<tr>
<td>RH2</td>
<td>16.65</td>
<td>11.22</td>
<td>13.65</td>
<td>1.48</td>
<td>1.22</td>
</tr>
<tr>
<td>RH3</td>
<td>16.5</td>
<td>11.22</td>
<td>13.65</td>
<td>1.47</td>
<td>1.21</td>
</tr>
<tr>
<td>FRH1</td>
<td>15.84</td>
<td>11.22</td>
<td>13.65</td>
<td>1.41</td>
<td>1.16</td>
</tr>
<tr>
<td>FRH2</td>
<td>15.8</td>
<td>11.22</td>
<td>13.65</td>
<td>1.41</td>
<td>1.16</td>
</tr>
<tr>
<td>FRH3</td>
<td>15.65</td>
<td>11.22</td>
<td>13.65</td>
<td>1.39</td>
<td>1.15</td>
</tr>
</tbody>
</table>

FRH1, FRH2 and FRH3 are the beam specimens of steel RHS filled with fibrous concrete. It was noticed during the experiments that steel fibres did not play significant role in the overall behaviour of the composite section which is due to the negligible contribution of the tensile concrete in the overall sectional strength. Results demonstrated prediction of deflections using analysis 1 gives conservative results. This is possibly due to the consideration of full bond at the interface. It was observed (chapter 4) that slip between steel and concrete took place early in the loading period. Deflection results using analysis 2 is always lower than the experimental results although it agrees reasonably well in comparison to analysis 1. This discrepancy is due to the assumption of slip strain of 1000 microstrain in analysis 2 which is recorded in the connection specimen CN4 (chapter 4). For the composite beam slip strain may exceed 1000 microstrain.
6.4 DEFLECTION AT THE ULTIMATE LOAD

In this section efforts have been made to calculate the deflection at the point of loading of the concrete-filled RHS flexural member at the ultimate load where the load was applied at the mid span. The basic assumption of this analysis is the consideration of plastic hinge in the vicinity of the loading point. A detailed diagram regarding the concept of the analysis is presented in the following figure (Fig. 5).

![Diagram of Beam Element, Bending Moment Diagram, and Idealised Curvature Diagram]

Fig. 6.8: Deflection at the ultimate load

In Fig. 5 $M_u$ is the ultimate moment of resistance of the composite section and externally represents as $P_u \frac{L}{4}$. $M_y$ is the moment capacity of the composite section at
which extreme fibre tensile steel started to yield. $M_y$ for particular composite section can be calculated from the derivation presented in the previous section. In this analysis both assumptions of bond conditions at the interface which was described in the section 6.3 was applied to compare the experimental results.

The plastic hinge of rotation at the point of loading can be expressed as,

$$\theta_p = (\varphi_u - \varphi_y)l_p,$$

or, $$\theta_p = \left( \frac{\varepsilon_u}{d - k_u d} - \frac{\varepsilon_y}{d - k_y d} \right)l_p$$ \hspace{1cm} (6.19)

where $\varepsilon_u$ is the extreme fibre strain in steel in the event of ultimate moment and $\varepsilon_y$ is the strain corresponding to yield stress in steel. $k_u$ and $k_y$ are the neutral axis positions from the top of the compressive concrete fibre corresponding to $M_u$ and $M_y$ respectively. The deflection can be calculated by multiplying the half area of the curvature diagram with the distance of the c.g. of the same area from the end support.

6.4.1 Comparison with test results

In this section analytical results are compared with the experimental results of concrete-filled RHS beams (chapter 3). Results of midspan deflections at the ultimate load of six test specimens were compared with two analysis results. Comparisons are presented in Table 6.3. In Table 6.3 analysis1 represents the full bond assumption at the steel concrete interface whilst analysis 2 considered slip between steel and concrete.
Table 6.6  Comparison between analyses and test results

<table>
<thead>
<tr>
<th>Test specimens</th>
<th>$\Delta u_{\text{test}}$</th>
<th>$\Delta u_{\text{analysis 1}}$</th>
<th>$\Delta u_{\text{analysis 2}}$</th>
<th>$\frac{\Delta u_{\text{test}}}{\Delta u_{\text{analysis 1}}}$</th>
<th>$\frac{\Delta u_{\text{test}}}{\Delta u_{\text{analysis 2}}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>RH1</td>
<td>104.93</td>
<td>115.14</td>
<td>122.17</td>
<td>0.91</td>
<td>0.86</td>
</tr>
<tr>
<td>RH2</td>
<td>110.45</td>
<td>115.14</td>
<td>122.17</td>
<td>0.96</td>
<td>0.9</td>
</tr>
<tr>
<td>RH3</td>
<td>110.43</td>
<td>115.14</td>
<td>122.17</td>
<td>0.96</td>
<td>0.9</td>
</tr>
<tr>
<td>FRH1</td>
<td>130.8</td>
<td>115.14</td>
<td>122.17</td>
<td>1.14</td>
<td>1.07</td>
</tr>
<tr>
<td>FRH2</td>
<td>85.61</td>
<td>115.14</td>
<td>122.17</td>
<td>0.74</td>
<td>0.7</td>
</tr>
<tr>
<td>FRH3</td>
<td>132.98</td>
<td>115.14</td>
<td>122.17</td>
<td>1.15</td>
<td>1.09</td>
</tr>
</tbody>
</table>

Results showed both analyses agrees well with the experimental results. Specimens with fibrous concrete generally reached the ultimate load at larger deflection values when compared to the beam specimens without the steel fibres. Only in beam FRH2 the deflection was significantly less at the ultimate load.

6.6  SUMMARY

Analytical studies were carried out on ultimate strength and deflection at the mid span of concrete-filled RHS flexural members. Simple mathematical expressions were derived so that it can be coded into a spreadsheet program or in a programmable calculator. In the analyses, at the steel-concrete interface two different bond conditions was assumed. In the first, full strain compatibility between steel and concrete was considered whilst the other assumption was partial bond with a strain difference between steel and concrete. This strain difference was named as slip strain. The results of the analyses were compared with the experimental results of chapter 3 and 4.
Comparison shows the slippage at the interface did not influence the ultimate strength significantly and the analysis which assumes full bond between steel and concrete agrees well with the experimental results. However, in the prediction of deflections the analysis with the slippage assumption provided a better comparison with the test results. This indicated the slip at the interface may influence the stiffness of the composite flexural member.
CHAPTER 7

Finite Element Analysis on The Flexural Members of Steel RHS Filled with High Strength Concrete

7.1 INTRODUCTION

Most of the studies on concrete-filled RHS flexural members were conducted as a part of the investigation on composite beam-columns. In the derivation of ultimate moment of these composite sections Eurocode (Eurocode 4, 1994) suggests strain compatibility exists at the steel-concrete interface. This assumption can be questioned because the overall performance of the flexural member constructed from concrete-filled steel tubular sections may be influenced by bond between steel and concrete when no mechanical connectors are used.
The ultimate strength results of a finite element study (Ge and Usami, 1994) on concrete-filled thin walled steel box columns showed good agreement with the experimental results. In this study both steel and concrete materials were assumed as elastic-perfectly plastic. The tensile and compressive residual stresses in the steel section were assumed as equal to the yield stress and 30 percent of the yield stress respectively. In a finite element study on concrete-filled RHS beam-columns (Shakir-Khalil and Al-Rawadan, 1995) two dimensional thin shell and three dimensional brick elements were adopted to model the steel and the concrete respectively. Full bond was assumed at the interface.

In recent experimental studies (Lu and Kennedy, 1994; Kilpatrick and Rangan, 1995) on concrete-filled RHS flexural members efforts were made to investigate the effect of slippage at the steel-concrete interface. These studies indicated minimal significance of slippage to the overall behaviour of the composite section. However, it is difficult to measure slippage with accuracy during the experiments. Also, the slippage in the composite beam may vary in different locations due to cracking and crushing of concrete. This interface behaviour between steel and concrete may be better understood by finite element studies. In this chapter efforts were made to model the steel-concrete interface by using nonlinear spring elements between steel and concrete.

The objective of this study is to examine the overall flexural behaviour of RHS filled with high strength concrete with an emphasise on the behaviour of bond between the steel shell and the concrete core.
In this chapter the results of a finite element study are presented on beams of steel RHS filled with high strength concrete. Models were developed with different bond stress between steel and concrete at the interface for parametric studies. The results were then compared with an experimental study.

7.2 MODELLING

7.2.1 Geometrical Modelling

A proprietary finite element analysis package ANSYS (ANSYS 5.1, 1995) was used to carry out the non-linear finite element analysis of beams of RHS filled with high strength concrete. Due to symmetrical configuration of the composite beam, only one fourth model was developed and analysed by using appropriate boundary conditions. The boundary conditions for the one fourth model of concrete-filled RHS flexural member is presented in Table 7.1. The steel shell of the RHS was modelled by a large-strain-plastic shell (Shell 43) element. The shell 43 is well suited to model nonlinear, flat or warped, thin to moderately thick shell structures. The element may have variable thickness. The element has plasticity, creep, stress stiffening, large deflection and large strain capabilities. The concrete core was meshed by three dimensional solid concrete (solid 65) element. The element has plasticity, creep, swelling, stress stiffening, large deflection and large strain capabilities. In addition, the solid 65 element has special cracking and crushing capabilities. One fourth model of the beam is presented in the Fig. 7.1.
Fig. 7.1 One fourth model of RHS filled with high strength concrete

Table 7.1 Boundary conditions (Fig. 7.1)

<table>
<thead>
<tr>
<th>Line FB</th>
<th>Steel</th>
<th>$F_x$, $\theta_y$, $\theta_z$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Line GC</td>
<td>Steel</td>
<td>$F_x$, $\theta_y$, $\theta_z$</td>
</tr>
<tr>
<td>Line HG</td>
<td>Steel</td>
<td>$F_x$, $F_y$</td>
</tr>
<tr>
<td>Line AB, AD, DC</td>
<td>Steel</td>
<td>$F_x$, $\theta_x$, $\theta_y$, $\theta_z$</td>
</tr>
<tr>
<td>Plane FGCB</td>
<td>Concrete</td>
<td>$F_x$, $\theta_y$, $\theta_z$</td>
</tr>
<tr>
<td>Plane ABCD</td>
<td>Concrete</td>
<td>$F_z$, $\theta_x$, $\theta_y$, $\theta_z$</td>
</tr>
</tbody>
</table>
The model presented in Fig. 7.1 is in the global coordinate system where x, y and z axes are parallel to the lines HG, EH and GC respectively. Symbols used in the Table 7.1 are in terms of $F$ and $\theta$ which are translation and rotation respectively. For example, $F_x$ and $\theta_x$ represent restriction of translation and rotation respectively against the x-axis.

### 7.2.2 Material Modelling

Material properties used for steel and concrete in the finite element analysis were taken from the results of material testing as given in chapter 3. The stress-strain curve for the steel RHS used in this model was determined by the idealization from tensile coupon tests. An elastic-perfectly plastic model was used which is shown in Fig. 7.2.

![Stress-strain curve for steel](image)

**Fig. 7.2** Stress strain curve for steel
The compressive strength of high strength concrete was considered as 94.5 MPa which is the mean compressive strength obtained from cylinder tests. Tensile strength of concrete was assumed as 4.0 MPa which is also based on test results. The stress-strain curve for concrete material provided by the ANSYS is a linear response and has limitations. However, for the analysis of high strength concrete these limitations do not have significant effect due to sharp descending curve after peak stress (Fig. 7.3).

![Stress-strain curve for high strength concrete](image)

**Fig. 7.3** Stress strain curve for high strength concrete

For concrete a poisson's ratio of 0.2 was assumed which is applicable to higher strength concretes (High Strength Concrete, 1992) and the modulus of elasticity was calculated from the equation of best-fit for high strength concrete which is

\[ E_c = 3.38 \rho^{2.5} \left( f_{cm} \right)^{0.325} \times 10^{-5} \text{ MPa} \] (High Strength Concrete, 1992).
7.2.3 Bond Modelling

A full bond is often assumed in the design of concrete-filled steel tubular sections for the simplification of the analysis. In this analysis, to model the interface a nonlinear spring (Combin 39) was used. These elements were connected node to node between the steel and the concrete. The length of the nonlinear spring elements are 2 mm which is the half of the wall thickness of the steel RHS. This is to ensure that the steel shell and the concrete core are just in contact. Combin 39 is a unidirectional element with nonlinear generalised force-deflection capability (ANSYS user manual, 1995). This nonlinear spring (Combin 39) element has three degrees of freedom at each node, these are translations in the nodal x, y and z directions. No bending and torsion is considered. The element has large displacement capability. Combin 39 element allows for a multilinear element stiffness. These stiffnesses can be generated by force-displacement curve which is the input for nonlinear spring elements. This force-deflection input was obtained from load-slip curves from pushout tests on concrete-filled steel tubes. Combin 39 elements are shown in the Fig. 7.4.

Maximum bond strength in the nonlinear spring was assumed as 0.2 MPa which is a more realistic value derived from previous tests on RHS filled with high strength concrete (Shakir-Khalil and Hassan, 1995) and also from pushout test results of this study presented in Chapter 5. This bond strength value of 0.2 MPa was multiplied by the effective area of each node at the interface to obtain the force value. The force and slip of the concrete core were used as y and x coordinate input respectively in the combin 39 element. It is also assumed that once the peak stress is reached the concrete core travel
through the steel shell with no increase in stress and the load slip curve become parallel to the horizontal axis.

Fig. 7.4 Nonlinear elements

The initial bond stiffness was determined from the results of the pushout tests (Chapter 5). This bond model is applied throughout the contact surface between steel and concrete. This approach, used for the simplification of the modelling is questionable due to the fact that the bond may be different in the compressive zone compared to the tensile zone of a flexural member. Bond modelling at the interface in two shearing directions is assumed as mentioned above. The stiffness of the nonlinear spring in the normal direction was calculated from the stiffness of high strength concrete which is the weaker of the two materials (steel and concrete) in the composite section. The stiffness
was derived from the crushing strength and shortening values of high strength concrete. In one of the two other beam models, the bond strength at the interface was considered as 0.8 MPa to study the comparison. No nonlinear spring elements were used in the other composite beam model where full bond was assumed between steel and concrete.

7.3 FINITE ELEMENT ANALYSIS

Due to the symmetrical configuration of the concrete-filled RHS beam, a quarter model (Fig. 7.1) was analysed by incorporating necessary boundary conditions (Table 7.1). Load was applied on the top flange of the composite beam as a line load on the line AB (Fig. 7.1). Care was taken for the convergence of the analysis due to the complex nature of both geometry and material nonlinearity. Due to the limited linear response of concrete material (Fig. 7.3) provided by the ANSYS load was applied very slowly by using small load steps to avoid divergence in the analysis.

The flexural member has a cross-sectional dimension of 100 mm width by 150 mm depth with steel wall thickness of 4.0 mm. The span of the beam was 2750 mm. A reasonably fine mesh of 25 mm was adopted for modelling the steel shell and the concrete core. For the concrete core and the steel shell separate volume and surface were created, respectively and these two were then connected by nonlinear spring (Combin 39) elements node to node. For full bond analysis, volume was first generated for the concrete core and it was meshed by solid concrete (Solid 65) element. The outer surfaces of the volume were meshed by using steel shell (Shell 43) element to ensure the
steel and the concrete are bonded together. However, this full bond analysis produced errors which were found later in the concrete stress results due to a much stiffer interface.

The load was applied in 16 load steps. Each load step had a flexibility of up to 400 substeps to avoid divergence. The analysis was terminated at the last load step due to cracking and crushing of concrete through the concrete section which is expected due to the instability of the stiffness matrices after the concrete component is cracked and crushed fully through the section.

7.4 RESULTS AND DISCUSSION

In this section results of concrete-filled RHS beams are presented as graphs and coloured contours. Results of deflections, strains and slippages at the interface are plotted against the load levels. Analysis results were also compared with the results obtained in the experiments given in chapter 3. The load versus deflection response of the composite beams under different interface bonds are plotted in the Fig. 7.5. The load-deflection curves are not completed due to the divergence of the analysis. Divergence occurred when the concrete component was cracked and/or crushed through the full section. From the results it can be noticed that with the increase of bond stress the curves become stiffer. The reason for this is the increased stiffness at the interface which led to the superior overall stiffness of the beam. Results also indicated that the more the bond the more likely the early divergence in the analysis.
Fig. 7.5 Load deflection response of composite beams with different bond

Fig. 7.6 Top and bottom flange strains

In Fig. 7.6 top and bottom flange strains for beams with 0.2 MPa and 0.8 MPa bond stress are plotted. Large amount of strain values were obtained in the analysis of beams
with 0.2 MPa interface bond strength whilst in the beam with higher bond the strain values are comparatively lower due to earlier divergence of the analysis. Results showed an increase of stiffness in the strain results of the compressive top flange for beam with higher bond but in the tensile flange the stiffnesses are almost identical. The reason for this can be the cracking of concrete in the tensile zone which starts very early during the loading period. The load slip curves are presented in Fig. 7.7 for beams with 0.2 MPa and 0.8 MPa interface bond stress. Slip results are recorded at the interface between steel and concrete in the bottom flange and at the end support of the composite beam. The difference in bond behaviour in two beams is significant. For beam with a higher bond a linear response was obtained before the analysis diverges whilst for a more realistic 0.2 MPa bond, a nonlinear behaviour is noticed.

![Graph: Load versus slip at the interface](image)

**Fig. 7.7 Load versus slip at the interface**

Analysis also carried out for a RHS beam without any filling. For the hollow beam also, one fourth model was developed by applying necessary boundary conditions. The steel
shell was meshed by Shell 43 element. The results of the finite element study were compared with the experimental study presented in chapter 3. The load deflection response of the comparison is presented in Fig. 7.8. The bond stress in the concrete-filled beam model was 0.2 MPa. For the hollow section, finite element results agrees well with the experiment results until the peak load is reached. After the peak load the experimental result showed a sharp loss of stiffness and ductility which is due to the local crushing of the compressive top flange. In comparison, the analysis indicates the beam is stiffer than that observed in the experiments. For the concrete-filled section the analysis indicates a higher initial stiffness in the beginning. Then due to debonding at the interface and cracking and crushing of concrete, beam becomes significantly less stiffer than the beam in the experiment before the analysis diverges. This inaccuracy is due to the limitations in linear stress-strain relationship (Fig. 7.3) for the concrete material and the load slip curve assumed for the bond model.

Fig. 7.8 Comparison between FEA and experiments (Load deflection response)
In the experiments significant interlocking between steel and concrete exists after debonding occurs due to the non-uniform strain across the section of the flexural member which is different from the bond model considered for the analysis. In the analysis it was assumed that once debonding occurs between steel and concrete, bond strength remains the same. In Fig. 7.9 steel strain values of top and bottom flanges are compared with the experiments. Large amount of plastic strains can be noticed in both flanges in the analysis results. Results showed that after steel yielding load strain graph is stiffer in the compressive top flange for the experimental results which is the effect of better interlocking between steel and concrete than the analysis bond model. In the tensile flange strain gauge peeled off early during the experiment after achieving a plastic strain of 6000 microstrain.

![Graph of Load vs Strain](image)

**Fig. 7.9** Comparison between analysis and test results (top and bottom flange strains)

By examining the stress distribution, it reveals that failure occurred due to heavy steel yielding combined with concrete cracking. Areas of steel yielding were plotted in Fig.
in terms of plastic strain contours. Results showed heavy steel yielding in the flanges and web near the loading point. In the compressive (top) flange large amount of plastic compressive strain (10 000 microstrain) occurred which is due to the presence of localised compressive stress generated by the applied load. In the tensile (bottom) flange maximum plastic strain occurred at the point of loading and strain as high as 14 000 microstrain was observed.

Finite element results indicated concrete cracking in the tensile zone of the concrete-filled RHS beam. As expected cracks are appeared to be larger near the loading point. An enlarged view of the concrete cracking near the area of loading is presented in the Fig. 7.11.

![Figure 7.10](image_url)

Fig. 7.10 Plastic strain distribution (horizontal direction) on the steel flange
7.5 SUMMARY

Results are presented of a finite element study into the flexural behaviour of RHS filled with high strength concrete. The emphasis was given on the bond between the steel shell and the concrete core. Both full bond and partial bond models were analysed. Finite element results were compared with an experimental study of concrete-filled RHS beams.

Comparison with experimental results demonstrate that a good agreement occurs between finite element analysis and test results for both hollow and concrete-filled RHS sections. For the hollow section, the finite element model overestimates the post-peak load behaviour. For the concrete-filled section, the analysis presents a stiffer model initially whilst in the post-failure range experimental results indicate a superior overall performance. The reason for this is due to the interface bond model which assumes once debonding occurs between steel and concrete slippage occurs without any increase in bond strength. But in reality in a composite flexural member, bond strength increases as the slippage increases. In another discrepancy, the bond model used in the finite element
study was applied to the entire interface but actually the bond behaviour is different in the compressive zone to the tensile zone of the composite beam.

Finite element results indicated that although the full bond model can analyse the overall behaviour to the certain degree of accuracy, it gives inaccurate results for the concrete stress distribution. However, the model with nonlinear spring element incorporating the load slip behaviour at the interface is more successful to predict the overall behaviour of the composite beam and also the stress distribution for concrete. The nonlinear analysis experiences divergence problem when the analysis reaches the stage of concrete cracking and crushing through the section of the concrete component.
CHAPTER 8

Finite Element Analysis on The Beam-Column Connections of RHS Filled with High Strength Concrete

8.1 INTRODUCTION

Previous research on concrete-filled steel tubular columns (Knowles and Park, 1969), beam-columns (Furlong, 1967; Neogi et al, 1969; Rangan and Joyce, 1992) and beams (Lu and Kennedy, 1994; Prion and Boehme, 1994) demonstrated excellent ductility and post-failure behaviour. This has led researchers to think a rigid beam-column connection which can be particularly useful in the earthquake prone areas. Research has been conducted experimentally (Shakir-Khalil, 1993) and also by finite element analysis (Kawaguchi and Morino, 1995) on the behaviour of simple shear connections between
steel I-beam and concrete-filled steel tubular columns. In the finite element study (Kawaguchi and Morino, 1995) the concrete-filled RHS was modelled by two separate elements for steel and concrete and a full bond interaction was assumed at the interface. It was concluded that the composite column failed in flexure with heavy steel yielding.

In the rigid connections between composite beams and composite columns one important factor which can influence the overall performance is the shear transfer mechanism between steel and concrete. This shear transfer depends heavily on bond between the steel shell and the concrete core when no shear connectors are used.

In this chapter finite-element studies are presented for beam-column connections where both beam and column portions were constructed from RHS filled with high strength concrete. Models of concrete-filled RHS connections were developed in two different geometrical configurations following the test specimens beam-column connections mentioned in chapter 4. In this finite element study emphasis was given on the interface bond and the nonlinear spring elements were used to connect the steel shell and the concrete core. Finally the analysis was compared to an experimental study (chapter 4) to validate the modelling.
8.2 MODELLING

8.2.1 Geometrical Modelling

Finite element analysis package ANSYS (ANSYS 5.1 User Manual, 1995) was used to conduct this analytical study on connections of steel RHS filled with high strength concrete. Two geometrical configurations were adopted for the beam-column connection. In the first model the beam portion was placed on top of column portion forming a rigid knee connection (Fig. 8.1) whilst in the second the cantilever beam was connected to the side of the column portion (Fig. 8.2).

![Fig. 8.1 Rigid knee connection (Half model)](image1.png) ![Fig. 8.2 Beam-column connection (Half model)](image2.png)

Due to the symmetrical configurations only half of the models were analysed by adopting necessary boundary conditions. The boundary conditions of the half models are presented in Table 8.1. The steel shell of the composite section was modelled by a nonlinear plastic shell (shell 43) element whilst a nonlinear solid (solid 65) element was
adopted to model the concrete core. The solid 65 element has the cracking and crushing capabilities of concrete. Both shell 43 and solid 65 elements have the large deformation capabilities. More detail on elements shell 43 and solid 65 are given in chapter 7.

**Table 8.1  Boundary conditions (Fig. 8.1 and Fig. 8.2)**

<table>
<thead>
<tr>
<th>Line AB, DC, JK, ML</th>
<th>Steel</th>
<th>( F_z, \theta_z, \theta_y )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Line EH, FG, NQ, PR</td>
<td>Steel</td>
<td>( F_z, \theta_z, \theta_y )</td>
</tr>
<tr>
<td>Plane ABCD, JKLM</td>
<td>Concrete</td>
<td>( F_z, \theta_z, \theta_y )</td>
</tr>
<tr>
<td>Plane EFGH, NQPR</td>
<td>Concrete</td>
<td>( F_z, \theta_z, \theta_y )</td>
</tr>
</tbody>
</table>

The symbols used in Table 8.1 are similar to that discussed in Chapter-7. It should be noted that the points for steel and concrete are not coincident (Fig. 8.1 and Fig. 8.2). They are connected by nonlinear spring (combin 39) elements of length 2 mm which is half of the wall thickness of the steel RHS. However, for the simplification of showing the boundary conditions this has been adopted.

### 8.2.2 Material Modelling

The stress-strain curve for steel RHS was obtained from tensile coupon tests. The stress-strain curve for finite element analysis was then idealised as an elastic-perfectly plastic model which is shown in Fig. 7.2. The compressive strength of concrete was the mean compressive strength which was determined by cylinder tests during experiments (chapter 4). The mean compressive strength for high strength concrete was 87.0 MPa
and 85.0 MPa respectively for rigid knee connection and beam-to-column connection. The tensile strength of concrete was 4.0 MPa which was obtained from tensile tests on concrete. The stress-strain curve provided by the ANSYS was used which is a limited linear response. However, due to the sharp descending path for high strength concrete after the peak load is reached, the error obtained from this limitation is minimal. The typical stress-strain curve of high strength concrete with the curve provided by the ANSYS is shown in Fig. 7.3. For concrete material a poisson's ratio of 0.2 is considered which is applicable for higher strength concretes (High strength concrete, 1992) and the modulus of elasticity was derived from \( E_c = 3.38 \rho^{2.5} (f_{cm})^{0.325} \times 10^{-5} \) MPa which is the best equation of the best fit for high strength concrete (High strength concrete, 1992).

### 8.2.3 Bond Modelling

In the design of steel-concrete composite structures a full bond at the interface is mostly assumed (Eurocode 4, 1994; ACI 318, 1989) to simplify the analysis although the concrete-filled steel tube members under large bending may experience slippage between steel and concrete. In this study of concrete-filled RHS connections the steel-concrete interface was modelled by a nonlinear spring (combin 39) element. These spring elements were connected between steel and concrete by node to node. The detail of nonlinear spring (combin 39) element is presented in Chapter 7.

The maximum bond strength in the nonlinear spring elements was assumed as 0.2 MPa. This value was obtained from previous studies (Shakir-Khalil and Hassan, 1995) on
RHS filled with high strength concrete. In the bond stress versus slip curve it was assumed that once the peak stress is achieved the bond is lost permanently with no further increase in stress. The bond stress versus slip curve was then considered parallel to the horizontal axis. For the simplification of the analysis this bond characteristic was applied in throughout the interface although some error can occur in this approach due to the different bond conditions in tensile and compressive zone of the members under bending. The bond stresses in the transverse directions are assumed as mentioned above whilst in the normal direction the stiffness of concrete was adopted in the nonlinear spring. More informations on the stiffnesses assumed in the nonlinear spring are given in Chapter 7. In this study one analysis with full bond condition at the interface was conducted for rigid knee connection model to compare with other results.

8.3 FINITE ELEMENT ANALYSIS

In this finite element study models of concrete-filled RHS beam-column connections in two different configurations were analysed. In the first model which is a rigid knee connection, beam was placed on top of the short column while the beam remains cantilever. The length of the short column is 300 mm whilst the total length of the beam is 500 mm with a cantilever portion of 300 mm (Fig. 8.1). In the other model cantilever composite beams were connected to the side of the short composite columns. In this model the length of the beam is 300 mm and the length of the column 500 mm with an unrestrained length of 300 mm from the base to the bottom flange of the beam (Fig.
8.2). Both the models were developed according to the specimens tested in the experiments (Chapter 4).

The cross-sectional dimension of the RHS is 100 mm width by 150 mm depth with a wall thickness of 4.0 mm. A fine mesh of 25 mm was used to model the steel shell and the concrete core. Separate area and volume were generated for steel shell and the concrete core respectively. The steel and concrete were then connected node to node by nonlinear spring element. For full bond analysis of the rigid knee connection, the concrete volume was first generated and meshed by solid concrete (solid 65) element. The outer surfaces were then meshed by steel shell (shell 43) element to achieve a full bond condition at the interface.

Due to the symmetrical configuration of the concrete-filled RHS beam-column connections only the half model was analysed by incorporating necessary boundary conditions (Table 8.1). The load was applied as a line load at a distance 25 mm from the free end of the cantilever beams which is similar to the loading conditions during the experiments. Due to the complicated nature of both geometry and material nonlinearity care was taken on the convergence of the analysis.

The load was applied very slowly by using 9 equal load steps which is due to the limited linear stress-strain response of the concrete material. Every load step has a flexibility of 400 substeps upto first 4 load steps and 500 substeps thereafter. The analyses were terminated at the beginning of seventh load step due to cracking and crushing through the section of the concrete component. This is expected due to the instability in the
stiffness matrices after the concrete component crushed and/or cracked through the full section.

8.4 RESULTS AND DISCUSSION

In this section the results of finite element analysis on concrete-filled RHS connections are presented. Results of connection joint rotations and interface slippage were plotted against the moment at the connection. Efforts were also made to compare the analysis results with the results obtained during experiments (Chapter 4).

![Graph](image)

Fig. 8.3 : Moment versus rotation

In Fig. 8.3 rotations at the composite connections of different models are plotted against the applied moment. All the analysis diverges earlier than the failure moment achieved in the experiments which is due to the limitations in the concrete material modelling.
adopted in the ANSYS. Rigid knee connection failed due to heavy cracking and crushing of concrete in the beam portion whilst the beam-column connection failed in the column portion by cracking and crushing of the concrete component. Results showed the moment-rotation response in the rigid knee connection model with full bond at the interface is almost linear which is due to early divergence of the analysis due to concrete component failure in the beam portion. The moment rotation curve of full bond model also showed a much higher stiffness than the models with interface elements which is due to the stiffening up the interface by fully bonding steel and concrete elements together. It was found in the analysis that this full bond model produced some errors in the concrete stress results due to the stiffening of concrete elements at the interface.

![Graph showing moment versus slip at the end of the beam](image)

**Fig. 8.4 : Moment versus interface slip at the end of the beam**

Fig. 8.4 demonstrates the moment versus slip at the interface for the models with nonlinear spring elements between steel and concrete. For both the models slippage
were measured at the free end of cantilever beams. Results showed the rigid knee connection generates more slippage at the interface than the beam-column specimen. This is due to the more flexible beam portion in the rigid knee connection. In the analysis rigid knee connection beam portion failed first which is due to larger length of the beam (500 mm) against a length of 300 mm short column. Whilst in the beam-column connection column portion (500 mm in length) failed earlier than the smaller (300 mm) beam portion. The moment versus slip response in the beam-column connection is nearly linear whilst in the rigid knee connection significant nonlinear response is noticed.

![Graph showing moment versus rotation](image)

**Fig. 8.5 : Comparison with test results**

In this study effort was taken to compare the analysis results with the results of an experimental study. The moment rotation curves of the rigid knee and beam-column composite connections are presented in Fig. 8.5. The results of the finite element
analysis showed the early divergence of the analysis is due to concrete cracking and/or crushing through the full section of the concrete component.

Fig. 8.6 Stress distribution (along the length of the beam and the column) in the steel portion of the rigid knee connection

Fig. 8.7 Stress distribution (along the length of the beam and the column) in the steel portion of the beam-column connection

The stress distributions in the steel portion of the rigid knee and beam-column connections are presented in Fig. 8.6 and Fig. 8.7 respectively. The beam and column portions are presented separately due to the different values of contours obtained in different directions. In the contours dark red and dark blue colours indicate extreme tensile and compressive areas respectively. From the diagrams it can be noticed that the maximum stresses are generated around the connection region which is expected because of the occurrence of maximum moment around the connection.
8.5 SUMMARY

Results are presented of a finite element study on the rigid beam-column connections where both beam and column portions are from steel RHS filled with high strength concrete. The study emphasizes the interface bond between the steel shell and the concrete core. Beam-column connections in two different geometrical configurations were analysed. In the first one, the beam was placed on top of the column portion forming a rigid knee whilst in the other, the beam was connected to the side of the short column. Both full bond and partial bond models were analysed. Finally, the results of the finite element study were compared with the experimental results.

Analysis diverges earlier than the peak moment achieved in the experiments. The reason behind this is the failure of the concrete component through the full section. Experimental result agrees well with the finite element results although the analysis model for rigid knee connection indicate a stiffer model. This is due to the limitations in the bond model which assumes no increase in bond strength occur once the peak stress is achieved. But in a composite the bond strength increases as the slippage increases. Finite element results showed the rigid knee connection failed in the beam portion whilst the beam-column connection failed at the short column due to heavy concrete cracking and/or crushing.

It was found that the full bond model gives erroneous results in determining the concrete stresses although it can predict the overall behaviour with certain degree of accuracy. However, it can be concluded that the model with nonlinear spring elements at the
interface which incorporates the load slip behaviour from pushout tests is more successful in predicting the overall behaviour as well as the concrete stress condition.
CHAPTER 9

Finite Element Analysis on The Pushout Behaviour of the Steel RHS Filled with High Strength Concrete

9.1 INTRODUCTION

One of the concerns with concrete-filled steel tubular sections is the influence of bond between the inside of the steel tube and the concrete core when the composite member is progressively loaded to failure. A number of experimental studies (Virdi and Dowling, 1980; Shakir-Khalil, 1991; Hunaiti, 1991) were carried out to investigate the interface bond between steel and concrete by conducting pushout tests on concrete-filled steel tubes. Recommendations for design bond strength for concrete-filled steel tubes are also given in codes of practices (Eurocode 4, 1994).
Study (Virdi and Dowling, 1980) showed the bond strength of the concrete-filled steel tube depends primarily on the roughness of the inside surface of steel and also on the size and the shape of the tube cross-section. Investigations (Shakir-Khalil, 1991) seemed to indicate that the wall thickness of the steel tube may influence the bond strength by increasing the stiffness of the steel shell. Recent study (Shakir-Khalil and Hassan, 1995) on the pushout resistance tests where the infill was high strength concrete indicate a significant decrease in bond strength when compared with the bond strength value assumed in the codes of practices (Eurocode 4, 1994).

For further investigations of this complex interface behaviour finite element studies were conducted on the pushout resistance of RHS filled with high strength concrete and the results are presented in this chapter. Effort was made to explain the bond behaviour at the interface and also to compare the results with the experimental study presented in chapter 5. A number of parameters which can influence the interface bond such as the size and the shape of the RHS cross-section, wall thickness of the RHS and the compressive strength of concrete were also investigated.

### 9.2 MODELLING

#### 9.2.1 Geometrical Modelling

A finite element software ANSYS (ANSYS 5.1 User manual, 1995) was used to model and analyse the pushout behaviour of steel RHS filled with high strength concrete. The circumferential steel portion was meshed by 2-D plastic shell (Shell 43) elements whilst
for the concrete core 3-D solid (Solid 65) elements were adopted. Solid 65 element with cracking and crushing capabilities was used specially for concrete. A reasonably fine mesh of 25 mm was adopted to model the composite section. Both the elements, Shell 43 and Solid 65 have nonlinear and large deformation capabilities.

9.2.2 Material Modelling

The material properties assumed for the steel RHS were determined from the tensile coupon tests during the experiments. An elastic-perfectly plastic stress-strain curve for steel was adopted for the analysis which was obtained by the idealisation from the tensile coupon results. The details of the stress-strain curve is presented in Fig. 7.2. The mean compressive strength and the tensile strength of concrete were considered as 87.0 Mpa and 4.0 Mpa respectively which was determined from material tests on high strength concrete. The modulus of elasticity of concrete was obtained from 

\[ E_c = 3.38 \rho^{2.5} (f'_{cm})^{0.325} \times 10^{-5} \text{ Mpa} \] (High strength concrete, 1992) and a value of 0.2 was assumed for the poisson’s ratio of high strength concrete (High strength concrete, 1992). The idealised stress-strain curve for concrete used in the analysis is similar to that presented in Fig. 7.3.

9.2.3 Bond Modelling

The full contact area of steel-concrete interface was modelled by nonlinear spring (Combin 39) elements which were connected by node to node between the steel shell
and the concrete core. Two types of bond models were used in the analyses. Both models were obtained from load versus slip curve of the pushout tests (chapter 5). The plots of bond stress versus concrete travel were idealised for bond models and presented in Fig. 9.2 for two cross-section geometries of RHS. For concrete-filled 75×75×2.5 RHS model the bond properties assumed from pushout test results of specimen SST2 (Chapter 5) whilst for concrete-filled 100×150×4.0 RHS model test results of specimen ST1 was adopted. Although the bond strength achieved for the specimen ST1 during the experiments was significantly lower than the results of previous pushout tests (Shakir-Khalil and Hassan, 1995) on RHS filled with high strength concrete of bond strength 0.146 to 0.196 MPa, it was adopted for the comparative study.

![Graph showing bond stress vs. concrete travel](image)

**Fig. 9.2** Bond models for concrete-filled RHS pushout models

The bond strength in the pushout direction and the other shearing direction was assumed as mentioned in Fig. 9.2. In the normal direction the spring stiffness of Combin 39 was considered as the stiffness of high strength concrete which has been mentioned in
Chapter 7. The assumption of this bond model is questionable due to the reason that the load-slip curve obtained from tests may not be the result of one way motion only in the pushout direction. Some confinement pressure from the steel shell is also expected. However, for the simplification and due to the fact that once the bond is released, the concrete core may travel in one direction despite some frictional resistance at the interface.

9.3 FINITE ELEMENT ANALYSIS

The length of the concrete-filled RHS pushout models were 450 mm including interface contact length of 400 mm. 50 mm gap was kept at the bottom of the steel shell to enhance the concrete core travel. Load was applied as uniform pressure load on top of the concrete to push the concrete core through the steel shell. The steel shell was restrained against translation in all three (X,Y and Z) directions at the base. Maximum pressure load was considered as 20% more than the peak load assumed in the bond model. Load was applied slowly in five load steps to avoid divergence due to complex nonlinearities present in geometry and materials. 20 substeps were used in every load steps to enhance convergence of the analysis. The concrete-filled 75 × 75 × 2.5 RHS model is presented in Fig. 9.2.
Fig. 9.2 The full model and the sectional view of concrete filled $75 \times 75 \times 2.5$ RHS

9.4 RESULTS AND DISCUSSION

Results of the finite element analysis on the pushout behaviour of RHS filled with high strength concrete are presented in this section. The stress distribution contours at the peak load in the direction of the pushout load (y-direction) for the steel $75 \times 75 \times 2.5$ RHS is presented in Fig. 9.3.
Results at the peak load showed a steady increase in the level of stress values from the top to the base of the model. The stress on the steel shell is due to the reactions at the base which was gradually balanced at every node by forces in the interface elements generated from the pushout load. It can be noticed that the maximum stress value obtained at the peak load is only 42 MPa which indicates the steel is in the early elastic stage.
The section through the concrete-filled RHS model is presented in Fig. 9.4 which shows the stress distribution contours in the concrete. The stress distribution pattern in concrete portion is reverse to that of the steel shell. In concrete core stress values are gradually decreased from top to bottom. The stress in the concrete core occurred due to the pushout load which was balanced at the interface by the reactions from non-linear spring elements.
The stress distribution in steel and concrete are plotted against the y-direction of the specimen at the peak load and presented in Fig. 9.5. The stress results indicated a decrease in stress level at the support node in the base compared to stress values at the two previous nodes. This is due to the localised effect of the unrestrained steel shell of 50 mm at the base.

![Graph showing stress distribution in steel and concrete](image)

**Fig. 9.5 Stress distribution in steel and concrete**

The strain distribution in steel and concrete portion is plotted against the y-coordinate at the peak load in the Fig. 9.6. The results indicated strain increased proportionally with stress. The localised effect on the steel shell in the unfilled portion of 50 mm in the base is also visible. Strain values showed the steel material is still in the linear elastic range. It was also noticed that concrete was not cracked and/or crushed after examining the stress values in all the directions.
Fig. 9.6 Strain distribution in steel and concrete

Fig. 9.7 Axial deformations on the steel shell and the concrete core

The relative deformations in the steel shell and in the concrete core in each nodes are plotted against the y-coordinate of the concrete-filled RHS model in Fig.9.7. The
deformations occurred due to the pushout load on the concrete core and thus causing axial shortening on the steel shell which is supported at the bottom. The concrete core was supported at the interface by nonlinear spring elements as it traveled through the RHS. In this study the results of the finite element analysis were compared with the experiment results (Chapter 5). Comparison is presented for two models of RHS filled with high strength concrete. The analysis results of concrete-filled $75 \times 75 \times 2.5$ RHS and concrete-filled $100 \times 150 \times 4.0$ RHS models were compared with the experimental results of the test specimens SST2 and ST1 respectively. The comparison is presented in Fig. 9.8. The load-slip results showed the analysis results agreed well with the experiments. This indicates when the concrete is only loaded the motion of the concrete core is almost one way downward. The analysis model of concrete-filled $75 \times 75 \times 2.5$ RHS shows a slightly stiffer curve in the post-failure range which is due to the simplification of bond model used in the analysis.

![Comparison between FEA and experiments](image)

**Fig. 9.8** Comparison between FEA and experiments
9.5 PARAMETRIC STUDIES

9.5.1 General

Previous experimental studies (Virdi and Dowling, 1980; Shakir-Khalil, 1991) indicated that the bond strength in concrete-filled steel tubes may depend on different parameters. These are the cross-sectional size and shape of the steel tube, the steel wall thickness and properties of concrete. However, it is often difficult to determine the influence of those parameters to the interface bond only by experiments. The reason for this is the different level of roughness present in the inside surface of steel tubes which may result different interface bond in identical test specimens. In this situation, to investigate the effect of abovementioned parameters on the bond between steel and concrete, finite element studies may be more appropriate than experimental studies.

In this chapter efforts were made to investigate different parameters of the concrete-filled RHS pushout model which can influence the interface bond. The parameters studied in this section are the cross-sectional shape (b/D ratio) and the wall-thickness of the RHS, and the compressive strength of concrete.

9.5.2 Wall thickness of the RHS

In this study three wall thicknesses, 2.5 mm, 4.0 mm and 5.0 mm of 75 × 75 × 2.5 steel RHS were investigated for pushout resistance of the concrete core through the steel shell. All other parameters, such as concrete strength, interface spring element
properties and model dimensions were kept identical. Same uniform pressure as pushout load was applied on top of the concrete core for all three models. A comparison of results of the concrete travel at the peak load is shown in Fig. 9.9. Results of the slip of the concrete core were plotted in a close range (3.7 mm to 4.5 mm) to visualize the differences. Y-coordinate was plotted in the horizontal axis. Results showed the slip of concrete against the steel is higher when the wall thickness of the RHS is thinner. The explanation for this can be the stiffer steel shell which generated superior bond in the RHS with thicker steel shell.

![Graph showing travel of concrete for models with different thicknesses of steel RHS](image)

**Fig. 9.9** Travel of concrete for models with different thicknesses of steel RHS

In Fig. 9.10 stress distributions in the steel portion for three different shell thicknesses are plotted against the y-coordinate of the model. The plot result indicate larger stresses occurred in the thinner sections with a much higher stress level in the model with 2.5 mm steel shell thickness when compared to other two model results.
9.5.3 The size of the RHS cross-section

Experimental studies (Virdi and Dowling, 1980; Shakir-Khalil, 1991) concluded that the size and shape of the cross-section of the steel tube have a major significance on the bond strength of the concrete-filled steel tube sections. In this series of finite element study three types of cross-sections of the RHS were adopted, 75 mm square, 75 mm by 100 mm rectangular and 75 mm by 125 mm rectangular. The RHS wall thickness of all concrete-filled model were adopted as 2.5 mm. All other parameters remained identical.

The axial deformations are shown in Fig. 9.11 for all three configurations at the peak load. From the results it can be noticed that in the larger sections more axial shortening occurred when compared to the smaller sections. The reason for this is the effect of
interlocking between steel and concrete at the corners which is more effective in the smaller cross-sections.

Fig. 9.11 Deformations on the steel shell

Fig. 9.12 Travel of the concrete core
The values of concrete slip were plotted against the y-coordinate which is at the horizontal axis (Fig. 9.12). Results once again indicated better performances of the squarer steel section models when compared to the rectangular models which is possibly due to superior bond stiffness at the interface provided by the narrower sections with better confinement at the corners.

![Graph showing stress distributions in the steel portion](image)

Fig. 9.13 Stress distributions in the steel portion

In this study stress distributions in steel and concrete portions are also examined for different cross-sections of RHS (Fig. 9.13). Analysis results indicated significant increase in stress level for the rectangular sections in the steel portion. The reason for this is the travel of the concrete core, which is less in the smaller section when compared to larger sections (Fig. 9.12). However, in the concrete core, the differences in the stress level is not significant which is due to small amount of stresses generated in concrete portion.
Fig. 9.14 Stress distributions in concrete

9.5.4 Compressive strength of concrete

Three different compressive strengths of 40, 70 and 100 MPa were considered for this series of pushout study on concrete-filled RHS. All other parameters including the dimensions, material properties etc kept identical. This comparative study did not indicate any significant differences in the results of the models. The reason for this is the very small amount of stress occurred in the concrete core under pushout load. However, strain distribution in the concrete for the models are presented in Fig. 9.15 which shows the strain differences are larger between 40 and 70 MPa concretes when compared between 70 and 100 MPa concrete.
Fig. 9.15 Concrete strain distribution

9.6 SUMMARY

Results of a finite element study about the pushout resistance of RHS filled with high strength concrete is presented. Finite element results were compared with the results of an experimental study of pushout tests on concrete-filled RHS. Models were also analysed to examine the effect of different parameters on the bond strength of concrete-filled RHS. These parameters were the shape of the cross-section and the wall thickness of the RHS, and the compressive strength of concrete.

Comparison between this finite element study and the experiments demonstrate a good agreement has been achieved although a questionable approach was undertaken to
model the steel-concrete interface. The analysis results show the stress levels of steel and concrete is low at the peak load and does not reach the nonlinear stage.

The parametric studies indicated that superior bond at the interface can be achieved when smaller hollow sections are used when compared to larger sections which is due to the better confinement at the corners of the steel shell. Also, the thicker wall of RHS shows an improved bond stiffness between steel and concrete which is due to the increased stiffness of the steel shell. Studies with the different compressive strength of concrete did not indicate any significant change in the bond behaviour. The reason for this is the limitations of the concrete material properties used in the analysis which did not include the effect of shrinkage.
CHAPTER 10

Conclusions

10.1 GENERAL

This thesis is mainly concerned with the study of the flexural members and the beam-column connections of steel RHS filled with high strength concrete. Emphasis is given on ductility and post-failure strength reserve and also on the interface bond between steel and concrete.

For the design of the composite section current codes of practices (Eurocode 4, 1994; ACI 318, 1989) assumed a full bond interaction between steel and concrete for the simplification of the analysis. But in the flexural members with large bending moment and in the connections with large rotations slippage may occur at the interface. In this study simple mathematical expressions were derived for the design of the composite
section assuming both full and partial bond at the interface. For the consideration of bond strength at the interface in the concrete-filled steel tube sections, Eurocode (Eurocode 4, 1994) assumed a value of 0.4 MPa which is based on the filler material as normal strength concrete. Previous research (Shakir-Khalil and Hassan, 1995) indicate the bond strength became almost half of the value when high strength concrete of about 90 MPa compressive strength is used. In this study a bond strength of 0.2 MPa is assumed at the steel-concrete interface to analyse the flexural member and the connection of steel RHS filled with high strength concrete by finite element modelling.

There was concern while using the higher strength concretes as the filler material which is due to the brittle behaviour of the members of high strength concrete (Attard and Mendis, 1993). The experimental results in this study indicate under the confinement of steel shell, the overall performance of the composite section is ductile, and with significant post-failure strength reserve.

In this thesis, study is presented experimentally and analytically on flexural and connection behaviour of RHS filled with high strength concrete. To model the bond at the interface pushout tests were also conducted. Experimental results are compared analytically by finite element modelling. Simple design equations were derived for the ultimate moment capacity of the composite section. General conclusions drawn from this study is presented in the following sections.
10.2 EXPERIMENTAL STUDIES

10.2.1 Concrete-filled RHS beams

In this scope of study, flexural members of steel RHS filled with high strength concrete with different configurations were tested to failure. The detail of this experimental study is presented in chapter 3.

Study indicated that the addition of steel fibres to the high strength concrete did not affect the overall behaviour of the composite beams. The Y16 bar which was provided in the tensile zone of the composite section did improve the ultimate strength marginally.

Test results showed concrete-filled flexural members have much superior overall performance than hollow flexural members in terms of strength, ductility and initial stiffness. Hollow members failed by elastic local buckling of the compression flange with dramatic loss of ductility whilst in the concrete-filled beams extreme fibres of steel, both in compression and in tension yielded heavily.

10.2.2 Concrete-filled RHS beam-column connections

Beam-column connections where both beam and column portions are from steel RHS filled with high strength concrete were tested to failure in this scope of experimental
study which has been described in chapter 4. Connections in different configurations, which include geometry and inside surface of steel were tested.

Results of this study showed that all the concrete-filled RHS connections failed in a ductile manner with large rotational capacity. Even in the grease surfaced specimens significant amount of ductility and post-failure strength reserve were observed. Results indicated the greased connections have a much lower initial stiffness than other specimens although there was no significant difference in the ultimate strength. Much larger relative slip between the steel shell and the concrete core was observed in the greased specimens when compared to the normal surfaced specimens. Incompatibility in strains between steel and concrete was noticed throughout the loading period.

Some of the connection specimens in the experiments failed due to inadequate weld. So, it can be concluded that a proper design of the connection weld to transfer the load is needed to achieve a reliable and ductile composite connection.

10.2.3 Bond strength of the concrete-filled RHS

In this experimental study the pushout tests were conducted on concrete-filled RHS specimens with different inside steel surface configurations to examine the bond behaviour at the interface. This study in detail is presented in chapter 5.

Experimental results showed the specimens with greased surface has significantly lower bond strength than the specimens with other configurations. This is due to almost no
bond situation between steel and concrete at the first place. No significant difference was observed in the pushout behaviour between the specimens with acidwashed and untreated interface. Study also demonstrated that specimens with smaller cross-sections have superior bond strength when compared to the specimens with larger cross-sections.

Pushout tests showed for high strength concrete a significantly lower bond strength was obtained when compared to the bond in the composite sections with normal strength concrete (Eurocode 4, 1994).

10.3 ANALYTICAL STUDIES

10.3.1 Analysis of the ultimate moment of resistance and deflections of concrete-filled RHS beams

Mathematical formulations were presented for ultimate moment and deflections for concrete-filled RHS flexural members in this analytical study and is presented in chapter 6. Analysis was performed considering full bond and partial bond at the steel-concrete interface and compared with the experimental results.

Results showed the slippage at the interface influence the ultimate moment of resistance of the composite section marginally. Analysis assuming full bond at the interface agrees well with the experimental results when compared to the analysis considering partial bond between steel and concrete for the derivation of ultimate moment capacity.
However, the analysis which assumes partial bond agrees more favourably with the experimental results in the comparison of deflections.

10.3.2Finite element analysis

10.3.2.1Concrete-filled RHS beams

Finite element models were developed and analysed using the software package ANSYS for flexural members of steel RHS filled with high strength concrete. Both full bond and partial bond models were analysed. Analysis was then compared with the experimental study of concrete-filled RHS beams. This finite element study is described in chapter 7.

Results indicate a good agreement occurred between the finite element study and the test results. Initially the analysis presents a stiffer beam but in the post-failure stage experimental results indicate a superior overall performance. The reason for this is the bond model used in the analysis at the interface to connect the steel shell and the concrete core. The bond model assumes after debonding at the interface the slippage between steel and concrete occurs without any increase in bond strength. But in the reality, bond strength may increase as slippage increases in a composite beam. Moreover, the bond model used in this analysis was applied to the whole interface of the composite flexural member for the simplification of the analysis but actually the bond is different in the compressive zone to the tensile zone in the composite beam. The divergence problem occurred in the analysis when the concrete component was cracked and crushed through the full section.
10.3.2.2 Beam-column connections

Chapter 8 presents finite element study on beam-column connections where both beam and column components are from RHS filled with high strength concrete. Comparison between analysis and test results showed a good agreement has been reached although divergence occurred in the analysis because of instability in the stiffness matrices of concrete due to cracking and crushing through the full section of the concrete component.

It was observed that models with nonlinear spring elements which incorporates the load-slip behaviour at the interface are more successful in predicting the overall behaviour when compared to the model with full bond assumption between steel and concrete.

10.3.2.3 Pushout behaviour of concrete-filled RHS

Finite element models were developed to compare an experimental study on pushout resistance of RHS filled with high strength concrete and presented in chapter 9. Models were also analysed to examine different parameters such as, the cross-sectional size and wall-thickness of the RHS, and the compressive strength of concrete.

Finite element results agreed well with the experiments. The study indicated when the pushout force is only on the concrete core the confinement pressure from the steel shell on concrete is negligible and the travel of concrete core is almost a one way motion along the pushout direction.
The study showed that superior bond behaviour can be achieved by using the smaller hollow sections when compared to the larger sections which is due to the better confinement at the corners of the RHS. Study also indicated that bond behaviour can be improved by adopting thicker wall of the steel RHS which is due to the increasing stiffness of the steel. Models with different compressive strength of concrete does not indicate any significant change in bond. This is due to the limitations in the concrete material used in the analysis which does not include shrinkage effect of higher strength concretes.
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162


CONNECTION AND FLEXURAL BEHAVIOUR OF
STEEL RHS FILLED WITH HIGH STRENGTH
CONCRETE

by

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Thesis Presented for the Degree of Doctor of Philosophy

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UNIVERSITY OF WESTERN SYDNEY, HAWKESBURY

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**PLEASE NOTE**

The greatest amount of care has been taken while scanning this thesis,

and the best possible result has been obtained.
To

My Parents

For their contributions in my life
Synopsis

Steel hollow section members filled with concrete have been frequently used in recent construction industry as columns and beam-columns because of their superior performance and constructability. Previous research demonstrated that such system has large energy absorption capacity which is critical in the event of an earthquake. By filling steel RHS with concrete, the failure of the steel shell due to local buckling can be delayed and the ductility of the concrete core can be improved as a result of the confinement of the steel shell. This type of composite section may be used in various structures including frames of highrise buildings, bridges, offshore structures, cast-in-situ piles in foundation etc.

Design methods for concrete-filled steel tubular sections are recommended in a number of codes of practices. However, there are significant discrepancies in the assumptions for the derivations of the ultimate strength of concrete-filled hollow members. Current design theories assumed full strain compatibility between the steel shell and the concrete core for the simplification of the calculation. This assumption is questionable particularly for the composite flexural members where slippage at the interface is inevitable after the tensile cracking of concrete. Design methods provided in the codes of practices assumed the normal strength concrete as the filler material in the concrete-filled steel hollow members. The same code is also used in many design situations when high strength concrete is used. Due to the significant differences in the material
properties between normal strength concrete and high strength concrete, there is a need to study the behaviour of composite sections with higher strength concretes.

In this thesis the study is presented on the behaviour of the flexural members and the beam-column connections of RHS filled with high strength concrete. The study emphasises ultimate strength, ductility, post-failure strength reserve and interface bond. Investigations were carried out in two different methods, experimentally and by analysis. The analysis work was also conducted in two separate methods. The analysis of ultimate moment and deflections of the composite beams were performed from the principle of structural mechanics. Finite element analysis were conducted to model the physical behaviour and also to compare the experimental results.

In the experimental investigations, study was carried out on the flexural behaviour of steel RHS filled with high strength concrete. Steel fibres were mixed with high strength concrete in some of the specimens to examine its effect. A Y16 bar was also provided in the tensile zone of the composite section in some of the beam specimens. The study emphasises ductility and post-failure strength of the composite beams due to the brittle behaviour of higher strength concretes when compare to normal strength concrete. Spreadsheet graphs were used to present the results such as load versus strains, load versus deflections etc.

Experimental studies were also conducted on beam-column connections where both beam and column portions were constructed from steel RHS filled with high strength concrete. Connections were tested with different configurations, they were in geometry
and also the inside surface condition of the steel shell. Emphasis was given on rotational capacity and post-failure behaviour of these rigid composite connections. Interface bond between the steel shell and the concrete core was also investigated.

In this study, pushout resistance tests were conducted on steel RHS filled with high strength concrete. This is to determine the load-slip behaviour at the interface which was later used in the finite element analysis. In the pushout tests, different inside steel surface treatment were adopted for comparison.

In this thesis analytical study is presented on the calculation of ultimate moment of resistance of the concrete-filled RHS beams. Among the main considerations of the derivation, the steel portion was assumed either elastic-perfectly plastic or perfectly plastic and the concrete carries no strength in the tensile zone. At the interface both full bond and partial bond were assumed for comparison. Efforts were also made to calculate the midspan deflections of the composite beams. Simple analytical expressions derived from this study can be coded to a programmable calculator or in a small spreadsheet program for design use.

Finite element studies were carried out by using a proprietorship software package ANSYS. In the analysis of concrete-filled RHS, three types of elements with large deformation and nonlinear capabilities were used. A plastic shell element, a solid concrete element with cracking and crushing capabilities, and a nonlinear spring contact element were used to model the steel shell, the concrete core and the interface respectively. Finite element models were developed and analysed on beams, beam-
column connections and for pushout resistance of steel RHS filled with high strength concrete. Efforts were made to compare the experimental results. In the study of beams and connections, interface was modelled by nonlinear spring elements where the input of the spring stiffnesses were obtained from the pushout tests. Models were also analysed for full bond situation between steel and concrete. In the finite element study on the pushout behaviour of concrete-filled RHS, models were also analysed to examine the influence of different parameters such as the shell thickness and the size of the cross-section of the steel RHS, and the compressive strength of concrete.
Preface

This thesis is submitted to the University of Western Sydney, Hawkesbury, Australia, for the degree of Doctor of Philosophy. The work described in this thesis was carried out in James Goldston Faculty of Engineering at Central Queensland University during the period 1993-1995, and in the School of Construction and Building Sciences at University of Western Sydney, Hawkesbury in the period 1995-1997. The candidate was supervised by Dr. John Zhang throughout the period (1993-1997) of candidature.

The author affirms the work presented in this thesis is entirely original unless otherwise referenced within the text. The author also affirms that this work has not been submitted at any other institution for a higher degree.

The following papers published were derived from the abovementioned research study.


Koushik Brahmachari
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Throughout the period of the candidature (1993-1997) the author was supervised by Dr. John Zhang, currently Senior Lecturer of the School of Construction and Building Sciences, University of Western Sydney, Hawkesbury. The author wishes to express his profound gratitude, appreciation and respect to his supervisor Dr. John Zhang for his patient guidance, constant inspiration, valuable suggestions and help in all possible ways during the entire candidature. Dr. Zhang was instrumental and inspirational during both the experimental and the analytical work carried out by the author.

The major analytical work of this thesis was performed by using a proprietorship software package ANSYS. Funds to purchase this package was provided by the School
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Contents

Synopsis ........................................ i

Preface .......................................... v

Acknowledgement ............................... vii

Contents ........................................ ix

1 Introduction

1.1 BACKGROUND .................................. 1

1.2 STATEMENT OF THE PROBLEM ............ 3

1.3 TOPICS TREATED IN THIS THESIS .......... 6
   1.3.1 General .................................. 6
   1.3.2 Experimental Studies .................... 7
   1.3.3 Analytical Studies ....................... 8

2 Literature Review

2.1 INTRODUCTION ............................. 11
2.2 EXPERIMENTAL STUDIES

2.2.1 Concrete-filled steel hollow flexural members

2.2.1.1 General

2.2.1.2 Initial stiffness

2.2.1.3 Local buckling of the steel shell

2.2.1.4 Ductility and post-failure strength reserve

2.2.1.5 Ultimate strength

2.2.1.6 Slip at the interface

2.2.2 Concrete-filled RHS beam-column connections

2.2.2.1 General

2.2.2.2 Rotational capacity

2.2.2.3 Ultimate strength

2.2.2.4 Local buckling

2.2.3 Pushout resistance of concrete-filled RHS

2.2.3.1 General

2.2.3.2 Size and shape of the cross-section

2.2.3.3 Wall thickness of the RHS

2.2.3.4 Properties of concrete

2.3 ANALYSIS AND DESIGN

2.3.1 General

2.3.2 Ultimate moment capacity

2.3.3 Interface bond
3 Experimental Study on The Flexural Behaviour of Rectangular Hollow Sections Filled with High Strength Concrete

3.1 INTRODUCTION. .......................... 31
3.2 PREPARATION OF SPECIMENS AND TEST SET UP ........ 33
  3.2.1 Preparation of specimens .......................... 33
  3.2.2 Test set up .................................. 36
3.3 TEST RESULTS AND DISCUSSION ........................ 36
3.4 SUMMARY .................................. 45

4 Experimental Study on The Beam-Column Connections of Steel Rectangular Hollow Sections Filled with High Strength Concrete

4.1 INTRODUCTION. .......................... 47
4.2 PREPARATION OF SPECIMENS ............... 49
4.3 TEST SET UP ................................ 52
4.4 RESULTS AND DISCUSSION .................... 54
5 Experimental Study on The Pushout Behaviour of Steel

Rectangular Hollow Sections Filled with High Strength Concrete

5.1 INTRODUCTION. . . . . . . . . . . . . . . . . . . 64
5.2 PREPARATION OF TEST SPECIMENS . . . . . . . 66
5.3 TEST SET UP AND INSTRUMENTATION. . . . . . . 68
5.4 TEST RESULTS AND DISCUSSION . . . . . . . . 69
5.5 SUMMARY . . . . . . . . . . . . . . . . . . . . . . . . 74

6 Analytical Study on The Flexural Behaviour of Concrete- Filled

Steel Rectangular Hollow Sections

6.1 INTRODUCTION. . . . . . . . . . . . . . . . . . . 75
6.2 THE ULTIMATE MOMENT OF RESISTANCE. . . . . 77
  6.2.1 Analysis based on strain compatibility at the interface . 77
  6.2.2 Analysis by considering slippage at the interface . . 83
  6.2.3 Comparison with test results . . . . . . . . . . . 88
6.3 DEFLECTIONS AT THE LIMIT OF SERVICEABILITY . . . 92
  6.3.1 Analysis based on strain compatibility at the interface . 93
  6.3.2 Analysis by considering slippage at the interface . . 95
  6.3.3 Comparison with test results . . . . . . . . . . . 96
7 Finite Element Analysis on The Flexural Members of RHS Filled with High Strength Concrete

7.1 INTRODUCTION. ........................................ 102
7.2 MODELLING ........................................... 104
  7.2.1 Geometrical modelling ......................... 104
  7.2.2 Material modelling ............................... 106
  7.2.3 Bond modelling .................................. 108
7.3 FINITE ELEMENT ANALYSIS ........................ 110
7.4 RESULTS AND DISCUSSION .......................... 111
7.5 SUMMARY ............................................. 117

8 Finite Element Analysis on The Beam-Column Connections of RHS Filled with High Strength Concrete

8.1 INTRODUCTION. ........................................ 119
8.2 MODELLING ........................................... 121
  8.2.1 Geometrical modelling ......................... 121
  8.2.2 Material modelling ............................... 122
  8.2.3 Bond modelling .................................. 123
8.3 FINITE ELEMENT ANALYSIS .................................................. 124
8.4 RESULTS AND DISCUSSION .................................................. 126
8.5 SUMMARY ............................................................................ 130

9 Finite Element Analysis on The Pushout Behaviour of RHS Filled with High Strength Concrete

9.1 INTRODUCTION. ................................................................. 132
9.2 MODELLING ......................................................................... 133
  9.2.1 Geometrical modelling ..................................................... 133
  9.2.2 Material modelling .......................................................... 134
  9.2.3 Bond modelling .............................................................. 134
9.3 FINITE ELEMENT ANALYSIS ................................................ 136
9.4 RESULTS AND DISCUSSION ................................................ 137
9.5 PARAMETRIC STUDIES ...................................................... 143
  9.5.1 General ........................................................................ 143
  9.5.2 Wall thickness of the RHS ............................................... 143
  9.5.3 Size of the RHS cross-section ......................................... 145
  9.5.4 Compressive strength of concrete .................................... 148
9.6 SUMMARY ............................................................................ 149

10 Conclusions

10.1 GENERAL ........................................................................... 151

xiv
10.2 EXPERIMENTAL STUDIES . . . . . . 153

10.2.1 Concrete-filled RHS beams . . . . . . 153

10.2.2 Concrete-filled RHS beam-column Connections . . 153

10.2.3 Bond strength of the concrete-filled RHS . . . . 154

10.3 ANALYTICAL STUDIES . . . . . . . . . . 155

10.3.1 Analysis of the ultimate moment of resistance and deflections of concrete-filled RHS beams . . . . . . 155

10.3.2 Finite Element Analysis . . . . . . . . 156

10.3.2.1 Concrete-filled RHS beams . . . . . . 156

10.3.2.2 Beam-column connections . . . . . . 157

10.3.2.3 Pushout behaviour of the concrete-filled RHS . 157

References . . . . . . . . . . . . . . . . . . . . . . . . . 159