BEHAVIOUR OF HYBRID STAINLESS-CARBON STEEL COMPOSITE BEAM-COLUMN JOINTS

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Behaviour of Hybrid Stainless-Carbon Steel Composite Beam-Column Joints

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STATEMENT OF AUTHENTICATION

I hereby declare that all materials presented in this thesis are of my own works, except for quotations and summaries which have been duly acknowledged. I also declare that this thesis contains no materials that have been submitted previously, either in full or in part, for any degree at Western Sydney University or other institution.

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ABSTRACT

Concrete-filled steel tubular (CFST) columns have been widely used in high-rise building structure due to its excellent structural properties such as high strength, high stiffness, high ductility and large energy absorption capacity. For this reason, the combined use of CFST columns and steel beams in composite building construction is increasing. Conventionally, welded connections are often used to connect steel beams to CFST columns. Such connections are very costly and also require on-site welding. In recent years, blind-bolted connections have been developed to simplify the in-situ installation procedure, while allowing the connection to maintain good structural performance. Although extensive studies have been conducted on steel beam-CFST column joints without slab, the opposite is true for steel beam-CFST column joints with slab. As a result, the development of blind-bolted connections is still in its infancy stage. The main objective of this study is to develop reliable connection between steel beams and CFST columns using blind bolts and to provide detailed information from experimental and numerical investigations, using mathematical models for predicting the initial rotational stiffness and moment-rotation relationship of blind-bolted connections in steel beam-CFST column joints with slab.

In the experimental work, two types of composite steel beam-CFST column joints were investigated using blind-bolted connections and through-plate connections. The tests demonstrated that blind-bolted connections with slab had outstanding structural properties in terms of their initial rotational stiffness, ultimate hogging moment and rotational capacity under static monotonic loading, compared to through-plate connections. However, the hogging moment capacity (37.7 kN-m) of blind-bolted connections without slab was very low (seven times lower) compared to the moment capacity of the connections with slab (264.0 kN-m). The moment capacity of the blind-bolted connections with slab was found to be double than that of through-plate connections with slab. It was also found that the moment capacity of blind-bolted connections using square columns was lower than blind-bolted connections with circular columns. By introducing binding bars into the square columns, the ultimate capacity of blind-bolted connections was enhanced. For both square and circular columns, blind-bolted connections can be used as rigid connections in braced frames. Through-plate connections can be considered as semi-rigid connections.
In the numerical analysis, finite element (FE) models developed using ABAQUS were validated by comparing their results with test data both from this work and available in the literature. The validated FE models were used to conduct very detailed numerical parametric studies in which a wide range of the parameters were used to further investigate the influences of those parameters on the behaviour of composite steel beam-CFST column joints. The FE analyses results indicate that Hollo-Bolts have a better performance when bolts connected to CFST columns compared to SHS columns.

It was also seen from FE analyses results of the blind-bolted connections of the CFST column joints that the addition of binding bars to the steel tube of the CFST columns can significantly enhance the moment capacity of the blind-bolted joints and minimise the steel tube outward deformation compared to those of the blind-bolted joints without binding bars. The numerical results indicate that binding bars are more effective when binding bars are placed in CFST columns at the top and bottom of the bolts subjected to tensile load. For composite CFST column joints with slab, binding bars can be used near bottom bolt row only; which help to improve the sagging moment capacity as well.

It was seen for joints with slab that the main influencing parameters include slab reinforcement, slab depth, number of shear connectors, profiled sheet orientation, steel tube size, beam depth and the material properties of the connecting components. In Eurocode 4, the effect of the profiled steel sheeting is ignored; however, the FE model results demonstrated that it has significant influence on the ultimate moment capacity of composite joints. This indicates that the effect of the profiled steel sheeting should be considered in moment capacity design calculations for the composite joints.

Finally, mathematical models were developed for predicting the initial rotational stiffness, plastic moment, ultimate moment and ultimate rotation of blind-bolted connections to CFST column; based on the component-based approach. In addition, full-range moment-rotation relationships of blind-bolted connections are proposed which require only three parameters: initial stiffness \( S_{j,in} \), ultimate moment \( M_u \) and ultimate rotation \( \phi_u \). The different stages of the moment-rotation curves of the proposed model matched the test results very well. The proposed moment-rotation model can be used in the design and analysis of semi-rigid frames using blind-bolted endplate connections, CFST columns and steel-concrete composite beams.
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CHAPTER 1

INTRODUCTION

1.1 Introduction
Concrete-filled steel tubular (CFST) columns have been widely used in high-rise building construction in recent years. The combined utilisation of CFST columns and composite beams are gaining popularity due to the structural and constructional benefits provided by CFST columns and composite beams. But their implementations depend on how composite beams are connected to CFST columns. In recent years, an emerging technology which eliminates the need of access to the interior of the CFST column tube has been developed, using blind bolts to connect steel beams to CFST columns. This type of connection is known as blind-bolted endplate connections and recognised as semi-rigid connections. Extensive research has been conducted on steel beam-CFST column joints without slab, but limited research have been conducted on steel beam-CFST column joints with slab; indeed, no research has been conducted on hybrid composite joints to connect the composite beams to concrete-filled stainless steel tubular (CFSST) column.

What is a hybrid composite joint? It is a composite joint consisting of two or more components and combining the use of stainless steel to fabricate the CFSST column and carbon steel to fabricate the steel beams and other steel components, and thus provide composite action. In the development of hybrid composite steel beam-CFSST column joints as shown in Figure 1.1, the joints are designed using endplate, blind bolts and binding bars. The endplate and blind bolts are used to connect the steel beams to the thin steel tube of CFSST column. Binding bars are welded to the opposite face of the steel tube acting as stiffener to provide the composite action between the thin steel tube and concrete. This research has investigated the behaviour of hybrid composite steel beam–CFST column joints with slab by conducting physical tests and finite element analysis. Finally mathematical models were developed to predict the initial
rotational stiffness of joints and to predict the moment-rotation curve of a hybrid composite joint.

This chapter provides an overview of the work described in this thesis on the hybrid composite steel beam-CFST column joints, mainly highlighting the research background, problem statement of the research, objectives of the research, research methodology and finally outline of the thesis.

![Figure 1.1 Typical example of hybrid stainless-carbon steel composite beam-column joints](image)

### 1.2 Research Background

In the building construction there are two types of structure; reinforced concrete structure and steel structure. Steel structure uses beams and columns of steel materials only. In recent years, concrete-filled steel tubular (CFST) columns have become widely used to minimise the amount of steel used in the columns. CFST columns are generally fabricated from mild steel tubes and concrete. The use of stainless steel tubes for CFST columns is becoming attractive solution due to its high strength, large stiffness and ductility, corrosion resistance, and the reduction of local buckling of the steel tube that is provided by the infill concrete core (Bradford et al., 2006; Ellobody and Young, 2006; Uy, 2008; Zhao et al., 2010; Tao et al., 2009 and 2011). Stainless steel CFST columns also provide not only an increase in the load-carrying capacity but also eliminate the use of the formwork during construction and economies of total maintenance, thus saving additional cost (Han et al., 2014). These advantages have been recognised when compared with both conventional steel i.e. carbon or mild steel structures and concrete structures, and thus have led to an increasing use of concrete-
filled stainless steel tubular (CFSST) columns in some tall buildings and super high rise buildings.

In the current practice of CFST column-based composite building structures, the connections between steel beams to CFST or CFSST columns apply conventional connections systems that are used for steel structures. Most commonly connections used in the steel beam-CFST column joints are welded connections, fin plate connections, through plate connections, angle cleat connections and blind-bolted endplate connections (Xiao et al., 1994; da Silva et al., 2001; Cabrera and Bayo, 2005; Jones, 2008; Yao et al., 2008; Wang and Li, 2007; Mirza and Uy, 2011). Of these connections, welded connections are widely used in steel beam-CFST column joints. However, the configuration of welded connection imposes high deformation demands on the steel tube of the CFST column, possibly causing fracturing of the steel tube wall (Wang et al., 2009a). This results in a deterioration of strength and stiffness of the composite beam-CFST column joints. To overcome this problem, external diaphragm, internal diaphragms or other complex stiffeners are required (Qin et al., 2014). These produce excellent structural performance in terms of initial stiffness, moment, rotation and ductility of joints, but such complicated arrangements slow the workability and increase the total construction time. In addition, welded connections are very costly due to extensive welding and low tolerances required in detailing and they add further complications to the construction (Schneider and Alostaz, 1998, Mirza and Uy, 2011).

Blind-bolted endplate connections are much easier to install on site than welded connections for connecting steel beams to steel tube of CFST columns. In blind bolted endplate connection, blind bolts and endplate are used to connect the steel beam with CFST column. Blind-bolts (Huck high strength blind bolt, Lindapter Hollo-Bolt also known as Hollo-Bolt, and Ajax ONESID bolt, etc.) are structural bolts that tightened from the outer side only. Hollo-Bolts and Ajax bolts are most commonly used as blind-bolts in blind-bolted endplate connections. For steel beam-CFST column joints, several studies of blind-bolted endplate connections were reported (France et al., 1999a & b; Loh et al., 2006a; Goldsworthy and Gardner, 2006; Yao et al., 2008; Wang et al., 2009b; Mirza and Uy, 2011; Tizani et al., 2013a & b). In most of these, except Loh et al. (2006b) and Mirza and Uy (2011), the connections were tested without considering the slab effect on the joints. Loh et al. (2006b) tested six joint specimens under
monotonic static loading in which the shear connector and reinforcement were considered as the main parameters. Mirza and Uy (2011) conducted two tests, one under static loading and one under cyclic loading.

1.3 Problem Statement of Research

To overcome the need for extensive welding and to give better solution in the design of steel beam-to-CFST joints, blind bolt technology used in endplate connections to connect the steel beams with CFST columns has been investigated by France et al. (1999a & b), Loh et al. (2006b), Goldsworthy and Gardner (2006), Yao et al. (2008), Wang et al. (2009b), Mirza and Uy (2011), Tizani et al. (2013a & b). However, a study of blind-bolted connections conducted by Wang et al. (2009b) revealed that the outward deformation of the tube walls at the location of the top tension bolts was very locally concentrated. This is largely due to the fact that the blind bolts, especially Ajax blind bolts, are not anchored into the concrete core, and the steel tube of the CFST column is not perfectly bonded to the concrete core. In addition, the bonding between the steel tube and concrete is not very strong. Thus, separation between the concrete core and steel tube is normally observed. In addition, for a thin-walled steel tube with limited membrane stiffness, the application of a blind bolt can be problematic. To address this issue, Goldsworthy and Gardner (2006) proposed the use of a cogged extension welded to the head of the blind bolt, but these were tested without considering the slab effects. Adding a cogged extension to the blind bolts improves the behaviour of blind bolted connections by transferring the load between the steel tube wall and the cogged anchorage within the concrete. Although, excessive local deformation of the tube wall is avoided using this technique, the presence of the extension largely counteracts the advantage of ease of installation provided by blind bolts. To resolve the steel tube separation problem, extended bolt shank was introduced by Tizani et al. (2013a), in which a nut was provided at the end of the extended bolt shank. Thus, commercially available blind bolts must be modified depending on the demand for connections of a given building structure. Since it would be very costly to produce special blind bolts for individual building construction, alternative solutions are still required that would both solve the steel tube outward-deformation problem and enhance the integrity of the joints without needing to modify commercially available blind bolts.
In recent years, researchers have begun to investigate the feasibility of using Hollo-Bolts and Ajax blind-bolts to connect the steel beams with CFST columns. It was seen that Ajax blind-bolts are similar to, and behave like normal bolts, but Hollo-Bolts are geometrically quite different from other blind-bolts; Hassan et al. (2014) reported that the behaviour of Hollo-Bolts differed from the other blind-bolts because of this. It is also reported that the behaviour of Hollo-Bolts when connected to CFST column was different from Hollo-Bolted connected to open section or square hollow section (SHS) column because the geometry of Hollo-Bolts and the embedding condition of Hollo-Bolts into the concrete core of the CFST column. The behaviour of Hollo-Bolts has been investigated in previous study by conducting tests similar to tests with normal bolts, in which the Hollo-Bolt was connected either to the open section or square hollow section (SHS) column. This does not represent the behaviour of Hollo-Bolts when connected to CFST columns. Therefore, extensive research is also needed to investigate the behaviour of Hollo-Bolts, especially when they are connected to CFST column.

In the analysis and design of a composite structure with blind-bolted steel beam-CFST column joints, it is also very important to know the initial rotational stiffness, the ultimate moment and the rotation of composite joints. Preferably, the moment-rotation relation of joints should be provided. In the current practice, the existing mathematical models developed for the steel beam-to-steel column joints have been used uncritically to determine the initial rotational stiffness, ultimate moment and rotation of the steel beam-CFST column joints. So it is quite necessary to develop mathematical models to determine the structural properties of blind-bolted endplate connections between steel beams and CFST columns, because the behaviour of blind bolts in CFST columns are quite different from that of normal bolts or blind bolts in steel column. Meanwhile, most previous tests of blind-bolted endplate connections did not consider the effects of the presence of the slab on the steel beam-CFST column joints. It is also clear that the behaviour of the steel beam-CFST column joints without a slab is not similar to that when the slab is present. In particular, the initial rotational stiffness, ultimate moment and rotation of joints with and without a slab are quite different. So, more studies are to be conducted to investigate the actual behaviour of steel beam-CFST column joints when the effect of the slab is taken into account.
1.4 Objectives of Research

The main objective of this research is to develop, by both experimental and numerical investigations, a reliable connection between carbon steel beams and CFSST columns using blind bolts, and to provide detailed information about the mathematical models developed to predict initial rotational stiffness and the moment-rotation relationship of blind-bolted endplate connection of steel beam-CFST column joints when a slab is present. The objectives of this research are outlined as:

i. To carry out a feasibility study on a composite beam-column joint to investigate the behaviour of different types of connections. Finally the effective connection type will be identified.

ii. To conduct experimental tests on innovative hybrid composite joints based on blind-bolted endplate connections to connect steel beams to steel CFST columns. Through-plate connections are also considered in the tests to compare with blind-bolted connections. The test results are used to validate a finite element (FE) simulation model of hybrid composite beam-column joints.

iii. To investigate the behaviour, under pure tension and shear loading, of blind bolts when connected to SHS columns or CFST columns.

iv. To find out an alternative solution that avoids modifying commercially available blind bolts to resolve the steel tube outward deformation problem in CFST columns. The alternative solution is used to enhance the integrity of the joint panel zone.

v. To determine the parameters those effectively control the behaviour of composite beam–CFST column joints. A limited range of parameters was considered in the testing of the joints due to the limitations imposed by the cost of such experiments; however, this objective covers all the parameters related to composite beam–CFST column joints when a slab is present.

vi. To develop the mathematical models for the rotational stiffness and moment-rotation relationship curves of the blind-bolted endplate connection of composite steel beam-CFST column joints.
1.5 Research Methodology

To achieve the objectives, the research program was divided into three groups: experimental works, numerical studies using finite element (FE) analysis and theoretical analysis for mathematical model development using Component Method. A summary of the research methodology that was used in this work to achieve its objectives is shown in Figure 1.2.

![Flow chart of research methodology](image)

**1.5.1 Experimental work**

In the experimental research work, innovative blind-bolted hybrid composite joints with a slab were considered for testing. A total of 10 cross-shaped beam-column composite joint specimens with blind-bolted connections (seven specimens) and through plate connections (three specimens) were tested. The height of the CFST column was 2200 mm; the length of the steel beam was 1320 mm; and thickness of the composite slab
was 120 mm. Details of the joint specimen dimensions and parameters are illustrated in Chapter 3. The following parameters were investigated in the tests:

1. Connection type: blind-bolted connections and through plate connections
2. Column steel types: stainless steel and carbon steel
3. Column section: square and circular
4. Binding bar: with and without binding bars
5. Loading types: Monotonic static loading and cyclic loading.

1.5.2 Numerical studies using FE analysis
Numerical studies were conducted using finite element (FE) analysis to investigate the behaviour of different components in the composite beam-CFST column joints with blind-bolted endplate connections. The solid FE models were established using ABAQUS software (2012) for the composite beam-CFST column joints with blind-bolted connections. The behaviour of blind bolts was investigated under pure tension and shear loading. The behaviour of blind-bolted endplate connections of the steel beam-CFST column joints (without slab) was investigated for different parameters. The steel beam-CFST column joints with slab were also investigated to determine the effects of various parameters that influence the connection behaviour.

1.5.3 Theoretical analysis
Theoretical studies were conducted to calculate the initial rotational stiffness, ultimate moment and ultimate rotation of the composite steel beam-CFST column joints. A mathematical model was developed using Component Method to determine the initial rotational stiffness of composite steel beam-CFST column joints with blind-bolted endplate connections. Mathematical expressions to determine the ultimate moment and rotation of composite joints were also established based on the existing equations. A moment-rotation relationship model was developed for these types of composite joints. The proposed moment-rotation relationship model of joints was verified with test results that collected from literature and reported in this thesis.

1.6 Outline of Thesis
The work presented in this thesis is organised into 10 chapters: Chapter 1-introduction, Chapter 2-literature review, Chapter 3-experimental program and material testing,
Chapter 2 presents the background information for blind-bolted connections to connect steel beams to CFST columns. This chapter includes a brief description of CFST columns, beam-column joints, and the types of connections used in steel beam-CFST column joints, and the bolting system for endplate connections. Previous experimental and numerical research works are summarised to provide a general understanding of the behaviour of composite steel beam-CFST column joints. The basic principles of Component Method are illustrated for developing the analytical model for the composite steel beam-CFST column joints. Finally, concluding remarks are noted based on the literature review, providing the research directions and the need for research works.

Chapter 3 presents the experimental program details and materials testing results. In the experimental program, two types of innovative hybrid composite steel beam-CFST column joints with slab are designed for blind-bolted endplate connections and through plate connections, and tested under monotonic static loading and cyclic loading. The material properties of connecting components of hybrid composite steel beam-CFST column joints are tested.

Chapter 4 describes the test results of two types of innovative hybrid composite steel beam-CFST column joints with slab. Discussion of the test results highlights the blind-bolted endplate connections and the through-plate connections. The key findings of the test results are summarised in this chapter.

Chapter 5 demonstrates the development of finite element (FE) modelling to simulate the steel beam-to-CFST column joints with blind-bolted endplate connections. In FE model development, the details of material modelling, element types, blind-bolt modelling technique, contact modelling and load and boundary conditions for non-linear analysis are illustrated. In material modelling, the full-range stress-strain curves...
of the steel tube, endplate, steel beam and blind bolts are proposed to simulate the material degradation parts and to capture the full behaviour including necking and failure. A sensitivity analysis is conducted to select appropriate type of solid element. A simplified FE model is proposed to simulate Hollo-bolts. Finally, the developed FE model is verified against previous test results collected from literature as well as the results of the present test conducted in this research. Three types of test data are considered to verify the FE simulation: (1) blind-bolts tested in pure tension and shear loading; (2) CFST column joints (without slab) tested in bending loading separately, (3) CFST column joints (with slab) tested in bending loading.

Chapter 6 illustrates the parametric analysis of the blind bolts (Hollo-bolts) subjected to pure tension and shear loading. Different parameters related to Hollo-Bolts and connecting parts including steel tube of SHS or CFST column and endplate are considered to assess their effect on the behaviour of blind bolts. The behaviour of blind bolts when connected to a square hollow steel column or a CFST column is mainly investigated under these two loading conditions. Finally the tensile and shear strengths of Hollo-bolts are suggested for SHS column and CFST column separately, based on the parametric analysis of Hollo-bolts at two different connecting conditions (SHS column and CFST column).

Chapter 7 describes the parametric analysis results of blind bolted endplate connections to CFST column joints with binding bars. Binding bars in CFST column are considered to minimise the outward deformation of the steel tube of square CFST column and to enhance the integrity of the joint panel zone. Parametric analyses are conducted to investigate the influence of important parameters, such as the number, diameter and location of the binding bars, on the performance of blind-bolted endplate connections to steel beam-CFST column joints. Based on the parametric analysis results, the appropriate number, locations and diameters of binding bars are suggested to design the binding bars for CFST column joints.

Chapter 8 mainly focuses on the parametric analysis of the blind-bolted endplate connections to composite CFST column joints with slab, using the stainless steel for the steel tube of CFST column and mild steel for the remaining steel components of the composite CFST column joints. Parametric studies are conducted to cover all
parameters related to the composite CFST column joints such as connecting parameters (endplate, blind-bolts), slab parameters (slab depth, slab reinforcement ratio, material strength of slab concrete, steel profile sheet orientation and its yield strength), composite beam parameters (shear connector ratio, beam depth, beam yield strength) and CFST column parameters (width-to-thickness ratio of steel tube, steel tube’s yield strength, column concrete strength).

**Chapter 9** presents the mathematical models developed to calculate the initial rotational stiffness of composite steel beam-CFST column joints using Component Method. The moment-rotation relationship model of joints is also developed and validated with test results of composite steel beam-CFST column joints.

**Chapter 10** summarises the conclusions drawn from this research and finally provides some recommendations and suggestions for future works.
CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

In Chapter 1, a brief description of the research background and problem statements of the present research are highlighted. This research mainly focused on the behaviour of hybrid composite steel beam-to-concrete filled steel tubular (CFST) column joints. This chapter provides information for the steel beam-CFST column joints, introduced as CFST column joints, and includes a brief description of CFST columns, CFST column joints and the types of connections that are used for CFST column joints, and bolting systems for endplate connections. Previously reported experimental and numerical research are summarised to provide a general understanding of composite CFST column joints when a slab is present. The basic principles of the component method are illustrated to develop the analytical model for the composite steel beam-CFST column joints. The concluding remarks are noted based on the literature review and provide the research directions and the need for research works.

2.2 Concrete-Filled Steel Tubular (CFST) Columns

Combinations of steel and concrete are incorporated in the construction of composite columns, in the form of concrete encased steel (CES) columns as shown in Figure 2.1(a) and concrete-filled steel tubular (CFST) columns as shown in Figure 2.1(b). Compared with CES columns, CFST columns have many structural and constructional advantages, and are being increasingly used in many countries. CFST columns are mainly used in industrial buildings, structural frame and supports, electricity transmission poles and spatial constructions (Han et al, 2014). Several examples of CFST column based structures (see Figure 2.2 and in Table 2.1) are found in the literature: these include Latitude Tower in Sydney, Australia; SEG Plaza, Guangzhou West Tower and Goldin 117 Tower in China; W-Comfort Towers, Otemachi Tower and Abeno Haruk in Japan; Taipei 101 Tower in Taiwan; and the Pacific First Centre, Two-Union building, Gateway Tower and Hearst Tower constructed in USA.
Recently, with the rising demand for durability, efficiency and sustainability of structures, there has been an increasing use of stainless steel to replace the carbon steel (Rasmussen, 2000; Gardner, 2008). The application of stainless steel in composite frame structures has also increased because of its inherent qualities, such as natural corrosion resistance, which allows the surface to be exposed without the need for protective coating (Real and Mirambell, 2005; Baddoo, 2008; Lam and Gardner, 2008; Tao et al., 2009, 2011). Stainless steel was first used in 1930 for the upper façade of the Chrysler Building, in New York in 1930 (Gardner, 2008). Stainless steel was also used in the construction of the 46 storey Hearst Tower in New York City in 2006, shown in Figure 2.2 (b) (Gardner, 2008; Uy, 2008) and in the Stonecutters Bridge, Hong Kong (SCI, 2010). The use of stainless steel for the steel tubes of CFST columns is an attractive solution for its high strength, large stiffness and ductility, corrosion resistance and resistance to local buckling of the steel tube provided by the infill concrete core (Bradford et al., 2006; Ellobody and Young, 2006; Uy, 2008; Zhao et al., 2010; Tao et al., 2009 & 2011). It has been reported that CFST columns using stainless steel provides not only an increase in the load-carrying capacity but also eliminates the use of the formwork during construction and economy in terms of total maintenance cost, thus saving additional cost (Han et al., 2014). Due to these structural and non-structural benefits provided by stainless steel, it is increasing used in CFST columns for the construction of composite frame structure buildings.

![Typical examples of composite columns](image)

**Figure 2.1 Typical examples of composite columns**
Table 2.1 List of CFST column based composite structures constructed in different countries

<table>
<thead>
<tr>
<th>Building name, construction year and country name</th>
<th>Building height (m)</th>
<th>Concrete strength (MPa)</th>
<th>Steel strength (MPa)</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>SEG Plaza, 2000, China</td>
<td>356</td>
<td>–</td>
<td>–</td>
<td></td>
</tr>
<tr>
<td>Latitude Tower, 2005, Australia</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td></td>
</tr>
<tr>
<td>Hearst Tower, 2006, USA</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td></td>
</tr>
</tbody>
</table>

2.3 CFST Column Joints

A CFST column joint is defined as a zone where steel beams and CFST column are connected to each other. Figure 2.3 shows the configuration of two typical CFST column joints. A joint consists of either one connection or two connections (Figure
2.3). Eurocode 3 (2005) defines a connection as the location at which two or more elements meet.

![Figure 2.3 Configuration of CFST column joints](image)

Eurocode 3 (2005) classifies joints on the basis of three main characteristics such as strength, stiffness and ductility. In most cases, joints are classified or evaluated based on the capacity of its strength and stiffness. Based on strength capacity, a joint is classified as (1) full strength joint, (2) partial strength joint or (3) pinned joint. Based on stiffness capacity, a joint is classified as (1) rigid joint, (2) semi-rigid joint or pinned joint. Based on ductility capacity, a joint is classified as (1) ductile joint, (2) semi-ductile joint or (3) brittle joint (Jaspart, 2002).

In full strength joints, the design moment resistance is higher than that of the connected members and the plastic hinges locate in the beam. Pinned joints transmit the internal shear and axial forces; without resisting any significant moment. According to Eurocode 3 (2005), the design moment resistance of a pinned joint is less than 25% of the strength required for full strength joint. The strength capacity of partial strength joint lies between that of full-strength joint and pinned joint. In partial strength joint, the plastic hinges initially develop in the joint.

In engineering practice, rigid, semi-rigid and pinned joints are classified on the basis of their initial rotational stiffness ($S_{j,ini}$). In rigid joints, the initial rotational stiffness of joints is greater than that of the frame structure. Eurocode 3 (2005) specifies that the initial rotational stiffness of a rigid joint should be more than eight times the beam
stiffness \((8K)\) for braced frame structures and 25 times the beam stiffness \((25K)\) for unbraced frame structures. For pinned joints, the initial rotational stiffness of a joint is less than half of the beam stiffness \((0.5 K)\). When the initial rotational stiffness of a joint lies between the pinned joint stiffness and rigid joint stiffness criteria, this type of joints is classified as a semi-rigid joint. Table 2.2 shows the boundary limits proposed by Eurocode 3 (2005) for the stiffness criteria of rigid, semi-rigid and pinned joints. The classification of joints by stiffness and their typical moment diagrams are shown in Figure 2.4.

Table 2.2 Stiffness criteria of rigid, semi-rigid and pinned joints (Eurocode 3, 2005)

<table>
<thead>
<tr>
<th>Type of joint</th>
<th>Stiffness Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Braced frames</td>
</tr>
<tr>
<td>1. Rigid joint</td>
<td>(S_{j,ini} \geq 8 K)</td>
</tr>
<tr>
<td>2. Semi-rigid joint</td>
<td>(8 K &gt; S_{j,ini} &gt; 0.5 K)</td>
</tr>
<tr>
<td>3. Pinned joint</td>
<td>(S_{j,ini} \leq 0.5 K)</td>
</tr>
</tbody>
</table>

The beam stiffness, \(K\) is calculated from the following equation:

\[
K = \frac{E}{L} \quad \text{(2.1)}
\]
where $E$ is the elastic modulus of connected beam, $I$ is the second moment of area of connected beam, and $L$ is the span of connected beam.

The ductility of a joint is an important parameter that represents the ability of a joint to withstand plastic deformation without rupture of joint components such as bolts, endplate, tube and beam. A lack of ductility is often termed brittleness. Depending on the ductility behaviour of the joint, it is classified as ductile, semi-ductile or brittle joints (Jaspart, 2002). Ductile joints show large deformation before fracture, brittle joints do not allow large deformation before fracture and fail without warning. Semi-ductile joints show moderate deformation and always give a warning before they fail. In ductile joints, endplate or column flange usually yield before the bolts yield. In this case bolts are stronger than the endplate or column flange. In semi-ductile joints, the bolts fail after the endplate or column flange yields, normally when the bolts are weaker than the endplate or column flange. In the brittle joints, the bolts fail first before the endplate or column flange yields. This type of joints is normally observed when the endplate and column flange are stronger than the bolts. Figure 2.5 shows the moment-rotation curves for joints of different ductility.

![Figure 2.5 Classification of joints by ductility](image)

2.4 Types of Connections used in CFST column Joints

A wide range of different types of connections is used for the design of the joints. These include simple welded connections, complex welded connections (using external and internal diaphragms), fin plate connections, through plate connections, angel web cleat connections, endplate connections and haunched connections. The most
commonly used for connecting steel beams to CFST columns are welded connections, fin plate connections, through plate connections and endplate connections. The detailed descriptions of these connections are given in the following sections.

2.4.1 Welded connections
Welded connections are generally fully rigid connections. In welded connections, steel beams are welded to the steel tube of the CFST column (as shown in Figure 2.6). In simple welded connections without stiffening components, if steel beams are directly welded to the steel tube of composite columns, tensile and shear forces are transferred to the steel tube. As a result, the steel tube separates from the concrete core and overstressing occurs in the steel tube. Due to the steel tube deformation, the rotation of the connection is increased and its stiffness decreases. Thus simple welded connections are not suitable for thin tube CFST columns. To reduce the connection rotation and to increase its stiffness, the use of complex welded connections with external or internal diaphragms and stiffeners, as shown in Figure 2.7, have been suggested by other researchers, reported in Qin et al. (2014). The welding of these types of connections is more complicated in the construction field; it can be very difficult to control the quality of the welding, and it is very expensive. In Australian practice, welding is preferably conducted in the fabrication shop, and at the construction site the connecting members are bolted together.

Figure 2.6 Welded connections to connect steel beams to CFST columns (Kurobane et al., 2004)
Figure 2.7 Complicated welded connections for CFST column joints using different types of diaphragm (Qin et al., 2014)

2.4.2 **Fin plate connections**

Fin plate connections are used for simple pinned connections. This type of connection consists of a fin plate welded to the steel tube of the CFST column and bolted to the web of the steel beam using two or more structural bolts (as shown in Figure 2.8). This type of connection is very easily installed in the field. However, fractures at the flange tip on the connection, and tube-wall tearing are often observed (Kurobane et al., 2004). In addition, the stiffness of this connection is comparatively lower than welded connections in which the steel beam is directly attached to the steel tube of the CFST column (Kurobane et al., 2004). Furthermore, experimental results reported by Jones (2008) noted that weld pull-out was observed around the weld at the top of the connection to the steel tube of the CFST column at ambient and elevated temperatures.

Figure 2.8 Typical fin plate connections (Kurobane et al., 2004)
2.4.3 Through-plate connections

In a through-plate connection, a steel plate passes through the CFST column and is welded to the face of the steel tube (shown in Figure 2.9(a)). The through plate extends outside the column and is also welded or bolted to the steel beam web (as shown in Figure 2.9(a)). The through plate transfers the internal forces in the beam to the column through the bearing and friction between the through plate and concrete (Minouei and Mirghader, 2009). Thus some load is transferred directly to the concrete through contact of the steel plate on the concrete.

(a) Full through-plate connection (Minouei and Mirghader, 2009)

(b) Single plate connection (Kosteski and Packer, 2003)

Figure 2.9 Configuration of typical through-plate connections used in CFST column

Through-plate connections are often used in Australia in replace of the fin plate connections (BHP Structural Development Group, 1996). This type of connection is usually regarded as a simple pinned connection owing to its relatively small rotation stiffness. Moreover, if a joint is needed to connect beams coming from different
directions, welding two or even more through plates inside the steel tube is inevitable. So, through-plate connections are difficult to fabricate at a construction site and thus it raises the overall construction cost of the steel beam-to-CFST column connections. To address this issue, Kosteski and Packer (2003) proposed the use of two different types of plate in the through plate connections: a full-through plate and a single plate (Figure 2.9 (b)). Although, full-through plate connection shows a good capability to resist the deformation of the steel tube, but the single plate is not able to resist the deformation of the steel tube when the steel plate is only welded to the face of the tube.

2.4.4 Endplate connections

Endplate connections are often regarded as semi-rigid connections in the composite frame building structures. In endplate connections to join a steel beam to a CFST column, the endplate is welded to the end of the steel beam, and then bolted to the steel tube using blind bolts. Endplate connections are classified into three categories: (a) header, (b) extended and (c) flush endplate connections, as shown in Figure 2.10.

Figure 2.10 Different types of endplate connections used to connect the steel beam to CFST column

(a) Header endplate connections

In header endplate connections, the length of the endplate is less than the depth of the steel beam as shown in Figure 2.10(a). The header endplate transmits the bending moment into the column flange as well as into column web, but is unable to transmit the bending moment into the steel beam. The initial stiffness of this connection is comparatively low due to the small area of the header plate. The bolts are usually
placed within the depth area of the steel beam. Failure of the header endplate connections may be due to the failure of the bolts, the endplate or the steel tube.

(b) Extended endplate connections
In extended endplate connections, the endplate height is significantly larger than the depth of the steel beam, as shown in Figure 2.10(b). The space above and/or below the beam is used for an additional row of bolts. The extended endplate is able to transmit a significant amount of bending moment to the column and beam. The initial stiffness of this type of connection is higher than that of header endplate connections. The header endplate connections are used less often than extended endplate connections because of the lower strength and stiffness of header endplate connections. Wang and Chen (2012) conducted tests on extended endplate connections and reported that it has high strength and stiffness and it also satisfies the connection rotation capacity and the ductility that is required for earthquake resistance in seismic regions. Most failures of extended endplate connections occur in the top bolts, the endplate and the steel tube.

(c) Flush endplate connections
In the flush endplate connections, the height of the endplate is approximately the same as the depth of the steel beam as shown in Figure 2.10(c). For steel beam-CFST column joints without slab, the initial stiffness and ultimate capacity of flush endplate connections are observed comparatively higher than those for header endplate connection, but lower than those for extended endplate connections. However, the behaviour of flush endplate connections and extended endplate connections used in composite steel beam-CFST column joints with slab are similar, due to the effect of the slab reinforcement that carries the tensile force of the joint. Therefore, flush endplate connections can be adopted for the composite steel beam-CFST column joints. This type of connection is commonly chosen in the design of composite frame building structures because they are economical, easily fabricated and have a good structural performance.
2.5 Bolting Systems for Endplate Connections of CFST column Joints

Either a welding system or a bolting system can be used to connect the steel beam to the CFST column. Bolting is more convenient than welding during the construction of connections. Bolting is suitable when bolts and nuts are used to connect to an open steel section, but it is very difficult to make the connection to a CFST section using normal bolts, because there is no access to the inside of the CFST section for tightening the connecting bolts. To avoid the need to gain access to the inside of CFST section, blind bolts have been developed by commercial companies. Blind bolts are structural bolts that can be installed from the outside of the steel tube. Four major bolting systems are available for connecting the steel beam to the steel tube of the CFST column: Flow-drill system, Huck blind-bolt system, Ajax ONESIDE bolt system and Lindapter Hollo-bolt system (Figure 2.11). Detailed information about these types of bolts is given in the following sub-sections.

![Flow drill system](image1)

(a) Flow drill system

![Huck blind-bolt system](image2)

(b) Huck blind-bolt system

![Ajax ONESIDE bolt system](image3)

(c) Ajax ONESIDE bolt system

![Lindapter Hollo-bolt system](image4)

(d) Lindapter Hollo-bolt system

*Figure 2.11 Available bolting systems for connecting the steel beam to CFST column*

2.5.1 Flow-drill system

The Flow-drill system as shown in Figure 2.11(a) is a thermal drilling process that uses a tungsten carbide bit rotating at high speed, making a hole through the wall of
structural hollow tube section without removing material. The hole formed in this way is then threaded using a roll thread-forming tool. A standard structural bolt can then be used to connect the steel beam to the steel tube. The Flow drill connector system involves a fully threaded bolt holding components together without nuts being required. This system allows members to be bolted to the face of a structural hollow tube section or CFST section (British Steel Tubes and Pipes, 1996 & 1997; Kurobane et al., 2004; Park and Wang, 2010; Lee, 2011). Flow drill is suitable for both hot-finished and cold-formed structural hollow tube sections from 5.0 to 12.5 mm thick. Only limited research has been conducted by France et al. (1999a & 1999b) to investigate the strength and rotational stiffness of simple joints using Flow drill connectors. No further research was conducted due to the complicated bolt installation system which requires a thermal drilling process.

2.5.2 Huck blind-bolts
Huck blind bolt as shown in Figure 2.11(b) is a type of blind fastener that overcomes the difficult access problem inside hollow tube sections. Two types of blind fastener (Huck high-strength blind bolt and Huck Ultra-Twist bolt) have been developed by Huck International Inc (www.huck.com/industrial). The tension, shear and bearing capacities of Huck blind bolts are equivalent to the capacity of ISO grade 8.8 of M16, M20 and M24 structural bolts, requiring minimum thicknesses of 10.4 mm, 13.4 mm and 14.4 mm respectively for structural hollow tube sections. These thicknesses are normally higher than those required by Flow drill method. Huck blind bolts are not applicable in low-rise structures, where the thickness of structural hollow tube section is less than 10 mm (Lee, 2011). No further research using Huck blind bolts was conducted due to the thickness limitation of the steel tube of the CFST columns.

2.5.3 Ajax ONESIDE bolts
Ajax ONESIDE bolts are structural blind fasteners consisting of a bolt with circular head, internal collapsible stepped washer, external solid stepped washer, shear sleeve and standard bolt nut as shown in Figure 2.11(c). The bolt is installed simply and effectively using a special installation tool. The bolt components are initially placed on the installation tool and then the collapsible washer is folded in the thin area of the installation tool. The bolt and folded collapsible washer are inserted through a predrilled oversized hole. After removing the installation tool, a rattle gun is used to
tighten the nut to the desired tension. The full tension capacity of structural bolts as per AS 4100 (see Table 2.3) can be achieved using Ajax ONESIDE bolt. Research on Ajax ONESIDE bolted endplate connections has been conducted by Gardner and Goldsworthy (2005), Goldsworthy et al. (2006), Lee et al. (2010, 2011), Mirza and Uy (2011), and Yao et al. (2008, 2009 & 2011). To improve and provide a better solution in the design of CFST connection using Ajax ONESIDE bolt, Goldsworthy and Gardner (2006) proposed the use of a cogged extension welded to the head of the Ajax ONESIDE bolt. This improves the behaviour of blind-bolted connections, since the load is transferred between the steel tube wall and cogged anchorage with the concrete. Although, excessive local deformation of the tube wall is avoided using this technique, the presence of the extension greatly counteracts the ease of installation that blind bolts provide.

Table 2.3 Specification of Ajax ONESIDE bolt size 16 to 24 mm as per AS4100 (Yao and Goldsworthy, 2009)

<table>
<thead>
<tr>
<th>Bolt diameter (mm)</th>
<th>Bolt head diameter (mm)</th>
<th>Head thickness (mm)</th>
<th>Washer diameter</th>
<th>Sleeve diameter</th>
<th>Hole diameter</th>
<th>PC8.8 Proof load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Outside (mm)</td>
<td>Inside (mm)</td>
<td>Outside (mm)</td>
<td>Inside (mm)</td>
</tr>
<tr>
<td>16</td>
<td>23</td>
<td>9</td>
<td>32</td>
<td>17</td>
<td>23</td>
<td>16</td>
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<td>24</td>
<td>34</td>
<td>13</td>
<td>51</td>
<td>25</td>
<td>34</td>
<td>24</td>
</tr>
</tbody>
</table>

2.5.4 Lindapter Hollo-Bolts

Lindapter Hollo-Bolts (known as Hollo-Bolts) are a pre-assembled unit having a standard grade 8.8 bolt, collar, sleeve or main body, threaded truncated cone and rubber washer as shown in Figure 2.11(d). The pre-assembled unit is inserted through predrilled holes in the endplate and hollow tube section (Figure 2.12(b) (i)). The configuration of the Hollo-Bolt, after tightening, differs from other blind bolts or standard bolts, mainly in the inner part of the bolt, especially the nut area. When tightened using a torque wrench as shown in Figure 2.12(b) (ii), a threaded truncated cone forces the base of the sleeve to expand to form a secure clamp against pull-out, see Figure 2.12(b) (iii). Information from Lindapter International plc at www.lindapter.com about the M16 and M20 grade 8.8 Hollo-Bolts is summarised in Table 2.4.
Table 2.4 Specifications of M16 and M20 grade 8.8 Hollo-Bolts (Lindapter)

<table>
<thead>
<tr>
<th>Product code</th>
<th>Bolt size</th>
<th>Clamping thickness, W (mm)</th>
<th>Sleeve Length, L (mm)</th>
<th>Outer diameter, d (mm)</th>
<th>Height, H (mm)</th>
<th>Outer diameter, D (mm)</th>
<th>A/F</th>
<th>Tightening torque, T (N-m)</th>
<th>Safe working load, TL (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HB16-1</td>
<td>M16</td>
<td>12–29</td>
<td>41.5</td>
<td>63</td>
<td>25.75</td>
<td>8</td>
<td>38</td>
<td>36</td>
<td>190</td>
</tr>
<tr>
<td>HB16-2</td>
<td>M16</td>
<td>29–50</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>HB16-3</td>
<td>M16</td>
<td>50–71</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>HB20-1</td>
<td>M20</td>
<td>12–34</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>HB20-2</td>
<td>M20</td>
<td>34–60</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>HB20-3</td>
<td>M20</td>
<td>60–68</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 2.12 Dimensions and installation procedure of Hollo-Bolts (www.lindapter.com)

Tizani and Ridley-Ellis (2003) and Loh et al. (2006b) conducted a series of tests to investigate the behaviour of flush endplate connections to CFST columns with Hollo-Bolts. The test results showed that Hollo-Bolts demonstrated good performance in flush endplate connections. Extensive research on flush endplate connections using Hollo-Bolts has also been reported by other scholars (Tizani and Ridley-Ellis, 2003; Ellison...
and Tizani, 2004; Wang et al., 2009a,b; Wang and Chen, 2012, Tizani et al., 2013a, 2013b). The results show that excellent strength and rotational capacity was achieved using Hollo-Bolts when used in flush endplate connections between the steel beam and CFST column. To improve the performance of the Hollo-Bolts, Tizani and Ridley-Ellis (2003) modified commercially available Hollo-Bolts and proposed the Extended Hollo-Bolts shown in Figure 2.13.

The Extended Hollo-Bolts provided greater stiffness and ultimate load capacity in the flush endplate connections, but the ductility was observed to be lower than that for Hollo-Bolted flush endplate connections due to the failure of the bolt shank (Tizani and Ridley-Ellis, 2003; Tizani et al., 2004).

![Diagram of Extended Hollo-Bolts](image)

(a) Before tightening  (b) After tightening

Figure 2.13 Extended Hollo-Bolts (Tizani et al., 2013a)

### 2.6 Experimental Work of CFST Column Joints with Blind-Bolted Connections

Ajax ONESIDE bolts and Hollo-Bolts are widely used in blind-bolted endplate connections for connecting steel beams to CFST columns and to avoid the need for welding at construction site. Extensive experimental tests have been carried out on steel beam-CFST column joints without slab using these two types of blind-bolts in endplate connections, but few tests have been conducted on composite steel beam-CFST column joints when a slab is present. But the fact is that such joints without slab behave differently from composite steel beam-CFST column joints with slab. Recent research
on steel beam-CFST column joints without and with slab is discussed in sub sections 2.6.1 and 2.6.2. Typical examples of CFST column joints without slab and with slab are shown in Figure 2.14(a) and (b).

![Diagram of CFST column joints](image)

(a) CFST column joints without slab  (b) CFST column joints with slab

Figure 2.14 Typical CFST column joints with Hollo-Bolted endplate connections

### 2.6.1 CFST column joints without slab

Research on blind-bolted endplate connections and steel beam-CFST column joints without slab is reviewed in this section. Most reports in the literature refer to blind-bolted endplate connections for steel structures and composite building structures. The tests were conducted either in pure tension or in combined shear and bending.

T-stub connections were first used by Zoetemerijer (1974) to investigate the tensile behaviour of the column flange and endplate of bolted endplate connections. The bending behaviour of the column flange and endplate was also investigated using bolted T-stub connections (Barnett, 2001). The behaviour of T-stub connections with the square hollow section (SHS) columns was investigated by Barnett (2001) who used Hollo-Bolts and Lee (2011) who used Ajax ONESIDE bolts. Barnett (2001) carried out experimental study of different types of blind bolts used on the T-stub connection and found that the stiffness of the T-stub connection with Hollo-Bolts was lower than that of the T-stub connection using normal bolts.
Wang et al. (2009a, b) conducted a full-scale experimental study of composite joints to connect square or circular CFST columns to a square steel tube (200 × 200 × 8 mm) or circular steel tube (200 × 8 mm), and investigated the behaviour of flush endplate joints under monotonic loading and cyclic loading. The two parameters that were varied in their analysis were column section type and endplate thickness. The behaviour of the flush endplate connections under monotonic loading was evaluated in terms of stiffness, moment capacity and ductility. The test results showed that this type of connection, which behaved as a semi-rigid and partial strength manner according to the Eurocode 3 specifications, displayed reasonable strength and stiffness (Wang et al., 2009b). The rotational capacity of this type of connection to square or circular tube exceeded 70 mrad which satisfies the ductility requirements for earthquake-resistance in most seismic regions. It was also reported that the outward deformation of the tube wall at the location of the top tension bolts was very locally concentrated for the square CFST column flush endplate connection. Wang et al. (2009a) reported that the steel tube wall of the circular CFST column was undamaged. This was due to the good resistance of circular sections to local buckling.

The studies on blind-bolted connection to square CFST column conducted by Wang et al. (2009a, b) also indicated that the strength and stiffness of the connections were limited by pulling-out of blind bolts due to the outward deformation of the steel tube of the CFST column. To resolve these issues, welding cogged extensions to blind bolts were proposed by Goldsworthy and Gardner (2006) and the use of extended bolt shanks was introduced by Tizani and Ridley-Ellis (2003). To avoid the need to modify commercially available blind bolts, alternative solutions are still being sought. Such solutions should enhance the integrity of the joint panel zone and reduce the outward deformation of the steel tube in CFST column.

2.6.2 CFST column joints with slab

A small number of tests have been conducted on steel beam-to-CFST column joints with slab in place to investigate the behaviour of blind-bolted flush endplate connections by Loh et al. (2006b) and Mirza and Uy (2011). Conversely, many tests have been conducted on steel beam-to-steel column joints with slab to investigate the behaviour of normal bolted flush endplate connections, for example by Anderson and Najafi (1994), Xiao et al. (1994), Li et al. (1996a), Liew et al. (2000), Brown and
Loh et al. (2006b) and Mirza and Uy (2011) used compact CFST columns with a width-to-thickness ratio of 22.22 (Loh et al., 2006b) and 25 (Mirza and Uy, 2011), respectively, for the square hollow steel tubes.

Loh et al. (2006b) tested five joint specimens to investigate the behaviour of the joints for the differently spaced shear connectors (265, 480 and 800 mm) and different slab reinforcement ratios (0.65, 1.29 and 1.98%) using the novel approach of blind-bolting (Hollo-Bolt M20, grade 8.8). The moment capacity of the joints was increased by using the partial shear connection and by increasing the slab reinforcement ratio. It was observed that when the slab reinforcement ratio was 0.65%, the joint failed due to the fracture of the slab reinforcement. Other joints failed due to the beam local buckling when slab reinforcement ratio was used 1.29% and 1.98%. In Table 2.5, joints failed due to the fracture of the slab reinforcement and the beam local buckling (noted as ‘B’ and ‘E’ respectively in Table 2.5).

Table 2.5 Previous experimental studies of flush endplate steel beam-to-CFST column composite joints

<table>
<thead>
<tr>
<th>Test conducted by</th>
<th>Test specimen No</th>
<th>$A_c = \left( \frac{b_e \times D_c}{D_c} \right)$ (mm)</th>
<th>$A_r \times 100 %$</th>
<th>$N_{sc}$</th>
<th>$p_0$ (mm)</th>
<th>$\eta_y$</th>
<th>$M_u$ (kN-m)</th>
<th>$\phi_u$ (mrad)</th>
<th>Slip (mm)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loh et al. (2006)</td>
<td>CJ1</td>
<td>2.07 × 120</td>
<td>1.29</td>
<td>5</td>
<td>100</td>
<td>1.1</td>
<td>185.5</td>
<td>30</td>
<td>0.5</td>
<td>E</td>
</tr>
<tr>
<td></td>
<td>CJ2</td>
<td>2.07 × 120</td>
<td>1.29</td>
<td>3</td>
<td>140</td>
<td>0.66</td>
<td>187.9</td>
<td>38</td>
<td>4.5</td>
<td>E</td>
</tr>
<tr>
<td></td>
<td>CJ3</td>
<td>2.07 × 120</td>
<td>1.29</td>
<td>2</td>
<td>300</td>
<td>0.44</td>
<td>178.9</td>
<td>45</td>
<td>7.4</td>
<td>E</td>
</tr>
<tr>
<td></td>
<td>CJ4</td>
<td>2.07 × 120</td>
<td>0.65</td>
<td>3</td>
<td>140</td>
<td>1.33</td>
<td>143.3</td>
<td>21</td>
<td>1.1</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td>CJ5</td>
<td>2.07 × 120</td>
<td>1.94</td>
<td>8</td>
<td>100</td>
<td>1.18</td>
<td>192.1</td>
<td>19</td>
<td>0.2</td>
<td>B, E</td>
</tr>
<tr>
<td>Mirza and Uy (2011)</td>
<td>CJ1</td>
<td>1000 × 120</td>
<td>1.45</td>
<td>4</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
</tbody>
</table>

Note: $\rho = \frac{A_r}{A_c} \times 100 \%; \eta_y = \frac{N_{sc}F_{sc,max}}{A_{fr,y}}; F_{sc,max} = A_{sc}f_{sc,y}$

Mirza and Uy (2011) tested two joint specimens under static monotonic loading and quasi-static cyclic loading. They reported that although both joints had the same configuration, the ultimate hogging moment capacity of the joint subjected to static monotonic loading was higher than that of the joint tested under quasi-static cyclic loading. This indicated that the behaviour of composite steel beam-square CFST column joints with slab can vary depending on the type of loading. It was also observed for the joint tested under quasi-static cyclic loading that the ultimate hogging moment
capacity of the joint was higher than the ultimate sagging moment capacity of joint. Two possible reasons were suggested: the pulling out of the bottom bolts from the concrete core, and the separation of the steel tube from the concrete, near the lower bolts. Test results found in the literature for CFST column joints with slab are summarised in Table 2.5.

2.7 Finite element (FE) Modelling of CFST Column Joints with Blind-Bolted Connections

Finite element (FE) analysis has been proven to be a reliable, acceptable and comparatively accurate method for investigating structural properties. In general, FE models of bolted endplate connections are often developed using commercial software package such as ABAQUS, ANSYS, LUSUS, LS-DYNA and DIANA. Of those, ABAQUS and ANSYS software have been widely used for numerical analysis because of their flexibility in modelling different components in composite joints. The results of FE modelling of composite joints using blind-bolted endplate connections, depends on correctly selecting the element type, geometry modelling such as blind bolts, contact interaction, material modelling and the analysis algorithm.

FE modelling of Hollo-Bolted endplate connections of the steel beam to CFST column comprises geometrical modelling of the steel tube, concrete core, steel beam, endplate, blind bolts and other components. In modelling the geometry of Hollo-Bolted endplate connections, the most critical part is the Hollo-Bolt modelling, because the Hollo-Bolt is quite different from other types of blind bolts (i.e. Ajax ONESIDE bolts) and normal standard bolts. In past studies, numerical analysis for blind-bolted endplate connection to a CFST column has been conducted by Mirza and Uy (2011), Wang and Spencer (2013), Ataei and Bradford (2013) and Tizani et al. (2013a). Mirza and Uy (2011) modelled Ajax ONESIDE bolts in the endplate connections as normal standard bolts because their geometries are similar. Wang and Spencer (2013) modelled Hollo-Bolts as normal standard bolts, as shown in Figure 2.15 (a). Ataei and Bradford (2013) proposed a simplified model for Hollo-Bolts, as shown in Figure 2.15 (b & d). Tizani et al. (2013a) proposed Extended Hollo-Bolts and modelled them as shown in Figure 2.15(c). The details of the FE model of the Hollo-Bolt reported by Hassan et al. (20014) are shown in Figure 2.15(e).
2.7.1 Bolt pretention forces

Bolts are normally tightened by applying torque to the head and/or nut. This stretches the bolts and applies tensile force (the so-called tightening force, tension force or pretension force). The final tightening force depends on the friction coefficients of the threads of the nut and bolt, and on contact surfaces between the bolt head and nut with the flange. In any tightening method, it should be ensured that the stress generated in the bolt never exceeds the elastic limit or yield point. It is recommended in BS 3692 (BSI2001) that the stress should be below the maximum allowable tensile stress, which is 70% of the minimum tensile strength of the bolt. The tightening force in a bolting system is a function of tightening torque ($T$), bolt diameter ($d_n$) and nut factor ($K$).

The basic equation to calculate the preload for a standard steel fastener is

$$F_p = \frac{T}{Kd_n}$$

The value of $K$ for standard steel fasteners is recommended as 0.15 to 0.25. Equation (2.2) can also be used to calculate the pre-load of Hollo-Bolts. Pitrakkos (2012) found
that the nut factor \((K)\) for Hollo-Bolts is larger than that normally applied in a standard bolting system, and proposed a value of \(K = 0.443 (1/2.26)\) based on test data and regression analysis. The recommended tightening torque during the installation of M16 and M 20 bolts is summarised in Table 2.6.

<table>
<thead>
<tr>
<th>Bolt type</th>
<th>Bolt grade</th>
<th>Bolt diameter</th>
<th>Tightening torque (N·m)</th>
<th>Pre-tension force (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Standard bolt</strong></td>
<td>Grade 8.8</td>
<td>M 16</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>M 20</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Grade 10.9</td>
<td>M 16</td>
<td>244(^b)</td>
<td>85(^b)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>M 20</td>
<td>315(^b)</td>
<td>110(^b)</td>
<td></td>
</tr>
<tr>
<td><strong>Hollo-Bolt</strong></td>
<td>Grade 8.8</td>
<td>M 16</td>
<td>190(^a)</td>
<td>27(^c)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>M 20</td>
<td>300(^a)</td>
<td></td>
</tr>
<tr>
<td>Grade 10.9</td>
<td>M 16</td>
<td>244(^b)</td>
<td>85(^b)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>M 20</td>
<td>315(^b)</td>
<td>110(^b)</td>
<td></td>
</tr>
<tr>
<td><strong>Extended Hollo-Bolt</strong></td>
<td>Grade 8.8</td>
<td>M 16</td>
<td>190(^a)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>M 20</td>
<td>300(^a)</td>
<td></td>
</tr>
<tr>
<td>Grade 10.9</td>
<td>M 16</td>
<td>300(^a)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>M 20</td>
<td>300(^a)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: Tests conducted by \(a:\) Lindapter; \(b:\) Liu et al. (2012a)

### 2.7.2 Material properties
Hollo-Bolts are fabricated from two types of materials: one is structural bolt material and another is mild steel material. The grade 8.8 and grade 10.9 are widely used as structural bolt materials in Hollo-Bolts to fabricate the bolt head, collar, cone and bolt shank. Mild steel material is used to fabricate the sleeve. The Hollo-Bolt material test results collected from the literature are summarised separately for bolts material (Grade 8.8 and 10.9) and sleeve material in Table 2.7. It was found from the coupon test results that the yield stress of bolt shank varies from 725 to 950 MPa for 8.8 Grade bolt and 785 to 1086 MPa for 10.9 Grade bolt; and for sleeve, it varies from 382 to 450 MPa.
<table>
<thead>
<tr>
<th>Coupon material</th>
<th>Bolt type</th>
<th>Bolt grade</th>
<th>Bolt diameter</th>
<th>Yield stress (MPa)</th>
<th>Ultimate tensile strength (MPa)</th>
<th>Young’s modulus (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard bolt</td>
<td>Grade 8.8</td>
<td>M 16</td>
<td>851.35&lt;sup&gt;e&lt;/sup&gt;</td>
<td>925.92&lt;sup&gt;e&lt;/sup&gt;</td>
<td>208115&lt;sup&gt;e&lt;/sup&gt;</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td></td>
<td>M 20</td>
<td>660&lt;sup&gt;g&lt;/sup&gt;</td>
<td>830&lt;sup&gt;g&lt;/sup&gt;</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td>Grade 10.9</td>
<td>M 16</td>
<td>940&lt;sup&gt;g&lt;/sup&gt;</td>
<td>1040&lt;sup&gt;g&lt;/sup&gt;</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td></td>
<td>M 20</td>
<td>940&lt;sup&gt;g&lt;/sup&gt;</td>
<td>1040&lt;sup&gt;g&lt;/sup&gt;</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Bolt shank</td>
<td>Grade 8.8</td>
<td>M 16</td>
<td>752.33&lt;sup&gt;c&lt;/sup&gt;</td>
<td>1918.18&lt;sup&gt;c&lt;/sup&gt;</td>
<td>194204&lt;sup&gt;c&lt;/sup&gt;</td>
<td>–</td>
</tr>
<tr>
<td>Hollo-bolt (HB)</td>
<td></td>
<td>M 20</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td>Grade 10.9</td>
<td>M 16</td>
<td>449.40&lt;sup&gt;c&lt;/sup&gt;</td>
<td>548.33&lt;sup&gt;c&lt;/sup&gt;</td>
<td>208830&lt;sup&gt;c&lt;/sup&gt;</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td></td>
<td>M 20</td>
<td>390&lt;sup&gt;f&lt;/sup&gt;</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td>Grade 8.8</td>
<td>M 16</td>
<td>382&lt;sup&gt;a&lt;/sup&gt;</td>
<td>509&lt;sup&gt;a&lt;/sup&gt;</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td></td>
<td>M 20</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td>Grade 10.9</td>
<td>M 16</td>
<td>382&lt;sup&gt;b&lt;/sup&gt;</td>
<td>512&lt;sup&gt;b&lt;/sup&gt;</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td></td>
<td>M 20</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
</tbody>
</table>

Note: Material tests conducted by a, b: Liu et al. (2012a, 2012b); c: Tizani et al. (2013b); d: Pitrakkos and Tizani et al. (2013c); e: Rahman (2012); f: Lindapter; g: novolts

### 2.8 Analytical Model for Blind-Bolted Connections to CFST Column

Three mechanical properties (strength, stiffness and rotation capacity) of a joint are widely used to evaluate its behaviour. Analytical models are often developed based on the component method to accurately reflect the mechanical properties of joints. Most
analytical models have been developed for composite steel beam-to-steel column joints. These models have been used blindly for composite steel beam-to-CFST column joints, despite the fact that the behaviours of CFST columns and steel columns are known to be quite different. Figure 2.16 shows the different components used in mechanical models of composite joints (Eurocode 3, Yao et al., 2006).

![Diagrams showing different components used in mechanical models of composite joints](image)

Figure 2.16 Different components used in mechanical models of composite joints

It can be seen that the main difference is in the calculation of the component stiffness values of composite joints. In both cases, the stiffness values of the slab reinforcement ($k_r$) and shear connector ($k_s$) can be calculated from the expressions proposed for the flush endplate steel beam-to-steel column composite joints. But for the current joints to CFST columns, the effects of the utilisation of blind bolts and CFST columns on $k_b$ (stiffness of the component in top bolt row level) and $k_c$ (stiffness of the components in bottom beam flange level) should be considered.
A few widely used mathematical models are found in the literature for predicting the rotational stiffness of flush endplate connections for steel beam-to-steel column composite joints. These were proposed by Aribert and Lachal (1992), Benussi and Noe (1994), Anderson and Najafi (1994), Ren and Crisinel (1996), Ahmed and Nethercot (1997b), Liew et al. (2000) and Al-Aasam (2013). The mathematical expressions for the rotational stiffness ($S_{j,c}$) proposed by Aribert and Lachal (1992); Anderson and
Najafi (1994); Ren and Crisinel(1996); Ahmed and Nethercot (1997b); Liew et al. (2000) and Al-Aasam (2013) are given in Equation 2.3 to 2.8 respectively.

Aribert and Lachal (1992):
\[ S_{j,c} = S_{j,s} + \frac{D_r^2}{\frac{1}{k_r} + \frac{\alpha}{N_{sc}k_{sc}} \left( \frac{D_r}{H_b} \right)} \] (2.3)

Anderson and Najafi (1994):
\[ S_{j,c} = k_bD_r^2 + \frac{D_rH_b}{\frac{1}{k_r} + \frac{1}{k_s}} \] (2.4)

Ren and Crisinel (1999):
\[ S_{j,c} = \frac{D_r^2}{\frac{1}{k_r} + \frac{1}{k_s} + \frac{1}{k_c}} \] (2.5)

Ahmed and Nethercot (1997b):
\[ S_{j,c} = \frac{D_rH_b \left( \frac{1}{k_b} + \frac{1}{k_c} \right) + D_b^2 \left( \frac{1}{k_r} + \frac{1}{k_s} + \frac{1}{k_c} \right) - D_b (H_b + D_r) \frac{1}{k_c}} {\left( \frac{1}{k_r} + \frac{1}{k_s} + \frac{1}{k_c} \right) \left( \frac{1}{k_b} + \frac{1}{k_c} \right) - \frac{1}{k_r^2}} \] (2.6)

Liew et al. (2000):
\[ S_{j,c} = S_{j,s} + \frac{\left( k_rh_r^2 + N_{sc}k_{sc}h_s^2 \right)^2}{\frac{1}{N_{sc}k_{sc}} + \frac{1}{k_r}} \] (2.7)

Al-Aasam (2013):
\[ S_{j,c} = \frac{D_b^2k_bk_c}{(k_b + k_c)} + \frac{\left( D_r - D_b \left( \frac{k_b}{k_b + k_c} \right) \right)^2}{\left( \frac{1}{k_r} + \frac{1}{k_s} + \frac{1}{k_b + k_c} \right)} \] (2.8)

Aribert and Lachal (1992) proposed an expression based on assumptions of the moment resistance of the steel connections and moment resistance of slab. Benussi and Noe (1994) established a simple spring model without considering the slip of the shear connectors as shown in Figure 2.17(a). The effect of the slip of the shear connectors were taken into account by including an axial spring (\(k_s\)) at the level of the slab–beam interface. Anderson and Najafi (1994) proposed a spring model, but the column web deformation due to compression at the level of the bottom flange of the beam was not incorporated in their spring model, as shown in Figure 2.17(b). Ren and Crisinel (1996) developed a mathematical expression by considering the effect of slab (\(k_r\)), shear
connectors \((k_s)\) and column web deformation \((k_c)\). This model did not consider the
stiffness effect \((k_b)\) of the steel components at the level of the top row of bolts. To
overcome all of the above limitations, Ahmed and Nethercot (1997b) proposed a simple
mechanical model in which effects of slab \((k_r)\), shear connectors \((k_s)\), column web
deformation \((k_c)\) and the steel components in the top row of bolt \((k_b)\) were
incorporated to derive the spring model as shown in Figure 2.17(c). This model is
widely used to predict the rotational stiffness of flush endplate connections for steel
beam-to-steel column composite joints, however, the effect of shear deformation of the
column panel zone was not incorporated into their spring model, and even their
mathematical expression did not take into account the fixed moment required to keep
the rigid bars which connects the stud spring with reinforcement spring (Al-Aasam,
2013).

Most recently, Al-Aasam (2013) proposed a mathematical model in which the fixed
moment mechanism was incorporated to connect the stud spring with reinforcement
spring. The effect of shear deformation in the column panel zone was also considered,
as shown in Figure 2.17(d). All of these mathematical models are derived based on
experimental test data and finite element (FE) model results of the steel beam-steel
column composite joints in which a steel beam was connected to a steel column using
normal bolts and endplate.

### 2.9 Summary of Research Gaps

A review of the literature shows that the use of the steel beams and CFST columns in
the building construction has been increasing in recent years. The behaviour of CFST
columns has been extensively investigated, indicating that CFST columns are able to
provide high strength, large stiffness and ductility, and can resist the local inward
buckling of the steel tube due to the infill concrete core. However, the connections
between the steel beams and CFST columns are more complicated due to the use of thin
steel tubes in CFST columns. Steel beams are connected to the thin steel tube by
welding or bolting. Bolting systems are more convenient than welding system for
constructing connections between a steel beam and a CFST column.
Welded connections are widely used as rigid connections in which the steel beams are welded to the thin steel tube of CFST column. External and/or internal diaphragms are required to overcome the outwards deformation of the steel tube. The use of these types of welded connections is more complicated in the construction field, and very expensive.

In bolting systems, endplate connections (known as semi-rigid connections) are widely used to connect the steel beam to the CFST column. The endplate is welded to the beam, and then bolted with blind bolts to the steel tube. Blind bolts usually differ from conventional or normal bolts. It is very difficult to make the connection to a CFST section using normal bolts because there is no access to the inside of the CFST section to tighten the bolt. Blind bolts are structural bolts that are capable of being installed from the outside of the steel tube of the CFST column. Ajax ONESIDE bolts and Hollo-Bolts are widely used in flush endplate connections to connect the steel beam to CFST column, which eliminates the need for on-site welding.

Flush endplate connections using blind bolts are becoming more commonly used to connect the steel beam to the CSFT column, due to their ease of installation in the construction field and the save in construction cost and time that this produces. Extensive research has been conducted on the blind-bolted flush endplate connections in the past, and it has been reported that this type of connections gives a good structural performance with regards to stiffness, ductility and strength of the joints. However, most studies and testing of blind-bolted connections with CFST columns have been conducted without considering how the presence of a slab affects the joint. The behaviour of composite steel beam-CFST column joints incorporating the slab effect is quite different from that of the non-composite joints where there is no slab. In composite joints, the tensile forces are carried by the reinforcing bars in the slab, whereas in non-composite joints, the tensile forces are carried by the upper tensile bolts and the steel tube. In addition, the failure mode and strength capacity of composite steel beam-CFST column joints with slab are both significantly varied from those of the non-composite steel beam-CFST column joints without slab.

Several tests on composite steel beam to steel column joints have previously conducted to investigate the behaviour of flush endplate connections, but very few were on
composite steel beam-CFST column joints with slab. In tests of composite steel beam to steel column joints, conventional bolts M20, grade 8.8 were used. The behaviour of these is not similar to that of composite steel beam-CFST column joints with blind-bolted endplate connections, due to the presence of the CFST column and the geometry of blind bolts (especially Hollo-Bolts). Existing analytical methods for composite joints are limited to steel beam-to-steel column joints with flush endplate connections. These methods cannot be blindly used in the design of composite steel beam-CFST column joints with blind-bolted endplate connections.

From these considerations, it seems imperative that more research on composite steel beam-CFST column joints using blind-bolted endplate connections be conducted to provide vital information regarding the use of CFST columns in composite building structures.
CHAPTER 3

EXPERIMENTAL PROGRAM AND MATERIAL TESTING

3.1 Introduction

This chapter provides details of the experimental program and materials for the hybrid composite steel beam-CFST column joints. For this program, two types of hybrid composite steel beam-CFST column joints with slab were designed using blind-bolted endplate connections and through-plate connections. These were tested in conditions of monotonic static loading and cyclic loading. The hybrid stainless-carbon steel composite beam-column joint consists of three materials such as stainless steel, carbon steel and concrete. Stainless steel was used in the steel tube of the CFST column, and carbon steel material was used for the steel elements of the joint components. Profile steel sheeting concrete slab was cast over the steel beams, and connected to the steel beams by shear connectors to provide composite action.

To develop the test program, a frame analysis was performed for the composite steel beams, the CFST columns and the blind-bolted connections. Following analysis and design of the composite frame structures, one joint was selected as the prototype of the test specimens to develop the test program. Due to the limitations imposed by the capacity of the testing machine, the selected joint was scaled down accordingly. After planning of the test program was completed and before testing of joint specimens began, the mechanical properties of each component of the hybrid stainless-carbon steel composite beam-column joint were investigated. The frame analysis, test specimen details, and material properties are illustrated step by step below.

3.2 Frame Analysis for Selection of Joints

Test specimens were designed to reflect the real joints of the composite 39-storey frame-structure building shown in Figure 3.1(a), which was analysed using SAP2000 software. The total height of the frame was 146.2 m (Floor-to-floor height was 3.1 m
for first three floors, 5.4 m for next four floors and 3.6 m for upper 32 floors). A plan view and elevation view of the frame are shown in Figure 3.1(b) and (c).

Figure 3.1 Configuration of semi-rigid frame of hybrid composite steel beam and CFST column based 39-storey building structure
The concrete-filled steel tubular (CFST) columns and steel beams were considered to build the composite frame structure of the building. Square CFST columns were considered throughout the analysis. Blind-bolted endplate connections were adopted in the frame analysis of the building for connecting the steel beams to the CFST columns. Blind-bolted endplate connections are recognised as semi-rigid connections.

![Flowchart for analysis and design of semi-rigid frame](image-url)

Figure 3.2 Flowchart for analysis and design of semi-rigid frame
The frame analysis of the building structure was performed using SAP200 software on the assumptions of rigid connections and semi-rigid connection. The analysis of the frame with rigid connections was very straightforward compared to the frame with semi-rigid connections, since there is no requirement to define the connection properties, such as stiffness, of the rigid connections in the rigid frame analysis. On the other hand, the properties of semi-rigid connections were required for the semi-rigid frame. The key parameter for the semi-rigid frame was the connection stiffness, which was defined by considering the rotational stiffness of the connections. The rotational stiffness value of each connection is determined using the component method based on the connecting components of the joints. Ahmed and Nethercot (1997b) proposed a mathematical expression for composite steel joint using the component method (details are given in Eq. (2.6) in Chapter 2). This equation was adopted in this section to calculate the rotational stiffness of the blind-bolted endplate connections. The loading combinations which were considered in the composite frame structure of the building are given in Appendix A.3.

The analysis procedure for a semi-rigid hybrid composite steel beam and CFST column frame building structure is illustrated step by step in Figure 3.2. Firstly a rigid frame analysis was performed and all connecting members such as composite steel beams and CFST columns were designed. On the basis of these preliminary designed sections of composite steel beams and CFST columns, semi-rigid connections were designed to resist the joint forces that were found in the rigid-frame analysis for the critical joints of each level of the 39-storey building. The rotational stiffness of the semi-rigid connections were then determined for the components of the joints (blind-bolts and endplate) and the connecting members (composite steel beam and CFST column) using the component method. The rotational stiffness values of the connections were used as input to the frame analysis software SAP2000; to represent the behaviour of the semi-rigid connections. After calculating the rotational stiffness of each joint in the whole frame, the frame was re-analysed as a semi-rigid structure, using the stiffness of the semi-rigid connections. The beam and column sizes adopted in the semi-rigid frame analysis are given in Appendix A. The designed resisting capacities of the composite steel beams and the CFST columns were then checked against design code AS4100. After the analysis and design of the semi-rigid frame were completed, one particular
joint (comprising steel beam (460UB67.1) and CFST column (600 × 600 × 12 mm)) as shown in Figure 3.1(d) was selected as the prototype joint for developing the test program.

3.3 Experimental Program

Due to the limitation imposed by the capacity of the testing machine, the sizes of the CFST column (600 × 600 × 12 mm) and the composite beam (460UB67.1) of the prototype joint were scaled down based on the line stiffness ratio of the column and beam. After reducing the sizes, the line stiffness ratio of the scaled-down test specimens was 0.45. The sections of test specimen SB1-1 (the prototype for all test specimens) were: square CFST column (360 × 360 × 6 mm), composite beam (310UB40.4), and composite slab (900 × 120 mm) with profile sheet. The column was 2200 mm high, the steel beam was 1320 mm long, and the reinforced concrete slab was 120 mm thick. The ultimate shear force was 385 kN and moment capacity 270 kN.m, calculated in accordance with Eurocode 3 (2005). The design details are given in Appendix B.

To compare blind-bolted and through-plate connections, through-plate connections were also tested. The basic sections of all test specimens were as for prototype specimen SB1-1. All calculations for blind-bolted connections and through-plate connections were done in accordance with Eurocode 3 (2005). The calculation details for both types of joints are given in Appendix B.

Blind-bolted connection test specimens were constructed using an endplate, blind bolts and with or without binding bars. Binding bars were used only for the blind-bolted endplate connections with square CFST column, and were omitted in circular CFST column tests. Endplates were welded to the beams. Blind bolts connected the endplate to the steel tube of the CFST column. Binding bars are normal steel bars welded to the steel tube of the column to connect opposite faces of the steel tube. They also act as a stiffener to provide the composite action between the steel tube and concrete core of the CFST column.

Through-plate connections were constructed by the combing the full-through plate and half-through plate, inserted and welded to the steel tube of the CFST column and bolted
to the steel beam web. For the composite slab, a steel profile sheet (bondek profile sheet) were cast over the steel beams, and connected to them by shear connectors to produce a composite action between the steel beam and the slab.

The experimental details, including configuration and fabrication of test specimens, test setup, instrumentation and testing procedure are given in the following subsections.

3.3.1 Configuration of test specimens

A total of 10 joint specimens were designed and fabricated for the experimental program. These were divided into three series (Series-SB1, Series-CB2 and Series-ST3) for blind-bolted endplate connections and through-plate connections. The parameters investigated in the tests are listed in Table 3.1. These parameters were: (i) effect of the presence of a slab; (ii) connection types (blind-bolted endplate connections and through-plate connections); (iii) column section (square CFST column and circular CFST column); (iv) the effect of binding bars on the blind-bolted endplate connections for square CFST column; (v) column steel material types (stainless steel and carbon steel) vi) through-plate types (full-through plate and half-through plate) and vi) loading types (static and cyclic).

The configuration details of the blind-bolted endplate connections and through-plate connections are described below. The basic dimensions of the hybrid composite joints for square CFST column (360 × 360 × 6 mm) and circular CFST column (360 × 6 mm) are shown in Figure 3.3. The plan view of the composite slab (900 × 3500 mm) 120 mm thick and 1.0 mm steel profile sheet is illustrated in Figure 3.3(a). The slab reinforcements were placed in the longitudinal direction (6Ø12@160) and in the transverse direction (18Ø10@200). The elevation view of joints is given in Figure 3.3(b). Each CFST column was 2200 mm high; beams (310UB40.4) were 1570 mm long for blind-bolted endplate connections and 1545 mm long for through-plate connections. Shear connectors (16Ø19@190) were welded to steel beams to provide composite action.
Table 3.1 Details of joint test specimens (all units are in mm)

<table>
<thead>
<tr>
<th>Series</th>
<th>Specimen label</th>
<th>Joint type</th>
<th>Column Section (mm)</th>
<th>Column Steel type</th>
<th>Binding bars</th>
<th>Slab (b_p x h_p x t_p) (mm)</th>
<th>Plate section n</th>
<th>Column load N (kN)</th>
<th>Beam Loading types</th>
</tr>
</thead>
<tbody>
<tr>
<td>1) SB1-0</td>
<td>#8Ø20</td>
<td>No</td>
<td>-</td>
<td>-</td>
<td>120</td>
<td>230x304</td>
<td>0.6</td>
<td>4603</td>
<td>CL</td>
</tr>
<tr>
<td>2) SB1-1</td>
<td>□- Stainless steel</td>
<td>120</td>
<td>×10</td>
<td>0.6</td>
<td>4603</td>
<td>ML</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3) SB1-2</td>
<td>#8Ø20</td>
<td>120</td>
<td>×10</td>
<td>0.6</td>
<td>4603</td>
<td>ML</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4) SB1-3</td>
<td>#8Ø20</td>
<td>120</td>
<td>×10</td>
<td>0.6</td>
<td>4603</td>
<td>ML</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5) CB2-1</td>
<td>□- Stainless steel</td>
<td>120</td>
<td>×10</td>
<td>0.6</td>
<td>4603</td>
<td>ML</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6) CB2-2</td>
<td>□- Carbon steel</td>
<td>120</td>
<td>×10</td>
<td>0.6</td>
<td>4603</td>
<td>ML</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7) CB2-3</td>
<td>□- Carbon steel</td>
<td>120</td>
<td>×10</td>
<td>0.6</td>
<td>4603</td>
<td>ML</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8) ST3-1</td>
<td>□- Stainless steel</td>
<td>120</td>
<td>700x212a×212b</td>
<td>0.6</td>
<td>3805</td>
<td>ML</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9) ST3-2</td>
<td>□- Stainless steel</td>
<td>120</td>
<td>244x12</td>
<td>0.6</td>
<td>3805</td>
<td>CL</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10) ST3-3</td>
<td>□- Stainless steel</td>
<td>120</td>
<td>590x267a×244b</td>
<td>0.6</td>
<td>3830</td>
<td>ML</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notes:
(1) ‘B’ = blind-bolted endplate connection
(2) ‘T’ = through-plate connection
(3) □ = square section
(4) ○ = circular section
(5) ‘ML’ = monotonic static loading
(6) ‘CL’ = cyclic loading
(7) ‘a’, ‘b’ = width of full-through plate and half-through plate, respectively
(8) ‘n’ = axial load level
(9) ‘SB’ = blind-bolted endplate connection specimen for square column
(10) ‘CB’ = blind-bolted endplate connection specimen for circular column
(11) ‘ST’ = through-plate connection specimen for square column.
Blind-bolted endplate connection specimens were divided into ‘Series-SB1’ (square CFST column), ‘Series-CB2’ (circular CFST column). In Series-SB1, four composite joint specimens (SB1-0, SB1-1, SB1-2, SB1-3) were designed for square CFST column and fabricated using blind-bolted endplate connections by considering three parameters including binding bar, slab and loading type. The configuration of specimens (SB1-0, SB1-1, SB1-2, SB1-3) are shown in Figure 3.4 to 3.6; in which these different parameters were investigated: the effect of binding bars without the slab (SB1-0), the effect of the slab without binding bars (SB1-1) and the effect of the slab with binding...
bars (SB1-2, SB1-3) were investigated. The configuration of the joint in specimen SB1-0 is shown in Figure 3.4. This was designed to investigate the effect of binding bar without considering the slab effect on the joint during monotonic static loading. The dimension details and binding bars and blind bolts position are shown in Figures 3.4(a)-(d). The configuration of joint specimen SB1-1 (Figure 3.5) was designed to investigate the effect of the slab without considering the effect of binding bar on the joint during cyclic loading. Figures 3.5(a)-(d) show the dimension details of beam-column connection, including connection without binding bar. Figure 3.6 shows the configuration of joint specimens SB1-2 and SB1-3, which were designed to investigate the effect of the slab and the binding bar on the joint during monotonic static loading and cyclic loading respectively. Figures 3.6(a)-(d) show the dimensions of the beam-column connection, the position of binding bars and the location of blind bolts.

Figure 3.4 Blind bolted endplate connection specimen (SB1-0) with square CFST column of Series-SB1 (unit: mm)
Figure 3.5 Blind bolted endplate connection specimen (SB1-1) with square CFST column of Series-SB1 (unit: mm)

(a) Top view of connection  (b) Elevation view of section A-A

(c) Elevation view of section B-B  (d) Front view of (i) section C-C; (ii) section D-D

Figure 3.6 Blind bolted endplate connections (specimens SB1-2, SB1-3) with square CFST column of Series-SB1 (unit: mm)
In Series-CB2, another three joint specimens were designed for circular CFST column and fabricated using blind-bolted endplate connections, with the same configuration as specimen SB1-1 except the circular column section. The configuration of these joint specimens (CB2-1, CB2-2, CB2-3) is shown in Figure 3.7. In Series-CB2, the influence of the steel tube material of the circular CFST column was investigated. For specimens CB2-1 and CB2-2, the steel tube of each CFST column was fabricated from the stainless steel. CB2-1 was tested under monotonic static loading; CB2-2 was tested under cyclic loading. The tube of specimen CB2-3 was fabricated from carbon steel and the specimen was tested under monotonic static loading.

![Blind bolted endplate connections](image)

Figure 3.7 Blind bolted endplate connections (specimens CB2-1, CB2-2, CB2-3) with circular CFST columns of Series-CB2 (unit: mm)
(b) Configuration of CFST column joints with through-plate connections

Through-plate connection specimens in Series-ST3 were tested for square CFST column only. Three specimens (ST3-1, ST3-2, ST3-3) were fabricated using full-through-plate and half-through-plates. Specimens ST3-1 and ST3-2 had one full-through-plates in the direction of (Figure 3.8) the major axis of joint and two half-through-plates in the direction of the minor axis. In specimen ST3-3, two half-through-plates were placed in the direction of the major axis of the joint and one full-through-plate was used in the direction of the minor axis of the joint (Figure 3.9). The influence of the full-through-plate and half-through-plates on the behaviour of the joint was investigated under monotonic static loading (ST3-1, ST3-3) and cyclic loading (ST3-2).

Figure 3.8 Full-through-plate connections (specimens ST3-1, SB3-2) with square CFST column of Series-ST3 (unit: mm)
3.3.2 Fabrication of test specimens

All test specimens were prepared at the Institute for Infrastructure Engineering Laboratory, Western Sydney University. Carbon steel tube and stainless steel tubes for the CFST columns were fabricated in the workshop. For Series-SB1 tests, steel tubes with binding bars were also fabricated in the workshop according to the design requirement as shown in Figure 3.4 and Figure 3.6; in which binding bars were passed through the cutting hole of steel tube and then welded to steel tube. The cutting of Hollo-Bolts was made according to their position and hole diameter, which was assumed to be 2 mm greater than the outer diameter of the sleeve of the Hollo-Bolt. Commercially available 310UB40.4 steel beams were used. Flush endplates were welded to the one end of the beams by full penetration butt welds. Lindapter Hollo-Bolts (M20), also known as Hollo-Bolts (M20), were used to connect the end plates to the steel tube. These bolts were tightened to the design torque of 300 N.m (Figure 3.10), which is specified by Lindapter for M20 Hollo-Bolts. The bolt tightening forces
should not exceed about 0.7 times the bolt yield stress. The composite slab with a 1.0 mm thick steel profile sheet was cast over the steel beams. Reinforcing bars (longitudinal reinforcement: 6Ø12@160; transverse reinforcement: 18Ø10@200) were used (Figure 3.11). Shear connectors (16Ø19@190) were welded to steel beams to provide composite action.

Figure 3.10 Blind bolts installation and tightening system: (a) insert blind bolt; (b) fasten the blind bolt using tightening torque; (c) apply the design torque using torque meters

Figure 3.11 Preparation of hybrid composite steel beam-CFST column joint specimens
3.3.3 Test setup and instrumentation

The testing of the joint specimens was carried out at the Institute for Infrastructure Engineering Laboratory, Western Sydney University. The loading system comprised one included one 10000 kN capacity hydraulic jack applying an axial load to the column and two 500 kN capacity mechanical actuators applying loads to the beams 1500 mm from the centre of the column. The upper and lower ends of the column were hinged, as shown in Figure 3.12. The lower end of the column was restrained in all directions except vertical; the upper end was restrained in all directions, including rotation. The distance from the upper to the lower hinge was 2660 mm, which was regarded as the effective height of the column.

During testing of specimens SB1, CB3 and ST2, four sets of data were recorded by four different types of sensor: strain gauges, linear variable displacement transducers (LVDTs), inclinometers and linear potentiometers. Their locations are shown in Figure 3.13 for the blind-bolted endplate connections (Series-SB1 and CB2) and in Figure 3.14 for the through plate connections (Series-ST3).

Figure 3.12 Experimental set up of composite joints for monotonic static loading and cyclic loading (unit: mm)
Figure 3.13 Layout of strain gauges, LVDT, inclinometers and linear potentiometers for blind-bolted endplate connections (Series-SB1 and CB2) (unit: mm)

Figure 3.14 Layout of strain gauges, LVDT, inclinometers and linear potentiometers for through plate connections (Series-ST3) (unit: mm)
The recorded test data were (i) strain values for different steel components (tube, endplate, beam flanges and webs near the connection), (ii) displacements (horizontal displacement of steel tubes, endplates and bolts, vertical displacement of composite beams), rotation of steel beams and slip between steel beam and composite slab. Based on the recorded data, typical curves from different measurements (load-strain curves for different components, moment-rotation curves for joints, load-deformation curves for beams) are discussed in Chapter 4. The rotation of the joint was inferred from the rotations of the beam and tube. Each sensor layout is described in the following subsections.

(a) Strain gauges

For each joint specimen of Series-SB1 and CB2 (SB1-1 to SB1-3, CB2-1, CB2-2 and CB2-3), a total of 22 strain gauges (SG) were attached to measure the strains in the tube (8 strain gauges), endplate (8 strain gauges), beam flanges (4 strain gauges) and beam web (2 strain gauges) near the connection, and for the bare steel joint specimen (SB1-0) the same strain gauges were used to measure the strains in the tube, endplate and beam. Twenty strain gauges were used for each specimen in Series-ST3 (ST3-1 to ST3-3); the main difference was in the number of strain gauges (6) used in through-plate and their locations. All readings were recorded by microcomputer. The strain gauge layout is shown in Figure 3.13 for Series-SB1 and CB2 and in Figure 3.14 for Series-ST3.

(b) Linear variable displacement transducers (LVDTs)

Fourteen LVDTs was used for the blind-bolted endplate connections in Series-SB1 and CB2 to measure the vertical displacement of the beams (6 LVDTs) and the horizontal displacement of the tube (2), bolts (2) and endplates (4). Ten LVDTs were used for the through-plate connection of Series-ST2 to measure the vertical displacement of the beams (6 LVDTs) and through-plates (2), and the horizontal displacement of the tube (2). For all joint specimens, six LVDTs were placed in different locations on the beams to measure their vertical displacement. The horizontal movement of the tube and the endplates near the connection was measured using a further two LVDTs on the tube and four on the endplate. The layout of the 14 LVDTs used for Series-SB1 and CB2 is shown in Figure 3.13, and the 10 LVDTs for Series-ST3 in Figure 3.14.
(c) **Inclinometers**
A total of four inclinometers (IM) for each joint specimen in Series-SB1, CB2, ST2 (SB1-0, SB1-1, SB1-2, SB1-3; CB2-1, CB2-2 & CB2-3; and ST3-1, ST3-2, ST3-3), were used to measure the rotation of the beams. Inclinometers were symmetrically placed at various positions along both spans of the beams. Location details of the inclinometers for Series-SB1 & CB2 are shown in Figure 3.13, and for Series ST3 in Figure 3.14.

(d) **Linear potentiometer**
Two linear potentiometers (LP) were used on each joint specimen of Series-SB1 (SB1-1 to SB1-3), Series-CB2 (CB2-1 to CB2-3) and Series-ST2 (ST2-1 to ST2-3). These measured the slip between the beam and the slab. They were located for specimens of Series-SB1 and CB2 where shown in Figure 3.13, and for Series-ST3 in Figure 3.14.

3.3.4 Test procedure
(a) **Static monotonic loading procedure**
During the tests on each specimen, two types of load were applied: axial compressive column load, and beam loads. The axial column loads were applied by a 10000 kN capacity hydraulic jack, as discussed above, reacting against a rigid steel frame onto an upper support at the bottom of the CFST column. An axial compressive load was firstly applied step-by-step at the top of the CFST column up to a constant load calculated in accordance with Eurocode 4 (2005) (approximately 60% of the column capacity based on the strengths of the concrete and steel tube). The load (monotonic static or cyclic) was applied to the beams at a distance of 1320 mm from the CFST column face. The cyclic loading was considered to simulate seismic action.

(b) **Quasi-static cyclic loading procedure**
Cyclic load was applied to some specimens by the vertical actuator at the tip of the beams to investigate the effect of seismic loading on the composite joints. For cyclic loading, the actuator load was able to be controlled either by standard ATC loading history or standard SAC loading history. The displacement history shown in Figure
3.15 is used in ATC loading history (ATC, 1992). The rotation history as shown in Figure 3.16 is recommended in the SAC loading history (SAC, 1997). In the standard ATC loading history, the cyclic load is applied in two stages, elastic and plastic. In the elastic stage, six cycles are conducted under displacement control at displacement levels of 0.25Δy, 0.5Δy and 0.7Δy (where Δy is the vertical yielding displacement) with two cycles at each level. In the second (plastic) stage, beams are loaded to displacement levels of Δy, 1.5Δy, 2Δy, 3Δy, 5Δy, 7Δy and 8Δy for a total of 17 cycles (three cycles at each Δy, 1.5Δy, 2Δy displacement levels and two cycles each at 3Δy, 5Δy, 7Δy and 8Δy displacement levels). These loading histories mainly depend on Δy, which is estimated from the vertical yielding load, assumed to equal to 0.7P_{uc}, where P_{uc} is the estimated ultimate vertical loading capacity of a connection. Since P_{uc} generally varied from specimen to specimen, it was clear that considerable difficulty would be experienced in controlling the tests to produce identical displacement histories for all specimens.

The standard SAC loading history (SAC, 1997) shown in Figure 3.16 was adopted in this study to apply the same rotational effect to all joint specimens subjected to cyclic loading.

In the standard SAC loading history, the rotation history is divided into load steps and the peak rotation of each load step j is given as a predefined value $\theta_j$, defined as beam deflection divided by beam span. The beam deflection was then used to control the rotation history. At each loading step, the beam deflections were calculated as defined
in the SAC loading history. The beam deflection corresponding to predefined rotation is given in Table 3.2. The first three load steps were set out with six cycles of 0.00375, 0.005 and 0.075 rad rotation per step. The next four cycles were at 0.01 rad rotation in the fourth step, two cycles of 0.015 rad rotation in the fifth load step, then two cycles each of successively increased rotations (0.02, 0.03, 0.04 and 0.05 rad) until the joint fails.

![Rotation history curve of the cyclic loading (SAC, 1997)](image)

Figure 3.16 Rotation history curve of the cyclic loading (SAC, 1997)

<table>
<thead>
<tr>
<th>Load step ( (j) )</th>
<th>No of Cycles in load step</th>
<th>Rotations ( \theta_j ) defined in SAC loading history (rad)</th>
<th>Calculated corresponding displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6</td>
<td>0.00375</td>
<td>4.95</td>
</tr>
<tr>
<td>2</td>
<td>6</td>
<td>0.005</td>
<td>6.6</td>
</tr>
<tr>
<td>3</td>
<td>6</td>
<td>0.0075</td>
<td>9.9</td>
</tr>
<tr>
<td>4</td>
<td>4</td>
<td>0.01</td>
<td>13.2</td>
</tr>
<tr>
<td>5</td>
<td>2</td>
<td>0.015</td>
<td>19.8</td>
</tr>
<tr>
<td>6</td>
<td>2</td>
<td>0.02</td>
<td>26.4</td>
</tr>
<tr>
<td>7</td>
<td>2</td>
<td>0.03</td>
<td>39.6</td>
</tr>
<tr>
<td>8</td>
<td>2</td>
<td>0.04</td>
<td>52.8</td>
</tr>
<tr>
<td>9</td>
<td>2</td>
<td>0.05</td>
<td>66</td>
</tr>
<tr>
<td>10</td>
<td>2</td>
<td>0.06</td>
<td>79.2</td>
</tr>
</tbody>
</table>

### 3.4 Materials Properties Testing

Stainless steel, carbon steel and concrete were utilised in the construction of the hybrid composite beam-CFST column joints. Tests coupons of the steel materials were
prepared and tested as per AS1391. The steel materials were steel beam (310UB40.4), carbon steel tube and stainless steel tube (both 6 mm thick), endplate (10 mm thick) and through-plate (12 mm thick), steel profile sheet (1 mm thick), M20 Hollo-Bolt, binding bar (20 mm diameter), and reinforcing steel bars (12 mm diameter for longitudinal bar and 10 mm diameter for distributing bar). The total number of test coupons is listed in Table 3.3. The location and layout of the structural steel test coupons are given in Appendix C. The test coupons of the beam flange and web were cut from the ends of three different test specimens. The tested material properties are illustrated in the following subsections.

<table>
<thead>
<tr>
<th>Items for coupon testing</th>
<th>Material types</th>
<th>Total coupons (39)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Square and circular steel (6 mm thick)</td>
<td>Stainless steel</td>
<td>3+3=6</td>
</tr>
<tr>
<td>2. Circular tube (6 mm thick)</td>
<td>Carbon steel</td>
<td>3</td>
</tr>
<tr>
<td>3. Steel beam (310UB40.4)</td>
<td>Carbon steel</td>
<td>3+3=6</td>
</tr>
<tr>
<td>4. Endplate (10 mm)</td>
<td>Carbon steel</td>
<td>3</td>
</tr>
<tr>
<td>5. Through plate (12 mm)</td>
<td>Carbon steel</td>
<td>3</td>
</tr>
<tr>
<td>6. Steel profile sheet (1 mm thick)</td>
<td>Carbon steel</td>
<td>3</td>
</tr>
<tr>
<td>7. Shear connector (19 mm dia.)</td>
<td>Carbon steel</td>
<td>3</td>
</tr>
<tr>
<td>8. Blind bolt (M20 Hollo-Bolt)</td>
<td>Carbon steel</td>
<td>3</td>
</tr>
<tr>
<td>9. Binding bar (20 mm dia.)</td>
<td>Carbon steel</td>
<td>3</td>
</tr>
<tr>
<td>10. Reinforcing steel (10 and 12 mm dia.)</td>
<td>Carbon steel</td>
<td>3+3=6</td>
</tr>
</tbody>
</table>

3.4.1 Concrete

The compressive strength of the concrete in the CFST column and slab was 32 MPa. The compressive strength and modulus of elasticity of the concrete were determined by compression tests on standard cylinders (diameter 150 mm × height 300 mm) at various intervals during the curing process until the experiments were complete. A total of 45 concrete cylinder tests were performed. Of these, six tests (3 for compressive strength and 3 for modulus of elasticity) was conducted at 28 days, and 39 cylinder tests for 10 joint specimens (3 tests for compressive strength and 9 tests for modulus of elasticity of three joint specimens) were conducted on the day that the experiments were completed. A compressive cylinder test on the day of completion of experiments was also
performed before the joint specimens were tested to take into account of the total compressive load applied to the CFST column during the testing of joint specimens. A summary of the mechanical properties of the concrete at 28 days is given in Table 3.4. Other cylinder test results are reported in Appendix C.3.

Table 3.4 Average concrete cylinder strengths at the testing day of each joint specimen

<table>
<thead>
<tr>
<th>Specimen no.</th>
<th>Ultimate compressive strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. SB1-0</td>
<td>32.8</td>
</tr>
<tr>
<td>2. SB1-1</td>
<td>33.6</td>
</tr>
<tr>
<td>3. SB1-2</td>
<td>35.7</td>
</tr>
<tr>
<td>4. SB1-3</td>
<td>39.8</td>
</tr>
<tr>
<td>5. CB2-1</td>
<td>42.3</td>
</tr>
<tr>
<td>6. CB2-2</td>
<td>41.2</td>
</tr>
<tr>
<td>7. CB2-3</td>
<td>43.8</td>
</tr>
<tr>
<td>8. ST3-1</td>
<td>29.93</td>
</tr>
<tr>
<td>9. ST3-2</td>
<td>33.3</td>
</tr>
<tr>
<td>10. ST3-3</td>
<td>34.4</td>
</tr>
</tbody>
</table>

Figure 3.19 Concrete cylinder testing

3.4.2 Stainless steel

Six coupon tests were performed for stainless steel materials (grade 1.4301), three tests for the square stainless steel tube and three for the circular stainless steel tube. All test
results are listed in Appendix C.1. The average values obtained for the square tube and the circular tube are listed in Table 3.5.

Table 3.5 Mechanical properties of stainless steel materials

<table>
<thead>
<tr>
<th>Items of stainless steel materials</th>
<th>0.01 % Proof stress (MPa)</th>
<th>0.2 % Proof stress (MPa)</th>
<th>Ultimate stress (MPa)</th>
<th>Elastic modulus (MPa)</th>
<th>n</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stainless steel tube, thickness 6 mm, grade 1.4301 Square tube</td>
<td>215.4</td>
<td>339</td>
<td>714</td>
<td>200985</td>
<td>6.6</td>
</tr>
<tr>
<td>Circular tube</td>
<td>245.3</td>
<td>367</td>
<td>732</td>
<td>200663</td>
<td>7.4</td>
</tr>
</tbody>
</table>

3.4.3 Carbon steel

Carbon steels i.e. mild steels were used to fabricate the steel tube, steel beam, endplate, through plate, steel profile sheet. Three coupon tests were conducted for each item of these steel materials. Coupon test results of each item are reported in Appendix C.2. The average yield strength, ultimate strength and elastic modulus value obtained from three coupons of each item are summarised in Table 3.6.

Table 3.6 Mechanical properties of carbon steel materials

<table>
<thead>
<tr>
<th>Items of carbon steel materials</th>
<th>Yield strength (MPa)</th>
<th>Tensile strength (MPa)</th>
<th>Elastic modulus (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Carbon steel tube</td>
<td>379</td>
<td>473</td>
<td>206338</td>
</tr>
<tr>
<td>(circular tube, 6 mm)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2. Steel beam (310UB40.4) Flange</td>
<td>352</td>
<td>535</td>
<td>198494</td>
</tr>
<tr>
<td>Web</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3. Endplate (10 mm)</td>
<td>388</td>
<td>506</td>
<td>206298</td>
</tr>
<tr>
<td>4. Through plate (12 mm)</td>
<td>375</td>
<td>497</td>
<td>207461</td>
</tr>
<tr>
<td>5. Steel profile sheet</td>
<td>352</td>
<td>535</td>
<td>198494</td>
</tr>
</tbody>
</table>

3.4.4 Structural bolts and shear studs

Three coupon tests were conducted the structural M20 HB bolts and three for the 19 mm diameter shear studs. All coupon test results are reported in Appendix C.2. The average values are given in Table 3.7.
Table 3.7 Mechanical properties of structural bolts and shear studs

<table>
<thead>
<tr>
<th>Items of carbon steel materials</th>
<th>Yield strength (MPa)</th>
<th>Tensile strength (MPa)</th>
<th>Elastic modulus (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Hollo-Bolt (M20)</td>
<td>890</td>
<td>953</td>
<td>218871</td>
</tr>
<tr>
<td>2. Shear stud (19 Diameter)</td>
<td>375</td>
<td>518</td>
<td>203941</td>
</tr>
</tbody>
</table>

3.4.5 Reinforcement bars

N12 (12 mm) and N10 (10 mm) reinforcement bars were used in the slab. N20 bars (20 mm) were used to fabricate the binding bars. Coupon tests of N10, N12 and N20 bars were conducted. All results are given in Appendix C.2. The average yield strength, ultimate strength and elastic modulus value obtained from three coupons of the N10, N12 and N20 bars are listed in Table 3.8.

Table 3.8 Mechanical properties of N10, N12 and N20 bars

<table>
<thead>
<tr>
<th>Items of carbon steel materials</th>
<th>Yield strength (MPa)</th>
<th>Tensile strength (MPa)</th>
<th>Elastic modulus (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Slab reinforcement (N10, 10 mm diameter)</td>
<td>524</td>
<td>631</td>
<td>199577</td>
</tr>
<tr>
<td>2. Slab reinforcement (N12, 12 mm diameter)</td>
<td>538</td>
<td>653</td>
<td>200371</td>
</tr>
<tr>
<td>3. Binding bar (N20, 20 mm diameter)</td>
<td>545</td>
<td>647</td>
<td>203261</td>
</tr>
</tbody>
</table>

3.5 Summary

This chapter contains detailed information about the design of the testing program, the experimental setup and instrumentation, and the test procedures for the hybrid steel beam-CFST column joints with concrete slab.

Information is also provided of the mechanical properties of the materials used. These include concrete cylinder strength, and yield and ultimate strength and modulus of elasticity of the carbon steel and stainless steel components, bolts and shear studs, determined from coupon tests.
CHAPTER 4

TEST RESULTS AND DISCUSSION

4.1 Introduction
This chapter presents the test results of 10 specimens tested for hybrid composite beam-CFST column joints using blind-bolted endplate connections and through-plate connections. The details of all specimens are given in Chapter 3, including the geometry of the test specimens, testing procedure and instrumentation. Seven specimens were tested to investigate the behaviour of blind-bolted endplate connections. Another three specimens were tested to investigate the behaviour of the joints with through-plate. The test results are discussed in section 4.2 for the blind-bolted endplate connections and in section 4.3 for the through-plate connections. Finally, the comparative discussion among the test results is highlighted in section 4.4 (blind-bolted endplate connections) and in sections 4.5 (through-plate connections).

4.2 Test Results for CFSST Column Joints with Blind bolted Endplate Connections
The test results of concrete-filled stainless steel tubular (CFSST) column joints with blind bolted endplate connections are presented in this section to provide information about the failures modes, moment-rotation relationship, the slip between slab and steel beam, bolt slip from the concrete core of the CFST column, endplate deformation behaviour and strain behaviour of connection components. Among seven specimens, five specimens (SB1-0, SB1-1, SB1-2, CB2-1 and CB2-3) were tested under static monotonic loading and two specimens (SB1-3 and CB2-2) were tested under cyclic loading. Specimen SB1-0 was fabricated without a slab; all other specimens were fabricated with a slab. Square CFSST columns were used in specimens SB1-0, SB1-1, SB1-2, and SB1-3; circular CFSST columns were used in specimens CB2-1, CB2-2 and CB2-3. The test results of the blind bolted endplate connections are given in the following sub-sections.
4.2.1 Failure modes

The failure modes were noted during testing of the blind-bolted endplate connections. The failure behaviour of the joint without slab (SB1-0) was quite different from the failure behaviour of joints with slab (specimen SB1-1, SB1-2, SB1-3, CB2-1, CB2-2 and CB2-3). When the steel beam-CFST column joint did not have a slab (specimen SB1-0) and was tested under monotonic static loading, the failure of the joint was due to the outward deformation of the steel tube, as shown in Figure 4.1. On the other hand, when the joints with slab were tested either in monotonic static loading or cyclic loading, the failure of joints were observed mainly due to the severe cracking of the slab accompanied by fracture of reinforcement bars, as shown in Figure 4.2.

![Failure causes: Steel tube outward buckling; Hollo-bolt being pulled out from concrete](image)

At the presence of the slab, initial damage to the joint occurred at the extreme tensile fibers of the composite slab, resulting in transverse concrete cracking. The concrete cracks propagated deeper into the section near the column face until through-depth
cracks occurred in the slab. The large cracks on both side of the column were clearly visible and extended through the whole slab section until reaching the profiled steel sheeting. When the slab reinforcement bars failed in tension, the connection reached its ultimate load capacity. The load capacity of the connection then dropped suddenly due to the loss of the load-carrying capacity of the steel reinforcement. Later, the profiled sheeting fractured due to the increased deformation.

![Failure causes: Slab fracture (Concrete cracking, reinforcing bar fracture)](image1)

(a) Blind bolted endplate connections with square CFSST column

![Failure causes: Slab failure (Concrete cracking, reinforcing bar fracture)](image2)

(b) Blind bolted endplate connections with circular CFSST column

Figure 4.2 Failures modes of blind bolted endplate connections with slab

### 4.2.2 Moment-rotation relationships of joints

The moment-rotation curve for each joint was calculated from the bending moment acting on the connection and the corresponding relative rotation between the connected members. In the test, the vertical displacement of the beam and load at the beam tip were measured during the entire loading phase. The bending moment \( M_j \) of the joint was calculated from the load \( P \) multiplied by the distance \( l_b \):

\[
M_j = Pl_b
\]
From the rotations of the column and beam, the relative rotation of joints was calculated as:

$$\phi_{j1} = \phi_b - \phi_c$$

where $\phi_c$ is the rotation of the column and $\phi_b$ is the rotation of the beam, calculated from the displacements ($\delta_{ct}$, $\delta_{cc}$ and $\delta_b$) measured by the linear variable displacement transducers (LVDTs). $\phi_c$ and $\phi_b$ were calculated from:

$$\phi_c = \frac{\delta_{ct} - \delta_{cc}}{h_{cb}}$$

$$\phi_b = \frac{\delta_b}{l_b}$$

where $\delta_{ct}$ and $\delta_{cc}$ are the horizontal displacements of the column above the composite slab level (tension zone) and below the composite beam level (compression zone), respectively; and $h_{cb}$ is the gauge length between the measurement locations for $\delta_{ct}$ and $\delta_{cc}$. $\delta_b$ is the vertical displacement of the composite beam measured near the connection and $l_b$ is the length from the face of the steel tube of the CFST column to the measurement location of vertical displacement of the beam.

The moment-rotation curves of seven specimens of blind-bolted endplate connections are reported in Figure 4.3. Table 4.1 shows the moment and rotation capacity of all test specimens of the blind-bolted connections. Figure 4.3(a) shows the result of the specimen SB1-0 without a slab. The yielding moment (30 kN-m) reached when the blind bolt separated (being pulled out of) from the concrete core of the column and then reached the ultimate moment (37 kN-m). The joint failed due to the yielding of the steel tube near the bolts and excessive outward deformation of the tube.

<table>
<thead>
<tr>
<th>Joint</th>
<th>Ultimate moment (kN-m)</th>
<th>Rotation at ultimate moment (mrad)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Hogging moment</td>
</tr>
<tr>
<td>SB1-0</td>
<td>37.7</td>
<td>–</td>
</tr>
<tr>
<td>SB1-1</td>
<td>238.4</td>
<td>–</td>
</tr>
<tr>
<td>SB1-2</td>
<td>264.0</td>
<td>–</td>
</tr>
<tr>
<td>SB1-3</td>
<td>254.4</td>
<td>81.0</td>
</tr>
<tr>
<td>CB2-1</td>
<td>277.2</td>
<td>–</td>
</tr>
<tr>
<td>CB2-2</td>
<td>262.0</td>
<td>152.0</td>
</tr>
<tr>
<td>CB2-3</td>
<td>270.7</td>
<td>–</td>
</tr>
</tbody>
</table>
Figure 4.3 Moment-rotation behaviour of blind bolted endplate connections tested under static and cyclic loading
Figure 4.3 Moment-rotation behaviour of blind bolted endplate connections tested under static and cyclic loading (continued)

The test results for the composite joints with a square CFSST column (SB1-1, SB1-2 and SB1-3) are shown in Figure 4.3(b)-(d). The moment-rotation curve for specimen SB1-1 (without binding bars) as shown in Figure 4.3(b) was observed to be similar to that for SB1-2 (with binding bars) as shown in Figure 4.3 (c), but the ultimate moment capacity of SB1-2 was higher (10.74%). It is also seen that the first yielding moment occurred due to cracking of the concrete slab, then the ultimate moment of both specimens was reached when the slab reinforcement fractured. The rotation capacity at the ultimate moment was found to be almost the same for all specimens (SB1-1, SB1-2 and SB1-3) under static monotonic loading and quasi-static cyclic loading. The hogging and sagging moment-rotation curves of specimen SB1-3 tested under quasi-static cyclic loading are shown in Figure 4.3(d), for north beam. The hogging moment capacity was observed to exceed the sagging capacity. This is because of the higher tensile force generated by the hogging moment (mainly carried by the slab reinforcement and the profile sheet) than the tensile force (resisted by the steel tube near the bottom flange of the beam) generated under the sagging moment.

The test results of the composite joints with a circular CFSST column (CB2-1, CB2-2 and CB2-3) are shown in Figure 4.3(e)-(g). The hogging moment-rotation behaviour of specimens of CB2-1 and CB2-3 were found almost the same as for specimen SB1-2. The hogging moment-rotation behaviour of CB2-2 was observed similar to SB1-3, but the sagging moment of CB2-2 was 87.65% higher than that of SB1-3. The higher
sagging moment was observed in CB2-2 due to the use of circular section for the CFST column. Similar to joints with a square column, when the slab reinforcement fractured, the moment capacity of the joints with a circular column suddenly dropped, and then the tensile load generated by the hogging moment was mainly carried by the profiled sheeting before it fractured.

4.2.3 Slip between slab and steel beam

The composite action between the steel beam and slab can be developed by introducing shear connectors on the steel beam. One end of the shear connector is normally welded to the top flange of the beam, and the other end has a round head which is embedded into the concrete of the slab. The composite action between the steel beam and slab was determined by measuring the relative slip between them in this study. Two linear potentiometers (LP) were used to measure the slip value; LP1 was placed on the northern side of the beam and LP2 was placed on the opposite (southern) side of the beam (see Figure 3.13, Chapter 3). Figure 4.4 shows typical beam load vs. slip curves obtained from the tests. The slip at the ultimate load was observed to be less than 0.1 mm for all specimens except SB1-2. This is due to the fact that a high degree of the shear connectors, i.e. \( \eta_y = \frac{N_{scF_{sc,max}}}{A_r f_{y,r}} = 300\% \), was adopted for all specimens. So, it can be concluded that the slip between the steel beam and concrete was negligible before the ultimate strength was obtained.

Figure 4.4 Beam load vs. slip (between slab and beam) curves for blind bolted endplate connections
Figure 4.4 Beam load vs. slip (between slab and beam) curves for blind bolted endplate connections (continued)

**4.2.4 Bolt slip from concrete core of CFST column**

Hollo-Bolts were used in the endplate connections to join the steel beams to the steel tube of the CFST column. Since the inner surface of the steel tube is not perfectly bonded to the outer surface of the concrete core of the CFST column, there is a possibility that the steel tube may separate from the core. This occurs if the steel tube is thin and the bolt is pulled out of the concrete core. This bolt pull-out action referred to here as bolt slip was measured using linear variable displacement transducers HDT2 and HDT7, shown in Figure 3.13 in Chapter 3. The plots of beam load vs. bolt slip are shown in Figure 4.5.

It was seen that the bolt slip at the ultimate load of the joints was higher for the joint specimens with square columns (SB1-0, SB1-1, SB1-2) than for the joints with circular columns (CB2-1, CB2-3). This was due to the different deformation capacities of the two tube sections. Bolt slip at the ultimate load of specimen SB1-0 (no slab) was the highest due to the absence of the slab. It was also noticed that the bolt slip in specimen SB1-1 (without binding bars) at ultimate load was greater than the specimen SB1-2 with binding bars. In specimen SB1-2, the binding bars acted as stiffeners and enhanced the composite action between the steel tube and the concrete core of the column. As a result, the separations of the steel tube from the concrete core were minimised due to
the presence of the binding bars on the steel tube, and thus the bolt slip from the concrete core was observed lower in SB1-2 compared to SB1-1.

Figure 4.5 Beam load vs. bolt slip curves of the blind bolted endplate connections
4.2.5 Endplate deformation behaviour

Endplates could deform due to the bending moment of the joint. During the testing, the deformation of the endplates was measured using two LVDTs (labelled as HDT1 and HDT6). To measure the deformation of the endplate, transducers were placed above the top bolt and below the beam top flange, where the locations are shown in Figure 3.13, Chapter 3. Figure 4.6 shows the beam load vs. endplate deformation curves of the blind bolted endplate connections. Figure 4.6(b) shows the maximum deformation of 3.25 mm at the ultimate load occurred in specimen SB1-1 with square CFSST column and flat endplate. Smaller deformations were observed in specimens CB2-1 and CB2-3 (Figure 4.6(d) and (e)) with circular column section and curved endplates. The higher endplate deformation in specimen SB1-1 may have been due to the outward deformation of the steel tube combined with the flat endplate, because the bending resistance capacity of the flat plate on the square column was less than for the curved plate on the circular column. When binding bars were welded to the square steel tube of the column in specimen SB1-2, the endplate deformation was minimised as shown in Figure 4.6 (c). From that it was concluded that the endplate deformation depends on the type of column section (square or circular) and the type of endplate (flat or curved) used in the blind bolted endplate connections.

![Figure 4.6 Beam load vs. endplate deformation curves of the blind bolted endplate connections](image)

(a) Specimen SB1-0: square CFSST column, without slab
(b) Specimen SB1-1: square CFSST column, with slab

Figure 4.6 Beam load vs. endplate deformation curves of the blind bolted endplate connections
(c) Specimen SB1-2: square CFSST column, with slab  
(d) Specimen CB2-1: circular CFSST column, with slab

(e) Specimen CB2-3: circular CFSST column, with slab

Figure 4.6 Beam load vs. endplate deformation curves of the blind bolted endplate connections (continued)

### 4.2.6 Strain developments in blind bolted connection components

Strains in the blind bolted connection components (steel beams, endplates and steel tube wall) were measured using strain gauges. The locations of the strain gauges for the steel beams, endplates and steel tube wall are shown in Figure 3.13, Chapter 3. The values obtained from the strain gauges are reported in the following.

**(a) Strains in the steel beam**

The strain gauges (SG) on the steel beams were placed 125 mm from the column face. Strain gauges were located on the flanges of each beam: one on the top flange (SG1 on the northern beam, SG4 on the opposite, or southern beam) and one on the bottom flange (SG3 on the northern beam and SG6 on the southern beam). One strain gauge
was located at the centre of the beam web on each side (SG2 on the northern beam, SG5 on the southern beam). Figure 4.7 shows the beam strains plotted against load. It is seen from Figure 4.7(a)-(e) that the compressive strains in the bottom flange were higher than the tensile strains in the top flange: the maximum compressive strains in the bottom flange at ultimate load were 373 µε for specimen SB1-0, 1706 µε for specimen SB1-1, 1870 µε for specimen SB1-2, 1833 µε for specimen CB2-1, and 1811 µε for specimen CB2-3. The yield strain of the flange and web were 1774 µε and 1816 µε, respectively, from the coupon tests for the beam flange and web; thus yielding of the beam bottom flange was observed in specimens SB1-2, CB2-1 and CB2-3.

Figure 4.7 Beam load vs. beam strain curves of the blind bolted endplate connections
Figure 4.7 Beam load vs. beam strain curves of the blind bolted endplate connections (continued)
(b) Strains in the endplate

Figure 4.8 shows the tensile and compressive strains of the endplate in the blind bolted endplate connections. Figure 4.8(a)-(e) show that the tensile strains in SG7, SG9, SG11 and SG13 for all test specimens were higher than the compressive strains in SG8, SG10, SG12 and SG14. The maximum tensile strains at the ultimate load of each specimen were 283 µε for specimen SB1-0, 645 µε for SB1-2, 724 µε for CB2-1 and 837 µε for specimen CB2-3. The coupon test of the endplate showed that the yield strain for the endplate was 1881 µε, so it is seen that the maximum tensile and compressive strains in the endplate were below the yield strain, indicating that the endplate remained intact without yielding.

Figure 4.8 Beam load vs. endplate strain curves of the blind bolted endplate connections
Figure 4.8 Beam load vs. endplate strain curves of the blind bolted endplate connections (continued)
(c) Strains in the steel tube

Figure 4.9 shows the steel wall strains in the longitudinal direction of the column wall close to the top and bottom flange of the beam. The yield strains of the steel tube wall were 1686 με for the square stainless steel tube, 1828 με for circular stainless steel tube, 1836 με for the circular carbon steel tube. It is seen from Figure 4.9 for all test specimens (except specimens SB1-1 and SB1-2) that the maximum tensile and compressive strains of the steel tube wall were generally below their yield strains, which indicates that the steel tube wall remained intact without yielding for all specimens except SB1-1 and SB1-2.

Figure 4.9 Beam load vs. tube wall strain curves of the blind bolted endplate connections
Figure 4.9 Beam load vs. tube wall strain curves of the blind bolted endplate connections (continued)

4.3 Test Results for CFSST Column Joints with Through-plate Connections

The test results of CFSST column joints with through-plate connections are presented in this section to provide information about failures modes, moment-rotation relationships, slip between slab and steel beam, bolt slip from the concrete core of the CFST column, beam deformation behaviour, through-plate deformation behaviour and strain developments of different connection components. Of three specimens, two specimens (ST3-1 and ST3-3) were tested under static monotonic loading and one specimen (ST3-2) was tested under cyclic loading. Specimen ST3-2 was fabricated
using half through-plate connections. Full through-plate connections were investigated in specimens ST3-1 and ST3-2. The details of the through-plate connections are illustrated in section 3.3.2 of Chapter 3. The results of all through-plate connection tests are presented in the following subsections.

### 4.3.1 Failure modes

The failure modes observed during the testing of the through-plate connections are shown in Figure 4.10 for full through-plate connections (ST3-1) and in Figure 11 for half through-plate connections (ST3-3). Both figures show that major failure modes include the cracking of the composite slab, bearing deformation of the beam web and twisting of the through-plate. Initially, transverse cracking was observed on the topmost fibres of the slab caused by the hogging moment resulting in high tensile stresses above the neutral axis.

![Failure modes](image)

(a) Specimen ST3-1 after failure  
(b) Through plate twisting  
(c) Bearing deformation of bolt hole  
(d) Concrete cracking

Figure 4.10 Failures modes of full through-plate connections (specimen ST3-1)
With increasing moment, these cracks in the slab propagated deeper and wider, leaving less of the concrete section effective. The slab crack pattern at failure is seen in Figure 4.10(d) for specimen ST3-1 and in Figure 4.11(d) for specimen ST3-3. The most severe cracking occurred close to the face of the column, where the largest moment was exerted. Although the concrete slab exhibited significant cracking, the steel reinforcement remained intact. Large cracks on either side of the column are clearly visible and extended through the whole section until reaching the profiled sheeting, which remained intact. This was due to its close proximity to the plastic neutral axis, and thus it experienced only small deformation.

The full through-plate itself sustained some damage, as seen in Figure 4.10(b), but remained good integrity. The half through-plate also suffered some damage, as seen in Figure 4.11(b). For both specimens, the through-plates have undergone torsional buckling as shown in Figures 4.10(b) and 4.11(b). This could have been due to the
combined action of flexural compression forces and the eccentric position of the plate. Meanwhile, significant bolt hole bearing deformation occurred due to the M20 bolt shearing through the beam web, as seen in Figures 4.10(c) and 4.11(c).

4.3.2 Moment-rotation relationships of joints

Figure 4.12 shows the moment-rotation behaviour of the through-plate connections tested under static monotonic loading (ST3-1 and ST3-3) and quasi-static cyclic loading (ST3-2). Table 4.2 shows the moment and rotation capacity of all tested specimens of the through-plate connections. For the full through-plate connections (ST3-1), the moment–rotation relationships measured for the full through-plate (VDT3) and beam (VDT1 and VDT2) were dissimilar as shown in Figure 4.12(a); the rotation stiffness was the same up to 35 kN-m, but as the moment gradually increased to the ultimate moment 123.70 kN-m, the stiffness of the beam (VDT1 and VDT2) were observed to be lower than that of the full through-plate (VDT3). This difference may be due to the shear failure of the beam web. As through-plate was firmly welded to both sides of the steel tube, the rotation deformation of the through-plate was less than that of the beam; as a result, the stiffness of the full through-plate exceeded that of the beam.

For the half through-plate connection (ST3-3), the rotational stiffness of the beam (VDT1 and VDT2) was closer to that of the half through-plate (VDT3) as shown in Figure 4.12(b). The rotational stiffness was the same up to a moment of 40 kN-m; as the moment was gradually increased to the ultimate moment 128.0 kN-m, the stiffness of the beam (VDT1 and VDT2) was observed to be slightly below that of the half-through-plate (VDT3). This difference may be due to the buckling of the beam web and pulling-out of the half through-plate. As the half through-plate was welded only to one side of the steel tube wall, there was a possibility that the half through-plate would be pulled out of the concrete. The rotation deformation rate of the half through-plate was similar to that of the beam.

Figure 4.12(c) shows the test results of full through-plate connections (ST3-2) under cyclic loading. It is seen that the ultimate hogging capacity of ST3-2 was 115 kN-m, less than that for specimen ST3-1 (123.70 kN-m), despite the joint configurations of the two specimens being identical. This was caused by the material fatigue. However, the
ultimate sagging moment capacity (168 kN-m) of specimen ST3-2 was higher than its hogging moment capacity (115 kN-m), possibly because the tensile load generated by the sagging moment was carried by the through-plate whose tensile-resisting capacity was higher than the slab reinforcement when the slab reinforcement was in the hogging condition; the compressive load generated by the sagging moment was carried by the slab and the beam, whose compressive-resisting capacity were higher than either the beam web or the plate.

![Diagram](image)

(a) Specimen ST3-1: Full through-plate connection, static loading  
(b) Specimen ST3-3: Half through-plate connection, static loading  
(c) Specimen ST3-2: Full through-plate connection, cyclic loading

Figure 4.12 Moment-rotation behaviour of the through-plate connections tested under static monotonic loading and quasi-static cyclic loading
Table 4.2 Moment and rotation capacity of the through-plate connections

<table>
<thead>
<tr>
<th>Joint</th>
<th>Ultimate moment capacity (kN⋅m)</th>
<th>Rotation at ultimate moment (mrad)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Hogging moment</td>
<td>Sagging moment</td>
</tr>
<tr>
<td>ST3-1</td>
<td>123.7</td>
<td>–</td>
</tr>
<tr>
<td>ST3-2</td>
<td>115.0</td>
<td>168.0</td>
</tr>
<tr>
<td>ST3-3</td>
<td>128.0</td>
<td>–</td>
</tr>
</tbody>
</table>

4.3.3 Slip between slab and steel beam

Two linear potentiometers (LP) were used to measure the slip value; LP1 was placed on the northern side of the beam and LP2 was placed on the opposite (southern) side of the beam (see Figure 3.13, Chapter 3). Figure 4.13 shows the beam load vs. slip (between slab and beam) curves of the through-plate connections. It is seen that the slip at the ultimate loading of the joints was observed to be less than 0.05 mm for the full-through-plate connection (ST3-1) (Figure 4.13(a)) and 0.09 mm for the half-through-plate connection (ST3-3) (Figure 4.13(b)). In both test specimens, a high degree of the shear connectors, i.e. \( \eta_y = \frac{N_{sc}E_{sc,max}}{A_r f_{y,r}} = 300\% \), was adopted. From this it is concluded that the small slip value was observed between slab and steel beam although the higher degree of the shear connector was considered in both specimens. So, it can be concluded that the slip between the steel beam and concrete was negligible before the ultimate strength was obtained.

![Figure 4.13 Beam load vs. slip (between slab and beam) curves of the through-plate connections](image)

(a) Specimen ST3-1: Full through-plate connections

(b) Specimen ST3-3: Half through-plate connections
4.3.4 Through-plate deformation behaviour

The vertical deformation of the through-plate under static loading was recorded using two LVDTs (labelled as VDT3 and VDT4). To measure the deformation, the transducers were placed 125 mm from the column face. Their locations are shown in Figure 3.14 in Chapter 3. Figure 4.14 shows plots of beam load vs. through-plate deformation. The maximum vertical deformation at the corresponding ultimate load was observed for specimen ST3-3 half through-plate connection as shown in Figure 4.14(b). The reasons for the higher through-plate deformation in specimen ST3-3 could be due to the pulling-out of the half through-plate. As the full through-plate was firmly welded to both sides of the steel tube wall of the column, it was not possible to pull it out of the concrete. It was also observed that the stiffness of the half-through-plate was lower than for the full through-plate. This may be due to a sudden bond loss between the plate and the concrete causing it to be pulled out. From this it is concluded that the capacity to resist deformation for the full through-plate connection was higher than that of the half through-plate due to the pulling-out of the half through-plate.

(a) Specimen ST3-1: Full through-plate connection  (b) Specimen ST3-3: Half through-plate connection

Figure 4.14 Beam load vs. through-plate deformation curves of the through-plate connections (ST3-1 and ST3-3)

4.3.5 Strain developments in through-plate connection components

Strains developed in the through-plate connection components, including the steel beams, the through-plates and the steel tube wall were measured using strain gauges,
whose locations are shown in Figure 3.14, Chapter 3. The strain values obtained from the strain gauges are discussed in the following.

**(a) Strains in steel beam**

Six strain gauges (SG) were attached to the steel beam 160 mm away from the column face: two on each beam flange, one (SG1) on the top of the flange of the northern beam (and SG4 on the south beam) and one (SG3) on the bottom flange of the northern beam (and SG6 on the south beam). One strain gauge was attached to the middle of each beam web (SG2 on the northern beam and SG5 on the southern beam). Figure 4.15 shows the beam strains plotted against the beam load.

![Graph showing beam load vs. beam strain curves of through-plate connections](image)

(a) Specimen ST3-1 (Full through-plate connection)

![Graph showing beam load vs. beam strain curves of through-plate connections](image)

(b) Specimen ST3-3 (Half through-plate connection)

Figure 4.15 Beam load vs. beam strain curves of the through-plate connections
The yield strain of the flange and web were 1774 µε and 1816 µε, respectively, from the coupon tests for the beam flange and web. The maximum strains generated in the beam web for connection ST3-1 (full through-plate) at the ultimate beam load was 3615 µε in the northern beam web and 3279 µε in the southern beam web, indicating that the beam web yielded before reached the ultimate load (93.7 kN). For the half through-plate connection (ST3-3), the maximum tensile strains at the ultimate beam load was 594 µε in the northern beam web and 537 µε in the southern beam web. The beam web did not yield at the ultimate load (97 kN) for the half through-plate connection.

![Diagram](image_url)

(a) Specimen ST-3-1 (Full through-plate connection)

![Diagram](image_url)

(b) Specimen ST3-3 (Half through-plate connection)

Figure 4.16 Beam load vs. plate strain for through-plate connections
(b) Strains in through-plates

Figure 4.16 shows the strains in the plates of the through-plate connections. Figure 4.16(a) & (b) show that the tensile strains recorded by SG7, SG9, SG11 and SG13 were higher than the compressive strains as recorded by SG8, SG10, SG12 and SG14. The maximum tensile strains occurred at the top of the through-plate near the top bolt (ST3-1, ST3-3). For the full through-plate (ST3-1), the maximum tensile strain at the ultimate beam load was 2362 µε, which exceeded the yield strain (1808 µε). This indicates that the full through-plate yielded before the ultimate load (93.7 kN) was reached. For the half through-plate (ST3-3), the maximum tensile strain at the ultimate beam load was 1960 µε at its ultimate load (97 kN), slightly above the yield strain (1808 µε), indicating that the half through-plate began to yield at its ultimate load.

Figure 4.17 Beam load vs. tube wall strain curves of the through-plate connections
(c) Strains in steel tube

Figure 4.17 shows the longitudinal strains in the steel tube wall close to the through-plate. The yield strain of the square stainless steel tube wall was 1686 με, obtained from coupon tests of the steel tube. It is seen from Figure 4.17(a) and (b) that the strains in the tube wall were below the yield strain, indicating that the steel tube wall remained intact without any yielding.

4.4 Discussion on CFSST Column Joints with Blind bolted Endplate Connections

4.4.1 Effect of slab

Most of the blind bolted endplate connections investigated in other studies were tested without considering the effect of the concrete slab on the steel beam-column joints. The results of such tests are unrealistic and impractical to be used to evaluate the behaviour of beam-column joints. In the present study, the effect of the slab on the joints was investigated and compared to the behaviour when without a slab being present. Both of the joints (SB1-0, SB1-2) tested were identical blind bolted endplate connections with binding bars, but only one included a concrete slab. Figure 4.18 shows the moment vs. rotation behaviour for these two joints.

![Graphs showing moment vs. rotation and bolt slip curves for CFSST column joints with and without slab](image)

(a) Moment vs. joint rotation curves  (b) Moment vs. bolt slip curves

Figure 4.18 Comparative test results of CFSST column joints with and without slab
Figure 4.18(a) shows that the initial stiffness and ultimate moment capacity of the joint without considering the slab effect was very much lower than the joint with slab effect. In addition, the bolt slip from the concrete core of the CFSST column, as shown in Figure 4.18(b), was found to be higher for the joint having no slab (SB1-0). The failure modes of this joint were observed on the steel tube of the CFST column and endplate deformation due to bending moment. The joint with slab (SB1-2) failed due to the cracking of the concrete in the slab and fracture of the reinforcing steel. This indicates that the presence of the slab significantly influenced the behaviour of the composite steel beam-CFSST column joints.

4.4.2 Effect of column sections

Figure 4.19 shows a comparison between the test results of joints for different column cross-sections. It is seen in Figure 4.19 (a) that the initial stiffness and ultimate load capacity of the square-section CFSST column were greater than those for the circular-section column. Figure 4.19 (b) shows the moment vs. rotation behaviour of joints SB1-1 (square section) and CB2-1 (circular section).

(a) Axial column load vs. axial column deformation curves  
(b) Moment vs. joint rotation curves

Figure 4.19 Comparative test results of CFSST column joints with different column sections
The test results show that the ultimate moment capacity and stiffness of joints with circular columns exceeded those for joints with square columns, although the configuration of the joints (width (or diameter)-to-thickness ratio ($B/t$ or $D/t$) = 60, endplate thickness = 10 mm, total number of M20 blind bolts = 6) were identical. This difference is due to the less bending resisting capacity of the square tube wall compared to that of the circular tube wall, when subjected to similar bending moment. This was confirmed by the measured bolt slips in the two specimens as shown in Figure 4.19 (c).

4.4.3 Effect of binding bars
Outwards deformation was observed in the square steel tube of the CFST column when steel beams were joined to CFST column using endplate connections, and the tensile load or bending moment was transferred from the beam to the column (Hassan et al., 2013, 2014). This was severe when no slab was presented. To reduce this steel tube separation or outward deformation, binding bars were proposed by Hassan et al. (2013) who conducted a finite element analysis of steel beam–square CFST column joints without a slab (Hassan et al., 2013, 2014). The simulated results indicated that binding bars would be an effective method of overcoming the separation of the tube from the concrete core of the column (Hassan et al., 2015) and thus would enhance the integrity of the panel zone of beam-square column joints when no slab was present.
The comparative test results of joints SB1-1 (square column without binding bars), SB1-2 (square column with binding bars) and CB2-1 (circular column) are shown in Figure 4.20. The plots of moment vs. joint rotation in Figure 4.20(a) and moment vs. bolt slip in Figure 4.20(b) show that without binding bars, the ultimate moment capacity of specimen SB1-1 was 14% lower than that for CB2-1, but bolt slip was higher in specimen SB1-1. When binding bars were incorporated in SB1-2, the ultimate moment capacity and stiffness of the joint increased and bolt slip was reduced. It is also seen that the moment vs. rotation curve of joint SB1-2 (square) is very similar to that of joint CB2-1 (circular). This indicates that the binding bars reduced the outward deformation of the square tube and enhanced the moment capacity for this type of blind bolted connection to square CFSST column.

![Graphs](a) Moment vs. joint rotation curves (b) Moment vs. bolt slip curves

Figure 4.20 Comparative test results of CFSST column joints with and without binding bars

### 4.4.4 Effect of steel tube’s material type

Figure 4.21 shows comparative test results of joints with either stainless steel or carbon steel tubes. The steel tube of specimen CB2-1 was fabricated from stainless steel (367 MPa yield strength), whilst that of specimen CB2-3 was fabricated from carbon steel (379 MPa yield strength). The test results are shown in Figure 4.21. It was also observed that the axial behaviour of the stainless steel column was very similar to that of the carbon steel column (Figure 4.21(c)). In general, the material type in the steel
tube had no significant influence on the joint performance. This is owing to the fact that the joints’ failure was controlled by the failure of the slab.

![Moment vs. joint rotation curves](image1)

(a) Moment vs. joint rotation curves

![Moment vs. bolt slip curves](image2)

(b) Moment vs. bolt slip curves

![Column load vs. axial deformation of column](image3)

(c) Column load vs. axial deformation of column

Figure 4.21 Comparative test results of CFSF column joints with stainless steel tube and carbon steel tube

### 4.4.5 Effect of loading type

The effect of the loading type was investigated during the testing of specimens SB1-2 and CB2-1 (static monotonic load), and SB1-3 and CB2-2 (quasi-static cyclic load). Specimens SB1-2 and SB1-3 had square columns with binding bars; specimens CB2-1 and CB2-2 had circular columns without binding bars. Figure 4.22(a) for specimens SB1-2 and SB1-3 shows that the hogging moment for SB1-2 tested under static
monotonic loading was higher (264 kN-m) than those of SB1-3 tested under quasi-static cyclic loading (254.4 kN-m). This was due to the material fatigue resulted from cyclic loading. A similar discrepancy was observed between circular-tube specimens CB2-1 tested under static monotonic loading (277.2 kN-m) and CB2-2 tested under quasi-static cyclic loading (262 kN-m): see Figure 4.22(b). The sagging moment of specimen CB2-2 (152 kN-m) was found to be higher than that for specimen SB1-3 (81 kN-m). The higher sagging moment in specimen CB2-2 was due to the better resisting capacity provided by the circular tube. From these results it is concluded that the type of loading had some influence on the moment capacity of the blind bolted connections to both square and circular columns.

Figure 4.22 Comparative test results of CFSST column joints for static loading and cyclic loading
4.5 Discussion on CFSST Column Joints with Through-plate Connections

4.5.1 Effect of full through-plate or half through-plate

The effect of using full through-plate (specimen ST3-1) or half through-plate (specimen ST3-3) on the behaviour of the through-plate connections was investigated. The results are shown in Figure 4.23. The plots of moment vs. beam rotation for specimens ST3-1 and ST3-3 are shown in Figure 4.23(a); Figure 4.23(b) shows the plots of moment vs. plate rotation for both specimens. It was noted that the ultimate moment capacity of ST3-1 and ST3-3 were almost identical; however, the stiffness behaviour of beam and plate for the full through-plate connection were quite different from that for the half-through-plate connection. Figure 4.23(a) shows that the beam rotational stiffness of the full through-plate connection (ST3-1) was lower than that of the half through-plate connection (ST3-3) due to buckling of the beam web near the bolt hole. However, Figure 4.23(b) shows the reverse behaviour for the plate stiffness of the full through-plate connection (ST3-1) and half through-plate connection (ST3-3). It is seen in Figure 4.23(b) that the plate stiffness beyond a moment of 35 kN-m of the half through-plate connection (ST3-3) dropped when the half through-plate separated from the concrete core in the column.

Figure 4.23 Comparative test results of CFSST column joints for full through-plate connection and half through-plate connection
It was found that the behaviours of the full and half through-plates were very similar. During static testing, both failed in a same manner under similar conditions. This suggests that there is no need to continue to use the more complex full through-plate connection in composite construction, as it brings no significant benefit to the overall structural performance of the joint. The installation of a full through-plate connection is a more involved construction process than the half through-plate. Meanwhile, a full through-plate connection uses more material, which is therefore less economical than a half through-plate connection.

4.5.2 Effect of loading type

Figure 4.24 shows the effect of the type of loading on through-plate connections. A comparison of the test results for specimens ST3-1 (full through-plate under static loading) and ST3-2 (full through-plate under cyclic loading) shows that the cyclic loading has no adverse effect on the initial stiffness, but it has effect on the ultimate moment capacity and rotation. It is evident that ST3-2 failed prematurely after attaining only 92.96% of the moment capacity of specimen ST3-1, yet recording 8.2% larger deflection. This further demonstrates the considerable cumulated damage generated by the nature of the cyclic loading.

Figure 4.24 Comparative test results of CFSST column joints with full through-plate connections tested under static loading (ST3-1) and cyclic loading (ST3-2)
4.6 Joint Classification According to Eurocode 3

Figure 4.25 shows the comparative test results of column joints with different types of connection. It can be seen from Figure 4.25 that the initial rotational stiffness of the through-plate connections ST3-1 and ST3-2 was almost the same as those of the blind-bolted endplate connections without binding bars (SB1-1) or with binding bars (SB1-2). However, the ultimate moment capacity of the blind-bolted endplate connections (i.e., 264.0 kN-m for SB1-2) was more than double that of the through-plate connections (i.e., 123.7 kN-m for ST3-1).

In the blind-bolted endplate connections, joints failed due to the fracture of the concrete slab reinforcement. Through-plate joints failed due to the through-plate twisting and bearing deformation of the bolt holes, although the beam and column were identical for both types of connections. This occurred because of the different configurations of the connections. In through-plate connections, the load was carried by the beam web and through-plate (whether full through-plate or half through-plate). Beyond a certain load, the beam web started to buckle and bolt-hole bearing deformation occurred because the beam load was transferred by means of the beam web to the through-plate through bolts. Conversely, in blind-bolted connections, as the beam web and flanges were welded to the endplate, the beam web did not buckle due to the better load-transfer mechanism at the presence of the endplate. A conclusion can be drawn that blind bolted endplate connection is superior to through-plate connection in terms of the stiffness and hogging moment.

![Figure 4.25 Comparative test results of CFSST column joints for different type of connections](image-url)

Figure 4.25 Comparative test results of CFSST column joints for different type of connections
4.6.1 Classification for blind-bolted joint

Test data of blind-bolted joints were compared, to assess the behaviour of beam-CFST column joints with blind bolted endplate connections, with the boundary limits (stiffness and strength) that were calculated from Eurocode 3 Part 1-8 (2005) for rigid connections and pinned connections. Eurocode 3 specifies the stiffness of the rigid joint to be more than eight times the beam stiffness \( (> 8K_b) \) for braced frame and greater than 25 times the beam stiffness \( (> 25K_b) \) for unbraced frames. For pinned joints, the initial rotational stiffness of a joint is less than 0.5 times the beam stiffness \( (< 0.5K_b) \). When the initial rotational stiffness of a joint lies between the stiffness criteria for pinned joints and rigid joints, the joint is classified as a semi-rigid joint. Beam stiffness is determined from \( K_b = EI_b/L_b \), where \( EI_b \) is the flexural stiffness of the composite beam and \( L_b \) is the beam span length (columns centre-to-centre). Beam length was assumed to be 6 m for low-rise residential building structures (Lee et al., 2010) and 8.5 m for high-rise buildings. Eurocode 3 also specifies the boundary limit for a full-strength connection as the ultimate moment capacity of the connection \( (M_u) \) is greater than the beam plastic moment \( (M_{p,beam}) \). For non-composite beam without slab, the value of \( M_{p,beam} \) is determined from \( M_{p,beam} = A_f f_{y,f} (h_w - t_f) + 0.25 A_w f_{y,w} h_w \); where \( f_{y,f}, A_f \) and \( t_f \) are the yield stress, area and thickness of the beam flange respectively; and \( f_{y,w}, A_w \) and \( h_w \) are the yield stress, area and height of the beam web respectively. For composite beam with slab, the value of \( M_{p,beam} \) is determined from \( M_{p,beam} = F_b d_b + F_{ss} d_{ss} + F_r d_r \); where \( F_b, F_{ss} \) and \( F_r \) are the total resisting forces of the steel beam, profile sheet and reinforced bars respectively; and \( d_b, d_{ss} \) and \( d_r \) are the distances from the plastic neutral axis to the steel beam, profile sheet and reinforced bars respectively. The plastic moment capacity of composite beam is observed higher than those of the non-composite beam; due to the presence of the slab reinforcement and profile sheet in the composite beam. A calculation detail is illustrated in Appendix B.2.

The boundary limit values in terms of stiffness and strength are compared with the test results as shown in Figure 4.26 for blind-bolted joint without slab and in Figure 4.27 for blind-bolted joint with slab. It can be seen from Figure 4.26(a) and (b) for joint without slab (SB1-0) that the initial rotational stiffness was very lower to the criterion for rigid braced and unbraced frames. However, the initial rotational stiffness of joints with slab (SB1-2, CB2-1) a shown in Figure 4.27(a) and (b) was very close to the
criterion for rigid braced and unbraced frames, even the ultimate capacity was close to the boundary limit of a full-strength connection. It is noticed that the initial stiffness and ultimate moment capacity of the joint with slab (SB1-2) is much higher than those of the joint without slab (SB1-0), although the configuration of both joint except slab is same. It indicates that the presence of slab to the joint has significant effect in the joint types. The current test results indicate that blind-bolted connections should be treated as semi-rigid connections to rigid connection if the contribution from the concrete slab is considered.

![Classification with beam span of 6 m](image1)

![Classification with beam span of 8.5 m](image2)

Figure 4.26 Classifications of joint without slab for blind bolted endplate connections

![Classification with beam span of 6 m](image3)

![Classification with beam span of 8.5 m](image4)

Figure 4.27 Classifications of joint with slab for blind bolted endplate connections
4.6.2 Classification for through plate joint

Test data of through plate joints were compared, to assess the behaviour of beam-CFST column joints with through-plate connections, with the boundary limits that were also calculated from Eurocode 3 Part 1-8 (2005) for rigid connections and pinned connections. The calculation detail of beam plastic moment capacity is illustrated in Appendix B.2. Beam length was assumed to be 6 m for low-rise residential building structures (Lee et al., 2010) and 8.5 m for high-rise buildings.

The boundary limit values in terms of stiffness and strength are compared with the test results as shown in Figure 4.28. It can also be seen from Figure 4.28(a) and (b) that, for the through-plate connections ST3-1 and ST3-3, initial rotational stiffness closely matched the stiffness of a rigid connection for braced frames; however, their ultimate moment was around 56% lower than the moment capacity for a full-strength connection. In engineering practice, through-plate connections are often simplified as pinned connections. The current test results indicate that through-plate connections should be treated as semi-rigid connections if the contribution from the concrete slab is considered.

![Figure 4.28](image)

(a) Classification with beam span of 6 m  
(b) Classification with beam span of 8.5 m

Figure 4.28 Classifications of joint with slab for through-plate connections
4.7 Summary

This chapter summarises the results of tests on 10 specimens of hybrid composite beam-column joints using blind-bolted endplate connections and through-plate connections. It was seen from the test results of the blind-bolted endplate connections that the moment capacity of the connections without the presence of a concrete slab was very low (seven times lower) compared to the moment capacity of the connections when the slab was provided. The moment capacity of the blind-bolted endplate connections with slab was found to be double that of through-plate connections with slab. Blind-bolted endplate connections without slab failed due to outward deformation of the steel tube. This was observed to be due to the thin steel tube used in CFSST column, and separation of the steel tube from the concrete core by pull-out of the blind bolts. However, the presence of the slab on the blind-bolted endplate connections had a significant effect on the failure mode of joints. Blind-bolted endplate connections with slab failed due to the slab reinforcement fracture. It was also seen that the moment capacity of blind-bolted endplate connections for square-section CFSST column was observed to be 14% lower compared to blind-bolted endplate connections for circular-section CFSST column. The addition of binding bars to the square-section steel tube column, enhanced the ultimate moment capacity of blind-bolted endplate connections to square-section CFSST column by only 4.7% lower compared to connections using circular-section CFSST columns without binding bars. Blind-bolted endplate connections with slab were tested under static and cyclic loading. The hogging moment capacity at cyclic loading was 3.6% lower for square column and 5.4% lower for circular column, than connections tested under static monotonic loading. The sagging moment capacity of joints tested under cyclic loading was also observed to be 68% less than the hogging moment for square columns, and 42% less for circular columns. The sagging moment reduction was observed to be the result of separation of the steel tube from the concrete near the bottom flange of the beam due to upward loading.

Based on the test results and stiffness classifications proposed by Eurocode 3 Part 1-8 (2005), blind-bolted endplate connections may be classified as rigid connection for braced frames and semi-rigid connection for unbraced frames, and can be adopted in high-rise building structures. Through-plate connections can be considered as semi-rigid connections.
CHAPTER 5

FE MODEL DEVELOPMENT AND VALIDATION FOR BLIND-BOLTED JOINTS WITH CFST COLUMNS

5.1 Introduction

Limited experimental studies of blind-bolted endplate connections to join steel beams to CFST columns have been conducted in the past in which the behaviour of different components of the joints was investigated. As experimental investigations are expensive and time-consuming, the development of finite element (FE) models, well validated against experimental data, would be a useful alternative solution to overcome any experimental limitations to explore the detailed behaviour of CFST column joints, as well as the behaviour of Hollo-Bolts under tension and shear load.

This chapter mainly demonstrates the development of FE modelling details of the different types of connections used in CFST column joints. In FE model development, the details of material modelling, element types, blind-bolt modelling technique, contact modelling, load and boundary conditions for non-linear analysis are illustrated. In material modelling, the full-range stress-strain curves of the steel tube, endplate, steel beam and blind-bolt are proposed to simulate the material and to capture its full behaviour including necking and failure. The developed FE model is verified against existing test results collected from the literature as well as the results of test conducted in this study. Three types of test data are collected: (1) blind bolt tests in pure tension and shear loading, (2) CFST column joints without slab tested in tension or bending loading, and (3) CFST column joints with slab tested in bending loading. The FE modelling details and its validation are illustrated step by step in the following sections.

5.2 FE Model Development for CFST Column Joints

To develop an FE model for CFST column joints, ABAQUS software (ABAQUS, 2012) was used to model the geometry and material nonlinearity, contact behaviour, and boundary and load conditions. The main components of the composite CFST
column joints are the column (steel tube and concrete core), composite steel beam (steel beam with shear studs); composite slab (profiled sheeting, slab reinforcement, slab concrete); endplates and blind-bolts. The details of material modelling, element types, geometrical modelling of blind-bolts, contact interaction, boundary and load conditions and analysis method are illustrated in the following subsections.

5.2.1 Material modelling

The material properties for various components of the steel beam-CFST column joints were determined from the material test data in the form of the engineering stress \( \sigma_{\text{eng}} \) and strain \( \varepsilon_{\text{eng}} \). In the FE analysis, the material property has to be defined by the use of the true stress and the true plastic strain relationships (ABAQUS, 2012). The values of the true stress \( \sigma_{\text{true}} \) and the true plastic strain \( \varepsilon_{\text{pl, true}} \) are determined from the engineering stress and logarithmic strain relationship using the equation (5.1):

\[
\sigma_{\text{true}} = \sigma_{\text{eng}} \left(1 + \varepsilon_{\text{eng}}\right); \quad \varepsilon_{\text{pl, true}} = \ln \left(1 + \varepsilon_{\text{eng}}\right) \frac{\sigma_{\text{true}}}{E}
\]

where \( E \) is the elastic modulus. The stress-strain relationship models used in FE analysis to define the material behaviour of concrete, structural steel sections and structural bolts are illustrated below.

(a) Material model of concrete

Concrete is a complex non-homogeneous material compared to other structural materials such as steel. The behaviour of this complex material can be assigned in ABAQUS using either ‘Concrete smeared cracking model’ or ‘Drucker Prager model’ or ‘Concrete damaged plasticity model’. The concrete damaged plasticity model was used throughout the FE analysis. As the concrete damage plasticity model is the only model for concrete that assumes two failure mechanisms, tensile cracking and compressive crushing. The concrete damaged plasticity model was developed by Lubliner et al. (1989), in which ‘damaged plasticity’ is used to characterise the uniaxial tensile and compressive behaviour of concrete. Four major material parameters are used in the concrete damaged plasticity model: the dilation angle \( \psi \), eccentricity \( e \), ratio of the compressive strength under biaxial loading to uniaxial compressive strength \( f_{b0}/f_{c0} \) and ratio of the second stress invariant on the tensile meridian to that on the
compressive meridian ($K_c$). The default values of $30^\circ$, 0.1, 1.16 and 0.67 for $\psi, e, K_c, f_{b0}/f_{c0}$ respectively have been used in many studies. In simulating CFST columns under axial compression, $e$ was recommended to be taken as 0.1, and suitable equations were proposed by Tao et al. (2013) to determine values of $\psi, K_c, f_{b0}/f_{c0}$. Based on sensitivity analysis, it is found that equations of $f_{b0}/f_{c0}$ and $K_c$ proposed by Tao et al. (2013) are still suitable for use in simulating blind-bolted joints with CFST columns. However, it is found that the simulation results are very sensitive to the selection of dilation angle ($\psi$) used for concrete in blind-bolted joints with CFST column. The sensitivity analyses results in Figure 5.1 show that the load-carrying capacity of blind-bolted joints (Wang et al., 2009b) tends to be over-predicted if higher $\psi$-values are used in the simulation of CFST column. This is probably due to the fact that a higher $\psi$ can lead to higher dilation of concrete, which will delay the damage of concrete under the pulling-out force of blind bolts. From the sensitivity analysis, it is seen that when a dilation angle of $25^\circ$ was used for CFST column, the FE model results closely matched the test results. Therefore the value $\psi$ of $25^\circ$ was adopted throughout the current analysis.

The elasticity modulus of concrete ($E_c$) is calculated from $4700\sqrt{f_c'}$ as recommended in ACI 318 (2011). Poison's ratio ($\nu_c$) is equal to 0.2. The compressive behaviour and tensile behaviour of concrete are assigned in ABAQUS based on the following stress-strain relationships.

![Figure 5.1 Influence of dilation angle ($\psi$) on the load-deformation curves of blind-bolted endplate steel beam-CFST column joints](image-url)
(1) **Concrete compressive behaviour**

The compressive behaviour of concrete was assigned using a nonlinear compressive stress–strain relationship. In composite joints, two types of concrete are identified: one is unconfined concrete used in reinforced concrete (RC) slabs and RC beams; and confined concrete is used in concrete-filled steel tubular (CFST) columns. The stress–strain relationship of confined concrete differs from that of unconfined concrete. The strain-hardening of both types of concrete are found to be the same, but their softening behaviour is quite different.

(i) **Stress-strain relationship of unconfined concrete**

The stress ($\sigma$)–strain ($\varepsilon$) relationship of unconfined concrete used in composite slabs and RC beams has been reported by different researchers in different ways. Among those material models, Carreira and Chu (1985) provided the following formula, which was adopted throughout the modelling of the slab concrete material.

Carreira and Chu (1985) proposed the following unconfined concrete model:

$$\sigma = \frac{f'_c}{\beta - 1 + \left(\frac{\varepsilon'_c}{\varepsilon'_c}\right)^\beta}$$

$$\beta = \left|\frac{f'_c}{32.4}\right|^3 + 1.55$$

$$\varepsilon'_c = (0.71f'_c + 168) \times 10^{-5}$$

where $\varepsilon'_c$ is the ultimate strain when the concrete cylinder strength is attained, and $f'_c$ is the concrete cylinder compressive strength. The values of $\varepsilon'_c$ and $\beta$ is determined from Eqs. (5.3) and (5.4)

(ii) **Stress-strain relationship of confined concrete**

Different types of stress–strain relationship models for confined concrete are found in literature to simulate the compressive behaviour of confined concrete. These include models proposed by Eurocode (2005), Mander et al. (1984), Han et al. (2008) and Tao et al. (2013). Han et al.’s model is practical for confined concrete in which the confinement effect provided by the steel tube to the concrete core in CFST columns was considered. The confinement effect depends on the cross-sectional area of the steel
and concrete, the yield strength of the steel and the characteristic strength of concrete. Tao et al. (2013) proposed a model that differed from Han et al.'s model. In Tao et al.'s model, the peak stress of the confined concrete was taken as the compressive strength of the unconfined concrete. Any strength increase of concrete from confinement can be captured in the FE simulation through the interaction between the steel tube and concrete. In his model, three parts were considered to represent the stress-strain behaviour of concrete confined by steel. The first part i.e. strain hardening (“OA” as shown in Figure 5.2) represents the unconfined concrete behaviour up to its strength ($f'_c$) reached. At this stage the interaction between the steel and the concrete in the CFST column is very small or negligible. After it reaches its peak strength ($f'_c$), contact pressure and interaction develop between the steel and concrete and then there is some confined effect observed in the concrete due to the increased peak strain of concrete. This strain behaviour is defined in the second part of the model (“AB” as shown in Figure 5.2). The strain softening for confined concrete (“BC” as shown in Figure 5.2) is the third part.

![Figure 5.2 Stress-strain model of confined concrete proposed by Tao et al. (2013)](image)

The full stress-strain model of confined concrete as proposed by Tao et al. (2013) was adopted in this study. The concrete model is expressed by Eq. (5.5).

$$
\sigma = \begin{cases} 
\frac{A \varepsilon + B \varepsilon^2}{1 + (A-2) \varepsilon + (B+1) \varepsilon^2} f'_c & 0 < \varepsilon \leq \varepsilon_{co} \\
\varepsilon_{co} < \varepsilon < \varepsilon_{cc} \\
f_r + (f'_c - f_r) \exp \left[ -\left( \frac{\varepsilon - \varepsilon_{cc}}{\varepsilon_c} \right)^\beta \right] & \varepsilon \geq \varepsilon_{cc}
\end{cases}
$$

(5.5)
where \( f_c' \) is the concrete cylinder compressive strength (N/mm\(^2\)); \( f_r \) is the residual stress (N/mm\(^2\)); \( X = \frac{e}{\varepsilon_{co}} \); \( A = \frac{E \varepsilon_{co}}{f_c^2} \); \( B = \frac{(A-1)^2}{0.55} - 1 \); \( \varepsilon_{co} \) is peak strain of unconfined concrete and \( \varepsilon_{cc} \) is the peak strain of confined concrete.

The values of \( \varepsilon_{co}, \varepsilon_{cc}, f_r, \alpha, \beta \) are determined by equations as follows.

\[
\varepsilon_{co} = 0.00076 + \sqrt{(0.626 f_c' - 4.33) \times 10^{-7}} \tag{5.6}
\]

\[
\varepsilon_{cc} = \varepsilon_{co} e^k \tag{5.7}
\]

\[
k = (2.9224 - 0.00367 f_c') \left( \frac{f_B}{f_c'} \right)^{0.3124 + 0.002 f_c'} \tag{5.8}
\]

\[
f_r = \begin{cases} 
0.7 (1 - e^{-1.38 \xi_c}) f_c' \leq 0.25 f_c' & \text{for circular CFST column} \\
0.1 f_c' & \text{for rectangular CFST column} 
\end{cases} \tag{5.9}
\]

\[
\alpha = \begin{cases} 
0.04 - \frac{0.036}{1 + e^{6.08 \xi_c - 3.49}} & \text{for circular CFST column} \\
0.005 + 0.0075 \xi_c & \text{for rectangular CFST column} 
\end{cases} \tag{5.10}
\]

\[
\beta = 1.2 \text{ for circular columns and } 0.92 \text{ for rectangular columns} \tag{5.11}
\]

Here, \( f_B \) is the confining stress and \( \xi_c \) is the confinement factor. The value of \( f_B \) and \( \xi_c \) are calculated from:

\[
f_B = \begin{cases} 
\frac{(1 + 0.027 f_y) e^{-0.02 D}}{1 + 1.6 e^{-10} (f_c')^{4.8}} & \text{for circular CFST column} \\
0.25 \frac{(1 + 0.027 f_y) e^{-0.02 \sqrt{B^2 + D^2} / t}}{1 + 1.6 e^{-10} (f_c')^{4.8}} & \text{for rectangular CFST column} 
\end{cases} \tag{5.12}
\]

\[
\xi_c = \frac{A_s f_y}{A_c f_c'} \tag{5.13}
\]

where \( D \) is the diameter of the circular column or cross-sectional depth of the rectangular column, and \( B \) is the width of the rectangular column; \( t \) is the thickness of the steel tube; \( f_y \) is the yield strength of steel; \( A_s \) is the cross-sectional area of the steel tube; and \( A_c \) is the cross-sectional area of the concrete.

(2) **Concrete tensile behaviour**

The tensile behaviour of concrete is assigned in ABAQUS by considering the tension softening, defined as a phenomenon of concrete when the concrete between cracks
continues to carry tensile stresses and offer stiffness. The tension softening of concrete is specified either by means of a post-failure stress–strain relationship or by applying a fracture energy cracking criterion (Han et al., 2007 & 2008). The post-failure stress–strain relationship is widely used for unconfined concrete in reinforced members. For unconfined concrete in tension, the tensile stress is assumed to increase linearly with strain until the concrete cracks, after which the tensile stress decreases linearly to zero at a strain of 16 times the strain at cracking (total tensile strain \( \varepsilon_t = 16\varepsilon_{cr} \)), as shown in Figure 5.3 (a). In the model proposed by Massicotte et al. (1990), there are three parts: the uncracked stage (OA), the primary cracking stage (AB) and the secondary cracking stage (BC). Using this equation, convergence during FE analysis is raised. To overcome this problem, a simplicity curve as shown in Figure 5.3 (a) is adopted in this study to determine the full uniaxial tensile stress–strain relationship (Eq. 5.14).

\[
\sigma_{ct} = \begin{cases} 
\varepsilon E_c & 0 < \varepsilon < \varepsilon_{cr} \\
 f'_{ct} \exp \left[ -\left( \frac{\varepsilon - \varepsilon_{cr}}{\alpha} \right)^{\beta} \right] & \varepsilon \geq \varepsilon_{cr}
\end{cases}
\] (5.14)

where \( f'_{ct} \) is the tensile strength of the concrete and \( \varepsilon_{cr} \) is the cracking strain at the tensile strength of concrete determined as \( f_{ct}'/E_c \), and \( \sigma_{ct} \) is the stress at the strain \( \varepsilon \). The values of \( \alpha \) & \( \beta \) are considered as \( 3.5\times10^{-4} \) and 0.85 respectively.

For confined concrete in tension, the tension stiffening can be specified by applying a fracture energy cracking criterion (Han et al., 2007 & 2008). The fracture energy criterion was used in this study for the CFST column only and calculated from the following expression presented by Tao et al. (2013):
\[ G_p = (0.0469d_{\text{max}}^2 - 0.5d_{\text{max}} + 26) \left( \frac{f_c}{10} \right)^{0.7} \text{ (N/m)} \]

where \( \gamma = 1.25d_{\text{max}} + 10 \) and \( d_{\text{max}} \) is the maximum diameter of coarse aggregate used in concrete mixing, usually assumed to be 20 mm.

(b) Material model of mild carbon steel

Mild carbon steel is the most common form of steel, often used when large quantities of steel are needed, for example structural steel. It contains approximately 0.16–0.29% carbon. Structural steel sections such as steel tubes, steel beams and endplates are used in the construction of endplate connections to join steel beams to CFST columns. The density of mild carbon steel is approximately 7850 kg/m\(^3\) and its elastic modulus varies from 200 to 210 GPa. The Poisson’s ratio of all structural steels is taken as 0.30. There are many material models used to describe the constitutive behaviour of steel (Eurocode 3, 2003; Uy, 1998; Loh et al., 2004; Wang et al., 2012; Tao et al., 2013). Tao et al. (2013) developed the following stress-strain curve for structural steel (Figure 5.4):

\[
\sigma = \begin{cases} 
E_s \varepsilon & 0 \leq \varepsilon < \varepsilon_y \\
\frac{f_y}{f_u} (\varepsilon - \varepsilon_y) & \varepsilon_y \leq \varepsilon < \varepsilon_p \\
\left( \frac{f_u - f_y}{f_u} \right) \left( \frac{\varepsilon - \varepsilon_y}{\varepsilon_p} \right)^p & \varepsilon_p \leq \varepsilon < \varepsilon_u \\
\frac{f_u}{f_y} & \varepsilon \geq \varepsilon_u 
\end{cases} 
\]

(5.15)

where \( \sigma \) is the engineering stress; \( \varepsilon \) is the engineering strain; \( \varepsilon_y \) is the yield strain \( (\varepsilon_y = f_y/E_s) \); \( \varepsilon_p \) is the strain at the onset of strain hardening; \( \varepsilon_u \) is the strain at ultimate strength \( (f_u) \); and \( p \) is the strain-hardening exponent determined from:

\[
p = E_p \left( \frac{\varepsilon_u - \varepsilon_p}{f_u - f_y} \right) 
\]

(5.16)

in which \( E_p \) is the initial modulus of elasticity at the onset of strain-hardening. The values of \( E_p, \varepsilon_p \) & \( \varepsilon_u \) are determined from Eqs. (5.17), (5.18) and (5.19) respectively.

\[
E_p = 0.02E_s 
\]

(5.17)

\[
\varepsilon_p = \begin{cases} 
15 \varepsilon_y & f_y \leq 300 \text{ MPa} \\
[15 - 0.018(f_y - 300)]\varepsilon_y & 300 \text{ MPa} < f_y \leq 800 \text{ MPa} \\
100\varepsilon_y & f_y \leq 300 \text{ MPa} 
\end{cases} 
\]

(5.18)

\[
\varepsilon_u = \begin{cases} 
[100 - 0.15(f_y - 300)]\varepsilon_y & 300 \text{ MPa} < f_y \leq 800 \text{ MPa} 
\end{cases} 
\]

(5.19)
Figure 5.4 Stress-strain model proposed by Tao et al. (2013) for structural steel sections

In this model, three parameters ($E_s, f_y, f_u$) are mainly required to determine the stress-strain curve. Tao et al. (2013) proposed the following expression for $f_u$:

$$f_u = \begin{cases} \left[ 1.6 - 2 \times 10^{-3} (f_y - 200) \right] f_y & 200 \text{ MPa} < f_y \leq 400 \text{ MPa} \\ \left[ 1.2 - 3.75 \times 10^{-4} (f_y - 400) \right] f_y & 400 \text{ MPa} < f_y \leq 800 \text{ MPa} \end{cases} \quad (5.20)$$

It should be noted that the model in Figure 5.4 only includes three parts: elastic, yielding and hardening. To accurately simulate the failure of structures, the material degradation of steel, i.e., necking and failure, should be considered.

Based on the Tao et al. (2013)'s model, a full-range stress-strain curve shown in Figure 5.5 is proposed with a failure stage as follows:

$$\sigma = \begin{cases} E_s \varepsilon & 0 \leq \varepsilon < \varepsilon_y \\ f_y & \varepsilon_y \leq \varepsilon < \varepsilon_p \\ f_u - (f_u - f_y) \left( \frac{\varepsilon - \varepsilon_y}{\varepsilon_u - \varepsilon_p} \right)^p & \varepsilon_p \leq \varepsilon < \varepsilon_u \\ f_u & \varepsilon_u \leq \varepsilon < \varepsilon_{u1} \\ f_u - (f_u - f_t) \left( \frac{\varepsilon - \varepsilon_{u1}}{\varepsilon_{u2} - \varepsilon_{u1}} \right)^{p'} & \varepsilon_{u1} \leq \varepsilon < \varepsilon_t \\ f_t - f_t \left( \frac{\varepsilon - \varepsilon_t}{\varepsilon_{u2} - \varepsilon_t} \right) & \varepsilon_t \leq \varepsilon \leq \varepsilon_{u2} \end{cases} \quad (5.21)$$

where $\varepsilon_u$ is the strain at ultimate strength $f_u$; $\varepsilon_{u1}$ is the strain at the onset of strain softening; $\varepsilon_t$ is the strain at fracture stress $f_t$; $\varepsilon_{u2}$ is the total strain; $p$ is the strain-hardening exponent; and $p'$ is the strain-softening exponent.
The values of $p$, $p'$ and $f_t$ are given by:

$$p = E_p \left( \frac{\epsilon_u - \epsilon_p}{f_u - f_y} \right)$$  \hspace{1cm} (5.22)

$$p' = E_{u1} \left( \frac{\epsilon_t - \epsilon_{u1}}{f_u - f_t} \right)$$  \hspace{1cm} (5.23)

$$f_t = f_u - (\epsilon_t - \epsilon_{u1}).E'_{u1}$$  \hspace{1cm} (5.24)

in which $E_p$ is the elastic modulus at the onset of strain-hardening, and $E_{u1}$ is the elastic modulus at the onset of strain-softening. The value of $E'_{u1} = aE_{u1}$ can be determined from the elastic modulus at the onset of strain-softening, where $a$ is a fracture stress adjustment factor.

Test data of full-range stress-strain curves is used to determine key parameters, such as $\epsilon_p, \epsilon_u, \epsilon_{u1}, \epsilon_t, \epsilon_{u2}, E_p, E_{u1}, E'_{u1}$. These parameters are required to calculate $p$, $p'$ and $f_t$, in which $p$ and $p'$ are very important to define the strain-hardening part and strain-softening part of the curve. Based on regression analysis, it is found that $\epsilon_u, \epsilon_{u1}, \epsilon_t$ and $\epsilon_{u2}$ are functions of yield stress ($f_y$) and strain ($\epsilon_y$), whilst $E_p$, $E_{u1}$ and $E'_{u1}$ are related to the initial elastic modulus ($E_s$). The values of $\epsilon_p, \epsilon_u, E_p$ are determined from Eqs. (5.17), (5.18) and Eq. (5.19) respectively. By analysing the test data collected from the coupon tests in this study, expressions for $\epsilon_{u1}, \epsilon_t, \epsilon_{u2}$, $E_{u1}$, and
$E'_u$ are proposed and presented in Table 5.1, based on the trial and error method and best-fit curves.

Table 5.1 Parameters used in the stress-strain curve of structural steel sections

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Expressions</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\varepsilon_p$</td>
<td>$\begin{cases} 15 \varepsilon_y \ (15 - 0.018(f_y - 300))\varepsilon_y \end{cases}$ $f_y \leq 300$ MPa</td>
</tr>
<tr>
<td>$\varepsilon_u$</td>
<td>$\begin{cases} 100 \varepsilon_y \ (100 - 0.15(f_y - 300))\varepsilon_y \end{cases}$ $300$ MPa $&lt; f_y \leq 800$ MPa</td>
</tr>
<tr>
<td>$\varepsilon_u1$</td>
<td>$\begin{cases} 110 \varepsilon_y \ (110 - 0.18(f_y - 300))\varepsilon_y \end{cases}$ $300$ MPa $&lt; f_y \leq 800$ MPa</td>
</tr>
<tr>
<td>$\varepsilon_t$</td>
<td>$\begin{cases} 145\varepsilon_y \ (145 - 0.25(f_y - 300))\varepsilon_y \end{cases}$ $300$ MPa $&lt; f_y \leq 800$ MPa</td>
</tr>
<tr>
<td>$\varepsilon_u2$</td>
<td>$\begin{cases} 167\varepsilon_y \ (167 - 0.3(f_y - 300))\varepsilon_y \end{cases}$ $300$ MPa $&lt; f_y \leq 800$ MPa</td>
</tr>
<tr>
<td>$E_p$</td>
<td>0.02 $E_s$</td>
</tr>
<tr>
<td>$E_{u1}$</td>
<td>2.2 $E_s$</td>
</tr>
<tr>
<td>$E'_{u1}$</td>
<td>0.15 $E_s$</td>
</tr>
</tbody>
</table>

The proposed stress-strain relationship model was validated by comparing with coupon material test data for structural steel sections. In the proposed model, only three parameters, yield strength ($f_y$), ultimate strength ($f_u$), and elastic modulus ($E_s$) are required to determine the full-range stress-strain curve. The comparisons in Figure 5.6 indicate a good correlation between test results and model results.

(a) Coupon-1 of 310UB40.4 flange  
(b) Coupon-2 of 310UB40.4 flange

Figure 5.6 Comparison between proposed model and test data for structural steels obtained from coupon tests in this study
Figure 5.6 Comparison between proposed model and test data for structural steels obtained from coupon tests in this study (continued)
Figure 5.6 Comparison between proposed model and test data for structural steels obtained from coupon tests in this study (continued)

(c) Material model of stainless steel
Stainless steel is an iron-based steel alloy, containing at least 10.5–11% chromium content by mass which gives it corrosion-resistant properties. The stress-strain behaviour of stainless steel differs from that of carbon steels in a number of respects. The most important difference is in the shape of the stress-strain curve (Figure 5.7). Whereas carbon steel typically exhibits linear elastic behaviour up to the yield stress and a plateau before strain hardening, stainless steel has a more rounded response, with
no well-defined yield stress. The yield strengths of stainless steel are measured as 0.2% proof strength at an offset permanent strain of 0.2%. Many studies (Ramberg and Osgood, 1943; Rasmussen, 2003; Garder and Nethercot, 2004a; Quach et al., 2008) have proposed material models for stainless steel. It was found that Ramberg and Osgood’s (1943) model predicts stainless steel stress-strain behaviour accurately up to the 0.2% proof stress ($\sigma_{0.2}$), but it is very inaccurate for stresses beyond that level. To model the stress-strain behaviour accurately, Rasmussen (2003) proposed a full-range stress-strain relationship, adopting Ramberg and Osgood’s (1943) model up to the proof stress ($\sigma_{0.2}$), and adding a new expression for the proof stress-to-ultimate strength stage ($\sigma_u$).

Figure 5.7 Typical stress-strain curve of stainless steel

The full-range stress-strain curve proposed by Rasmussen (2003) is given in Eq. (5.25).

$$
\varepsilon = \begin{cases} 
\frac{\sigma}{E_0} + 0.002 \left( \frac{\sigma}{\sigma_{0.2}} \right)^n & \sigma \leq \sigma_{0.2} \\
\frac{\sigma - \sigma_{0.2}}{E_{0.2}} + \varepsilon_u \left( \frac{\sigma - \sigma_{0.2}}{\sigma_u - \sigma_{0.2}} \right)^m & \sigma > \sigma_{0.2}
\end{cases}
$$

(5.25)

where $\sigma$ and $\varepsilon$ are engineering stress and strain respectively; $E_0$ is the initial elastic modulus; $\sigma_{0.2}$ is the material 0.2% proof stress and $n$ is the strain hardening, which represents the shape of the stress-strain curve, determined by $\sigma_{0.2}$ and 0.1% proof stress ($\sigma_{0.1}$) calculated from:

$$
n = \frac{\ln(20)}{\ln \left( \frac{\sigma_{0.2}}{\sigma_{0.1}} \right)}
$$

(5.26)
The value of $E_{0.2}$ (stiffness at 0.2% proof stress), $e$, $\sigma_u$ (ultimate strength), $\varepsilon_u$ (strain at ultimate strength), $\varepsilon_{0.2}$ (strain at 0.2% proof stress) and $m$ are calculated from the following expressions:

$$E_{0.2} = \frac{E_0}{1 + 0.002\frac{n}{e}}$$  \hspace{1cm} (5.27)

$$e = \frac{\sigma_{0.2}}{E_0}$$  \hspace{1cm} (5.28)

$$\frac{\sigma_{0.2}}{\sigma_u} = \frac{0.2 + 185e}{1 - 0.375(n - 5)}$$  \hspace{1cm} (5.29)

$$\varepsilon_{0.2} = \frac{\sigma_{0.2}}{E_0} + 0.002$$  \hspace{1cm} (5.30)

$$\varepsilon_u = 1 - \frac{\sigma_{0.2}}{\sigma_u}$$  \hspace{1cm} (5.31)

$$m = 1 + 3.5\frac{\sigma_{0.2}}{\sigma_u}$$  \hspace{1cm} (5.32)

From the expression proposed by Rasmussen (2003) in Eq. (5.25), it is seen that only three basic parameters $E_0$, $\sigma_{0.2}$ and $n$ are needed in the Rasmussen (2003) model to represent the full stress-strain curve of stainless steel. In this research, Rasmussen (2003) model was used throughout the present work to model stainless steel behaviour.

**d) Material model of structural bolts and shear connectors**

The mechanical behaviour of the steel materials in structural bolts and shear connectors is usually different from structural steel section materials. The ductility of structural bolts and shear connectors is normally much lower than that for structural steel section materials. The mechanical behaviour of structural bolts and shear connectors in both tension and compression are assumed to be similar. In general, bilinear and trilinear stress–strain curves are used to describe these materials (Loh et al., 2006a; Mahmoud Baei et al., 2012; Mehdi Ghassemieh et al., 2012). These material models do not define a fracture point. Hanus et al. (2011) reported that a multilinear stress-strain curve proposed by Riaux (1980) considered three parameters: yield stress, ultimate strength and fracture stress. However, no equations were given for these parameters to derive the stress–strain curves.
In the present research, the full-range stress-strain curves for structural bolts and shear connectors were derived based on Eq. (5.21) for structural steel. The proposed stress-strain relationship models are shown in Figure 5.8 (a) for structural bolts and Figure 5.8 (b) for shear connectors, respectively.

\[
\sigma = \begin{cases} 
E_s \varepsilon & 0 \leq \varepsilon < \varepsilon_y \\
 f_u - (f_u - f_y) \cdot \left( \frac{\varepsilon_u - \varepsilon_y}{\varepsilon_u - \varepsilon_y} \right)^p & \varepsilon_y \leq \varepsilon < \varepsilon_u \\
 f_u - (f_u - f_t) \cdot \left( \frac{\varepsilon - \varepsilon_{u1}}{\varepsilon_t - \varepsilon_{u1}} \right)^p' & \varepsilon_u \leq \varepsilon < \varepsilon_{u1} \\
 f_t - f_t \cdot \left( \frac{\varepsilon - \varepsilon_{u2}}{\varepsilon_{u2} - \varepsilon_t} \right) & \varepsilon_{u1} \leq \varepsilon \leq \varepsilon_{u2}
\end{cases}
\] (5.33)

![Figure 5.8 Proposed full-range stress-strain curves for structural bolts and shear connectors](image)

(a) Structural bolts  (b) Shear connectors

Figure 5.8 Proposed full-range stress-strain curves for structural bolts and shear connectors

The values of \(p\), \(p'\) and \(f_t\) are calculated from Eqs. (5.22), (5.23) and (5.24), respectively. In determining the full-range stress-strain curves, parameters such as \(\varepsilon_u, \varepsilon_{u1}, \varepsilon_t, \varepsilon_{u2}, E_p, E_{u1}\) and \(E'_{u1}\) need to be calculated. Based on the test data collected from the coupon tests in this study and from other reports in the literature, empirical expressions for \(\varepsilon_u, \varepsilon_{u1}, \varepsilon_t, \varepsilon_{u2}, E_p, E_{u1}\) and \(E'_{u1}\) are proposed on the basis of trial and error method to fit curves. The expressions for various parameters are presented in Table 5.2.
Table 5.2 Parameters used in the stress-strain models for structural bolts and shear connectors

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Expressions for $\varepsilon_{u}, \varepsilon_{u1}, \varepsilon_t, \varepsilon_{u2}, E_p, E_{u1}, E_{u1}'$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Structural bolts</td>
</tr>
<tr>
<td>$\varepsilon_{u}$</td>
<td>$4.5 \varepsilon_y$</td>
</tr>
<tr>
<td>$\varepsilon_{u1}$</td>
<td>$8 \varepsilon_y$</td>
</tr>
<tr>
<td>$\varepsilon_t$</td>
<td>$34 \varepsilon_y$</td>
</tr>
<tr>
<td>$\varepsilon_{u2}$</td>
<td>$36 \varepsilon_y$</td>
</tr>
<tr>
<td>$E_{p}$</td>
<td>$0.125 E_s$</td>
</tr>
<tr>
<td>$E_{u1}$</td>
<td>$1.25 E_{u2}$</td>
</tr>
<tr>
<td>$E_{u1}'$</td>
<td>$0.017 E_s$</td>
</tr>
<tr>
<td></td>
<td>Shear connectors</td>
</tr>
<tr>
<td>$\varepsilon_{u}$</td>
<td>$22 \varepsilon_y$</td>
</tr>
<tr>
<td>$\varepsilon_{u1}$</td>
<td>$55 \varepsilon_y$</td>
</tr>
<tr>
<td>$\varepsilon_t$</td>
<td>$120 \varepsilon_y$</td>
</tr>
<tr>
<td>$\varepsilon_{u2}$</td>
<td>$130 \varepsilon_y$</td>
</tr>
<tr>
<td>$E_{p}$</td>
<td>$0.025 E_s$</td>
</tr>
<tr>
<td>$E_{u1}$</td>
<td>$2.5 E_{u2}$</td>
</tr>
<tr>
<td>$E_{u1}'$</td>
<td>$0.006 E_s$</td>
</tr>
</tbody>
</table>

The proposed stress-strain relationship models for structural bolts and shear connectors were validated by comparison with their coupon material test data. In both proposed stress-strain models, only three parameters (yield stress ($f_y$), ultimate strength ($f_u$), modulus of elasticity ($E_s$)) are required to determine the full-range stress-strain curve. Figure 5.9 shows test and predicted stress-strain curves for structural bolt material. Figure 5.10 shows test and predicted stress-strain curves for shear connector material. The comparisons with the predicted curves show good agreement.
Figure 5.9 Comparison between model and test stress-strain curves for structural bolts (continued)

Figure 5.10 Comparison between model and test stress-strain curves for shear connectors
5.2.2 Element types

The ABAQUS software library contains a wide range of elements, including continuum (solid) elements, shell elements, beam elements, rigid elements, membrane elements, infinite elements, spring and dashpot elements, connector elements and truss elements for solving many different problems. The most common types of element used in FE models of steel beam-CFST column joints are shell elements (S4R), 3D solid elements (C3D8, C3D8H, C3D8I, C3D8R, C3D20, C3D20H, C3D20R), truss elements (T3D2) and connector elements. Shell elements (S4R) are generally used to model the beam and steel profile sheet, in which one dimension (the thickness) is significantly smaller than the other dimensions and the stresses in the thickness direction are negligible. The S4R elements have five degrees of freedom (three translational and two in-plane rotations but no rotation about the shell normal) at each node. Solid elements can be used to model the bolted end plate connection components such as the steel tube, concrete, endplate, bolts, shear connectors and binding bars. Slab reinforcing bars are modelled using a 2-node linear truss element (T3D2). Connector elements can be used instead of solid elements to model the shear connectors, and in this case the load-slip behaviour of the shear connectors must be defined. This load-slip relationship is derived from push-out test results. In this work, solid elements are used to model the shear connectors, and there is no need to define the load-slip relationship.

Continuum or solid elements include eight-node linear (first-order) elements: brick elements (C3D8), hybrid formulation elements (C3D8H), incompatible mode elements (C3D8I), reduced integration elements (C3D8R); and 20-node quadratic (second-order)
elements: brick elements (C3D20), hybrid formulation elements (C3D20H) and reduced integration elements (C3D20R). These elements have three translational degrees of freedom at each node. These elements may be used to model almost any shape for linear analysis and also for complex nonlinear analyses involving contact, plasticity and large deformation. Since each element has some advantages and disadvantages, it is very important to select the appropriate element for a particular application, especially for the steel tube used in the CFST column connected to a steel beam in bending conditions. An element sensitivity analysis for cantilever steel beams and steel beams with a square hollow section (SHS) column joint is conducted to find out the appropriate element types for the modelling of the joint components, especially the steel tube, endplate and bolts. An elastic material is used for cantilever beam acting in transverse loading (as provided by ABAQUS manual). For the steel beam-SHS column joints, elastic-perfectly plastic materials were adopted for the steel beam (dimension: 250 × 125 × 6 × 9/600 mm, $f_y = 260$ MPa, $E_s = 210$ GPa), steel tube (dimension: 200 × 200 × 5/1400 mm, $f_y = 308$ MPa, $E_s = 216$ GPa), 14 mm endplate ($f_y = 270$ MPa, $E_s = 209$ GPa) and M20 bolts ($f_y = 750$ MPa, $E_s = 218$ GPa), see Figure 5.11.

![Configuration details of steel beam-SHS column joint](image)

**Steel beam:** $f_y = 200$ MPa, $E_s = 210$ GPa  
**Endplate:** $f_y = 270$ MPa, $E_s = 209$ GPa  
**Steel tube:** $f_y = 308$ MPa, $E_s = 216$ GPa  
**Bolt:** $f_y = 750$ MPa, $E_s = 218$ GPa

Figure 5.11 Configuration details of steel beam-SHS column joint
Table 5.3 shows comparative results of the element sensitivity analysis for the cantilever beam (length 150 mm, depth 5 mm and width 2.5 mm) under transverse loading. It is seen that the FE model results of the cantilever beam modelled using the C3D8 and C3D8H elements are under-predicted very badly for coarse meshes, and even for fine meshes. The least accurate results of tip deflection are caused by shear locking of C3D8 elements that are subjected to bending, since these elements are too stiff to bend (ABAQUS, 2012). The shear locking problem is overcome by using other elements (C3D8I, C3D20 and C3D20H). The model results for elements C3D8I, C3D20, C3D20H and C3D20R show a good agreement with the theoretical value, using both coarse and fine mesh. In addition, the elements C3D20R also give a good prediction with fine mesh only, but with a coarse mesh, a convergence problem arises due to excessive distortion at some integration points in the C3D20R solid elements.

Another case study was conducted for the beam-SHS column joint in which axial loading is applied to the beam. Figure 5.12 shows the element sensitivity analysis results for different mesh sizes (30, 25, 20, 15 and 10 mm elements). It was also observed that if the C3D8 and C3D8R elements are used in the model, the simulated result for the load–displacement curve varies with mesh size: the coarse mesh (30 mm) model shows a high stiffness value, but the fine (10 mm) mesh model gives a lower value. On the other hand, elements C3D8I, C3D20 and C3D20R give better results for both coarse and fine meshes. The accuracy of the results is higher for C3D20 and C3D20R elements for both meshes; however, computation time for these elements is also higher than if C3D8I elements are used. It is also seen that using C3D8I elements produces no significant difference in results compared to C3D20 and C3D20R elements for both coarse mesh and fine mesh, because the C3D8I element, which is an improved

<table>
<thead>
<tr>
<th>Element types</th>
<th>Mesh size (depth x length)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1 × 6</td>
</tr>
<tr>
<td>C3D8</td>
<td>0.237</td>
</tr>
<tr>
<td>C3D8H</td>
<td>0.237</td>
</tr>
<tr>
<td>C3D8I</td>
<td>3.068</td>
</tr>
<tr>
<td>C3D20</td>
<td>3.074</td>
</tr>
<tr>
<td>C3D20H</td>
<td>3.075</td>
</tr>
<tr>
<td>C3D20R</td>
<td>a*</td>
</tr>
</tbody>
</table>

Note: For load = 5 N, tip displacement of cantilever beam is 3.09 mm.
a*: Convergence problems arise for that particular mesh size
(a) Mesh sensitivity for element C3D8  
(b) Mesh sensitivity for element C3D8R  
(c) Mesh sensitivity for element C3D8I  
(d) Mesh sensitivity for element C3D20  
(e) Mesh sensitivity for element C3D20R  
(f) Comparison & mesh sensitivity: C3D8I & C3D20  

Figure 5.12 Element sensitivity analyses results for beam-SHS column joints
version of the C3D8 element, is introduced to remove shear locking and reduce volumetric locking. In these elements, internal deformation modes are added to the standard displacement nodes of the element in order to eliminate parasitic shear stresses that occur in bending. Thus the incompatible mode elements (C3D8I) can be used to model the joint components such as the steel tube, endplate, bolt and concrete core. The C3D8I elements have also been recommended for bending and contact problems by many other researchers (Bursi and Jaspart, 1998; Yang et al., 2000; Kishi et al., 2001; Chung and Iq, 2001a, b; Swanson et al., 2002; Citipitioglu et al., 2002; Ju et al., 2002; Komuro et al., 2004; Sarraj, 2007).

5.2.3 Geometry modelling of blind bolts

In the present study, Hollo-Bolts manufactured by Lindapter (Lindapter International plc) were adopted as the blind bolts to connect steel beams and CFST columns. These were selected because the sleeves of the fastener are plastically deformed when tightening the connector. The expanding sleeves then provide resistance to pull-out, which can increase the stiffness and load-carrying capacity of the joint (Hassan et al., 2014). Figure 5.13(a) shows a typical five-part Hollo-Bolt for structural application, consisting of collar, rubber washer, sleeve, cone and bolt shank. As can be seen, the interactions between different components are relatively complicated.

A simplified model was proposed for FE modelling of Hollo-Bolts to reflect the actual geometry of Hollo-Bolt as shown in Figure 5.13, in which the Hollo-Bolt is divided into two parts as shown in Figure 5.14. The sleeve is modelled as one part and the gap between two adjacent legs of the sleeve is also considered as shown Figure 5.14(a). The other Hollo-Bolt components (bolt head, collar, shank and cone) were modelled as one solid part in ‘Part-2’ as shown in Figure 5.14(b). The rubber washer was ignored in the model (Figure 5.14). A gap was also introduced between the sleeve and bolt shank to represent the gap between the sleeve and bolt shank, because the inner diameter of the actual sleeve is slightly larger than the diameter of the bolt shank. The hexagonal bolt heads and collars of actual Hollo-Bolts were modelled as circular bolt heads and collars to simplify the analysis. The angle of the solid cone of the modified blind bolt was considered the same as the cone slope of the original Hollo-Bolt. This simplified FE model can be used in the modelling of Hollo-Bolts. Table 5.4 shows the dimension details of M16 and M20 grade 8.8 Hollo-Bolts.
Figure 5.13 Dimension details of simplified model of a Hollo-Bolt

Figure 5.14 Simplified FE model of a Hollo-bolt used in blind-bolted connections to open section or SHS column or CFST column
Table 5.4 Dimension details of M16 and M20 grade 8.8 Hollo-Bolt

<table>
<thead>
<tr>
<th>Product code</th>
<th>Bolt size</th>
<th>Bolt diameter (D₀)</th>
<th>Clamping thickness range (W)</th>
<th>Sleeve</th>
<th>Collar</th>
<th>Clamping thickness (t₁)</th>
<th>Cone thickness (t₂)</th>
<th>Cone slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>HB16-1 M16</td>
<td>16 b</td>
<td>16</td>
<td>12-29</td>
<td>41.5</td>
<td>25.75 b (25.5 c)</td>
<td>17.0 c</td>
<td>4.25 c</td>
<td>8 c</td>
</tr>
<tr>
<td>HB16-2 M16</td>
<td>(15.80 c)</td>
<td>29-50</td>
<td>63</td>
<td>50-71</td>
<td>84</td>
<td>a</td>
<td>L-t₂</td>
<td>15</td>
</tr>
<tr>
<td>HB16-3 M16</td>
<td>20 b</td>
<td>12-34</td>
<td>50</td>
<td>34-60</td>
<td>76</td>
<td>32.75 b (32.60 c)</td>
<td>21.0 c</td>
<td>5.8 c</td>
</tr>
<tr>
<td>HB20-1 M20</td>
<td>(19.73 c)</td>
<td>34-60</td>
<td>76</td>
<td>60-68</td>
<td>102</td>
<td>a</td>
<td>L-t₂</td>
<td>15</td>
</tr>
</tbody>
</table>

Note: a= the sum of the steel tube thickness and endplate thickness; b=nominal value; c= measured value

Gap between two legs of sleeve (Gₛₜ) = 1.50 mm for M16 and 2.75 mm for M20
All dimensions are in mm.

Another simplified model was proposed to model of Hollo-Bolt for FE modelling in which all components of Hollo-Bolt, including the bolt head, collar, sleeve, cone and bolt shank were considered as one solid part as shown in Figure 5.15. The rubber washer was also ignored in this model.

![Simplified FE model](a) Simplified FE model
![Hard contact between collar and sleeve](b) FE model with mesh

Figure 5.15 Simplified FE model of Hollo-Bolt used only in blind-bolted connection to CFST column

A gap between sleeve and bolt shank was also introduced. Meanwhile, the gap between two adjacent legs of the sleeve was ignored. To allow the separation of the sleeve from the collar in the case of pull-out forces or shear forces, the sleeve and collar were not joined and introduced a negligible gap (0.0001 mm) between collar and sleeve because if the gap was not introduced, the sleeve could not freely move in either the horizontal or vertical directions. The collar will be in contact with the sleeve once the pretension force is
applied. The reason for modelling the whole Hollo-Bolt as a single piece was to avoid a convergence problem that occurs when the Hollo-Bolts are embedded in the concrete core of the CFST column. This model was mainly used throughout FE modelling of the Hollo-Bolts in the blind-bolted connections to the CFST column, except for FE modelling and investigation of Hollo-Bolt in pure tension and pure shear.

5.2.4 Contact modelling

To define the interaction between different components of the blind-bolted endplate connection, the “surface-to-surface contact” option provided in ABAQUS was employed. As shown in Figures 5.16 and 5.17, the contact pairs in the FE model comprise the steel tube-to-concrete core (Contact A), endplate-to-steel tube (Contact B), collar-to-endplate (Contact C), sleeve-to-bolt hole (Contact D), bolt shank and sleeve-to-steel tube and concrete core (Contact E), collar-to-sleeve (Contact F) and bolt shank-to-sleeve (Contact G). Hard contact in the normal direction and Coulomb friction in the tangential direction were defined in the surface-to-surface contacts.

In past research, a range of friction coefficients have been adopted: 0.25 (Schneider, 1998; Hassan et al. 2013, 2014), 0.3 (Lam et al., 2012), 0.6 (Han et al., 2007; Tao et al., 2013) for contact between steel and concrete; and 0.1 (Ataie & Bradford, 2013), 0.25 (Hassan et al., 2013, 2014) for contact between steel surfaces. Rabbat and Russell (1985) reported that, based on their test results, the coefficient of friction between steel and concrete usually varied from 0.57 to 0.68. In the present FE model, a friction...
coefficient of 0.6 was used for the contact between carbon steel and concrete, and 0.25 for the contact between stainless steel and concrete. For bolt-to-steel, a sensitivity analysis is conducted to determine a suitable friction coefficient for these contact pairs. The influence of the friction coefficient (\( \mu \)) on the load-deformation curves for the blind-bolted joints with CFST columns is shown in Figure 5.18 based on the results of the sensitivity analysis. It is seen that when \( \mu \) increases from 0.1 to 0.45, the FE model results for all values of \( \mu \) change significantly, and the best match with the test results is for \( \mu = 0.25 \). Accordingly, the value of 0.25 is used for bolt-to-steel contacts in the present FE model.

![Figure 5.17 FE model and contact details of Hollo-Bolts connected to open section](image)

- (a) Top view of double shear test specimens
- (b) Elevation view of sections A-A

Figure 5.17 FE model and contact details of Hollo-Bolts connected to open section

![Figure 5.18 Influence of friction coefficient (\( \mu \)) on the load-deformation curves of blind-bolted endplate steel beam-CFST column joints](image)

Figure 5.18 Influence of friction coefficient (\( \mu \)) on the load-deformation curves of blind-bolted endplate steel beam-CFST column joints
In surface-to-surface contact, the master and slave surfaces must be carefully defined for contact formulation. The contact sliding formulation option and the contact interaction properties are also required. The master surfaces are defined as surfaces belonging to the body of the stronger material, or bodies of a finer mesh. The contact sliding formulation uses finite sliding (Chen et al., 2012; Hassan et al., 2013). In surface-to-surface interaction of steel tube and concrete, the inner surface of the steel tube was chosen to be the master surface as it was the stronger material (Sarraj, 2007), and the outer surface of the concrete core was defined as the slave surface. The contact surfaces of the steel tube are chosen to be the master surface and the endplate was selected to be the slave surface. The contact surfaces of the bolt head/nut/shank were chosen to be the master surfaces and the endplate/steel tube/bolt hole were selected to be the slave surfaces (Sarraj, 2007). Tie contact was used for the steel beam and endplate, and for shear connectors to beams. The surfaces of the endplate that come in contact with the steel beam were considered to be the master surface and the steel beam surfaces that contact the endplate were considered as slave surfaces (Sarraj, 2007). In the contact between reinforced steel bar and concrete in the slab, the bonding between slab reinforced steel bar and concrete are defined using “embedded region” constraints. The reinforcing steel bars were selected as the embedded elements and the concrete as the host elements. Other parameters used ABAQUS default values.

5.2.5 Boundary conditions and load

Boundary conditions and applied loads in the FE models are shown in Figure 5.19 for joints without slab and in Figure 5.20 for joints with slab. Pinned supports were applied on the bottom column and restricted from moving in the x-, y-, and z-directions. The top of the column was restricted only in the z-direction and free to move in the other two directions. In some cases, only half of a specimen was considered in FE modelling due to the symmetry of boundary and loading conditions with an aim to reduce the size of the model and computational time. In the half-model of steel beam–CFST column connections, the vertical plane was restricted from moving in the z-direction, and was free from rotational restrain.

In the FE modelling, the loads are applied at the reference point to reflect actual loading conditions that were applied to the test specimens. In the tests of blind-bolted endplate
connections, three types of loads, (1) bolt tightening (pre-tension) force, (2) column axial load and (3) beam load; were generally considered. These loads were applied in three steps in the FE models: in the first step of the FE analysis of Hollo-Bolted connections, pre-tension forces were applied to the Hollo-Bolts using the “bolts load” option in ABAQUS. There were three steps in applying the bolt pre-tension forces: (i) bolt pre-tension forces are defined at the middle section of the Hollo-Bolt shank using the “apply force” method; (ii) the length of the bolt is then fixed at its current position in the second step; and (iii) By applying the “fixed at current length” method in the third step, the force in the bolt is allowed to change with vertical load on the beam in the subsequent analysis.

Column axial load (approximately 60% of the CFST column capacity) and beam load were applied in the second step and third steps respectively, by using either the load control method or displacement control method. The loading process for bolt load or displacement control or load control was performed by adopting a general static analysis using the Newton-Raphson method. The Newton-Raphson method is a powerful technique for solving equations numerically and for obtaining the solutions to nonlinear problems.

Figure 5.19 Load and boundary conditions of FE models of steel beam-CFST column joints without slab
Nonlinear FE analysis of the steel beam to CFST column connections can be incorporated in the FE models by considering the three basic types of nonlinearity: material nonlinearity, geometric nonlinearity and boundary nonlinearity. Material nonlinearity of each component of a joint is assigned by applying the nonlinear stress–strain relationship. Geometric and boundary nonlinearities are assigned in the FE models using NLGEOM and CONTACT PAIR commands respectively in ABAQUS. Boundary nonlinearity is associated with a change in the boundary conditions (contact/gaps) during the analysis. The loading process is performed under either displacement control or load control by adopting a general static analysis. The solution of the nonlinear analysis is done by applying the load in increments until obtaining the final solution, and many iterations are required to solve the equations at a given load increments.
5.3 FE Model Validation for Simulating Blind-Bolts

A numerical model of the Hollo-Bolts was developed according to FE model shown in Figure 5.14 and its accuracy was tested by comparing the FE model results with test results. The FE models of Hollo-Bolts subjected to pure tension and shear were established for the test configurations reported by Occhi (1995), Ballerini and Piazza (1996), Elghazouli et al. (2009), Liu et al. (2012) and Tao et al. (2014). Occhi (1995) conducted experiments on Hollo-Bolts (M12, M16, M20, grade 8.8) under tensile and shear loads, and used square hollow section (SHS 140 × 140) with different tube thicknesses (6.3 to 12.5 mm). Ballerini and Piazza (1996) also performed shear tests on Hollo-Bolts with the same testing setup used by Occhi (1995). Elghazouli et al. (2009) reported four sets of tensile test data for Hollo-Bolts (M12 and M16, grade 8.8 and 10.9). Single and double shear tests of grade 10.9 Hollo-Bolts were conducted by Liu et al. (2012), in which different plate thicknesses were also considered. Recently, Tao et al. (2014) conducted tests at ambient and elevated temperatures. At ambient temperature, one test was in shear and one test was in tension. All of these test results are used to verify the FE model of Hollo-Bolts under tension and shear. Reasonable agreement was obtained between the test results and predicted values using the current FE model. Here, only a few test specimens are reported to demonstrate the validation of the FE model.

5.3.1 Blind-bolts in tension

The FE model results of Hollo-Bolted connections under pure tension are compared with the test results conducted by Elghazouli et al. (2009) and Tao et al. (2014). The comparative results are shown in Figures 5.21 and 5.22. It is seen from the figures that the initial simulated stiffness and ultimate tensile capacity were similar to the test results. It was reported by Elghazouli et al. (2009) that the failure of Hollo-Bolts under tension was due to sleeve failures. It was also observed from a stress contour plot obtained from the FE model that a high stress concentration was developed in the sleeve, and failure of Hollo-Bolts was also due to the failure of the sleeve, not the bolt shank. The sleeve failed due to crushing and fracture, which implies that the sleeves were weaker than the shank of the Hollo-Bolt. Since the FE model results of Hollo-Bolted connections showed a good agreement with the test results, the proposed FE model will be used to conduct a parametric analysis of the Hollo-Bolted connections.
under pure tension in Chapter 6 to investigate the influence of the different parameters on the ultimate tensile capacity of Hollo-Bolted connections.

Figure 5.21 Comparison between a test conducted by Elghazouli et al. (2009) and FE prediction of Hollo-Bolts under tensile load

Figure 5.22 Comparison between a test conducted by Tao et al. (2014) and FE prediction of Hollo-Bolts under tensile load

5.3.2 Blind-bolts in shear

The FE model results of Hollo-Bolted connections under pure shear were compared with the test results conducted by Occhi (1995), Ballerini and Piazza (1996), Elghazouli et al. (2009), Liu et al. (2012) and Tao et al. (2014). The comparative results are shown in Figures 5.23 to 5.26. The FE-predicted results of Hollo-Bolted connections under pure shear showed a good agreement with the test results. The initial stiffness and yield
points due to the sleeve yielding and bolt shank yielding, and ultimate shear capacity, were predicted very well. It is seen from the FE model results and test results that the behaviour of Hollo-Bolted connections under shear is unlike that of Hollo-Bolted connections under tension. It was observed in shear tests and in the FE model results that the initial stiffness of Hollo-Bolts is maintained to a certain point and then changes, and again linearly increases before the bolt shank yields. The fall in initial stiffness is due to yielding of the sleeve with significant local deformation observed. As a result, the sleeve comes into full contact with the bolt shank, which then carries the load until it fails. It can be concluded, both from previous test results and the present FE model results that the shear load behaviour of Hollo-Bolts depends on the sleeve and bolt shank.

(a) SHS140×6.3; two M20 8.8 bolts
(b) SHS150×12.5; two M20 8.8 bolts
Figure 5.23 Comparison between tests conducted by Occhi (1995) and FE results for Hollo-Bolts under shear load

(a) SHS140×6.3; two M16 8.8 bolts
(b) SHS150×12.5; two M16 8.8 bolts
Figure 5.24 Comparison between tests conducted by Ballerini & Piazza (1996) and FE results for Hollo-Bolts under shear load
In this section, 26 test specimens from studies conducted by Lee et al. (2010), Yao et al. (2008), Li et al. (2015), Wang et al. (2009) and Tizani et al. (2013a, b) were used to test the FE model of Hollo-Bolted endplate connections to CFST columns. FE models of those test specimens were developed according to FE modelling details (Figure 5.14) described in section 5.2.3. The results were compared with their test results. Nine specimens conducted by Lee et al. (2010), Yao et al. (2008) and Li et al. (2015) for blind-bolted endplate connections in tension were modelled and reported in subsection 5.4.1. In subsection 5.4.2, the FE model results for blind-bolted endplate connections
subjected to bending loading are compared with 17 test data reported by Wang et al. (2009), Tizani et al. (2013a & b) and Li et al. (2015).

The configurations of typical blind-bolted endplate connections reported in the literature are given in Figure 5.27 (a) for joints tested under tensile loading and in Figure 5.27 (b) for the joints tested in bending loading.

Figure 5.27 Typical blind-bolted endplate connections tested under tensile load and bending loading

5.4.1 Blind-bolted endplate connections in tension
Lee et al. (2010) tested the behaviour of blind-bolted endplate connections in tension for square hollow section (SHS) columns and considered different parameters to investigate their influence on the bolted connection. Two of the specimens (S1 and S2) tested by Lee et al. (2010) were used to verify the FE model. Materials and geometric nonlinearity were considered in the FE models. Load was applied according to the tests conducted by Lee et al. (2010). Their test results and the FE results are compared in Figures 5.28 and 5.29. Figure 5.28 shows the failure mode of the tube, that is punching failure of bolt holes. Comparison between the failure mode of the tube of the test result and the FE model results shows good agreement. Figure 5.29 (a) shows a comparison between the load–displacement curves between test result and FE result for specimen S1 in which the blind bolt did not have sleeves. Figure 5.29 (b) shows a comparison between test result and FE result for specimen S2. The main difference between these
two test specimens was in the bolt sleeves. In specimen S1, bolt sleeves were not used, but bolt sleeves were used in specimen S2 to tighten the bolt hole. The presence of the sleeves in specimen S2 which fill the gap between the oversized hole and bolt shank (reducing the gap from a total 8 mm to 0.5 mm) reduced the bending of the bolts and hence increased their ultimate strength above that of specimen S1 shown in Figure 5.29 (a).

Yao et al. (2008) tested the behaviour of cogged extension bars welded to the head of the blind bolts and anchored in the infill concrete to prevent the steel tube separating from the concrete, resulting in improved stiffness and strength (Yao et al., 2008). According to the test details reported in Yao et al. (2008), an FE half-model of a symmetrical blind-bolted connection was built in order to reduce the size of the model and, consequently, the computational cost, by applying the appropriate boundary conditions. Ajax bolts were used in their test and modelled here as normal bolts. The test result obtained from Yao et al. (2008) and FE simulation of load vs. relative outward displacement between the centre of the curved endplate and the concrete-filled steel tube due to tensile loading is shown in Figure 5.30. The stress contour plot obtained from the FE analysis indicated a high stress concentration that exceeded the yield stress of the cogged bar near the weld between the bolt head and the end of the cogged bar connection. A high stress concentration was also observed at the middle of both edges of the curved endplate.

(a) Test (S1) (Lee et al., 2010)  (b) FE model (S1)

Figure 5.28 Failure mode of the steel tube for specimen S1
Li et al. (2015) tested eight steel beam–CFST column joint specimens in tension and bending. Seven specimens (JS1, JS2, JS3, JC5, JC6, JS7, JC8) were designed as blind-bolted connections with endplate (280 × 200 × 14 mm), blind bolts (M20, grade 8.8) and binding bars (ϕ20). Four binding bars were considered in specimens JS2/JS7 and JC6/JC8 and eight binding bars were used in specimen JS3. One of the joints used a through-bolted (ϕ20) endplate connection (specimen JS4). In the FE modelling of specimens JS1, JS2, JS3, JS4, JC5 and JC6, two types of loads (bolt tightening loads and beam loads) were considered in accordance with the tests, and applied in two steps.
Figure 5.31 Comparison between tests conducted by Li et al. (2015) and FE results of SHS joints with blind-bolted endplate connections in tension
In the FE modelling of six symmetrical connections, half-models were used, by applying the appropriate boundary conditions, to reduce the size and computational cost. The comparative results of the FE models and test of six specimens (JS1, JS2, JS3, JS4, JC5, JC6) subjected to tensile loading are reported in Figure 5.31. The load–displacement curve obtained from FE models of these specimens agreed very closely with the test results of those specimens, with the exception of specimen JS3, for which the model predicted the initial stiffness very well, but not the ultimate load capacity. The degradation of load in the joint was similar to the test results. It was noted that the ultimate load obtained from FE analysis agreed reasonably well with the experiment test results. In addition, the failure mode as shown in Figure 5.32 was found same as the test.

![Failure modes of CFST joint with blind-bolted connection (JS3) in tension](image)

**Figure 5.32** Failure modes of CFST joint with blind-bolted connection (JS3) in tension

### 5.4.2 Blind-bolted endplate connection in bending

FE half-models of 17 specimens of blind-bolted endplate connections tested in bending by Wang et al. (2009), Tizani et al. (2013a, b) and Li et al. (2015) were developed. Loading and boundary conditions were applied according to their test setup. In the FE modelling of the Wang et al. (2009) tests, three types of loads were considered (bolt tightening load, column axial load and beam load), but for the Tizani et al. (2013a, b) and Li et al. (2015) tests, only bolt tightening loads and beam loads were considered in accordance with their test details. The column axial load was applied through the load control method.
Wang et al. (2009) conducted tests on blind-bolted endplate connections to CFST column. Among those, two test specimens (CJM1, CJM2) were modelled. The comparative results of FE model and test of the two specimens subjected to bending are shown in Figure 6.3. The predicted load–displacement curves obtained from the FE models of these specimens agree very closely with the test results. The predicted initial stiffness shows a good correlation with the test results, and the predicted ultimate capacity is only 1-4% higher than the test strengths.

Tizani et al. (2013a, b) conducted 13 tests on extended blind-bolted connections to CFST columns (specimens S1-S7 and T1-T6). These were all modelled; comparisons between the FE predictions and test results for the 13 bending tests are shown in Figure 5.3. The load–displacement curves obtained from the FE models agree reasonably well with the test results. It is seen from the figures that the predicted ultimate load of all specimens was very close to the test results. For all specimens, the ultimate capacity obtained from the FE model of specimens CJM1 and CJM2 varied only 3–8% from their measured strengths. The predicted initial stiffness shows a good correlation with the test results as well. In the stress contour plots of the FE results (Figure 5.3 (b)), it was observed that most connections were predicted to fail due to failure of the bolt shank. In addition, the failure modes were found to be same as those observed in the tests. The failure mode of the blind bolt of one specimen is shown in Figure 5.3.

Li et al. (2015) reported the test results of two specimens tested under bending. The comparative results for the FE model and the test of both specimens (JS7, JC8) subjected to bending are shown in Figure 5.3. The load–displacement curve obtained from the FE model of specimen JC8 was observed to agree very closely with the test results of the specimen JS8, but not specimen JS7. However, the predicted initial stiffness for both specimens shows a good correlation with the test results. The ultimate capacity obtained from the FE model of specimens JC8 is only 7% higher than the measured strengths obtained from tests. The predicted ultimate capacity of specimen JS7 was somewhat higher than the test result, but the load degradation behaviour (Figure 5.3 (a)) and failure mode (Figure 5.3) were similar to the test. From the above, it was concluded that the FE model predictions showed a good correlation with experimental results.
Figure 5.33 Comparison between tests conducted by Wang et al. (2009b) and FE results of CFST joint with blind-bolted endplate connections in bending load.

(a) CJM1

(b) CJM2

Figure 5.34 Comparison between tests conducted by Tizani et al. (2013a & b) and FE

(a) Specimen S1

(b) Specimen S2

(c) Specimen S4

(d) Specimen S5
model results of blind-bolted endplate connections in bending load

Figure 5.34 Comparison between tests conducted by Tizani et al. (2013a & b) and FE model results of blind-bolted endplate connections in bending load (continued)
Figure 5.34 Comparison between tests conducted by Tizani et al. (2013a & b) and FE model results of blind-bolted endplate connections in bending load (continued)

Figure 5.35 Failure mode of extended Hollo-bolt tested in bending loading
Figure 5.36 Comparison between tests conducted by Li et al. (2015) and FE results of CFST joint with blind-bolted endplate connections in bending load

Figure 5.37 Failure modes of CFST joints with blind-bolted connections (JS7) in bending load

5.5 FE Model Validation for CFST Column Joints with Slab

Seven sets of test data for composite steel beam–CFTS column joints with slab, were considered to validate the FE model. One set of data was reported by Mirza and Uy (2011) and six sets of data are from the present work, as described in Chapter 4. FE models of those test specimens were developed according to FE modelling details (Figure 5.15) that discussed in section 5.2.3. In the FE model of the specimen tested by Mirza and Uy (2011), a quarter of the symmetrical specimen was modelled and the
appropriate boundary conditions were applied; whereas the tests conducted in this study were modelled as complete FE models. In all modelled cases, three types of loads were considered in accordance with the reported test details: bolt tightening load, column axial load and beam load. The column axial load (60% of the CFST column capacity) was applied by the load control method. The predictions from the FE models were compared to their test results.

Figures 5.38 and 5.39 show the FE results of the flush endplate connection with slab reported by Mirza and Uy (2011). The stress contour plot of a quarter-model of the flush endplate connection is given in Figure 5.39. The load-displacement curves for the test and FE result of the connection is shown in Figure 5.38. The FE model results show good correlation with the experimental results.

**Figure 5.38 Comparison between tests conducted by Mirza & Uy (2011) and FE results of CFST joint with slab in bending**

**Figure 5.39 Stress contour plot of a quarter FE model of flush endplate connection**
Figure 5.40 shows comparisons between test results and proposed model results for the hybrid steel beam-CFST column composite joints with blind-bolted endplate connections. Figure 5.41 shows a comparison between test results and proposed model results of hybrid steel beam-CFST column composite joints with through-plate connections. It is seen from Figure 5.40 that the ultimate moment capacity and initial stiffness found from FE modelling of all joint specimens closely matched their test results. In the test, failures of joints were observed to occur due to slab failures, such as slab concrete cracking and slab reinforcement fracture. Failure modes predicted by the FE analysis of the hybrid steel beam-CFST column composite joints with blind-bolted endplate connections were similar to the test failure modes as observed in the tests. The FE model results of the through-plate connections also matched their test results as shown in Figure 5.41. The failures of the joints with through-plate connections were due to buckling of the web of the steel beam, and tear failure of the bolt hole.
Figure 5.41 Comparison between test and FE results for hybrid steel beam-CFST column composite joints with through-plate connections

5.6 Summary

This chapter provides detailed information for modelling CFST column joints with Hollo-Bolts. The FE model result mainly depends on the modelling of materials, interactions, elements types and geometry of the Hollo-Bolts. Full-range stress-strain curves were proposed for the structural steel sections, structural steel bolts and shear studs. The proposed stress–strain models were validated by comparison with test stress–strain results. An element sensitivity analysis was conducted to find out the appropriate type of solid element to be used in the model. It was seen that incompatible mode elements (C3D8I) can be used as solid elements to model the joint components, including the steel tube, endplate, Hollo-Bolts, concrete core and steel beams. This type of element is an improved version of the C3D8 element. In these elements, internal deformation mode is added to the standard displacement modes of the element in order to eliminate parasitic shear stresses that occur in bending. C3D8I elements were selected for the solid model of the CFST column joints, with Hollo-Bolts.

The developed FE models of CFST column joints, including Hollo-Bolts, were verified by many test results reported in the literature and test results in the current research. The results obtained from the developed FE models show good correlation with test results. So, the proposed FE model can be confidently adopted for parametric analyses of CFST column joints.
CHAPTER 6

BEHAVIOUR OF BLIND BOLTS SUBJECTED TO PURE TENSION AND SHEAR

6.1 Introduction
The capacity of blind bolts under tensile and shear forces is very important to the designer of blind-bolted endplate connections for composite steel beam–CFTS column joints. Tests on blind bolts (Hollo-Bolts) have been reported in past studies to investigate their behaviour when subjected to pure tensile and shear loads. Most of those tests were conducted either for open section or square hollow steel (SHS) column. However, it has been observed that the Hollo-Bolts in concrete-filled steel tubular (CFST) columns in pure tension or shear behave differently from Hollo-Bolts connected to the SHS column. When a Hollo-Bolt is embedded in the concrete core of the CFST column, the gap between two adjacent sleeves around the cone of the bolt is filled with concrete and more interaction develops between the concrete, sleeve and cone. These additional constraints introduced by the concrete restrain sleeve movement from the concrete, and this changes the behaviour of the Hollo-Bolt itself. So, the behaviour of Hollo-Bolts embedded in the concrete core of the CFST column needs to be investigated. This chapter describes this investigation.

In this chapter, the behaviour of Hollo-Bolts embedded in the concrete core of the CFST column is mainly investigated under pure tension and shear using FE modelling. The results are compared to the behaviour of the Hollo-Bolt when connected to an SHS column. For the two conditions, different parameters are considered to understand the behaviour of the Hollo-Bolts and connecting parts, including the steel tube of both SHS and CFST column, and the endplate. The characteristic behaviour of Hollo-Bolts in pure tension and shear is identified for the load-deformation responses when connected to SHS or CFST columns, and finally the capacities of M16 and M20 Hollo-Bolts in pure tension and shear are recommended for SHS or CFST columns. The parametric
analysis for Hollo-Bolts under pure tension is described in section 6.2, and the analysis for Hollo-Bolts under pure shear loading is described in section 6.3.

6.2 Parametric Studies on Blind Bolts under Pure Tensile Loading

A parametric analysis was conducted to investigate the behaviour of the M16 and M20 Hollo-Bolts for the configurations shown in Figure 6.1, for connections to SHS and CFST columns when the bolts are in tension. Other parameters related to the Hollo-Bolts and connecting parts were also considered: sleeve material (275–500 MPa), Hollo-Bolt diameter (grade 8.8) and tube thickness (6.3–25 mm), to evaluate their influence on initial stiffness and ultimate capacity of Hollo-Bolts in tension. The dimensions used to build the FE model are shown in Figure 6.1(a) for a CFST column connection and in Figure 6.1(b) for an SHS column connection. In both cases, the dimensions of the cross-section of the column were maintained at 150 × 150 mm. The width and height of the endplate for both cases were 150 × 200 mm, and endplate thickness was maintained at 15 mm throughout the analysis. The material properties of the steel tube and endplate listed in Table 6.1 were constant throughout the analysis unless otherwise specified. The yield strength of both the steel tube and endplate was 350 MPa. For the CFST column, the compressive strength of the infill concrete was kept constant as 40 MPa throughout the analysis. The tensile load was applied to the bolt through the endplate, at points 10 mm from the top and bottom of the endplate. The boundary conditions and tensile load for the FE analysis is shown in Figure 6.2. Boundary conditions for the supports were applied at mid-column. FE models were developed according to the details set out in Chapter 5.

Table 6.1 Material properties used in the parametric analysis of Hollo-Bolts subjected to pure tension

<table>
<thead>
<tr>
<th>Component</th>
<th>Yield stress (MPa)</th>
<th>Ultimate strength (MPa)</th>
<th>Elastic modulus (MPa)</th>
<th>Elongation at fracture (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel tube</td>
<td>350</td>
<td>450</td>
<td>200000</td>
<td>30</td>
</tr>
<tr>
<td>End plate</td>
<td>350</td>
<td>450</td>
<td>200000</td>
<td>30</td>
</tr>
<tr>
<td>Bolt shank</td>
<td>752</td>
<td>918</td>
<td>205000</td>
<td>14</td>
</tr>
<tr>
<td>Bolt sleeve</td>
<td>450</td>
<td>600</td>
<td>208830</td>
<td>30</td>
</tr>
</tbody>
</table>
Figure 6.1 Configuration of FE models of Hollo-Bolts subjected to pure tension while connected to (a) CFST column, and (b) SHS column (unit: mm)
6.2.1 Effect of CFST column and SHS column

The effects of the CFST column and SHS column were investigated to determine their influence on the tensile load capacity of the Hollo-Bolts. It was seen in the literature review that the behaviour of Hollo-Bolt in tension had previously been investigated by tests where the Hollo-Bolt was connected either to an open-section or SHS column. In this section, the behaviour of Hollo-Bolt embedded into the concrete core of a CFST column is compared to its behaviour when connected to an SHS column.
Figure 6.3 shows the different influences of CFST and SHS columns on the load–deformation curves of Hollo-Bolts. It is seen that the ultimate tension capacity of Hollo-Bolts as shown in Figure 6.3(a) and (b) are observed lower when M16 & M20 Hollo-Bolts connected to SHS columns, but found higher when these two Hollo-Bolts connected to CFST columns. This difference is also observed in Figure 6.3(c), despite a thicker steel tube (20 mm) being used in both types of column. The lower capacity of the Hollo-Bolts in the SHS column is the lower capacity of the steel tube, and inadequate contact surface of Hollo-Bolt sleeve with steel tube. As a result, punching failure was observed in the bolt sleeve, together with tearing and bearing failure of the steel tube of the column. The higher capacity of the Hollo-Bolt in the CFST column connection is because the contact surfaces between the Hollo-Bolt sleeve and the concrete core increases the bearing surface and enhances the interaction of Hollo-Bolt sleeve with the steel tube and concrete core of the CFST column. Also, the concrete restrains the movement of the Hollo-Bolt sleeve and cone during tensile loading. Table 6.2 shows the ultimate tensile load capacity of Hollo-Bolts when connected to SHS and CFST column. The tensile load capacity of Hollo-Bolt increases 8.6-22.85% when using CFST column, compared to SHS column.

The capacity of M16 Hollo-Bolts when connected to CFST column was greater than its when connected to SHS column. This was observed in CFST column for M16 due to the larger bearing surfaces of M16 Hollo-Bolt that come from the contact between Hollo-Bolt sleeve and concrete core. But for M16 Hollo-Bolt with SHS column, the contact surfaces between M16 Hollo-Bolt sleeve and steel tube is very small compared to the contact surface area in CFST column. The capacity of M20 Hollo-bolt when connected to CFST column was significantly increased compared to the capacity of M20 Hollo-Bolt connected to SHS column. This indicates that the performance of both types of Holl-Bolts was improved when connected to CFST column.
Figure 6.3 Influence of CFST and SHS column on the load-deformation curves of Hollo-Bolt in tension
Table 6.2 Ultimate tensile load capacity of Hollo-Bolts when connected to SHS column and CFST column

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Ultimate tensile capacity of Hollo-Bolts (kN)</th>
<th>Load increase using CFST column (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>For SHS column</td>
<td>For CFST column</td>
</tr>
<tr>
<td>M16 Tube thickness-12.5 mm</td>
<td>140</td>
<td>152</td>
</tr>
<tr>
<td>Hollo-Bolt Tube thickness-20 mm</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>M20 Tube thickness-12.5 mm</td>
<td>210</td>
<td>258</td>
</tr>
<tr>
<td>Hollo-Bolt Tube thickness-20 mm</td>
<td>273</td>
<td>317</td>
</tr>
</tbody>
</table>

6.2.2 Effect of sleeve material

In Hollo-bolts, sleeve is one of the most important components. This section describes the investigation of the influence of the sleeve material on the behaviour of Hollo-Bolts. Figure 6.4 shows the effect of different sleeve material of Hollo-Bolt in which four different mild steel materials (275 MPa; 350 MPa; 450 MPa & 500 MPa, yield strength) were considered as the sleeve material for M20 8.8 grade Hollo-Bolt. In all cases, the sleeve thickness was kept constant as 5.80 mm (inner diameter 21 mm; outer diameter 32.6 mm).

![Figure 6.4](image)

Figure 6.4 Effect of sleeve material on the load-deformation curves of Hollo-Bolts in tension

The FE model results for these sleeve materials, shown in Figure 6.4 indicate that the use of sleeve material with higher yield strength has no significant effect on the ultimate tensile capacity of Hollo-Bolts when connected to SHS column, but it has
some effect on the ultimate tensile capacity of the Hollo-Bolt when connected to CFST column. It is also observed that the ultimate tension load capacity of Hollo-Bolt is found higher when Hollo-Bolts are connected to CFST column compared to the Hollo-Bolt connected to SHS column.

6.2.3 Effect of bolt material and bolt diameter

The effects of bolt material (grades 8.8 and 10.9) and bolt diameter (M16 and M20) on the behaviour of Hollo-Bolt were investigated. Bolt yield strengths of 752 MPa for grade 8.8 bolts and 883 MPa for grade 10.9 bolts were examined. The same sleeve material (450 MPa) was considered in both bolt material grades (8.8 and 10.9). The parametric analyses results are shown in Figures 6.5 and 6.6. In all cases, the thicknesses of the steel tube (150 × 150 mm) and sleeve are kept constant as 12.5 mm and 5.80 mm, respectively. It is seen from Figure 6.5 that the ultimate tensile capacity of M16 Hollo-Bolt are observed same for both types of bolt materials (grades 8.8 and 10.9); same phenomenon is found for M20 Hollo-Bolts. This is observed due to the failure of sleeve and steel tube rather than the bolt shank.

Figure 6.6 shows that the ultimate tension capacity of M16 Hollo-Bolt was lower when the bolt was connected to a SHS column; and higher when the bolt was connected to CFST column. The lower strength in the SHS column is due to the lower capacity of the steel tube. In this case, the failure occurred by tearing and bearing of the steel tube and bolt sleeve. The capacity of the Hollo-Bolt increased when the bolt diameter increased from 16 mm (M16 Hollo-Bolt) to 20 mm (M20 Hollo-Bolt) for a same SHS section (150 × 150 × 12.5 mm). The higher load capacity observed for M20 Hollo-Bolt is due to the higher contact surface area of M20 Hollo-Bolt contacted with the steel tube. When M20 Hollo-Bolt was used in SHS column, the bearing surface area also increased by the larger bolt hole diameter made for larger bolt sleeve diameter. This larger bearing surface area increased the capacity of M20 Hollo-Bolts despite the steel tube thickness of the SHS column remaining unchanged (12.5 mm).
6.2.4 Effect of tube wall thickness

Figure 6.7 shows the effect of steel tube thickness on the behaviour of Hollo-Bolt. Three wall thicknesses (12.5, 20 and 25 mm) were considered for M20 grade 8.8 Hollo-Bolts. In all cases, the inner diameter and outer diameter of sleeve for M20 Hollo-Bolt were kept constant as 21 mm & 32.6 mm respectively. The sleeve material in all cases was also kept constant as 450 MPa of yield strength. Figure 6.7 shows that the initial stiffness and ultimate capacity of the bolts increased with increasing tube thickness from 12.5 mm to 20 mm. However, the initial stiffness and ultimate capacity of Hollo-Bolt were not affected by the steel tube when the thickness of steel tube is higher than 20 mm.
6.3 Parametric Studies on Blind Bolts under Pure Shear Loading

This section describes a parametric analysis of Hollo-Bolts subjected to pure shear loading, to investigate the behaviour of M16 and M20 Hollo-Bolts in SHS and CFST column connections. Sleeve material yield strength (275–500 MPa), diameter of the grade 8.8 Hollo-bolts and tube wall thickness (6.3–25 mm) were all considered to determine their influences on the initial stiffness and ultimate capacity of Hollo-Bolts in shear. The parametric study was performed based on the configuration as shown in Figure 6.8 for Hollo-Bolts connected to SHS and CFST columns. The dimensions used to build the FE model are shown in Figure 6.8 (a) for CFST column connection and in Figure 6.8 (b) for SHS column connection. In both cases, the cross-sectional dimension of the column was kept the same at 150 × 150 mm. The size of the endplate (150 × 150 mm) for both type of column was also considered the same and the endplate was 15 mm thick throughout the analysis.

The material properties of the steel tube and endplate shown in Table 6.1 in section 6.1 were kept constant throughout the analysis unless otherwise specified. The yield strengths of both the steel tube and endplate were 350 MPa. For the CFST column, the compressive strength of the infill concrete core was taken as 40 MPa throughout the analysis. The bolt shear load was applied at the bottom of the endplate.
The boundary conditions and shear load considered in this analysis are illustrated in Figure 6.9. The boundary conditions for the supports were applied at the top of column as shown in Figure 6.9(a) for the CFST column and in Figure 6.9(b) for the SHS column. FE models were developed in accordance with the details set out in Chapter 5.

Figure 6.8 Configurations of Hollo-Bolts connected to SHS column and CFST column and subjected to pure shear (unit: mm)
6.3 Effect of CFST column and SHS column

The effects of the CFST column and the SHS column were investigated to determine their influences on the load capacity of Hollo-Bolts. In this section, the behaviour of Hollo-Bolts embedding in the concrete core of a CFST column was compared with its behaviour when Hollo-Bolts were connected to an SHS column. Figure 6.10 shows the influences of CFST column and SHS column on the shear load-deformation curves of Hollo-Bolts subjected pure shear load. Here, shear load reported in all figures is the load applied to one single plate and deformation is the vertical plate deformation.
Figure 6.10 Effect of CFST column and SHS column on the load-deformation curves of Hollo-Bolt in shear

(a) M16 Blind-bolt

(b) M20 Blind-bolt

(c) Tube Thickness (20 mm)
It is seen that the ultimate shear capacity of Hollo-Bolts as shown in Figure 6.10(a) and (b) were observed lower when Hollo-Bolts were connected to SHS columns, but found higher when connected to CFST columns. This difference was also observed in Figure 6.10(c) although higher steel tube thickness (20 mm) was being used for both the CFST and SHS columns. This occurred in the SHS column because of the lower shear capacity of the steel tube and the inadequate contact surface between the bolt sleeve and the tube. As a result, punching failure was observed in the sleeve of the Hollo-Bolt, and tearing and bearing failure of the steel tube of the SHS column.

On the other hand, the higher capacity of Hollo-Bolt was observed in the case of CFST column because the contact between Hollo-Bolt sleeve and concrete core increased the bearing surface and enhanced the interaction of Hollo-Bolt sleeve with steel tube and concrete core of CFST column. Also, the concrete restrained the movement of the Hollo-Bolt sleeve and cone during shear loading. As a result, the capacity of Hollo-Bolt increased when connected to CFST column rather than SHS column. Table 6.3 shows the ultimate shear capacity of Hollo-Bolts when connected to SHS and CFST column. The shear capacity of Hollo-Bolt increased 15.81-45.89% when using CFST column, compared to SHS column.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Ultimate shear capacity of Hollo-Bolts (kN)</th>
<th>Load increase using CFST column (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SHS column</td>
<td>CFST column</td>
</tr>
<tr>
<td>M16 Hollo-Bolt</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tube thickness-12.5 mm</td>
<td>146</td>
<td>213</td>
</tr>
<tr>
<td>Tube thickness-20 mm</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>M20 Hollo-Bolt</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tube thickness-12.5 mm</td>
<td>217</td>
<td>286</td>
</tr>
<tr>
<td>Tube thickness-20 mm</td>
<td>253</td>
<td>293</td>
</tr>
</tbody>
</table>

### 6.3.2 Effect of sleeve material

Figure 6.11 shows the effect of different sleeve materials of Hollo-Bolt in which four different mild steel materials (275 MPa; 350 MPa; 450 MPa & 500 MPa, yield strength) were considered as the sleeve material for M20 8.8 grade Hollo-Bolt. In all
cases, the sleeve thickness was kept constant as 5.80 mm where the inner diameter and outer diameter of the sleeve were considered as 21 mm and 32.6 mm respectively. The FE model results for the different sleeve materials shown in Figure 6.11 indicate that the use of higher yield strength sleeve material slightly influences the shear capacity of the Hollo-Bolt when connected either to SHS column or CFST column. When Hollo-Bolts are subjected to shear, the load is carried by the combination of the sleeve and the bolt shank; thus the sleeve material influences the shear capacity of Hollo-Bolt. The effect of the sleeve material was more significant for SHS column connections rather than for CFST column connections, but a higher ultimate load capacity of Hollo-Bolt was found in the case of the CFST column connection.

Figure 6.11 Effect of sleeve material on the load-deformation curves of Hollo-Bolts in shear

6.3.3 Effect of bolt diameter
The effects of bolt diameter (M16 and M20) on the behaviour of Hollo-Bolt subjected to shear load were investigated. The findings are shown in Figure 6.12. In all cases, the thickness of the steel tube (150 × 150 mm) was kept constant at 12.5 mm. The sleeve thickness was kept constant at 5.80 mm. Figure 6.12 shows the ultimate shear capacity of M16 and M20 Hollo-Bolts. The capacity of Hollo-Bolt increased with increasing bolt diameter from 16 mm to 20 mm for a constant steel tube section (150 × 150 × 12.5 mm). A higher load capacity was observed for the M20 Hollo-Bolt due to its larger bolt hole diameter and consequently greater contact surface area. The capacity of the M16 Hollo-Bolt when connected to a CFST column was similar to that when connected to SHS column. This occurred for the M16 bolt because of the very much larger contact area between the sleeve and the concrete core in a CFST connection.
Figure 6.12 Effect of bolt diameter on the load-deformation curves of Hollo-Bolts in shear

6.3.4 Effect of tube wall thickness

Figure 6.13 shows the effect of steel tube wall thickness (6.3, 12.5, 20 and 25 mm) on the behaviour of M20 grade 8.8 Hollo-Bolts. In all cases, the inner diameter and outer diameter of the sleeve were kept constant at 21 mm and 32.6 mm. The sleeve material in all cases was regarded as having constant yield strength of 450 MPa. Figure 6.13 shows that the initial stiffness and ultimate capacity of M20 8.8 grade Hollo-Bolt significantly increased with increasing tube thickness from 6.3 mm to 20 mm, but the differences were small when the wall of the steel tube was more than 20 mm thick.

Figure 6.13 Effect of steel tube thickness on the load-deformation curves of M20 Hollo-Bolts in shear
6.4 Characteristic Behaviour of Blind Bolts
The characteristic behaviour of Hollo-Bolts is discussed here to identify the load-deformation response of different components of a Hollo-Bolt when connected either to SHS column or CFST column; and subjected to pure tension and shear.

6.4.1 Characteristic behaviour of blind bolts in tension
The characteristic behaviour of a Hollo-Bolt connected to an SHS or CFST column and subjected to pure tension is described in Figure 6.14. It can be seen that the load-deformation curve of Hollo-Bolt connected to SHS column differs from when Hollo-Bolt is connected to CFST column. In the case of the SHS column, Figure 6.14 shows four characteristic points, each indicating the different yielding stages of different components of the bolt and its connecting elements, such as steel tube or endplate. Point A occurs due to slip between the sleeve and the steel tube; point B is due to the first yielding of the Hollo-Bolt sleeve. At point C, the sleeve has failed by punching shear failure of the steel tube. The third stage CD is due to punching failure of the steel tube and bolt shank failure.

Figure 6.14 Characteristics of load-deformation curve for Hollo-Bolt in tension, when connected to SHS column and CFST column

For a Hollo-Bolt connected to a CFST column, there are three characteristic points that indicated the different yielding stages of different components of a Hollo-Bolt. First yielding point A’ occurs due to the slip between the sleeve and concrete. There is then a movement in the sleeve which is in firm contact with the concrete, and the load is
carried by bolt shank up to the point \( B' \), which marks the yielding of the bolt shank. At point \( C' \), the Hollo-Bolt shank fractures.

### 6.4.2 Characteristic behaviour of blind bolts in shear

Figure 6.15 shows the characteristic behaviour of a Hollo-Bolt connected to an SHS column or a CFST column and subjected to pure shear loading. The load–deformation curve of a Hollo-Bolt connected to an SHS column has five characteristic points that indicate the different yielding stages of components of the bolt and its connecting elements, such as steel tube or endplate. In stage OA, the load is carried by the sleeve, which starts to yield at point A. Due to local deformation of the sleeve (stage AB), the sleeve comes in contact with the bolt shank, and load is then carried by the bolt shank up to point C, where slip occurs between the bolt sleeve and the steel tube. After point C, there is a movement in the sleeve, which comes firmly in contact with the bolt shank and the steel tube, and the load is carried by the bolt shank up to point D, where further yielding occurs as the sleeve fractures and the bolt shank and bearing of the steel tube around the bolt hole yield. The load is then carried by the bolt shank to point E, where the bearing deformation on the bolt hole causes severe fracturing of the sleeve. The Hollo-Bolt fails to carry any further load.

![Figure 6.15 Characteristics of load-deformation curve for Hollo-Bolt in shear, when connected to SHS column and CFST column](image)

The load-deformation curve of a Hollo-Bolt connected to a CFST column has four characteristic points that indicate the yielding stages of the components of the bolt. The first yield point A occurs when the sleeve first yields. In stage \( AB' \), local deformation
in the Hollo-Bolt sleeve causes it to come in contact with the bolt shank, which then carries the load to point C’. From B’ to C’, no slip takes place between the sleeve and the steel tube because the sleeve is restrained by the core concrete. Point C’ marks the yielding of the bolt shank and the steel tube around the bolt hole. The load is then carried by the bolt shank to point D’, where the bearing deformation of the bolt hole causes the shank to fracture.

6.5 Summary
A parametric analysis is conducted to investigate the behaviour of Hollo-Bolts under pure tension and shear. The effects of the CFST column and SHS column are mainly investigated to determine their influence on the tensile and shear load capacity of the Hollo-Bolts. Sleeve material, bolt material, bolt diameter and steel tube are also considered for both types of columns. It is seen from analyses results that Hollo-Bolt when connected to CFST column in pure tension and shear behave differently from Hollo-Bolts connected to SHS column. The tensile load capacity of Hollo-Bolt increases by 8.6-22.9% when using CFST column, compared to SHS column. The shear load capacity of Hollo-Bolt increases by 15.8-45.9% when using CFST column, compared to SHS column. This indicates that the performance of Hollo-Bolt is improved when connected to CFST column compared to SHS column.
CHAPTER 7

BEHAVIOUR OF BINDING BARS IN CFST COLUMN JOINTS

7.1 Introduction

In the current research, a number of tests were conducted on steel beam-CFST column joints with slab and joint specimen without slab is one. In Chapter 4, it was seen that the behaviour of the steel beam-CFST column joint without slab differed considerably from joints with slab. In particular, the initial stiffness and moment capacity of the joint without a slab were observed to be very much lower than those of the joints with a slab. Significant outward deformation was observed in the steel tube of the CFST column joint without slab. Past research on blind-bolted connections of CFST column joints without slab had also revealed that the outward deformation of the tube walls at the position of the top tension bolts was very locally concentrated. To overcome the steel tube deformation problem and to enhance the capacity of joint, welding cogged extension to blind bolt was proposed by Goldsworthy and Gardner (2006) and extended bolt shank was introduced by Tizani et al. (2004, 2013). In both of these techniques, commercially available blind bolts must be modified.

To avoid the modification of commercially available blind bolts, the current research in this thesis proposes to use binding bars to enhance the integrity of the joint panel zone, which might resolve these issues. The test results demonstrated in Chapter 4 indicate that binding bars have a significant effect on the moment capacity of CFST column joints. However, only limited test results for binding bars are available. In this chapter, a parametric analysis of CFST column joints without slab was conducted to explore their behaviour. This chapter also describes an investigation of the different parameters of CFST column joints with and without binding bars. Based on the results of this analysis, the appropriate number, location and diameter of binding bars are suggested for the design of binding bars for CFST column joints without slab. The parameters related to the CFST column joints with slab are investigated in Chapter 8.
7.2 Parametric Studies on Different Parameters of CFST Column Joints with Binding bars

This section describes parametric analyses that were conducted on joints with various parameters included the blind-bolt type, number of bolts, width-to-thickness ratio of the steel tube, material of the steel tube, column section type and loading type. In particular, these were investigated to find out their influence on the CFST column joints with and without binding bars. The parametric study was performed based on the test specimens presented by Li et al. (2015) and Wang et al. (2009b), in which specimen JS1 in Li et al. (2015) was used as the prototype for joints in tension, and specimen CJM1 in Wang et al. (2009b) for joints subjected to bending. The dimensions used to build the finite element (FE) model are shown in Figure 7.1 for tensile loading and Figure 7.2 for bending loading. The dimensions of the reference joint (JS1) subjected to tensile loading in the axial direction of the beam were 200 × 200 × 5 mm square CFST column and endplate 280 × 200 × 14 mm. CFST column height and beam length (250 × 125 × 6 × 9 mm) were 1400 mm and 600 mm respectively. The basic components of the reference joint (CJM1) subjected to bending loading were: square CFST column (200 ×200 × 8 mm), endplates (340 × 20 × 18 mm), and H-shaped beams (300 × 150 × 6.5 × 9 mm). In both reference joints, four binding bars (two above the upper bolts and two below the lower bolts) were modelled. The beam length and the column height of the reference joint CJM1 were assumed to be 1300 mm and 1400 mm respectively throughout the analysis. The material properties were adopted from test data reported by Wang et al. (2009b). The material properties of the steel tube, beam, endplate, bolt and binding bars in the FE models are listed in Table 7.1.

<table>
<thead>
<tr>
<th>Component</th>
<th>Yield stress (MPa)</th>
<th>Ultimate strength (MPa)</th>
<th>Elastic modulus (MPa)</th>
<th>Elongation at fracture (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel tube</td>
<td>301.7</td>
<td>436.5</td>
<td>200000</td>
<td>15</td>
</tr>
<tr>
<td>Steel beam flange</td>
<td>262.3</td>
<td>377.6</td>
<td>196000</td>
<td>17</td>
</tr>
<tr>
<td>Steel beam web</td>
<td>272.8</td>
<td>380.7</td>
<td>190000</td>
<td>19</td>
</tr>
<tr>
<td>End plate</td>
<td>312</td>
<td>448.4</td>
<td>202000</td>
<td>17</td>
</tr>
<tr>
<td>Bolt M16</td>
<td>640</td>
<td>800</td>
<td>200000</td>
<td>15.68</td>
</tr>
<tr>
<td>Binding bar, φ16 mm</td>
<td>500</td>
<td>630</td>
<td>200000</td>
<td>17.5</td>
</tr>
</tbody>
</table>
Figure 7.1 Configuration of CFST column joint without slab and subjected to tensile loading (unit: mm)

Figure 7.2 Configuration of CFST column joint without slab and subjected to bending loading (unit: mm)
7.2.1 Effect of type of blind-bolts

As seen in the literature review, no studies have been reported comparing the influence of different types of blind bolts on CFST column joints. Indeed the effects of blind-bolts on CFST column joints with or without binding bars have not been investigated. In this section, the effects of two types of blind bolts (Hollo-Bolts and Ajax bolts) on blind-bolted endplate connections of CFST column joints are first investigated, then binding bars are included in the same joints and their effect is analysed. FE models of CFST column joints in tension with Hollo-Bolted connections as shown in Figure 7.3 (a), and Ajax bolted connections as shown in Figure 7.3 (b) are developed to investigate their effect on the CFST column joints. In both cases, M20 bolts are used. Similarly for bending loading, two FE models of CFST column joints in bending for the different types of blind-bolts in the endplate connections are shown in Figure 7.4.

It is observed from the FE results as shown in Figure 7.5 that the load-displacement behaviour of the Hollo-Bolted connections differed from that of the Ajax bolted connection. The main differences were in the initial stiffness and ultimate load capacity for connections in tension. The initial stiffness of the Hollo-Bolted connection was found to be higher than that of the Ajax bolted connection. Because the cone-shaped nuts of the Hollo-Bolts were embedded in the concrete core of the CFST column and formed a secure clamp against pull-out. The nuts of other blind bolts including Ajax bolts, nuts were in hexagonal shape (usually modelled as a round shape in FE modelling) which have no interaction with the concrete and thus have no secure clamping resistance against pull-out. As a result the stiffness and ultimate load capacity of the Hollo-Bolted connection was higher than those of Ajax bolted connection.

Figure 7.6 shows the influence of the blind bolts on the load \((N)\)-displacement \((\Delta_b)\) curves of CFST joints without binding bars when subjected to bending loading. Here, \(N\) is beam load and \(\Delta_b\) is the beam displacement measured at the beam loading point. It is seen in Figure 7.6 (a) that the bending load capacity of the Hollo-Bolted connections exceeded that of the Ajax bolted connections. For example at 100 mm displacement, the bending load was found as 71.5 kN for Hollo-Bolted connections and 64.2 kN for Ajax bolted connections. Due to the insufficient mechanical bonding between the concrete and nut of Ajax bolts, bolt-slip from the concrete of the CFST column in
Figure 7.3 FE models of CFST column joint in tension with different types of blind-bolted endplate connections

Figure 7.4 FE models of CFST column joint in bending with different types of blind-bolted endplate connections
bending was found high for Ajax bolted connections. As a result, more obvious separation of the steel tube from the concrete core was observed for the Ajax bolted connections (see Figure 7.6 (b)). The steel tube outward deformation ($\Delta t$) of the CFST column was observed for both types of connections, which measured at the tube hole of the top bolt hole. This is more clearly seen in the deformation modes shown in Figure 7.7. It was concluded that the tube outward deformation of the steel tube was reduced when Hollo-Bolts were used, but the difference was not significant.

Figure 7.5 Comparative results of CFST column joints in tension with different types of blind-bolts used in endplate connections

(a) Load vs. beam displacement curves  (b) Load vs. tube outward deformation curves
Figure 7.6 Influence of blind-bolts on the load versus displacement ($\Delta b$) curves of CFST joints without binding bars in bending loading
Figure 7.7 Deformation modes of CFST column joints due to the bending loading with different types of bolts

Figure 7.8 Comparative load versus displacement curves of CFST column joints with or without binding bars for different types of blind-bolted connections in bending loading

Figure 7.8 shows the comparative load vs. beam displacement curves of two types of blind-bolted connections with and without binding bars when a bending load was applied on the beam. The bending load capacity of Hollo-Bolted connections was found to be 71.50 kN without binding bars and 87.90 kN with binding bars. The load capacity of Ajax bolted connections was found to be 64.2 kN without binding bars and 71.6 kN with binding bars. It is seen that when four binding bars (two binding bars are used
above the top bolts and two binding bars are used below the bottom bolts) were used in both types of bolted connections, the capacity of Hollo-Bolted connections with binding bars was enhanced by 23% above of the value without binding bars; the capacity of Ajax bolted connections was enhanced by 11.50%. This indicates that the binding bars are more effective in Hollo-Bolted connections than in Ajax bolted connections, due to the combined actions of the clamping mechanisms of Hollo-Bolts and the binding bars.

7.2.2 Effect of number of blind-bolts

This section describes the investigation of the effect of the total number of Hollo-Bolts in the beam-column joints. CFST column joints were modelled based on six Hollo-Bolted connections and four Hollo-Bolted connections. The basic components of both types of joints were similar to those illustrated in Figure 7.2, except for the number of Hollo-Bolts connecting the endplate to the CFST column.

The FE model results of CFST column joints with different numbers of Hollo-Bolted connections are shown in Figures 7.9 and 7.10 for bending. Figure 7.9(a) shows the joint load-displacement behaviour of the steel beam for six-bolt and four-bolt connections. It is seen that the ultimate load capacity of the CFST column joints increased by increasing the number of Hollo-Bolts from four to six. Figure 7.9(b)
shows the joint load-outward deformation (i.e., horizontal deformation $\Delta_t$) in the steel tube of the CFST column, measured at a point 25.5 mm from the centre of the top bolts. It was predicted that the maximum outward deformation of the steel tube would be found near the top bolts of both types of blind-bolted connection, as shown in Figure 7.10. For the same joint load, a larger steel tube deformation (Figure 7.9(b)) was predicted for the four-Hollo-Bolted connection than six-Hollo-Bolted connection. It is also seen in Figure 7.9(b) that the outward deformation of the steel tube rapidly increased with the joint load increments. Two conclusions may be drawn from these results: one is that the joint capacity of blind-bolted endplate connections to CFST columns is reduced by the outward deformation of the steel tube of the CFST column; another is that the interaction between the steel tube and the concrete core of CFST columns is enhanced by increasing the number of Hollo-Bolts used. Since the tensile strength of concrete is very low, outward deformation of the steel tube of the CFST column may be reduced by adding stiffeners to the tube of the CFST column.

![Deformation modes of CFST column joints due to the bending loading](image)

Figure 7.10 Deformation modes of CFST column joints due to the bending loading with different number of Hollo-Bolts (at 80 mm beam tip's deflections)

### 7.2.3 Effect of width-to-thickness ratio of the CFST column

The effect of width-to-thickness ratio ($B/t_c = 43.33$, 50 and 83.33) of the steel tube of CFST column was investigated in this section. In this case, the Figure 7.11 shows a comparison of the load-displacement behaviour of CFST column joints for three different width-to-thickness ratios. The ratio has a significant effect on the ultimate
capacity of CFST column joints. The ultimate load capacity of CFST joints without binding bars is increased with decreasing width-to-thickness ratio of the steel tube (i.e., by increasing the steel tube thickness), but the initial stiffness of the joint shows little change. However, when binding bars are attached to the steel tube of the CFST column, the connections show significantly increased ultimate load capacity and initial stiffness with decreased width-to-thickness ratio of the steel tube compared to the values when no binding bars are present. This implies that the ultimate load capacity of CFST column joints are enhanced by increasing the steel tube thickness and adding binding bars to the CFST column.

Figure 7.11 Load–displacement curves for different width-to-thickness ratios ($B/t_c$) of the steel tube of CFST column joints in tension

**7.2.4 Effect of steel tube material**

Carbon steel and stainless steel were used to fabricate the steel tube of CFST columns. This section describes the examination carried out on the effect of the different steel tube materials to determine their influence on the behaviour of CFST column–beam connections.

FE models of CFST columns were developed for the connections of carbon steel columns and stainless steel columns to carbon steel beams. Figure 7.12 shows a comparison of the load-displacement behaviour of the CFST column joints for the two materials. These results indicate that the initial stiffness of the CFST column joints is similar for both types of steel tube, but CFST column joints with columns fabricated
from stainless steel tubes have a higher ultimate load capacity due to the higher strain-hardening effect of stainless steel.

Figure 7.12 Effect of steel tube material on the load–displacement behaviour of CFST column joints with and without binding bars ($t_c = 6$ mm, $t_p = 14$ mm)

### 7.3.5 Effect of column section and loading type

The influence of binding bars was investigated for different combinations of column cross-section and applied loading type, specifically for square and circular column sections and tensile and bending loading. Figure 7.13 shows the influences of the column section and loading type on the load-displacement ($N/\Delta_b$) behaviour of CFST joints, both with and without binding bars. Figure 7.13(a) shows that the presence of binding bars showed no significant difference in the load-carrying capacity of column-beam connection joints for circular CFST columns compared to square CFST columns, when both were subjected to tensile loading. However, when both types of CFST column joints were subjected to bending loading, the presence of binding bars had a considerable influence on the load-displacement behaviour of CFST joints for both square and circular column sections used in the CFST column joints (Figure 7.13(b)). This indicated that the binding bars had a greater effect on square-column CFST joints for both loading cases, but only in bending loading for joints with circular CFST columns.
(a) Square CFST column joints in tension  (b) Circular CFST column joints in tension

(c) Square CFST column joints in bending  (d) Circular CFST column joints in bending

Figure 7.13 Influence of column section and loading type on the load \( (N) \)-displacement \( (A_b) \) curves of CFST joints with and without binding bars

7.3 Determination of Number, Location and Diameter of Binding Bars

The parametric analyses discussed in previous sections indicate that the presence of binding bars is only one parameter that may significantly enhance the initial stiffness and ultimate load capacity of CFST column joints. It can be concluded that the use of binding bars in blind-bolted endplate connections may help to solve the problem of outward deformation of the steel tube of the column and thereby increase the loading capacity of the joint. However, before binding bars are recommended to be adopted in construction practice, the first concern should be how to design the binding bars. To propose suitable design guidelines for the use of binding bars in blind-bolted endplate connection joints of square CFST columns to steel beams, the number, position and diameter of binding bars are investigated in this section.
Figure 7.14 Configurations of CFST column joints with different number of Hollo-Bolts (unit: mm)
FE models were developed for square CFST column-to-steel beam joints with blind-bolted endplate connections (CFST joints, for simplification). Two types of CFST joints were designed with either six or four blind bolts. For the CFST joint with six blind bolts, the side length and thickness of steel tube for the square column were 200 mm and 8 mm, respectively; the height, width, thickness of flange and thickness of web of the H-shaped steel beam were 300, 150, 9 and 6.5 mm, respectively; the height and width of the endplate were 340 and 200 mm, respectively. More details of the dimensions of the CFST joint with six blind bolts are shown in Figure 7.14(a). The CFST joint with four blind bolts was identical except for the number of bolts. The two bolts closest to the centreline of the beam were removed, leaving four blind bolts in the CFST joint as shown in Figure 7.14(b).

The compressive strength of the core concrete was 56 MPa; the yield stress of the steel tube was 302 MPa; the yield stress of the steel in the flange and web of the beam were 262 MPa and 272 MPa, respectively. M16 Hollo-Bolts were chosen with a yield stress 754 MPa. The pre-tension force was 45 kN.

7.3.1 Total number of binding bars
A total of 14 FE models are developed for both types of joints (Joint-1 and Joint-2) and analysed to determine the appropriate number of binding bars and their positions. Seven FE models were developed for each type of CFST joint, one for each binding bar configuration. Figure 7.15(a) shows the CFST column joint without binding bars (configuration BB0). Figure 7.15(b)-(h) shows the configurations of Joint-1 CFST column joints with different arrangements of binding bars (BB1, BB2, BB3, BB4, BB5, BB6, BB7). Figure 7.16 illustrates two of these FE models for Joint-1 analysis: BB0 without binding bars, and BB3 with binding bars. The same binding bar arrangements were modelled for Joint-2 connections BB1, BB2, BB3, BB4, BB5, BB6 and BB7.

The FE model results for the CFST column joints with different arrangements of binding bars are shown in Figures 7.17, 7.18 and 7.19. Figure 7.17 shows the load–displacement behaviour of the CFST column joints without binding bars (BB0) and with binding bars (BB1 to BB7). It is seen in Figure 7.17(a) for Joint-1 and Figure 7.17(b) for Joint-2 that the load capacity of blind-bolted endplate connections with binding bars are found to be 15.1-29.4% (for Joint-1) and 7.5-35.5% (for Joint-2)
higher than that for the corresponding connection without binding bars (BB0). This indicates that the load capacity of blind-bolted endplate connections can be enhanced by increasing the number of binding bars on the steel tube of the square CFST column. It was also noticed in Figure 7.17(a) and (b), and in Table 7.2, that the load capacities of joints with the BB3, BB5, BB6 and BB7 binding bar arrangements are greater than those for the joints with the BB1, BB2 and BB4 arrangements. In addition, the steel tube outward deformations (shown in Figure 7.18) are compared at the corresponding joint load capacities of the CFST column joints with different binding bars arrangements (BB0, BB1, BB2, BB3, BB4, BB5, BB6 and BB7). It is also noted that the maximum joint load capacities occurred at the minimum steel tube outward deformation for the joints with the binding bar arrangements BB3, BB5, BB6 and BB7 rather than the other joints with binding bar arrangements BB0, BB1, BB2 and BB4 (as shown in Table 7.3).

Figure 7.15 Different binding bar arrangements used for six-blind-bolted connections: a) BB0, b) BB1, c) BB2, d) BB3, e) BB4, f) BB5, g) BB6, h) BB7 (unit: mm)
Figure 7.19 shows the percentage of joint load increment at the minimum steel tube outward deformations (2 and 4 mm) for different numbers of binding bars used in Joint-1 and Joint-2 configurations. It can be seen that the outward deformation of the steel tube can be reduced by providing either four, six or eight binding bars; however, four binding bars (BB3) are sufficient to enhance the joint load capacity by about 49.3% for Joint-1 and 66.6% for Joint-2 at the minimum steel tube deformation (allowing 2 mm deformation at the serviceability limit).

Figure 7.20 shows the deformation modes of CFST column joints with four binding bars (BB3) and without binding bars (BB0). It is clear from the deformation modes that the outward deformation of the steel tube is reduced when binding bars are used on the steel tube. From this it is inferred that, to optimise the use of the binding bars, they should be placed only above and below the bolts that are subjected to tensile load. The outward deformation of the steel tube is then reduced and the tensile forces in the steel tube are carried by the binding bars and then transferred by the binding bars to the core concrete in the CFST column.

![FE model of CFST joints](image)

Figure 7.16 FE model of CFST joints (Joint-1) without binding bars (BB0) and with binding bars (BB3)
Figure 7.17 Load versus displacement curves of connections with different binding bars arrangements

Figure 7.18 Steel tube outward deformation of CFST column for different binding bars arrangement

Table 7.2 Ultimate load capacity of blind-bolted connections at 90 mm beam deflection

<table>
<thead>
<tr>
<th>FE model no</th>
<th>Connection capacity (kN)</th>
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<tr>
<td></td>
<td>Six-bolt connection</td>
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<tr>
<td>BB0</td>
<td>70.37</td>
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<tr>
<td>BB1</td>
<td>81.03</td>
</tr>
<tr>
<td>BB2</td>
<td>81.60</td>
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<td>BB3</td>
<td>90.19</td>
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<td>BB4</td>
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<td>BB5</td>
<td>93.70</td>
</tr>
<tr>
<td>BB6</td>
<td>90.45</td>
</tr>
<tr>
<td>BB7</td>
<td>91.06</td>
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Table 7.3 Allowable connection capacity of ‘Joint-1’ and ‘Joint-2’ for different binding bars arrangement

<table>
<thead>
<tr>
<th>FE model no</th>
<th>Connection capacity (kN)</th>
<th>Six-bolt connection</th>
<th>Four-bolt connection</th>
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<td></td>
<td>for 2 mm and 4 mm steel tube deformation</td>
<td>Tube deformation (2 mm)</td>
<td>Tube deformation (4 mm)</td>
</tr>
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<td>BB0</td>
<td></td>
<td>52.30</td>
<td>60.50</td>
</tr>
<tr>
<td>BB1</td>
<td></td>
<td>60.50</td>
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<td>87.00</td>
</tr>
<tr>
<td>BB7</td>
<td></td>
<td>79.10</td>
<td>87.90</td>
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</tbody>
</table>

(a) For 2 mm tube deformation of Joint-1
(b) For 2 mm tube deformation of Joint-2
(c) For 4 mm tube deformation of Joint-1
(d) For 4 mm tube deformation of Joint-2

Figure 7.19 Load increase (%) at the minimum steel tube outward deformation (2 mm and 4 mm) when different numbers of binding bars used in Joint-1 and Joint-2
7.3.2 Location of binding bars

To determine the spacing of the binding bars relative to the bolt hole in the steel tube, four different types of binding bar spacing ($P = 2.0D$, $2.5D$, $3.0D$ and $3.5D$) were modelled for CFST column joints P1, P2, P3 and P4 respectively. The configurations of these four CFST column joints were considered to be almost identical to the configuration of joint BB3 as illustrated in Figure 7.15(d), except the only difference being the spacing of the binding bars. Figure 7.21 shows the configuration of the CFST
column joint P1 with six Hollo-Bolts (Joint-1) and four Hollo-Bolts (Joint-2). Table 7.4 shows the binding bars spacing \( P \) used for different connection configurations.

The FE model results of CFST column joints P1, P2, P3 and P4 are also compared with the result for connection BB0 (without binding bars). The joint load capacity-beam deflection results for connections BB0, P1, P2, P3 and P4 are shown in Figure 7.22 (a) for Joint-1 and in Figure 7.22 (b) for Joint-2. The outward deformation of the steel tube at the corresponding joint load is shown in Figure 7.23 (a) for Joint-1 and in Figure 7.23 (b) for Joint-2. It is seen that the joint load capacity is enhanced by the smallest binding bar spacing \( P = 2.0D \), but a very significant stress concentration on steel tube is seen where the binding bars are connected to the steel tube. When the spacing of binding bars is increased from 2.5\( D \) to 3.5\( D \), the fracture at the connection between the steel tube and binding bars is reduced considerably. In those cases, the ultimate capacity of CFST column joints P2, P3, P4 is reduced slightly, but the reduction is not significant compared to the capacity of joints BB0. This indicates that consideration of the best binding bar spacing could overcome the steel tube fracture where the steel tube is joined to the binding bars. The binding bars spacing can be considered as \( P = 3.0D \) to avoid the fracture in the steel tube of the CFST column, and to enhance the load capacity of both Joint-1 and Joint-2.

![Figure 7.21 Location of binding bars on CFST column joints](image-url)
Table 7.4 Spacing details of binding bars considered in the parametric analysis

<table>
<thead>
<tr>
<th>FE model no for CFST column joints</th>
<th>Total binding bars</th>
<th>Spacing of binding bars from the centre of top bolts</th>
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</thead>
<tbody>
<tr>
<td>P1</td>
<td># 4Φ16 binding bars</td>
<td>$P = 2.0D$</td>
</tr>
<tr>
<td>P2</td>
<td># 4Φ16 binding bars</td>
<td>$P = 2.5D$</td>
</tr>
<tr>
<td>P3</td>
<td># 4Φ16 binding bars</td>
<td>$P = 3.0D$</td>
</tr>
<tr>
<td>P4</td>
<td># 4Φ16 binding bars</td>
<td>$P = 3.5D$</td>
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</table>

Note: $D$ is bolt diameter; horizontal spacing of binding bars are considered the same as bolts spacing.

(a) Joint-1(six-bolt-bolted connections)  (b) Joint-2 (four-bolt-bolted connections)

Figure 7.22 Load versus displacement curves of CFST column joints for different spacing of binding bars

(a) Joint-1(six-bolt-bolted connections)  (b) Joint-2 (four-bolt-bolted connections)

Figure 7.23 Outward deformation of steel tube in CFST column joints for different spacing of binding bars
7.3.3 Diameter of binding bars

To find out the most effective binding bar diameter, four diameters \( d_{bb} = 1.00D, 1.25D, 1.50D \) & \( 1.75D \) as shown in Table 7.5) are considered for CFST column joints D1, D2, D3 and D4 respectively, where the binding bar spacing \( P \) is taken to be \( P = 3.0D \) as CFST column joints P3.

Table 7.5 Diameters of binding bars considered in the parametric analysis

<table>
<thead>
<tr>
<th>FE model no for CFST column joints</th>
<th>Total number of binding bars</th>
<th>Spacing of binding bars (( P ))</th>
<th>Diameter of binding bars ( (d_{bb}) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>D0 (BB0)</td>
<td>Without binding bars</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>D1</td>
<td>4( \phi )16 binding bars</td>
<td>3.0( D )</td>
<td>1.00( D )</td>
</tr>
<tr>
<td>D2</td>
<td>4( \phi )16 binding bars</td>
<td>3.0( D )</td>
<td>1.25( D )</td>
</tr>
<tr>
<td>D3</td>
<td>4( \phi )16 binding bars</td>
<td>3.0( D )</td>
<td>1.50( D )</td>
</tr>
<tr>
<td>D4</td>
<td>4( \phi )16 binding bars</td>
<td>3.0( D )</td>
<td>1.75( D )</td>
</tr>
</tbody>
</table>

The effect of binding bar diameter on the CFST column joints was investigated to enhance the joint load capacity and to minimise the steel tube deformation and fracture at the connection between the steel tube and binding bars, and compared to the CFST column joints without binding bars (D0). The joint load capacity-beam deflection results of CFST column joints D0, D1, D2, D3 and D4 are shown in Figure 7.24 (a) and (b) for Joint-1 and Joint-2 respectively. The outward deformation of the steel tube is shown in Figure 7.25 (a) (Joint-1) and Figure 7.25 (b) (Joint-2). The load capacity of Joint-2 was improved by increasing the binding bar diameter from 1.25D to 1.75D compared to Joint-1, compared to the joint D0. The outward deformation of the steel tube corresponding to the joint load capacities of connections D2 and D3 were found to be almost identical for binding bar diameters 1.25D and 1.5D used on CFST column joints D2 and D3 respectively. To optimise the binding bars diameter and to minimise the steel tube outward deformation and fracture, the diameter of 1.25D can be considered as effective binding bars diameter.
Figure 7.24 Load versus displacement curves of CFST column joints for different diameters of binding bars

(a) Joint-1 (six-bolt-bolted connections)  
(b) Joint-2 (four-bolt-bolted connections)

Figure 7.25 Steel tube outward deformations of CFST column joints for different diameters of binding bars

(a) Joint-1 (six-bolt-bolted connections)  
(b) Joint-2 (four-bolt-bolted connections)

7.4 Design Recommendations for Binding Bars

The significant contribution of binding bars to improve the load carrying capacity of CFST joints and simultaneously reduce the outward deformation of the steel tube is demonstrated in the parametric analysis. Design recommendations and a design example are illustrated in this section.

The design recommendations of binding bars are derived based on the parametric analysis conducted in the previous section, and the following three aspects should be determined accordingly:
(1) The numerical analysis results indicate that binding bars are more effective when binding bars are placed only at the top and bottom of the tensile bolts. Actually, two binding bars are required at the top and bottom of the tensile bolts respectively to transfer the tensile bolt forces into the core concrete of the CFST column.

(2) The proposed position for the binding bars is that the top binding bars are placed above the top bolts (tensile bolts) at a maximum distance of $2.5D$ away from the centre of top bolts, and the bottom binding bars are placed below the top bolts (tensile bolts) at a maximum distance of $2.50D$ away from the centre of top bolts (Figure 7.26). The minimum clearance distance from the outer diameters bolt holes to the outer diameters of binding bars should be considered as 15 mm. The horizontal spacing of the two top or two bottom binding bars can be considered same as the horizontal spacing of the two blind bolts.

(3) The diameters of binding bars are suggested as 1.25D.

Figure 7.26 Configuration of binding bars according to the designed recommendation

Through the above three steps, the binding bars can be designed to minimise the steel tube outward deformation and improve the bearing capacity of CFST joints. An example using the design recommendations is given as follows.

The design of binding bars for CFST joints with six blind bolts is demonstrated in this section. The known conditions include: the diameter of blind bolt M16 is 16 mm, the horizontal spacing of two blind bolts is 100 mm, and two blind bolts in the top row are in tension. The number, position and diameter of binding bars are decided as follows:
(1) Determine the number of bars:

Total number of binding bars \((n)\) is determined based on the total number of tensile bolts \((N)\), and the equation is: \(n = 2N = 2 \times 2 = 4\).

(2) Determine the position of bars:

Vertical spacing \((P_1)\) of the top and bottom binding bars from the centre of the top bolts is calculated from \(P_1 = 3.0D = 3.0 \times 16 = 48\) mm.

Horizontal spacing of the binding bars \((P_2)\) = horizontal spacing of the two blind bolts, \(\therefore P_2 = 100\) mm.

(3) Determine the bar diameter:

The diameter \((d_{bb})\) of binding bars is determined from the diameter of the blind bolts.

The equation is: \(d_{bb} = 1.25D = 1.25 \times 16 = 20\) mm.

### 7.5 Summary

The separation of the steel tube from the concrete core in the CFST column is one of the most common phenomena observed in steel beam-CFST column joints when the tensile load or bending moment is transferred to the steel tube from the beams. To reduce the steel tube separation, binding bars were utilised as stiffeners for the steel tube of the CFST column near the connections. The behaviours of the steel beam-CFST column connections with and without binding bars were investigated using FE analysis. The parametric analyses results showed that binding bars can be welded to the steel tube of the CFST column to reduce the problem of outward deformation of the CFST column and to enhance endplate connection capacity. A design example of binding bars has been demonstrated for blind-bolted endplate connections. The simplified calculation details for binding bar design can be helpful for correctly connecting binding bars to the steel tube of the CFST column.
CHAPTER 8

PARAMETRIC STUDY FOR CFST COLUMN JOINTS WITH SLAB

8.1 Introduction

In Chapter 7 the parametric analysis results of steel beam-CFST columns joints without slab were set out. This chapter mainly focuses on a parametric analysis of the steel beam-CFST column joints with composite slab. Very limited experimental and numerical results on these joints, with the presence of a slab, have been reported in the past. The behaviour of the CFST column joints with slab is normally influenced by several parameters such as those relating to the connection, composite slab parameters, composite beam parameters and CFST column parameters. Few of these parameters have been investigated in the past for the composite CFST column joints (CFST column joints with slab). Due to the high cost and time to conduct experimental work, it is very difficult to include all of these parameters in the testing of composite joints. However, the influence of these parameters may be modelled and investigated by FE analysis.

In this chapter, a parametric study using FE analysis was conducted to cover all the parameters related to the composite CFST column joints. In the analysis, blind-bolted endplate connections were considered to connect the steel beams to CFST column. Stainless steel material was considered for the steel tube of the CFST column and mild steel material was considered for all other steel components of the composite CFST column joints. Throughout the parametric analysis, specimen SB1-2 was considered as the reference joint. The configuration details of specimen SB1-2 are illustrated in Figure 3.6, Chapter 3. The analyses results of composite CFST column joints are illustrated step by step in the following.
8.2 Effects of Connection’s Parameters

In blind-bolted endplate connections, blind-bolts and endplate are two of the connecting components to connect the steel beams to the CFST columns. These two connecting components are illustrated here as connection parameters. The connecting parameters such as the number of bolts and endplate thickness were investigated to determine their influences on the composite CFST column joints. Table 8.1 shows the details of the connecting parameters that were considered in the analysis to observe their influence on the composite joints.

Table 8.1 Connection parameters

<table>
<thead>
<tr>
<th>Parameter name</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Number of bolt rows</td>
<td>Two bolt rows, three bolt rows</td>
</tr>
<tr>
<td>2 Endplate thickness ($t_p$)</td>
<td>10, 14, 18 mm</td>
</tr>
</tbody>
</table>

8.2.1 Number of bolt rows

To investigate the behaviour of composite joints with different numbers of blind-bolts, one connection of composite CFST column joint was considered with six M20 blind bolts in three rows, as shown in Figure 8.1(a) and another considered four M20 blind bolts placed in two rows as shown in Figure 8.1(b). In both cases, slab reinforcements were kept constant in the longitudinal direction (6Ø16@160) and in the transverse direction (18Ø10@200), with four active slab reinforcement bars in the longitudinal direction and 16 in the transverse direction. Other parameters are the same as for reference joint (specimen SB1-2).

Figure 8.2 shows the moment-rotation behaviour of the composite CFST column joints for the different numbers of bolt rows in the endplate connections. It is seen from the FE model results of the connections with two and three rows of bolt that the ultimate moment capacity of the joint was not increased by increasing the number of rows of bolts. Indeed, even the initial rotational stiffness was not changed despite increasing the number of blind-bolt rows from two to three. The initial rotational stiffness of a composite CFST column joint mainly depends on the capacity of the top row of tensile bolts and slab reinforcements only, not on the middle row of bolts. On the other hand, the ultimate moment capacity of a composite CFST column joint depends on all of the connecting components, as well as on the plastic neutral axis. It is seen from Figure 8.2
that the moment capacity of joints was not increased because the plastic neutral axis is located at the centre of the middle bolt row, causing the bolts in that row to go into compression. Thus they do not contribute to resisting tensile force and, as a result, they do not influence the moment capacity. It is concluded that the composite CFST column joint capacity is increased by adding more rows of bolts only if they are located above the plastic neutral axis. The locations of blind bolts should be optimised in a design.

![Figure 8.1 Configurations of bolts used in the parametric analysis of composite CFST column joints (unit: mm)](image)

**Figure 8.1** Configurations of bolts used in the parametric analysis of composite CFST column joints (unit: mm)

![Figure 8.2 Effect of number of bolt rows on the moment-rotation behaviour of composite CFST column joints](image)

**Figure 8.2** Effect of number of bolt rows on the moment-rotation behaviour of composite CFST column joints

### 8.2.2 Endplate thickness

Three FE models were developed to investigate the effect of endplate thickness on the composite CFST column joints, in which only the endplate thickness was varied between 10, 14 and 18 mm. In all cases, the slab reinforcement ratio in the longitudinal
direction ($\rho = 1.35\%$) was kept constant. Other parameters are considered to be the same as the reference joint (specimen SB1-2). Figure 8.3 compares the moment–rotation behaviour of composite CFST column joints for different endplate thickness.

The initial rotational stiffness and ultimate moment capacity of the joints were not changed significantly when the endplate thickness was increased from 10 to 18 mm. This may be due to the presence of the composite slab, because the bending and tensile forces were carried by the beam, whereas the endplate mainly carried the shear and bearing force rather than bending force. In most cases, joint failure was observed as beam flange buckling, not the endplate.

![Figure 8.3](image)

**Figure 8.3** Effect of endplate thickness on the moment-rotation behaviour of composite CFST column joints

### 8.3 Effects of Slab Parameters

There are five parameters related to the composite slab: thickness, reinforcement ratio, concrete strength, profiled sheeting strength and orientation of the profiled sheeting orientation. The effects of these parameters, listed in Table 8.2, were investigated to understand the structural behaviour of composite CFST column joints.

<table>
<thead>
<tr>
<th>Table 8.2 Slab parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parameter name</td>
</tr>
<tr>
<td>1  Slab thickness ($D_c$) in mm</td>
</tr>
<tr>
<td>2  Slab reinforcement ratio ($\rho$) in %</td>
</tr>
<tr>
<td>3  Slab concrete material strength ($f_c'$) in MPa</td>
</tr>
<tr>
<td>4  Profile sheet material strength ($f_y$) in MPa</td>
</tr>
<tr>
<td>5  Profile sheet orientation</td>
</tr>
</tbody>
</table>
8.3.1 Slab thickness

Figure 8.4 shows the comparative results of the moment-rotation curves of composite CFST column joints for five different slab thicknesses (0, 100, 120, 150 and 200 mm). In all cases, the slab reinforcement was kept constant including four active longitudinal bars (6Ø16@160) and 16 transverse bars (18Ø10@200. Other parameters were considered to be the same as the reference joint (specimen SB1-2).

It is seen from the comparison of results as shown in Figure 8.4 that the moment-rotation behaviour of CFST column joints without slab ($D_c = 0$ mm) was much lower than that of the CFST column joints, for various slab thicknesses ($D_c = 100, 120, 150,$ and $200$ mm). The moment and rotational stiffness of the joints significantly increased, but the rotation capacity reduced, when the slab thickness was increased. The lever arm of the slab reinforcement bars from plastic neutral axis increased with increasing slab thickness. As a result, the moment capacity and initial rotational stiffness were increased. The rotation capacity of the joints was reduced with greater slab thickness because the deformation of the reinforcement was limited for all slabs, but the distance from the slab reinforcement to the bottom flange of the beam was increased with increasing slab thickness. It can be concluded from results that the presence of slab and its thickness have significant influence on the behaviour of CFST column joints.

Figure 8.4 Effect of slab thickness on the moment-rotation behaviour of composite CFST column joints
8.3.2 Slab reinforcement ratio

Joints with six different slab reinforcement ratios were modelled to determine the effect of slab reinforcement on the behaviour of composite CFST column joints. The slab reinforcement ratio was calculated from the total reinforcement area and effective slab cross-sectional area of $b_t \times (D_c-h_c)$, where $D_c$ is the slab thickness and $h_c$ is the rib height of the profiled sheeting. Slab reinforcement was varied only in the longitudinal direction; reinforcement in the transverse direction was kept constant at 18Ø10@200. In all cases, the active slab reinforcement comprised four longitudinal bars and 16 transverse bars. Eight shear studs (8Ø19) were used in all cases. Other parameters were considered to be the same as for the reference joint (specimen SB1-2).

Figure 8.5 shows the comparison for the joints with six slab reinforcement ratios $\rho = 0.53, 0.76, 1.04, 1.35, 2.12$ and 3.31%). When the ratio was less than 1.0 %, the joints failed due to the reinforcement failure only. When the slab reinforcement ratio was increased to over 1.0%, the joint failed by the combined failure of the slab reinforcement and local buckling of the bottom flange of the beam. The FE model also showed that the moment capacity and initial rotational stiffness of the joints were increased when the slab reinforcement ratio was increased. This indicates that the slab reinforcement plays a big role in improving the moment capacity and initial rotational stiffness of the composite CFST column joints. In the analysed examples, the joint failure due to slab reinforcement fracture can be avoided by increasing the slab reinforcement ratio to 1.0% and above.

![Figure 8.5 Effect of slab reinforcement ratio on the moment-rotation behaviour of composite CFST column joints](image)

Figure 8.5 Effect of slab reinforcement ratio on the moment-rotation behaviour of composite CFST column joints
8.3.3 Slab concrete material strength

In general, the strength of the slab concrete is ignored in the calculation of the joint hogging moment capacity because the concrete tensile resistance capacity is very low compared to the slab reinforcement capacity. In this section, the effect of slab concrete materials was investigated for a constant slab reinforcement ratio of 1.2%.

Figure 8.6 shows the comparison of the moment-rotation behaviour for joints with different slab concrete strengths (24, 32, 44, 50 and 60 MPa). It is seen from the results obtained from the FE models that the moment capacity and initial rotational stiffness of the composite joints were not affected by the slab concrete strength. This indicates that the strength of the slab concrete can be ignored when calculating the moment and stiffness capacity of composite CFST column joints.

![Figure 8.6](image)

Figure 8.6 Effect of slab concrete material strength on the moment-rotation behaviour of composite CFST column joints

8.3.4 Profiled sheeting material strength

The contribution from the profiled sheeting is generally ignored in the calculation of the moment capacity of composite joints since it is difficult to ensure the continuity of the sheeting in the joint area. In this section, the effect of profile sheet material on the moment–rotation curves of the composite CFST column joints was investigated. Six FE models of composite joints were developed for different profile sheet materials having yield strengths of 0 (i.e., no profile sheet used), 50, 150, 275, 450 and 690 MPa. Its elastic modulus was kept constant at 222532 MPa. In all cases, longitudinal (6Ø16@160) and transverse slab reinforcement (18Ø10@200) remained constant.
active longitudinal slab reinforcement bars and 16 transverse bars were used. Other
parameters are considered to be the same as the reference joint (specimen SB1-2).

Figure 8.7 Effect of profile sheet material strength on the moment-rotation curves of the
composite CFST column joints

Figure 8.7 shows the comparison of the moment-rotation behaviour of joints with six
profile sheet yield strengths. The FE modelling results indicated that the moment
capacity of the joints was significantly influenced by the presence of the profile sheet.
However, the initial rotational stiffness was not changed by changing the profile sheet
yield strength. The results for the composite joint with no profile sheet (i.e., $f_y = 0$ MPa
in Figure 8.7) indicated that the moment capacity of joints was lower than joints with a
profile sheet, regardless of its yield strength. Increasing the yield strength of the profile
steel raised the total tensile resisting capacity above the plastic neutral axis. The
additional tensile strength provided by the profile sheet plays a key role in enhancing
the ultimate moment capacity of composite joints. This was observed due to the tensile
strength of the profile sheet and the lever arm from the plastic neutral axis to the level
of the profile sheet. It can be concluded that if the continuity of the profiled sheeting
can be guaranteed, the profiled sheeting strength may be considered in the calculation
of the moment capacity of composite CFST column joints but not in the calculation of
the initial rotational stiffness. Further research is required on the continuity of profiled
sheeting.
8.3.5 Orientation of profiled sheeting

In the previous section, it was seen that the existence of profile sheet influenced the moment capacity of composite joints, and the profile sheet may be incorporated in the calculation of the moment capacity of the joints. This could raise a question as to whether or not the orientation of the profile sheet influences the moment capacity of the composite joints. To clarify this point, the effect of profile sheet orientation was modelled for two orientations, with the ribs parallel and normal to the primary beam, as shown in Figure 8.8. In all cases, the slab reinforcement was kept constant in the longitudinal direction (6Ø16@160) and in the transverse direction (18Ø10@200), with four longitudinal active bars and 16 transverse bars, as before. All other parameters were considered to be same as the reference joint (specimen SB1-2).

![Figure 8.8 Profile sheet orientations used in FE modelling of composite CFST column joints](image)

Figure 8.8 Profile sheet orientations used in FE modelling of composite CFST column joints

Figure 8.9 shows the comparison of the moment-rotation behaviour for the two profile sheet orientations. It is seen from the results obtained from the FE models that the moment capacity of the composite joint is significantly affected by the orientation of...
the profile sheet. With the ribs of the profile sheet running transverse to the beam, the moment capacity of joints was 10.65% less than when the ribs were parallel to the beam. This is similar to the observed difference when the profile sheet was ignored in the composite slab. It may be concluded that when a profile sheet is placed on the steel beam with the ribs running transverse to the beam, it is predicted that the moment capacity would be reduced by about 10.65% of the moment capacity of the joint with a composite slab with its ribs running parallel to the beam.

Figure 8.9 Effect of profile sheet orientations on the moment-rotation behaviour of the composite CFST column joints

### 8.4 Effects of Composite Beam Parameters

Three parameters related to composite beams were considered in this section. The shear connectors (19 mm diameter) are important in developing composite action between steel beams and the composite slab. It was seen in the literature review that the composite action between the steel beam and slab greatly influences the capacity of composite joints. Other parameters related to the beam are its depth and its material strength, both of which affect the performance of the joint. The effects of these parameters on the composite CFST column joints were investigated. Table 8.3 shows the details of the different composite parameters.
Table 8.3 Composite beam parameters

<table>
<thead>
<tr>
<th>Parameter name</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Shear connector ratio, ( \eta_y ) (%)</td>
<td>40, 80, 120, 180, 200</td>
</tr>
<tr>
<td>2 Beam depth, ( H_b ) (mm)</td>
<td>250UB37.3, 310UB40.4, 360UB50.7, 460UB82.1, 530UB82.0</td>
</tr>
<tr>
<td>3 Beam material strength, ( f_y ) (MPa)</td>
<td>275, 350, 450, 550, 690</td>
</tr>
</tbody>
</table>

8.4.1 Shear connector ratio

Six different degrees of shear connectors (\( \eta_y = \frac{N_{sc}F_{sc,max}}{A_{r}f_{y,r}} \times 100\% = 48, 76, 96, 120, 144, 192\% \)) were considered to investigate the effect of shear connectors on the behaviour of composite CFST column joints. Here, \( N_{sc} \) is the total number of shear connectors; \( F_{sc,max} \) is shear capacity of a single shear connector; \( A_{r} \) is the total area of slab reinforcement and \( f_{y,r} \) is the yield strength of slab reinforcement. In all cases, the first stud spacing was kept constant at 100 mm and the slab reinforcement ratio in the longitudinal direction was kept constant at \( \rho = 1.35\% \). All other parameters including material properties, joint configurations were identical to those in the reference joint (SB1-2). The shear connector ratio was calculated from the total shear resisting capacity of the shear stud and the total tensile capacity of the slab reinforcement.

Figure 8.10 shows a comparison of the moment-rotation behaviour of joints with different shear connector ratios. It can be noted from the results that the moment...
capacity of the composite CFST column joints increased slightly with increasing degree of shear connectors. This indicates that the moment capacity of joints also depends on the composite action between the steel beam and the slab. When lower degrees of shear connectors (48% to 96%) were modelled for the steel beam, the shear studs yielded before the reinforcement yielded; the shear studs even reached their ultimate strength before the reinforcement yielded. The joints failed due to the failure of shear connectors. When the degree of shear connectors was increased to 120% or above, the slab reinforcement bars gradually reached their yield stress, and reinforcement yielded before the shear connectors; in those cases, the joints failed due to the yielding of reinforcement. It can be concluded that the failure mode of the joint can be controlled by considering the appropriate degree of shear connectors on the steel beam to provide the necessary composite action between the steel beam and slab.

8.4.2 Beam depth
To investigate the effect of the depth of steel beams on composite CFST column joints, five beam depths were considered for the composite joints: 250UB37.3, 310UB40.4, 360UB50.7, 460UB82.1 and 53UB82.0. In all cases, the slab reinforcement ratios as previously were kept constant. Other parameters were considered to be identical to those in the reference joint (specimen SB1-2). Based on the five beam depths, five finite (FE) element models of joints were developed.

![Figure 8.11 Effect of beam depth on the moment-rotation behaviour of composite CFST column joints](image)
The results for moment-rotation behaviour are shown in Figure 8.11, where it is seen that the moment capacity and initial rotational stiffness of the composite joints were increased with increasing depth of the steel beam. Indeed, the ductility of joints was also significantly affected by the beam depth. The results show that the stiffness of the joints was predicted to be least when the lowest beam depth (256 mm) was adopted. In this case, the joint failed by buckling of the beam flange. This was due to the lower compressive capacity of the bottom flange of the beam compared to the sum of the tensile capacities of the slab reinforcement, profile sheet and bolts. The compressive capacity of the bottom flange was higher in beams with deeper sections (e.g., 310UB40.4, 360UB50.7, 460UB 82.1 and 53UB82.0). Increasing the depth from 304 mm to 528 mm also increased the lever arm from the plastic neutral axis to the slab reinforcement level, and increased the moment capacity of the joint accordingly. It is seen that increased beam depth significantly reduced the rotational capacity of the joint. This might also be due to the higher lever arms of the tensile components, as measured from the centre of the bottom flange of the beam to the centre of the each component, since the slab reinforcement allows only a certain amount of deformation and the rotation of the joint was calculated by dividing this deformation by the distance of the slab reinforcement from the centre of the bottom flange. Thus the rotation capacity of the joint was reduced with increasing beam depth. It can be concluded that the beam section is one of the most important parameters and have a significant effect on the moment-rotation behaviour of the composite joints.

8.4.3 Beam material strength
To study the effect of the material strength of the beam, five yield strengths (275, 350, 450, 550 and 690 MPa) were considered in FE modelling to determine their influence on the moment and initial rotational capacity of the composite joints. The beam flange and web were assumed to consist of the same material, and all other parameters were assumed to be identical to those of test specimen SB1-2, with the exception of the total slab reinforcement ratio ($\rho = 1.35\%$). Five FE models were developed to examine how the different yield strengths affected the joints.

The FE analysis results are illustrated in Figure 8.12, in which it is seen that the initial rotational stiffness of the composite joint was not affected either by increasing or
decreasing the yield strength of the steel in the beams. However, the ultimate moment capacity of the joints was significantly increased with increasing yield strength of the beam; it is observed that when the yield strength of beam was 275 MPa, the joints failed due to buckling of the bottom flange of the beam. When the yield strength of beams was increased from 350 MPa to 690 MPa, no buckling occurred in the bottom flange. In these cases, the joint failed due to the failure of the slab reinforcement.

Figure 8.12 Effect of beam material strength on the moment-rotation behaviour of composite CFST column joints

8.5 Effects of CFST Column Parameters

Three parameters related to the CFST column were investigated to determine their effects on the composite CFST column joints: width-to-thickness of the steel tube of CFST column; the steel tube material; and the strength of the infill concrete in the column. The details of these parameters are listed in Table 8.4.

Table 8.4 CFST column parameters

<table>
<thead>
<tr>
<th>Parameter name</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Width-to-thickness ratio of steel tube, $B/t$</td>
<td>30, 40, 60, 80, 100</td>
</tr>
<tr>
<td>2 Steel tube yield strength, $f_y$ (MPa)</td>
<td>275, 350, 450, 690</td>
</tr>
<tr>
<td>3 Column concrete material strength, $f'_{c}$ (MPa)</td>
<td>40, 50, 60, 80, 100</td>
</tr>
</tbody>
</table>

8.5.1 Width-to-thickness ratio of steel tube of CFST column

Figure 8.13 shows a comparison of moment-rotation behaviour of composite CFST column joints for five different width-to-thickness ratios ($B/t=100, 80, 60, 40$ and 30)
of the steel tube with endplate thickness, \( t_p = 10 \) mm. The results show that the effect of width-to-thickness ratio \((B/t)\) was not significant, possibly due to the presence of the slab and consequent composite action between the slab and the beam. When a slab is present, the major tensile forces generated due to bending are carried by the slab reinforcement bars and the profile sheet; however, in the absence of a slab, these tensile forces are carried either by the steel tube or upper bolts or endplate. The initial rotational stiffness and moment capacity of joints without a slab are controlled by the steel tube with significant outward deformation, as detailed in section 7.3.4, Chapter 7. When no slab is incorporated in the joints, the joints failed due to deformation of the steel tube caused by the tensile load acting at the upper bolt level. On the other hand, for composite joints with a slab, the tensile force generated on the steel tube at the upper bolt level is observed to be minimal. As a result, no deformation is observed in the steel tube due to hogging bending moment. From this it is concluded that the width-to-thickness ratio of the steel tube \((B/t)\) has significant influence on the initial rotational stiffness and moment capacity of joints if there is no slab at the joints, but it does not have a significant effect on the composite joints with slab when hogging moment acted on the joints.

![Figure 8.13 Effect of steel tube width-to-thickness ratio on the moment-rotation behaviour of composite CFST column joints](image)

**8.5.2 Steel tube’s yield strength**

The effect of the yield strength of the steel tube of the CFST column on the behaviour of composite column joints was investigated in this section. Five FE models of
composite joints were developed for five steel tube materials. Figure 8.14 shows a comparison of the moment-rotation behaviour for different yield strengths of the steel tube (275, 350, 450, 550 and 690 MPa). The FE model results show that the strength of the steel tube material do not influence the moment capacity. The initial rotation stiffness is not changed either with increased or decreased yield strength of the steel tube. The main reason is the presence of the slab at the joint.

![Figure 8.14](image-url)

Figure 8.14 Effect of steel tube material strength on the moment-rotation curves of the composite CFST column joints

### 8.5.3 Column infill-concrete material strength

In general, both normal-strength concrete (30, 40 and 50 MPa) and high-strength concrete (60, 80 and 100 MPa) can be used to fill the steel tube of CFST columns. It is seen in the literature that the ultimate axial strength of CFST columns is greatly influenced by the strength of the infill-concrete, but don’t have any information regarding the effect of the column concrete strength on the ultimate moment capacity of composite CFST column joints. In this section, the effect of column concrete strengths was investigated by FE modelling of joints, with all other parameters unchanged.

Figure 8.15 shows the comparison of the moment-rotation behaviour for different infill concrete strengths (40, 50, 60, 80 and 100 MPa). It is seen from the results that the moment capacity or the initial rotational stiffness of composite joints was not obviously affected by the column concrete strength, despite the strength of the column concrete being increased from 40 MPa to 100 MPa.
8.5 Effect of Binding Bars

The effect of binding bars on CFST column joints without slab was investigated in Chapter 7. It was seen from those analysis results that binding bars are most effective when they are placed only near the top and bottom of the bolts that are subjected to tensile load. In this section, the effects of binding bars on composite CFST column joints with slab were investigated for different loading conditions, such as applying hogging and sagging moments to the beam. The moment generated due to downward loading is referred to as hogging moment, and the moment generated due to upward loading is referred to as sagging moment. In all cases, the longitudinal slab reinforcement ratio was kept at 1.35%. All other parameters were identical to those of the reference joint (specimen SB1-2), except the number of binding bars and their positions. In this section, the effect of binding bars on the moment-rotation behaviour of composite CFST column joints was investigated when sagging moment acted on the joints.

Figure 8.16 shows the sagging moment-rotation behaviour of composite CFST column joints with and without binding bars. The maximum sagging moment capacity of joints with 0, 2 and 4 binding bars were found to be 76.15, 98.16 and 125.50 kN-m, respectively. It is seen that the sagging moment capacity of the joint without binding bars was lower (76.15 kN-m) than joints with either two or four binding bars. The sagging moment capacity was enhanced by about 22.74% when two binding bars were
placed below the bottom bolts, and by about 64.81% when four binding bars are placed above and below the bottom bolts. These variations were related to outward deformation of the steel tube, and bolt pull-out from the concrete. When steel beams were connected to CFST column without adding binding bars to the column, outward deformation of the steel tube was observed due to pull-out of the bolt from the concrete (Figure 8.17 (a)) near the bottom beam flange. The tensile force at the bottom beam flange is resisted by the steel tube, which has lower resisting capacity than the endplate or blind bolt. However, when binding bars were added near the bottom bolts, then the sagging moment capacity of the joints increased with increasing numbers of binding bars. This was due to the presence of the binding bars at the top and bottom of the bolts subjected to tensile load, because the binding bars act as stiffeners which helped to avoid separation of the tube from the concrete core. Thus the sagging moment capacity of the composite CFST column joints is increased by adding binding bars near the top and bottom tensile bolts.

Figure 8.16 Effect of binding bars on the sagging moment-rotation behaviour of composite CFST column joints

Figure 8.17 shows the deformation modes of composite CFST column joints with binding bars and without binding bars. Figure 8.18 shows the deformation modes of steel tube of the column for the joints with different binding bars arrangements. It is
Figure 8.17 Deformation modes of composite CFST column joints with different numbers of binding bars, when subjected to sagging moment on the joints
clear that the outward deformation of the steel tube is the largest for the joints with no
binding bars, and decreases when binding bars are included in the joints. When two
binding bars are attached below the bottom bolts, the steel tube deformation is reduced
by 37.8%. When four binding bars are attached above and below the lower bolts, the
deflection is reduced significantly by 68.70%. This indicates that the outward
deformation of the steel tube can be reduced significantly by proper utilisation of
binding bars on the steel tube of the CFST column. The binding bars should be placed
only above and below the bolts subjected to a tensile load.

Figure 8.19 shows a comparison of the sagging moment-rotation behaviour of
composite CFST column joints for two different width-to-thickness ratios of the steel
tube ($B/l_c = 30$ and 60), with and without binding bars. The ratio has a significant effect
on the ultimate sagging moment of composite CFST column joints. The ultimate sagging moment capacity of composite CFST joints without binding bars is increased with decreasing width-to-thickness ratio of the steel tube (i.e., by increasing the steel tube thickness), but the initial stiffness of the joints shows no change. However, when binding bars are attached to the steel tube of the CFST column, the connections show significantly increased ultimate sagging moment capacity with decreased width-to-thickness ratio of the steel tube compared to the values when no binding bars are present. This implies that the ultimate sagging moment capacity of composite CFST column joints are enhanced by increasing the steel tube thickness and adding binding bars to the CFST column.

![Figure 8.19 Effect of tube width-to-thickness ratio on the sagging moment-rotation behaviour of composite CFST column joints with and without binding bars](image)

**Figure 8.19** Effect of tube width-to-thickness ratio on the sagging moment-rotation behaviour of composite CFST column joints with and without binding bars

### 8.6 Summary

This chapter contains parametric analysis results of composite steel beam-CFST column joints with slab. The effects of different parameters related to the composite CFST column joints are investigated using FE analysis. The parameters examined are divided into four major groups: connecting parameters (number of bolt rows, endplate thickness); slab parameters (slab thickness, reinforcement ratio, profile sheet material strength and orientation); beam parameters (degree of shear connectors, beam depth, beam material strength) and CFST column parameters (width-to-thickness ratio and material strength of the steel tube, column infill concrete strength). The analyses indicate that slab depth, reinforcement ratio, profile sheet properties, shear connectors,
beam sections and beam materials all have a significant effect on the initial rotational stiffness and ultimate moment capacity of composite CFST column joints. In general, the profile sheet is ignored in the design codes for the calculation of the moment capacity of composite joints; however, the profile sheet is found to be one of the most important parameters, with a significant effect on the ultimate moment capacity of a composite joint. From these results, it is recommended that the profile sheet should be taken into account in the moment capacity calculation for the design of composite joints.

The effect of sagging bending load on the composite joints was investigated for different combinations of binding bars and width-to-thickness ratio of the steel tube of CFST column. The results showed that the outward deformation was noticeable in the steel tube of the composite CFST column joints when sagging bending load acted on the joints, resulting in significant reduction in the ultimate moment capacity of the joints. The addition of binding bars to the steel tube of the CFST column significantly improved the sagging moment capacity of the joints and reduced the steel tube deformation. The analysis results demonstrated that binding bars are most effective when they are placed only near the top and bottom of the lower bolts of the composite CFST column joints when subjected to sagging moment.
CHAPTER 9

MATHEMATICAL MODEL DEVELOPMENT

9.1 Introduction
Rotational stiffness, moment and rotational capacity are widely used to evaluate joint
types, resisting capacity and ductility of a joint. These mechanical properties are also
used to develop the moment-rotation relationship of a joint. The rotational stiffness is a
very important parameter and plays the key role in classifying the joint as rigid, semi-
rigid or pinned. Many mathematical models have been developed to calculate the
rotational stiffness, moment and rotational capacity and moment-rotation relationship of
a joint. Existing mathematical models were dominantly derived using the component
method based on different components in steel beam-to-steel column joints. The
mathematical models were then validated by comparison with experimental test data
and finite element (FE) model results. Models were usually developed for conventional
steel beam-to-steel column joints, where steel beams are connected to steel columns
using normal bolts and endplates. In the current study of the steel beam-to-concrete-
filled steel tubular (CFST) column joints, steel beams are connected to CFST columns
using blind bolts and endplates. Mathematical models derived for steel beam-to-steel
column joints cannot be used indiscriminately to evaluate the behaviour of steel beam-
CFST column joints, particularly if blind bolts (Hollo-Bolts) are used. Therefore the
existing mathematical models must be modified to take into account the use of CFST
columns and Hollo-Bolts.

This chapter presents the development of mathematical models to calculate these
mechanical properties of the steel beam-CFST column composite joints using blind-
bolted endplate connections. The proposed mechanical models are validated by
comparisons with test data for the composite steel beam-to-CFST column joints. The
details are set out step by step in the following.
9.2 Mathematical Model Development

In this study, the main purpose of the mathematical model is to evaluate the behaviour of composite joints in which CFST columns are used instead of steel columns, and where blind bolts are used to connect the steel beam to the steel tube of the CFST column. The stiffness parameters that influence the initial rotational stiffness of blind-bolted composite joints with CFST column are shown in Figure 9.1 and described in Table 9.1. Figure 9.1(a) shows the internal force of the joints including their locations. Figure 9.1(b) shows the individual component stiffness of the joints: slab reinforcement ($k_r$), shear connection ($k_s$), Column flange in bending ($k_{cb}$), endplate in bending ($k_{pb}$), blind-bolts in tension ($k_{bt}$), endplate bearing on steel tube in compression ($k_{cp}$) and concrete core in compression ($k_{cc}$). Figure 9.1(c) shows the main parameters ($k_r, k_s, k_b$ (component stiffness at level of top row of bolts due to tension) and $k_c$ (component stiffness at the level of the bottom flange of the beam due to compression)) used in the mathematical model of the initial rotational stiffness. Other parameters shown in Figure 9.1(a) are beam height ($H_b$), the distance of the reinforcement from the bottom flange of the beam ($D_r$), and the distance of the top row of bolts from the bottom flange of the beam ($D_b$). Based on the rotational stiffness ($S_{j,c}$), moment ($M$) and rotational capacity ($\phi$) of steel beam-CFST column composite joints, the moment-rotation relationship is proposed in section 9.2.4.

![Figure 9.1 Description of the components of the mechanical model of composite CFST column joints](image)
Table 9.1 Description of different parameters used in the mathematical model for steel beam-CFST column composite joints with blind-bolts

<table>
<thead>
<tr>
<th>Main Parameter</th>
<th>Component stiffness</th>
<th>Symbol</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Stiffness of slab reinforcement ($k_r$)</td>
<td>1(a) Reinforcement in the concrete slab</td>
<td>$k_r$</td>
</tr>
<tr>
<td>2. Stiffness of shear connectors ($k_s$)</td>
<td>2(a) Shear connection</td>
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</tr>
<tr>
<td>3. Stiffness of the components at level of top row of bolts ($k_b$)</td>
<td>3(a) Column flange in bending</td>
<td>$k_{cb}$</td>
</tr>
<tr>
<td></td>
<td>3(b) Endplate in bending</td>
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</tr>
<tr>
<td></td>
<td>3(c) Blind-bolts in tension</td>
<td>$k_{bt}$</td>
</tr>
<tr>
<td>4. Stiffness of components at level of bottom beam flange ($k_c$)</td>
<td>4(a) Endplate bearing on steel tube in compression</td>
<td>$k_{cp}$</td>
</tr>
<tr>
<td></td>
<td>4(b) Concrete core in compression</td>
<td>$k_{cc}$</td>
</tr>
</tbody>
</table>

9.2.1 Rotational stiffness calculation models

From a review of the literature, it was seen that the rotational stiffness model proposed by Ahmed and Nethercot (1997) is widely used (Loh et al., 2006b) to calculate or to predict the rotational stiffness of flush endplate connections for steel beam-to-steel column composite joints. However, the effect of shear deformation of the column panel zone was not incorporated in their spring model; indeed, their mathematical expression did not take into account the fixed moment required to keep the rigid bars which connect the spring representing the stud to the spring representing the reinforcement (Al-Aasam, 2013). Recently, Al-Aasam (2013) proposed a mathematical model in which the fixed moment mechanism was incorporated in his model to connect the stud spring to the reinforcement spring. In addition, the effect of shear deformation in the column panel zone was also considered. The spring models proposed by Ahmed and Nethercot (1997) and Al-Aasam (2013) are shown in Figure 9.2(a) and (b), respectively. The simplified equations proposed by them and used to calculate the rotational stiffness of the steel beam-steel column joints are given in Eqs. (9.1) and (9.2).

Rotational stiffness proposed by Ahmed and Nethercot (1997):

\[
S_{jc} = \frac{D_r H_b \left( \frac{1}{k_b} + \frac{1}{k_c} \right) + D_b^2 \left( \frac{1}{k_r} + \frac{1}{k_s} + \frac{1}{k_r} \right) - D_b(H_b + D_r) \frac{1}{k_r} \frac{1}{k_c} \left( \frac{1}{k_b} + \frac{1}{k_c} \right) + \frac{1}{k_c} \frac{1}{k_c}}{\left( \frac{1}{k_r} + \frac{1}{k_s} + \frac{1}{k_c} \right) \left( \frac{1}{k_b} + \frac{1}{k_c} \right) - \frac{1}{k_c} \frac{1}{k_c}} \]  

(9.1)
Rotational stiffness proposed by Al-Aasam (2013):

\[ S_{j,c} = \frac{D_r^2 k_b k_c}{(k_b + k_c)} + \left( \frac{D_r - D_b \left( \frac{k_b}{k_b + k_c} \right)}{\left( \frac{1}{k_r} + \frac{1}{k_s} + \frac{1}{(k_b + k_c)} \right)} \right)^2 \]  \hfill (9.2)

where \( k_r \) is the stiffness of the slab; \( k_s \) is the stiffness of shear connectors; \( k_b \) is the stiffness of the steelwork components at the top bolt row level; and \( k_c \) is the stiffness of column web deformation (\( k_c \)).

Ahmed and Nethercot (1997) and Al-Aasam (2013) derived the above mathematical expressions based on the following compatibility conditions.

Ahmed and Nethercot (1997):

\[ \frac{\Delta_r + \Delta_s + \Delta_c}{H_b} = \frac{\Delta_b + \Delta_c}{D_b} = \phi \]  \hfill (9.3)

Al-Aasam (2013):

\[ \frac{\Delta_r + \Delta_s + \Delta_c}{D_r} = \frac{\Delta_b + \Delta_c}{D_b} = \phi \]  \hfill (9.4)

Here \( \Delta_r \), \( \Delta_s \) and \( \Delta_c \) are the deformations of the related components such as slab reinforcement, shear connectors and column web deformation respectively; \( \phi \) is the rotation of the connections.

It is seen from Eq. (9.3) used by Ahmed and Nethercot (1997) that the deformations (\( \Delta_r, \Delta_s \) and \( \Delta_c \)) are divided by the beam height although the deformation level of the slab reinforcement is above the beam. For Al-Aasam (2013) as shown in Eq. (9.4), the deformations (\( \Delta_r, \Delta_s \) and \( \Delta_c \)) is divided by the distance (\( D_r \)) of the reinforcement from the bottom flange of the beam although the deformation of the shear connectors is observed at the beam level. In the present work, the mathematical model proposed by Ahmed and Nethercot (1997) has been modified to predict accurately the rotational stiffness of steel beam-CFST column joints, in which \( \Delta_r \) is divided by \( D_r \) and \( \Delta_s \) is divided by \( H_b \).
Figure 9.2 Major spring models available in the literature for flush endplate steel beam-to-steel column composite joints

(a) Proposed model for composite steel beam-CFST column joints

A spring model is proposed for calculating the rotational stiffness of flush endplate connections in composite steel beam-to-CFST column joints, shown in Figure 9.3.
Figure 9.3 Proposed spring model to calculate the rotational stiffness of flush endplate steel beam-to-CFST column composite joints

When the connection subjected to loading in the initial stage, the sum of the tensile forces in the reinforcement ($F_r$) and top row of bolts ($F_b$) is equal to the compression force transferred by the beam bottom flange ($F_c$). The value of $F_r$ is equal to the shear force transferred by the shear studs ($F_s$). At equilibrium, the following expressions may be written:

$$F_r + F_b = F_c \quad (9.5)$$

$$F_r = F_s \quad (9.6)$$

From the force-deformation relationship, the stiffness is defined as the ratio of force to deformation. For each component of a connection, the following expressions can be written:

$$k_r = \frac{F_r}{\Delta_r}$$

$$k_s = \frac{F_s}{\Delta_r}$$

$$k_b = \frac{F_b}{\Delta_b}$$

$$k_c = \frac{F_c}{\Delta_c}$$
From Figure 9.3 and compatibility conditions, the following expression can be written:

\[
\frac{\Delta r + \Delta c}{D_r} + \frac{\Delta s}{H_b} = \frac{\Delta b + \Delta c}{D_b} = \phi \tag{9.7}
\]

From Eq. (9.7), the following expression can be rewritten:

\[
(\Delta r + \Delta c)H_b + \Delta sD_r = H_bD_r \phi \\
\frac{F_r + F_c}{k_r} + \frac{F_s}{k_s}D_r = H_bD_r \phi \\
\frac{F_r + F_r + F_b}{k_c}H_b + \frac{F_r}{k_s}D_r = H_bD_r \phi \\
F_r \left(\frac{1}{k_r} + \frac{1}{k_c}\right)H_b + F_r \frac{1}{k_s}D_r + F_b \frac{1}{k_c}H_b = H_bD_r \phi
\]

Therefore,

\[
F_r \left(\frac{1}{k_r} + \frac{1}{k_c}\right)H_b + \frac{1}{k_s}D_r \right) + F_b \frac{1}{k_c}H_b - H_bD_r \phi = 0 \tag{9.8}
\]

Again, from Eq. (9.7), the following expression can be rewritten:

\[
\Delta b + \Delta c = D_b \phi \\
F_b + \frac{F_c}{k_c} = D_b \phi \\
F_b + \frac{F_r + F_b}{k_c} = D_b \phi
\]

Therefore,

\[
F_r \frac{1}{k_c} + F_b \left(\frac{1}{k_b} + \frac{1}{k_c}\right) - D_b \phi = 0 \tag{9.9}
\]

By solving for \( F_r \) and \( F_b \) from Eqs. (9.8) and (9.9),

\[
F_r = \frac{\left(\frac{1}{k_b} + \frac{1}{k_c}\right)D_r H_b - \frac{1}{k_c}D_b H_b}{\left(\frac{1}{k_r} + \frac{1}{k_c}\right)\left(\frac{1}{k_b} + \frac{1}{k_c}\right)H_b + \frac{1}{k_s}\left(\frac{1}{k_b} + \frac{1}{k_c}\right)D_r - \frac{1}{k_c}H_b - \phi} \tag{9.10}
\]

\[
F_b = \frac{D_b H_b \left(\frac{1}{k_r} + \frac{1}{k_c}\right) + \frac{1}{k_s}D_b D_r - \frac{1}{k_c}D_r H_b}{\left(\frac{1}{k_r} + \frac{1}{k_c}\right)\left(\frac{1}{k_b} + \frac{1}{k_c}\right)H_b + \frac{1}{k_s}\left(\frac{1}{k_b} + \frac{1}{k_c}\right)D_r - \frac{1}{k_c}H_b - \phi} \tag{9.11}
\]

At the initial stage of loading, the internal tensile forces are small, and no compression forces develop in the beam web (Ahmed & Nethercot, 1997), which are resisted only by
the bottom flange of the beam. Therefore the joint moment at the initial loading stage may be determined from the centre of the bottom flange of the beam, using the tensile forces in the reinforcement \((F_r)\) and top row of bolts \((F_b)\). Taking the moment from the centre of the beam bottom flange,

\[ M = F_r D_r + F_b D_b \]  

(9.12)

Substituting \(F_r\) and \(F_b\) from Eqs. (9.10) and (9.11) into Eq. (9.12) gives

\[ M = \frac{1}{k_r} D_b^2 H_b + \frac{1}{k_s} D_b^2 D_r + \frac{1}{k_b} D_r^2 H_b + \frac{1}{k_c} H_b (D_r - D_b)^2 \left( \frac{1}{k_r} + \frac{1}{k_c} \right) \left( \frac{1}{k_b} + \frac{1}{k_c} \right) D_r - \frac{1}{k_c^2} H_b \phi \]  

(9.13)

From the moment-rotation relationship, the stiffness is defined as the ratio of moment to rotation \((S_{j,c} = \frac{M}{\phi})\). Therefore, from Eq. (9.11), the joint stiffness can be written:

\[ S_{j,c} = \frac{\frac{1}{k_r} D_b^2 H'_b + \frac{1}{k_s} D_b^2 D_r + \frac{1}{k_b} D_r^2 H_b + \frac{1}{k_c} H'_b (D_r - D_b)^2 \left( \frac{1}{k_r} + \frac{1}{k_c} \right) \left( \frac{1}{k_b} + \frac{1}{k_c} \right) D_r - \frac{1}{k_c^2} H'_b}{\left( \frac{1}{k_r} + \frac{1}{k_c} \right) \left( \frac{1}{k_b} + \frac{1}{k_c} \right) H'_b + \frac{1}{k_s} \left( \frac{1}{k_b} + \frac{1}{k_c} \right) D_r - \frac{1}{k_c^2} H'_b} \]  

(9.14)

The rotation stiffness of the steel beam-CFST column composite joint may be determined using Eq. (9.12), which mainly depends on the stiffness of the slab reinforcement \((k_r)\) and shear connectors \((k_s)\), the stiffness of the components at the level of the top row of bolts \((k_b)\) due to tension, and the stiffness of the components at the level of the bottom flange of the beam \((k_c)\) due to the compression. The calculation methods of the stiffness values of \(k_r, k_s, k_b,\) & \(k_c\) are illustrated step by step in the following.

(b) Determination of component stiffness

It is seen from Figure 9.1(b) for composite stiffness steel beam-to-steel CFST column joints that the stiffness values of the seven components listed in Table 9.1, need to be calculated to obtain the value of \(k_r, k_s, k_b,\) & \(k_c\). The stiffness value of reinforcement in slab \((k_r)\) and shear connector \((k_s)\) can be calculated from the expressions that was proposed for the flush endplate in steel beam-to-steel column composite joints. Since the stiffness of \(k_b\) and \(k_c\) depend on the CFST column, endplate and blind bolts, the values of \(k_b\) and \(k_c\) need to be calculated from expressions derived from the steel beam-CFST column
composite joint. Specifically, the stiffness of $k_b$ depends on the stiffness of the steel tube wall of the CFST column in bending ($k_{cb}$), the endplate in bending ($k_{pb}$), and the bolts in tension ($k_{bt}$); whilst the stiffness of $k_c$ depends on the stiffness of the endplate bearing on the steel tube in compression ($k_{cp}$) and the concrete core in compression ($k_{cc}$).

(1) Stiffness of slab reinforcement ($k_r$)
The stiffness of the slab reinforcement mainly depends on the total number of reinforcement bars in the effective slab cross-sectional area, taking account of the effective length of the reinforcement bars. The stiffness of the slab reinforcements ($k_r$) is obtained from:

$$k_r = \frac{E_s A_r}{l_r} \quad (9.15)$$

where $A_r$ is the total area of the slab reinforcement bars and $E_s$ is the elastic modulus of the reinforcement bar and $l_r$ is the effective length of the reinforcement bar. There are many expressions for calculating the value of $l_r$. In this study, the expression proposed in Eurocode 4 (2005) is adopted, in which the depth of the column is taken into consideration. The value of $l_r$ can be determined from the following expression:

$$l_r = \begin{cases} \frac{h_c}{2} & \text{for internal joints} \\ 3.6 h_c & \text{for exterior joints} \end{cases} \quad , \text{where} \ h_c \text{is the depth of the CFST column.}$$

(2) Stiffness of shear connectors ($k_s$)
The stiffness of shear connectors ($k_s$) is determined from:

$$k_s = \frac{F_{sc}}{\Delta_{sc}} \quad (9.16)$$

where $F_{sc}$ is the equivalent shear force resisted by the total number of shear connectors ($N_{sc}$), and $\Delta_{sc}$ is the slip of the shear connectors. The values of $F_{sc}$ and $\Delta_{sc}$ can be calculated from the following expression proposed by Al-Aasam (2013) for partial shear connectors and full shear connections:

$$F_{sc} = \begin{cases} 0.5 N_{sc} F_{sc,\text{max}} & \text{when} \ \eta_y < 1 \ \text{for partial shear connectors} \\ \frac{0.5}{\eta_y} N_{sc} F_{sc,\text{max}} & \text{when} \ \eta_y \geq 1 \ \text{for full shear connectors} \end{cases} \quad (9.17)$$
$$\Delta_{sc} = \begin{cases} 
\frac{\ln \left[ 1 - (0.5)^{\frac{1}{\alpha}} \right]}{-\lambda} & \text{when } \eta_y < 1 \\
\frac{\ln \left[ 1 - \left( \frac{0.5}{\eta_y} \right)^{\frac{1}{\alpha}} \right]}{-\lambda} & \text{when } \eta_y \geq 1 
\end{cases}$$

(9.18)

where $\alpha$ is a non-dimensional parameter taken as 0.8, $\lambda = 0.7$ in this study, $N_{sc}$ is the total number of shear connectors and $\eta_y$ is the degree of the shear connectors to be calculated from:

$$\eta_y = \frac{N_{sc} F_{sc,\text{max}}}{A_r f_{y,r}}$$

(9.19)

where $A_r$ is the total area of the slab reinforcement and $f_{y,r}$ is the yield strength of the slab reinforcement.

Therefore, the stiffness of shear connectors ($k_s$) can be calculated from the following expression proposed by Al-Aasam (2013):

$$k_s = \begin{cases} 
-\frac{0.5\lambda N_{sc} F_{sc,\text{max}}}{\ln \left[ 1 - (0.5)^{\frac{1}{\alpha}} \right]} & \text{for } \eta \leq 1 \\
-\frac{0.5\lambda N_{sc} F_{sc,\text{max}}}{\eta \ln \left[ 1 - \left( \frac{0.5}{\eta} \right)^{\frac{1}{\alpha}} \right]} & \text{for } \eta > 1 
\end{cases}$$

(9.20)

The maximum shear strength of the shear connection ($F_{sc,\text{max}}$) is determined from the formula provided by Eurocode 4 (2005):

$$F_{sc,\text{max}} = \min \left\{ 0.8 f_u \frac{\pi d^2}{4}, 0.37 \alpha \sqrt{f' c E_c} \frac{\pi d^2}{4} \right\}$$

where $\alpha = \begin{cases} 
0.2 \left( \frac{h_{sc}}{d} + 1 \right) & \text{for } \frac{h_{sc}}{d} \leq 4 \\
1 & \text{for } \frac{h_{sc}}{d} \geq 4 
\end{cases}$

where $h_{sc}$ is the height of the shear connector; $d$ is the diameter of the shear connector; $f' c$ is the characteristic cylinder compressive strength of the concrete; $E_c$ is the elastic modulus of the concrete; and $f_u$ is the ultimate tensile strength of the shear connector.
In most cases, the height-to-diameter ratio \( \frac{h_{sc}}{d} \) of the shear studs used in composite buildings is greater than 4, so the reduction factor \( (\alpha) \) is equal to unity. The above equation can be rewritten:

\[
F_{sc,\text{max}} = \text{Min} \left\{ 0.8 \, f_u \left( \frac{\pi d^2}{4} \right), \quad 0.37 \sqrt{f'_c E_c} \left( \frac{\pi d^2}{4} \right) \right\}
\]  

(9.21)

(3) **Stiffness of the components at the level of the top bolt row (\( k_b \)) due to tension**

The stiffness value \( (k_b) \) of the components at the level of the top row of bolts can be calculated in accordance with Eurocode 3 (2005) as given in Eq. (9.22). The stiffness of \( k_b \) depends on the stiffness of the column face in bending \( (k_{cb}) \), the endplate in bending \( (k_{ep}) \), and the blind bolts in tension \( (k_{bt}) \). The value of \( k_{cb}, k_{ep}, k_{bt} \) should be calculated based on the CFST column and blind bolts rather than a steel column and normal bolts:

\[
k_b = \frac{1}{\frac{1}{k_{cb}} + \frac{1}{k_{ep}} + \frac{1}{k_{bt}}} \tag{9.22}
\]

The calculation details for the stiffness value of \( k_{cb}, k_{ep} \) & \( k_{bt} \) are illustrated step by step in the following.

(i) **Stiffness of column face in bending (\( k_{cb} \))**

The stiffness of the column face in bending is calculated based on the plate approach due to the restraint of column edges in the width side of the column face and continuous face along the column height. The stiffness value of the column face in bending proposed by da Silva et al. (2004) is:

\[
k_{cb} = \frac{16E t_{tb}^3}{(B - t_{tb})^2} \alpha + (1 - \beta) \tan \theta + \frac{10.4(1.5 - 1.6\beta)}{\mu^2} \tag{9.23}
\]

where \( \mu \) is the steel tube face slenderness ratio; \( B \) is the width of the steel tube of the CFST column; and \( t_{tb} \) is the thickness of the steel tube of CFST column, as shown in Figure 9.4. The geometry-related parameters \( \alpha, \beta \) and \( \mu \) were later defined by Elamin (2013) as below:

\[
\alpha = \frac{c}{B - t_{tb}}; \quad \beta = \frac{g+c}{B - t_{tb}}; \quad \mu = \frac{B - t_{tb}}{t_{tb}}; \quad \theta = 35 - 10 \frac{g+c}{B - t_{tb}}; \quad c = k_{is} \times d_h
\]
where $g$ is the bolt gauge (horizontal); $d_h$ is the bolt hole diameter; and $k_{is}$ is the initial stiffness calibration factor. For Hollo-Bolts, the value of $c$ depends on the value of $k_{is}$ and $d_h$, and for other blind bolts it depends on the average bolt nut diameter ($d_{bn}$).

The load transfer mechanism of Hollo-Bolts is quite different from that of other blind bolts. For Hollo-Bolts, the load is transferred by the opened sleeves on the face of the steel tube of CFST column through the concrete core, whereas for other blind bolts, the load is transferred through the bolt nut in contact with the face of the steel tube of the CFST column. The following equation is used to calculate the value of $k_{is}$. Elamin (2013) provided three individual equations with three different charts for three parameters included ($\mu$, $\beta$ and $f'_c$), which modified here into one equation as below.

$$k_{is} = 1.75656 + 0.0046268 \mu_1 - 1.0416 \beta_1 - 0.000060718f'_c^2$$

$$+ 0.0083156f'_c$$ (9.24)

where $\beta_1$ is the ratio of bolt gauge to hollow section width ($\frac{g}{B}$); and $\mu_1$ is the hollow section face slenderness ratio ($\frac{B}{t_b}$).

The proposed equation was validated against test data and model data reported by Elamin (2013). It is seen in Figure 9.5 that the model predicts very well against test data up to the elastic stage.

![Figure 9.4 Mechanism of load transfer from Hollo-Bolts to the steel tube of a CFST column (Elamin, 2013)](image-url)
**ii) Stiffness of endplate in bending \( (k_{pb}) \)**

In flush endplate connections, the stiffness of the endplate in bending \( (k_{cb}) \) can be calculated from the basic principles of cantilever beam bending theory:

\[
k_{pb} = \frac{P}{\delta_p}
\]

\[
k_{pb} = \frac{6EI_p}{m^3}
\]

(From cantilever beam theory, the maximum deflection is \( \delta_p = \frac{Pm^3}{3EI_p} \), where \( P = \frac{F}{2} \).

where \( E \) is the elastic modulus of the endplate; \( I_p \) is the moment of the inertia of the endplate; and \( m \) is the distance from one bolt line to the centre of the beam flange, shown in Figure 9.6.)
The value of $I_p$ can be calculated from the following formulas: $I_p = \frac{b_p t_p^3}{12}$ for flat endplate; $I_p = \frac{(R^4 - r^4)}{4} \left( \frac{\pi \alpha}{180} + \sin \alpha \cos \alpha \right) - \frac{80 \sin^2 \alpha (R^3 - r^3)^2}{\pi \alpha (r^2 - r)}$ for curved endplate (reported in Yao et al. (2006)), where $b_p$ and $t_p$ are the width and thickness of the flat endplate and $R$ & $r$ are the outer and inner diameters of the curved endplate measured from the centre of the circular column.

Figure 9.6 Configuration of curved endplate, flat endplate used for the calculation of endplate stiffness

(iii) Stiffness of Hollo-Bolts in tension ($k_{bt}$)

The stiffness for Hollo-Bolts in tension is different from that of other blind bolts or normal bolts. Section 6.2 of Chapter 6 states that the stiffness of Hollo-Bolts is higher when connected to a CFST column than when it is connected to a square hollow column. When Hollo-Bolts are connected to CFST column, their stiffness depends on the stiffness of both the bolt shank and the sleeve bearing on the infill concrete in the CFST column, because Hollo-Bolts are embedded into the concrete core of the CFST column. The stiffness ($k_{bt}$) of Hollo-Bolt when used to connect a steel beam to CFST column using an endplate connection is calculated based on the following proposed equation (9.27):

$$k_{bt} = \frac{1}{\frac{1}{k_{bsh}} + \frac{1}{k_{cst}}}$$  \hspace{1cm} (9.27)

where $k_{bsh}$ is the stiffness of the bolt shank, and $k_{cst}$ is the stiffness of the sleeve bearing on the concrete.
The values of \( k_{bsh} \) and \( k_{cst} \) are calculated from Eqs. (9.28) and (9.29) respectively.

(iii) Stiffness of bolt shank in tension \( (k_{bsh}) \)

The value of \( k_{bsh} \) can be determined from the bolt shank elongation in the direction of bolt tensile load.

\[
k_{bsh} = \frac{E_s A_s}{L_b}
\]

(9.28)

where \( E_s \) is the elastic modulus of the bolt shank; \( A_s \) is the tensile stress area of the bolt shank; and \( L_b \) is the effective length of the Hollo-Bolt calculated from the following equation proposed by Pitrakkos (2012), shown in Figure 9.7.

\[
L_b = t_p + t_{tb} + t_w + \frac{t_{bh} + t_c}{2}
\]

where \( t_{bh} \) is the thickness of the hexagonal bolt head; \( t_w \) is the thickness of the collar of the Hollo-Bolt; \( t_c \) is the depth of the cone of the Hollo-Bolt; \( t_p \) is the thickness of the endplate; and \( t_{tb} \) is the thickness of the steel tube.

![Figure 9.7 Effective length \( (L_b) \) of the Hollo-Bolt (Pitrakkos, 2012)](image)

(iiib) Stiffness of sleeve bearing on the concrete in compression \( (k_{slb}) \)

The value of \( k_{slb} \) can be calculated from the stiffness of the sleeve bearing on the concrete in tension. The following equation is proposed to calculate the value of \( k_{slb} \).

\[
k_{cst} = \frac{E_c A_{slb}}{L_{slb}}
\]

(9.29)
where $E_c$ is the elastic modulus of the concrete; $A_{sib}$ is the bearing area of the sleeve; and $L_{sib}$ is the effective length of the sleeve bearing.

Here $A_{sib} = \pi (R - r) \sqrt{R^2 + (R - r)^2}$; $r = \frac{d_h}{2}$; $R = \frac{d_{tcm}}{2} + t_s$; $L_{sib} = \frac{t_c}{\cos \alpha}$

where $d_h$ is the bolt hole diameter; $t_s$ is the thickness of sleeve; $d_{tcm}$ is the maximum diameters of the upper threaded cone; and $\alpha$ is the slope angle of the cone (15°).

The stiffness ($k_{bt}$) of Hollo-Bolts was calculated from Eqs. (9.25)-(9.27) and compared with test data reported by Pitrakkos (2012). It is seen from Figure 9.8 that the model agrees very well with the test data.

![Comparison between test data (Pitrakkos, 2012) and calculated tensile stiffness of Hollo-Bolts embedded in concrete](image-url)
(4) **Stiffness of components at level of bottom beam flange (k_c)** in compression

The stiffness value of k_c depends on the stiffness of the endplate bearing on the steel tube in compression (k_cp), and the concrete core in compression (k_cc). The stiffness of k_c can be determined from Eq. (9.30). This equation is proposed in this study according to Eurocode 3 (2005). The values of k_cp & k_cc are calculated based on the area of an equivalent T-stub in compression (as shown in Figure 9.9) on the endplate and transferred to the core concrete through the steel tube of the CFST column. The calculation details are illustrated step by step in the following.

\[
k_c = \frac{1}{k_{cp}} + \frac{1}{k_{cc}}
\]

(9.30)

Figure 9.9 Area of equivalent T-stub in compression (Eurocode 3, 2005)

(i) **Stiffness of Endplate Bearing on Steel Tube in Compression (k_{cp})**

The stiffness value (k_{cp}) of the endplate bearing on the steel tube in compression is determined from:

\[
k_{cp} = \begin{cases} 
\frac{E_p b_{eff} l_{eff}}{(t_p + t_{tb})} & \text{for flat endplate} \\
\frac{\pi E_p a b_{eff} r}{180(t_p + t_{tb})} & \text{for curved endplate (Yao et al., 2006)}
\end{cases}
\]

where \(E_p\) is the elastic modulus of the endplate, \(t_p\) & \(t_{tb}\) are the thicknesses of the endplate and steel tube, respectively; \(r\) is the radius of the internal surface of the curved
endplate; \(b_{eff}\) and \(l_{eff}\) are width and length of the area of the equivalent T-stub in compression; and \(\alpha\) is the angle of the curved endplate from side edge to its centerline as shown in Figure 9.6.

The values of \(b_{eff}\) and \(l_{eff}\) can be determined from the area of equivalent T-stub, defined by Eurocode 3 (2005), subjected to compression force as shown in Figure 9.9.

\[
b_{eff} = \begin{cases} 
  t_{bf} + 2t_w & \text{for short projection} \\
  t_{bf} + 2t_p & \text{for large projection}
\end{cases}
\]

\[
l_{eff} = \begin{cases} 
  b_{bf} + 2t_w & \text{for short projection} \\
  b_{bf} + 2t_p & \text{for large projection}
\end{cases}
\]

where \(b_{bf}\) & \(t_{bf}\) are the width and thickness of the beam flange; \(t_p\) is the endplate thickness; and \(t_w\) is the throat thickness of the weld.

(ii) Stiffness of Concrete in Compression \((k_{cc})\)

The stiffness \((k_{cc})\) of the endplate bearing on the steel tube in compression is determined the following proposed expression:

\[
k_{cc} = \begin{cases} 
  E_c(B - 2t_{tb}) & \text{for flat endplate} \\
  2sin\alpha(D - 2t_{tb})E_c & \text{for curved endplate}
\end{cases} \quad (9.32)
\]

where \(B\) is the width of the square steel tube; \(D\) is the diameter of the circular steel tube; \(t_{tb}\) is the thickness of the steel tube; \(E_c\) is the elastic modulus of the core concrete of the CFST column; and \(\alpha\) is the angle as shown in Figure 9.6.

The formulas for the stiffness of all the components of a flush endplate steel beam-to-CFST column composite joint are summarised in Table 9.2.
Table 9.2 Formulas for the component stiffness of flush endplate steel beam-to-CFST column composite joints

<table>
<thead>
<tr>
<th>Component Stiffness</th>
<th>Formula</th>
<th>References</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Reinforcement in the concrete slab</strong></td>
<td>[ k_r = \frac{E_s A_r}{l_r} ] for ( l_r = \frac{h_c}{2} )</td>
<td>Eurocode 4 (2005)</td>
</tr>
<tr>
<td><strong>Shear connectors</strong></td>
<td>[ k_s = \begin{cases} 0.5N_{sc} k_{sc, max} &amp; \text{for } \eta \leq 1 \ \frac{0.5N_{sc} k_{sc, max}}{\eta} \left( 1 - \left( \frac{0.5}{\eta} \right)^{\frac{3}{2}} \right) \right) &amp; \text{for } \eta &gt; 1 \end{cases} ]</td>
<td>Al-Aasam (2013)</td>
</tr>
<tr>
<td><strong>Column face in bending</strong></td>
<td>[ k_{cb} = \frac{16E_{tp}^3}{(B - t_{tb})^2} \left( \alpha + (1 - \beta) \tan \theta \right) \left( 1 - \beta \right)^3 + \frac{104(1.5 - 1.6\beta)}{\mu^2} ]</td>
<td>da Silva et al. (2004), Elamin (2013)</td>
</tr>
<tr>
<td><strong>Endplate in bending</strong></td>
<td>[ k_{pb} = \frac{6EI_p}{m^3} ]</td>
<td>Proposed</td>
</tr>
<tr>
<td><strong>Top row bolts in tension</strong></td>
<td>[ k_{bt} = \frac{1}{k_{bsb} + k_{cst}} ] for flat endplate [ k_{bst} = \frac{E_s A_s}{l_s} ] for curved endplate [ k_{cst} = \frac{E_c A_{cst}}{l_{cst}} ]</td>
<td>Proposed</td>
</tr>
<tr>
<td><strong>Endplate bearing on steel tube in compression</strong></td>
<td>[ k_{cp} = \begin{cases} \frac{E_p b_{eff} l_{eff}}{(t_p + t_{tb})} &amp; \text{for flat endplate} \ \frac{\pi E_p a b_{eff} \gamma}{180(t_p + t_{tb})} \right) &amp; \text{for curved endplate} (Yao et al., 2006) \end{cases} ]</td>
<td>Proposed</td>
</tr>
<tr>
<td><strong>Concrete core in compression</strong></td>
<td>[ k_{cc} = \begin{cases} \frac{E_c (B - 2t_{tb})}{2sina(D - 2t_{tb})E_c} &amp; \text{for flat endplate} \end{cases} ]</td>
<td>Proposed</td>
</tr>
</tbody>
</table>

(c) Validation of the proposed model

The validity of the proposed rotational stiffness model in Eq. (9.14) was tested by comparing the results with test data collected from steel beam-steel column composite joints and steel beam-CFST column composite joints. The proposed model in Eq. (9.12) was also compared with other models as summarised in Eq. (9.1) from Ahmed and Nethercot (1997) and in Eq. (9.2) from Al-Aasam (2013). The comparisons between test data and calculated results of rotational stiffness of both types of composite joints are illustrated separately below. For CFST column joints, the stiffness of each component of the steel beam-CFST column composite joints was calculated from the expressions illustrated in the previous section. The stiffness values of \( k_r, k_s, k_b, \& k_c \) were calculated using the formulas in Table 9.2 for CFST column joints. It can be seen
from the comparison between test and model results that the proposed rotational stiffness model predicts very well, both for steel beam–CFST column composite joints and for steel beam-steel column composite joints.

(1) Validation for steel beam-steel column joints

Twenty test specimens from studies by Xiao et al. (1994), Li et al. (1996), Liew et al. (2000), and Fu and Lam (2006) were considered for validating the proposed model. Table 9.3 shows the stiffness values of $k_r$, $k_s$, $k_b$, & $k_c$ calculated by Al-Aasam (2013) for steel beam-steel column joints. These values were considered in calculating the rotational stiffness of steel beam-steel column joints using the current proposed model as well as using the models proposed by Ahmed and Nethercot (1997) and Al-Aasam (2013). The calculated value of the initial rotational stiffness value of the steel beam-steel column joints is given in Table 9.4.

Table 9.3 Component stiffness values calculated by Al-Aasam (2013) for steel beam-steel column joints

<table>
<thead>
<tr>
<th>Reference</th>
<th>Specimen</th>
<th>$H_b$ (mm)</th>
<th>$D_c$ (mm)</th>
<th>$D_r$</th>
<th>$D_b$</th>
<th>$k_r$ (kN/mm)</th>
<th>$k_s$ (kN/mm)</th>
<th>$k_b$ (kN/mm)</th>
<th>$k_c$ (kN/mm)</th>
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<tr>
<td>Xiao et al.</td>
<td>SCJ3</td>
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<td>120</td>
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<td>254</td>
<td>238</td>
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<td>155</td>
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<td>(1994)</td>
<td>SCJ4</td>
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<td>120</td>
<td>398</td>
<td>254</td>
<td>1201</td>
<td>662</td>
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<td>639</td>
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<tr>
<td>Li et al.</td>
<td>SCJ1</td>
<td>257</td>
<td>110</td>
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<td>Liew et al.</td>
<td>SCCB1</td>
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<td>Fu and Lam</td>
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<td>187</td>
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Table 9.4 Comparison of calculated rotational stiffness values for steel beam-steel column joints

<table>
<thead>
<tr>
<th>Reference</th>
<th>Specimen</th>
<th>Rotational stiffness of steel beam-steel column joints (kN·m/mrad)</th>
</tr>
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<tbody>
<tr>
<td>Xiao et al. (1994)</td>
<td>SCJ3</td>
<td>25.9</td>
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<td>Li et al. (1996)</td>
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<td>31.3</td>
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<td></td>
<td>SCJ3</td>
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<td>SCJ6</td>
<td>18.7</td>
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<td>Liew et al. (2000)</td>
<td>SCCB1</td>
<td>36.3</td>
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<td>SCCB2</td>
<td>53.9</td>
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<td></td>
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<td>66.2</td>
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<td>Fu and Lam (2006)</td>
<td>CJ1</td>
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<td>38.5</td>
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</table>

The comparison between test and model data for the rotational stiffness of the steel beam-steel column composite joints is illustrated in Figure 9.10. It can be seen from the figure that, in most cases, the rotational stiffness calculated from the model proposed by Ahmed and Nethercot (1997) is lower than the rotational stiffness determined by test, and also lower than values calculated using the model of Al-Aasam (2013). It is also observed that the rotational stiffness values calculated from the expression proposed by Al-Aasam (2013) agree with test data in some cases, but in other cases the calculated stiffness is lower than the stiffness test data. However, it can be seen from all the figures that the rotational stiffness calculated from the proposed model is found to be very much closer to the test stiffness. In some cases, the stiffness of the proposed model is found to be similar to the Al-Aasam (2013) model. This indicates that the proposed rotational stiffness model can be used to predict the rotational stiffness of steel beam-steel column composite joints.
Figure 9.10 Comparison between test and model results for rotational stiffness of steel beam-steel column composite joints

(a) Test specimen SCJ3 (Xia et al., 1994)

(b) Test specimen SCJ4 (Xia et al., 1994)

(c) Test specimen SCJ5 (Xia et al., 1994)

(d) Test specimen SCJ6 (Xia et al., 1994)

(e) Test specimen SCJ7 (Xia et al., 1994)

(f) Test specimen SCJ1 (Li et al., 1996)
Figure 9.10 Comparison between test and model results for rotational stiffness of steel beam-steel column composite joints (continued)
Figure 9.10 Comparison between test and model results for rotational stiffness of steel beam-steel column composite joints (continued)
(2) Validation for steel beam–CFST column joints

Ten sets of test data, four from the present study and six sets collected from reports in the literature were examined to validate the predicted rotational stiffness of the steel beam-CFST column composite joints. The stiffness of each component of the steel beam-CFST column composite joints was calculated from the expressions illustrated in the previous section. The stiffness values of $k_r$, $k_s$, $k_{br}$, and $k_c$ were calculated using the formulas in Table 9.2. These values, which are listed in Table 9.5, were used to determine the rotational stiffness of steel beam-CFST column composite joints.

The comparison between test and model data for the rotational stiffness of the steel beam-CFST column composite joints is illustrated in Figure 9.11. It can be seen from the figures that, in some cases, the rotational stiffness calculated using the expression Eq. (9.1) proposed by Ahmed and Nethercot (1997) is lower than the rotational stiffness determined in the tests and also lower than the value obtained from the model of Al-Aasam (2013). It is also seen that when the rotational stiffness of joints is calculated using Eq. (9.2) proposed by Al-Aasam (2013) and the present model proposed in Eq. (9.12), the rotational stiffness of joints is almost identical to the test results. However, the stiffness values calculated using the model proposed by Al-Aasam (2013) is observed to be higher (see in Table 9.6) than the values calculated using the proposed model. This indicates that the proposed model is safer than the model proposed by Al-Aasam (2013), as the stiffness values are lower than those calculated from the model.
proposed by Al-Aasam (2013), and lie between those of Ahmed and Nethercot (1997) and Al-Aasam (2013). From this it is concluded that the proposed rotational stiffness model can safely be used to predict the rotational stiffness of steel beam-CFST column composite joints.

Table 9.5 Details of main parameters used in the calculation of the rotational stiffness of steel beam-CFST column joints

<table>
<thead>
<tr>
<th>Reference</th>
<th>Specimen</th>
<th>(H_b) (mm)</th>
<th>(D_c) (mm)</th>
<th>(D_r) (mm)</th>
<th>(D_b) (mm)</th>
<th>(k_r) (kN/mm)</th>
<th>(k_s) (kN/mm)</th>
<th>(k_b) (kN/mm)</th>
<th>(k_c) (kN/mm)</th>
</tr>
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<tbody>
<tr>
<td>Loh et al. (2006)</td>
<td>CJ1</td>
<td>248</td>
<td>120</td>
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<td>188</td>
<td>1649</td>
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<td>Mirza and Uy (2011)</td>
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<td>689</td>
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<td>2413</td>
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<td>SB1-1</td>
<td>304</td>
<td>120</td>
<td>398</td>
<td>234</td>
<td>504</td>
<td>896</td>
<td>14.9</td>
<td>2742</td>
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<td>SB1-2</td>
<td>304</td>
<td>120</td>
<td>398</td>
<td>234</td>
<td>504</td>
<td>896</td>
<td>14.9</td>
<td>2742</td>
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<td>CB2-1</td>
<td>304</td>
<td>120</td>
<td>398</td>
<td>234</td>
<td>504</td>
<td>896</td>
<td>15.28</td>
<td>3680</td>
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<td>120</td>
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<td>234</td>
<td>504</td>
<td>896</td>
<td>15.28</td>
<td>3680</td>
</tr>
</tbody>
</table>

Table 9.6 Comparison of calculated rotational stiffness values for steel beam-CFST column joints

<table>
<thead>
<tr>
<th>Reference</th>
<th>Specimen</th>
<th>Rotational stiffness of CFST column joints (kN.m/mrad)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loh et al. (2006)</td>
<td>CJ1</td>
<td>23.9</td>
</tr>
<tr>
<td></td>
<td>CJ2</td>
<td>17.2</td>
</tr>
<tr>
<td></td>
<td>CJ3</td>
<td>13.5</td>
</tr>
<tr>
<td></td>
<td>CJ4</td>
<td>17.3</td>
</tr>
<tr>
<td></td>
<td>CJ5</td>
<td>33.7</td>
</tr>
<tr>
<td>Mirza and Uy (2011)</td>
<td>–</td>
<td>213.0</td>
</tr>
<tr>
<td>Current research</td>
<td>SB1-1</td>
<td>35.5</td>
</tr>
<tr>
<td></td>
<td>SB1-2</td>
<td>35.5</td>
</tr>
<tr>
<td></td>
<td>CB2-1</td>
<td>36.5</td>
</tr>
<tr>
<td></td>
<td>CB2-3</td>
<td>36.5</td>
</tr>
</tbody>
</table>
Figure 9.11 Comparison between test and model results of rotational stiffness of the steel beam-CFST column composite joints
9.2.2 Plastic moment capacity calculation models

The plastic moment capacity of the endplate steel beam-CFST column composite joint can be calculated from a summation of the tensile resistance and compressive resistance of the components of the joint using a plastic analysis approach based on the strain block. The tensile and compressive resistance of the components of the joint are determined from their tensile forces and compressive forces, multiplied by the lever arms from the plastic neutral axis. The location of the plastic neutral axis generally depends on the total tensile resistance force \( F_t = F_r + F_{ss} + F_b \) and total compressive resistance force of beam flange \( F_c = F_{bf} \), where \( F_r \) is the resistance capacity of the reinforcement, \( F_{ss} \) is the resistance capacity of profile sheet, \( F_b \) is the resistance...
capacity at top row bolt level, and $F_{bf}$ is the compressive force in the bottom flange of the beam.

If $F_c \geq F_t$, the plastic neutral axis is located in the centre of the bottom flange of the beam. The plastic moment is then calculated by taking the moment about the centre of the bottom flange of the beam, as shown in Figure 9.12.

![Figure 9.12 Internal force distribution when $F_c \geq F_t$](image)

If the total compressive force ($F_c$) is less than the total tensile force ($F_t$), the plastic neutral axis is located on the web below the top row of bolts, or above the top row of bolts, or on the top flange of the beam. The position of the plastic neutral axis can be determined from a force equilibrium equation in which the total tensile force $F_t$ is equal to the total compressive force $F_c$ ($F_t = F_c$). When the total tensile force ($F_t = F_r + F_{ss} +$
If $F_b$ exceeds the total compressive resistance force ($F_{bf}$) of the beam flange, a plastic stress block is assumed, as shown in Figure 9.13, in the lower part of the beam web. The additional force ($F_t - F_{bf}$) is carried by the beam web as force $F_{bw}$ below the plastic neutral axis.

(1) Plastic moment capacity of the connection when $F_c \geq F_t$

If $F_c \geq F_t$, the plastic neutral axis is located at the centre of the bottom flange of the beam. The joint plastic resistance moment is calculated from:

$$M_p = F_r d_r + F_{ss} d_{ss} + F_{b1} d_{b1} + F_{b2} d_{b2} \tag{9.33}$$

where the lever arms ($d_r, d_{ss}, d_{b1}, d_{b2}$) of forces ($F_r, F_{ss}, F_{b1}, F_{b2}$) from the plastic neutral axis as shown in Figure 9.12 are calculated as below:

$d_r = D_r$

$d_{ss} = H_b + y_{ss}$

$d_{b1} = H_b - e_1$

$d_{b2} = H_b - p_1 - e_1$

$$D_r = H_b + D_c - \text{cover} - \frac{t_{bf}}{2}$$

where $H_b$ is the beam height, and $D_c$ is the slab depth.

(2) Plastic moment capacity of the connection when $F_c < F_t$

If $F_c < F_t$, the position of the plastic neutral axis on the beam web must be calculated based on the equilibrium of forces (total tensile forces = total compressive forces). After calculating the position of the plastic neutral axis and the individual forces in the components of the joint, the plastic moment of the connection is calculated by taking the moment about the plastic neutral axis:

$$M_p = F_r d_r + F_{ss} d_{ss} + F_b d_b + F_w d_w + F_f d_f \tag{9.34}$$

where the lever arms ($d_r, d_{ss}, d_b, d_{bw}, d_{bf}$) of forces ($F_r, F_{ss}, F_b, F_{bw}, F_{bf}$) from the plastic neutral axis in Figure 9.13 can be calculated from:

$d_r = d_n$

$d_{ss} = d_n - (D_c - \text{cover} - y_{ss})$

$d_b = d_n - (D_c - \text{cover} + e_1)$

$d_f = D_r - d_n$
\[
\begin{align*}
d_w &= \frac{1}{2} \left( d_f - \frac{t_{bf}}{2} \right) \\
D_r &= H_b + D_c - \text{cover} - \frac{t_f}{2}
\end{align*}
\]

**Plastic Neutral Axis**

By equilibrium of force acting to the connection as shown in Figure 9.13,

\[
\text{Tension} = \text{Compression} \\
F_r + F_{ss} + F_b = F_{bf} + F_{bw}
\] (9.35)

The value of the \(F_r, F_{ss}, F_b, F_{bf}, F_{bw}\) is calculated from the following:

\[
\begin{align*}
F_r &= N_r A_r E_r \varepsilon_r \\
F_{ss} &= A_{ss} E_{ss} \varepsilon_{ss} \\
F_b &= N_b A_b E_b \varepsilon_b \\
F_{bf} &= A_f E_f \varepsilon_f \\
F_{bw} &= A_w E_w \varepsilon_w
\end{align*}
\]

The value of \(\varepsilon_r, \varepsilon_{ss}, \varepsilon_b, \varepsilon_w, \varepsilon_f\) can be calculated from the stress block, as shown in Figure 9.13, and based on the failure mode of the connection.

The plastic moment capacity of a composite joint depends on the failure modes. It is observed from the test results that there are four failure modes found in the flush endplate composite joints; due to either shear stud failure (Figure 9.14(a)), or reinforcement fracture (Figure 9.14(b)), or buckling of beam flange (Figure 9.14(c)), or yielding of reinforcement with buckling of beam flange (Figure 9.14(d)). These failure modes can be classified according to the value of reinforcement ratio \((\rho)\), shear connection ratio \((\eta_y)\) and beam to slab depth ratio. The reinforcement ratio \((\rho)\) is defined as the percentage of the reinforcement relative to the concrete area above the steel profile sheet.
Figure 9.14 Different failure modes of typical steel beam-CFST column composite joints
(a) Shear stud failure

It is seen from the tests reported by Fu and Lam (2006) for steel beam–steel column composite joints that joints sometimes failed due to shear stud failure. This is observed when there were too few shear studs in the shear span, and when the reinforcement ratio \( \rho = \frac{A_r}{A_c} \times 100 \% \) is less than 0.8 and the shear connection ratio \( \eta_y = \frac{N_{sc,sc,max}}{A_{fr,fr,y}} \) is less than 1, in which \( A_r \) is the total longitudinal reinforcement area and \( A_c \) is the effective slab cross-sectional area \((b_e \times D_c)\). It is seen from Table 9.7 that the joints failed following shear stud failure when \( \rho < 1 \) and \( \eta_y \leq 1 \).

Table 9.7 Test data of the flush endplate steel beam-to-steel column composite joints due to the shear connections failure

<table>
<thead>
<tr>
<th>Test conducted by</th>
<th>Test specimen no.</th>
<th>( A_c ) ((b_e \times D_c)) (mm)</th>
<th>( H_b/D_c )</th>
<th>( \rho ) (%)</th>
<th>Shear connectors ( \frac{\rho}{N_{sc}} ) ( \eta_y )</th>
<th>( M_u ) (kN·m)</th>
<th>( \phi_u ) (mrad)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fu and Lam (2006)</td>
<td>CJ3</td>
<td>2.53</td>
<td>0.6</td>
<td>2</td>
<td>0.71</td>
<td>250</td>
<td>-</td>
<td>G</td>
</tr>
<tr>
<td></td>
<td>CJ4</td>
<td>1890×150</td>
<td>2.53</td>
<td>0.6</td>
<td>1.07</td>
<td>368</td>
<td>37.40</td>
<td>G</td>
</tr>
<tr>
<td></td>
<td>CJ5</td>
<td>2.53</td>
<td>0.6</td>
<td>3</td>
<td>1.07</td>
<td>363</td>
<td>31.70</td>
<td>G</td>
</tr>
</tbody>
</table>

Note: \( \rho = \frac{A_r}{A_c} \times 100 \% \); \( \eta_y = \frac{N_{sc,sc,max}}{A_{fr,fr,y}} \)

G = Shear stud failure.

(b) Reinforcement fracture

This failure mode is observed when there is insufficient slab reinforcement on the hogging moment area of joint, as observed in tests conducted by Anderson and Najafi (1994), Xiao et al. (1994), Liew et al. (2000), Brown and Anderson (2001), Fu and Lam (2006), and Loh et al. (2006). It is seen from these tests that joint failure was due to reinforcement fracture. It is seen from Table 9.8 that the joints failed due to the reinforcement failure when \( \rho \leq 1 \) and \( \eta_y > 1 \).
Table 9.8 Test data for flush endplate steel beam-steel column composite joint failure due to concrete slab reinforcement fracture

<table>
<thead>
<tr>
<th>Test conducted by</th>
<th>Test specimen No.</th>
<th>$A_c$ ($b_e \times D_c$) (mm)</th>
<th>$H_b/D_c$</th>
<th>$\rho$ (%)</th>
<th>Shear connectors $N_{sc}$ $\eta_y$</th>
<th>$M_u$ (kN-m)</th>
<th>$\phi_u$ (mrad)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>Anderson and Najafi (1994)</td>
<td>S4F</td>
<td>1100 x 120</td>
<td>2.53</td>
<td>0.55</td>
<td>7</td>
<td>3.76</td>
<td>179</td>
<td>15.70 B</td>
</tr>
<tr>
<td></td>
<td>S8FD</td>
<td>1200 x 120</td>
<td>3.81</td>
<td>1.1</td>
<td>7</td>
<td>1.91</td>
<td>416</td>
<td>- B</td>
</tr>
<tr>
<td>Xiao et al. (1994)</td>
<td>SCJ3</td>
<td>1200 x 120</td>
<td>2.07</td>
<td>0.3</td>
<td>6</td>
<td>6.41</td>
<td>85.7</td>
<td>7.20 B</td>
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<tr>
<td>Liew et al. (2000)</td>
<td>SCCB1</td>
<td>1500 x 120</td>
<td>2.54</td>
<td>0.5</td>
<td>7</td>
<td>1.69</td>
<td>271</td>
<td>- B</td>
</tr>
<tr>
<td>Brown and Anderson (2001)</td>
<td>Test-3</td>
<td>1100 x 120</td>
<td>3.75</td>
<td>1.0</td>
<td>7</td>
<td>1.86</td>
<td>390</td>
<td>47.7 B</td>
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<td></td>
<td>Test-4</td>
<td></td>
<td>3.75</td>
<td>1.0</td>
<td>7</td>
<td>1.86</td>
<td>370</td>
<td>35.5 B</td>
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<td>7</td>
<td>1.86</td>
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<td>20.1 B</td>
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<td>Fu and Lam (2006)</td>
<td>CJ1</td>
<td>1890 x 150</td>
<td>3.09</td>
<td>0.6</td>
<td>7</td>
<td>2.5</td>
<td>370</td>
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<tr>
<td></td>
<td>CJ3</td>
<td></td>
<td>3.09</td>
<td>0.6</td>
<td>4</td>
<td>1.43</td>
<td>363</td>
<td>- B</td>
</tr>
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<td></td>
<td>CJ6</td>
<td></td>
<td>3.09</td>
<td>0.6</td>
<td>6</td>
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<td>425</td>
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<td>3.09</td>
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<td>2</td>
<td>1.12</td>
<td>274</td>
<td>30.00 B</td>
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<td>CJ8</td>
<td></td>
<td>3.09</td>
<td>0.6</td>
<td>4</td>
<td>1.12</td>
<td>439</td>
<td>42.30 B</td>
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<td>Loh et al. (2006)</td>
<td>CJ4</td>
<td>1000 x 120</td>
<td>2.07</td>
<td>0.65</td>
<td>3</td>
<td>1.33</td>
<td>143.3</td>
<td>21 B</td>
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<td>Current research</td>
<td>SB1-1</td>
<td>900 x 120</td>
<td>2.53</td>
<td>0.76</td>
<td>8</td>
<td>3.41</td>
<td>238.4</td>
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<td>0.76</td>
<td>8</td>
<td>3.41</td>
<td>264.0</td>
<td>23.2 B</td>
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<td>0.76</td>
<td>8</td>
<td>3.41</td>
<td>277.2</td>
<td>23.6 B</td>
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<tr>
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<td>CB2-3</td>
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<td>2.53</td>
<td>0.76</td>
<td>8</td>
<td>3.41</td>
<td>270.7</td>
<td>22.4 B</td>
</tr>
</tbody>
</table>

Table 9.9 Test data of the flush endplate steel beam-to-steel column composite joints due to the local buckling of beam flange

<table>
<thead>
<tr>
<th>Test conducted by</th>
<th>Test specimen no.</th>
<th>$A_c$ ($b_e \times D_c$) (mm)</th>
<th>$H_b/D_c$</th>
<th>$\rho$ (%)</th>
<th>Shear connectors $N_{sc}$ $\eta_y$</th>
<th>$M_u$ (kN-m)</th>
<th>$\phi_u$ (mrad)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liew et al. (2000)</td>
<td>SCCB2</td>
<td>1500 x 120</td>
<td>2.54</td>
<td>1.1</td>
<td>14</td>
<td>1.52</td>
<td>441</td>
<td>51.90 E</td>
</tr>
<tr>
<td></td>
<td>SCCB3</td>
<td></td>
<td>2.54</td>
<td>1.56</td>
<td>20</td>
<td>1.55</td>
<td>449</td>
<td>- E</td>
</tr>
<tr>
<td>Loh et al. (2006)</td>
<td>CJ1</td>
<td>1000 x 120</td>
<td>2.07</td>
<td>1.29</td>
<td>5</td>
<td>1.1</td>
<td>185.5</td>
<td>30 0.5 E</td>
</tr>
<tr>
<td></td>
<td>CJ3</td>
<td></td>
<td>2.07</td>
<td>1.29</td>
<td>3</td>
<td>0.66</td>
<td>187.9</td>
<td>38 4.5 E</td>
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<tr>
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<td>CJ3</td>
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<td>1.29</td>
<td>2</td>
<td>0.44</td>
<td>178.9</td>
<td>45 7.4 E</td>
</tr>
</tbody>
</table>

Note: E = Buckling of beam flange.

(c) Local buckling of beam flange

Local buckling of the bottom flange of the beam is one of the major reasons observed in joint failures. This failure mode is observed when the bottom flange of the beam has insufficient compressive strength capacity, even with adequate slab reinforcement, on
the hogging moment area of the joint. This failure mode is noticed when $\rho > 1$ and $\eta_y \leq 1.6$, as seen in Table 9.9.

**d) Yielding of reinforcement with buckling of beam flange**

This failure mode is observed due to the insufficient compressive strength of the bottom flange of the beam and with insufficient slab reinforcement in the hogging moment area of the joint. This failure mode is noticed when $\rho > 1$ and $\eta_y > 1.2$, as shown in Table 9.10.

Table 9.10 Test data for flush endplate steel beam–steel column composite joints due to the yielding of reinforcement and local buckling of beam flange

<table>
<thead>
<tr>
<th>Test conducted by</th>
<th>Test specimen no.</th>
<th>$A_c = (b_e \times D_c)$ (mm)</th>
<th>$H_b/D_c$</th>
<th>$\rho$ (%)</th>
<th>$\frac{N_{sc}}{\eta_y}$</th>
<th>$M_u$ (kN-m)</th>
<th>$\phi_u$ (mrad)</th>
<th>Slip Failure Mode</th>
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<tbody>
<tr>
<td>Anderson and Najafi (1994)</td>
<td>S8F</td>
<td>1100 x120</td>
<td>2.53</td>
<td>1.1</td>
<td>7</td>
<td>1.91</td>
<td>262</td>
<td>–</td>
</tr>
<tr>
<td>Li et al. (1996)</td>
<td>SCJ1</td>
<td>1000 x110</td>
<td>2.34</td>
<td>1.2</td>
<td>14</td>
<td>4.61</td>
<td>181.5</td>
<td>47.00</td>
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<td></td>
<td>SCJ3</td>
<td></td>
<td>2.34</td>
<td>1.2</td>
<td>14</td>
<td>4.61</td>
<td>176</td>
<td>42.00</td>
</tr>
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<td></td>
<td>SCJ4</td>
<td></td>
<td>2.34</td>
<td>1.2</td>
<td>14</td>
<td>4.61</td>
<td>177.5</td>
<td>58.00</td>
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<td>SCJ5</td>
<td></td>
<td>2.34</td>
<td>1.2</td>
<td>14</td>
<td>4.61</td>
<td>197.5</td>
<td>60.00</td>
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<tr>
<td></td>
<td>SCJ6</td>
<td></td>
<td>2.34</td>
<td>1.2</td>
<td>14</td>
<td>4.61</td>
<td>174</td>
<td>–</td>
</tr>
<tr>
<td>Loh et al. (2006)</td>
<td>CJ5</td>
<td>1000 x120</td>
<td>2.07</td>
<td>1.94</td>
<td>8</td>
<td>1.18</td>
<td>192.1</td>
<td>19</td>
</tr>
</tbody>
</table>

Note: B- Fracture of the mesh reinforcement; E-Buckling of the beam flange;

**9.2.3 Rotational capacity calculation models**

Empirical models are available for calculating the rotation capacity of flush endplate connections for steel beam-steel column composite joints. The ultimate rotation ($\phi_u$) of a flush endplate connection for composite joints depends on the deformation of the reinforcement ($\Delta_r$) and slip of the shear connector measured as the slip between the slab and the beam ($\Delta_s$) as well as the deformation of the beam bottom flange ($\Delta_a$). Anderson et al. (2000) proposed two expressions for two different conditions, based on the tension and compression resisting capacity of a joint. In the present work, the formula proposed by Anderson et al. (2000) for the steel beam-steel column composite joints was used to determine the ultimate rotation capacity of the steel beam–CFST column composite joints. The value of $\phi_u$ for steel beam-CFST column composite joints can be determined from the expression proposed by Anderson et al. (2000):
(i) If $F_f \geq F_r$.

$$\phi_u = \frac{\Delta_r}{D_r} + \frac{\Delta_s}{H_b - 0.5\ t_{bf}}$$  \hspace{1cm} (9.36)

(ii) If $F_f < F_r$.

$$\phi_u = \frac{\Delta_r}{D_r} + \frac{\Delta_s + \Delta_a}{H_b - 0.5\ t_{bf}}$$  \hspace{1cm} (9.37)

where $D_r$ is the distance of the reinforcement from the centre of the bottom flange of the beam; $H_b$ is the depth of the steel beam and $t_{bf}$ is the thickness of the beam flange.

(a) Determination of deformation of reinforcement ($\Delta_r$)

The deformation of reinforcement ($\Delta_r$) generally depends on the elongation of the longitudinal bar ($L_{eff}$), ‘transmission’ length ($L_t$), and the average ultimate strain ($\varepsilon_{smu}$). Formulas suggested by Anderson et al. (2000) and Fu et al. (2010) for the calculation of $\Delta_r$ are as follows.

(i) Shear stud failure (for $\rho < 1\%$ and $\eta_y \leq 1$):

$$\Delta_r = \left(\frac{D_{cd}}{2} + P_0 + P_1\right) \times \varepsilon_r$$  \hspace{1cm} (9.38)

where $\varepsilon_r = 0.002$; $D_{cd}$ is the column depth; $P_0$ is the distance from the face of column to the first shear stud; and $P_1$ is the shear stud spacing from centre to centre.

(ii) Reinforcement fracture (for $\rho \leq 1\%$ and $\eta_y > 1$):

$$\Delta_r = 2 \times L_t \times \varepsilon_{smu}$$  \hspace{1cm} (9.39)

The value of $\varepsilon_{smu}$ and $L_t$ can be determined as below.

- **Determination of $L_t$:**

$$L_t = \frac{f_{ctm}k_c\phi}{4\tau_{sm}\rho}$$  \hspace{1cm} (9.40)

where $\phi$ is the diameter of the reinforcement; $f_{ctm}$ is the mean tensile strength of concrete; $\tau_{sm}$ is the average bond stress along the transmission length; $\rho$ is the reinforcement ratio; and $k_c$ is a coefficient that allows for the self-equilibrating stresses and the stress distribution in the slab prior to cracking.
\[ \rho = \frac{A_s}{A_c} \]

\[ k_c = \frac{1}{1 + \frac{h_c}{2z_0}} \]

\[ \tau_{sm} = 1.8 f_{ctm} \]

where \( h_c \) is the thickness of the concrete slab, excluding any haunch or ribs \((h_c = D_c - \text{rib})\), \( D_c \) is the thickness of the composite slab, \( z_0 \) is the vertical distance from the centroid of the uncracked, unreinforced concrete slab to the centroid of the uncracked, unreinforced composite section.

\[ z_0 = y_1 - y_2 \]

where \( y_1 = H_b + D_c - h_c/2 \) and \( y_2 = \frac{A_s H_b/2 + A_c y_1}{A_s + A_c} \).

**Determining \( \varepsilon_{smu} \):**

\[ \varepsilon_{smu} = \varepsilon_{sy} - \beta_t \Delta \varepsilon_{sr} + \delta \left( 1 - \frac{\sigma_{sr1}}{f_{ys}} \right) (\varepsilon_{su} - \varepsilon_{dy}) \]  \hfill (9.41)

where \( \varepsilon_{sy} \) is the strain at yield stress; \( \varepsilon_{su} \) is the ultimate strain of the steel reinforcement bars; \( \Delta \varepsilon_{sr} \) is the increased strain in the reinforcement at the crack; and \( \sigma_{sr1} \) is the stress in the reinforcement at the crack when the crack is open. \( \beta_t \) and \( \delta \) are constants, where \( \beta_t = 0.4 \) and \( \delta = 0.8 \). The values of \( \sigma_{sr1} \) and \( \Delta \varepsilon_{sr} \) can be determined from the following expressions.

\[ \sigma_{sr1} = \frac{f_{ctm} k_c}{\rho} \left( 1 + \rho \frac{E_{sr}}{E_c} \right) \]  \hfill (9.42)

\[ \Delta \varepsilon_{sr} = \frac{f_{ctm} k_c}{E_s \rho} \]  \hfill (9.43)

The mean tensile strength of concrete \( (f_{ctm}) \) can be determined from the expression provided in Eurocode 4 (2005):

\[ f_{ctm} = \begin{cases} 0.3(f_{ck})^{2/3} & \leq C50/60 \\ 2.12 \ln(1 + (f_{ck} + 8)/10) & > C50/60 \end{cases} \]  \hfill (9.44)

\[ \Delta_r = \left( \frac{D_{cd}}{2} + L_t \right) \times \varepsilon_{smu} + (P_0 + P_1 - L_t) \times \varepsilon_r \]
(iii) Reinforcement yielding and buckling of beam flange (for $\rho > 1\%$ and $\eta_y > 1.2$)

$$\Delta_r = \begin{cases} 
\left(\frac{D_{cd}}{2} + L_t \right) \times \varepsilon_{smu} & \text{when } P_0 + P_1 < L_t \\
\left(\frac{D_{cd}}{2} + L_t \right) \times \varepsilon_{smu} + (P_0 + P_1 - L_t) \times \varepsilon_r & \text{when } P_0 + P_1 < L_t 
\end{cases}$$

where $\varepsilon_r$ is the average strain rather than the yield strain of the longitudinal bar; $D_{cd}$ is the column depth; $P_0$ is the distance between the column face and the centreline of the first shear stud; and $P_1$ is the distance between the centreline of the first shear stud and the second shear stud. $L_t$ and $\varepsilon_{smu}$ can be determined using Eqs. (9.40) and (9.41).

(b) Determination of slip of shear connector ($\Delta_s$)

An empirical formula has been used by Al-Aasam (2013) to determine the value of the slip between slab and steel beam ($\Delta_s$):

$$\Delta_s = \frac{F}{N_{sc} k_{sc}}$$

where $N_{sc}$ is the number of the shear connectors in the hogging region; $k_{sc}$ is the stiffness of a single shear connector; and $F$ can be calculated from:

$$F = \text{Min} \begin{cases} 
A_r f_{y,r} & \text{when } \eta_y \geq 1 \\
N_{sc} f_{sc,max} & \text{when } \eta_y < 1 
\end{cases}$$

The maximum shear strength of the shear connection ($F_{sc,max}$) can be calculated from the formula provided in Eurocode 4 (2005):

$$F_{sc,max} = \min \begin{cases} 
0.8 f_u \left(\frac{\pi d^2}{4}\right) \\
0.37 \sqrt{f'c} E_c \left(\frac{\pi d^2}{4}\right)
\end{cases}$$

The stiffness of a single shear connector can be calculated from the equation proposed by Al-Aasam (2013):

$$k_{sc} = 1.47 F_{sc,max}$$
9.2.4 Moment-rotation relationship models

The moment-rotation relationship of blind-bolted flush endplate steel beam–CFTS column composite joints was developed in the present work based on the concept of material model illustrated in Chapter 5. The full-range moment-rotation behaviour for a blind-bolted flush endplate connection in steel beam-CFST column composite joints is shown in Figure 9.15, expressed as:

\[
M = \begin{cases} 
S_{j,ini} \phi & \text{for } 0 < \phi \leq \phi_e \\
M_p - (M_p - M_e) \left( \frac{\phi_p - \phi}{\phi_p - \phi_e} \right)^n & \text{for } \phi_e < \phi \leq \phi_p \\
M_u - (M_u - M_p) \left( \frac{\phi_u - \phi}{\phi_u - \phi_p} \right)^n & \text{for } \phi_p < \phi \leq \phi_u 
\end{cases}
\]  

(9.50)

where:

- \( \phi_e \) is the rotation corresponding to the elastic moment \( M_e \);
- \( \phi_p \) is the plastic rotation corresponding to the plastic moment \( M_p \);
- \( \phi_u \) is the ultimate rotation corresponding to the ultimate moment \( M_u \);
- \( S_{j,ini} \) is the initial rotational stiffness known as rotational stiffness of joint;
- \( S_{j,s} \) is the secant stiffness of joint;
- \( S_{j,p} \) is the plastic stiffness known also as strain hardening stiffness of joint;
- \( \eta \) is the shape factor which depends on the value of \( S_{j,ini} \) and \( M_p \) in which:

\[
\eta = \ln \left( \frac{M_p}{S_{j,ini}} \right)
\]

\[
S_{j,p} = \beta S_{j,ini}
\]
(a) Elastic moment ($M_e$)

The elastic moment ($M_e$) of the connection is related to the plastic moment ($M_p$), expressed as $M_e = \alpha M_p$. The value of $\alpha$ normally depends on the types of sections used to develop the connection. In general, it can be assumed that $\alpha = 0.67$ for the steel section used in joints, but in composite steel beam-CFST column composite joints, different types of sections are used. The value of $\alpha$ was suggested by Wen (2009) to be 0.62 for steel beam–steel column composite joints. In the present work, the value of $\alpha$ is taken to be 0.61 due to the composite material used in the CFST column. Then,

$$M_e = 0.61 M_p$$  \hspace{1cm} (9.51)

(b) Plastic moment ($M_p$)

The plastic moment capacity of blind-bolted endplate steel beam-CFST column composite joints can be calculated by summing the tensile and compressive resistances of the components of the joint, using the plastic analysis approach. The calculations are set out in section 9.2.2. This moment can be calculated approximately from $0.85M_u$.

(c) Ultimate moment ($M_u$)

The ultimate moment resistance capacity of the flush endplate connection is determined from the resisting capacity of each component in the connection (Figure 9.16). If $F_c \geq F_t$, the ultimate moment resistance capacity of composite joints can be determined using Eq. (9.49), taking the moment from the centre of the bottom flange:

$$M_u = R_r D_r + R_{ss} (H_b' + y_{ss}) + R_{b1} d_{b1} + R_{b2} d_{b2}$$  \hspace{1cm} (9.52)

If $F_c < F_t$, the ultimate moment resistance capacity of the composite joint is determined by Eq. (9.50), by taking the moment from the centre of the bottom flange. In this case, the position of the neutral axis from the centre of the beam bottom must be calculated:

$$M_u = R_r D_r + R_{ss} (H_b' + y_{ss}) + R_b d_b - R_{bw} \left(\frac{y_c + t_{bf}}{2}\right)$$  \hspace{1cm} (9.53)

The position of the neutral axis from the centre of the beam bottom flange is found by the following expression based on equilibrium of tensile and compressive forces:

$$y_c = \frac{(R_r + R_{ss} + R_b - R_f)}{t_{bw} f_{bw}}$$  \hspace{1cm} (9.54)
where $y_c$ is the depth of the stress block of the beam web in compression; $t_{bw}$ is the beam web thickness; and $f_{bw}$ is the yield strength of the beam web.

It is seen from Eqs. (9.52) and (9.53) that the ultimate moment resistance capacity of a joint depends on the tensile resistance capacity of reinforcement or shear connectors ($R_r$), tensile resistance capacity of profile sheet ($R_{ss}$), tensile resistance capacity of the top row of bolts ($R_b$), and the compressive resistance capacities of the beam flange ($R_f$) and beam web ($R_w$). The details for calculating $R_r, R_{ss}, R_b, R_f, R_w$ are illustrated in below.

(i) $F_c \geq F_t$

(ii) $F_c < F_t$

![Figure 9.16 Stress blocks of components of flush endplate connection](image)

(c1) **Tensile resistance of reinforcement ($R_r$)**

The calculation of the tensile resistance capacity of the reinforcement ($R_r$) depends on the ratio of shear connectors ($\eta_y$). The value of $R_r$ is determined using Eq. (9.55), from the minimum resisting capacity of reinforcement and shear connectors. If $\eta_y < 1$, the tensile resistance ($R_r$) is calculated from shear connectors; otherwise from slab reinforcement:

$$R_r = \min \left\{ \begin{array}{ll} A_r f_{y,r} & \text{when } \eta_y \geq 1 \\ N_{sc} A_{sc} f_{u,sc} & \text{when } \eta_y < 1 \end{array} \right.$$  \hspace{1cm} (9.55)

where, $f_{y,r}$ is the yield strength of reinforcement bars; $A_r$ is the total cross-sectional area of the reinforcement; $N_{sc}$ is the total number of shear connectors; $A_{sc}$ is the cross-
sectional area of shear connectors; and $f_{u,sc}$ is the ultimate strength of the shear connectors.

(c2) Tensile resistance of profile sheet ($R_{ss}$)
The tensile resistance of the profile sheet is normally ignored in the calculation of the moment capacity of a composite joints; however, the parametric analysis in Chapter 8 found that the presence of the profile sheet affects the moment capacity of composite joints, such that the moment capacity of the joint is significantly enhanced when the yield strength of the profile sheet is increased. The profile sheet strength is therefore included in the calculation of the moment capacity of composite CFST column joints. The tensile resistance capacity of a profile sheet ($R_{ss}$) is given by:

$$R_{ss} = A_{ss}f_{y,ss}$$ (9.56)

where $A_{ss}$ is the effective cross-sectional area of the profile sheet, and $f_{y,ss}$ is its yield strength. The effective cross-sectional area of the profile sheet ($A_{ss}$) at the column section is calculated as $A_{ss} = (b_e - B) \times$ cross-sectional area of profile steel per meter, in which $b_e$ is the effective width of the slab and $B$ is the width of the CFST column.

(c3) Tensile resistance of the top row of bolts ($R_b$)
Tensile resistance capacity of the top row of bolts depends on the resistance capacity of the endplate in bending ($R_{pb}$), the tensile resistance capacity of the bolts, and the resistance capacity of the tube face in bending ($R_{cb}$). The value of $R_r$ is found from:

$$R_b = \min \left( R_{pb}, R_b, R_{cb} \right)$$ (9.57)

and the method of calculating $R_{pb}, R_b, R_{cb}$ is given in the following.

(i) Determination of resistance capacity of endplate in bending ($R_{pb}$)
The resistance capacity of an endplate in bending ($R_{pb}$) is calculated from a relationship proposed by Li et al. (1996b), also used by Amhed and Nethercot (1996):

$$R_{pb} = 5.5 - 0.021 m_c + 0.017e) t_{pb}^2 f_{y,pb}$$ (9.58)
where $m_c$ is the distance from the bolt centre to the beam web (mm); $e$ is the distance from the centreline of the bolt to the outer edge of the plate (mm); $t_{pb}$ is the thickness of the endplate (mm); and $f_{y, pb}$ is the endplate yield strength (MPa).

(ii) Determination of resistance capacity of blind-bolts in tension ($R_b$)
The tensile resistance capacity of a Hollo-Bolt is calculated from Table 6.2 reported in Chapter 6. The tensile resistance capacity a Hollo-Bolt can also be determined using Eq. (9.56), however this value is found lower than the actual tensile value of Hollo-Bolt.

$$R_b = A_b f_{y,b}$$  \hspace{1cm} (9.59)

(iii) Determination of resistance capacity of steel tube in bending ($R_{cb}$)
The resistance capacity of steel tube at the top row of bolts, due to bending, can be determined using a yield line mechanism. A number of studies on different types of bolt used for connections to hollow section columns have led to proposed mathematical expressions using yield line theory for the resistance capacity of hollow tubes. Three different models found in literature: Yeomans (1996) for hollow sections with Flowdrill bolts, Mourad (1994) for hollow sections with Huck high-strength blind bolts (HSBB), and Elamin (2013) for CFST sections using Hollo-Bolts. The expression proposed by Elamin (2013) from yield line mechanism for Hollo-Bolts and CFST column was adopted in the present study since the current research is related to Hollo-Bolts and CFST column. The resistance capacity of stainless steel tube in bending ($R_{cb}$) may be determined from Eq. (9.57) proposed by (Elamin, 2013) for carbon steel tube.

$$R_{cb} = \frac{f_{y,c}}{2} \left[ \pi \left( 1 + \frac{r}{r - \frac{c}{2}} \right) + \frac{g}{r} \right]$$ \hspace{1cm} (9.60)

where $r$ is the radius of circular yield line determined from $r = \frac{B - g - t_{cb}}{2}$; $c = k_{yf} d_h$; $B$ is the width of the CFST column; $t_{cb}$ is the thickness of the steel tube of the CFST column; $g$ is the bolt gauge (horizontal); $d_h$ is the outer diameter of the sleeve of the Hollo-bolt; and $k_{yf}$ is a yield line force calibration factor.

To calculate the value of $k_{yf}$, Elamin (2013) provided three individual equations with three different charts for the three parameters included ($\mu, \beta$ and $f'_c$). For simplicity,
the following simplified expression was developed based on the charts given by Elamin (2013):

\[ k_y = 0.84274 + 0.0054463\mu + 0.3164\beta + 0.0010883f'_{c} \]  

(9.61)

where \( \mu \) is the slenderness ratio of the hollow section (\( \mu = \frac{B}{t_{tb}} \)); \( \beta \) is the ratio of bolt gauge (horizontal) to width of the hollow section (\( \beta = \frac{b}{B} \)); and \( f'_{c} \) is the characteristic strength of concrete (MPa).

(c4) Compressive resistance of beam bottom flange \( (R_f) \) and beam web \( (R_w) \)

The design strength of the bottom flange of a beam depends on the capacity of the beam flange in compression as well as the beam flange in local buckling. The equation recommended by Eurocode 3 (2001) is used to calculate the resistance capacity of the beam bottom flange:

\[
R_f = \min \begin{cases} 
 t_{bf}b_{bf}f_{bf} & \text{for beam flange in compression} \\
 22t_{bf}^2f_{pb} & \text{for beam flange in local buckling}
\end{cases} \quad \frac{b_{bf}}{t_{bf}} < 22 \sqrt{\frac{235}{f_{bf}}}
\]

(9.62)

where \( b_{bf} \) and \( t_{bf} \) are the width and thickness of the beam flange, and \( f_{bf} \) is its yield strength.

The resistance capacity of the beam web \( (R_w) \) is calculated based on the stress block generated on the beam web: \( R_w \)

\[ R_{bw} = y_c t_{bw} f_{bw} \]

(9.63)

where \( y_c \) is the depth of the stress block of the beam web in compression; \( t_{bw} \) is the thickness of the beam web; and \( f_{bw} \) is its yield strength.

(d) Elastic rotation \( (\phi_e) \)

Since the elastic rotation \( (\phi_e) \) is related to the elastic moment \( (M_e) \) and rotational stiffness \( (S_{j,ini}) \) of the joint, the value of \( \phi_e \) can be determined from:

\[ \phi_e = \frac{M_e}{S_{j,ini}} \]

(9.64)

(e) Plastic Rotation \( (\phi_p) \)

The plastic rotation \( (\phi_p) \) is related to the plastic moment \( (M_p) \) and secant stiffness \( (S_{j,sec}) \) of the joint. The value of \( \phi_p \) is determined from:
\[ \phi_p = \frac{M_p}{S_{j,s}} = \frac{\mu M_p}{S_{j,ini}} \]

where \( \mu \) is the stiffness ratio. The value of \( \mu \) can be calculated from the expression proposed by Eurocode 3 (2005): 
\[ \mu = \left( \frac{M_p}{M_e} \right)^{2.7} \]

This expression is proposed only for the steel beam-steel column joints. Since the behaviour of the steel beam-CSFT column joints differs from that of steel beam-steel column joints, in this study the following expression has been used. This expression is developed based on the best fitted curves method.
\[ \mu = \left( \frac{M_p}{M_e} \right)^{1.5} \quad (9.65) \]

(f) **Secant stiffness (\( S_{j,s} \)) and plastic stiffness (\( S_{j,p} \))**

The secant stiffness (\( S_{j,s} \)) is usually depends on the initial rotational stiffness (\( S_{j,ini} \)) and the plastic and elastic moments, and \( S_{j,s} \) is calculated from (Eurocode 3, 2005):
\[ S_{j,s} = \frac{S_{j,ini}}{\mu} \quad (9.66) \]

The plastic stiffness (\( S_{j,p} \)) can be calculated from:
\[ S_{j,p} = a S_{j,ini} \]

in which the value of \( a \) lies between 0.03 and 0.05 (Eurocode 3, 2005), and is assumed in this work to be 0.04 as recommended by Wang et al. (2013). Therefore:
\[ S_{j,p} = 0.04 S_{j,ini} \quad (9.67) \]

(g) **Validation of proposed moment-rotation relationship model**

The full-moment-rotation curve of blind-bolted flush endplate steel beam-CFTS column composite joints was developed based on the proposed moment-rotation relationship model illustrated in the previous section. In the proposed model, the only three parameters required are the initial stiffness (\( S_{j,init} \)), ultimate moment (\( M_u \)) and ultimate rotation (\( \phi_u \)). The other parameters can be calculated from the expressions given above.

The proposed moment-rotation relationship model was validated by comparing with available test data for blind-bolted flush endplate steel beam-CFTS column composite joints.
Figure 9.17 Comparison between test (Loh et al., 2006) and results of proposed model for moment-rotational relationship of steel beam-CFST column composite joints
Figure 9.17 shows a comparison between the tests reported by Loh et al. (2006) and the results of the proposed model. The initial stiffness, strain hardening and ultimate moment stages of the moment-rotation curves of the proposed model matched the test data very well. In this study, some specimens of blind-bolted flush endplate steel beam-CFTS column composite joints were tested under monotonic static loading. The results of those tests have also been compared with the proposed model results, as shown in Figure 9.18 and in Table 9.11. The proposed model results match those test results very closely, except the initial stiffness of specimen SB1-1. It is concluded from these comparisons that the proposed model predicts values very well up to the ultimate rotation capacity at the ultimate moment capacity.

![Comparison between test and proposed model results of moment-rotational relationship of the steel beam-CFST column composite joints](image)

Figure 9.18 Comparison between test and proposed model results of moment-rotational relationship of the steel beam-CFST column composite joints
Table 9.11 Test and predicted moment and rotation capacity of the blind-bolted endplate connections of steel beam-CFST column joints

<table>
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<tr>
<th>Joints</th>
<th>Ultimate moment capacity (kN-m)</th>
<th>Rotation at ultimate moment (mrad)</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>Test</td>
<td>Predicted</td>
</tr>
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</tr>
<tr>
<td>CJ2</td>
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</tr>
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</table>

9.3 Summary

This chapter provides the details of the mathematical model for calculating initial rotational stiffness, moment and rotation capacity of blind-bolted endplate connections to the steel beam-CFST column joints with slab. The developed mathematical model for joint rotational stiffness successfully predicts the initial rotational stiffness of the proposed composite joints.

From this it is concluded that the proposed joint stiffness model can be used to define the stiffness value in frame analysis, especially for semi-rigid connections. Finally, a full-range moment-rotation relationship model is proposed, in which only three parameters are required: initial stiffness \((S_{j,ini})\), the ultimate moment \((M_u)\) and the ultimate rotation \((\phi_u)\). The proposed model predicts the moment-rotation relationship very well up to the ultimate rotation capacity at the ultimate moment capacity. This model is suitable for use in the analysis of semi-rigid frames with blind-bolted endplate connections for CFST columns and composite steel beams.
10.1 Conclusions

This thesis has presented detailed information of an experimental and numerical study to investigate the behaviour of hybrid stainless-carbon steel composite beam-column joints. Stainless steel material was used to fabricate the steel tube of the concrete-filled steel tubular (CFST) column, and carbon steel material was used to fabricate the steel beams and other steel components. This thesis mainly focused on blind-bolted endplate connections, in order to develop reliable connections for joining steel beams to CFST columns. The main purpose of this work was to provide detailed information on mathematical models for the prediction of the initial rotational stiffness and moment–rotation relationship for blind-bolted endplate connections of steel beam–CFST column joints when a concrete slab is present. Key findings and observations of the experimental, numerical and analytical studies in this thesis are summarised in the following.

(a) From Experimental Studies:
In the test program, hybrid composite joints were investigated for blind-bolted endplate connections and through-plate connections. Through-plate connections were considered in the test program only for the purpose of comparison with endplate connections. The following conclusions were drawn from the test results.

- The test results showed that the moment capacity of the blind-bolted endplate connections with slab was double than that of through-plate connections with slab. Based on the test results and stiffness classifications proposed by Eurocode 3, blind-bolted endplate connections may be classified as rigid connection for braced frames and semi-rigid connection for unbraced frames, and can be adopted in high-
rise building structures. Through-plate connections may be classified as semi-rigid connection and can be considered for low-rise building structures if no lateral force resisting system is provided.

- The tests of the blind-bolted endplate connections showed that the moment capacity of connections without slab was very low (around seven times lower) than the moment capacity of connections when a slab is present. Blind-bolted endplate connections without slab failed due to outward deformation of the steel tube. This was observed to be due to the thin steel tube used in CFSST column, and separation of the steel tube from the concrete core by pull-out of the blind bolts. However, the presence of the slab on the blind-bolted endplate connections had a significant effect on the failure mode of joints. Blind-bolted endplate connections with slab failed due to the slab reinforcement fracture and through-plate connections with slab failed due to the buckling of the beam web.

- It was also seen that the moment capacity of blind-bolted endplate connections for square-section CFSST column was observed to be 14% lower compared to blind-bolted endplate connections for circular-section CFSST column. The addition of binding bars to the square-section steel tube column, enhanced the ultimate moment capacity of blind-bolted endplate connections to square-section CFSST column by only 4.7% lower compared to connections using circular-section CFSST columns without binding bars.

- Blind-bolted endplate connections with slab were tested under static and cyclic loading. The hogging moment capacity at cyclic loading was 3.6% lower for square column and 5.4% lower for circular column, than connections tested under static monotonic loading. The sagging moment capacity of joints tested under cyclic loading was also observed to be 68% less than the hogging moment for square columns, and 42% less for circular columns. The sagging moment reduction was observed to be the result of separation of the steel tube from the concrete near the bottom flange of the beam due to upward loading.

**(b) From Numerical Studies:**

Numerical studies were conducted using finite element (FE) analysis to investigate the behaviour of different components of the composite beam-CFST column joints with
blind-bolted endplate connections. Solid FE models were developed using ABAQUS software (2012) for the analysis of the composite beam-CFST column joints with blind-bolted connections; and validated by comparing with the test results. The behaviour of the blind-bolts (Hollo-bolts) under pure tension and pure shear loading was investigated. The effectiveness of binding bars for improving the behaviour of the blind-bolted endplate connections in steel beam–CFST column joints was investigated for the cases with and without the presence of a slab. Other parameters related to the composite slab, composite beam and connecting components and CFST column were also investigated to find out effective parameters that influence the connection behaviour especially the initial rotational stiffness and moment capacity of joints. The following conclusions can be drawn based on the FE modelling results.

- The parametric analysis results of Hollo-Bolts under pure tension and pure shear loading showed that their behaviour in pure tension was quite dissimilar from their behaviour in pure shear. Hollo-Bolts in tension failed due to fracturing of the sleeve, but in shear they failed following local deformation of the sleeve and bolt shank. For both loading cases, the FE model results of connections with Hollo-Bolts embedded in the concrete core of a CFST column were compared with their behaviour when connected to a square-section hollow steel column. The results showed that the ultimate tensile and shear capacity of Hollo-Bolts was higher when connected to CFST columns than when connected to a square-section hollow column. From this it may be concluded that Hollo-Bolts perform better in CFST columns than in square-section hollow columns.

- It was seen from both test and FE model results of the blind-bolted endplate connections in the steel beam-CFST column joints without slab that the steel tube separated from the concrete core in CFST columns when the tensile or bending load was transferred to the steel tube of the column from the beams. This steel tube separation was observed to be more pronounced when using Ajax ONESIDE bolted endplate connections than Hollo-Bolted endplate connections. To reduce the steel tube separation, binding bar stiffeners were proposed in this work to be attached (welded) to the column tube near the connections. FE modelling results of the connections with and without binding bars showed that the binding bars reduce the outward deformation problem, and also enhance endplate connection capacity.
Binding bars were predicted to be more effective on square-section CFST columns rather than on circular-section columns. Recommendations for binding bar design and positioning were proposed.

- From parametric analyses results of the composite steel beam-CFST column joints with a slab, it is seen that slab depth, reinforcement ratio, profile sheet, shear connectors, beam sections and beam materials all have a significant effect on the initial rotational stiffness and ultimate moment capacity of the composite CFST column joints. In general, the presence of a profile sheet is ignored in the design codes for the calculation of the moment capacity of the composite joints. However, the profile sheet was found to be one of the most effective parameters and has significant effect on the ultimate moment capacity of composite joints. It may be concluded from the parametric analysis that the profile sheet should be considered in the calculation of design moment capacity for composite joints.

- The effect of sagging bending load on the composite joints was investigated for different combinations of binding bars and width-to-thickness ratio of the steel tube of CFST column. The results showed that outward deformation was noticeable in the steel tube of the composite CFST column joints when sagging bending load acted to the joints, in resulting, sagging moment of the joints reduced significantly. The addition of binding bars to the steel tube of the CFST column significantly improved the sagging moment capacity of the joints and reduced the steel tube deformation. The analysis results demonstrated that binding bars are most effective when they are placed only near the top and bottom of the lower bolts of the composite CFST column joints.

(c) From Analytical Studies:

Analytical studies were carried out for the theoretical calculation and determination of the stiffness, moment and rotation capacity of blind-bolted endplate connections in the steel beam-CFST column joints with slab. The following conclusions can be drawn from the analytical studies.

- The developed mathematical model for joint stiffness successfully predicted the initial rotational stiffness of the proposed composite joints. The proposed joint
stiffness model may therefore be used to determine the stiffness in frame analysis, especially for semi-rigid connections.

- The calculation details of the yield moment and ultimate moment of the proposed joints are given. These can be helpful in determining the moment capacity of the proposed joints without doing any testing or FE analysis.
- The rotational capacity of the proposed joints was successfully calculated using existing mathematical expressions.
- A full-range moment-rotation relationship model is proposed in which only three parameters are required: initial stiffness \( S_{\text{ini}} \), ultimate moment \( M_u \) and ultimate rotation \( \phi_u \).
- At different stages, the moment-rotation curves of the proposed model matched test results very well. The proposed model predicts outcomes very well up to the ultimate rotation capacity at the ultimate moment capacity. This moment-rotation relationship model can be used in the analysis of semi-rigid frames with blind-bolted endplate connections, CFST columns and composite steel beams.

### 10.2 Recommendations for Future Research

As research is an ongoing process, further research is always required. A number of research areas were generated by the work in this thesis. The following are suggested recommendations for future research topics on composite steel beam–CFST column joints.

- From the test results of through-plate connections, it was observed that reasonable stiffness and moment capacity can be achieved for this type of connection, which are suitable for low-rise building construction. To implement these connections, more experimental work is needed, as limited testing was conducted in the present work.
- The present work mainly focused on blind-bolted endplate connections under static monotonic loading. Tests and FE results have shown that this type of connection under static monotonic loading has a good stiffness and moment capacity. In addition, a few tests were conducted under cyclic loading; these indicated that cyclic loading can potentially affect the ultimate moment and failure behaviour of
the blind-bolted endplate connections to CFSST column. Therefore more experimental work under cyclic loading needs to be carried out to further investigate the effects of cyclic loading, especially to improve the sagging moment capacity of the joints.

- FE model results show that Hollo-bolts in CFST columns have a better structural performance than in square-section hollow steel columns. No tests were carried out on Hollo-Bolts in pure shear loading, and few tests in pure tension, for CFST columns. To implement the Holo-Bolts in CFST column joints, more tests are needed to investigate their behaviour in CFST columns subjected to pure shear and tension.

- Although much effort has been made in this thesis on testing and FE development and modelling, and in developing mathematical models for predicting the initial rotational stiffness and moment-rotation relationship of blind-bolted endplate connections of composite beam-CFSST column joints, more test and FE analysis under static and cyclic loading are still needed to develop the design guidelines for blind-bolted endplate connections of composite beam-CFSST column joints.

- This research was limited to an investigation of isolated joints of blind-bolted endplate connections to CFSST columns when subjected to static monotonic and cyclic loading. To further the understanding and uses of this type of connection in structural frames, frame testing is needed on studying the behaviour of frames using CFST columns, composite steel beams and blind-bolted endplate connections.
REFERENCES


American Concrete Institute. 2011. Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary. MI, USA: Farmington Hills.


SCI. 2010. Stonecutters bridge towers. Steel Construction Institute, Structural stainless steel case study 01.


Standards Australia (1993). Australian Standard *AS 1170.4-1993*, Minimum design loads on structures (known as the SAA Loading Code) - Earthquake loads, Standards Australia International Ltd.


APPENDIX A

SEMI-RIGID FRAME ANALYSIS AND DESIGN

A.1 General Information of the Semi-Rigid Frame Analysis

Total height of the frame - 146.1 m (39-storey building)

Types of structure- Framed structure with composite beams and CFST columns

- Frame system: Moment resisting frame including partial braced frame
- Frame analysis method: Second order (P-Delta) analysis
- Connection type: Semi-rigid connections
- CFST column: stainless steel tube, concrete
- Frame analysis and design has been performed using software, SAP2000

Table A.1 Column and beam sizes used in different levels of the 39-storey building

<table>
<thead>
<tr>
<th>Level No</th>
<th>CFST column Size in mm</th>
<th>Steel beam size (d x b_f x t_f x t_w) in mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level 1-7</td>
<td>800 x 800 x 16</td>
<td>460UB67.1 454 x 190 x 12.7 x 8.5</td>
</tr>
<tr>
<td>Level 8-14</td>
<td>700 x 700 x 14</td>
<td>460UB67.1 454 x 190 x 12.7 x 8.5</td>
</tr>
<tr>
<td>Level 15-21</td>
<td>600 x 600 x 12</td>
<td>460UB67.1 454 x 190 x 12.7 x 8.5</td>
</tr>
<tr>
<td>Level 22-28</td>
<td>600 x 600 x 12</td>
<td>360UB56.7 352 x 171 x 9.7 x 6.9</td>
</tr>
<tr>
<td>Level 29-34</td>
<td>500 x 500 x 10</td>
<td>360UB44.7 352 x 171 x 9.7 x 6.9</td>
</tr>
<tr>
<td>Level 35-39</td>
<td>450 x 450 x 9</td>
<td>360UB44.7 352 x 171 x 9.7 x 6.9</td>
</tr>
</tbody>
</table>
Figure A.1 39-storey building: (a) 3D view; (b) Plan view
Figure A.2 Elevation view of the 39-storey building

A.2 Slab Thickness

The concrete slab thickness has been determined according to the Australian Concrete Structures Standard AS3600 (2001). According to AS3600, the following formula is used to calculate the slab thickness.

\[
\frac{L_{ef}}{d} \leq \left[ k_3 k_4 \left( \frac{\Delta_{\text{lim}}}{L_{ef}} \right)^{1/3} \right]
\]

Eq. 3.22.a  Cl. 3.22.a

where:

\[
f'_c = 32 \text{ MPa}
\]

\[
E_c = 4730\sqrt{f'_c}
\]

\[
E_c = 4730\sqrt{35}
\]

\[
E_c = 27983 \text{ MPa}
\]

\[
\Delta_{\text{lim}} / L_{ef} = 1/250
\]

Table 2.4.2
\[ \begin{align*}
& \text{• } k_3 = 0.95 \quad \text{Cl. 9.3.4.1} \\
& \text{• } k_4 = 2.40 \\
& \text{• } F_{d, ef} = 3G + 1.5Q \\ 
& \hspace{1cm} = 3 \times 3.5 + 1.5 \times 3 \\
& \hspace{1cm} = 0.015 \text{ N/mm}^2
\end{align*} \]

Therefore, \( \frac{L_{ef}}{d} \leq \left[ 0.95 \times 2.4 \left( \frac{1/250 \times 27983}{0.015} \right)^{1/3} \right] \)

\[ \frac{L_{ef}}{d} \leq 44.55 \]

\[ d = 83 \text{ mm} \]

Assuming concrete cover is 30 mm, total slab thickness \( D_c = 83 + 30 = 113 \approx 120 \text{ mm} \)

### A.3 Design Loads and Load Combinations

#### A.3.1 Design loads

The design loads are determined according to AS/NZS 1170.0 (Standard Australia/Standards NZ, 2002). Four types of loading are considered for the frame analysis:

1. Dead loads (G)
2. Live loads (Q)
3. Wind loads (W)
4. Earthquake loads (F_{eq})

The dead loads have been determined from the summation of the self-weight of structure of 3 kPa and the assumed superimposed dead load of 0.5 kPa. The live loads have been determined for office building as 3 kPa according to AS1170.1 (Standard Australia/Standards NZ, 2002a). The wind load and the earthquake loads are determined according to AS/NZS1170.2 (Standard Australia / Standards NZ, 2002b) and AS1170.4 (Standard Australia, 1993), respectively. The calculation details of the wind load and the earthquake loads are given in sections A.3.3 and A.3.4 of this Appendix.
A.3.2 Design load combinations
The design load combinations have been done according to AS/NZS 1170.0 (Standard Australia/ Standards NZ, 2002). The following load combinations have been considered for the frame analysis.

UDSTL1 = 1.3(DL + SDL)
UDSTL2 = 1.2(DL + SDL) + 1.5 LL
UDSTL3 = 1.2(DL + SDL) + 0.4 LL + 1.0 WL(+X)
UDSTL4 = 1.2(DL + SDL) + 0.4 LL + 1.0 WL(−X)
UDSTL5 = 1.2(DL + SDL) + 0.4 LL + 1.0 WL(+Y)
UDSTL6 = 1.2(DL + SDL) + 0.4 LL + 1.0 WL(−Y)
UDSTL7 = 1.2(DL + SDL) + 0.4 LL
UDSTL8 = 1.2(DL + SDL) + 0.4 LL
UDSTL9 = 1.2(DL + SDL) + 0.4 LL
UDSTL10 = 1.2(DL + SDL) + 0.4 LL
UDSTL11 = 1.2(DL + SDL) + 1.0 WL(+X)
UDSTL12 = 1.2(DL + SDL) + 1.0 WL(−X)
UDSTL13 = 1.2(DL + SDL) + 1.0 WL(+Y)
UDSTL14 = 1.2(DL + SDL) + 1.0 WL(−Y)
UDSTL15 = 1.2(DL + SDL)
UDSTL16 = 1.2(DL + SDL)
UDSTL17 = 1.2(DL + SDL)
UDSTL18 = 1.2(DL + SDL)
UDSTL19 = 0.9(DL + SDL) + 1.0 WL(+X)
UDSTL20 = 0.9(DL + SDL) + 1.0 WL(−X)
UDSTL21 = 0.9(DL + SDL) + 1.0 WL(+Y)
UDSTL22 = 0.9(DL + SDL) + 1.0 WL(−Y)
UDSTL23 = 0.9(DL + SDL)
UDSTL24 = 0.9(DL + SDL)
UDSTL25 = 0.9(DL + SDL)
UDSTL26 = 0.9(DL + SDL)
UDSTL27 = 1.0(DL + SDL) + 0.4 LL + 1.0 EQ(+X)
UDSTL28 = 1.0(DL + SDL) + 0.4 LL + 1.0 EQ(−X)
UDSTL29 = 1.0(DL + SDL) + 0.4 LL + 1.0 EQ(+Y)
UDSTL30 = 1.0(DL + SDL) + 0.4 LL + 1.0 EQ(−Y)
UDSTL31 = 1.0(DL + SDL) + 1.0 EQ(+X)
UDSTL32 = 1.0(DL + SDL) + 1.0 EQ(−X)
UDSTL33 = 1.0(DL + SDL) + 1.0 EQ(+Y)
UDSTL34 = 1.0(DL + SDL) + 1.0 EQ(−Y)
UDSTL35 = 1.0(DL + SDL) + 0.4 LL + 1.0 RS(+X)
UDSTL36 = 1.0(DL + SDL) + 0.4 LL + 1.0 RS(+Y)
UDSTL37 = 1.0(DL + SDL) + 1.0 RS(+X)
UDSTL38 = 1.0(DL + SDL) + 1.0 RS(+Y)
UDSTL39 = 1.0(DL + SDL)
UDSTL40 = 1.0(DL + SDL) + 1.0 LL
A.3.3 Wind loads

The calculation of earthquake loads has been done according to AS AS/NZS 1170.2 (Standard Australia/ Standards NZ, 2002b). The calculation details are given in the following sub-sections.

A.3.3.1 Site wind speed

\[ V_{sit, \beta} = V_R M_d M_{z, cat} M_s M_t \]

where,

\[ V_R = \text{regional 3 seconds gust wind speed, in m/s} \]

\[ = 48 \text{ m/s for most regions in Australia (1 to 7), Cl.3.2, Table 3.1} \]

\[ (R = \text{average recurrence interval taken as 2000, importance level = 2, design working life = 50 years}) \]

\[ M_d = \text{wind directional multiplier} \]

\[ = 1.0 \text{ (assumed to act from any cardinal direction, Cl.3.3 Table 3.2)} \]

\[ M_{z, cat} = \text{terrain/height multiplier} \]

\[ = 1.206 \text{ (assumed buildings height is 146.1m, and terrain category 3, areas of suburban housing, Cl.4.2)} \]

\[ M_s = \text{shielding multiplier} \]

\[ = 1.0 \text{ (effects of shielding are ignored, Cl.4.3)} \]

\[ M_t = \text{topographic multiplier} \]

\[ = 1.16 \text{ (assumed as the average value for upwind slope, Table 4.4)} \]

\[ V_{sit, \beta} = 48 \times 1 \times 1.206 \times 1 \times 1.32 \text{ m/s} \]

\[ = 67.15 \text{ m/s} \approx 67.2 \text{ m/s} \]

A.3.3.2 Design wind speed

\[ V_{des, \theta} = 67.2 \text{ m/s} \text{ (assumed to be equal to } V_{sit, \beta}, \text{ and for ultimate limit states design, it shall not be less than 30 m/s, Cl.2.3)} \]

A.3.4 Earthquake loads

The calculation of earthquake loads has been done according to AS 1170.4 (Standard Australia, 2007). The calculation details are given in the following sub-sections.
A.3.2.1 Engineering assumptions for earthquake loads

Engineering assumptions are considered in the frame analysis for earthquake loads.

- **Building type**: High-rise building
- **Number of levels**: 39-storey high
- **Slab thickness**: 120 mm
- **Soil type**: Rock (Class Bc)
- **Building location**: Newcastle (One of the High risk Area)
- **Live load**: 3.0 kPa for office building
- **Superimposed dead load**: 1.0 kPa
- **Self-weight**: 3.0 kPa
- **Return period**: 1/800 years

A.3.2.2 Earthquake design category

- **Importance level**: 2
- **Annual probability of exceedance**: 1/800
- **Probability factor**: $k_p = 1.25$ (for 1/800 year earthquake)
- **Hazard factor**: $Z = 0.11$ for Newcastle location
- **Soil factor**: $k_pZ = 0.14$
- **Site subsoil class**: Bc
- **Total height of the structure above the structural base**: $h_n = 146.1$ m
- **Earthquake design category (EDC)**: III

Based on the above parameters, linear static analysis (EQ), response spectrum analysis (RS) were conducted in frame analysis using SAP2000; in which non-linear static analysis (P-DELTA), modal analysis (EIGENMODE) were also considered.

A.4 Connections Properties Used in Semi-rigid Frame Analysis

Rotational spring stiffness is used to define connection property for semi-rigid frame analysis. Rotational spring stiffness is introduced as ‘Partial Fixity Spring’ in SAP2000 for semi-rigid connections. Rotational spring stiffness values are to be defined at both ends of a beam. This stiffness value is calculated based on the steel beam size and CFST column sizes and type of blind-bolted endplate connections. The parameters used in the calculation of the connections’ spring stiffness are given in Table A.1. Six
different spring stiffness values are calculated based on connection configurations as shown in Table A.2.

Table A.2 Parameters used in the calculation of the rotational stiffness of connections in different levels of the 39-storey building

<table>
<thead>
<tr>
<th>Component</th>
<th>Level 1 to 7</th>
<th>Level 8 to 14</th>
<th>Level 15 to 21</th>
<th>Level 22 to 28</th>
<th>Level 28 to 34</th>
<th>Level 35 to 39</th>
</tr>
</thead>
<tbody>
<tr>
<td>CFST column</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Width (mm)</td>
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<td>700</td>
<td>600</td>
<td>600</td>
<td>500</td>
<td>450</td>
</tr>
<tr>
<td>Thickness (mm)</td>
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<td>14</td>
<td>12</td>
<td>12</td>
<td>10</td>
<td>9</td>
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<tr>
<td>Beam</td>
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<tr>
<td>Flange width</td>
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<td>190</td>
<td>190</td>
<td>171</td>
<td>171</td>
<td>171</td>
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<tr>
<td>Flange thickness</td>
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<td>12.7</td>
<td>12.7</td>
<td>9.7</td>
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<tr>
<td>Depth of web</td>
<td>454</td>
<td>454</td>
<td>454</td>
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<td>Depth</td>
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<td>Bolt gauge (g)</td>
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<td>Bolt pitch (p)</td>
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<td>165</td>
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<td>Bolt edge distance (e)</td>
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<td>Bolt</td>
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<td>Bolt hole diameter</td>
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<tr>
<td>Total number of shear stud</td>
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</tr>
<tr>
<td>Spacing from column face</td>
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<td>Other parameters</td>
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<tr>
<td>$D_c$</td>
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<tr>
<td>$H_b$</td>
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<td>454</td>
<td>454</td>
<td>352</td>
<td>352</td>
<td>352</td>
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<td>$D_b$</td>
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<td>382.6</td>
<td>282.2</td>
<td>282.2</td>
<td>282.2</td>
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<tr>
<td>$D_r$</td>
<td>301.6</td>
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<td>201.2</td>
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<tr>
<td>Component stiffness (kN/mm)</td>
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<td>615.0</td>
<td>615.0</td>
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<td>615.0</td>
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<tr>
<td>$K_c$</td>
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<td>3429.3</td>
<td>3179.0</td>
<td>3060.74</td>
<td>2986.6</td>
</tr>
</tbody>
</table>
Table A.3 Rotational spring stiffness (Partial Fixity Spring) for semi-rigid connections (calculated based on the formula proposed by Ahmed & Nethercot (1997))

<table>
<thead>
<tr>
<th>Level No</th>
<th>CFST column size in mm</th>
<th>Steel beam size</th>
<th>Rotational spring stiffness (kN-m/rad)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Beam type</td>
<td>Beam section (d×b×t_f×t_w) in mm</td>
</tr>
<tr>
<td>Level 1-7</td>
<td>800×800×16</td>
<td>460UB67.1</td>
<td>454×190×12.7×8.5</td>
</tr>
<tr>
<td>Level 8-14</td>
<td>700×700×14</td>
<td>460UB67.1</td>
<td>454×190×12.7×8.5</td>
</tr>
<tr>
<td>Level 15-21</td>
<td>600×600×14</td>
<td>460UB67.1</td>
<td>454×190×12.7×8.5</td>
</tr>
<tr>
<td>Level 22-28</td>
<td>600×600×14</td>
<td>360UB56.7</td>
<td>352×171×9.7×6.9</td>
</tr>
<tr>
<td>Level 29-34</td>
<td>500×500×10</td>
<td>360UB44.7</td>
<td>352×171×9.7×6.9</td>
</tr>
<tr>
<td>Level 35-39</td>
<td>450×450×10</td>
<td>360UB44.7</td>
<td>352×171×9.7×6.9</td>
</tr>
</tbody>
</table>

A.5 Frame Analysis Results

Table A.4 Maximum sagging and hogging moment at both ends of typical beams for frame 10-10 (long direction)

<table>
<thead>
<tr>
<th>Beam number</th>
<th>Frame type</th>
<th>Moment at both ends of beam (kN-m)</th>
<th>I-End (0.5 m)</th>
<th>J-End (8 m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Sagging moment (+ve)</td>
<td>Hogging moment (-ve)</td>
</tr>
<tr>
<td>Beam-4156</td>
<td>Rigid</td>
<td>184.03 (LC14)</td>
<td>-181.94 (LC5)</td>
<td>10.96 (LC13)</td>
</tr>
<tr>
<td></td>
<td>Semi-rigid</td>
<td>158.24 (LC14)</td>
<td>-166.07 (LC5)</td>
<td>7.73 (LC13)</td>
</tr>
<tr>
<td>Beam-3766</td>
<td>Rigid</td>
<td>192.70 (LC14)</td>
<td>-197.48 (LC5)</td>
<td>26.28 (LC13)</td>
</tr>
<tr>
<td></td>
<td>Semi-rigid</td>
<td>160.79 (LC14)</td>
<td>-178.67 (LC5)</td>
<td>20.30 (LC13)</td>
</tr>
<tr>
<td>Beam-3376</td>
<td>Rigid</td>
<td>168.18 (LC14)</td>
<td>-197.22 (LC5)</td>
<td>22.97 (LC13)</td>
</tr>
<tr>
<td></td>
<td>Semi-rigid</td>
<td>142.42 (LC14)</td>
<td>-179.80 (LC5)</td>
<td>17.98 (LC13)</td>
</tr>
<tr>
<td>Beam-2986</td>
<td>Rigid</td>
<td>157.21 (LC14)</td>
<td>-200.54 (LC5)</td>
<td>22.80 (LC13)</td>
</tr>
<tr>
<td></td>
<td>Semi-rigid</td>
<td>132.90 (LC14)</td>
<td>-182.55 (LC5)</td>
<td>17.68 (LC13)</td>
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<tr>
<td>Beam-2596</td>
<td>Rigid</td>
<td>147.60 (LC14)</td>
<td>-205.21 (LC5)</td>
<td>24.26 (LC13)</td>
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<tr>
<td></td>
<td>Semi-rigid</td>
<td>124.56 (LC14)</td>
<td>-186.52 (LC5)</td>
<td>18.91 (LC13)</td>
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<tr>
<td>Beam-2206</td>
<td>Rigid</td>
<td>139.82 (LC14)</td>
<td>-211.39 (LC5)</td>
<td>27.69 (LC13)</td>
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<tr>
<td></td>
<td>Semi-rigid</td>
<td>117.92 (LC14)</td>
<td>-191.86 (LC5)</td>
<td>21.96 (LC13)</td>
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<tr>
<td>Beam-1808</td>
<td>Rigid</td>
<td>133.29 (LC14)</td>
<td>-217.43 (LC5)</td>
<td>30.71 (LC13)</td>
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<tr>
<td></td>
<td>Semi-rigid</td>
<td>112.03 (LC14)</td>
<td>-196.98 (LC5)</td>
<td>24.55 (LC13)</td>
</tr>
</tbody>
</table>
Table A.5 Maximum sagging and hogging moments at both ends of typical beams for frame L-L (short direction)

<table>
<thead>
<tr>
<th>Beam number</th>
<th>Frame type</th>
<th>Moment at both ends of beam (kN-m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Sagging moment (+ve)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>I-End (0.5 m)</td>
</tr>
<tr>
<td>Beam-146</td>
<td>Rigid</td>
<td>304.14 (LC3)</td>
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<td>Semi-rigid</td>
<td>259.24 (LC3)</td>
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<td>Beam-536</td>
<td>Rigid</td>
<td>307.0 (LC3)</td>
</tr>
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<td></td>
<td>Semi-rigid</td>
<td>261.78 (LC3)</td>
</tr>
<tr>
<td>Beam-1077</td>
<td>Rigid</td>
<td>305.01 (LC3)</td>
</tr>
<tr>
<td></td>
<td>Semi-rigid</td>
<td>260.18 (LC3)</td>
</tr>
<tr>
<td>Beam-1467</td>
<td>Rigid</td>
<td>298.57 (LC3)</td>
</tr>
<tr>
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<td>Semi-rigid</td>
<td>270.27 (LC3)</td>
</tr>
<tr>
<td>Beam-1933</td>
<td>Rigid</td>
<td>298.08 (LC3)</td>
</tr>
<tr>
<td></td>
<td>Semi-rigid</td>
<td>269.70 (LC3)</td>
</tr>
<tr>
<td>Beam-25766</td>
<td>Rigid</td>
<td>301.86 (LC3)</td>
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<td>Semi-rigid</td>
<td>257.35 (LC3)</td>
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<td>Beam-26172</td>
<td>Rigid</td>
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</tr>
<tr>
<td></td>
<td>Semi-rigid</td>
<td>253.08 (LC3)</td>
</tr>
</tbody>
</table>
B.1 Design of Concrete-Filled Steel Tubular Columns

(All calculations are done according to the 'Eurocode 4: BS EN 1994-1-1: 2005')

Basic design conditions:

a) Geometry:
   - Height of CFST columns ($H$): 2200 mm
   - External diameter for circular sections/ overall width for square sections ($D$): 360 mm
   - Thickness of the stainless steel tube for square columns ($t_s$): 6 mm
   - Thickness of the stainless/ carbon steel tube for circular columns ($t_s$): 6 mm

b) Boundary conditions:
   - Both ends of the CFST column are considered as hinged support. Bottom and top ends are restrained against horizontal translation and rotation.

c) Materials:
   - Core concrete characteristic cylinder compressive strength ($f'_c$): 35 MPa
   - $E_c = 4730\sqrt{f'_c} = 4730 \times \sqrt{30} = 27983$ MPa
   - Yield strength of stainless steel (1.4301) square tube ($f_y$): 339 MPa; $E_s = 200985$ MPa
   - Yield strength of stainless steel (1.4301) circular tube ($f_y$): 367 MPa; $E_s = 200663$ MPa
   - Yield strength of carbon steel (S355) square tube ($f_y$): 379 MPa; $E_s = 206338$ MPa

B.1.1 Square concrete-filled stainless steel tubular column

B.1.1.1 Flexural stiffness of square CFST column

\[
(EI)_{column} = E_s I_s + 0.6E_c I_c
\]

\[
= 200985 \times \left(\frac{360 \times 360^3}{12} - \frac{(360 - 2 \times 6) \times (360 - 2 \times 6)^3}{12}\right)
+ 0.6 \times 27983 \times \frac{(360 - 2 \times 6) \times (360 - 2 \times 6)^3}{12}

= 5.619 \times 10^{13} \text{ N-mm}^2
\]
\[
\frac{(EI)_{column}}{L} = \frac{5.619 \times 10^{13}}{2200} = 2.554 \times 10^{10} \text{ N mm}
\]

### B.1.1.2 Plastic cross-sectional resistance to compression

\[
N_{pl,R} = A_{sfy} + A_{cf'c} = 8496 \times 339 + 121104 \times 35 = 7118784 \text{ N} = 7118.78 \text{ kN}
\]

\[
A_c = (360 - 2 \times 6) \times (360 - 2 \times 6) = 121104 \text{ mm}^2
\]

\[
A_s = 360 \times 360 - 121104 = 8496 \text{ mm}^2
\]

### B.1.3 Resistance of column under axial compression considering slenderness reduction

\[
N_{cu} = \chi \cdot N_{pl,R} = 0.88 \times 7118.78 = 6264.53 \text{ kN}
\]

Relative slenderness \(\bar{\lambda} = \sqrt{\frac{N_{pl,R}}{N_{cr}}} = \sqrt{\frac{7118.78}{114590.81}} = 0.25 > 0.2\]

\[
N_{cr} = \frac{\pi^2 \times (EI)_{column}}{l^2} = \frac{\pi^2 \times 5.619 \times 10^{13}}{2200^2} = 114590.81 \text{ kN}
\]

\(l = H = 2200 \text{ mm}\)

\[
\therefore \bar{\lambda} = 0.25 > 0.2
\]

\[
\therefore \chi = \frac{1}{\phi + \sqrt{\phi^2 + \bar{\lambda}^2}} = 0.88
\]

\[
\phi = 0.5 \times (1 + 0.21 \times (\bar{\lambda} - 0.2) + \bar{\lambda}^2) = 0.54
\]

### B.1.4 Plastic bending resistance

\[
M_{pl,R} = W_{psf'y} + 0.5W_{pcf'c} - W_{psnf'y} - 0.5W_{pcnf'}
\]

\[
= 1127952 \times 339 + 0.5 \times 10536048 \times 35 - 130591.94 \times 339 - 0.5 \times 3787166.52 \times 35 = 456.21 \text{ kN} \cdot \text{m}
\]

\[
W_{pc} = \frac{(D - 2ts) \cdot (D - 2ts)^2}{4} = \frac{(360 - 2 \times 6) \times (360 - 2 \times 6)^2}{4} = 10536048 \text{ mm}^3
\]

\[
W_{ps} = \frac{D \cdot D^2}{4} - W_{pc} = \frac{360 \times 360^2}{4} - 10536048 = 1127952 \text{ mm}^3
\]

\[
W_{pcn} = (D - 2ts) \cdot h_n^2 = (360 - 2 \times 6) \times 104.32^2 = 3787166.52 \text{ mm}^3
\]

\[
W_{psn} = D \cdot h_n^2 - W_{pcn} = 360 \times 104.32^2 - 3787166.52 = 130591.94 \text{ mm}^3
\]

\[
h_n = \frac{A_c \cdot f_c'}{2D \cdot f_c' + 4ts \cdot (2f_y - f_c')} = \frac{121104 \times 35}{2 \times 360 \times 35 + 4 \times 6 \times (2 \times 339 - 35)} = 104.32 \text{ mm}
\]

\[
A_c = (360 - 2 \times 7) \times (360 - 2 \times 7) = 119716 \text{ mm}^2
\]
B.1.2 Circular concrete-filled stainless steel tubular column

B.1.2.1 Flexural stiffness of CFST columns

\[ (EI)_{\text{column}} = E_s I_s + 0.6 E_c I_c \]

\[ = 200663 \times \left( \frac{\pi \times 180^4}{4} - \frac{\pi \times 174^4}{4} \right) + 0.60 \times 27983 \times \frac{\pi \times 174^4}{4} \]

\[ = 3.307 \times 10^{13} \text{N-mm}^2 \]

\[ \frac{(EI)_{\text{column}}}{L} = \frac{3.307 \times 10^{13}}{2200} = 1.503 \times 10^{10} \text{ N-mm} \]

B.1.2.2 Plastic cross-sectional resistance to compression

\[ A_c = \pi (360 - 2 \times 6)^2 / 4 = 95114.86 \text{ mm}^2 \]

\[ A_s = \frac{\pi \times 360^2}{4} - 95114.86 = 6672.74 \text{ mm}^2 \]

\[ \eta_a = 0.25(3 + 2\bar{\lambda}) = 0.895, \text{ and less than 1, therefore, } \eta_a = 0.895 \]

\[ \eta_c = 4.9 - 18.5\bar{\lambda} + 17\bar{\lambda}^2 = 0.96, \text{ and greater than 0, therefore, } \eta_c = 0.96 \]

\[ \bar{\lambda} = \sqrt{\frac{A_s f_y + A_c f'_c}{N_{cr}}} = \sqrt{\frac{6672.74 \times 367 + 95114.86 \times 35}{67430906.56}} = 0.29 \]

\[ N_{cr} = \frac{\pi^2 \times (EI)_{\text{column}}}{l^2} = \frac{\pi^2 \times 3.307 \times 10^{13}}{2200^2} = 67430906.56 N \]

\[ l = H = 2200 \text{ mm} \]

\[ \therefore \bar{\lambda} = 0.29 < 0.5 \text{ and } e/D = 0 < 0.1 \]

\[ \therefore N_{plR} = \eta_a A_s f_y + A_c f'_c \left( 1 + \eta_c \frac{t_{jf} f_y}{f'_c} \right) \]

\[ = 0.895 \times 6672.74 \times 367 + 95114.86 \times 35 \times (1 + 0.96 \times \frac{6 \times 367}{360 \times 35}) \]

\[ = 6079296.1 N = 6079.29 kN \]

B.1.2.3 Resistance of column under axial compression considering slenderness reduction

\[ N_{cu} = \chi \cdot N_{plR} = 0.85 \times 6079.29 = 5167.40 kN \]

\[ \therefore \bar{\lambda} = 0.29 > 0.2 \]

\[ \therefore \chi = \frac{1}{\phi + \sqrt{\phi^2 + \bar{\lambda}^2}} = 0.85 \]

\[ \phi = 0.5 \times (1 + 0.21 \times (\bar{\lambda} - 0.2) + \bar{\lambda}^2) = 0.55 \]
B.1.2.4 Plastic bending resistance

\[ M_{pl,R} = W_{psf_y} + 0.5W_{pcf'_c} - W_{psnf_y} - 0.5W_{pcnf'_c} \]
\[ = 751968 \times 367 + 0.5 \times 7024032 \times 35 - 75480.916 \times 367 - 0.5 \times 2188946.48 \times 35 = 332.88 \text{ kN} \cdot \text{m} \]

\[ W_{pc} = \frac{(D - 2t_s)^3}{4} - \frac{2r^3}{3} = \frac{(360 - 2 \times 6)^3}{4} - \frac{2 \times 174^3}{3} = 7024032 \text{ mm}^3 \]

\[ W_{ps} = \frac{D^3}{4} - \frac{2(r + t_s)^3}{3} - W_{pc} = \frac{360^3}{4} - \frac{2(174 + 6)^3}{3} - 7024032 = 751968 \text{ mm}^3 \]

\[ W_{pcn} = (D - 2t_s) \cdot h_n^2 = (360 - 2 \times 6) \times 79.31^2 = 2188946.48 \text{ mm}^3 \]

\[ W_{psn} = D \cdot h_n^2 - W_{pcn} = 360 \times 79.31^2 - 2188946.48 = 75480.916 \text{ mm}^3 \]

\[ r = \frac{D}{2} - t_s = \frac{360}{2} - 6 = 174 \text{ mm} \]

\[ h_n = \frac{A_c \cdot f'_c}{2D \cdot f'_c + 4t_s \cdot (2f_y - f'_c)} = \frac{95114.86 \times 35}{2 \times 360 \times 35 + 4 \times 6 \times (2 \times 367 - 35)} = 79.31 \text{ mm} \]

\[ A_c = \pi (360 - 2 \times 6)^2 / 4 = 95114.86 \text{ mm}^2 \]

B.1.3 Circular concrete-filled carbon steel tubular column

B.1.3.1 Flexural stiffness of CFST columns

\[ (EI)_{\text{column}} = E_s I_s + 0.6 E_c I_c \]
\[ = 206338 \times \left( \frac{\pi \times 180^4}{4} - \frac{\pi \times 174^4}{4} \right) + 0.60 \times 27983 \times \frac{\pi \times 174^4}{4} \]
\[ = 3.37 \times 10^{13} \text{ N} \cdot \text{mm}^2 \]

\[ \frac{(EI)_{\text{column}}}{L} = \frac{3.37 \times 10^{13}}{2200} = 1.53 \times 10^{10} \text{ N} \cdot \text{mm} \]

B.1.3.2 Plastic cross-sectional resistance to compression

\[ A_c = \pi (360 - 2 \times 6)^2 / 4 = 95114.86 \text{ mm}^2 \]

\[ A_s = \frac{\pi \times 360^2}{4} - 95114.86 = 6672.74 \text{ mm}^2 \]

\[ \eta_a = 0.25 (3 + 2\lambda) = 0.895, \text{ and less than 1, therefore, } \eta_a = 0.895 \]

\[ \eta_c = 4.9 - 18.5\lambda + 17\lambda^2 = 0.96, \text{ and greater than 0, therefore, } \eta_c = 0.96 \]

\[ \lambda = \sqrt{\frac{A_{sf_y} + A_{cf'_c}}{N_{cr}}} = \sqrt{\frac{(6672.74 \times 379 + 95114.86 \times 35)}{68640852.35}} = 0.29 \]
\[ N_{cr} = \frac{\pi^2 \times (EI)_{column}}{l^2} = \frac{\pi^2 \times 3.37 \times 10^{13}}{2200^2} = 68640852.35 \text{ N} \]

\( l = H = 2200 \text{ mm} \)

\[ \bar{\lambda} = 0.29 < 0.5 \text{ and } e/D = 0 < 0.1 \]

\( N_{plR} = \eta_a A_s f_y + A_c f'_c \left( 1 + \eta_c \frac{t_s f_y}{D f'_c} \right) \)

\[ = 0.895 \times 6672.74 \times 379 + 95114.86 \times 35 \times \left( 1 + 0.96 \times \frac{6 \times 379}{360 \times 35} \right) \]

\[ = 6169223.38 \text{ N} = 6169.22 \text{ kN} \]

**B.1.3.3 Resistance of columns under axial compression considering slenderness reduction**

\( N_{cu} = \chi \cdot N_{plR} = 0.85 \times 6169.22 = 5243.84 \text{ kN} \)

\[ \therefore \bar{\lambda} = 0.29 > 0.2 \]

\[ \therefore \chi = \frac{1}{\phi + \sqrt{\phi^2 + \bar{\lambda}^2}} = 0.85 \]

\[ \phi = 0.5 \times (1 + 0.21 \times (\bar{\lambda} - 0.2) + \bar{\lambda}^2) = 0.55 \]

**B.1.3.4 Plastic bending resistance**

\( M_{plR} = W_{ps} f_y + 0.5 W_{pc} f'_c - W_{psn} f_y - 0.5 W_{pcn} f'_c \)

\[ = 751968 \times 379 + 0.5 \times 7024032 \times 35 - 73439.194 \times 379 - 0.5 \times 2129736.65 \times 35 = 342.81 \text{ kN} \cdot \text{m} \]

\( W_{pc} = \frac{(D - 2 t_s)^3}{4} - 2 \frac{r^3}{3} = \frac{(360 - 2 \times 6)^3}{4} - 2 \frac{174^3}{3} = 7024032 \text{ mm}^3 \)

\( W_{ps} = \frac{D^3}{4} - 2 \left( r + t_s \right)^3 \cdot \frac{3}{3} - W_{pc} = \frac{360^3}{4} - 2 \left( 174 + 6 \right)^3 \cdot \frac{3}{3} - 7024032 = 751968 \text{ mm}^3 \)

\( W_{psn} = (D - 2 t_s) \cdot h_n^2 = (360 - 2 \times 6) \times 78.23^2 = 2129736.65 \text{ mm}^3 \)

\( W_{pcn} = D \cdot h_n^2 - W_{pc} = 360 \times 78.23^2 - 2129736.65 = 73439.194 \text{ mm}^3 \)

\( r = \frac{D}{2} - t_s = \frac{360}{2} - 6 = 174 \text{ mm} \)

\( h_n = \frac{A_c \cdot f'_c}{2D \cdot f'_c + 4t_s \cdot (2 f_y - f'_c)} = \frac{95114.86 \times 35}{2 \times 360 \times 35 + 4 \times 6 \times (2 \times 379 - 35)} = 78.23 \text{ mm} \)

\( A_c = \pi (360 - 2 \times 6)^2 / 4 = 95114.86 \text{ mm}^2 \)
B.2 Design of Composite Beam

**Reference standard:**

**Basic design conditions:**
Length of composite beam \((L)\): 1000 mm
Beam type: 310 UB 40.4 \((h = 304 \text{ mm}; \ b_f = 165 \text{ mm}; \ t_w = 6.1 \text{ mm}; \ t_f = 10.2 \text{ mm})\)
Slab type: Composite slab with 1.0 BMT BONDEK profile sheet \((\text{Slab depth } D_{cs} = 120 \text{ mm}; \ \text{Slab width } b_{\text{slab}} = 900 \text{ mm}; \ \text{Rib height of profiled sheeting } h_r = 54 \text{ mm})\)

Reinforcement: Longitudinal bar \((\text{N-Grade, } \phi = 12 \text{ mm})\);
Distributing bar \((\text{N-Grade, } \phi = 10 \text{ mm})\)
Shear connectors: Stud EN ISO 13918 - SD 19×100
Concrete characteristic cylinder compressive strength \((f'_c)\): 35 MPa
\[ E_c = 4730 \cdot \sqrt{f'_c} = 4730 \cdot \sqrt{35} = 27983 \text{ MPa} \]
Yield strength of steel beam: \(f_{yf} = 352 \text{ MPa} ; \ f_{yw} = 370 \text{ MPa} ; \ E_s = 203765 \text{ MPa} \)
Yield strength of steel profile sheeting: \(f_{yss} = 615 \text{ MPa} ; \ E_{ss} = 216122 \text{ MPa} \)
Yield strength of longitudinal and distributing bar: \(f_{yr} = 538 \text{ MPa} ; \ E_{sr} = 200371 \text{ MPa} \)
Boundary conditions: Cantilever beam

![Figure B.1 Bondek steel profile sheet (Unit: mm)](image-url)
B.2.1 Resistance moment of composite beam to hogging bending

Composite beam is designed with full shear connection.

(a) Effective width of concrete compression flange:

\[ b_{cf} = 900 \text{ mm} \] is used in all calculations.

(b) Resistance moment of composite beam to hogging bending:

Force details in the composite beam hogging moment cross-section are shown in Fig. B.2.

![Figure B.2](image)

Figure B. 2 Force details in the composite beam hogging moment cross-section

Cross-section area of each steel reinforced bar, \( A_r = \pi r^2 = \pi \times 6^2 = 113.10 \text{ mm}^2 \)

Cross-section area of steel profile sheet, \( A_{ss} = 1678 \times 0.9 = 1510.2 \text{ mm}^2 \)

Assumed that the plastic neutral axis would be at a distance of \( x \) from the top flange of beam.

Tension =Compression

\[ F_r + F_{ss} + F_{f1(x)} = F_{f1(t_f-x)} + F_w + F_{f2} \]

where,

\[ F_r = 6A_r f_{yr} = 6 \times 113.10 \times 538 = 35965.8 \text{ N} \]

\[ F_{ss} = A_{ss} f_{ys} = 1510.2 \times 615 = 928773 \text{ N} \]

\[ F_{f1} = F_{f2} = A_f f_{yf} = (b_f t_f) f_{yf} = 165 \times 10.2 \times 352 = 592416 \text{ N} \]

\[ F_{f1(x)} = (b_f x) f_{yf} = 165 \times x \times 352 = 58080x \text{ N} \]

\[ F_{f1(t_f-x)} = (b_f (t_f - x)) f_{yf} = 165 \times (10.2 - x) \times 352 = 592416 - 58080x \]

\[ F_w = A_w f_{yw} = (h_w t_w) f_{yw} = 304 \times 6.1 \times 370 = 686128 \text{ N} \]
Therefore,
\[ 35965.8 + 928773 + 58080x = 592416 - 58080x + 686128 + 592416 \]
or, \[ 58080x + 58080x = 592416 + 686128 + 592416 - 35965.8 - 928773 \]
or, \[ x = 7.80 \]
Assumption is correct.

\[ F_{st} = (A_{f1} + A_{f2})f_{fy} + A_wf_{yw} \]
\[ = (2 \times b_f \times t_f)f_{fy} + h_w \times t_w f_{yw} \]
\[ = (2 \times (165 - 6.1) \times 10.2) \times 340 + 304 \times 6.1 \times 360 \]
\[ = 1769714.4 \, N \]

Distances between forces and plastic neutral axis:
\[ d_r = 7.80 + 120 - 26 = 101.8 \, mm \]
\[ d_{ss} = 7.80 + 14.3 = 22.1 \, mm \]
\[ d_{st} = \frac{304}{2} - 7.80 = 144.2 \, mm \]

Hogging moment capacity of composite beam:
\[ M_{beam \, hogging} = F_r d_r + F_{ss} d_{ss} + F_{st} d_{st} \]
\[ = 35965.8 \times 101.8 + 928773 \times 22.1 + 1769714.4 \times 144.2 \]
\[ = 279.38 \, kN \cdot m \]

**B.2.2 Flexural stiffness of composite beam**

Based on the method proposed by Chinese code for design of steel structures (GB 50017-2003, 2003), ignore the contribution of concrete for calculating flexural stiffness of composite beam to hogging bending.

\[ (EI)_b = E_s I_{eq} = 203765 \times 1.274 \times 10^8 = 2.96 \times 10^{13} \, \text{MPa} \cdot \text{mm}^4 \]

\[ I_{eq} = I_s + A_s x_e^2 + A_{rt} \cdot y_2^2 + A_{ss} \cdot y_3^2 \]
\[ = 86.4 \times 10^6 + 5095.96 \times (57.39)^2 + 678.58 \times (188.61)^2 + 1510.2 \times (108.91)^2 \]
\[ = 1.452 \times 10^8 \, \text{mm}^4 \]

\[ x_e = \frac{A_{rt} (y_s + y_r) + A_{ss} (y_s + y_{ss})}{A_s + A_{rt} + A_{ss}} \]
\[ = \frac{678.58 \times (152 + 94) + 1510.2 \times (152 + 14.3)}{5095.96 + 678.58 + 1510.2} = 57.39 \, mm \]
\[ y_2 = y_s + y_r - x_e = 152 + 94 - 57.39 = 188.61 \text{ mm} \]
\[ y_3 = y_s + y_{ss} - x_e = 152 + 14.3 - 57.39 = 108.91 \text{ mm} \]
\[ A_s = 2 \times (b_f - t_w) \times t_f + h_w \times t_w = 2 \times (165 - 6.1) \times 10.2 + 304 \times 6.1 = 5095.96 \text{ mm}^2 \]
\[ y_s = \frac{304}{2} = 152 \text{ mm} \]
\[ A_{rt} = 6A_r = 678.58 \text{ mm}^2 \]
\[ y_r = 120 - 26 = 94 \text{ mm} \]
\[ A_{ss} = 1678 \times 1.2 = 2013.6 \]
\[ y_{ss} = \frac{(54.83 \times 2 \times 6) \times \frac{54}{2} + (32 \times 6) \times 54}{(1200 - 13 \times 6) + (54.83 \times 2 \times 6) + (32 \times 6)} = 14.26 = 14.30 \text{ mm} \]

**B.2.3 Shear connector design for full shear connection**

Total tensile forces on slab for the hogging moment section,
\[ F_r = 4A_r f_{yr} = 4 \times 113.10 \times 538 = 243391.2 \text{ N} \]
Diameter of shear connectors, \(d_{sc} = 19 \text{ mm}\)
\[ A_{sc} = \pi r^2 = \pi \times (9.5)^2 = 283.52 \text{ mm}^2 \]
\[ f_{y,sc} = 375 \text{ MPa} \]
\[ f_{usc} = 517 \text{ MPa} \]

The nominal shear capacity \(f_{vsc}\) of welded headed studs shall be as the minimum value from the following two values:
\[ f_{vsc} = 0.63 \frac{d_{sc}^2 f_{usc}}{2} = 0.63 \times 19^2 \times 375 = 85286.25 \text{ N}; \text{ or} \]
\[ f_{vsc} = 0.31 \frac{d_{sc}^2 \sqrt{f_{c}'} E_c}{2} = 0.31 \times 19^2 \times \sqrt{35 \times 27983} = 110751.6 \text{ N} \]
Therefore, nominal shear capacity, \(f_{vsc} = 110751.6 \text{ N} \)

Minimum number of shear connectors = \(\frac{243391.2}{85286.25} = 2.85 = 3\)

Therefore, totally 8 shear connectors with a spacing of 190 mm are configured on the hogging region of a beam. The first shear connector is placed at 75 mm away from the face of the steel tube of the CFST column.
B.3 Design of Blind-Bolted Beam-Column Connection


Basic design conditions:
Endplate thickness \( (t_p) \): 10 mm
Blind bolt: HB20-1, Lindapter grade M20 Class 8.8 Hollo-bolt
Yield strength of endplate: \( f_{yp} = 388 \text{ MPa}; f_{up} = 506 \text{ MPa}; E_s = 206298 \text{ MPa} \)
Yield strength of blind bolt: \( f_{yb} = 890 \text{ MPa}; f_{ub} = 953 \text{ MPa}; E_s = 218871 \text{ MPa} \)

Design principle:
Semi-rigid connection, moment capacity of connection should be lower than that of composite beam, and the shear resistance of joint should be controlled by the beam.

B.3.1 Resistance moment of blind-bolted endplate connection to hogging bending
Cross-section area of each steel reinforced bar, \( A_r = \pi r^2 = \pi \times 6^2 = 113.10 \text{ mm}^2 \)
Cross-section area of steel profile sheet, \( A_{ss} = 1678 \times (0.9 - 0.36) = 906.12 \text{ mm}^2 \)
Cross-section area of each blind bolt, \( A_b = \pi \times 10^2 = 314 \text{ mm}^2 \)
Dimension and force details in the endplate cross-section are shown in Figure B.3.
Yield strain of steel reinforcement, \( \varepsilon_r = \frac{f_{yr}}{E_r} = \frac{538}{200371} = 0.002685 \cong 0.00269 \)

![Figure B.3 Dimension and force details at the endplate cross-section](image)

\[ \text{Cross-section area of each steel reinforced bar, } A_r = \pi r^2 = \pi \times 6^2 = 113.10 \text{ mm}^2 \]
\[ \text{Cross-section area of steel profile sheet, } A_{ss} = 1678 \times (0.9 - 0.36) = 906.12 \text{ mm}^2 \]
\[ \text{Cross-section area of each blind bolt, } A_b = \pi \times 10^2 = 314 \text{ mm}^2 \]
\[ \text{Yield strain of steel reinforcement, } \varepsilon_r = \frac{f_{yr}}{E_r} = \frac{538}{200371} = 0.002685 \cong 0.00269 \]
Assumed that the bolts at bottom row and middle row are in compression and top bolts are in tension.

\[ x = 120 + 304 - 26 - d_n - \frac{10.2}{2} = 392.9 - d_n \]

Tension = Compression

\[ F_r + F_{ss} + F_b = F_w + F_f \]

Tension:

\[ F_r = 4A_rE_s \times 0.00269 = 4 \times 113.10 \times 200371 \times 0.00269 = 243842.69 \text{ N} \]

\[ F_{ss} = A_{ss}E_s \times \frac{0.00269}{d_n} (d_n - (D_{cs} - \text{cover}) + d_{ss}) \]

\[ = 906.12 \times 216122 \times \frac{0.00269}{d_n} (d_n - (120 - 26) + 14.30) \]

\[ = \frac{1}{d_n} (0.5268 \times 10^6 d_n - 41.985 \times 10^6) \]

\[ F_b = 2A_bE_s \times \frac{0.00269}{d_n} (d_n - (D_{cs} - \text{cover}) - 65) \]

\[ = 2 \times 314 \times 218871 \times \frac{0.00269}{d_n} (d_n - (120 - 26) - 65) \]

\[ = \frac{1}{d_n} (0.369 \times 10^6 d_n - 58.79 \times 10^6) \]

Compression:

\[ F_w = A_wE_s \times \frac{0.00269}{d_n} \times \frac{2x}{3} \]

\[ = (392.9 - d_n) \times t_w \times E_s \times \frac{0.00269}{d_n} (\frac{2}{3} (392.9 - d_n)) \]

\[ = (392.9 - d_n) \times 6.1 \times 203765 \times \frac{0.00269}{d_n} (\frac{2}{3} (392.9 - d_n)) \]

\[ = \frac{1}{d_n} (344.10 \times 10^6 - 1.752 \times 10^6 d_n + 2.229 \times 10^3 d_n^2) \]

\[ F_f = A_fE_s \times \frac{0.00269}{d_n} x \]

\[ = (165 - 6.1) \times 10.2 \times 206338 \times \frac{0.00269}{d_n} \times (392.9 - d_n) \]

\[ = \frac{1}{d_n} (353.46 \times 10^6 - 0.899 \times 10^6 d_n) \]

Tension = Compression

\[ F_r + F_{ss} + F_b = F_w + F_f \]

\[ 0.2438 \times 10^6 + \frac{1}{d_n} (0.5268 \times 10^6 d_n - 41.985 \times 10^6) + \frac{1}{d_n} (0.369 \times 10^6 d_n - 58.79 \times 10^6) \]

\[ = \frac{1}{d_n} (344.10 \times 10^6 - 1.752 \times 10^6 d_n + 2.229 \times 10^3 d_n^2) \]

\[ + \frac{1}{d_n} (353.46 \times 10^6 - 0.899 \times 10^6 d_n) \]
or,
\[0.2438 \times 10^6 d_n + 0.5268 \times 10^6 d_n - 41.985 \times 10^6 + 0.369 \times 10^6 d_n - 58.79 \times 10^6 = 344.10 \times 10^6 - 1.752 \times 10^6 d_n + 2.229 \times 10^3 d_n^2 + 353.46 \times 10^6 - 0.899 \times 10^6 d_n\]

or,
\[0.2438 \times 10^6 d_n + 0.5268 \times 10^6 d_n - 41.985 \times 10^6 + 0.369 \times 10^6 d_n - 58.79 \times 10^6 = 344.10 \times 10^6 - 1.752 \times 10^6 d_n + 2.229 \times 10^3 d_n^2 + 353.46 \times 10^6 - 0.899 \times 10^6 d_n\]

or,
\[-2.229 \times 10^3 d_n^2 + 3.7906 \times 10^6 d_n - 798.445 \times 10^6 = 0\]

\[d_n = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}\]

\[\therefore d_n = 246.31 \, mm\]

Therefore

Tension:
\[F_r = 243842.69 \, N = 243.84 \times 10^3 \, N = 243.84 \, kN\]
\[F_{ss} = \frac{1}{d_n} (0.5268 \times 10^6 d_n - 41.985 \times 10^6) = 352.5 \times 10^3 \, N = 356.34 \, kN\]
\[F_b = \frac{1}{d_n} (0.369 \times 10^6 d_n - 58.79 \times 10^6) = 125.29 \times 10^3 \, N = 130.31 \, kN\]

Compression:
\[F_w = \frac{1}{d_n} (344.10 \times 10^6 - 1.752 \times 10^6 d_n + 2.229 \times 10^3 d_n^2) = 194.05 \times 10^3 \, N\]
\[= 194.05 \, kN\]
\[F_f = \frac{1}{d_n} (353.46 \times 10^6 - 0.899 \times 10^6 d_n) = 536.02 \times 10^3 \, N = 536.02 \, kN\]
Distances between forces and plastic neutral axis:
\[ d_r = d_n = 246.31 \text{ mm} \]
\[ d_{ss} = d_n + 26 - 120 + 14.30 = 246.31 + 26 - 120 + 14.30 = 166.61 \text{ mm} \]
\[ d_b = d_n + 26 - 120 - 65 = 246.31 + 26 - 120 - 65 = 87.31 \text{ mm} \]
\[ d_f = x = 392.9 - d_n = 392.9 - 246.31 = 146.59 \text{ mm} \]
\[ d_w = \frac{2x}{3} = \frac{2 \times 146.59}{3} = 97.73 \text{ mm} \]

Resistance moment of blind-bolted endplate connection to hogging bending:
\[ M_{\text{joint-hogging}} = F_r \times d_r + F_{ss} \times d_{ss} + F_b \times d_b + F_w \times d_w + F_f \times d_f \]
\[ = 243.84 \times 10^3 \times 246.31 + 356.34 \times 10^3 \times 166.61 + 130.31 \times 10^3 \]
\[ \times 87.31 + 194.05 \times 10^3 \times 97.73 + 536.02 \times 10^3 \times 146.59 \]
\[ = 228.35 \times 10^6 \text{ N} \cdot \text{mm} = 228.35 \text{ kN} \cdot \text{m} \]

B.3.2 Joint shear resistance for blind-bolted endplate connection
Assumed that the endplate thickness of the flush endplate connection, \( t_p \), is 10 mm

B.3.2.1 Bolt in shear
\[ V_{Rd_1} = 0.8nF_{bv,Rd} = 0.80 \times 6 \times 101.28 = 486.14 \text{ kN} \]
\[ F_{bv,Rd} = \frac{\alpha_v F_{bv,Rk}}{\gamma_{M2}} = \frac{0.6 \times 211}{1.25} = 101.28 \text{ kN} \]
\[ n = 6 \]
\[ \alpha_v = 0.6 \]
\[ F_{bv,Rk} = 211 \text{ kN} \]
\[ \gamma_{M2} = 1.25 \]

B.3.2.2 Endplate in bearing:
\[ V_{Rd_2} = nF_{pb,Rd} = 6 \times 98.18 = 589.08 \text{ kN} \]
\[ F_{pb,Rd} = \frac{k_1 \alpha_b d_b t_p f_{up}}{\gamma_{M2}} = \frac{2.3 \times 0.58 \times 20 \times 10 \times 460}{1.25} = 98.18 \text{ kN} \]
\[\alpha_b = \min(\alpha_1; \alpha_2; \alpha_3; 1) = \min(0.58; 0.62; 2.17; 1) = 0.58\]
\[\alpha_1 = e_1/(3d_{b,o}) = 65/(3 \times 35) = 0.62\]
\[\alpha_2 = \frac{p_1}{3d_{b,o}} \times \frac{1}{4} = \frac{87}{3 \times 35} \times \frac{1}{4} = 0.58\]
\[\alpha_3 = \frac{f_{ub}}{f_{up}} = \frac{1000}{460} = 2.17\]
\[k_1 = \min\left(2.8 \frac{p_2}{d_{b,o}} - 1.7; 1.4 \frac{p_2}{d_{b,o}} - 1.7; 2.5 \right)\]
\[= \min\left(2.8 \frac{65}{35} - 1.7; 1.4 \frac{100}{35} - 1.7; 2.5 \right) = \min(3.5; 2.3; 2.5) = 2.3\]
\[d_b = 20 \text{ mm}\]
\[d_{b,o} = 35 \text{ mm}\]
\[t_p = 10 \text{ mm}\]
\[e_1 = 65 \text{ mm}\]
\[e_2 = 65 \text{ mm}\]
\[p_1 = 87 \text{ mm}\]
\[p_2 = 100 \text{ mm}\]
\[f_{ub} = 1000 \text{ MPa}\]
\[f_{up} = 460 \text{ MPa}\]
\[\gamma_{M2} = 1.25\]

**B.3.2.3 Gross section of endplate in shear:**
\[ V_{Rd,3} = 2F_{pv,Rd} = 2 \times 483.7 = 967.4 \text{ kN} \]
\[ F_{pv,Rd} = \frac{A_v f_{yp}}{1.27 \sqrt{3} \gamma_{M0}} = \frac{3040 \times 355}{1.27 \times \sqrt{3} \times 1} = 483.7 \text{ kN} \]
\[ A_v = h_p t_p = 304 \times 10 = 3040 \text{ mm}^2\]
\[ h_p = 528 \text{ mm}\]
\[ t_p = 10 \text{ mm}\]
\[ f_{yp} = 355 \text{ MPa}\]
\[ \gamma_{M0} = 1\]

**B.3.2.4 Net section of endplate in shear:**
\[ V_{Rd,4} = 2F'_{pv,Rd} = 845.6 \text{ kN} \]
\[ F'_{pv,Rd} = \frac{A_v,net f_{up}}{\sqrt{3} \gamma_{M2}} = \frac{1990 \times 460}{\sqrt{3} \times 1.25} = 422.8 \text{ kN} \]
\[ A_{v,net} = (h_p - n_1 d_{b,o}) \ t_p = (304 - 3 \times 35) \times 10 = 1990 \ \text{mm}^2 \]

\[ h_p = 304 \ \text{mm} \]

\[ t_p = 10 \ \text{mm} \]

\[ f_{up} = 460 \ \text{MPa} \]

\[ \gamma_{M2} = 1.25 \]

\[ d_{b,o} = 35 \ \text{mm} \]

\[ n_1 = 3 \]

**B.3.2.5 Plate in bending:**

\[ \therefore 1.36p_2 = 1.36 \times (100) = 136 \leq h_p = 304 \]

\[ \therefore V_{Rd 5} = \infty \]

**B.3.2.6 Beam web in shear:**

\[ V_{Rd 6} = F_{v,Rd} = 385.43 \ \text{kN} \]

\[ F_{v,Rd} = \frac{A_v \ f_{ybw}}{\sqrt[3]{\gamma_{M0}}} = \frac{5068.8 \times 360}{\sqrt[3]{1}} = 385.43 \ \text{kN} \]

\[ A_v = h_{bw}t_{bw} = 304 \times 6.1 = 1854.4 \ \text{mm}^2 \]

\[ f_{ybw} = 360 \ \text{MPa} \]

\[ h_{bw} = 304 \ \text{mm} \]

\[ t_{bw} = 9.6 \ \text{mm} \]

**B.3.2.7 Throat thickness of weld:**

\[ a > 0.4t_{bw} \beta_w \sqrt[3]{\gamma_{M2}} \frac{f_{ybw} \gamma_{M2}}{f_{ubw} \gamma_{M0}} = 0.4 \times 6.1 \times 0.9 \times \sqrt[3]{1.25} \times \frac{360}{460} \times \frac{1.25}{1.0} = 3.7 \approx 4 \ \text{mm} \]

\[ t_{bw} = 6.1 \]

\[ \beta_w = 0.9 \ for \ S355 \ grade \ steel \]

\[ f_{ybw} = 360 \]

\[ f_{ubw} = 460 \]

\[ \gamma_{M2} = 1.25 \]

\[ \gamma_{M0} = 1.0 \]

**Minimum weld size for S355 Grade steel:**

\[ a = 0.543t_{bw} = 0.543 \times 6.1 = 3.3 \approx 3.50 \ \text{mm} \]
Welding in shear:
\[ V_{Rd7} = 2a \frac{f_{ubw}}{\gamma_{M2}\beta_w\sqrt{3}} = 2 \times 4 \times \frac{460}{1.25 \times 0.9 \times \sqrt{3}} = 1888.58 \text{ kN} \]

B.3.2.8 Joint shear resistance:
Shear resistance of the joint:
\[ V_{Rd} = \min(V_{Rd1}, V_{Rd2}, V_{Rd3}, V_{Rd4}, V_{Rd5}, V_{Rd6}, V_{Rd7}) = V_{Rd6} = 385.43 \text{ kN} \]
Failure mode: Beam web in shear.

B.4 Design of Through-Plate Beam-Column Connection
Reference standard:

Basic design conditions:
Through plate thickness \((t_p)\): 12 mm
Bolt: M20 Class 8.8 bolt
Yield strength of through plate: \(f_{yp} = 355\text{ MPa}; f_{up} = 460\text{ MPa}; E_s = 200000 \text{ MPa}\)
Yield strength of bolt: \(f_{yb} = 900 \text{ MPa}; f_{ub} = 1000\text{ MPa}; E_s = 200000 \text{ MPa}\)

Design principle:
Semi-rigid connection, moment capacity of connection should be lower than that of composite beam, and the shear resistance of joint should be controlled by the beam.

B.4.1 Resistance moment of through-plate connection to hogging bending
Cross-section area of each steel reinforced bar, \(A_r = \pi r^2 = \pi \times 6^2 = 113.10 \text{ mm}^2\)
Cross-section area of steel profile sheet, \(A_{ss} = 1678 \times (0.9 - 0.36) = 906.12 \text{ mm}^2\)
Cross-section area of each bolt, \(A_b = \pi \times 10^2 = 314 \text{ mm}^2\)
Dimension and force details in the through plate cross-section are shown in Figure B.5:
Figure B.5 Dimension and force details at the through plate cross-section

Assume that the plastic neutral axis would be in the through plate.

\[ x = h'_{p} + D_{cs} - \text{cover} - \frac{h_{p}}{2} - d_{n} = 304 + 120 - 26 - \frac{244}{2} - d_{n} = 276 - d_{n} \]

Tension = Compression

\[ F_{r} + F_{ss} = F_{p} \]

**Tension:**

\[ F_{r} = 4A_{r}E_{s} \times 0.0025 = 4 \times 113.10 \times 200000 \times 0.0025 = 226200 \text{ N} \]

\[ F_{ss} = A_{ss}E_{s} \times \frac{0.0025}{d_{n}}(d_{n} - (D_{cs} - \text{cover}) + d_{ss}) \]

\[ = 1409.52 \times 200000 \times \frac{0.0025}{d_{n}}(d_{n} - (120 - 26) + 14.30) \]

\[ = \frac{1}{d_{n}}(0.705 \times 10^{6}d_{n} - 56.19 \times 10^{6}) \]

**Compression:**

\[ F_{p} = A_{p}E_{s} \times \frac{0.0025}{d_{n}}x = h_{p} \times t_{p} \times E_{s} \frac{0.0025}{d_{n}}(276 - d_{n}) \]

\[ = 244 \times 12 \times 200000 \times \frac{0.0025}{d_{n}}(276 - d_{n}) \]

\[ = \frac{1}{d_{n}}(404.06 \times 10^{6} - 1.464 \times 10^{6}d_{n}) \]

\[ \therefore F_{r} + F_{ss} = F_{p} \]

\[ 0.226 \times 10^{6} + \frac{1}{d_{n}}(0.705 \times 10^{6}d_{n} - 56.19 \times 10^{6}) \]

\[ = \frac{1}{d_{n}}(404.06 \times 10^{6} - 1.464 \times 10^{6}d_{n}) \]
or,
\[ 0.226 \times 10^6 d_n + 0.705 \times 10^6 d_n - 56.19 \times 10^6 = 404.06 \times 10^6 - 1.464 \times 10^6 d_n \]
for,
\[ 0.226 \times 10^6 d_n + 0.705 \times 10^6 d_n + 1.464 \times 10^6 d_n = 404.06 \times 10^6 + 56.19 \times 10^6 \\
2.395 \times 10^6 d_n = 460.25 \times 10^6 \\
\therefore d_n = 192.17 \text{ mm} \]

Therefore

Tension:
\[ F_r = 226 \times 10^3 \text{ N} \]
\[ F_{ss} = \frac{1}{d_n} (0.705 \times 10^6 d_n - 56.19 \times 10^6) = 412.60 \times 10^3 \text{ N} \]

Compression:
\[ F_p = \frac{1}{d_n} (404.06 \times 10^6 - 1.464 \times 10^6 d_n) = 638.62 \times 10^3 \text{ N} \]

Distances between forces and plastic neutral axis:
\[ d_r = d_n = 192.17 \text{ mm} \]
\[ d_{ss} = d_n + 26 - 120 + 14.30 = 192.17 + 26 - 120 + 14.30 = 112.47 \text{ mm} \]
\[ d_x = x = 276 - d_n = 276 - 192.17 = 83.83 \text{ mm} \]

Resistance moment of blind-bolted connection to hogging bending:
\[ M_{\text{joint-hogging}} = F_r \times d_r + F_{ss} \times d_{ss} + F_p \times x \]
\[ = 226 \times 10^3 \times 192.17 + 412.60 \times 10^3 \times 112.47 + 638.62 \times 10^3 \times 83.83 \]
\[ = 143.37 \times 10^6 \text{ N} \cdot \text{mm} = 143.37 \text{ kN} \cdot \text{m} \]
B.4.2 Joint shear resistance for through-plate connection

Dimension details of through plate connection are shown in Figure B.6.

Assumed that the plate thickness of the through plate connection, $t_p$, is 12 mm.

![Figure B.6 Dimension details of through plate connections](image)

**B.4.2.1 Bolt in shear**

\[
V_{Rd1} = \frac{F_{bv,Rd}}{\sqrt{\left(\frac{z}{2l} + \frac{1}{n}\right)^2 + \left(\frac{z}{2l} \left( n_1 - 1 \right)\right)^2}} = \frac{101.28}{\sqrt{\left(\frac{97.5 \times 55}{2 \times 28253.5} + \frac{1}{4}\right)^2 + \left(\frac{97.5 \times 77}{2 \times 28253.5} (3 - 1)\right)^2}} = 252.32 \text{kN}
\]

\[
F_{bv,Rd} = \frac{\alpha_v A_b f_{ub}}{\gamma_{M2}} = \frac{0.6 \times 211}{1.25} = 101.28 \text{kN}
\]

- $n = 6$
- $n_1 = 3$
- $p_1 = 77 \text{ mm}$
- $p_2 = 55 \text{ mm}$
- $\alpha_v = 0.6$
- $A_b = 245 \text{ mm}^2$
- $f_{ub} = 1000 \text{ MPa}$
- $\gamma_{M2} = 1.25$
- $z = HG + e_2 + \frac{p_2}{2} = 25 + 45 + \frac{55}{2} = 97.5 \text{ mm}$
- $l = \frac{n_1}{2} p_2^2 + 6 n_1 (n_1^2 - 1) p_1^2 = \frac{3}{2} (55)^2 + 6 \times 3 \times (3^2 - 1) \times 77^2 = 28253.5$
B.4.2.2 Plate in bearing

\[ V_{Rd} = \sqrt{\left( \frac{1}{n^2} + \frac{\beta}{\sqrt{F_{pb,ver,Rd}}} \right)^2 + \left( \frac{1}{\frac{I}{2} + 0.095} \right)^2} = \sqrt{\left( \frac{1}{6} + \frac{0.226}{128.06} \right)^2 + \left( \frac{1}{55} \right)^2} = 313.51 \text{kN} \]

\[ n = 6 \]

\[ \alpha = \frac{z}{l} \frac{p_2}{2} = \frac{97.5}{28253.5} \times \frac{55}{2} = 0.095 \]

\[ \beta = \frac{z}{l} \frac{n_1 - 1}{2} \frac{p_1}{2} = \frac{97.5}{28253.5} \times \frac{3 - 1}{2} \times 77 = 0.266 \]

\[ l = \frac{n_1}{2} p_2^2 + 6 \ n_1 (n_1^2 - 1) p_1^2 = \frac{3}{2} (55)^2 + 6 \times 3 \times (3^2 - 1) \times 77^2 \]

\[ = 28253.5 \]

\[ F_{pb,ver,Rd} = \frac{k_1 \alpha_b d_b t_p f_{up}}{\gamma M_2} = \frac{1.8 \times 0.68 \times 20 \times 12 \times 460}{1.25} = 108.1 \text{kN} \]

\[ \alpha_b = \min(\alpha_1; \alpha_2; \alpha_3; 1) = \min(0.68; 0.92; 2.17; 1) = 0.68 \]

\[ \alpha_1 = \frac{e_1}{(3d_{b,o})} = 45/(3 \times 22) = 0.68 \]

\[ \alpha_2 = \frac{p_1}{3d_{b,o}} - \frac{1}{4} = \frac{77}{(3 \times 22)} - \frac{1}{4} = 0.92 \]

\[ \alpha_3 = \frac{f_{ub}}{f_{up}} = \frac{1000}{460} = 2.17 \]

\[ k_1 = \min \left( 2.8 \frac{e_2}{d_{b,o}} - 1.7; 1.4 \frac{p_2}{d_{b,o}} - 1.7; 2.5 \right) \]

\[ = \min \left( 2.8 \frac{45}{22} - 1.7; 1.4 \frac{55}{d_{b,o}} - 1.7; 2.5 \right) = \min(4.03; 1.8; 2.5) \]

\[ = 1.8 \]

\[ d_b = 20 \text{ mm} \]

\[ d_{b,o} = 22 \text{ mm} \]

\[ e_2 = 45 \text{ mm} \]

\[ p_1 = 77 \text{ mm} \]

\[ p_2 = 55 \text{ mm} \]

\[ t_p = 12 \text{ mm} \]

\[ f_{ub} = 1000 \text{ MPa} \]

\[ f_{up} = 460 \text{ MPa} \]

\[ \gamma M_2 = 1.25 \]
\[
F_{pb,\text{hor},Rd} = \frac{k_1 \alpha b \ d_b \ t_p \ f_{up}}{\gamma M_2} = \frac{2.5 \times 0.58 \times 20 \times 12 \times 460}{1.25} = 128.06 \text{ kN}
\]

\[
\alpha_b = \min(\alpha_1, \alpha_2, \alpha_3, 1) = \min(0.68; 0.58; 2.17; 1) = 0.58
\]

\[
\alpha_1 = \frac{e_2}{(3d_{b,o})} = 45/(3 \times 22) = 0.68
\]

\[
\alpha_2 = \frac{p_2}{3d_{b,o}} - \frac{1}{4} = \frac{55}{3 \times 22} - \frac{1}{4} = 4 = 0.58
\]

\[
\alpha_3 = \frac{f_{ub}}{f_{up}} = \frac{1000}{460} = 2.17
\]

\[
k_1 = \min(2.8 \frac{e_1}{d_{b,o}} - 1.7; 1.4 \frac{p_1}{d_{b,o}} - 1.7; 2.5)
\]

\[
= \min(2.8 \frac{45}{22} - 1.7; 1.4 \frac{77}{22} - 1.7; 2.5)
\]

\[
= \min(4.03; 3.2; 2.5) = 2.5
\]

B.4.2.3 Gross section of plate in shear

\[
V_{Rd3} = \frac{A_v \ f_{yp}}{1.27\sqrt{3} \gamma M_0} = \frac{2928 \times 355}{1.27\sqrt{3} \times 1} = 472.54 \text{ kN}
\]

\[
A_v = h_p \ t_p = 244 \times 12 = 2928 \text{ mm}^2
\]

\[
h_p = 244 \text{ mm}
\]

\[
t_p = 12 \text{ mm}
\]

\[
f_{yp} = 355 \text{ mm}
\]

\[
\gamma M_0 = 1
\]

B.4.2.4 Net section of plate in shear

\[
V_{Rd4} = \frac{A_{v,\text{net}} \ f_{up}}{\sqrt{3} \gamma M_2} = \frac{2136 \times 460}{\sqrt{3} \times 1.25} = 453.83 \text{ kN}
\]

\[
A_{v,\text{net}} = (h_p - n_1 d_{b,o}) \ t_p = (244 - 2 \times 22) \times 12 = 2136 \text{ mm}^2
\]

\[
h_p = 244 \text{ mm}
\]

\[
t_p = 12
\]

\[
n_1 = 3
\]

\[
d_{b,o} = 22 \text{ mm}
\]

\[
f_{up} = 460 \text{ mm}
\]

B.4.2.5 Plate in bending

Since \(2.73z = 2.73 \times 97.5 = 266.18 \geq h_p = 244\)

\[
V_{Rd5} = \frac{W_{el} \ f_{yp}}{z} = \frac{119072 \times 355}{97.5} \times \frac{1}{1} = 433.54 \text{ kN}
\]
\[
W_{el} = \frac{t_p h_p^2}{6} = \frac{12 \times 244^2}{6} = 119072
\]
\[
h_p = 244 \text{ mm}
\]
\[
t_p = 12 \text{ mm}
\]
\[
z = 97.5 \text{ mm}
\]
\[
f_{yp} = 355 \text{ MPa}
\]
\[
\gamma_{M0} = 1
\]

**B.4.2.6 Buckling of the plate**

\[
V_{Rd6} = \frac{W_{el} \sigma}{\gamma_{M0} z} = \frac{119072 \times 81 \times 235}{97.5 \times 1} = 352.14 \text{ kN}
\]
\[
W_{el} = \frac{t_p h_p^2}{6} = \frac{12 \times 244^2}{6} = 119072
\]
\[
\sigma = 81 \left( \frac{t_p}{z} \right)^2 \times 235 = 81 \times \left( \frac{12}{97.5} \right)^2 \times 235 = 288.34 \text{ MPa}
\]
\[
h_p = 244 \text{ mm}
\]
\[
t_p = 12 \text{ mm}
\]
\[
z = 97.5 \text{ mm}
\]
\[
\gamma_{M0} = 1
\]

**B.4.2.7 Beam web in bearing**

\[
V_{Rd7} = \frac{1}{\sqrt{\left( \frac{1}{n+\alpha} \right)^2 + \left( \frac{\beta}{F_{pb,ver,Rd}} \right)^2}} = \frac{1}{\sqrt{\left( \frac{1}{4+0.095} \right)^2 + \left( \frac{0.226}{65.1} \right)^2}} = 185.43 \text{ kN}
\]

\[
n = 6
\]
\[
\alpha = \frac{z}{\frac{p_2}{I}} = \frac{97.5}{28253.5} \times \frac{55}{2} = 0.095
\]
\[
\beta = \frac{z}{\frac{n_1 - 1}{2}} = \frac{97.5}{28253.5} \times \frac{3 - 1}{2} \times 77 = 0.266
\]
\[
I = \frac{n_1}{2} p_2^2 + 6 n_1 (n_1^2 - 1) p_1^2 = \frac{3}{2} (55)^2 + 6 \times 3 \times (3^2 - 1) \times 77^2 = 28253.5
\]
\[
F_{pb,ver,Rd} = \frac{k_1 \alpha_b d_b f_{bw} f_{ubw}}{\gamma_{M2}} = \frac{1.8 \times 0.92 \times 20 \times 6.1 \times 460}{1.25} = 74.35 \text{ kN}
\]
\[
\alpha_b = \min(\alpha_1; \alpha_2; 1) = \min(0.92; 2.17; 1) = 0.92
\]
\[
\alpha_1 = \frac{p_1}{3d_{b,o}} - \frac{1}{4} = \frac{77}{(3 \times 22)} - \frac{1}{4} = 0.92
\]
\[
\alpha_2 = \frac{f_{ub}}{f_{ubw}} = \frac{1000}{460} = 2.17
\]
\[
k_1 = \min \left( 2.8 \frac{e_{2b}}{d_{b,o}} - 1.7; 1.4 \frac{p_2}{d_{b,o}} - 1.7; 2.5 \right)
\]
\[
= \min \left( 2.8 \frac{45}{22} - 1.7; 1.4 \frac{55}{3 \times 22} - 1.7; 2.5 \right) = \min(4.03; 1.8; 2.5)
\]
\[
= 1.8
\]
\[d_b = 20 \text{ mm}\]
\[d_{b,o} = 22 \text{ mm}\]
\[e_{2b} = 45 \text{ mm}\]
\[p_1 = 77 \text{ mm}\]
\[p_2 = 55 \text{ mm}\]
\[t_{bw} = 6.1 \text{ mm}\]
\[f_{ub} = 1000 \text{ MPa}\]
\[f_{ubw} = 460 \text{ MPa}\]
\[\gamma_{M2} = 1.25\]
\[
F_{pb,hor,Rd} = \frac{k_1 \alpha_b d_b t_{bw} f_{ubw}}{\gamma_{M2}} = \frac{2.5 \times 0.58 \times 20 \times 6.1 \times 460}{1.25} = 65.1 \text{ kN}
\]
\[
\alpha_b = \min(\alpha_1, \alpha_2, 1) = \min(0.68; 2.17; 1) = 0.58
\]
\[
\alpha_1 = \frac{e_{2b}}{d_{b,o}} = \frac{45}{(3 \times 22)} = 0.68
\]
\[
\alpha_2 = \frac{p_2}{3d_{b,o}} - \frac{1}{4} = \frac{55}{3 \times 22} - \frac{1}{4} = 4 = 0.58
\]
\[
\alpha_3 = \frac{f_{ub}}{f_{ubw}} = \frac{1000}{460} = 2.17
\]
\[
k_1 = \min(1.4 \frac{p_1}{d_{b,o}} - 1.7; 2.5)
\]
\[
= \min(1.4 \frac{77}{22} - 1.7; 2.5)
\]
\[
= \min(3.2; 2.5) = 2.5
\]

**B.4.2.8 Gross section of the beam web in shear**

\[
V_{Rd,8} = \frac{A_{b,v} f_{yb,w}}{\sqrt{3} \gamma_{M0}} = \frac{1854.4 \times 360}{\sqrt{3} \times 1} = 385.43 \text{ kN}
\]
\[
A_{b,v} = h_b t_{bw} = 304 \times 6.1 = 1854.4 \text{ mm}^2
\]
\[ h_b = 304 \text{ mm} \]
\[ t_{bw} = 6.1 \text{ mm} \]
\[ f_{ybw} = 360 \text{ MPa} \]
\[ \gamma_{M0} = 1 \]

**B.4.2.9 Net section of the beam web in shear:**

\[
V_{Rd,9} = \frac{A_{v,net} f_{ubw}}{\sqrt{3} \gamma_{M2}} = \frac{1451.8 \times 460}{\sqrt{3} \times 1.25} = 308.46 \text{ kN}
\]

\[ A_{v,net} = (h_b - n_1 d_{b,o}) t_{bw} = (304 - 2 \times 22) \times 6.1 = 1451.8 \text{ mm}^2 \]

\[ h_b = 304 \text{ mm} \]
\[ t_{bw} = 6.1 \text{ mm} \]
\[ n_1 = 3 \]
\[ d_{b,o} = 22 \text{ mm} \]
\[ f_{ubw} = 460 \text{ MPa} \]

**B.4.2.10 Shear block of the beam web:**

\[
V_{Rd,10} = F_{eff,2,Rd} = 295.81 \text{ kN}
\]

\[
F_{eff,2,Rd} = 0.5 \frac{f_{ubw} A_{nt}}{\gamma_{M2}} + \frac{f_{ybw} A_{nv}}{\sqrt{3} \gamma_{M0}}
\]

\[
= 0.5 \times 460 \times 408.7 \div 1.25 + 360 \times 1061.4 \div \sqrt{3} \times 1 = 295.81 \text{ kN}
\]

\[ A_{nt} = t_{bw} \left( p_2 + e_{2b} - \frac{3 d_{b,o}}{2} \right) \]

\[ = 6.1 \times (55 + 45 - 3 \times \frac{22}{2}) = 408.7 \text{ mm}^2 \]

\[ A_{nv} = t_{bw} \left( e_{1b} + (n_1 - 1)p_1 - (n_1 - 0.5) d_{b,o} \right) \]

\[ = 6.1 \times (75 + (3 - 1) \times 77 - (3 - 0.5) \times 22) = 1061.4 \text{ mm}^2 \]

\[ t_{bw} = 6.1 \text{ mm} \]
\[ e_{2b} = 45 \text{ mm} \]
\[ p_1 = 77 \text{ mm} \]
\[ d_{b,o} = 22 \text{ mm} \]
\[ e_{1b} = e_1 + \text{vertical gap between beam and THP} = 45 + 30 = 75 \text{ mm} \]
\[ n_1 = 3 \]
\[ f_{ybw} = 360 \text{ MPa} \]
\[ f_{ubw} = 460 \text{ MPa} \]
\[ \gamma_{M0} = 1; \quad \gamma_{M2} = 1.25 \]
Joint shear resistance:
Shear resistance of the joint:
\[ V_{\text{Rd}} = \min(V_{\text{Rd}1}, V_{\text{Rd}2}, V_{\text{Rd}3}, V_{\text{Rd}4}, V_{\text{Rd}5}, V_{\text{Rd}6}, V_{\text{Rd}7}, V_{\text{Rd}8}, V_{\text{Rd}9}, V_{\text{Rd}10}) \]

\[ = V_{\text{Rd}7} = 185.43\ kN \]

(Failure mode: Beam web in bearing)

B.5 Summary of Hybrid Stainless-Carbon Steel Joints
Calculated results on hybrid stainless-carbon steel joints are listed in Table B.1.

<table>
<thead>
<tr>
<th>Joint types</th>
<th>Blind-bolted endplate connection for square column with stainless steel tube</th>
<th>Through-plate and half-through plate connection for square column with stainless steel tube</th>
<th>Blind-bolted endplate connection for circular column with stainless steel tube</th>
<th>Blind-bolted endplate connection for square column with carbon steel tube</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column section (mm)</td>
<td>□-360×6</td>
<td>304×165×6.1×10.2</td>
<td>□-360×6</td>
<td>304×165×6.1×10.2</td>
</tr>
<tr>
<td>Beam section (310UB40.4) (mm)</td>
<td>304×230×10</td>
<td>700/264×244×12 or 319/590×244×12</td>
<td>304×230×10</td>
<td>304×230×10</td>
</tr>
<tr>
<td>Slab section (mm)</td>
<td>900×120</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Length of column/beam/slab (mm)</td>
<td>2200/1500/1500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Endplate or through-plate/half-through plate size (h_p × b_p × t_p) (mm)</td>
<td>6/4</td>
<td>6/3</td>
<td>6/4</td>
<td>6/4</td>
</tr>
<tr>
<td>Number of bolts in each connection (Major direction/Minor direction of beam)</td>
<td>626.453</td>
<td>626.453</td>
<td>5167.40</td>
<td>5243.4</td>
</tr>
<tr>
<td>Column resistance to axial compression (kN)</td>
<td>5.619 × 10^{13}</td>
<td>3.307 × 10^{13}</td>
<td>3.37 × 10^{13}</td>
<td></td>
</tr>
<tr>
<td>Composite beam resistance moment to hogging bending (kN·m)</td>
<td></td>
<td></td>
<td></td>
<td>279.38</td>
</tr>
<tr>
<td>Composite beam flexural stiffness (MPa·mm^{4})</td>
<td></td>
<td></td>
<td></td>
<td>2.96 × 10^{13}</td>
</tr>
<tr>
<td>Connection resistance moment to hogging bending (kN·m)</td>
<td>228.35</td>
<td>143.37</td>
<td>228.35</td>
<td>228.35</td>
</tr>
<tr>
<td>Joint shear resistance</td>
<td>385.43</td>
<td>185.43</td>
<td>385.43</td>
<td>385.43</td>
</tr>
<tr>
<td>Failure mode of joint under shear force</td>
<td>Beam web in shear</td>
<td>Beam web in shear</td>
<td>Beam web in shear</td>
<td>Beam web in shear</td>
</tr>
<tr>
<td>Line stiffness ratio of beam to column</td>
<td>0.52</td>
<td>0.52</td>
<td>0.96</td>
<td>0.87</td>
</tr>
<tr>
<td>Failure mode of joint under vertical beam load</td>
<td>Connection failure due to the fact that connection moment reaches the resistance moment hogging bending</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
APPENDIX C

MATERIAL PROPERTIES

C.1 Mechanical Properties of Stainless Steel Materials

Table C.1 Stainless steel tube (Square tube-6 mm thickness)

<table>
<thead>
<tr>
<th></th>
<th>0.01 % Proof stress (MPa)</th>
<th>0.2 % Proof stress (MPa)</th>
<th>Ultimate stress (MPa)</th>
<th>Elastic modulus (MPa)</th>
<th>n</th>
<th>Elongation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coupon-1</td>
<td>201.2</td>
<td>338.1</td>
<td>720.2</td>
<td>200739</td>
<td>5.8</td>
<td>56.3(53.5)</td>
</tr>
<tr>
<td>Coupon-2</td>
<td>193.5</td>
<td>338.0</td>
<td>710.6</td>
<td>201519</td>
<td>5.4</td>
<td>55.3(53.3)</td>
</tr>
<tr>
<td>Coupon-3</td>
<td>251.4</td>
<td>341.1</td>
<td>712.0</td>
<td>200696</td>
<td>9.8</td>
<td>55.2(52.5)</td>
</tr>
<tr>
<td>Average</td>
<td>215.4</td>
<td>339</td>
<td>714</td>
<td>200985</td>
<td>6.6</td>
<td></td>
</tr>
</tbody>
</table>

Table C.2 Stainless steel tube (Circular tube-6 mm thickness)

<table>
<thead>
<tr>
<th></th>
<th>0.01 % Proof stress (MPa)</th>
<th>0.2 % Proof stress (MPa)</th>
<th>Ultimate stress (MPa)</th>
<th>Elastic modulus (MPa)</th>
<th>n</th>
<th>Elongation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coupon-1</td>
<td>248.1</td>
<td>366.0</td>
<td>735.2</td>
<td>200993</td>
<td>7.7</td>
<td>49(48.4)</td>
</tr>
<tr>
<td>Coupon-2</td>
<td>260.2</td>
<td>368.1</td>
<td>731.1</td>
<td>200783</td>
<td>8.6</td>
<td>49.5(50.2)</td>
</tr>
<tr>
<td>Coupon-3</td>
<td>227.5</td>
<td>367.1</td>
<td>729.4</td>
<td>200213</td>
<td>6.3</td>
<td>47.8(49.2)</td>
</tr>
<tr>
<td>Average</td>
<td>245.3</td>
<td>367</td>
<td>732</td>
<td>200663</td>
<td>7.4</td>
<td></td>
</tr>
</tbody>
</table>

Figure C.1 Stress-strain curves of the coupon test of stainless steel tube (Square tube-6 mm thickness)

Figure C.2 Stress-strain curves of the coupon test of stainless steel tube (Circular tube-6 mm thickness)
### C.2 Mechanical Properties of Carbon Steel Materials

#### Table C.3 Carbon steel tube (Circular tube-6 mm thickness)

<table>
<thead>
<tr>
<th></th>
<th>Yield stress (MPa)</th>
<th>Ultimate stress (MPa)</th>
<th>Elastic modulus (MPa)</th>
<th>Elongation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coupon-1</td>
<td>379.5</td>
<td>472.4</td>
<td>207322</td>
<td>22.3(18.5)</td>
</tr>
<tr>
<td>Coupon-2</td>
<td>378.8</td>
<td>472.0</td>
<td>204783</td>
<td>22.6(29.5)</td>
</tr>
<tr>
<td>Coupon-3</td>
<td>379.7</td>
<td>473.2</td>
<td>206910</td>
<td>23.4(29.4)</td>
</tr>
<tr>
<td>Average</td>
<td>379</td>
<td>473</td>
<td>206338</td>
<td></td>
</tr>
</tbody>
</table>

#### Table C.4 Steel profile sheet

<table>
<thead>
<tr>
<th></th>
<th>Yield stress (MPa)</th>
<th>Ultimate stress (MPa)</th>
<th>Elastic modulus (MPa)</th>
<th>Elongation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coupon-1</td>
<td>614.1</td>
<td>615.5</td>
<td>222532</td>
<td></td>
</tr>
<tr>
<td>Coupon-2</td>
<td>617.2</td>
<td>619</td>
<td>208208</td>
<td></td>
</tr>
<tr>
<td>Coupon-3</td>
<td>613.7</td>
<td>615.9</td>
<td>217626</td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td>615</td>
<td>617</td>
<td>216122</td>
<td></td>
</tr>
</tbody>
</table>

#### Table C.5 Steel beam flange (310UB40.4)

<table>
<thead>
<tr>
<th></th>
<th>Yield stress (MPa)</th>
<th>Ultimate stress (MPa)</th>
<th>Elastic modulus (MPa)</th>
<th>Elongation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coupon-1</td>
<td>363.1</td>
<td>537.5</td>
<td>193800</td>
<td>25.2(36.0)</td>
</tr>
<tr>
<td>Coupon-2</td>
<td>349.0</td>
<td>532.4</td>
<td>197315</td>
<td>25.2(36.0)</td>
</tr>
<tr>
<td>Coupon-3</td>
<td>344.0</td>
<td>534.0</td>
<td>204368</td>
<td>26.0(35.3)</td>
</tr>
<tr>
<td>Average</td>
<td>352</td>
<td>535</td>
<td>198494</td>
<td></td>
</tr>
</tbody>
</table>

#### Table C.6 Steel beam web (310UB40.4)

<table>
<thead>
<tr>
<th></th>
<th>Yield stress (MPa)</th>
<th>Ultimate stress (MPa)</th>
<th>Elastic modulus (MPa)</th>
<th>Elongation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coupon-1</td>
<td>367.2</td>
<td>535.9</td>
<td>203811</td>
<td>25.4(34.2)</td>
</tr>
<tr>
<td>Coupon-2</td>
<td>369.4</td>
<td>530.9</td>
<td>201015</td>
<td>23.8(32.2)</td>
</tr>
<tr>
<td>Coupon-3</td>
<td>372.6</td>
<td>536.0</td>
<td>206468</td>
<td>24.3(32.3)</td>
</tr>
<tr>
<td>Average</td>
<td>370</td>
<td>534</td>
<td>203765</td>
<td></td>
</tr>
</tbody>
</table>

#### Table B.7 Endplate (10 mm)

<table>
<thead>
<tr>
<th></th>
<th>Yield stress (MPa)</th>
<th>Ultimate stress (MPa)</th>
<th>Elastic modulus (MPa)</th>
<th>Elongation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coupon-1</td>
<td>391.4</td>
<td>505.5</td>
<td>206287</td>
<td>29.8 (42.5)</td>
</tr>
<tr>
<td>Coupon-2</td>
<td>388.8</td>
<td>507.1</td>
<td>209977</td>
<td>31.3(43.0)</td>
</tr>
<tr>
<td>Coupon-3</td>
<td>384.5</td>
<td>504.4</td>
<td>202631</td>
<td>30.0(43.3)</td>
</tr>
<tr>
<td>Average</td>
<td>388</td>
<td>506</td>
<td>206298</td>
<td></td>
</tr>
</tbody>
</table>
Table C.8 Through plate (12 mm)

<table>
<thead>
<tr>
<th>Coupon</th>
<th>Yield stress (MPa)</th>
<th>Ultimate stress (MPa)</th>
<th>Elastic modulus (MPa)</th>
<th>Elongation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coupon-1</td>
<td>373.3</td>
<td>498.2</td>
<td>203180</td>
<td>28.4 (41.9)</td>
</tr>
<tr>
<td>Coupon-2</td>
<td>370.5</td>
<td>496.3</td>
<td>209953</td>
<td>31.5 (43.6)</td>
</tr>
<tr>
<td>Coupon-3</td>
<td>381.8</td>
<td>496.4</td>
<td>209249</td>
<td>29.5 (43.5)</td>
</tr>
<tr>
<td>Average</td>
<td>375</td>
<td>497</td>
<td>207461</td>
<td></td>
</tr>
</tbody>
</table>

Table C.9 M20 Hollo-Bolts

<table>
<thead>
<tr>
<th>Coupon</th>
<th>Yield stress (MPa)</th>
<th>Ultimate stress (MPa)</th>
<th>Elastic modulus (MPa)</th>
<th>Elongation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coupon-1</td>
<td>896.4</td>
<td>962.2</td>
<td>217158</td>
<td>14.0 (21.8)</td>
</tr>
<tr>
<td>Coupon-2</td>
<td>913.4</td>
<td>978.1</td>
<td>213464</td>
<td>17.5 (20.95)</td>
</tr>
<tr>
<td>Coupon-3</td>
<td>858.8</td>
<td>919.5</td>
<td>225991</td>
<td>14.3 (23.48)</td>
</tr>
<tr>
<td>Average</td>
<td>890</td>
<td>953</td>
<td>218871</td>
<td></td>
</tr>
</tbody>
</table>

Table C.10 Binding bars (N20 Bar)

<table>
<thead>
<tr>
<th>Coupon</th>
<th>Yield stress (MPa)</th>
<th>Ultimate stress (MPa)</th>
<th>Elastic modulus (MPa)</th>
<th>Elongation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coupon-1</td>
<td>543.8</td>
<td>641.4</td>
<td>203587</td>
<td>(11.28)</td>
</tr>
<tr>
<td>Coupon-2</td>
<td>550.6</td>
<td>649.0</td>
<td>202503</td>
<td>(11.31)</td>
</tr>
<tr>
<td>Coupon-3</td>
<td>551.5</td>
<td>650.6</td>
<td>203694</td>
<td>(34.93)</td>
</tr>
<tr>
<td>Average</td>
<td>549</td>
<td>647</td>
<td>203261</td>
<td></td>
</tr>
</tbody>
</table>

Table C.11 Longitudinal reinforcement (N12 Bar)

<table>
<thead>
<tr>
<th>Coupon</th>
<th>Yield stress (MPa)</th>
<th>Ultimate stress (MPa)</th>
<th>Elastic modulus (MPa)</th>
<th>Elongation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coupon-1</td>
<td>536.6</td>
<td>656.5</td>
<td>200326</td>
<td>13.6 (10.19)</td>
</tr>
<tr>
<td>Coupon-2</td>
<td>538.5</td>
<td>653.6</td>
<td>200803</td>
<td>12.8 (8.5)</td>
</tr>
<tr>
<td>Coupon-3</td>
<td>538.7</td>
<td>649.3</td>
<td>199985</td>
<td>11.7 (8.4)</td>
</tr>
<tr>
<td>Average</td>
<td>538</td>
<td>653</td>
<td>200371</td>
<td></td>
</tr>
</tbody>
</table>

Table C.12 Transverse reinforcement (N10 Bar)

<table>
<thead>
<tr>
<th>Coupon</th>
<th>Yield stress (MPa)</th>
<th>Ultimate stress (MPa)</th>
<th>Elastic modulus (MPa)</th>
<th>Elongation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coupon-1</td>
<td>512.6</td>
<td>620.2</td>
<td>200947</td>
<td>15.0 (13.15)</td>
</tr>
<tr>
<td>Coupon-2</td>
<td>523.1</td>
<td>629.9</td>
<td>198735</td>
<td>12.3 (8.7)</td>
</tr>
<tr>
<td>Coupon-3</td>
<td>536.1</td>
<td>643.9</td>
<td>199050</td>
<td>13.4 (10.15)</td>
</tr>
<tr>
<td>Average</td>
<td>524</td>
<td>631</td>
<td>199577</td>
<td></td>
</tr>
</tbody>
</table>

Table C.13 Shear Connector (19 Diameter) (Coupon: 12 mm diameters)

<table>
<thead>
<tr>
<th>Coupon</th>
<th>Yield stress (MPa)</th>
<th>Ultimate stress (MPa)</th>
<th>Elastic modulus (MPa)</th>
<th>Elongation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coupon-1</td>
<td>354.8</td>
<td>513.8</td>
<td>199511</td>
<td>23.1 (28.9)</td>
</tr>
<tr>
<td>Coupon-2</td>
<td>381.9</td>
<td>516.8</td>
<td>205612</td>
<td>25.5 (27.3)</td>
</tr>
<tr>
<td>Coupon-3</td>
<td>387.6</td>
<td>521.5</td>
<td>206700</td>
<td>24.6 (27.5)</td>
</tr>
<tr>
<td>Average</td>
<td>375</td>
<td>517</td>
<td>203941</td>
<td></td>
</tr>
</tbody>
</table>
Figure C.3 Stress-strain curves of the coupon test of carbon steel tube (Circular tube-6 mm thickness)

Figure C.4 Stress-strain curves of the coupon test of profile sheet (1 mm Bondek profile sheet)

Figure C.5 Stress-strain curves of the coupon test of steel beam flange (310UB40.4)

Figure C.6 Stress-strain curves of the coupon test of steel beam web (310UB40.4)
Figure C.7 Stress-strain curves of the coupon test of endplate (10 mm)

Figure C.8 Stress-strain curves of the coupon test of through plate (12 mm)

Figure C.9 Stress-strain curves of the coupon test of M20 Hollo-Bolts

Figure C.10 Stress-strain curves of the coupon test of shear connector (19 Diameter shear stud, Coupon: 12 mm diameter)
C.3 Mechanical Properties of Concrete Materials

Table C.14 Concrete strength at 7 days

<table>
<thead>
<tr>
<th></th>
<th>Ultimate compressive strength (MPa)</th>
<th>Load (kN)</th>
<th>Area (mm²)</th>
<th>Testing Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cylinder-1</td>
<td>22.2</td>
<td>178.8</td>
<td>8065.9</td>
<td>2014-04-10</td>
</tr>
<tr>
<td>Cylinder-2</td>
<td>21.6</td>
<td>175.0</td>
<td>8082.6</td>
<td>2014-04-10</td>
</tr>
<tr>
<td>Cylinder-3</td>
<td>21.5</td>
<td>172.4</td>
<td>8003.9</td>
<td>2014-04-10</td>
</tr>
<tr>
<td>Average</td>
<td>21.77</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table C.15 Concrete strength at 28 day:

<table>
<thead>
<tr>
<th></th>
<th>Ultimate compressive strength (MPa)</th>
<th>Load (kN)</th>
<th>Area (mm²)</th>
<th>Testing Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cylinder-1</td>
<td>33.2</td>
<td>266.6</td>
<td>8040.4</td>
<td>2014-05-01</td>
</tr>
<tr>
<td>Cylinder-2</td>
<td>33.9</td>
<td>267.7</td>
<td>7885.4</td>
<td>2014-05-01</td>
</tr>
<tr>
<td>Cylinder-3</td>
<td>36.7</td>
<td>249.1</td>
<td>8054.7</td>
<td>2014-05-01</td>
</tr>
<tr>
<td>Average</td>
<td>34.6</td>
<td></td>
<td></td>
<td>2014-05-01</td>
</tr>
</tbody>
</table>
Table C.16 Concrete strength at 77 days

<table>
<thead>
<tr>
<th>Ultimate compressive strength (MPa)</th>
<th>Load (kN)</th>
<th>Area (mm²)</th>
<th>Testing Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cylinder-1</td>
<td>38.5</td>
<td>306.0</td>
<td>7939.8</td>
</tr>
<tr>
<td>Cylinder-2</td>
<td>35.0</td>
<td>275.2</td>
<td>7860.3</td>
</tr>
<tr>
<td>Cylinder-3</td>
<td>33.6</td>
<td>264.6</td>
<td>7884.6</td>
</tr>
<tr>
<td>Average</td>
<td>35.7</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table C.17 Concrete strength at 88 days

<table>
<thead>
<tr>
<th>Ultimate compressive strength (MPa)</th>
<th>Load (kN)</th>
<th>Area (mm²)</th>
<th>Testing Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cylinder-1</td>
<td>30.1</td>
<td>243.4</td>
<td>8093.8</td>
</tr>
<tr>
<td>Cylinder-2</td>
<td>29.7</td>
<td>234.3</td>
<td>7897.2</td>
</tr>
<tr>
<td>Cylinder-3</td>
<td>30.0</td>
<td>236.0</td>
<td>7879.9</td>
</tr>
<tr>
<td>Average</td>
<td>29.93</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table C.18 Concrete strength at 132 days

<table>
<thead>
<tr>
<th>Ultimate compressive strength (MPa)</th>
<th>Load (kN)</th>
<th>Area (mm²)</th>
<th>Testing Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cylinder-1</td>
<td>39.8</td>
<td>322.5</td>
<td>8099.3</td>
</tr>
<tr>
<td>Cylinder-2</td>
<td>43.3</td>
<td>348.1</td>
<td>8036.5</td>
</tr>
<tr>
<td>Cylinder-3</td>
<td>43.8</td>
<td>348.3</td>
<td>7960.4</td>
</tr>
<tr>
<td>Average</td>
<td>42.3</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table C.19 Concrete strength at 146 days

<table>
<thead>
<tr>
<th>Ultimate compressive strength (MPa)</th>
<th>Load (kN)</th>
<th>Area (mm²)</th>
<th>Testing Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cylinder-1</td>
<td>40.3</td>
<td>276.0</td>
<td>8040.4</td>
</tr>
<tr>
<td>Cylinder-2</td>
<td>33.3</td>
<td>263.8</td>
<td>7925.6</td>
</tr>
<tr>
<td>Cylinder-3</td>
<td>30.8</td>
<td>248.4</td>
<td>8069.9</td>
</tr>
<tr>
<td>Average</td>
<td>32.8</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table C.20 Concrete strength at 153 days

<table>
<thead>
<tr>
<th>Ultimate compressive strength (MPa)</th>
<th>Load (kN)</th>
<th>Area (mm²)</th>
<th>Testing Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cylinder-1</td>
<td>40.3</td>
<td>325.8</td>
<td>8075.4</td>
</tr>
<tr>
<td>Cylinder-2</td>
<td>41.6</td>
<td>334.5</td>
<td>8044.4</td>
</tr>
<tr>
<td>Cylinder-3</td>
<td>41.6</td>
<td>334.5</td>
<td>8044.4</td>
</tr>
<tr>
<td>Average</td>
<td>41.2</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table C.21 Concrete strength at 174 days

<table>
<thead>
<tr>
<th>Ultimate compressive strength (MPa)</th>
<th>Load (kN)</th>
<th>Area (mm²)</th>
<th>Testing Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cylinder-1</td>
<td>41.8</td>
<td>325.8</td>
<td>8035.7</td>
</tr>
<tr>
<td>Cylinder-2</td>
<td>41.5</td>
<td>333.0</td>
<td>8019.8</td>
</tr>
<tr>
<td>Cylinder-3</td>
<td>36.2</td>
<td>286.6</td>
<td>7926.4</td>
</tr>
<tr>
<td>Average</td>
<td>39.8</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
APPENDIX D

CALCULATION DETAILS FOR JOINT PROPERTIES

D.1 General Information of Joints

Table D.1 Geometry related parameters of a joint

<table>
<thead>
<tr>
<th>Items</th>
<th>Value in mm</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>CFST column</strong></td>
<td></td>
</tr>
<tr>
<td>Width (mm)</td>
<td>360</td>
</tr>
<tr>
<td>Thickness (mm)</td>
<td>6</td>
</tr>
<tr>
<td><strong>Beam</strong></td>
<td></td>
</tr>
<tr>
<td>Flange width</td>
<td>165</td>
</tr>
<tr>
<td>Flange thickness</td>
<td>10.1</td>
</tr>
<tr>
<td>Depth of web</td>
<td>304</td>
</tr>
<tr>
<td>Thickness of web</td>
<td>6.1</td>
</tr>
<tr>
<td><strong>Endplate</strong></td>
<td></td>
</tr>
<tr>
<td>Width</td>
<td>230</td>
</tr>
<tr>
<td>Depth</td>
<td>310</td>
</tr>
<tr>
<td>Thickness</td>
<td>10</td>
</tr>
<tr>
<td>Bolt gauge (g)</td>
<td>100</td>
</tr>
<tr>
<td>Bolt pitch (p)</td>
<td>87</td>
</tr>
<tr>
<td>Bolt edge distance (h), e</td>
<td>65</td>
</tr>
<tr>
<td>Bolt edge distance (v), e</td>
<td>65</td>
</tr>
<tr>
<td><strong>Bolt</strong></td>
<td></td>
</tr>
<tr>
<td>Bolt diameter</td>
<td>20</td>
</tr>
<tr>
<td>Bolt hole diameter</td>
<td>32.6</td>
</tr>
<tr>
<td>Sleeve thickness</td>
<td>5.8</td>
</tr>
<tr>
<td><strong>Slab</strong></td>
<td></td>
</tr>
<tr>
<td>Width</td>
<td>900</td>
</tr>
<tr>
<td>Depth</td>
<td>120</td>
</tr>
<tr>
<td>Cover</td>
<td>26</td>
</tr>
<tr>
<td><strong>Reinforcement</strong></td>
<td></td>
</tr>
<tr>
<td>Diameter</td>
<td>12</td>
</tr>
<tr>
<td>Total number</td>
<td>4</td>
</tr>
<tr>
<td><strong>Shear stud</strong></td>
<td></td>
</tr>
<tr>
<td>Diameter</td>
<td>19</td>
</tr>
<tr>
<td>Total number of shear stud</td>
<td>8</td>
</tr>
<tr>
<td>Spacing from column face</td>
<td>100</td>
</tr>
<tr>
<td><strong>Other parameters</strong></td>
<td></td>
</tr>
<tr>
<td>$D_c$ = 120</td>
<td></td>
</tr>
<tr>
<td>$H_b$ = 304</td>
<td></td>
</tr>
<tr>
<td>$D_b$ = 233.95</td>
<td></td>
</tr>
<tr>
<td>$D_t$ = 392.95</td>
<td></td>
</tr>
</tbody>
</table>
Table D.2 Materials properties of different components of joints

<table>
<thead>
<tr>
<th>Items</th>
<th>Yield stress (MPa)</th>
<th>Ultimate stress (MPa)</th>
<th>Elastic modulus (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CFST column</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Steel tube</td>
<td>339</td>
<td>714</td>
<td>200985</td>
</tr>
<tr>
<td>Concrete</td>
<td>35</td>
<td>35</td>
<td>27983</td>
</tr>
<tr>
<td>Beam</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flange</td>
<td>352</td>
<td>535</td>
<td>198494</td>
</tr>
<tr>
<td>Web</td>
<td>370</td>
<td>534</td>
<td>203765</td>
</tr>
<tr>
<td>Endplate</td>
<td>388</td>
<td>506</td>
<td>206298</td>
</tr>
<tr>
<td>Bolt</td>
<td>890</td>
<td>953</td>
<td>218871</td>
</tr>
<tr>
<td>Profile sheet</td>
<td>615</td>
<td>617</td>
<td>216122</td>
</tr>
<tr>
<td>Reinforcement</td>
<td>538</td>
<td>653</td>
<td>200371</td>
</tr>
<tr>
<td>Shear stud</td>
<td>375</td>
<td>517</td>
<td>203941</td>
</tr>
</tbody>
</table>

D.2 Rotational Stiffness of Joints

D.2.1 Stiffness of slab reinforcement

\[ k_r = \frac{E_s A_r}{l_r} \]

= 504 kN/mm

Here

\( A_r = \) total area of the slab reinforcement bars = \( 4 \times \pi (6)^2 = 452.29 \text{ mm}^2 \)

\( E_s = \) modulus of elasticity of the reinforcement bar = 200371 MPa

\( l_r = \) effective length of the reinforcement bar = \( \frac{h_c}{2} = 180 \text{ mm} \)

\( h_c = \) depth of the CFST column = 360 mm

D.2.2 Stiffness of shear connectors

\[ k_s = \begin{cases} 
- \frac{0.5 \lambda N_{sc} F_{sc,max}}{\eta} \ln \left( 1 - \frac{(0.5) \bar{\sigma}}{\eta} \right) & \text{for } \eta \leq 1 \\
- \frac{0.5 \lambda N_{sc} F_{sc,max}}{\eta} \ln \left( 1 - \frac{(0.5) \bar{\sigma}}{\eta} \right) & \text{for } \eta > 1
\end{cases} \]

\( k_s = 896 \text{ kN/mm} \)

where \( \alpha \) is a non-dimensional parameter taken as 0.8 and \( \lambda \) as 0.7

\[ \eta = \frac{N_{sc} F_{sc,max}}{A_{refy,r}} = 3.41 > 1 \]
\[ F_{sc,\text{max}} = \min \left\{ 0.8 f_u \left( \frac{\pi d^2}{4} \right), 0.37 \sqrt{f' c E_c \left( \frac{\pi d^2}{4} \right)} \right\} \]

\[ F_{sc,\text{max}} = \min (117.3, 103.8) = 103.8 \text{ kN} \]

**D.2.3 Stiffness of column face in bending**

\[ k_{cb} = \frac{16 E t_b^2}{(B- t_{tb})^2 (1- \beta)^3 + 10.4 (1- \mu) f' c^2} = 17 \text{ kN/mm} \]

\[ \alpha = \frac{c}{B-t_{tb}} = 0.178 \]

\[ \beta = \frac{g+c}{B-t_{tb}} = 0.46 \]

\[ \mu = \frac{B-t_{tb}}{t_{tb}} = 59 \]

\[ \theta = 35 - 10 \frac{g+c}{B-t_{tb}} = 30.40 \]

\[ c = k_{ts} \times d_h = 62.84 \]

\[ B = \text{width of the steel tube of CFST column} = 360 \text{ mm} \]

\[ t_{tb} = \text{thickness of the steel tube of CFST column} = 6 \text{ mm} \]

\[ g = \text{bolts gauge (horizontal)} = 100 \text{ mm} \]

\[ d_h = \text{bolt hole diameter} = 32.6 \text{ mm} \]

\[ E = \text{modulus of elasticity of the steel tube} = 218871 \text{ MPa} \]

\[ k_{ts} = 1.75656 + 0.0046268 \mu_1 - 1.0416 \beta_1 - 0.000060718 f' c^2 + 0.0083156 f' c = 1.928 \]

where \( \beta_1 = \text{bolt gauge to hollow section width ratio} = \frac{g}{B} = 0.278 \)

\[ \mu_1 = \text{hollow section face slenderness ratio} = \frac{B}{t_{tb}} = 60 \]

\[ f' c = \text{compressive strength of concrete} = 35 \text{ MPa} \]

**D.2.4 Stiffness of endplate in bending**

\[ k_{pb} = \frac{6 E I_p}{m^3} = 273.9 \text{ kN/mm} \]

\[ E = \text{modulus of elasticity of endplate} = 206298 \text{ MPa} \]

\[ I_p = \text{moment of inertia of endplate} \]

\[ m = \text{the distance from one bolt line to the centre of the beam flange} = 50 \text{ mm} \]

for flat endplate
for curved endplate

\[ I_p = \frac{b_p t_p^3}{12} = 19166.67 \text{ mm}^4 \]

Here, \( b_p \) = width of flat endplate = 230 mm

\( t_p \) = thickness of flat endplate = 10 mm

\( R \) = outer diameter of curved endplate from the centre of the circular column=190 mm

\( r \) = inner diameter of curved endplate from the centre of the circular column= 180 mm

**D.2.5 Stiffness of blind-bolt in tension**

\[ k_{bt} = \frac{1}{k_{bsh} + k_{cst}} = 305 \text{ kN/mm} \]

\( k_{bsh} \) = stiffness of bolts shank = \( \frac{E_s A_s}{L_b} \) = 1140.9 kN/mm

\( E_s \) = elastic modulus of bolt shank = 218871 MPa

\( A_s \) = tensile stress area of bolt shank = 245 mm²

\( L_b \) = effective length of the Hollo-Bolt = \( t_p + t_{tb} + t_w + \frac{t_{bh} + t_c}{2} \) = 47 mm

\( t_{bh} \) = thickness of the hexagon bolt head = 15 mm

\( t_w \) = thickness of the collar of the Hollo-Bolt = 10 mm

\( t_c \) = depth of the cone of the Hollo-Bolt = 27 mm

\( t_p \) = thickness of the endplate = 10 mm

\( t_{tb} \) = thickness of the steel tube = 6 mm

\( k_{cst} \) = stiffness of the sleeve bearing on the concrete = \( \frac{E_c A_{sib}}{L_{sib}} \) = 416.8 kN/mm

\( E_c \) = elastic modulus of concrete = 27983 MPa

\( A_{sib} \) = bearing area of sleeve = \( \pi (R - r)\sqrt{R^2 + (R - r)^2} \) = 416.32 mm²

\( r = \frac{d_h}{2} = 16.3 \text{ mm} \)

\( R = \frac{d_{tcm}}{2} + t_s = 22.1 \text{ mm} \)

\( L_{cst} = \frac{t_c}{\cos \alpha} = 27.95 \text{ mm} \)

\( d_h \) = outer diameter of sleeve = 32.6 mm

\( t_s \) = thickness of sleeve = 5.8 mm

\( d_{tcm} \) = the maximum diameters of the upper threaded cone=32.6 mm

\( \alpha \) = the slope angle of cone=15°
D.2.6 Stiffness of endplate bearing on steel tube in compression

\[ k_{cp} = \begin{cases} \frac{E_p b_{eff} l_{eff}}{(t_p + t_{tb})} & \text{for flat endplate} = 3816.51 \text{ kN/mm} \\ \pi E_p \alpha b_{eff} r \frac{l_{eff}}{(t_p + t_{tb})} & \text{for curved endplate} = 5189.72 \text{ kN/mm} \end{cases} \]

- \( E_p \) = modulus of elasticity of endplate = 206298 MPa
- \( t_p \) = thickness of the endplate = 10 mm
- \( t_{tb} \) = thickness of the steel tube = 6 mm
- \( r \) = radius of the internal surface of the curved endplate = 180 mm
- \( \alpha \) = angle of the curved endplate from side edge to its centreline = 39.7 degree (see Figure 9.6, Chapter 9)
- \( b_{eff} = t_{bf} + 2t_p = 30.1 \) mm
- \( l_{eff} = b_{bf} + 2t_p = 185 \) mm
- \( b_{bf} \) = the width of the beam flange = 165 mm
- \( t_{bf} \) = the thickness of the beam flange = 10.1 mm
- \( t_p \) = the endplate thickness = 10 mm

D.2.7 Stiffness of concrete core in compression

\[ k_{cc} = \begin{cases} E_c (B - 2t_{tb}) & \text{for flat endplate} = 9738 \text{ kN/mm} \\ 2 \sin \alpha (D - 2t_{tb}) E_c & \text{for curved endplate} = 12655.27 \text{ kN/mm} \end{cases} \]

- \( B \) = the width of the square steel tube = 360 mm
- \( D \) = the diameter of the circular steel tube = 360 mm
- \( t_{tb} \) = the thickness of the steel tube = 6 mm
- \( E_c \) = the modulus of elasticity of core concrete of the CFST column
- \( \alpha \) = the angle as shown in Figures 9.6 in Chapter 9 = 39.7 degree

Therefore, rotational stiffness of joint

\[ S_{jc} = \frac{D_t^2 H_b^l \left( \frac{1}{k_b} + \frac{1}{k_c} \right) + D_b D_r H_b^l \frac{1}{k_r} + D_r^2 D_b \left( \frac{1}{k_s} - \frac{1}{k_c} \right)}{\left( \frac{1}{k_r} + \frac{1}{k_c} \right) \left( \frac{1}{k_b} + \frac{1}{k_c} \right) H_b^l + \frac{1}{k_s} \left( \frac{1}{k_b} + \frac{1}{k_c} \right) D_r - \frac{1}{k_c} H_b^l} \]

\[ = 42.7 \text{ kN-m/mrad} \]
D.3 Moment Capacity of Joints

As, \( F_c < F_t \),

Ultimate moment resistance capacity composite joints can be determined as:

\[
M_u = R_rD_r + R_{ss}(H_b + y_{ss}) + R_b.d_b - R_{bw}\left(\frac{y_c + \frac{t_{bf}}{2}}{2}\right)
\]

Here,

- \( D_r \) = distance of the reinforcement from beam bottom flange centre = 392.95 mm
- \( d_b \) = distance of the top bolt row from the beam bottom flange centre = 233.95 mm
- \( H_b + y_{ss} \) is the distance between centroids of the beam bottom flange and profile sheet
  \( = (H_b - \frac{t_{bf}}{2}) + y_{ss} = 304-10.1/2+14.3 = 313.25 \) mm
- \( y_c \) = position of the neutral axis from the centre of the beam bottom flange and determined as:
  \[
  y_c = \frac{(R_r + R_{ss} + R_b - R_f)}{t_{bw}f_{bw}}
  \]
- \( t_{bf} \) = beam flange thickness = 10.1 mm
- \( t_{bw} \) = beam web thickness = 6.1 mm
- \( f_{bw} \) = yield strength of the beam web = 370 MPa
- \( R_r \) = tensile resistance capacity of reinforcement or shear connectors
- \( R_{ss} \) = tensile resistance capacity of profile sheet
- \( R_b \) = the tensile resistance capacity of top bolt row
- \( R_f \) = the compressive resistance capacity of beam flange
- \( R_w \) = the compressive resistance capacity of beam web

Figure D.1 Stress blocks of flush endplate connection’s components

(i) \( F_c \geq F_t \)  
(ii) \( F_c < F_t \)
D.3.1 Tensile resistance of reinforcement ($R_r$):
The tensile resistance capacity of reinforcement ($R_r$) is calculated from shear connectors, otherwise from slab reinforcement.

$$R_r = \min \left\{ \frac{A_r f_{y,r}}{N_{sc} A_{sc} f_{u,sc}} = 243.4 \text{ kN} \right\}$$

$$R_r = 243.4 \text{ kN}$$

Here,

- $f_{y,r}$ = yield strength of reinforcement bars = 538 MPa
- $A_r$ = total cross-sectional area of reinforcement = 452.39 mm$^2$
- $N_{sc}$ = total number of shear connectors = 8
- $A_{sc}$ = cross-sectional area of shear connectors = 283.53 mm$^2$
- $f_{u,sc}$ = ultimate strength of the shear connectors = 517 MPa

D.3.2 Tensile resistance of profile sheet ($R_{ss}$):
The tensile resistance capacity of profile sheet ($R_{ss}$) can be determined as:

$$R_{ss} = A_{ss} f_{y,ss} = 557.3 \text{ kN}$$

Here,

- $A_{ss}$ = the effective cross-section area of the profile sheet
- $A_{ss} = (b_e - B) \times$ cross-sectional area of profile steel per meter
- $= (900 - 260) \times 1678 = 906.12 \text{ mm}^2$
- $b_e$ = effective width of slab = 900 mm
- $B$ = width of the CFST column = 360 mm
- $f_{y,ss}$ = the yield strength of profile sheet = 615 MPa

D.3.3 Tensile resistance of top bolt row ($R_b$):
The tensile resistance capacity of top bolt row ($R_b$) is determined as:

$$R_b = \min \left( R_{pb}, R_b, R_{cb} \right) = 94.1 \text{ kN}$$

i) Determination of resistance capacity of endplate in bending ($R_{pb}$):
The resistance capacity of endplate in bending ($R_{pb}$) is determined from the equation proposed by Li et al. (1996b).

$$R_{pb} = 5.5 - 0.021 m_c + 0.017e) t_{pb}^2 f_{y,pb} = 218 \text{ kN}$$

Here,

- $m_c$ = distance from the bolt centre to the beam web = $(g/2 - t_{tw}/2) = 46.95 \text{ mm}$
\( e \) = distance from the bolt centre to the outer edge of the plate \( = 65 \) mm
\( t_{pb} \) = thickness of the endplate \( = 10 \) mm
\( f_{y,pb} \) = endplate yield strength \( = 388 \) MPa

**ii) Determination of resistance capacity of blind-bolts in tension \((R_b)\):**

The tensile resisting capacity of blind-bolt is calculated as

\[
R_b = A_b f_{y,b} = 218.1 \text{kN}
\]

\( A_b \) = tensile cross-sectional area of Hollo-Bolt \( = 245 \text{ mm}^2 \)
\( f_{y,b} \) = yield strength of Hollo-Bolt \( = 890 \) MPa

**iii) Determination of resistance capacity of steel tube in bending \((R_{cb})\):**

The resistance capacity of steel tube in bending \((R_{cb})\) is determined from followings expression.

\[
R_{cb} = \frac{f_{y,c} t_{cb}^2}{2} \left[ \pi \left(1 + \frac{r}{r - \frac{c}{2}} \right) + \frac{g}{r} \right]
\]

\( = 94.1 \text{kN} \)

Here,
\( B \) = width of CFST column \( = 360 \) mm
\( t_{cb} \) = thickness of the steel tube of CFST column \( = 6 \) mm
\( g \) = bolt gauge (horizontal) \( = 100 \) mm
\( d_h \) = outer diameters of sleeve of the Hollo-bolt \( = 32.6 \) mm
\( r \) = radius of circular yield line can be determined from \( r = \frac{B - g - t_{tb}}{2} = 127 \) mm
\( c = k_{yf} d_h = 42.77 \)
\( k_{yf} \) = yield line force calibration factor.
\( k_{yf} = 0.84274 + 0.0054463\mu + 0.3164\beta + 0.0010883f'_c \)
\( = 1.312 \)
\( \mu \) = slenderness ratio of the hollow section \( = \frac{B}{t_{tb}} = 60 \)
\( \beta \) = bolts gauge (horizontal) to hollow section width ratio \( = \frac{g}{B} = 0.278 \)
\( f'_c \) = characteristics strength of concrete \( = 35 \) MPa.
D.3.4 Compressive resistance of beam bottom flange \( (R_f) \) and beam web \( (R_w) \): 

The design strength of beam bottom flange depends on the capacity of the beam flange in compression as well as the beam flange in local buckling. The equation recommended by Eurocode 3 (2001) is used to calculate the resisting capacity of beam bottom flange.

\[
R_f = \min \begin{cases} 
  t_{bf} b_{bf} f_{bf} \text{ for beam flange in compression} \\
  22 t_{bf}^2 f_{pb} \text{ for beam flange in local buckling}
\end{cases}
\]

Here,

\[
b_{bf} = \text{width of the beam flange} = 165 \text{ mm} \\
t_{bf} = \text{thickness of the beam flange} = 10.1 \text{ mm} \\
f_{bf} = \text{yield strength of the beam flange} = 352 \text{ MPa} \\
\frac{b_{bf}}{t_{bf}} = 16.3 < 18 \\
22 \sqrt[2]{\frac{235}{f_{bf}}} = 18
\]

Therefore, the compressive resistance of beam bottom flange

\[
R_f = t_{bf} b_{bf} f_{bf} = 586.6 \text{ kN}
\]

\[
F_c = R_f = 586.6 \text{ kN}
\]

\[
F_t = R_r + R_{ss} + R_b = 243.4 + 557.3 + 94.1 = 894.8 \text{ kN}
\]

As the value of \( F_c \) (586.6 kN) is lower than the value of the \( F_t \) (894.8 kN)

So, additional compressive force resisted by beam web

\[
= R_r + R_{ss} + R_b - R_f = 308.3 \text{ kN}
\]

The depth of the stress block of the beam web in compressive \( (y_c) \):

\[
y_c = \frac{(R_r+R_{ss}+R_b-R_f)}{t_{bw} f_{p,bw}} = 113.8 \text{ mm}
\]

Here,

\[
f_{p,bw} = 1.2 f_{bw} = 444 \text{ MPa} \\
t_{bw} = \text{thickness of the beam web} = 6.1 \text{ mm} \\
f_{bw} = \text{yield strength of the beam web} = 370 \text{ MPa}
\]

The resistance capacity of the beam web \( (R_w) \) = \( y_c t_{bw} f_{bw} \) = 308.22 kN
Therefore, ultimate moment capacity of joints
\[ M_u = R_r D_r + R_{ss} (H_b + y_{ss}) + R_b d_b - R_{bw} (y_c + t_{bf}) / 2 \]
\[ = 243.4 \times 392.5 + 557.3 \times ((304 - 10.1/2) + 14.3) + 94.1 \times -308.22 \times (113.8 + 10.1)/2 \]
\[ = 273.1 \text{kN-m} \]

**D.4 Rotation Capacity of Joints**

(i) If \( F_f \geq F_r \).
\[ \phi_u = \frac{\Delta_r}{D_r} + \frac{\Delta_s}{H_b - 0.5 t_{bf}} \]

(ii) If \( F_f < F_r \).
\[ \phi_u = \frac{\Delta_r}{D_r} + \frac{\Delta_s + \Delta_a}{H_b - 0.5 t_{bf}} \]

Here \( F_f \) is the compressive resistance of the beam bottom flange and \( F_r \) is the tensile resistance of slab reinforcement.

**D.4.1 Determination of slab reinforcement deformation (\( \Delta_r \))**

Case 1 – Shear stud failure (for \( \rho < 1\% \) and \( \eta_y \leq 1 \)):
\[ \Delta_r = \left( \frac{D_{cd}}{2} + P_0 + P_1 \right) \times \varepsilon_r \]

Case 2 – Reinforcement fracture (for \( \rho \leq 1 \% \) and \( \eta_y > 1 \))
\[ \Delta_r = 2 \times L_t \times \varepsilon_{smu} \]

Case 3 – Reinforcement yielding and buckling of beam flange (for \( \rho > 1 \% \) and \( \eta_y > 1.2 \))
\[ \Delta_r = \begin{cases} \left( \frac{D_{cd}}{2} + L_t \right) \times \varepsilon_{smu} & \text{when } P_0 + P_1 < L_t \\ \left( \frac{D_{cd}}{2} + L_t \right) \times \varepsilon_{smu} + (P_0 + P_1 - L_t) \times \varepsilon_r & \text{when } P_0 + P_1 < L_t \end{cases} \]

in which,
\[ p = \text{reinforcement ratio} = A_r / A_c = 0.76\% \]
\[ A_r = \text{area of longitudinal reinforcement} = 452.39 \text{ mm}^2 \]
\[ A_c = \text{area of concrete slab} = (900 \times (360-54) = 9400 \text{ mm}^2 \]
\[ D_{cd} = \text{overall column depth} = 360 \text{ mm} \]
\[ P_0 = \text{distance of the first shear connector from the column face} = 75 \text{ mm} \]
\[ P_1 = \text{the shear stud spacing from centre to centre} = 180 \text{ mm} \]
\[ L_t = \text{transmission length between cracks} = \frac{f_{ctm} k_c \phi}{4 \tau_{sm} \rho} = 117.53 \text{ mm} \]
\[ k_c = \frac{1}{1 + \frac{h_c}{2z_0}} = 0.53 \]

\[ \phi = \text{diameter of reinforcement bars} = 12 \text{ mm} \]

\[ \tau_{sm} = \text{average bond stress along} \ L_t = 1.8 \ f_{ctm} = 5.78 \text{ MPa} \]

\[ f_{ctm} = \text{mean tensile strength of concrete according Eurocode 4 (2004)} \]

\[ f_{ctm} = \begin{cases} 
(0.3 f_{ck})^{2/3} & \leq C50/60 \\
2.12 \ln(1 + (f_{ck} + 8)/10) & > C50/60 
\end{cases} \]

\[ = 3.2 \text{ MPa} \]

\[ h_c = \text{thickness of the concrete slab, excluding any haunch or ribs} \]

\[ = D_c - \text{rib} = 120 - 54 = 93 \text{ mm} \]

\[ D_c = \text{concrete slab thickness} = 120 \text{ mm} \]

\[ z_0 = \text{distance between centroids of uncracked concrete slab and uncracked composite section using modular ratio for short term effects} \]

\[ z_0 = y_1 - y_2 = 377.5 - 324.02 = 53.48 \]

where \( y_1 = H_b + D_c - h_c/2 = 304 + 120 - 93/2 = 377.5 \)

and \( y_2 = \frac{A_s H_b/2 + A_c y_1}{A_s + A_c} = 324.02 \)

\[ \varepsilon_{mu} = \text{ultimate average strain of reinforcement} \]

\[ = \varepsilon_{sy} - \beta_t \Delta \varepsilon_{sr} + \delta \left( 1 - \frac{\Delta \sigma_{sr1}}{f_{ys}} \right) (\varepsilon_{su} - \varepsilon_{DY}) = 0.02685 \]

\[ \varepsilon_{sy} = \text{strain at yield stress of reinforcement bar} = 10 \times 538 / 200371 = 0.02685193 \]

\[ f_{ys} = \text{reinforcement yield stress} = 538 \text{ MPa} \]

\[ E_s = \text{elastic modulus of reinforcement} = 200371 \text{ MPa} \]

\[ \varepsilon_{us} = \text{reinforcement strain at fracture} = 0.1726 \]

\[ \beta_t = 0.4 \text{ for short-term loading,} \]

\[ \delta = 0.8 \text{ for high ductility deformed bars} \]

\[ \Delta \varepsilon_{sr} = \text{increase of reinforcement strain at first crack} = \frac{f_{ctm} k_c}{E_s \rho} = 0.00113 \]

\[ \sigma_{sr1} = \text{reinforcement stress in the first crack} = \frac{f_{ctm} k_c}{\rho} \left( 1 + \rho \frac{E_{sr}}{E_c} \right) = 237.75 \text{ MPa} \]

\[ E_c = \text{elastic modulus of concrete} = 27983 \text{ MPa} \]

Therefore, \( \Delta_r = 2 \times L_c \times \varepsilon_{smu} = 6.286 \text{ mm} \) (since, \( 0.76 \% < 1 \) and \( \eta_y = 3.49 > 1 \))
D.4.2 Determination of shear connector slip of \( \Delta_s \)

\[
\Delta_s = \frac{F}{N_{sc} k_{sc}}
\]

\( N_{sc} \) = number of the shear connectors in the hogging region = 8

\( k_{sc} \) = stiffness of a single shear connector = 1.47

\[
F_{sc,\text{max}} = \min \left\{ 0.8 f_u \frac{\pi d^2}{4}, 0.37 \sqrt{f'_c E_c} \frac{\pi d^2}{4} \right\}
\]

\( F_{sc,\text{max}} = \min (117.3, 103.8) = 103.8 \text{ kN} \)

\[
F = \min \left\{ A_r f_{y,r} \left( \frac{\pi d^2}{4} \right), N_{sc} F_{sc,\text{max}} \right\}
\]

when \( \eta_y \geq 1 \)

when \( \eta_y < 1 \)

\[
= \min (243.4, 1172.7) = 243.4 \text{ kN}
\]
## D.5 Moment-Rotation Relationship Model

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