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HYBRID CONNECTION IN PRECAST CONSTRUCTION

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**SCHOOL OF CIVIL & ENVIRONMENTAL ENGINEERING
NANYANG TECHNOLOGICAL UNIVERSITY**

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School of Civil & Environmental Engineering

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SUMMARY

In this research, a new hybrid connection has been proposed to utilise the ease of steelwork installation for traditional precast concrete construction. The primary objective of this research is to study the behaviour and suitability of this kind of hybrid beam-column connection. Experimental results and analytical models have been used in evaluating the characteristics and suitability of the proposed hybrid connection.

An experimental programme was carried out to study the behaviour of hybrid connections under monotonic loading as well as cyclic sway loading. A total of twelve specimens comprising bare steel connections, partial hybrid connections and complete hybrid connections were tested. Three loading cases, namely monotonic hogging moment, monotonic sagging moment and cyclic sway loading were considered. The experimental findings were used in developing the component-based mechanical model to predict the behaviour of hybrid connection under various loadings.

A mechanical model and a component-based model have been proposed to predict the behaviour of single plate and hybrid connections, respectively. The mechanical model is developed to analyse the moment-rotation relationship of the single plate connection under both monotonic and cyclic loading. The predicted results have good agreement with the experimental results. This model can be incorporated into the component-based model which is proposed for the analysis of the hybrid connection. The component-based model is based upon the principles of force transfer mechanism, geometrical configuration and material properties. The adopted methodology is based on the assembly of contributions from various connection components, such as reinforcement, concrete, single plate connection, etc. The formulation takes into account the nonlinear behaviour of each component in the hybrid connection. For hybrid connections under monotonic loading, the effectiveness and accuracy of the component-based model has been demonstrated by the verifications against the test results. Moreover, the component-based model

can be extended to predict the behaviour of the hybrid connection under cyclic loading through the introduction of Damage Index (DI) to characterise the reduction of concrete stiffness and decay of concrete strength under cyclic loading due to the damage of concrete.

The predicted moment-rotation relationships are incorporated into the connector elements in a finite element program for semi-rigid frame analysis. Comparisons are conducted between frames with pinned, rigid and semi-rigid connections. The two extreme assumptions (pinned and rigid) of idealized connection behaviour may lead to substantial inaccuracies in the overall response assessment. Frame analysis with semi-rigid connections provides not only the accurate moment distribution, but also the realistic results of beam deflections as well as frame lateral deformations.

A simplified and yet reliable design method for hybrid connections is proposed for practical purpose. The simplified method is for the determination of moment capacity, rotation capacity as well as moment-rotation relationship for hybrid connections. The proposed semi-rigid frame design approach can more realistically account for the connection behaviour and produce more appropriate member sizes than the traditional pinned joint or rigid joint design method. In this semi-rigid method, the properties of connections are also treated as design variables, so that both the members and the connections can be optimised. Therefore, this approach is more general and provides a greater flexibility than the traditional methods.

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LIST OF SYMBOLS

$A_{s,top}$:	Area of top rebars in the concrete beam;
$A_{s,bot}$:	Area of bottom rebars in the concrete beam;
A_{sm} :	Area of mesh reinforcements in the topping concrete;
b_p :	Width of precast beam;
$b_{topping}$:	Width of topping concrete;
d_b :	Diameter of bolt;
d_{sm} :	Distance from bottom of connection to the centre of mesh;
d_{top} :	Distance from bottom of connection to the centre of top rebars;
d_{bottom} :	Distance from top of connection to the centre of bottom rebars;
d_{ch} :	Depth of compression centre under hogging moment;
d_{cs} :	Depth of compression centre under sagging moment;
DI :	Damage index;
E_s :	Elastic modulus of structural steel;
E_c :	Elastic modulus of concrete;
f_y :	Yield strength of steel;
f_{ym} :	Yield strength of mesh steel;
f_{cu} :	Cube strength of concrete;
f_{rp} :	Concrete modulus of rupture;
f_{uweb} :	Ultimate strength of beam web;
$F_{bearing}$:	Ultimate bearing capacity of ordinary single plate connection;
$F_{r,top}$:	Force in the top reinforcements;
F_s :	Force between HSFG bolt and connected part;
$F_{r,bot}$:	Force in the bottom reinforcements;
F_{ci} :	Force in the i th concrete layer;
F_{decay} :	Decayed slip load;

F_{slip} :	Slip resistance;
F_{min} :	Minimum slip load;
G :	Shear modulus of elasticity of steel;
h_c :	Depth of the column section;
K_{slip} :	Initial slip stiffness of single plate connection;
K_{fp} :	Post slip stiffness;
$K_{i,bearing}$:	Initial bearing stiffness of single plate connection with ordinary bolts;
K_{br} :	Bearing stiffness;
K_b :	Bending stiffness;
K_i :	Initial stiffness of steel connection;
K_v :	Shear stiffness;
L_e :	End distance of bolt;
L_c :	Length of concrete considered for calculating concrete strain; ($L_c = h_c/2$, h_c is the depth of column section)
L_r :	Length of reinforcement considered for calculating steel strain;
M_{hog} :	Maximum hogging moment capacity;
M_{sag} :	Maximum sagging moment capacity;
N_{cycles} :	Number of load cycles;
n :	Row number of bolts;
$nshape$:	Shape parameter defining the curvature of the decay function;
P :	Applied force of an interior joint;
P_y :	Strength resistance capacity of an interior joint;
t_{web} :	Thickness of beam web;
t_{plate} :	Thickness of steel plate;
θ :	Rotation of connection;
θ_{hog} :	Rotation capacity under hogging moment;
θ_{sag} :	Rotation capacity under sagging moment;

δ :	Slip deformation;
δ_{slip} :	Deformation before slip;
δ_{fu} :	Ultimate slip deformation;
$\delta_{clearance}$:	Bolt hole clearance;
$\delta_{bearing}$:	Bearing deformation;
Δ :	Measured top displacement of an interior joint;
Δ_y :	Reference yielding displacement of an interior joint at P_y ;
Δ_r :	Displacement of reinforcement;
x_h :	Depth of neutral axis under hogging moment;
x_s :	Depth of neutral axis under sagging moment;
y_i :	Distance from bottom of connection to the i th HSFG bolt;
$\bar{y}$:	Depth of neutral axis.

Chapter1: Introduction

1.1 General

Precast concrete technology has been used widely in the construction industry due to Singapore's economical development and its industrialisation over the past decades. In precast construction, factory-controlled conditions and the use of state-of-art manufacturing techniques enable the desired dimensions and shapes of precast concrete to be more easily achieved. Also, construction time can be shortened by the introduction of precast concrete technique and the need for workers on-site is greatly reduced. This is especially important in Singapore where the shortage of manpower and the shrinking pool of skilled workers from traditional sources are key factors. The limited resources of labour force and its high costs have pushed Singapore to improve and upgrade its precast construction technology into a more effective as well as less labour intensive way.

In design, the precast beam-column connections are usually assumed to behave as pins and can only transfer shear and axial forces. The stability of a precast frame due to horizontal load is provided by some form of bracing systems. However, in Singapore, precast frames are designed as moment resisting frames to resist the lateral load just like the monolithic frames. A typical precast beam-column connection used in Singapore is illustrated in Figure 1.1. The main shortcomings of such connection may be summarised as congestion of reinforcement in the connection, unstable structural system during construction stage, labour-intensive job on site and only able to resist hogging moments under sway loads.

As shown in Figure 1.1, it is quite common for beam and column rebars to be concentrated at the connection area during the construction stage, which may result in congestion or clashing of rebars. Reinforcing bars, which appear as lines on drawings, have real cross-sectional dimensions that are larger than the nominal dimensions because of the surface deformations. Placement of these reinforcements can often prevent the proper compaction of wet concrete. Blocked by the congested

reinforcing steel, coarse aggregates will hinder the flow of concrete and cause undesirable honeycombs within the connection.

During the construction stage, the precast beams are just simply placed on the corbel or the edge of column, which may cause the whole structural system to be unstable at this stage, thereby requiring additional temporary supports. Moreover, the congestion in the connection and poor fit in the field often slow down the construction process and more manpower may also be required. Therefore, it is necessary to look for a more systematic alternative connection device that can simplify the complexity and congestion at the joint. Based on above considerations, a new hybrid beam-column connection method is proposed by A/P Tan Teng Hooi and A/P Yip Woon Kwong at NTU. The motivations to adopt such hybrid connection for the construction industry are as follows:

- Installation is simple, direct and fast.
- Structural system is stable during the construction stage.

The configuration and details of proposed hybrid connection are shown in Figure 1.2 & 1.3 respectively. A short steel universal beam is partly embedded in the precast beam at each end. Based on previous research carried out in NTU (Hu, 1999), the optimum embedded length was suggested to be two times of the effective depth of the precast beam. The precast column is separated at the joint position by a steel universal column partly embedded in the precast column. Shear studs are used to effectively transfer the loads between the steel column and the precast column. During the construction stage, the precast beam is connected to the column using a single plate or fin plate connection. In the U.S., this type of connection is known as “shear tab”. Eventually, the hybrid connection is cast together with cast-in-place beam and the topping concrete. Continuity can be achieved across the beam-column joint through the use of top and bottom reinforcements in the hybrid connection.

Using this kind of hybrid connection, it is much easier and simpler to bolt steel sections altogether. It can also eliminate or reduce on-site reinforcement congestion.

The main advantage of this hybrid connection technology is its instant ability to support the weight of structural members immediately after installation and is also stable at the construction stage. However, during the construction stage, the self-weight of structural members and the construction live load are carried in shear by the single plate connections only. The hybrid connection resists only imposed gravity and sway load.

In order to use the proposed hybrid connection technology in practice, it is necessary to study and understand the behaviour of the hybrid connection. For the hybrid connection, the semi-rigid nature of the steel connection and the concrete encasement together have increased the complexity in predicting the moment-rotation relationships and failure modes compared with the conventional concrete or bare steel structures.

1.2 Objective and Scope of Investigation

The proposed hybrid connection utilises the ease of installation of steelwork for traditional reinforced concrete construction. However, there are no studies to support the use of this type of hybrid connection for precast concrete construction. The primary objective of this research is to study the behaviour and suitability of the proposed hybrid beam-column connection. This research is also intended to provide guidelines for the design and analysis of frames with such hybrid connections under vertical and lateral loads.

The scopes of this research are:

- To understand the behaviour of the hybrid connection through experimental works, which can provide key parameters for the derivation of a mechanical model.
- To develop a mechanical model of the connection.
- To incorporate the analytical model of hybrid connection into existing structural analysis software.
- To develop a simplified and yet reliable design method for the hybrid connections.
- To set up a design guide for the analysis of precast frames with hybrid connections.

1.3 Organisation

This section briefly describes the organisation of this thesis:

In Chapter 2, previous researches on semi-rigid steel and composite connections are reviewed. Factors affecting the connection behaviour are discussed.

In Chapter 3, existing models on the behaviour of semi-rigid connections are reviewed. Four types of models are compared and their merits are discussed.

In Chapter 4, the experimental program of this research is reported. It includes the configurations of specimens, test set-ups and loading procedures for specimens under monotonic and cyclic loading. In Chapter 5, the experimental results of tests on 12 specimens are presented and discussed.

In Chapter 6, a mechanical model has been proposed to simulate the behaviour of single plate connection under monotonic loading. The force-displacement relationship of High Strength Friction Grip (HSFG) bolts is represented by a three-stage analytical model. A separate model, which is component-based, is also proposed for the hybrid connection under monotonic loading.

In Chapter 7, the component-based mechanical model for hybrid connection under monotonic loading has been extended for cyclic loading. The mechanical model for single plate connection under monotonic loading has been modified to handle cyclic loading by introducing the rules of cyclic force-displacement relationship of HSFG bolts. A Damage Index (DI) is introduced into the model to characterise the reduction of concrete stiffness and decay of concrete strength under cyclic loading due to the damage of concrete.

In Chapter 8, a simplified and yet reliable practical design method for hybrid connection is presented. The simplified design method is for the design of moment

capacity, rotation capacity as well as moment-rotation relationship. This approach gives a convenient and safe prediction for design purpose.

In Chapter 9, the analytical moment-rotation relationship is incorporated into the connector elements in a finite element program for semi-rigid frame analysis. Stability analysis of the precast frame at the construction stage is carried out. Comparisons of moment distributions, beam deflections and frame lateral displacements are conducted between frames with pinned, rigid and semi-rigid connections. Design guidance for the analysis of a frame with hybrid connections is suggested.

Finally, in Chapter 10, the major findings from the experimental work and numerical analysis of the present research are summarised. Suggestions for future research are also presented.

Figure 1.1 Typical beam-column connection used in precast construction

Figure 1.2 Proposed hybrid connection in precast construction

Figure 1.3 Details of proposed hybrid connection

Chapter 2: Literature Review of Semi-Rigid Steelwork Connections

2.1 Introduction

In the analysis and design of structures, the joints between the components are generally assumed to behave in an idealised manner in order to reduce the analytical difficulties. The idealised behaviour refers to the “perfectly pin” and the “perfectly rigid” condition, in which the joint is either assumed to have zero moment capacity or zero rotational capacity respectively. However, the actual behaviour of all joints falls in between these two extremes. Therefore, it is more realistic to consider all joints as “semi-rigid”. Various combinations of moment resistance and rotational capacity can be achieved by using different joint detailing. It is well known that even a very flexible joint exhibits some kind of rotational stiffness while a very rigid joint possess some kind of rotational flexibility.

Several national codes for the design of steel structures permit semi-rigid action in the design, however, these provisions have seldom been utilised in practice.

AISC (1980) - Three types of frame construction are permitted by American Institute of Steel Construction (AISC):

Type 1 or “Rigid framing” - This requires beam-to-column joints to have sufficient rigidity to essentially prevent the rotation of the ends of intersecting members with respect to one another.

Type 2 or “Simple framing” - This requires that the joints to have free rotation of the ends of intersecting members with respect to one another, at least under gravity loads.

Type 3 or “Semi-rigid framing” - This requires that the joints to be intermediate in rigidity between rigid framing and simple framing. In this type of construction,

some rotation between the ends of intersecting members occurs and some end moments are generated. “Evidence” that the joint can produce a minimum proportion of full end restraint is a requirement for the design of type 3 framing.

AISC (1999) – The newly published AISC specifications (AISC, 1999), title “Load and Resistance Factor Design” (LRFD), revises the earlier provisions for the types of construction in an attempt to provide a more accurate account for the actual degree of joint restraint in the structural analysis and design. It recognises the fact that all joints possess some restraint, and depending on the amount of restraint offered by the joint, the LRFD provisions classify the steel joints as either:

- Type FR (Fully-Restrained) which is equivalent to Type 1 construction in the previous edition, or,
- Type PR (Partially-Restrained) which encompasses both Type 2 and Type 3 in the previous edition.

BS 5950 (2000) – The British Standards (BS 5950) also allows for three methods of design. In clause 2.1.2 of BS 5950 (2000), it is stated that the design of any structure or its parts may be carried out by one of the methods given below:

Simple design – The joints between members are assumed not to develop moments adversely affecting either the members or the structure as a whole.

Continuous design – Either elastic or plastic analysis may be used. For elastic analysis the joints should have sufficient rotational stiffness to justify analysis based on full continuity. For plastic analysis the joints should have sufficient moment capacity to justify analysis assuming plastic hinges in the members.

Semi-continuous design – Some degree of joint strength and stiffness is assumed, but insufficient to develop full continuity. The moment capacity, rotational stiffness and rotation capacity of the joints should be based on experimental evidence.

The experimental method for semi-continuous design is not practicable in normal cases and therefore does not appeal to engineers.

Eurocode 3 (1992) – In the semi-rigid design aspect, the “Eurocode 3: Design of steel structures – Part 1.1” (1992) has paved the way for other national codes to follow suit. In section 6.9 of EC3, detailed guidelines for the design of beam-to-column joints are given. Specific methods for the determination of moment resistance, rotational stiffness and rotation capacity of endplate joint using the “component method” approach is given in Annex J of EC3. The component-based approach simulates the connection behaviour by a set of rigid and deformable components. The overall connection response is achieved by assembling the contributions from discrete components of the connection.

In this chapter, a brief review of previous researches on the semi-rigid steel and composite connections is presented to obtain a comprehensive understanding of the factors affecting the joint behaviour.

2.2 Literature Review of Semi-Rigid Steel Connections

Most of the investigations and documentations of joint data before 1986 were compiled by Nethercot (1985) and Kishi (1986). The database by Kishi was further expanded in 1993 (Chen, 1993). The most common types of steel connections are illustrated in Figure 2.1. Figure 2.2 shows schematically the moment-rotation behaviour of these commonly used connections. The curves represent a wide range of stiffness offered by different types of connections. The horizontal and vertical axes represent the two ideal cases of pinned and rigid joints respectively. In practice, such ideal cases do not exist. According to the Eurocode 3 classification, the boundary curve between rigid and semi-rigid regions is a trilinear curve (Figure 2.3). The three regions define the rigid, semi-rigid and nominally pinned or flexible connections. A good representative of rigid-framing seems to be the T-stub or the

extended end-plate connection. The single fin plate connection represents simple-framing. The rest of curves are considered to be semi-rigid connections.

2.2.1 Factors Affecting the Behaviour of Steel Connections

To quantify the behaviour of a joint, the in-plane moment-relative rotation relationship ($M - \theta$) is normally used. The $M - \theta$ curve describes the behaviour of a joint through three important physical properties: initial stiffness, moment capacity and rotation capacity. The $M - \theta$ relationship of steel connections is typically non-linear over the entire range of loading. The non-linearity of connection is due to a number of factors (Chen & Lui, 1986; Barakat, 1989). Some of the important ones are as follows:

- Component discontinuity of a connection assemblage. The connection is composed of various combinations and arrangements of bolts and structural shapes like angles and T-stubs. This formation allows for irregular slippage and movement of components relative to each other at different stages of loading.
- Local yielding of some component parts of a connection assemblage. This is the primary factor related to the non-linear behaviour of a connection.
- Stress and strain concentrations caused by holes, fasteners, and bearing contacts of elements used in a connection assemblage.
- Local buckling of flanges and/or web of the beam and the column in the vicinity of a connection.
- Overall geometric changes under the influence of applied loads.

2.2.2 Lack of Fit in Steel Connections

Due to the fabrication deviation or foundation misalignment, some lack of fit in the structure may arise during the erection period. Therefore, it is necessary to select steel connections, which are tolerant to on-site adjustments. The influence of lack of fit on the rotational capacity of bolted beam-to-column connection has been reported (Davison et al., 1987). Generally, endplate connections and welded joints have little or no tolerance, while the bolted cleated type connections and single plate connections have some adjustments by virtue of the use of bolts in clearance holes. Therefore, if the semi-rigid nature of the connection has been relied on in practical design, the implications due to lack of fit in steel connections must be considered carefully by the engineer. For the precast concrete construction, a lack of flatness, alignment and squareness will inevitably lead to the accumulation of physical dimensional deviations. Slotted holes are usually provided for the horizontal and vertical tolerances in practice.

2.2.3 Single Plate Connection

The single plate or fin-plate connection consists of a length of plate, which is welded to the supporting member in the workshop, and then bolted to the beam web on site. The advantage of this type of connection is that they are simple to fabricate and easy for construction erection on site. Single plate connections derive their rotational capacity from the bolt deformation in shear and the hole distortion in bearing of the weaker of the steel plate and beam web. Tests and studies were carried out by Lipson (1977), which indicate that the single plate connection can develop a significant end moment in the beam and supporting member. The magnitude of the moment is generally dependent upon the number, size, configuration of bolt pattern, the thickness of plate or beam web and the beam span to beam depth ratio, etc. In the present research, the single plate connection is selected for the steel connection due to its simplicity of fabrication, ease of installation and tolerance of on-site adjustment.

2.3 Literature Review of Semi-Rigid Composite Connections

The concept of composite construction can be traced back to the beginning of last century. The synergetic composite action of steel and concrete makes inventive use of the merits of these two materials. Numerous researches have been carried out on the field of semi-rigid composite joints. In order to study the feasibility of achieving continuity across the column through composite joints, extensive investigations have also been conducted on the behaviour of composite connections under hogging moment condition. Initially, the researchers seek to understand the response of joints with simple steel joint detailing and a moderate amount of steel reinforcement. Subsequently, more parameters were investigated and the interaction with the beam and column was studied in more detail.

The use of the semi-rigid joints was first suggested in early 1970 (Barnard 1970, Johnson and Hope-Gill, 1972). Owens and Echeta (1981) carried out research works aiming at checking the feasibility of using plastic design approach on the composite joint. Different steel joints were used in their experiments. Various slab reinforcement ratio and shear/moment ratio were applied during the test. It was observed that there was no significant effect in varying shear/moment ratio due to the low failure load in most of the specimens.

Another test was carried out by Van Dalen and Godoy (1982) to study the degree of flexibility of steel joints by adopting two different forms of joint. Four cruciform specimens were tested under a symmetrical loading condition and two slab reinforcement ratios (0.46% and 0.80%) were used for investigation. It was found that the addition of more reinforcement increased the ultimate moment and concluded that the addition of a composite slab greatly increased the stiffness and moment capacity of the joint.

Kato and Tagawa (1984) reported their research on concrete slabs and column-to-composite beam subassemblies. A push-out test of four concrete slab specimens was carried out in order to investigate the effective width of concrete slabs when

composite beams were subjected to positive bending. Comparison of positive moment of composite beams was conducted under different combinations of parameters of slab thickness, column width and different details inside the H-section column at the level of slabs. It was indicated that the slab thickness had obvious influence on the moment strength, while the effect of the stiffness of the column bearing face was insignificant.

As a continuation of the pilot project in the University of Sheffield, Nethercot et al (1994) carried out another series of tests at the University of Nottingham (Figure 2.4). Four different steel joints, which are seating cleat, flush endplate, partial depth endplate and fin-plate were used. It was concluded that different failure modes were observed for different steel details and different reinforcement arrangements. Failure was usually through a combination of yielding of the reinforcement and failure of the steel joint part. For fin-plate composite connection, considerable moment resistance can be achieved if the connection is properly designed.

In order to prove the feasibility of semi-rigid composite connection under cyclic loading, Leon et al (1987) conducted an experimental program at the University of Minnesota (Figure 2.5). Three tests were carried out: two specimens with cruciform configuration named SPCC1C and SPCC3C, and one on a complete two-bay limited frame named SPCC2C. Cracking tended to occur earlier under cyclic loading compared to monotonic loading. This resulted in a lower stiffness compared with that in the monotonic test. The hysteresis loops display pinched form when the displacement increased further. The envelope of moment-rotation curves from cyclic tests followed closely to that obtained from the monotonic test. It was concluded that cyclic loading has a limited effect on hysteresis properties of connections. Yielding under hogging moment action was always associated with yielding of reinforcing bars, while under sagging moment, failure was mainly due to the fracture of steel connection components, such as the low cyclic fatigue of the cleats bolted to the beam flange and the fracture of the extended endplate.

As shown in Figure 2.6, some real test results (Nethercot et al, 1994; Leon et al, 1987) were plotted with respect to the classification in the Eurocode 3 limits (assuming the beam length equals to 8 m). It can be concluded that most composite connections fall into the semi-rigid category.

After the 1994 Northridge earthquake, there was a significant series of tests conducted on shear connections with floor slabs under cyclic loading by Liu and Astanteh (2000). The test results demonstrated that shear connections have significant moment capacities, both on their own and with the contribution of the floor slab. To explore the potential of precast, prestressed concrete being used in high seismic areas, a new precast framing system that used precast elements, connected by unbonded post-tensioning steel and bonded reinforcing bars was developed (Stanton, et al., 1997). Compared to the conventional monolithic frames, the new precast frame proved to be at least equal to the monolithic frames in almost all respects, and superior in most.

2.3.1 Characteristics of Semi-Rigid Composite Connection

Similar to the semi-rigid steel connection, the behaviour of semi-rigid composite connections under hogging moment is best described using a moment-rotation curve. Generally, the typical curve consists of three characteristic regions: elastic, inelastic and plastic region. These three regions are bounded by characteristic values of moment and rotation as shown in Figure 2.7. Different stages of the composite connection behaviour can be clearly reflected from the moment-rotation curve. Previous investigations indicate that the initial response is governed by the axial stiffness in tension of the concrete slab. The contribution of the steel connection tends to be modest and it becomes apparent after the slab has cracked.

- **Elastic Region**

When concrete remains uncracked, test results showed a very high value of initial stiffness of the composite connection, in the range of 226000~255000 kNm/rad

(Van Dalen et al., 1982; Benussi et al., 1986; Ammerman et al., 1987). Ammerman and Leon (1987) tested various configurations of composite joints using very flexible steel angle joints to very stiff steel endplate joints. Despite the use of very flexible angle joints, very high initial stiffness of the composite joint (approximately ten times of bare steel joint initial stiffness) was observed. This clearly indicated that the initial stiffness is governed by the axial stiffness in tension of the concrete slab. Once the concrete starts cracking, the axial stiffness of the slab is substantially reduced. Cracks usually initiate from the column face and spread rapidly towards the edge of the slab. The rotational behaviour now enters a second phase. This phase is characterised by stiffness substantially lower than the initial uncracked value but still fairly linear. Although the axial stiffness of the slab is substantially reduced by the cracking, the slab action is still a key component of the joint response. The reduced stiffness of the slab allows the steel joint to contribute to a greater extent. It can be concluded at this stage that the composite behaviour is still governed largely by the distribution of steel reinforcement in the slab (Van Dalen and Godoy, 1982) and the behaviour of the components of the steel connection (Johnson and Hope-Gill, 1972).

- **Inelastic Region**

This stage is the most unpredictable part of the curve. Most researchers were not able to quantify the factors attributing to this behaviour. The level of non-linearity at this stage also differs with each joint type. Therefore, Zandonini (1989) suggested that the various reasons attributing to the behaviour in the inelastic range are: material behaviour (yielding of reinforcement and steel members); inelastic phenomena (bolt slippage); change in mode of action of different joint (beam flange bearing against column flange due to contact).

- **Plastic Region**

The plastic stage is attained as a result of significant yielding of one of the key components of the composite joint, usually the slab reinforcement, beam

compression flange or the column web. However, the column webs are usually stronger through the use of web stiffeners or concrete encasement. Due to the effect of strain hardening, most joints are able to sustain further loading after the plastic moment is achieved. If the plastic stage is achieved due to the yielding of reinforcement, the steel joint may develop its inherent strength for relatively high rotations and then enable the joint to sustain further loading. Ammerman (1987) observed that all reinforcement bars have yielded at the attainment of ultimate moment. This confirmed that the steel bars further away from the column flange keep on increasing the resistance of slab, as the plastic stage becomes imminent.

2.3.2 Parameters Affecting the Behaviour of Composite Connections

Based on previous researches, a number of parameters were found to influence the physical behaviour of composite connections. These parameters may be summarised briefly as follows:

- **Concrete slab action**

As mentioned by Zandonini (1989), the slab action is major factor governing the behaviour of the joint. Two aspects of this factor can be considered:

- 1) The effect of reinforcement inside the slab which includes the steel ratio, yield strength of steel reinforcement and the arrangement of bars. Numerous experiments indicated that effect of reinforcement in improving the strength and stiffness of composite connections under negative bending moment.
- 2) The contribution of concrete slab itself, such as effective width of concrete slab, relative stiffness of the slab to the steel section and tensile strength of concrete slab, which is often neglected by researchers.

- **Steel connection**

The steel connection is the basic part of the composite connection. Not only it carries almost all shear force acting on the connection, but also influences the behaviour of the composite joint. A stronger steel connection may yield a stronger composite joint and improve the moment capacity. It may also reduce the ductility of the composite connection and weaken the benefit from the slab reinforcement.

- **Shear connector**

It was observed that the shear connection between the concrete slab and steel beam affects the behaviour of composite joint (Zandonini, 1989). This shear connection depends on the flexibility and strength of shear connectors. Generally, the stronger the interaction effect, the stiffer the composite joint.

- **Steel column**

There are two types of failure modes occurred in the column. The first failure mode is the local buckling of the web under compression and the second is the shear distortion of the panel zone mainly due to the unbalanced moment. Moreover, the presence of column encasement and axial load in the column also influence the behaviour of steel column.

2.3.3 Embedment length of steel section in precast concrete beam

Elliot (1996) studied the embedment problem in his research of precast reinforced concrete, and treated the problem in a practical design manner. Elliot's method was based on adequate bearing stresses both in the steel section and concrete beam, the prevention of spalling, bursting and splitting, and an adequate tie back in the concrete beam. The ultimate shear capacity of the section was based on the shear capacity of the inserted section itself, and was gradually transferred to the concrete beam. Tie back forces were distributed into the concrete beam either by an

appropriate concentration of vertical stirrups, bent bars, or by welding a wide plate to the bottom of the steel inserts.

The study of the problem begins from the end reaction. The end reaction V is transferred to the inserts either directly or through a flat plate welded to the bottom of the insert, provided that the local bearing stresses is not exceeded and the connection is stable in the temporary fixing condition. A tension hanger in the form of stirrup reinforcement, plate straps or bent bars with a full anchorage length and correct bend radius, which are welded to the steel insert.

The equilibrium compression force $C = T - V$ is provided at the remote end of the insert, either by direct bearing or through an additional bearing plate. The line pressure is suggested to be taken as $0.8f_{cu}b_p$ provided that the concrete beneath the bearing is confined laterally. Taking moment about the centre of the tension strap, as shown in Figure 2.8.

$$V = \frac{0.8f_{cu}b_pL_3(L_4 - 0.5L_3)}{L_1} \quad \text{Eq. 2.1}$$

Resolving vertically,

$$V = T - 0.8f_{cu}b_pL_3 \quad \text{Eq. 2-2}$$

Hence T may be computed. The area of reinforcement is

$$A_s = \frac{T}{0.87f_y} \text{ for rebar}$$

or

$$A_s = \frac{T}{P_y} \text{ for steel plate} \quad \text{Eq. 2-3}$$

Where V is the reaction force;

f_{cu} is the characteristic compressive cube strength of concrete;

b_p is the breadth of bearing;

L_1 is the distance from face of column to load;

L_2, L_3 is the length of stress block (insert design);

L_4 is the length of embedment of insert.

During the construction stage, in general, unpropped construction is chosen in order to be economical. In the case of unpropped construction, the steel beam has to carry the self weight of steel, concrete and construction live load. Therefore, the bare steel connection is pre-loaded during the construction stage.

2.4 Summary

Previous researches on the semi-rigid steel and composite connections are reviewed. Factors affecting the joint behaviour are discussed. Single plate connection is adopted in the present research due to its simplicity of fabrication, ease of installation and tolerance of on site adjustment. However, previous studies on the behaviour of single plate connection are rare.

Although numerous researches have been carried out on the composite joints, all these joints are the conventional composite joints which consist of concrete slab and steel connection only. There is no existing composite connection similar to the proposed hybrid connection, which consists of the steel connection and concrete encasement. For this hybrid connection, the semi-rigid behaviour of the steel component is complicated further by the reinforced concrete infill compared to those conventional composite connections.

Figure 2.1 Common types of steel connections

Figure 2.2 Behaviour of semi-rigid steel connections (Chen, 1987)

Figure 2.3 Beam-to-column joint classifications according to their rotational stiffness (Eurocode 3, 1993)

Figure 2.4 Cruciform arrangement of composite connection
(Nethercot, 1994)

Figure 2.5 Composite frame tested by Leon (1987)

Figure 2.6 Some real test results (Nethercot et al, 1994; Leon et al, 1987) with respect to Eurocode 3 classification

Figure 2.7 Typical moment-rotation characteristic of composite connection

Figure 2.8 Elliott's study of embedded structural steel section (1996)

Figure 2.9 Equilibrium condition of embedded structural steel section (Elliott, 1996)

Chapter 3: Literature Review of Existing Models

3.1 Introduction

Although experimental investigation may give the most reliable and accurate result about the behaviour of composite connections, it is too expensive and time-consuming in practice. Therefore, using suitable and reliable modelling to predict the connection behaviour is reasonable and necessary. To date, many models have been proposed by researchers for steel and composite joints. COST C1 (1997) is a major work in the field of semi-rigid behaviour of structural connections. Based on theoretical considerations, these models may be classified into four types, namely empirical, mathematical, numerical and mechanical.

3.2 Empirical Models

The most commonly used approaches in empirical models to describe the $M - \theta$ relationship involve the curve-fitting of experimental data to simple expressions, or the development of simple analytical procedures to predict the behaviour of the connections if no test data are available for specific connection details. Goverdhan (1983) collected the available test data on moment-rotation characteristic and tried to formulate prediction equation for each curve. Nethercot (1985) conducted a literature survey on steel and composite connections for the period 1915-1985 and reviewed the test data and the corresponding curve representation. Kishi and Chen (1986) collected a large selection of data and provided moment-rotation characteristics and corresponding parameters of the beam-to-column connections used frequently in steel construction. Various empirical models and their characteristics are summarised in Table 3.1.

Although the single plate connection has been used for many years, previous researches on the behaviour of single plate connection are scant. From Table 3.1, only Richard et al. (1980) developed a non-dimensional power model to represent the moment-rotation relationship of single plate connections under monotonic

loading. To the best knowledge of the author, the study on the behaviour of single plate connection under cyclic loading has not been carried out yet.

3.3 Mathematical Models

Mathematical models rely on calibrating mathematical expressions to experimental data. Johnson and Law (1981) proposed an approach for the prediction of flush endplate composite connections. The moment-rotation curve is presented by tri-linear modelling. The boundary of each linear part defines a zone, which is supposed to be representative of the physical behaviour. The first zone is the elastic zone, while the last zone is the plastic zone. The intermediate zone describes the transition between the loss of stiffness and the attainment of the plasticity. The initial stiffness of the connection is determined by superposition of the contributions provided by the steel components and the reinforced concrete slab. The tensile strength of the concrete and shear lag effect is neglected. Assuming that the end cross-section of the steel beam rotates about its bottom flange, the elastic rotational stiffness K of the connection can be calculated by using the equilibrium and compatible conditions at the column face:

$$K = \frac{h_b^2}{0.5\left(\frac{1}{K_b} + \frac{1}{K_{cf}}\right)\left(\frac{h_b}{h}\right)^2 + \frac{1}{K_p}} \quad \text{Eq. 3-1}$$

where K is the elastic rotational stiffness (Nmm/rad);

h_b is the depth of the steel beam (mm);

h is the depth of steel column (mm);

K_b is the stiffness of the bolts (N/mm);

K_{cf} is the stiffness of the column flange (N/mm);

K_p is the stiffness of the endplate (N/mm).

The ultimate moment resistance of the connection is determined by adding the resistance of the steel part and that of the reinforcing bars.

$$M_u = M_{ps} + A_r \cdot f_{yr} \cdot d_r \quad \text{Eq. 3-2}$$

where M_{ps} is the plastic moment capacity of the steel connection;

A_r is the area of reinforcing bars;

f_{yr} is the yield strength of reinforcing bars;

d_r is the arm length of the tensile force in bars.

It is assumed that yielding in the column flange precedes that in the endplate and bolts. This approach is not suitable for connections having thinner endplate.

Based on test results, Aribert and Lachal (1992) [COST C1] proposed a simple form of moment-rotation relation for flush endplate connections. Similar to Johnson and Law's model, it first suggests that the initial stiffness of the composite connection K_{cc} can be deduced from the stiffness of the steel part in the form:

$$K_{cc} = K_i + \frac{d_r}{\frac{h_c}{2E_r A_r d_r} + \frac{\alpha}{N k h_b}} \quad \text{Eq. 3-3}$$

in which, E_r is the elastic modulus (N/mm^2);

A_r is the area of reinforcing bars (mm^2);

K_i is the initial stiffness of steel connection (Nmm/rad);

d_r is the distance from bars to the bottom beam flange (mm);

N is number of active shear connectors;

k is the secant stiffness of one shear connector (N/mm);

α is an amplification factor taken as 2.

Rotation of the connection is obtained from compatible condition at the column face:

$$\theta = \frac{\Delta}{d_r} + \frac{s}{h_b} \quad \text{Eq. 3-4}$$

where Δ is the elongation of reinforcing bar over half depth of the column;

s is the slip of shear connectors near the joint;

h_b is the depth of the steel beam;

F_r is the total tensile force in the reinforcing bars;

Then the moment resistance is obtained as:

$$M = K_{cc} \cdot \theta + F_r \cdot d_r \quad \text{Eq. 3-5}$$

The formulation of this model is very simple, but its prediction accuracy is doubtful. The comparison between the predicted and tested initial stiffness had large discrepancies, sometimes more than 100% (Ahmed, 1996).

Anderson and Najafi (1994) [COST C1] have proposed a model to predict the moment capacity of the connection using the concepts of plastic analysis. When the total tensile resistance (bolts and rebar) exceeds the bottom flange compressive force, a plastic stress block is assumed in the lower part of the web. The moment resistance of the connection is given by:

$$M = R_r D_r + R_b D_b - (R_r + R_b - R_f) \left(\frac{x}{2} + \frac{T}{2} \right) \quad \text{Eq. 3-6}$$

where R_r is the rebar force;

R_b is the bolt force;

R_f is the compression in bottom flange;

x is the depth of web compression;

T is the thickness of beam flange.

A relation between the moment and the rotation of a composite connection was also proposed by Anderson and Najafi (1994) [COST C1]. The model assumed a rotation of the beam web about the bottom flange, but at the same time considered the slip of the studs. The method assumes that the developed moment is due to rebar and bolt force only. The relationship is given as:

$$M = \left(\frac{K_r K_s D_r D_s}{K_r + K_s} + K_b D_b^2 \right) \phi \quad \text{Eq. 3-7}$$

where M is the moment and ϕ is the rotation, subscripts r , s and b indicate rebar, shear stud and bolt. D represents the distance from bottom flange centre line (mm), and K represents the stiffness of the associated member (N/mm).

The above formulae ignore the yield of the column web, or the column web stiffness, more importantly, the influence of the different numbers of studs present in the connections is not properly reflected.

3.4 Numerical Models

Because of the numerous variables and potential failure modes associated with the composite connections, the simplified models which just consider limited numbers of parameters are unlikely to be able to thoroughly examine all detailed aspects. Numerical approaches, such as finite element method are able to provide powerful technique to include more variables than those covered by the experimental program.

Numerical calculation was carried out to study the behaviour of composite joint (Puhali et al., 1990) using the non-linear finite element package ABAQUS. The proposed analytical approach was incorporated into the finite element model. The steel beam and reinforced concrete slab were modelled by beam elements. The slip for the shear connectors was modelled by spring elements. The compressive

deformation of the column web was included by imposing restraint conditions on the composite beam. The analytical predictions had about 10% differences with the experimental results which indicated the capacity of this model.

Based on the model for bare steel joints, Ren et al. (1992) [COST C1] proposed a finite element model to calculate the moment resistance of composite flush endplate connections. The model adopts four kinds of elements, which allow respectively modelling of the column panel zone, the steel connection, the concrete slab and the partial shear connection in the composite beam. Since the model simulates the composite joint properties using different elements, the influence of each factor can be singled out. The unbalanced moment and axial force can also be accounted into the model. The predicted behaviour had a generally 85% agreement with the test results.

Nethercot and Ahmed (1997) proposed a finite element model to simulate the structural performance of composite flush endplate connections. The model was also incorporated into the multi-purpose finite element package ABAQUS. In this finite element model, four node rigid shell elements are used for the steel beam, column and endplate. Each bolt is modelled with a combination of springs and shell elements. Shear connectors are modelled by beam elements and reinforcing bars by truss elements. The concrete is taken as only playing the role of transferring the tensile force to the reinforcing bars, and its effect was therefore ignored in the model. The contact between endplate and column flange is modelled by an interface element, capable of transferring compression when in contact and transferring no tension during separation. The analytical results were compared with experimental data and only 5 to 10% differences were observed. The comparison indicated that the finite element model can accurately address the behaviour of composite connections.

3.5 Mechanical Models

Mechanical models are based on simulating the connection behaviour by a set of rigid and deformable elements. The non-linearity of the connection response is achieved by means of inelastic constitutive laws adopted in the deformable elements. By assembling the contributions from discrete elements of the connection, a mechanical model is capable of predicting the connection behaviour throughout the whole range of the moment-rotation curve based only on the geometrical configuration and material properties. In addition, the contribution from different components can be singled out.

Wales and Rossow (1983) proposed a component-based model to evaluate the non-linearity of double web angle connections (Figure 3.1). The model idealised the connection as two rigid bars joined by a homogeneous continuum of independent non-linear springs and the load-deformation relationship for the springs is modelled by a tri-linear function. Comparing with the test data, the model yields satisfactory results with about 80% agreement. An important novel feature of this model is that it accounts the influence of the axial force.

Tschemmerneegg (1988a) [COST C1] proposed a component-based model to represent the deformation characteristics of welded steel connections. As shown in Figure 3.2, two sets of springs are employed in the model. One set of springs (spring A) was used to account for the load-transferring effects from the beam end to the column, while another set of spring (spring B) was used to represent the shear deformation of the column web panel zone. The spring properties are formulated in mathematical forms, which are calibrated using the experimental data. The final moment-rotation curve of the joint is achieved by superposing the deformations of those two spring sets. The model was further extended to represent endplate bolted connections by an additional set of connection springs (spring C, Figure 3.3).

To further extend the research work on steel connection, Tschemmerneegg (1992) [COST C1] proposed a new spring model for composite connections (Figure 3.4).

The principle procedure is similar to the previous model for steel connections. However, the spring properties were redefined and a compressive spring was added (spring D). In the tension zone, the tensile force is simulated by a truss model. The loading spring simulates the flexibility of the truss system. The flexibility of reinforcing bars at both sides of the column is modelled by tension springs (spring E). In the compression zone, the compressive force is simulated by an external spring and an internal spring (spring A & C). The shear deformation of the column panel zone is simulated by a shear spring (spring B). However, this model neglects the effects of slip at the steel beam and concrete slab interface. The shear lag effect in concrete slab is also ignored. Moreover, the model does not account for the tension in column flange. All these simplifications affect the accuracy of this model (Wang, 1999).

Few mechanical models were developed to simulate the cyclic behaviour of semi-rigid composite joints. De Stefano et al. (1994) proposed a mechanical model to describe the cyclic behaviour of double-angle connections (Figure 3.5). Similar to the model suggested by Wales and Rossow (1983), the double-angle connection is modelled by a series of rigid and deformable elements, only the stress-strain relations and geometrical properties of the connection are required. An elastic gap element is introduced to represent the contact effect. When the gap is closed, the rotational stiffness is equal to the stiffness of the gap element and the connection regains its stiffness. The effect of deterioration and bolt slippage is neglected in the model. Comparisons of the model prediction against the experimental results showed an acceptable correlation (differences within 20%) in terms of stiffness, strength and ductility.

Madas (1993) proposed a mechanical model to predict the behaviour of steel and composite connections of web angle, top and seat angles, partially welded flush endplate and fully welded connections. The formulation takes into account the non-linear behaviour of the connecting elements and connected members, bolt extension and slippage, shear connector flexibility, coupled moment-axial force behaviour, but does not consider shear force in the connecting elements. The proposed model

can analyse the response of composite connections under monotonic and cyclic loading conditions. The model has also been implemented into the non-linear frame analysis program 'ADAPTIC' and compared with test results done by other researchers. The accuracy of the prediction is reasonably high (differences between 2.8% to 10.6%). However, the neutral axis of the connection is assumed unchanged which may violate the force equilibrium condition.

3.6 Summary

In this chapter, four types of models are compared and their merits are discussed. Generally, empirical models always have an explicit form and are able to fit any shape of moment-rotation curves provided that suitable expressions are used and there are sufficient experimental data to identify the model parameters. However, they suffer from the disadvantage that they cannot be directly extended for use beyond the range of calibration data. Additionally, they are unable to predict the substantially different behaviour due to possible change of failure mode, when connections with different geometrical configurations and material properties are considered.

The prediction of the connection behaviour by mathematical models is achieved through the determination of key parameters (such as initial stiffness, moment resistance capacity) and fitting of the experimental data with the predetermined model shape. Due to numerous connection types and combinations, it is difficult to propose a single model to cover a large number of connection types. Most models are suitable for a particular type of connection. Therefore, the application of this approach has its limitations.

Numerical models are able to account for more variability. The complex interaction between the connection components can also be accounted. The modelling of connections by finite elements can serve as a valuable and complimentary activity to laboratory testing. However, using this type of modelling requires a lot of

computing resources and can be time-consuming and expensive for practical applications.

Component-based mechanical model can predict the moment-rotation relationship without the need to restrain it to predetermined response patterns. The accuracy and versatility of the method is increased as the number of components taken into account increases. However, its accuracy depends on the accuracy of the load-deformation relationship for the key components. The determination of such characteristic requires a full understanding of the behaviour of each component, as well as the way in which they interact. Once this is achieved, the method is capable of predicting the various behaviour of connection due to change in the geometrical configuration, material properties and loading conditions.

Compared to empirical or mathematical models, component-based mechanical model provides sufficient accuracy and versatility without the need to restrain the moment-rotation relationship of connection to a predetermined form. Meanwhile, unlike finite element methods, it is relatively simple and easy to be implemented into a computer program for the frame analysis. Therefore, the component-based model compares favourably with other analytical methods because it combines the economy with the effective and predictive application.

Based on the literature review in this chapter, it can also be concluded that there is no attempt or model to predict the behaviour of the proposed hybrid connection. Moreover, the existing modelling techniques are inadequate in the analysis of such hybrid connection in terms of following aspects:

- Models for single plate connection are rare and none of them accounts for cyclic loading.
- Most models for composite connection are under monotonic hogging moment. The available models for composite connections under cyclic loading are very few.

- To date, there is no existing model to predict the behaviour of semi-rigid steel connection with concrete encasement.

Attempting to overcome the above limitations, a new component-based mechanical model for the hybrid connection is proposed. The details of the model will be described in Chapter 6.

Table 3.1 Summary of Empirical Models

Model	Reference	Advantages	Disadvantages
Linear	Rathbun (1936) Monforton and Wu (1963)	Simple to use; Stiffness matrix only requires initial modifications	Inaccurate at high rotation values
Bilinear	Lui and Chen (1983)	Simple to use; Curve follows $M - \theta$ more closely than linear model	Inaccurate at some rotation
Polynomial	Frye and Morris (1976)	Produce close approximation to the shape of $M - \theta$ data.	Can produce inaccurate connection tangent stiffness values
B-spine	Jones et al. (1982)	Produces a very close approximation to any $M - \theta$ set	Nonlinear, requires iterative evaluation; A large number of data are required for curve-fitting process
Power	Richard et al (1980) Kishi and Chen (1986)	Produces good fit to test data of various types of connections.	Nonlinear, requires iterative evaluation; Requires weighted least-squares evaluation
Exponential	Lui and Chen (1986)	Produces good fit to variety of test data	Nonlinear, requires iterative evaluation; Requires weighted least-squares evaluation

Figure 3.1 Mechanical model for web angle connections
(Wales and Rossow, 1983)

Figure 3.2 Mechanical model of a welded joint (Tschemmerneegg, 1988a)

Figure 3.3 Mechanical model of a bolted joint (Tschemmerneegg, 1988b)

A: LOAD INTRODUCTION SPRING B: SHEAR SPRING
 C: CONNECTION SPRING
 D: REDIRECTION SPRING (COMPRESSION ONLY)
 E: LOAD INTRODUCTION SPRING

Figure 3.4 Spring model for composite connection (Tschemmerneegg, 1992)

Figure 3.5 Mechanical model for double-angle connections under cyclic loading (De Stefano, De Luca and Astaneh, 1994)

Chapter 4: Experimental Programme

4.1 Introduction

The experimental programme of this research is to study of the behaviour of the hybrid connection under monotonic loading as well as cyclic loading. Based on the case study of a typical industrial building layout (Figure A.1, Appendix A), typical beam and column sizes are selected for the experimental programme. The typical floor plan of an industrial building is shown in Figure A.1. The slab span is 8 m and the beam span is 6.65 m. The floor-to-floor height is 3.2 m. The precast components consist of hollow core slabs, composite beams and precast columns. The hollow core slab is 265 mm thick with 75 mm topping concrete. The composite beam consists of two components, the 500 x 460 mm precast concrete beam and the 300 x 340 mm cast-in-place portion. The service imposed live load is 7.5 kN/m^2 and the construction live load is 1.5 kN/m^2 .

4.2 Test Series

The test programme consists of three series; bare steel connection, partial hybrid connection and complete hybrid connection. The bare steel connection, named series A, is to study the moment-rotation relationship of bare steel connection under monotonic and cyclic sway loading. The partial hybrid connection, named series B, is the hybrid connection without precast hollow core slabs & topping concrete. It should be noted that this connection arrangement is an intermediate stage and may not exist in reality. Its purpose is to study the behaviour of the connection components at that stage under monotonic and cyclic loading. Test series C is the complete hybrid connection with precast hollow core slabs & topping concrete, which is aimed to study the contribution of hollow core slabs as well as topping concrete to the moment and rotational capacity under monotonic and cyclic loading. A total number of 12 specimens have been tested and the details of each series are illustrated as follows.

4.2.1 Test Series A – Bare Steel Connection

Single plate connection is adopted in this research to connect the steel universal beam to the steel universal column in this research due to its simplicity of fabrication, ease of installation and tolerance of on site adjustment. The aim of this series is to study the moment-rotation relationship of bare steel connection under monotonic as well as cyclic loading. The details of steel connection are shown in Figure 4.1. The bare steel specimen consisted of two universal beams (Grade 43, 356 x 71 UB45), which were connected to the steel universal column via single plate connections. The single steel plates were welded (E43, 8 mm fillet weld) to the flanges of steel column first, and then the steel beams were connected to the single plates using High Strength Friction Grip (HSFG) bolts. The size of HSFG bolt was 20 mm and the clearance bolt hole was 22 mm in diameter. The torque-pretension relationship of HSFG bolt was found to be linear and illustrated in Appendix B. All HSFG bolts were pre-tensioned to 170 kN by applying a torque of 380 N.m on each bolt. The specimens of bare steel connection BS1 and BS2 (Figure 4.2) were tested under monotonic loading. The specimen BS3 (Figure 4.3) was tested under cyclic loading. The detailed arrangements of these specimens are shown in Table 4.1. The material strength of each component is illustrated in Appendix B.

4.2.2 Test Series B – Partial Hybrid Connection (without Precast Hollow Core Slabs & Topping Concrete)

The specimens in this series consisted of three cruciform specimens with partial hybrid connections (HC1, HC2 & HC3). These specimens had the same detailing of single plate connection as test series A. Two specimens HC1 and HC2 (Figure 4.4 & 4.5) were tested under monotonic hogging and sagging moment respectively. The third specimen HC3 (Figure 4.8) was tested under cyclic loading. All specimens in this series were constructed without the precast hollow core slabs & topping concrete. The aim of this series was to study the moment-rotation behaviour of partial hybrid connection under monotonic and cyclic loading.

The universal beam was partly embedded in the precast beam. The protruding part was connected to the steel column through the single plate connection. Based on the research results carried out by Hu (1999), the optimum embedded length was about two times of the effective depth of the precast beam, therefore, the embedded length was taken as 900 mm in the experiments. The precast beam size was 500 x 460 mm and cast-in-place portion size was 300 x 340 mm (concrete strength referring to Appendix B). Inside the precast beam, four reinforcing bars (top and bottom) were provided throughout the precast beam for the tying of stirrups. Two bottom bars were protruded out of the precast beam and were lapped across the connection to achieve the bottom continuity. Similarly, two reinforcing bars were placed at the top of the cast-in-place portion and across the beam-column joint for the top continuity (reinforcing bar strength referring to Appendix B). Stirrups were provided at T10 @ 100 mm centre to centre. The stirrups also functioned as shear connectors to connect the precast beam and the cast-in-place portion. The detailed arrangements are shown in Figure 4.4.

The configuration of HC3 was constructed for the test under cyclic loading. The steel universal column was embedded in the precast column. Based on the previous research (Ong H.C., 2001), the embedded length of steel universal column was taken as 600 mm and shear studs were provided to prevent relative slip between steel column and concrete. Headed shear studs with 19 mm diameter were used in the tests. The ultimate tensile strength (nominal strength) of shear stud was $f_u = 450$ N/mm². The number of shear studs needed was designed to ensure full shear connection according to Eurocode 4 (1992). However, in this experimental programme, since there was no axial load presented in the column, the shear studs were placed uniformly along both side of steel column with two shear connectors per trough at a spacing of 200 mm. The detailed arrangements are shown in Figure 4.6.

4.2.3 Test Series C – Complete Hybrid Connection (with Precast Hollow Core Slabs & Topping Concrete)

A total of six specimens were tested in this series. All the specimens in this series were complete with the topping concrete and the hollow core slabs to simulate the final condition in practice. Four specimens HCS1, HCS2, HCS3 & HCS4 (Figure 4.7 & 4.8) were tested under monotonic loading and two specimens HCS5 & HCS6 (Figure 4.9) were tested under cyclic loading. The precast beam size was 500 x 460 mm and cast-in-place portion was 300 x 340 mm. This kind of configuration is to facilitate the installation of hollow core slabs. Hollow core slabs (referring to Appendix B) can be placed at the edge of precast beam during installation and the 75 mm topping concrete over the hollow core slabs can be cast together with the cast-in-place portion and the hybrid connection at the final stage (concrete grade referring to Appendix B). The width of topping concrete (hence the flange width) was 1000 mm. Various steel reinforcements were used in different specimens in order to study the influence of steel ratio on the moment-rotation behaviour. The summary of all test specimens is shown in Table 4.2.

The material properties of the major components used in the specimens, detailed configurations of specimens, fabrication procedure of specimens and instrumental arrangements are presented in the Appendix B ~ E respectively.

4.3 Test Set-up and Testing Procedures

The test set-up for specimens under monotonic loading is shown in Figure 4.10 & Photo 4.1. Before testing, each specimen was white washed for easy identification of crack information and for marking crack pattern on the beam surface. The specimen was placed on the metal roller bearings at the support locations. Each test was started by applying the load in an increment of 5 kN until a load level when cracks began to appear. After some flexure cracks had been generated, the load

increment was then increased to 10 kN until the beam failed. At each load increment, the displacement and load were recorded.

The test set-up for specimens under cyclic sway loading is shown in Figure 4.11 & Photo 4.2. The bottom of the column was pinned to the strong floor. On each side of the beam, there are two supporting legs, each with a 500 kN load cell to measure the reacting forces. Connection moments were measured based on the reacting forces from load cells. The lateral loading was applied to the specimen at the column head by a hydraulic actuator with a maximum load capacity of ± 2000 kN. The lateral load P was calculated from the readings of load cells based on the force equilibrium condition. The cyclic testing procedure on structural steel elements recommended by European Convention for Construction Steelwork (1986) was adopted in this study. A short testing procedure was used as a reference to derive the loading procedure. In the first stage of loading, load increments based on force control were used. The strength resistance capacity P_y of the hybrid connection was estimated before testing. For an interior joint, the applied force resistance P_y was determined from:

$$P_y \approx \frac{M_{sag,y} - M_{hog,y}}{H_{col}} \quad \text{Eq. 4-1}$$

where, $M_{sag,y}$ and $M_{hog,y}$ are the estimated yielding moments of connections under the sagging and hogging actions. Values of $M_{sag,y}$ and $M_{hog,y}$ are based on the monotonic test results. H_{col} is the height of the column.

The first load cycle was carried out at a loading level of $\pm 0.4P_y$. The second load cycle was performed at a loading level of $\pm 0.65P_y$, and the corresponding measured peak displacement is (δ^-, δ^+) , where '+' denotes that a pushing force was applied and '-' denotes that a pulling force was acted. The reference yielding

displacement Δ_y at P_y is determined by extrapolating from δ_y as shown in Figure 4.12.

$$\Delta_y = \frac{\delta_y}{0.65} \quad \text{Eq. 4-2}$$

At the first and second loading levels, load applications were based on force-control. The successive loading cycles of maximum levels at $1.0 \Delta_y$, $1.5 \Delta_y$, $2.0 \Delta_y$, $3.0 \Delta_y$ and so on were then performed. Displacement control for the loading and reloading branches and force control for the unloading branches (from the peak displacement to the applied force equal to zero) were used alternately. In general, two or three cycles for each loading level were applied. A test was terminated if the failure of the specimen or the stroke limitation of the actuator was reached. A typical history of cyclic loading procedures is illustrated in Figure 4.13.

4.4 Summary

The test programme consists of three series, namely bare steel connection, partial hybrid connection and complete hybrid connection. Test specimens, test set-up and testing procedures for specimens under monotonic and cyclic loadings are described. The material properties of the major components used in the specimens, detailed configurations of specimens, fabrication procedure of specimens and instrumental arrangements are illustrated in the Appendix B ~ E respectively.

Table 4.1 Details of Bare Steel Connection Specimens (BS1, BS2 & BS3)

Specimen (BS1, BS2 & BS3)	Details
Steel Universal Beam (Grade 43)	356 x 171 UB 45 (Yield strength = 456.5 N/mm ²)
Steel Universal Column (Grade 43)	356 x 171 UB 67 (Yield strength = 403.3 N/mm ²)
Single Steel Plate (Grade 43)	130 x 300 x 10 mm thk (Yield strength = 322.3 N/mm ²)
Welds (E43)	8 mm fillet weld
Bolts	20 mm HSFG bolt
Clearance hole	22 mm in diameter

Table 4.2 Summary of Test Specimens

Test Series	Specimen	Beam Reinforcement		Loading
		Top	Bottom	
Series A – Bare steel connection	BS1			Mono
	BS2			Mono
	BS3			Cyclic
Series B – Partial hybrid connection (without hollow core slabs & topping concrete)	HC1	2T16	2T13	Mono (hogging)
	HC2	2T16	2T13	Mono (sagging)
	HC3	2T16	2T13	Cyclic
Series C – Complete hybrid connection (with hollow core slabs & topping concrete)	HCS1	2T16	2T13	Mono (hogging)
	HCS2	2T16	2T13	Mono (sagging)
	HCS3	2T25	2T16	Mono (hogging)
	HCS4	2T25	2T16	Mono (sagging)
	HCS5	2T16	2T13	Cyclic
	HCS6	2T25	2T16	Cyclic

Figure 4.1 Single plate connection

Figure 4.2 Specimens with bare steel connections under monotonic loading (BS1 & BS2)

Figure 4.3 Specimen with bare steel connection under cyclic loading (BS3)

Figure 4.4 Specimens with partial hybrid connections under monotonic loading (HC1 & HC2)

Figure 4.5 Section A-A & A1-A1 (HC1 & HC2)

Figure 4.6 Specimen with partial hybrid connections under cyclic loading (HC3)

Figure 4.7 Specimens with complete hybrid connections under monotonic loading (HCS1, HCS2, HCS3, HCS4)

Figure 4.8 Section B-B & B1-B1 (HCS1, HCS2, HCS3, HCS4)

Figure 4.9 Specimens with complete hybrid connections under cyclic loading (HCS5, HCS6)

Figure 4.10 Test set-up for specimens under monotonic loading

Figure 4.12 Determination of reference displacement

Figure 4.13 Typical loading procedure for cyclic test

Photo 4.1 Test set-up for specimens under monotonic loading

Photo 4.2 Test set-up for specimens under cyclic loading

Chapter 5: Experimental Results and Discussions

5.1 Introduction

In this chapter, the experimental results of test series A, B and C are presented and discussed. The experimental programme was carried out to study the behaviour of hybrid connection under monotonic loading as well as cyclic loading conditions. The main focus of the study is the moment-rotation ($M - \theta$) relationship of hybrid connection. The global behaviour such as lateral force-displacement ($P - \Delta$) response and hysteretic characteristic of hybrid connection are also discussed. Visual observations of cracks are reported. The contribution & behaviour of the components of the hybrid connections are evaluated based on strain measurements.

5.2 Defining the Moment-Rotation Relationship

In order to define the $M - \theta$ relationship, the two parameters M and θ must be defined first. Unfortunately, different researchers have different definitions based on different assumptions. It is important to have a consistent definition, which would give a common basis to develop methods suitable for the prediction of connection behaviour as well as the analysis of semi-rigid composite frames. In this thesis, the following definition of M and θ will be used.

Since the value of the bending moment varies along the beam axis, it is appropriate to define M as the moment value at the column face, which may be considered to govern the joint rotational response. The joint rotation θ is defined as the relative rotation between the end of the beam and the column face (Figure 5.1).

5.3 Experimental Results for Specimens under Monotonic Hogging Moment

The specimens of bare steel connections BS1 & BS2 in series A were tested under monotonic loading. Due to non-symmetrical section of the hybrid connection, the flexural behaviours under hogging and sagging moment are not the same. Therefore, the specimen HC1 in series B and specimens HCS1 & HCS3 in series C were tested under monotonic hogging moment (specimens were loaded up-side down). The specimen HC2 in series B and specimens HCS2 & HCS4 in series C were tested under monotonic sagging moment.

5.3.1 Specimens BS1 & BS2 in Series A

The bare steel connections BS1 & BS2 were tested to study the moment-rotation relationship of single plate connection under monotonic loading. The moment-rotation responses are shown in Figure 5.2. The tested results for BS1 and BS2 were quite close. It should be noted that the moment-rotation curve of single plate connection with HSFG bolts basically consisted of four stages. The first stage was almost a linear response until slippage occurred at the friction connection. The slippage occurred at about 23 kN.m and the rotation of steel connection was about 5 mrad. As the load was increased further, the slip started to occur gradually from the outmost bolt in a sudden and audible movement. The connection slipped rapidly with little increase in connection moment until all the HSFG bolts went into bearing condition. By observation of the bolt movement relative to its initial position together with the numbers of sound caused by the slippage, the rotation of steel connection at the end of this stage was about 20 mrad. This was considered as the second stage. At the third stage, all HSFG bolts bore against the edge of the bolt holes. The moment-rotation curve was non-linear at this stage. The applied connection moment was increased until the top beam flange touched the column flange. The rotation at the end of this stage was about 60 mrad. At the final stage, the bearing force between HSFG bolts and connected parts increased substantially due to the top beam flange bearing against the column flange. This resulted in rapid

increase of connection moment. The connection rotated about the top beam flange and eventually failure occurred due to shear failure of bottom bolts (Photo 5.1).

After failure, the connection was removed from the testing machine and inspected. The fractured surfaces and distortion of the bolt holes were observed. As shown in Photo 5.2, the distortions of bolt holes followed the pattern according to their respective bearing between bolts and connected parts.

5.3.2 Specimen HC1 in Series B

The specimen HC1 was tested to study the behaviour of partial hybrid connection under monotonic hogging moment. The moment-rotation relationship can be considered as a three-stage curve as shown in Figure 5.3. At the first stage, which was prior to the cracking of concrete, the stiffness of hybrid connection was very high and the slope of $M - \theta$ curve was almost linear. The strains in the rebars and single plate connection were very small which indicated that they had very little contribution at this stage. As the load was increased, the connection stiffness reduced as the cracks propagated and the steel bars were mainly responsible for carrying the tensile forces. The moment-rotation relationship was non-linear at this stage. The final plastic stage was attained as a result of yielding of the top rebars in the hybrid connection.

The first flexural crack of the specimen occurred at a moment of 70 kN.m at the critical section, which was the interface between the cast-in-place concrete and the steel column flange at the joint. This was due to the weak bond between the concrete and steel column flange. As the loading progressed, more flexural cracks occurred around the critical sections. As shown in Photo 5.3, the flexural cracks along the critical sections kept on developing and propagating towards the bottom of the connection. Some splitting cracks were also observed at the interfaces between the precast beam and the cast-in-place concrete. This was also due to the weak bond between these two parts. With further increase of loading, the neutral axis of the connection kept on moving towards the bottom of the connection.

Eventually, the moment resistance was reduced when the concrete reached the maximum compressive strain and crushed concrete was observed in the compressed region.

As shown in Figure 5.2 & 5.3, the initial stiffness and moment capacity of bare steel connection (BS1 & BS2) is much less than those of hybrid connection (HC1). It should be noted that there is a ten times difference in the axial limit of rotation in Figure 5.2 (100 mrad) and Figure 5.3 (10 mrad).

5.3.3 Specimens HCS1 & HCS3 in Series C

The specimen HCS1 was tested to study the behaviour of complete hybrid connection under monotonic hogging moment. As shown in Figure 5.3, the moment-rotation relationship consisted of linear, non-linear and plastic stages similar to HC1. The first flexural crack of the specimen occurred at a moment of 200 kN.m at the critical section. This cracking moment was much higher than that of HC1 due to the presence of mesh reinforcement in the topping concrete. After cracking, the mesh reinforcement in the topping concrete and the top T16 rebars in the beam were mainly responsible for carrying the tensile forces. With the progress of test, the flexural cracks started from the topping concrete and kept on developing and propagating towards the bottom of the connection along the critical sections (Photo 5.4). These flexural cracks reduced the shear capacity of hollow core slabs and may affect the development of full prestressing force as well as the bond transfer if these flexural cracks occurred within the transmission zone.

The configuration of HCS3 had the same configuration as HCS1 except for the different longitudinal rebars used in the beam (Table 4.2). The first flexural crack of the specimen occurred at a moment of 270 kN.m, which is higher than HCS1 because rebars with bigger diameter were used. The crack pattern was similar to HCS1 (Photo 5.5) and flexural cracks were found to cross through the whole topping concrete along the critical sections (Photo 5.6).

Due to the presence of mesh reinforcement, the steel ratio in HCS1 and HCS3 was much higher than HC1. As shown in Photo 5.4 & 5.5, the flexural cracks extended closely to the bottom of the connection, which meant that the neutral axis had been reduced significantly at the moment of specimen failure. Therefore, the failure of the specimen was due to the maximum compressive strain of concrete being reached. The test results of HC1, HCS1 and HCS3 are compared in Figure 5.3. It was observed that the moment capacity and stiffness of HCS1 were higher than those of HC1 due to the presence of mesh reinforcement in topping concrete, and the enhancement of moment capacity of HCS3 compared to HCS1 was due to the increase of reinforcements in HCS3.

5.3.4 Measured Strains for Specimens under Monotonic Hogging Moment

In the above sections, experimental results in terms of moment-rotation relationships and visual observations of cracks have been described. However, to interpret the component behaviour of a hybrid connection and for the purpose of calibrating the component-based model, it is generally not sufficient by just providing the information of the connection moment-rotation relationship. Performance of key components should be acknowledged in evaluating their contributions in the connection strength, stiffness and ductility. In view of the limited available information to reliably describe the behaviour of connection components, several parameters, including strains of longitudinal bars, strains of mesh reinforcements and strains of steel beam were monitored during the tests. These measurements provide a wide range of information to understand the behaviour of connection components and therefore the contributions of these components to the behaviour of entire hybrid connection can be evaluated.

The strain distributions on the longitudinal rebars, mesh reinforcements and steel beams of HC1, HCS1 and HCS3 were measured (referring to the locations of strain gauges in Appendix E, Figure E.5, E.7 & E.8). The strain measurements were investigated in detail as follows.

- **Strains on longitudinal rebars**

Strain measurements of top rebars (R1 & R2) on specimens HC1, HCS1 and HCS3 were shown in Figure 5.4. Prior to the cracking of concrete, top rebars together with concrete resisted the tensile forces and the tensile strains in the top rebars were very small at this stage. After the cracking of concrete, which were indicated by the turning points in the moment-strain curves, tensile forces at the cracking planes were mainly transferred to the top rebars, which caused significant increase of tensile strains in the top rebars. Eventually, all top rebars exceeded the yielding strength when the maximum connection moments were reached.

Strain measurements of bottom rebars (P1 & P2) on specimens HC1, HCS1 and HCS3 were also shown in Figure 5.5. All bottom rebars were under compression since the beginning of loading and remained in the elastic range when the maximum moments were reached. It was observed large strains were measured in HC1 compared to HCS1 & HCS3. The reason could be that the moment capacity of HC1 was much less than that of HCS1 & HCS3 due to the smaller steel ratio in the section, therefore, HCS1 & HCS3 failed in a more brittle fashion than HC1 and less strain on bottom rebars were measured.

- **Strains on mesh reinforcements**

As shown in Figure 5.6, the measured strains of mesh reinforcements (M1~M4) on HCS3 were plotted against the connection moment. Similar to top rebars, tensile strains in mesh reinforcements increased linearly before the cracking of concrete and became non-linear after cracking. Due to the shrinkage cracks and weak bond between the concrete and steel column, flexural cracks started from the concrete and steel column interfaces and propagated quickly towards the edge of topping concrete during the loading process. This may explain why the longitudinal tensile strain in mesh reinforcements along the topping concrete section reduced with the distance increasing away from the column after cracking (for the same connection moment, tensile strain $M1 > M2 > M3 > M4$). Eventually, cracks penetrated the

topping concrete and all mesh reinforcements exceeded the yielding strength when the maximum connection moment was attained.

- **Strains on beam web**

The measured strains of the steel beam web (B1~B3) on HCS3 were plotted against the connection moment in Figure 5.7. Before the cracking of concrete, the strain of B1 was very small and strain gauges B2 & B3 were under compression, which indicated that the single plate connection was initially under compression. After cracking, the strains of B1 and B2 turned into tension rapidly which indicated the depth of neutral axis dropped substantially towards the bottom of the connection. As the loading progressed, the depth of neutral axis continued to reduce until the strain of B3 eventually turned into tension. Therefore, the single plate connection finally turned into tension due to further reduction of the depth of neutral axis.

- **Variation of neutral axis**

Knowledge of the variation of neutral axis is essential to understand the behaviour of each component in the hybrid connection. In this research, the depth of neutral axis was calculated based on the measured strains in the top and bottom rebars at the connection section. In general, the reinforcing bar strains are a reasonable approximation to the neutral axis depth, however, the strain gauges provide rebar strains at one location on the rebar, which may not be representative of the average strains in the cross section. The variations of neutral axis of HC1, HCS1 and HCS3 were shown in Figure 5.8. Prior to the cracking of concrete, the neutral axis was around the initial position which was based on the internal force equilibrium of the uncracked connection section. After cracking, the neutral axis reduced significantly to the bottom of connection due to the loss of concrete tensile strength. As the loading progressed, the neutral axis shifted down further and the specimen failed due to the maximum compressive strain of concrete was reached. This was consistent with the experimental observations of flexural cracks shown in Photo 5.3 ~ Photo 5.5.

5.4 Experimental Results for Specimens under Monotonic Sagging Moment

5.4.1 Specimen HC2 in Series B

The specimen HC2 was tested to study the behaviour of partial hybrid connection under monotonic sagging moment. As shown in Figure 5.9, it was observed that the connection moment continued to increase until the specimen failed. This was due to the presence of the single plate connection in the tension zone, which increased the tensile capacity in the connection under sagging moment. Based on the strain measurements, it was found that the tensile reinforcements had already exceeded the yielding strength when the maximum moment capacity was attained, while single plate connection continued to enhance the connection moment.

The first flexural crack of the specimen occurred at a moment of 110 kN.m at the critical section. Unlike the test results under monotonic hogging moment (Figure 5.3), the cracking moment can not be identified easily from the moment-rotation curve due to the contribution of single plate connection in the tension zone (Figure 5.9). Similar to HC1, the critical sections were the interfaces between the cast-in-place concrete and the steel column flanges at the joint. In addition, the interface between the end of precast beam and cast-in-place concrete was also likely to develop splitting cracks due to the weak bond between them. After cracking, the bottom rebars and single plate connection were responsible for carrying the tensile forces. The turning point of the connection moment-rotation curve was due to the slip moment of single plate connection being reached instead of the yielding of steel reinforcements. As the loading progressed, the flexural cracks along the critical sections kept on developing and propagating towards the top of the connection, causing an increase of compressive stress in the concrete. Due to the reduction of neutral axis, the single plate connection was mainly under tension which continued to increase the connection moment until the failure of specimen. As shown in Photo 5.7, the depth of neutral axis was reduced to a very small distance and the failure of the specimen was due to compression failure of concrete.

5.4.2 Specimens HCS2 & HCS4 in Series C

The specimen HCS2 was tested to study the behaviour of complete hybrid connection under monotonic sagging moment. The first flexural crack of the specimen occurred at a moment of 130 kN.m at the critical section (Figure 5.9). Similar to HC2, the cracking of concrete was not clearly reflected in the moment-rotation curve. The slight increase of cracking moment compared to HC2 was due to the contribution of topping concrete as well as hollow core slabs. With the progress of test, the flexural cracks started from the topping concrete and propagated towards the top of the connection along the critical sections (Photo 5.8). The configuration of HCS4 had the same configuration as HCS2 except different longitudinal rebars used in the beam (Table 4.2). The first flexural crack of the specimen occurred at a moment of 150 kN.m and the crack pattern was similar to HCS2 as shown in Photo 5.9.

Under monotonic sagging moment, the whole single plate connection was under tension since the beginning of loading. The presence of the single plate connection in the tension zone was similar as increasing the tensile reinforcement in the hybrid connection. Unlike the test results under monotonic hogging moment, the cracking of concrete was not clearly reflected in the moment-rotation curve due to the contribution of single plate connection in the tension zone. Moreover, the turning point of the connection moment-rotation curve was due to the slip moment of single plate connection being reached instead of the yielding of steel reinforcements. The single plate connection continued to enhance the connection moment when the tensile reinforcement had already reached the yielding strength. As the depth of neutral axis continued to reduce, the specimen eventually failed due to the concrete reaching its maximum compressive capacity. Therefore, for small steel ratios, different steel ratios contributed little to the moment-rotation relationships by comparing HCS2 and HCS4 due to the contribution of single plate connection (Figure 5.9). However, through the comparison between HC2 and HCS2, it should be noted, the presence of topping concrete and hollow core slabs has enhanced the moment capacity due to the increase of concrete compressive area.

5.4.3 Measured Strains for Specimens under Monotonic Sagging Moment

The arrangements of strain gauges on the longitudinal rebars, mesh reinforcements and steel beams of HC2, HCS2 & HCS4 were also shown in Figure E.5, E.7 & E.8 in Appendix E. The strain measurements were described as follows.

- **Strains on longitudinal rebars**

Strain measurements of top rebars (R1 & R2) on specimens HC2 and HCS4 were shown in Figure 5.10. At the beginning of loading, all top rebars were under compression and the compressive strains increased linearly. As the loading progressed, the neutral axis continued to move towards the top of connection and the compressive strains in the top rebars eventually turned into tension.

As shown in Figure 5.11, strain measurements of bottom rebars (P1 & P2) on specimens HC2 and HCS4 were plotted. All bottom rebars under sagging moment were under tension and the turning points of strain curves indicated the cracking of concrete. However, the slopes of moment-strain curves were less reduced after cracking compared to those under hogging moment shown in Figure 5.4. This is because the single plate connection together with bottom rebars resisted the tensile forces at the cracking planes. Eventually, all bottom rebars exceeded the yielding strength when the maximum connection moment was reached.

- **Strains on mesh reinforcements & beam web**

Under sagging moment, all mesh reinforcements of HCS2 were under compression and within the elastic range when the maximum connection moment was attained (Figure 5.12). The measured strains on beam web (B1~B3) of HCS2 were also plotted against the connection moment in Figure 5.13. All B1, B2 and B3 were under tension since the beginning of loading and kept on increasing until the maximum moment capacity was attained. This indicated that the entire single plate

connection was under tension since the beginning of loading and continued to contribute to the connection moment until the failure of the specimen.

- **Variation of neutral axis**

The variations of neutral axis of HC2, HCS2 and HCS4 were shown in Figure 5.14. Unlike the rapid reduction of neutral axis depth under the hogging moment (Figure 5.8), the depth of neutral axis reduced gradually after the cracking of concrete. This is due to the presence of single plate connection in the tension zone which increases the tensile capacity of the connection under sagging moment. As the loading progressed, the neutral axis shifted up further and the single plate connection continued to increase the connection moment even when the bottom rebars had already reached the yielding strength. Eventually, the specimen failed due to the maximum compressive capacity of concrete was reached. This was consistent with the experimental observations of flexural cracks shown in Photo 5.7 ~ Photo 5.9.

5.5 Experimental Results for Specimens under Cyclic Loading

Specimens BS3 in series A, HC3 in series B and HCS5 & HCS6 in series C were tested under cyclic sway loading. For specimens under cyclic loading, connection moments were measured based on the reacting forces from load cells and connection rotations were calculated based on the LVDT readings. Moment-rotation ($M - \theta$) relationships, lateral force-displacement ($P - \Delta$) responses and hysteretic characteristics were described.

5.5.1 Specimen BS3 in Series A

The specimen BS3 had the same connection detail as BS1 & BS2. The purpose of this test was to study the cyclic behaviour of single plate connection and compare with the behaviour of BS1 & BS2 under monotonic loading. Connection moments were measured based on the reacting forces from load cells and connection rotations

were calculated based on the LVDT readings. The lateral force P was calculated from the reacting forces of the load cells based on the force equilibrium condition. The top displacement of column Δ was measured using a LVDT. Complete curves of $P - \Delta$ and $M - \theta$ for BS3 are shown Figure 5.15 & 5.16 respectively.

Similar to those under monotonic loading, the moment-rotation relationship remained linear before the slip load was exceeded. As the load was increased, the slip started to occur gradually from the outmost bolt in a sudden and audible movement. During this period, the connection moment did not increase very much until all the bolts went into bearing. The connection moment started to increase substantially as a result of bolts bearing against the connected parts. The cyclic $M - \theta$ relationship reflected a kind of cyclic slip and bearing behaviour. It was observed that the cyclic hysteretic behaviour combined an increasing slip deformation and a decreasing slip load. The elongation of slip deformation was due to the increasing bearing deformation and the slip load deterioration was attributed to the changing frictional characteristics of faying surfaces between the connected parts. The hysteretic loops of lateral displacement measured at the top of the column ($P - \Delta$) and moment-rotation relationship ($M - \theta$) curves were stable and ductile. As shown in Figure 5.16, the unloading part stiffness was almost the same as the initial stiffness. The moment-rotation curve of monotonic test (BS1) was also plotted for comparison and it was observed that the envelope of the cyclic curves correlated well with the monotonic curve. Therefore, it may be reasonable to consider the monotonic curve as the skeleton curve for single plate connection under cyclic loading.

5.5.2 Specimen HC3 in Series B

The specimen HC3 had the same connection details as HC1 & HC2. The purpose of this test was to study the cyclic sway behaviour of partial hybrid connection in order to compare with the behaviour of HC1 & HC2 under monotonic loading. Complete curves of $P - \Delta$ and $M - \theta$ for HC3 are shown Figure 5.17 & 5.18 respectively.

The first cycle was carried out at loading of $\pm 0.4P_y$ (95.2 kN), there was no cracks observed in the hybrid connection. The second load cycle was performed at a loading level of $\pm 0.65P_y$ (154.6 kN), cracks appeared along the critical sections which were the interfaces between cast-in-place concrete and steel column flanges at the joint. At the successive loading level at $1.0\Delta_y$ (206.5 kN), some flexural cracks occurred at the top of the beam in the vicinity of the joint and splitting cracks appeared at the interfaces between precast beam and cast-in-place concrete at the bottom of the beam. As further loading levels were applied as $1.5\Delta_y$ (264.7 kN), $2.0\Delta_y$ (260.7 kN), $3.0\Delta_y$ (255.3 kN), the cracks were developed and propagated. At the final state, the concrete around the column at the top of the beam was completely crushed due to compression (Photo 5.10).

The hysteretic loops were stable and ductile. Pinching which was mainly caused by the opening and closure of the cracks along the critical sections was observed. The moment-rotation curves for the left and right connections were in good agreement because of the symmetrical construction details. The ultimate hogging moment for the left and right connection occurred at the loading level of $\pm 1.5\Delta_y$ (264.7 kN). Thereafter, the strength started to degrade. The ultimate hogging moment was much lower than that of HC1, which was under the monotonic hogging moment. This was likely to be caused by the fast development of cracks along the critical sections under the cyclic loading which accelerated the damage of the concrete. Therefore, the stiffness and strength of hybrid connection were much reduced and unable to achieve the same ultimate strength as those under monotonic loadings.

The ultimate sagging moment appeared at the loading level of $\pm 1.5\Delta_y$ (264.7 kN). The ultimate sagging moment value was also lower than that of HC2, which was under monotonic sagging moment. The hysteretic loops were ductile and stable until the loading level of $\pm 1.5\Delta_y$ (264.7 kN). After which, significant strength degradation was observed.

5.5.3 Specimens HCS5 & HCS6 in Series C

The specimen HCS5 had the same connection details as HCS1 & HCS2. The purpose of this test was to study the cyclic sway behaviour of complete hybrid connection and compare with the behaviour of HCS1 & HCS2 under monotonic loading. Complete curves of $P - \Delta$ and $M - \theta$ for HCS5 are shown Figure 5.19 & 5.20 respectively.

The cracks were found concentrated along the critical sections. Similar to HC3, the splitting cracks were also found at the interfaces between precast beam and cast-in-place concrete. As further loading levels were applied, the cracks were developed and propagated. At the final state, the concrete around the column at the bottom of the beam was badly crushed due to compression and the topping concrete was completely separated by the cracks along the critical sections (Photo 5.11 & 5.12).

The hysteretic loops under cyclic loading were ductile and stable. The ultimate hogging and sagging moment for the left and right connection occurred at the loading level of $\pm 1.5\Delta_y$ (288.1 kN). Compared to the behaviour under monotonic loading (HCS1 & HCS2), the cyclic response characterises the reduction of concrete stiffness and the decay of concrete strength due to the damage of concrete (Figure 5.20).

The specimen HCS6 had the same configuration as HCS5 except different size of top and bottom rebars used. Complete curves of $P - \Delta$ and $M - \theta$ for HCS6 are shown Figure 5.21 & 5.22 respectively. Because of the increase of longitudinal rebars in the hybrid connection, the stiffness of the hybrid connection was therefore enhanced relatively to the precast column. During the cyclic loading, the column became the slender member and likely to fail prior to the hybrid connection. As shown in Photo 5.13, the failure of the specimen was due to the damage of the column head instead of the hybrid connection. Therefore, the moment capacity of HCS6 was much lower than those under monotonic loadings (HCS3 & HCS4) due to different failure mode (Figure 5.22).

After the test, the specimen HCS5 was removed from the testing machine and inspected. As shown in Photo 5.14, the concrete at the joint was removed to check the mesh reinforcements, steel rebars as well as the single plate connection. Some mesh reinforcements were found broken near the column due to the repeat tension and compression. After the removal of HSFG bolts, the fractured surfaces of both beam web and steel plate as well as the distortion of bolt holes of single plate connection displayed a similar pattern to those bare steel connections.

5.5.4 Measured Strains for Specimens under Cyclic Loading

- **Strains on beam web & variation of neutral axis**

The measured strains (B1~B3) on beam web of BS3 were plotted against the load stage (load increment) in Figure 5.23. Under cyclic loading, the top and bottom flange of beam were under tension and compression alternately in a symmetrical response. As shown in Figure 5.24, the movement of neutral axis was consistently around the middle height of connection except some points far away from the connection centreline, which were caused by those connection moments near zero under the cyclic loading.

- **Strains on longitudinal rebars & variation of neutral axis**

Figure 5.25 shows the strains of top rebars (R5 & R6) and bottom rebars (P5 & P6) of HC3 under cyclic loading. The top and bottom rebars were under tension and compression alternately at different loading levels. The tensile strains in the bottom rebars were much less than those in the top rebars due to the presence of single plate connection which increases the tensile capacity of connection under sagging moment. The variation of neutral axis under cyclic loading was also presented in Figure 5.26. As expected, the neutral axis of HC3 varied from the connection top to the connection bottom under hogging moment and altered from the connection

bottom to the connection top under sagging moment during the cyclic loading process.

5.5.5 Energy Dissipation of Hybrid Connections under Cyclic Loading

The energy capacity is the factor of utmost importance when evaluating the performance of a structure subject to seismic attacks. Its value is generally deformation-dependent and serves as an indication of dissipating energy for the structure through the inelastic range. The dissipated energy J_i in the i th loading cycle is defined as the area encircled by the restoring force-deformation curve defined in Figure 5.27. The energy per cycle may be further separated into two parts, J^+ for positive loading and J^- for negative loading.

Energy dissipating capacities of hybrid connections for HC3 and HCS5 are shown in Figure 5.28 and Figure 5.29 respectively. Due to non-symmetrical properties of hybrid connection under positive and negative loading, the values of dissipated energy for positive loading J^+ and negative loading J^- are different. The energy is dissipated by the hybrid connections, concrete cracking and friction between interfaces of components. It can be observed that specimens with hybrid connections possess certain energy dissipating capacity, which suggests that this kind of hybrid connection has the potential being used in moderate earthquake prone regions.

5.6 Summary

A total of twelve specimens comprising bare steel connections, partial hybrid connections and complete hybrid connections were tested. Three loading cases, namely monotonic hogging moment, monotonic sagging moment and cyclic loading were tested with four specimens each. The moment-rotation ($M - \theta$) relationships, experimental observations and strain measurements of key components were presented. Following conclusions can be made:

- Tests of specimens indicated that the proposed hybrid connection had certain results in the strength and ductility, which suggests that such hybrid connection may be suitable for the precast construction industry.
- The test results showed a typical three-stage behaviour for single plate connection under monotonic loading before the beam flange contacted the column flange. Under cyclic loading, the hysteretic loops of moment-rotation relationship were stable and ductile. The monotonic moment-rotation curve could be used as the skeleton curve for cyclic loading.
- The performance of hybrid connection depends mainly on the contribution of each component. Adjustment of variables, such as the amount of top and bottom reinforcement influenced the response noticeably. Cracks were developed along the critical sections which are the interfaces between the cast-in-place concrete and the steel column flanges at the joint due to the weak bond between concrete and steel.
- Due to the continuity of top and bottom reinforcements crossing the joint, hybrid connection can achieve significant moment resistance under both monotonic hogging and sagging moment. Under monotonic hogging moment, the steel reinforcement yielded before the concrete reached its maximum capacity. The stiffness and moment capacity increased as a result of increasing reinforcements in the beam and topping concrete.
- Under monotonic sagging moment, the presence of the single plate connection in the tension zone is similar to increasing the tensile reinforcements in the hybrid connection. The single plate connection continued to increase the connection moment after the tensile reinforcements exceeding the yielding strength. Therefore, the bottom reinforcement contributes little to the overall connection behaviour after the yield strength being exceeded. However, the presence of topping concrete and hollow core slabs enhanced the moment capacity due to the increase of concrete compressive area.
- The hysteretic loops of hybrid connection were found to be ductile and stable under cyclic loading. However, the stiffness and strength of hybrid connection were much reduced and unable to achieve the same ultimate strength as those

under monotonic loadings due to the fast development of cracks under the cyclic loading which may accelerate the damage of the concrete.

- Under monotonic hogging moment, top rebars and mesh reinforcements were in tension and bottom rebars were in compression. Before the cracking of concrete, the neutral axis was around the initial position and the single plate connection was under compression. After cracking, the neutral axis reduced significantly to the bottom of connection and the single plate connection gradually turned into tension due to further reduction of the depth of neutral axis.
- Under monotonic sagging moment, top rebars and mesh reinforcements were in compression and bottom rebars were in tension. Single plate connection was completely under tension since the beginning of the loading. Unlike the rapid reduction of neutral axis under hogging moment, the depth of neutral axis reduced gradually after the cracking of concrete due to the presence of single plate connection in the tension zone which enhances the tensile capacity of the hybrid connection under sagging moment.

Table 5.1 Experimental Results of Force-Displacements ($P - \Delta$) Curves

Specimen	Loading (kN)		Displacement (mm)		Initial stiffness (kN/mm)	
	$P_{\max}^+$	$P_{\min}^-$	Δ^+	Δ^-	K_i^+	K_i^-
BS3	48.6	-51.3	134.8	-134.3	1.68	1.19
HC3	262.7	-267.3	45.2	-42.6	9.9	10.0
HCS5	278.0	-290.7	35.8	-32.9	12.6	11.1
HCS6	317.3	-333.3	30.9	-42.1	13.6	11.7

Table 5.2 Experimental Results of Moment-Rotation ($M - \theta$) Curves

Specimen	Moment (kN.m)		Rotation (mrad)		Initial stiffness (kNm/rad)	
	$M_{\max}^+$	$M_{\min}^-$	θ^+	θ^-	K_0^+	K_0^-
BS3	53.1	-50.7	54.3	-55.0	4.9	6.7
HC3	300.9	-456	7.2	-8.8	81.2	146.3
HCS5	368.1	-530.0	6.4	-18.3	141.1	160.7
HCS6	458	-406	3.5	-2.5	194.2	165.7

Figure 5.1 Definition of connection rotation

Figure 5.2 Test results of BS1 & BS2

Figure 5.3 Comparison of specimens under monotonic hogging moment

Figure 5.4 Strain measurements on top rebars under monotonic hogging moment (HC1, HCS1 & HCS3)

Figure 5.5 Strain measurements on bottom rebars under monotonic hogging moment (HC1, HCS1 & HCS3)

Figure 5.6 Strain measurements on mesh reinforcements under monotonic hogging moment (HCS3)

Figure 5.7 Strain measurements on beam web under monotonic hogging moment (HCS3)

Figure 5.8 Variation of neutral axis under monotonic hogging moment (HC1, HCS1 & HCS3)

Figure 5.9 Comparison of specimens under monotonic sagging moment

Figure 5.10 Strain measurements on top rebars under monotonic sagging moment (HC2 & HCS4)

Figure 5.11 Strain measurements on bottom rebars under monotonic sagging moment (HC2, HCS2 & HCS4)

Figure 5.12 Strain measurements on mesh reinforcements under monotonic sagging moment (HCS2)

Figure 5.13 Strain measurements on beam web under monotonic sagging moment (HCS2)

Figure 5.14 Variation of neutral axis under monotonic sagging moment (HC2, HCS2 & HCS4)

Figure 5.15 $P - \Delta$ curve for BS3

Figure 5.16 Moment-rotation curve for BS3

Figure 5.17 $P - \Delta$ curve for HC3

Figure 5.18 Moment-rotation curve for HC3

Figure 5.19 $P - \Delta$ curve for HCS5

Figure 5.20 Moment-rotation curve for HCS5

Figure 5.21 $P - \Delta$ curve for HCS6

Figure 5.22 Moment-rotation curve for HCS6

Figure 5.23 Strain measurements on beam web under cyclic loading (BS3)

Figure 5.24 Variation of neutral axis under cyclic loading (BS3)

Figure 5.25 Strain measurements on top and bottom rebars under cyclic loading (HC3)

Figure 5.26 Variation of neutral axis under cyclic loading (HC3)

Figure 5.27 Definition of energy dissipation in ith cycle

Figure 5.28 Cyclic energy dissipation of specimen HC3

Figure 5.29 Cyclic energy dissipation of specimen HCS5

Photo 5.1 **Failure of single plate connection under monotonic loading (BS2)**

Photo 5.2 **Inspection of tested specimen (BS2)**

Photo 5.3 Cracking pattern for HCl (Side elevation)
Note: The specimen was loaded up-side down.

Photo 5.4 Cracking pattern for HCS1 (Side elevation)

Note: The specimen was loaded up-side down.

Photo 5.5 Cracking pattern for HCS3 (Side elevation)
Note: The specimen was loaded up-side down.

Photo 5.6 **Cracking pattern for HCS3 (Plan view)**

Photo 5.7 Cracking pattern for HC2 (Side elevation)

Photo 5.8 Cracking pattern for HCS2 (Side elevation)

Photo 5.9 Cracking pattern for HCS4 (Side elevation)

Photo 5.10 Cracking pattern for HC3 (Side elevation)

Photo 5.12 Cracking pattern for HCS5 (Plan view)

Photo 5.13 Cracking pattern for HCS6 (Side elevation)

Photo 5.14 Inspection of tested specimen (HCS5)

Chapter 6: Modelling of Moment-Rotation Relationship under Monotonic Loading

6.1 Introduction

As discussed in Chapter 3, the component-based mechanical model is chosen to analyse the behaviour of hybrid connection for its inherent merits. For the proposed mechanical model, the following basic assumptions are made:

- For the sections adjacent to the hybrid connection, plane sections remain plane after deformation.
- The column web was stiffened through concrete encasement, therefore, the column is assumed to be rigid and undeformable relative to the hybrid connection.
- There is no relative movement between steel beam and precast concrete beam.
- The local buckling and the shear deformation of beams are ignored.
- Geometrical non-linearity is not considered.

6.2 Idealization of Connections

As illustrated in Figure 6.1, the proposed component-based mechanical model is used to simulate the moment-rotation relationship of hybrid connection under monotonic hogging as well as sagging moment. Column centre-line is assumed to be rigid relative to the hybrid connection and represented by a rigid body AB. The end of the beam abutting the column face, which is within the cast-in-place concrete, can be represented by a rigid element CD. The connection zone is defined as the region between the column centre-line AB and the beam end CD, in which the distance between AB and CD is half of the depth of column section. The connection rotation θ is the relative rotation between the end section CD and the column face. The rigid body AB and rigid bar CD are joined by a series of distributed springs,

which simulate the connection components. There are three basic components in the proposed model: single plate connection, the longitudinal reinforcements and concrete infill. The force-displacement ($F - \delta$) relationship of each component under monotonic loading can be defined and represented by the equivalent spring in the model.

The concrete beam section is subdivided into a number of discrete layers n along the depth of section, each to be represented by a spring (Figure 6.2). The axial deformation of a spring element i located at a distance y_i of the connection can be given as:

$$\Delta_i = (y_i - \bar{y}) \cdot \theta \quad \text{Eq. 6-1}$$

Where Δ_i is the axial deformation of a spring element i . y_i is the distance of spring element i . $\bar{y}$ is the depth of neutral axis. θ is the rotation of the hybrid connection.

Once the deformation Δ_i is established, the corresponding force F_i to the connecting element can be derived based on the $F - \delta$ relationship of respective component.

$$F_i = K_i \cdot \Delta_i \quad \text{Eq. 6-2}$$

The total axial force and moment transmitted by the connecting elements can be expressed as:

$$F = \sum_{i=1}^n F_i \quad \text{Eq. 6-3}$$

$$M = \sum_{i=1}^n F_i \cdot y_i \quad \text{Eq. 6-4}$$

With an increase in rotation, the new neutral axis position is calculated. If the force-displacement relationship of connecting element is inelastic, iterations are needed until the axial force equilibrium condition is satisfied. As illustrated in the flow chart in Figure 6.3, the numerical procedure for the moment-rotation relationship of hybrid connection can be achieved. The force-displacement relationship of each component will be described in detail in the following sections.

6.3 Modelling of Single Plate Connection under Monotonic Loading

6.3.1 Introduction

Single plate connection is the fundamental component of the hybrid connection, therefore, the study of its contribution to the entire hybrid connection is essential. However, previous studies on the single plate connection are rare and incomplete. In this research, a mechanical model is proposed to simulate the behaviour of single plate connections with HSFG bolts as shown in Figure 6.4. Similar to the component-based model shown in Figure 6.1, the column centre-line is assumed to be rigid relative to the single plate connection and represented by rigid body EF. The end of beam can be represented by a rigid element GH. The distance between EF and GH is half of the depth of column section. Each spring represents the force transfer mechanism between one HSFG bolt and the connected parts. If the axial force-displacement ($F - \delta$) relationship between one HSFG bolt and connected parts can be defined, the moment-rotation ($M - \theta$) relationship of the entire connection can be calculated based on force equilibrium condition. The $F - \delta$ relationship of HSFG bolt will be described as follows.

6.3.2 Slip Mechanism

Before the slip, the $F - \delta$ relationship between HSFG bolt and connected parts is linear up to the slip resistance. The slip resistance is dependent on the bolt preload

and slip coefficient between the contacted surfaces. Based on the recommendations given by Fisher and Struik (1974), the slip resistance F_{slip} is given as:

$$F_{slip} = k_s . m . F_p \quad \text{Eq. 6-5}$$

where k_s : Slip coefficient ($k_s = 0.45$, for untreated surfaces in clearance holes);

m : Number of slip planes;

F_p : Pre-tension force in the bolt.

The initial slip stiffness K_{slip} is determined as a function of the deformation before slip δ_{slip} as follows:

$$K_{slip} = \frac{F_{slip}}{\delta_{slip}} \quad \text{Eq. 6-6}$$

Based on previous research on the load deformation response of bolted connections (Karsu, B., 1995), the value of deformation before slip δ_{slip} is determined as 0.0076 inch (0.193 mm) by conducting a statistical analysis of tested data.

6.3.3 Bearing Mechanism

After the slip, HSFG bolts will bear against the wall of bolt holes. The single plate connection progressively becomes a bearing type connection similar to those with ordinary bolts (without pre-tension force). Because the physical reality of a bolt bearing on the side of a bolt hole is a complex problem involving the material non-linearity, therefore, a three-parameter power model is proposed to simulate the non-linear force-displacement ($F - \delta$) relationship. These three parameters are the initial bearing stiffness $K_{i,bearing}$, ultimate bearing capacity $F_{bearing}$ and shape parameter n . In the proposed model, the force-displacement relationship between one bolt and connected parts can be given as follows:

$$\delta_{bearing} = \frac{F}{K_{i,bearing} \left[1 - \left(\frac{F}{F_{bearing}} \right)^n \right]} \quad \text{Eq. 6-7}$$

where $K_{i,bearing}$: Initial bearing stiffness of single plate connection with ordinary bolts;

$F_{bearing}$: Ultimate bearing capacity of single plate connection with ordinary bolts;

n : Shape parameter.

6.3.3.1 Initial Bearing Stiffness of Single Plate Connection with Ordinary Bolts

Based on the results from the finite element models and the experimental tests, Rex and Easterling (2003) proposed a model to predict the initial bearing stiffness, which compares favourably with the existing models given by Eurocode 3 Annex J (1994) and Tate and Rosenfeld (1946). It was concluded that the initial bearing stiffness depends on three primary stiffness values in the plate, namely bending, shearing and bearing stiffness. The initial bearing stiffness $K_{i,bearing}$ is given as:

$$K_{i,bearing} = \frac{1}{\frac{1}{K_{br}} + \frac{1}{K_b} + \frac{1}{K_v}} \quad \text{Eq. 6-8}$$

where K_{br} : Bearing stiffness;

K_b : Bending stiffness;

K_v : Shear stiffness.

The bearing stiffness K_{br} was derived by considering a bolt and bolt hole in their deformed state as shown in Figure 6.5 (a), where R1 and R2 are the radii of the bolt

and bolt hole respectively. The bending and shear stiffness were derived based on the assumption that the steel between the bolt and the end of the plate can be modelled as a rectangular elastic fixed end beam as shown in Figure 6.5 (b).

$$K_{br} = 120t_p f_y (d_b / 25.4)^{0.8} \quad \text{Eq. 6-9}$$

$$K_b = 32Et_p (L_e / d_b - 1/2)^3 \quad \text{Eq. 6-10}$$

$$K_v = 6.67Gt_p (L_e / d_b - 1/2) \quad \text{Eq. 6-11}$$

where d_b : Diameter of bolt (mm);
 E : Modulus of elasticity of steel (N/mm²);
 f_y : Yield stress of steel (N/mm²);
 G : Shear modulus of elasticity of steel (N/mm²);
 L_e : End distance of bolt (mm);
 t_p : Thickness of plate (mm).

6.3.3.2 Ultimate Bearing Capacity of Single Plate Connection with Ordinary Bolts

The most common strength model for predicting bearing failure was developed by Fisher and Struik (1974). They used a simple plate shearing model to develop an equation for predicting the bearing strength of plates. The AISC Specification (LRFD 1993) has adopted an equation recommended by Fisher and Struik to predict the ultimate bearing capacity $F_{bearing}$.

$$F_{bearing} = F_u \frac{L_e}{d_b} \leq 2.4F_u \quad \text{Eq. 6-12}$$

where L_e : End distance of bolt;

d_b : Diameter of bolt;

F_u : Tensile strength of steel.

6.3.3.3 Shape Parameter of Single Plate Connection with Ordinary Bolts

The ABAQUS finite element package is used to model the bearing behaviour of single plate connection with ordinary bolts. The 3D solid model incorporates the nonlinear material properties as well as the contact surfaces between contacting components. The details of modelling and parametric study will be described in the later sections. Based on the regression of numerical data, the shape parameter n can be derived as:

$$n = \frac{5.73\Delta}{1 + 0.439.\Delta.t_{web}^2} \quad \text{Eq. 6-13}$$

$$\Delta = \frac{F_{bearing}}{K_{i,bearing}} \quad \text{Eq. 6-14}$$

where $K_{i,bearing}$: Initial bearing stiffness of single plate connection with ordinary bolts (Eq. 6-8);

$F_{bearing}$: Ultimate bearing capacity of single plate connection with ordinary bolts (Eq. 6-12);

t_{web} : Thickness of beam web (mm).

6.3.4 Force-Displacement ($F - \delta$) Relationship of HSFG Bolt

From the experimental test, it was observed that the $F - \delta$ response of HSFG bolt consists of three typical stages (Figure 6.6). At stage I, the $F - \delta$ curve is linear up to the slip resistance F_{slip} and the deformation before slip is δ_{slip} . After the slip

resistance F_{slip} is exceeded, the bolt slippage occurs without increasing of load capacity. When the hole clearance $\delta_{clearance}$ is reached, the HSFG bolt starts to bear against the inner side of bolt hole until the slip load F_{slip} is exceeded. This is defined as stage II. At stage III, the HSFG bolt continues to bear against connected parts similar to the single plate connection with ordinary bolts. The $F - \delta$ relationship is non-linear at this stage. Based on Eq.6-7, the non-linear $F - \delta$ relationship of HSFG bolt at stage III can be calculated, therefore, the three-stage $F - \delta$ curve in Figure 6.6 can be completely defined.

Based on the mechanical model proposed in Figure 6.3, with the increase of rotation, the moment-rotation relationship of single plate connection can be calculated. Based on the measured material properties and the geometrical configuration of single plate connection tested in the experiments (BS1 & BS2), the predicted result has reasonable agreement with the test results (Figure 6.7). The computer program and predicted values are listed in Appendix F for verification. The difference between them may be attributed to the bolt misalignments with bolt holes which will be discussed in the later section. For single plate connections with different bolt diameter, beam web thickness and steel plate thickness, the initial bearing stiffness $K_{i,bearing}$, ultimate bearing capacity $F_{bearing}$ and shape parameter n at the third stage of $F - \delta$ relationship can be determined. Therefore, the proposed model could be extended to simulate the behaviour of single plate connection with different configurations.

6.3.5 Numerical Modelling of Single Plate Connection with Ordinary Bolts

As mentioned in previous section, the $F - \delta$ relationship of HSFG bolt can be considered as a three-stage curve. At the third stage, the $F - \delta$ relationship is non-linear and is similar to the bearing behaviour of single plate connection with ordinary bolts. In order to derive the shape parameter n in Eq. 6-7, numerical modelling of single plate connection with ordinary bolts was carried out.

6.3.5.1 Numerical Modelling using ABAQUS

A general purpose finite element package ABAQUS (Version 6.3) was used to model the behaviour of single plate connection with ordinary bolts (Figure 6.8). The 3D solid model incorporates the contact surfaces between contacting components and nonlinear material properties. Material properties were defined based on the material coupon test results. The sections representing the steel plate and steel beam were made up of C3D8, 8 node solid elements. The bolts were made up C3D15, 15 node wedge elements. Contact interactions were defined between:

- the underside of the bolt heads and the surface of steel plate surrounding the bolt holes;
- the bolt shanks and the inner surfaces of the bolt holes;
- the faying surface between the steel plate and beam web.

Since the shape parameter n is affected by many variables, a parametric study is therefore carried out to gain an insight into the behaviour of single plate connection. A parametric study was conducted to investigate the variations in shear span, bolt size, bolt pitch, thickness of steel plate as well as thickness of beam web. The detailed configuration of single plate connection is shown in Figure 4.1. The steel beam used was 356 x 171 UB45 and the steel column was 356 x 171 UB67. The dimension of steel plate was 130 x 300 x 10 mm. Five 20-mm ordinary bolts were used to connect the steel plate to the beam web. The bolt pitch was 50 mm which was 2.5 times of the bolt diameter. The material properties of steel components

were based on the stress-strain curves of coupon tests. The boundary condition of one end of steel plate was fixed due to the structural symmetry.

6.3.5.2 Parametric Study

Based on recommended or standard practices, the geometric variables of single plate connection were limited to the practical ranges. The parameters selected were the shear span, bolt size, bolt pitch, thickness of steel plate and thickness of beam web. For a 5-bolt single plate connection, the values of parameters selected for finite element analysis were summarised in Table 6.1.

**Table 6.1 Values of Parameters Selected for Finite Element Analysis
(5-bolt single plate connection with ordinary bolts)**

Variables	Values of variables					
Shear span	0.5 m	1.0 m	1.5 m			
Bolt size	16 mm	20 mm	24 mm			
Bolt pitch	50 mm	75 mm	100 mm			
Thickness of single plate	8 mm	10 mm	12 mm			
Thickness of beam web	4 mm	5 mm	6 mm	7 mm	8 mm	9 mm

- **Shear span**

As shown in Figure 6.9, the moment-rotation relationships of single plate connection with different shear spans of 0.5 m, 1.0 m and 1.5 m were plotted. It was observed that the differences between them are insignificant. The initial stiffness and moment capacities are almost the same. Therefore, it may be concluded the variations of shear span have little influences on the moment-rotation relationships if the shear force is not near to the connection. This is consistent with Richard's (1980) conclusion that the moment-rotation relationship of single plate connection was insensitive to the connection shear for shear span equal to or greater than the depth of bolt pattern.

- **Bolt size**

The moment-rotation relationships of single plate connection with different bolt sizes were plotted in Figure 6.10. It was observed that the bigger the bolt size, the higher the moment capacity and initial stiffness. This is primarily due to the higher bearing area from the bigger bolt size.

- **Bolt pitch**

The moment-rotation relationships of single plate connection with different bolt pitches were plotted in Figure 6.11. The bolt pitches were 50 mm, 75 mm and 100 mm respectively. Because the increasing in bolt pitch increases the length of level arm about the centroid of bolt group, therefore, resulting in a higher initial stiffness and moment capacity.

- **Thickness of steel plate**

In Figure 6.12, the moment-rotation relationships of single plate connection with different thickness of steel plate were plotted. In practice, the thickness of the steel plate is usually greater than the thickness of beam web. It was observed that there were no noticeable changes of moment capacity although the initial stiffness were slightly different, which indicated that it was the thinner beam web that actually governed the connection behaviour instead of the thicker steel plate. Therefore, it may be concluded that when the steel plate is sufficiently thick relative to the beam web, which causes most of the inelastic bearing deformations occurring in the beam web, the influence of steel plate thickness was considered insignificant and the moment-rotation relationship of single plate connection was independent of the thickness of steel plate.

- **Thickness of beam web**

As shown in Figure 6.13, the moment-rotation relationships of single plate connection with different web thickness were plotted. It was important to note that the moment-rotation relationships were dependent on the thickness of beam web. The thicker the beam web, the higher the initial stiffness and moment capacity can be achieved. This can be explained that the thicker beam web can provide larger bearing area between beam web and bolts, which increases the stiffness of the single plate connection as well as the moment capacity.

Based on the parametric study, it can be summarised that the shear span is the insignificant variable for the moment-rotation behaviour of single plate connection. If the steel plate is sufficiently thicker than the beam web, most of the inelastic bearing deformations will occur in the beam web and the steel plate thickness will contribute little to the behaviour of single plate connection. However, the bolt size, bolt pitch and beam web thickness can affect the behaviour of single plate connection significantly. Normally, the bolt size and bolt pitch are standardised in practice. Therefore, the bearing strength between bolt and beam web can be considered as the critical component, which controls the overall behaviour of single plate connection.

6.3.5.3 Shape Parameter

Since the thickness of beam web governs the moment-rotation behaviour of single plate connection, parametric study was carried out further to investigate the variation of beam web thickness on the behaviour of single plate connection with different bolt numbers as shown in Table 6.2.

Based on the analysis of numerical data, the shape parameter n is found to be the function of initial bearing stiffness $K_{i,bearing}$ and ultimate bearing capacity $F_{bearing}$. However, $K_{i,bearing}$ and $F_{bearing}$ are predicted based on empirical equations (Eq.6-8 & 6-12). To verify the accuracy of predicted $K_{i,bearing}$ and $F_{bearing}$, a finite element

model of a single plate bearing against a single bolt was developed and analysed using finite element package ABAQUS (Figure 6.14). The material properties of single plate and single bolt were the same as beam web and ordinary bolt used in the single plate connection model (Figure 6.8). As shown in Table 6.3, finite element models with various plate thickness and edge distances were analysed to verify the predicted values of $K_{i,bearing}$ and $F_{bearing}$.

Table 6.2 Values of Beam Web Thickness for Finite Element Analysis

Single plate connection	Thickness of beam web					
3-bolt	4 mm	5 mm	6 mm	7 mm	8 mm	9 mm
4-bolt	4 mm	5 mm	6 mm	7 mm	8 mm	9 mm
5-bolt	4 mm	5 mm	6 mm	7 mm	8 mm	9 mm
6-bolt	4 mm	5 mm	6 mm	7 mm	8 mm	9 mm
7-bolt	4 mm	5 mm	6 mm	7 mm	8 mm	9 mm
8-bolt	4 mm	5 mm	6 mm	7 mm	8 mm	9 mm

Table 6.3 Variations of Plate Thickness and Edge Distance

Edge distance (L_e)	Thickness of plate					
30 mm	4 mm	5 mm	6 mm	7 mm	8 mm	9 mm
40 mm	4 mm	5 mm	6 mm	7 mm	8 mm	9 mm
50 mm	4 mm	5 mm	6 mm	7 mm	8 mm	9 mm
60mm	4 mm	5 mm	6 mm	7 mm	8 mm	9 mm
70 mm	4 mm	5 mm	6 mm	7 mm	8 mm	9 mm

Suggested by Rex and Easterling (2003), the initial stiffness $K_{i,bearing}$ was measured by assuming standard hole size and an initial bearing deformation of 0.102 mm. The bearing capacity $F_{bearing}$ was assumed to be the load at 6.4 mm or at failure if failure occurred before 6.4 mm of deformation. Comparisons of $K_{i,bearing}$ and $F_{bearing}$ between finite element models and predictions are shown in Figure 6.15 & 6.16 respectively. Overall, the agreements between the finite element results and

predicted results are deemed satisfactory (average 15% differences). Based on the regression of numerical results, the shape parameter n in Eq.6-7 can be derived as:

$$n = \frac{5.73\Delta}{1 + 0.439.\Delta.t_{web}^2} \quad \text{Eq. 6-15}$$

$$\Delta = \frac{F_{bearing}}{K_{i,bearing}} \quad \text{Eq. 6-16}$$

where $K_{i,bearing}$: Initial bearing stiffness of single plate connection with ordinary bolts;
 $F_{bearing}$: Ultimate bearing capacity of single plate connection with ordinary bolts;
 t_{web} : Thickness of beam web (mm).

Based on the three-parameter power model in Eq.6-7, the moment-rotation relationship of single plate connection can be calculated. For a 5-bolt single plate connection with different beam web thickness, the model predictions are compared with the numerical results in Figure 6.13. For single plate connections with different bolt numbers, the comparisons are also presented in Figure 6.17. In general, the predicted results agree well with the numerical curves, which indicates that the proposed model may be extended to simulate the behaviour of single plate connection with different configurations.

6.3.6 Influences of Bolt Misalignment and Shear Force on the $F - \delta$ Relationship of HSFG Bolt

The $F - \delta$ relationship of HSFG bolt presented in Figure 6.6 is based on the conditions that bolts are perfectly aligned and there is no shear force acting on the single plate connection. However, both conditions are unlikely to happen in practice.

Therefore, it is necessary to evaluate the influences of bolt misalignment and shear force on the behaviour of single plate connection.

As shown in Figure 6.18, $F - \delta$ relationships of three possible bolt alignments on two plates, namely minimum slip deformation, perfect alignment and maximum slip deformation are illustrated. For the minimum slip deformation, the bolt is initially in contact with the bolt holes and the slip deformation is zero. For a perfect bolt alignment, the initial clear distance between bolt and bolt hole is the hole clearance $\delta_{clearance}$ (1.0 mm) and the total slip deformation on two plates will be $2\delta_{clearance}$. If the bolt is initially in contact with the left side of the bolt hole on top plate and the right side of the bolt hole on bottom plate, then the maximum slip deformation $4\delta_{clearance}$ will be achieved. For any other random bolt alignment, the slip deformation will fall between the lower bound zero and the upper bound $4\delta_{clearance}$. For a 5-bolt single plate connection, three bolt alignments with minimum slip deformation, perfect alignment and maximum slip deformation and their corresponding moment-rotation curves are shown in Figure 6.19 & 6.20 respectively. As shown in Figure 6.20, the connection rotation at the slip stage is increased as a result of increasing of bolt slip deformation. Compared with the test data, the prediction with minimum slip deformation has a better agreement with the experimental results than the perfect alignment, which indicates that the bolts were not perfectly aligned initially and likely in contact with the bolt holes during the installation process.

The influence of shear force acting on the single plate connection on the $F - \delta$ relationship of HSFG bolt is indicated in Figure 6.21. The slip resistance F_{slip} of each bolt can be considered as a resultant force of the horizontal force F'_{slip} and the vertical force $\frac{P}{n_b}$, where n_b is the number of bolts and the shear force P is divided equally between the bolts. As shown in the $F - \delta$ curve, the slip resistance F_{slip} and deformation before slip δ_{slip} will be reduced to F'_{slip} and δ'_{slip} respectively due

to the presence of shear force P . As shown in Figure 6.22, single plate connections with shear force of 0, 150 and 300 kN are analysed to evaluate the effect of shear force on the moment-rotation relationship. It can be noted that the slip moment is reduced as a result of increasing of shear force acting on the single plate connection.

6.4 Force-Displacement Relationship of Concrete Section

6.4.1 Concrete Model Proposed by Mander et al. (1988)

Different stress-strain relationships have been developed to describe the behaviour of confined concrete (Park et al. 1975, Mander et al. 1988, Paulay et al. 1992). In this study, the relationship proposed by Mander et al. (1988) is adopted in describing the behaviour of concrete beam section. Mander et al. proposed a unified stress-strain model for confined concrete which can be used for structural members with either circular or rectangular sections subjected to static or dynamic axial compressive loading (Figure 6.23). The concrete section may contain any general type of confinement with either spirals or circular hoops, or rectangular hoops with or without supplementary cross ties. For monotonic loading, the longitudinal compressive concrete stress was expressed as:

$$f_c = \frac{f'_{cc} x r}{r - 1 + x^r} \quad \text{Eq. 6-17}$$

where f'_{cc} is the peak longitudinal compressive stress for confined concrete which can be stated as:

$$f'_l = \frac{1}{2} K_e \rho_s f_{yh} \quad \text{Eq. 6-18}$$

$$f'_{cc} = f'_{co} \left(2.254 \sqrt{1 + \frac{7.94 f'_l}{f'_{co}}} - \frac{2 f'_l}{f'_{co}} - 1.254 \right) \quad \text{Eq. 6-19}$$

$$x = \frac{\epsilon_c}{\epsilon_{cc}} \quad \text{Eq. 6-20}$$

where f'_l is the effective lateral confining stress, K_e is the confinement effectiveness coefficient, x is the ratio of strain ε_c to the strain ε_{cc} at peak confined concrete stress f'_{cc} . ε_{cc} is determined in terms of ε_{co} as:

$$\varepsilon_{cc} = \varepsilon_{co} \left[1 + 5 \left(\frac{f'_{cc}}{f'_{co}} - 1 \right) \right] \quad \text{Eq. 6-21}$$

where f'_{co} and ε_{co} are the unconfined concrete strength and corresponding strain respectively (generally $\varepsilon_{co} = 0.002$ can be assumed), and

$$r = \frac{E_c}{E_c - E_{\text{sec}}} \quad \text{Eq. 6-22}$$

$$E_c = 5000 \sqrt{f'_{co}} \quad \text{Eq. 6-23}$$

$$E_{\text{sec}} = f'_{cc} / \varepsilon_{cc} \quad \text{Eq. 6-24}$$

The efficiency of the various possible arrangements of transverse reinforcement is taken into account by defining a confinement effectiveness coefficient K_e .

6.4.2 Modifications for Concrete Model Proposed by Mander et al.

The deterioration of concrete tensile strength ensures activation of reinforcing steel in all reinforced concrete structures. Thus, it is necessary to include the deterioration of concrete tensile strength in a concrete constitutive model. In general, the concrete constitutive model proposed by Mander et al. is reasonable for reinforced concrete. However, the deterioration of concrete tensile strength is inadequately illustrated. It is a known fact that the tensile stress-strain relationship of concrete shows a linear portion up to the tensile strength followed by a descending post peak curve until the ultimate tensile strain of concrete is reached

(Gopalaratnam et al. 1985). Mander's model simply assumes that the tensile stress drops to zero once the maximum tensile stress is exceeded, which ignores the contributions of the post peak concrete.

Some empirical formulations for the stress-strain relationship of concrete after tensile strength are available (Gopalaratnam et al. 1985, Rots et al. 1984). In this study, a bilinear relationship suggested by Rots et al. was adopted to describe the concrete tensile behaviour (Figure 6.24). The tensile stress will drop to $0.33 f_t'$ at tensile strain ε_{t1} after the maximum tensile strain ε_t is exceeded, where $\varepsilon_{t1} = 5.5\varepsilon_t$. After that, the tensile stress will decrease linearly to the ultimate tensile strain ε_{tu} which equals to $25\varepsilon_t$. This stress-strain relationship of concrete under tension will be incorporated into the concrete model proposed by Mander et al.

6.4.3 Force-Displacement Relationship of Concrete

As shown in Figure 6.2, the concrete section consists of concrete beam section and topping concrete. The effective width of topping concrete has been specified in different design codes. In BS5950 (1990), the width is taken as 1/8 or 0.2 times of the average beam spans. In some publications, it is assumed as 2.5 times of the width of the column flange (Kato and Tagawa, 1984). In this study, the effective width of topping concrete is taken as 1000 mm which is equivalent to 1/8 of the average beam spans. The effect of slab width is not included in the experimental programme.

The force-displacement relationship of an idealized concrete layer, shown in Figure 6.2, is derived as follows. Based on the component-based model shown in Figure 6.1 and the definition of connection zone in Section 6.2, the distance between the column centre-line AB and end section CD is half of the depth of column section. Therefore, for a displacement of i th concrete element Δ_i , the concrete strain ε_{ci} can be expressed as:

$$\varepsilon_{ci} = \frac{\Delta_i}{L_c} \quad \text{Eq. 6-25}$$

where Δ_i : Displacement of the i th concrete element;

L_c : Length of concrete considered for calculating concrete strain.

($L_c = h_c/2$, h_c is the depth of column section)

Based on the modified concrete model, the respective concrete stress f_{ci} can be calculated and the force of the i th concrete layer F_{ci} can be derived as:

$$F_{ci} = f_{ci} \cdot A_i \quad \text{Eq. 6-26}$$

where A_i : Cross section area of i th concrete layer.

6.5 Force-Displacement Relationship of Steel Reinforcement

6.5.1 Stress-strain Relationship of Steel Reinforcement

In this study, a monotonic model proposed by Kent and Park (1973) is adopted to represent the stress-strain relationship for steel reinforcement as shown in Figure 6.25. The model can be expressed in piecewise as follows:

(a) Elastic range

$$f_s = E_s \varepsilon_s, \text{ for } 0 \leq \varepsilon_s \leq \varepsilon_y; \quad \text{Eq. 6-27}$$

(b) Yield plateau

$$f_s = f_y, \text{ for } \varepsilon_y < \varepsilon_s \leq \varepsilon_{sh}; \quad \text{Eq. 6-28}$$

(c) Strain-hardening range

The strain hardening curve commences when the strain increases above the strain of yield plateau (ε_{sh}) and extends to the ultimate strain (ε_{su}). The equation proposed by Kent and Park (1973) for modelling the strain-hardening region is given as:

$$f_s = f_y \left[\frac{W_h (\varepsilon_s - \varepsilon_{sh}) + 2}{60(\varepsilon_s - \varepsilon_{sh}) + 2} + \frac{(\varepsilon_s - \varepsilon_{sh})}{(\varepsilon_{su} - \varepsilon_{sh})} \left(\frac{f_{su}}{f_y} - W_a \right) \right] \quad \text{Eq. 6-29}$$

where

$$W_h = \frac{\frac{f_{su}}{f_y} (30b + 1)^2 - 60b - 1}{15b^2} \quad \text{Eq. 6-30}$$

$$W_a = \frac{W_h b + 2}{60b + 2} \quad \text{Eq. 6-31}$$

$$b = \varepsilon_{su} - \varepsilon_{sh} \quad \text{Eq. 6-32}$$

As shown in Figure 6.26, the predicted stress-strain relationship agrees well with the T10 rebar tensile test result.

6.5.2 Force-Displacement Relationship of Steel Reinforcement

Based on the component-based model shown in Figure 6.1, the axial strain of steel reinforcement ε_r can be expressed as follows:

$$\varepsilon_r = \frac{\Delta_r}{L_r} \quad \text{Eq. 6-33}$$

where Δ_r : Displacement of reinforcement;

L_r : Length of reinforcement considered for calculating steel strain.

During the loading procedure, the length of L_r varies at different loading stages which results in a varying stiffness for the reinforcement. After the cracking of concrete, the propagation of cracks inside the concrete coupled with the gradual yielding of steel reinforcement reduces the overall stiffness of the hybrid connection. It was confirmed by Anderson and Najafi (1994) that if the length of rebar considered for calculating the rebar stiffness is equal to half the depth of the column, the resulting model overestimates the stiffness of the connection. They concluded that this could be corrected by increasing the length of rebar considered for the elongation. In this research, based on the strain measurements of longitudinal rebars under hogging and sagging moment, it was found that rebars yielded first at the region between the column centre-line and the position of first shear link, which

indicates that the tensile force in the rebars is the highest in this region. Therefore, it may be appropriate to take the value of L_r as:

$$L_r = h_c / 2 + a \quad \text{Eq. 6-34}$$

where h_c : Depth of the column section.

a : Distance of the first shear link from the column face ($a = 225$ mm).

Based on the monotonic stress-strain model, the respective steel stress of top rebar, bottom rebar and mesh reinforcement can be calculated and the corresponding forces $F_{r,top}$, $F_{r,bot}$ and F_{sm} can be obtained.

6.6 Analytical Procedure of Hybrid Connections under Monotonic Loading

As illustrated in the flow chart in Figure 6.3, the numerical procedure for the moment-rotation characteristics of hybrid connection under both monotonic hogging and sagging moment can be achieved. Taking the rotation of the connection θ as the controlling variable, the analytical scheme can be stated as follows:

- 1) The concrete section is subdivided into finite number of layers.
- 2) The force-displacement relationship of each component is calculated according to the geometrical size and material property of the component. Three components are included: single plate connection, concrete beam section and longitudinal reinforcements.
- 3) At the initial stage, the moment and rotation are zero. The neutral axis position y_0 can be calculated based on section properties of the hybrid connection.
- 4) Giving the rotation an increment $\Delta\theta_i$, the variation of displacement and force of each component, can be determined. Based on force equilibrium, the neutral axis position can be calculated.
- 5) If the force equilibrium requirements are satisfied, the results are recorded. A new rotation increment $\Delta\theta_i$ is added. Otherwise, the position of neutral axis is adjusted until the force equilibrium requirements are satisfied.
- 6) The calculation is continued until the failure of the connection. The failure of the connection is defined as the deformation limitation of the first component being exceeded.

6.7 Component Behaviour of Hybrid Connection under Monotonic Hogging & Sagging Moment

6.7.1 Comparison between Test Results and Model Predictions

The comparison of moment-rotation curves between test results and predicted values of hybrid connections under monotonic hogging and sagging moment are presented in Figure 6.27 & 6.28 respectively. Satisfactory agreements (average 15% differences) between test results and model predictions are observed. The analytical and experimental results of rebar tensile strains are also presented in Figure 6.29 & 6.30. The model yields fairly good agreements (within 10 % differences) with the test data for specimens under both monotonic hogging (HC1, HCS1 & HCS3) and sagging moment (HC2, HCS2 & HCS4).

The calculated depths of neutral axis versus connection moment for specimens under hogging and sagging moment are shown in Figure 6.31 & 6.32 respectively. Acceptable correlations (about 20% differences) can be observed between the model predictions and actual test results. Under hogging moment, the predicted neutral axis changes slightly from the initial position prior the cracking of concrete and reduces significantly after the cracking, which reflects the actual behaviour of hybrid connection under hogging moment. Under sagging moment, the predicted neutral axis reduces gradually after the cracking of concrete, which is consistent with the variation of neutral axis for specimen under sagging moment.

6.7.2 Contribution of Key Components to the Connection under Monotonic Hogging Moment

In general, the moment-rotation relationship is not sufficient to reveal the behaviour of key components in the hybrid connection. Using component-based model, the contribution of each component to the behaviour of entire hybrid connection can be identified, which enables a better understanding on the actual behaviour of hybrid

connection. To understand how each component behaves, it is essential to study the contribution of key components to the connection behaviour.

Under hogging moment, the moment contribution of key components, namely concrete, top rebars, mesh reinforcements and single plate connection can be singled out as shown in Figure 6.33. The connection moment is the summation of the moment contribution from each individual component. Prior the cracking of concrete, the stiffness of the hybrid connection is very high and the slope of the moment-rotation curve is almost linear. The tensile stress in the top rebars and mesh reinforcements are very small and the single plate connection is initially under compression at this stage. After cracking, concrete tensile strength starts to decrease following the post peak tensile stress-strain relationship defined in the concrete model. The tensile stress in the top rebars and mesh reinforcements increase significantly due to the cracking in the concrete and with the decrease of neutral axis depth. The single plate connection starts to contribute to the connection moment and its moment-rotation curve is non-linear at this stage. When the concrete layers defined in the component-based model have reached their ultimate tensile strain ε_{tu} (Figure 6.24), the moment contribution due to concrete becomes zero. Eventually, the top rebars and mesh reinforcements begin to yield and the turning point in the moment-rotation curve indicates the yielding of steel reinforcements. Although the single plate connection continues to increase the connection moment after the yielding of steel reinforcements, the moment enhancement is unnoticeable.

The contribution of each HSFG bolt to the non-linear behaviour of single plate connection is also illustrated in Figure 6.34. The moment-rotation curve of single plate connection is the summation of contribution from each individual HSFG bolt. Before the cracking of concrete, the location of neutral axis is above the position of the first bolt and all HSFG bolts are under compression. After cracking, all HSFG bolts gradually turn into tension from the first to the fifth due to the reduction of neutral axis depth. Following the same sequence, the HSFG bolt begins to slip when the slip resistance is exceeded and bears against the bolt hole when the hole

clearance is reached. It can be observed that most HSFG bolts are still within the slip stage when the maximum moment capacity is attained, which results in little moment enhancement after the yielding of top steel reinforcements.

Based on the moment contribution of the key components for the hybrid connection under hogging moment, single plate connection only contributes a small portion to the connection moment, however, the top tensile reinforcements actually govern the connection behaviour. In fact, the tested hybrid connection was highly under-reinforced by noting that the neutral axis depth was about 25% of the effective depth when connection moment exceeded 200 kN.m (Figure 6.31). Therefore, increasing of the top rebars will absolutely enhance the moment capacity as shown in Figure 6.27.

6.7.3 Contribution of Key Components to the Connection under Monotonic Sagging Moment

The contribution of key components to the entire hybrid connection under monotonic sagging moment is shown in Figure 6.35. Unlike the specimen under hogging moment shown in Figure 6.33, the cracking of concrete can not be identified easily from the moment-rotation curve. This is due to the presence of single plate connection in the tension zone which increases the stiffness of connection and the deterioration of concrete tensile strength will not affect the connection stiffness significantly. For the same reason, the depth of neutral axis reduces gradually after the cracking of concrete (Figure 6.32) instead of dropping abruptly as that under hogging moment (Figure 6.31). The turning point of the connection moment-rotation curve is actually due to the slip moment of single plate connection being reached instead of the yielding of steel reinforcements. As shown in Figure 6.35, there is a coincidence of turning points in the bottom rebars and single plate connection at around 2 mrad. It also can be observed that the bottom rebars yield at moment of 450 kN.m, which agrees with the earlier strain measurements in Figure 6.30. The single plate connection keeps on increasing the

connection moment effectively after the bottom rebars exceed the yielding strength until the failure of specimen.

The contribution of each HSFG bolt to the non-linear behaviour of single plate connection is shown in Figure 6.36. Under sagging moment, all HSFG bolts are under tension since the beginning of loading. Starting from the fifth to the first, the HSFG bolt starts to slip as the slip resistance is exceeded and subsequently bears against the bolt hole when the hole clearance is reached. It can be observed that all HSFG bolts are in the bearing stage when the maximum moment capacity is attained, which results in significant moment enhancement after the yielding of bottom steel reinforcements.

For a hybrid connection under sagging moment, the single plate connection continues to enhance the connection moment effectively even when the bottom tensile reinforcements reach their yielding strength. The connection behaviour is actually governed by the behaviour of single plate connection instead of bottom tensile reinforcements. Therefore, the increase of steel reinforcement contributes little to the connection behaviour after the yield of bottom rebars as shown in Figure 6.28.

6.8 Parametric Study

The hybrid connection consists of different structural components, including the longitudinal reinforcements, single plate connection and topping concrete. Changes in the details of some components, such as area of rebars or configuration of single plate connection may lead to a significant effect on the performance of the connection. In this section, an extensive parametric study is carried out using the proposed component-based model to check the influences on the connection behaviour for the variation of different key parameters. The parameters selected herein are the area of top rebars, the thickness of beam web and the effective width of topping concrete which, defacto, increases much reinforcement. The influence of

axial force on the connection behaviour is also evaluated. The configuration of the specimen used for parametric study is based on HCS1 and only one parameter is changed at a time so as to evaluate its effect clearly.

6.8.1 Area of Top Rebars

Based on the contribution of key components for connection under hogging moment (Figure 6.33), it is concluded that the top tensile reinforcements control the connection behaviour. To further understand the influence of top tensile reinforcements, hybrid connections with top rebar area of 402.1 (2T16), 628.3 (2T20) and 981.7 mm² (2T25) are analysed under monotonic hogging moment. The analytical results are presented in Figure 6.37.

The moment contribution from other key components, such as concrete, mesh reinforcements and single plate connection are the same in these three cases. By increasing the area of top rebars, the moment contribution of top rebars increases correspondently and therefore enhances the overall moment-rotation behaviour of the hybrid connection. It is assumed the balanced failure occurs when the steel reaches the yield strength and the concrete reaches the extreme fibre compression strain of 0.0035, simultaneously. The top rebar area required for the balanced failure of hybrid connection is 8928 mm² (11T32). If the steel content is greater than this value, the section becomes over-reinforced and the concrete may reach its maximum capacity before the steel yields, which may lead to a less ductile compression failure.

6.8.2 Beam Web Thickness in Single Plate Connection

As discussed in the previous sections, the thickness of beam web governs the behaviour of single plate connection. To investigate the influence of beam web thickness on the connection behaviour, three cases with web thickness of 4, 6 and 9 mm are analysed under both monotonic hogging and sagging moment. The analytical results are presented in Figure 6.38 & 6.39 respectively.

As indicated in Figure 6.6, the variation of web thickness only changes the bearing behaviour of HSFG bolt (stage III of $F - \delta$ relationship). Under hogging moment, the steel beam web is on the neutral axis, most HSFG bolts are still within the slip stage and the change of web thickness contributes very little to the behaviour of single plate connection (Figure 6.34). Therefore, the variation of web thickness does not affect the moment contribution of single plate connection and has no effect on the overall connection behaviour for connection under hogging moment (Figure 6.38). However, under sagging moment, all HSFG bolts are in the bearing stage and the change of web thickness affect the contribution of single plate connection significantly (Figure 6.36). As shown in Figure 6.39, the sagging moment-rotation curve changes according to the change of contribution from single plate connection, which is caused by the variation of beam web thickness.

6.8.3 Effective Width of Topping Concrete

Since the definitions of effective width of topping concrete are not consistent in different design codes. It is necessary to investigate the influence of effective width on the behaviour of hybrid connection. Hybrid connections with three different effective widths of 0.6, 0.8 and 1.0 m are analysed under both hogging and sagging moment and the analytical results are presented in Figure 6.40 & 6.41 respectively.

The increase of effective width of topping concrete increases the area of concrete beam section, which therefore enhances the capacity of cracking moment. Under hogging moment, the greater the effective width of topping concrete, the higher the value of cracking moment (Figure 6.40). The moment capacity increases with the increase of effective width due to more mesh reinforcements in the topping concrete. Under sagging moment, the topping concrete is under compression and the influences of topping concrete width on the cracking moment capacity and overall connection behaviour are unnoticeable (Figure 6.41).

6.8.4 Influence of Axial Force on the Hybrid Connection

In practice, axial forces always exist in the structural members due to the horizontal loads. Hybrid connections with 0, 100 kN and 200 kN compressive forces are analysed under both hogging and sagging moment to study the effect of axial force on the connection behaviour. The analytical results are presented in Figure 6.42 ~ 6.45.

The presence of axial force affects the initial behaviour of hybrid connection. Under hogging moment, the higher the axial force, the lower the initial position of neutral axis (Figure 6.42). This is because the initial depth of neutral axis is adjusted to a lower position to balance the higher axial force based on the force equilibrium condition. As shown in Figure 6.43, the value of cracking moment is also increased slightly as a result of increasing of axial force. However, there is no noticeable change on the overall moment-rotation behaviour as well as the moment contributions from top rebars and single plate connection. Under sagging moment, the depth of neutral axis is also adjusted based on the force equilibrium condition similar to that under hogging moment and the influence of axial forces on the connection behaviour is unnoticeable (Figure 6.44 & 6.45).

If the axial force is tensile force, the depth of neutral axis is adjusted based on the force equilibrium condition similar to that under axial compressive force. Under hogging moment, the higher the tensile force, the higher the initial position of neutral axis. Under sagging moment, the higher the tensile force, the lower the initial position of neutral axis. However, only the initial behaviour of hybrid connection will be affected and the overall connection behaviour under hogging and sagging moment are basically unaffected.

6.9 Summary

In this chapter, the following conclusions can be made:

- The ABAQUS finite element package is used to model the behaviour of single plate connection with ordinary bolts. Based on the parametric study, it can be summarised that the shear span and thickness of steel plate are insignificant variables for the moment-rotation relationships of single plate connection, however, the bolt size, bolt pitch and beam web thickness can affect the behaviour of single plate connection significantly.
- A three-parameter power model is proposed to describe the non-linear force-displacement ($F - \delta$) relationship of single plate connection with ordinary bolts. The shape parameter in the model can be obtained based on the regression of numerical data from the finite element model.
- A mechanical model is proposed to analyse the behaviour of single plate connection with HSFG bolts under monotonic loading. The proposed model can be extended to simulate the behaviour of single plate connection with different configurations. This model can be incorporated into the component-based mechanical model which is proposed for the analysis of hybrid connection.
- A component-based mechanical model is proposed to predict the moment-rotation relationship of hybrid connection under monotonic hogging and sagging moment. The model takes into account the nonlinear behaviour of each component in the hybrid connection. Mander's concrete model has been modified to include the deterioration of concrete tensile strength.
- Based on the moment contribution of the key components under hogging moment, single plate connection only contributes a small portion to the connection moment, however, the top tensile reinforcements actually govern the connection behaviour. Therefore, increasing of the top rebars will effectively enhance the moment capacity.

Based on the moment contribution of the key components under sagging moment, single plate connection continues to enhance the connection moment even when the bottom tensile reinforcements reach the yielding strength. The connection behaviour is actually governed by the behaviour of single plate connection instead of bottom tensile reinforcements. Therefore, the increase of steel reinforcement contributes little to the overall connection behaviour after the yield of bottom rebars.

Figure 6.1 Proposed component-based model under monotonic loading

$K_{r,top}$: Axial stiffness of top reinforcements;

K_s : Axial stiffness of HSFG bolt in single plate connection;

K_c : Axial stiffness of concrete;

$K_{r,bot}$: Axial stiffness of bottom reinforcements;

$F_{r,top}$: Force in the top reinforcements;

F_s : Force between HSFG bolt and connected part;

$F_{r,bot}$: Force in the bottom reinforcements;

F_{ci} : Force in the i th concrete layer;

Δ_i : Displacement of the i th element;

θ : Rotation of hybrid connection;

y_i : Depth of the i th element;

$\bar{y}$: Depth of neutral axis;

h_c : Depth of column section.

Figure 6.2 Subdivision of the concrete beam section into monitoring areas

Figure 6.4 Proposed mechanical model for single plate connection
(each spring represents the axial response of one HSFG bolt)

Figure 6.3 Flow chart to determine the moment-rotation relationship of hybrid connection

Figure 6.5 Bearing, bending and shear stiffness model
(Rex and Easterling, 2003)

Figure 6.6 $F - \delta$ relationship between one HSFG bolt and connected parts

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Figure 6.7 Comparison between test results and model prediction

Figure 6.8 Numerical modelling of single plate connection with ordinary bolts

Figure 6.9 Moment-rotation relationships with different shear spans
(5-bolt, bolt dia=20mm, bolt pitch=50mm, plate thk=10mm, beam web thk=6.9mm)

Figure 6.10 Moment-rotation relationships with different bolt diameters
(5-bolt, bolt pitch=50mm, plate thk=10mm, beam web thk=6.9mm)

Figure 6.11 Moment-rotation relationship with different bolt pitches
(5-bolt, bolt dia=20mm, plate thk=10mm, beam web thk=6.9mm)

Figure 6.12 Moment-rotation relationships with different plate thickness
(5-bolt, bolt dia=20mm, bolt pitch=50mm, beam web thk=6.9mm)

Figure 6.13 Comparison between numerical results and model predictions
(5-bolt, bolt dia=20mm, bolt pitch=50mm, plate thk=10mm)

Figure 6.14 Finite element model of a single plate bearing against a single bolt

Figure 6.15 Comparison of initial bearing stiffness $K_{i,bearing}$ between finite element models and predictions

Figure 6.16 Comparison of ultimate bearing capacity $F_{bearing}$ between finite element models and predictions

Figure 6.17 Comparison between numerical results and model predictions for single plate connection with different bolt numbers (bolt dia=20mm, bolt pitch=50mm, plate thk=10mm, beam web thk=7mm)

Figure 6.18 $F - \delta$ relationships of HSFG bolt with different bolt alignments

Figure 6.19 Different bolt alignments on a 5-bolt single plate connection

Figure 6.20 Moment-rotation relationships for a 5-bolt single plate connection with different bolt alignments

Figure 6.21 Influence of shear force on the $F - \delta$ relationship of HSFG bolt

Figure 6.22 Moment-rotation relationships for a 5-bolt single plate connection with different shear forces

Figure 6.23 Concrete model proposed by Mander et al. (1988)

Figure 6.24 Stress-strain relationship of concrete under tension
(Rots et al., 1984)

Figure 6.25 Stress-strain relationship for steel reinforcement under monotonic loading (Kent and Park, 1973)

Figure 6.26 Prediction of stress-strain relationship for steel reinforcement

Figure 6.27 Comparison of moment-rotation relationships between test results and model predictions under monotonic hogging moment

Figure 6.28 Comparison of moment-rotation relationships between test results and model predictions under monotonic sagging moment

Figure 6.29 Comparison of top rebar strains between test results and model predictions under monotonic hogging moment (HC1, HCS1 & HCS3)

Figure 6.30 Comparison of bottom rebar strains between test results and model predictions under monotonic sagging moment (HC2, HCS2 & HCS4)

Figure 6.31 Comparison of neutral axis between test results and model predictions under monotonic hogging moment (HC1, HCS1 & HCS3)

Figure 6.32 Comparison of neutral axis between test results and model predictions under monotonic sagging moment (HC2, HCS2 & HCS4)

Figure 6.33 Contribution of key components in the hybrid connection under monotonic hogging moment (HCS1)

Figure 6.34 Contribution of each HSFG bolt to the entire single plate connection under monotonic hogging moment (HCS1)

Figure 6.35 Contribution of key components in the hybrid connection under monotonic sagging moment (HCS2)

Figure 6.36 Contribution of each HSFG bolt to the entire single plate connection under monotonic sagging moment (HCS2)

Figure 6.37 Hogging moment-rotation curves for variation of top rebar area

Figure 6.38 Hogging moment-rotation curves for variation of beam web thickness

Figure 6.39 Sagging moment-rotation curves for variation of beam web thickness

Figure 6.40 Hogging moment-rotation curves for variation of effective width of topping concrete

Figure 6.41 Sagging moment-rotation curves for variation of effective width of topping concrete

Figure 6.42 Position of neutral axis under hogging moment for variation of axial force

Figure 6.43 Hogging moment-rotation curves for variation of axial force

Figure 6.44 Position of neutral axis under sagging moment for variation of axial force

Figure 6.45 Sagging moment-rotation curves for variation of axial force

Chapter 7: Modelling of Moment-Rotation Relationship under Cyclic Loading

7.1 Introduction

Existing models for composite connections under cyclic loading are few and mainly based on experimental observations of the global behaviour rather than analytical representation of the cyclic response of individual components. In Chapter 6, a component-based mechanical model has been proposed to predict the moment-rotation relationship of hybrid connection under monotonic loading. In this chapter, the component-based model is extended to present the behaviour of hybrid connection under cyclic loading. The properties of each component in the model under cyclic loading are illustrated as below.

7.2 Modelling of Single Plate Connection under Cyclic Loading

From the test result of specimen (BS3) under cyclic loading (Section 5.5.1), it was found that the monotonic moment-rotation curve could be used as the envelope curve for the behaviour of single plate connection under cyclic loading. The cyclic moment-rotation relationship reflects a combination of cyclic slip and bearing behaviour. The elongation of slip deformation is due to the increasing bearing deformation and the slip load deterioration may be attributed to the changing frictional characteristics of faying surfaces between the connected parts. The loss of slip resistance is the principle factor to cause hysteretic deterioration or pinching for the cyclic moment-rotation curve. In this study, the decay model proposed by Swanson (1999) was adopted to describe the decay of slip load for HSFG bolt under cyclic loading. As shown in Figure 7.1, the post slip behaviour is non-linear and can be given as a function of the slip deformation as follows:

$$F_{decay} = F_{slip} + \frac{(\delta - \delta_{slip})K_{fp}}{\left[1 + \left(\frac{(\delta - \delta_{slip})(-K_{fp})}{(F_{slip} - F_{min})}\right)^{nshape}\right]^{\left(\frac{1}{nshape}\right)}} \quad \text{Eq. 7-1}$$

$$F_{min} = 0.25F_{slip} \quad \text{Eq. 7-2}$$

$$K_{fp} = \frac{F_{slip}}{\delta_{slip} - \delta_{fu}} \quad \text{Eq. 7-3}$$

where F_{decay} : Decayed slip load;
 F_{slip} : Slip resistance;
 F_{min} : Minimum slip load;
 K_{fp} : Post slip stiffness;
 δ : Slip deformation;
 δ_{slip} : Deformation before slip;
 δ_{fu} : Ultimate slip deformation (Table 7.1);
 $nshape$: Shape parameter defining the curvature of the decay function.

Table 7.1 Ultimate Slip Deformation Definition

$t_{p1} + t_{p2}$ (mm)	δ_{fu} (mm)
$(t_{p1} + t_{p2}) < 12.7$	10.16
$12.7 \leq (t_{p1} + t_{p2}) < 38.1$	$10.16 - 0.3(t_{p1} + t_{p2} - 12.7)$
$38.1 \leq (t_{p1} + t_{p2})$	2.54

The typical value of shape parameter $nshape$ is 2.0 and the value of deformation before slip δ_{slip} was determined as 0.0076 inch (0.193 mm) (Swanson, 1999). The

ultimate slip deformation δ_{fu} was calculated as a function of the thickness of joined plates as shown in Table 7.1 (Karsu, B., 1995).

As shown in Figure 7.2, the stiffness K_{slip} (Eq. 6-6) and K_{br} (Eq. 6-9) are used to characterise the slip stiffness and bearing stiffness respectively. The slip stiffness K_{slip} is used to represent the stiffness when a HSFG bolt is not in contact with the bolt hole and the bearing stiffness K_{br} is used to represent the stiffness when a HSFG bolt and bolt hole are in contact. The rules of cyclic $F - \delta$ relationship between HSFG bolt and connected parts are given as follows:

- When loading to a new load level, the cyclic response follows the virgin monotonic curve;
- When reloading to a target load level, the response maintains a bearing stiffness K_{br} when HSFG bolt is in contact with the bolt hole and a slip stiffness K_{slip} when HSFG bolt and bolt hole are not in contact;
- The slip load decays according to the slip load decay model which is based on the bolt slip deformation.

As illustrated in Figure 7.2, when HSFG bolt is loaded, the initial stiffness K_{slip} is followed (branch AB). At this stage, the $F - \delta$ curve is linear up to the slip resistance F_{slip} . After the slip resistance F_{slip} is reached, slippage occurs without the increase of load capacity (branch BC). When HSFG bolt bears against the connected parts which results in a non-linear curve, the virgin monotonic $F - \delta$ curve is followed until unloading occurs at point D (branch CD). During the unloading, the stiffness is K_{br} when HSFG bolt is in contact with the bolt hole (branch DE) and the stiffness changes to K_{slip} when HSFG bolt and bolt hole are not in contact (branch EF). With the increase of load, the slippage occurs at a decayed slip load F_{decay} (branch FG), which is calculated based on bolt slip deformation (Eq. 7-1). Upon further loading, the same principle is followed (branch GHIJK).

Based on the mechanical model of single plate connection in Figure 6.4 and the rules of cyclic $F - \delta$ relationship in Figure 7.2, the cyclic moment-rotation relationship can be calculated. The predicted result is found to have a satisfactory agreement with the test data (Figure 7.3).

7.3 Constitutive Relationship of Concrete under Cyclic Loading

By carrying out a literature review of concrete cyclic models, it was suggested by Madas (1993) that the cyclic model proposed by Mander et al (1988) appears to be the most complete. In Mander's model, the reloading and unloading curves are defined in a manner that they can be used together with any monotonic stress-strain relationship for confined concrete. The limited capacity of concrete to carry tensile stresses is also taken into account. The cyclic model for concrete proposed by Mander et al. (1988) is adopted in this research to represent the behaviour of confined concrete under cyclic loading.

7.3.1 Cyclic Model Proposed by Mander et al. (1988)

The monotonic loading stress-strain curve is assumed to be the skeleton curve for the cyclic stress-strain response. For the definition of the unloading curves, a plastic strain ε_{pl} needs to be determined (Figure 7.4). This strain is based on the coordinates at the reversal point $(\varepsilon_{un}, f_{un})$, when unloading takes place from the envelope. Strain ε_{pl} lies on the unloading secant slope which in turn is dependent on the strain ε_a at the intersection of the initial tangent and the plastic unloading secant slopes. The strain ε_a is given by:

$$\varepsilon_a = a\sqrt{\varepsilon_{un}\varepsilon_{cc}} \quad \text{Eq. 7-4}$$

where ε_{cc} is the critical strain of the confined concrete and a is the greater of:

$$a = \frac{\varepsilon_{cc}}{\varepsilon_{cc} + \varepsilon_{un}} \quad \text{or} \quad a = \frac{0.09\varepsilon_{un}}{\varepsilon_{cc}} \quad \text{Eq. 7-5}$$

The plastic strain on the secant line between ε_a and ε_{un} is given by:

$$\varepsilon_{pl} = \varepsilon_{un} - \frac{(\varepsilon_{un} + \varepsilon_a)f_{un}}{(f_{un} + E_c \varepsilon_a)} \quad \text{Eq. 7-6}$$

The unloading curve is given by:

$$f_c = f_{un} - \frac{f_{un} x r}{r - 1 + x^r} \quad \text{Eq. 7-7}$$

in which:

$$r = \frac{E_u}{E_u - E_{sec}}, \quad E_{sec} = \frac{f_{un}}{\varepsilon_{un} - \varepsilon_{pl}}, \quad x = \frac{\varepsilon_c - \varepsilon_{un}}{\varepsilon_{pl} - \varepsilon_{un}}$$

where ε_c is the applied concrete strain and E_u is the initial modulus of elasticity at the onset of unloading, given by:

$$E_u = bcE_c \quad \text{Eq. 7-8}$$

in which:

$$b = \frac{f_{un}}{f_{co}} \geq 1 \quad \text{and} \quad c = \left(\frac{\varepsilon_{cc}}{\varepsilon_{un}} \right)^{0.5} \leq 1$$

In the case of tensile unloading, deterioration in the tensile strength due to previous compressive strain histories is assumed (Figure 7.5). When unloading from the compressive branch, the tension strength f_t is given by:

$$f_t = f_t' \left(1 - \frac{\varepsilon_{pl}}{\varepsilon_{cc}} \right) \quad \text{Eq. 7-9}$$

If $\varepsilon_{pl} < \varepsilon_{cc}$, then $f_t = 0$. Thus the stress-strain relation becomes:

$$f_t = E_t (\varepsilon_c - \varepsilon_{pl}) \quad \text{Eq. 7-10}$$

Where:

$$E_t = \frac{f_t}{\varepsilon_t} \quad \text{and} \quad \varepsilon_t = \frac{f_t'}{E_c}$$

When the tensile strain at the tensile strength is exceeded, cracks open and the tensile strength is assumed to be zero for all subsequent increments of strain.

Figure 7.6 shows the stress-strain curves including unloading and reloading branches. The coordinates of the point of reloading (ε_{ro}, f_{ro}) may be from either the unloading curve, or from the cracked state in which $\varepsilon_{ro} = (\varepsilon_{pl} - \varepsilon_t)$ and $f_{ro} = 0$. The reloading stress-strain curve is assumed to be a straight line between ε_{ro} and ε_{un} to a revised stress magnitude to account for cyclic degradation. The new stress point f_{new} is assumed to be given by:

$$f_{new} = 0.92f_{un} + 0.08f_{ro} \quad \text{Eq. 7-11}$$

A parabolic transition curve is used between the linear relation and the monotonic stress-strain curve at the return coordinate point (ε_{re}, f_{re}).

$$f_c = f_{ro} - E_r (\varepsilon_c - \varepsilon_{ro}) \quad \text{Eq. 7-12}$$

in which:

$$E_r = \frac{f_{ro} - f_{new}}{\varepsilon_{ro} - \varepsilon_{un}}$$

The return strain ε_{re} is assumed to be given by:

$$\varepsilon_{re} = \varepsilon_{un} + \frac{(f_{un} - f_{new})}{E_r \left(2 + \frac{f_{cc}'}{f_{co}'} \right)} \quad \text{Eq. 7-13}$$

where f_{co} is the uniaxial unconfined concrete strength and f_{cc}' is the compressive strength of the confined concrete in the corresponding model for monotonic loading.

The above described unloading and reloading stress-strain curves can be applied to any envelope curve to account for cyclic loading conditions.

7.3.2 Modifications of Cyclic Model Proposed by Mander et al. (1988)

In order to idealise the concrete model for cyclic loading proposed by Mander et al. (1988), the modified model is shown in Figure 7.7. The unloading from the compressive strain ε_{un} is associated with the plastic strain ε_{pl} which can be obtained from Mander's model. A linear relationship is assumed to describe the unloading branch up to cracking of the concrete due to tension (branch ABC). The tensile strength of the concrete is taken as 10% of the compressive strength, $f_t' = 0.1f_{cc}'$. After the peak tensile strength is exceeded, the deterioration of concrete tensile strength is included. The tensile stress-strain relation follows a bilinear curve shown in Figure 6.24 until ultimate tensile strain ε_{tu} is reached (point D). Reloading after cracking takes place only when ε_{pl} (point B) is reached and subsequently branch BA is followed. The tensile strength is assumed to be zero when the plastic strain magnitude exceeds the magnitude of the strain at the compressive strength ε_{cc} (branch EF).

When the concrete is under repeated loading, if the concrete has not cracked, it is capable of carrying tensile stress to point C, but if the concrete has previously cracked, or if cracks form during this stage, the tensile strain increases but no tensile stress will be developed.

7.3.3 Damage Index to Characterize the Loss of Capacity of Hybrid Connection under Cyclic Loading

In the component-based mechanical model, the concrete beam section is subdivided into finite number of layers. The tensile strain and compressive strain within one layer is assumed to be uniformly distributed. Under monotonic loading, this assumption is reasonable since the stress on each concrete layer is unidirectional and the micro cracks are uniformly distributed at the macro level before the peak load is reached. However, under cyclic loading, the compressive stiffness of cracked concrete is lower than that of crack-free concrete due to the concrete damage caused by the cracks. The damage of concrete at the beginning of a cyclic test is low. The stress-strain relationship of concrete may still follow the monotonic envelope. However, with the increase of cyclic loads, the concrete damage increases considerably and the damage of concrete can be reflected by the progressive deterioration of concrete stiffness and decay of strength. Therefore, the assumption that the monotonic loading stress-strain curve of concrete can be used as the skeleton curve for the cyclic loading is no longer valid.

Experimental results indicate that the loss of capacity of hybrid connection under cyclic loading was mainly due to the damage of concrete. To characterize the damage process of concrete members under cyclic loadings, Powell and Allahabadi (1988) proposed a deformation-only based damage index in terms of displacement of ductility, namely,

$$DI = \frac{u_{\max} - u_y}{u_0 - u_y} \quad \text{Eq. 7-14}$$

where $u_{\max}$: Maximum deformation of the component under cyclic loading;

u_y : Yield deformation of the component under monotonic loading;

u_0 : Maximum deformation of the component under monotonic loading.

However, the damage characterization by Eq. 7-14 is inadequate since it is implicitly assumed that the structural damage occurs only due to maximum deformation and is therefore independent of the energy dissipated by the structure.

Numerous Damage Indices considering both the maximum displacement response and total energy dissipation have been proposed over the last two decades. The best known and most widely used Damage Index (DI) is that proposed by Park and Ang (1985). The Park and Ang damage index is calculated as a linear combination of maximum displacement response and total hysteretic energy dissipation, namely,

$$DI = \frac{\delta_m}{\delta_u} + \beta \frac{\int dE}{F_y \delta_u} \quad \text{Eq. 7-15}$$

where DI : Damage index, $DI \geq 1$ indicates total damage or collapse;
 δ_m : Maximum deformation of the component under cyclic loading;
 δ_u : Maximum deformation of the component under monotonic loading;
 F_y : Calculated yield strength of the component;
 dE : Incremental absorbed hysteretic energy;
 β : A calibrated parameter.

In this research, the Damage Index proposed by Park and Ang (1985) is adopted to characterise the loss of capacity of hybrid connection due to the damage of concrete. The advantage of this model is its simplicity, and the fact that it has been calibrated against a significant amount of test results. The maximum displacement response and cumulative energy dissipation of specimen HC3 and HCS5 under positive and negative loading were shown in Table 7.2~7.5. Due to non-symmetrical properties of hybrid connection, the maximum displacement response and energy dissipation under positive and negative loading are different.

Based on Park and Ang's model, the parameter β in Eq. 7-15 can be calculated as:

$$\beta = (-0.447 + 0.073 \frac{l}{d} + 0.24n_0 + 0.314p_t) \times 0.7^{\rho_w} \quad \text{Eq. 7-16}$$

where l/d : Shear span ratio (replaced by 1.7 if $l/d < 1.7$);

n_0 : Normalised axial stress (replaced by 0.2 if $n_0 < 0.2$);

p_t : Longitudinal steel ratio as a percentage (replaced by 0.75% if $p_t < 0.75\%$);

ρ_w : Confinement ratio.

The calculated Damage Indices for specimen HC3 and HCS5 under positive and negative loading were shown in Table 7.6 & 7.7. The damage process for specimen HC3 and HCS5 with the increase of cycle numbers were shown in Figure 7.8 & 7.9 respectively. The damage index $DI = 0$ represents a state of zero damage of specimen under cyclic loading and $DI = 1.0$ represents a complete damage of specimen under cyclic loading. It can be observed that the Damage Indices for specimens with hybrid connections under positive and negative loading are quite close. These Damage Indices will be incorporated into the proposed component-based mechanical model to characterize the damage process of hybrid connection under cyclic loadings.

7.4 Model for Reinforcement under Cyclic Loading

When tension-compression load reversal is applied to the reinforcement, the stress-strain relationship shows the Bauschinger effect, in which the stress-strain curve becomes non-linear at a much lower than the initial yield strength. Kent and Park (1973) have used Ramberg-Osgood relationships to idealize the shape of the softened branches of the stress-strain curve. In their method, the unloading branches of the curve for stresses of both signs are assumed to follow the initial elastic slope. After the first yield excursion, the loading parts of the stress-strain curves are represented by the following form of the Ramberg-Osgood relationship:

$$\varepsilon_s - \varepsilon_{si} = \frac{f_s}{E_s} \left(1 + \left| \frac{f_s}{f_{ch}} \right|^{r-1} \right) \quad \text{Eq. 7-17}$$

in which,

$$f_{ch} = f_y \left[\frac{0.744}{\ln(1 + 1000\varepsilon_{ip})} - \frac{0.071}{1 - e^{1000\varepsilon_{ip}}} + 0.241 \right]$$

For odd-numbered loading runs ($n = 1, 3, 5, \dots$)

$$r = \frac{4.49}{\ln(1 + n)} - \frac{6.03}{e^n - 1} + 0.297 \quad \text{Eq. 7-18}$$

For even-numbered loading runs ($n = 2, 4, 6, \dots$)

$$r = \frac{2.20}{\ln(1 + n)} - \frac{0.469}{e^n - 1} + 3.04 \quad \text{Eq. 7-19}$$

where ε_s : Steel strain;

ε_{si} : Steel strain at zero stress at the beginning of loading run;

- ε_{ip} : Plastic strain in steel produced in previous loading run;
- f_s : Steel stress;
- E_s : Modulus of elasticity of steel;
- n : Loading run number.

As shown in Figure 7.10, the stress-strain relationship of reinforcement under cyclic loading including Bauschinger effect is presented. This relationship will be incorporated into the component-based model to represent the behaviour of reinforcement under cyclic loading.

7.5 Analytical Procedure of Hybrid Connections under Cyclic Loading

The component-based mechanical model proposed in Chapter 6 is extended for hybrid connection under cyclic loading. The numerical procedure for the moment-rotation characteristics of hybrid connection under cyclic loading is similar to that under monotonic loading. The force-displacement relationship for each component under cyclic loading is calculated according to the geometrical sizes and material properties of the component. Taking the rotation of the connection θ as the controlling variable, the analytical procedure for hybrid connection under cyclic loading can be completed.

7.6 Component Behaviour of Hybrid Connection under Cyclic Loading

7.6.1 Comparison between Test Results and Model Predictions

For specimens under cyclic loading (HC3 & HCS5), connection moments were measured based on the reacting forces from load cells and connection rotations were calculated based on the LVDT readings. The comparison between test results and model predictions are shown in Figure 7.11 & 7.12 respectively. In general, the model can give agreeable predictions (average 20% differences) with the test results.

However, it should be noted that the Damage Index adopted in the concrete model is empirical and used for this particular test series only. In fact, the determination of Damage Index is complicated and related to many factors, such as concrete stress, crack width, loading history, deterioration of bond strength between steel and concrete, etc. Therefore, advanced researches are necessary for the refinement of the Damage Index under cyclic loading in the future.

7.6.2 Contribution of Key Components to the Connection Behaviour under Cyclic Loading

Similar to hybrid connections under monotonic loading, the contribution of key component to the behaviour of entire hybrid connection under cyclic loading can be identified. Under cyclic loading, the moment contribution of key components, namely top rebars, bottom rebars, mesh reinforcements, concrete and single plate connection can be singled out as shown in Figure 7.13 & 7.14 (HC3 & HCS5). The moment contribution of each component is calculated by taking moment about the bottom of the hybrid connection. The connection moment is the summation of the moment contribution of each individual component.

When hybrid connection is initially under hogging moment, the slope of the moment-rotation curve is linear prior the cracking of concrete. After cracking, concrete tensile strength starts to decrease following the post peak tensile stress-strain relationship defined in the concrete model. The tensile stress in the top rebars and mesh reinforcements increase significantly due to the cracking in the concrete. The single plate connection starts to contribute to the hogging moment similar to the hybrid connection under monotonic hogging moment. When unloading starts, the tensile stress in the top rebars and mesh reinforcements gradually decrease following the initial elastic slope. When the loading is reversed, the connection moment changes from hogging moment to sagging moment, the top reinforcements are under compression and bottom rebars are under tension. The single plate connection begins to contribute to the sagging moment. During the sequential unloading and reloading, the stress-strain curve of each component changes based

on their respective constitutive relationship defined in the model to account for cyclic conditions. The Damage Index is incorporated into the concrete model to characterise the reduction of concrete stiffness and the decay of concrete strength.

As shown in Figure 7.13 & 7.14, the contribution of key component to the connection behaviour under cyclic loading can be singled out. It should be noted that the moment contribution of each component is always calculated by taking moment about the bottom of the hybrid connection. For a hybrid connection under cyclic loading, the single plate connection does not contribute significantly to the connection behaviour. However, the top and bottom reinforcements actually govern the performance of the hybrid connection. Therefore, the increase of top and bottom reinforcements will effectively enhance the performance and capacity of hybrid connection under cyclic loading.

7.7 Summary

In this chapter, the force-displacement of each component in the hybrid connection under cyclic loading has been described. The cyclic behaviour of hybrid connection can be analysed based on the component-based mechanical model. The main contents are summarised as follows:

- The decay model proposed by Swanson (1999) was adopted to describe the decay of slip load for HSFG bolt under cyclic loading. The mechanical model for single plate connection under monotonic loading can be extended for cyclic loading by introducing the rules of cyclic force-displacement relationship of HSFG bolt.
- Mander's concrete model for cyclic loading has been modified to include the deterioration of concrete tensile strength. An idealised linear relationship is also assumed to describe the unloading and reloading branch in Mander's model.
- The component-based model for hybrid connection under monotonic loading can be extended for cyclic loading as well. The cyclic response of single plate

connection, concrete and reinforcement are incorporated in the component-based model. A Damage Index (DI) is introduced into the concrete model to characterise the reduction of concrete stiffness and decay of concrete strength under cyclic loading due to the damage of concrete.

- Based on the moment contribution of the key components under cyclic loading, the single plate connection does not contribute significantly to the connection behaviour. The top and bottom reinforcements actually govern the performance of the hybrid connection. Therefore, the increase of top and bottom reinforcements will effectively enhance the performance and capacity of hybrid connection under cyclic loading.

Table 7.2 Maximum Displacement Response of HC3 under Cyclic Loading

Specimen	Cycle	δ_m (mm)	δ_u (mm)	δ_m / δ_u
HC3	2 ⁺	9.3	75.2	0.12
	2 ⁻	-11.5	-76.3	0.15
	4 ⁺	15.3	75.2	0.20
	4 ⁻	-14.9	-76.3	0.19
	6 ⁺	21.3	75.2	0.28
	6 ⁻	-20.2	-76.3	0.26
	8 ⁺	33.0	75.2	0.43
	8 ⁻	-30.3	-76.3	0.41
	10 ⁺	45.2	75.2	0.61
	10 ⁻	-42.6	-76.3	0.56
	12 ⁺	71.2	75.2	0.95
	12 ⁻	-69.5	-76.3	0.91

Table 7.3 Maximum Displacement Response of HCS5 under Cyclic Loading

Specimen	Cycle	δ_m (mm)	δ_u (mm)	δ_m / δ_u
HCS5	2 ⁺	8.6	108.2	0.08
	2 ⁻	-9.6	-112.5	0.09
	4 ⁺	15.3	108.2	0.13
	4 ⁻	-15.7	-112.5	0.13
	6 ⁺	21.9	108.2	0.20
	6 ⁻	-23.9	-112.5	0.21
	8 ⁺	35.8	108.2	0.33
	8 ⁻	-36.3	-112.5	0.32
	10 ⁺	46.6	108.2	0.43
	10 ⁻	-49.7	-112.5	0.45
	12 ⁺	73.8	108.2	0.68
	12 ⁻	-73.2	-112.5	0.65
	14 ⁺	104.8	108.2	0.97
	14 ⁻	-107.8	-112.5	0.96

Table 7.4 Cumulative Energy Dissipation of HC3 under Cyclic Loading

Specimen	Cycle	$\int dE$ (kNm)	$F_u \delta_u$ (kNm)	$\int dE / F_u \delta_u$
HC3	2 ⁺	0.1	18.1	0.01
	2 ⁻	0.2	16.8	0.01
	4 ⁺	0.5	18.1	0.03
	4 ⁻	0.5	16.8	0.03
	6 ⁺	1.1	18.1	0.06
	6 ⁻	1.3	16.8	0.08
	8 ⁺	3.2	18.1	0.18
	8 ⁻	3.7	16.8	0.22
	10 ⁺	6.4	18.1	0.35
	10 ⁻	7.2	16.8	0.43
	12 ⁺	17.0	18.1	0.93
	12 ⁻	14.2	16.8	0.85

Table 7.5 Cumulative Energy Dissipation of HCS5 under Cyclic Loading

Specimen	Cycle	$\int dE$ (kNm)	$F_u \delta_u$ (kNm)	$\int dE / F_u \delta_u$
HCS5	2 ⁺	0.1	30.0	0.01
	2 ⁻	0.1	31.2	0.01
	4 ⁺	0.6	30.0	0.02
	4 ⁻	0.6	31.2	0.02
	6 ⁺	1.7	30.0	0.06
	6 ⁻	1.7	31.2	0.05
	8 ⁺	5.1	30.0	0.17
	8 ⁻	5.0	31.2	0.16
	10 ⁺	9.2	30.0	0.31
	10 ⁻	8.5	31.2	0.27
	12 ⁺	17.9	30.0	0.60
	12 ⁻	17.8	31.2	0.57
	14 ⁺	28.9	30.0	0.96
	14 ⁻	28.5	31.2	0.92

Table 7.6 Calculated Damage Index for HC3 under Cyclic Loading

Specimen	Cycle	$\frac{\delta_m}{\delta_u}$	β from Eq. 7-16	$\beta \frac{\int dE}{F_y \delta_u}$	$DI = \frac{\delta_m}{\delta_u} + \beta \frac{\int dE}{F_y \delta_u}$
HC3	2 ⁺	0.12	0.03	0.0003	0.12
	2 ⁻	0.15	0.03	0.0003	0.15
	4 ⁺	0.20	0.03	0.0009	0.20
	4 ⁻	0.19	0.03	0.0009	0.19
	6 ⁺	0.28	0.03	0.0018	0.28
	6 ⁻	0.26	0.03	0.0024	0.26
	8 ⁺	0.43	0.03	0.0054	0.43
	8 ⁻	0.41	0.03	0.0066	0.42
	10 ⁺	0.61	0.03	0.0105	0.62
	10 ⁻	0.56	0.03	0.0129	0.57
	12 ⁺	0.95	0.03	0.0279	0.98
	12 ⁻	0.91	0.03	0.0285	0.95

Table 7.7 Calculated Damage Index for HCS5 under Cyclic Loading

Specimen	Cycle	$\frac{\delta_m}{\delta_u}$	β from Eq. 7-16	$\beta \frac{\int dE}{F_y \delta_u}$	$DI = \frac{\delta_m}{\delta_u} + \beta \frac{\int dE}{F_y \delta_u}$
HCS5	2 ⁺	0.08	0.03	0.0003	0.08
	2 ⁻	0.09	0.03	0.0003	0.09
	4 ⁺	0.13	0.03	0.0006	0.13
	4 ⁻	0.13	0.03	0.0006	0.13
	6 ⁺	0.20	0.03	0.0018	0.20
	6 ⁻	0.21	0.03	0.0015	0.21
	8 ⁺	0.33	0.03	0.0051	0.34
	8 ⁻	0.32	0.03	0.0048	0.32
	10 ⁺	0.43	0.03	0.0093	0.45
	10 ⁻	0.45	0.03	0.0081	0.46
	12 ⁺	0.68	0.03	0.018	0.70
	12 ⁻	0.65	0.03	0.0171	0.67
	14 ⁺	0.96	0.03	0.0268	0.99
	14 ⁻	0.96	0.03	0.0276	0.99

Figure 7.1 Slip load decay model proposed by Swanson (1999)

Figure 7.2 Rules of cyclic $F - \delta$ relationship between HSFG bolt and connected parts

Figure 7.3 Comparison between test result and model prediction

Figure 7.4 Stress-strain curve for unloading branch and determination of plastic strain (Mander et al., 1988)

Figure 7.5 Assumed deterioration in tensile strength of concrete
(Mander et al., 1988)

Figure 7.6 Stress-strain curves for reloading branch (Mander et al., 1988)

Figure 7.7 Modified concrete model for cyclic loading based on Mander et al. (1988)

Figure 7.8 Damage Index (DI) of specimen HC3 under cyclic loading

Figure 7.9 Damage Index (DI) of specimen HCS5 under cyclic loading

Figure 7.10 Stress-strain relationship of reinforcement under cyclic loading
(Kent and Park, 1973)

Figure 7.11 Comparison between test result and model prediction (HC3)

Figure 7.12 Comparison between test result and model prediction (HCS5)

Figure 7.13 Contribution of key components in the hybrid connection under cyclic loading (HC3)

Figure 7.14 Contribution of key components in the hybrid connection under cyclic loading (HCS5)

Chapter 8: Design Method for Hybrid Connections

8.1 Introduction

As discussed in the previous chapters, the component-based mechanical model is able to predict the behaviour of hybrid connections under different loading actions. However, a simplified and yet reliable design method for hybrid connections is also necessary for practical purpose. Design methods are provided for both single plate connection and hybrid connection as follows.

8.2 Design Method for Single Plate Connection at Construction Stage

During the construction stage, the self-weight of structural members and the construction live load are carried in shear by the single plate connections only. Since the single plate connection is incapable to transmit significant moment, it is convenient to assume that the single plate connection carries the shear load only in practice. The shear capacity of single plate connection is governed by the following items:

- Shear resistance of the bolt group;
- Bearing resistance of the bolts against the steel plate or beam web;
- Weld resistance in shear;
- Block shear resistance of steel plate;
- Block shear resistance of beam web.

The minimum of the above will give the actual shear capacity of the connection. Current design codes and standards can be used to check the detailing requirements and the shear capacity of the single plate connection.

8.3 Design Method for Hybrid Connections at Service Stage

After the casting of cast-in-place concrete, the hybrid connection behaves as a semi-rigid connection. At the service stage, the shear forces are carried by the composite beam and the shear deformation is negligible compared to the deformation due to bending based on the calculation of shear deformation given in Appendix A. A simple and practical method is proposed to predict the moment capacity, rotation capacity as well as moment-rotation relationship of hybrid connections.

8.3.1 Stiffness Variation of Hybrid Connection under Monotonic Loading

Based on the test results, the typical moment-rotation relationship of a hybrid connection under monotonic loading can be idealized as comprising of three stages. A tri-linear moment-rotation relationship is proposed to represent the behaviour of hybrid connections under monotonic hogging and sagging moment (Figure 8.1). The first stage can be considered as elastic stage, which is prior to the cracking of concrete. At this stage, the stiffness of the hybrid connection is very high and the $M - \theta$ curve is linear. Subsequently, the stiffness reduces as the cracks propagate. The reinforcement will be in direct tension and the stiffness of the hybrid connection is reduced significantly. This second stage can be simplified by a linear relationship up to the point when the reinforcements reach the yielding strength. After the yielding of reinforcement, the hybrid connection enters into plastic stage and the connection moment keeps constant.

8.3.1.1 Stiffness before the cracking of concrete (First stage)

The initial moment-rotation relationship of the hybrid connection prior to the cracking of concrete is modelled with the moment curvature relationship of the reinforced concrete section. The end of this stage is when the first crack appears in the concrete. The concrete in tension is assumed to be cracked when the concrete modulus f_{rp} is reached. The modulus of rupture f_{rp} suggested by Park (1975) is given as:

$$f_{rp} = 0.58\sqrt{f_c} \quad \text{Eq. 8-1}$$

where f_c is the concrete cylinder compressive strength (N/mm).

The cracking moment M_{cr-hog} can be calculated using the transformed area method, which is based on the proportionality between the elastic modulus of the concrete and steel. The corresponding rotation θ_{cr-hog} can be derived based on the moment-curvature relationship of uncracked section of hybrid connection. The stiffness at the first stage K_{cr-hog} can be obtained as well. Detailed calculations are given in the Appendix A. This linear relationship forms the initial stiffness of the hybrid connection prior to the cracking of concrete.

8.3.1.2 Stiffness after cracking and before the yielding of reinforcement (Second stage)

After the initiation of the first crack, the stiffness of the hybrid connection is reduced. When concrete starts cracking, the contribution of reinforcements and steel connection to the overall stiffness increases gradually. The stress state at this stage is complicated since each component contributes to the overall behaviour of the hybrid connection. As the connection moment increases, each component follows its constitutive force-displacement relationship and the non-linear moment-rotation behaviour of the hybrid connection is formed. The limiting point of this stage is when the steel reinforcements reach the yielding strength. The moment capacity and rotation capacity is termed as M_{hog} and θ_{hog} respectively. Although the moment-rotation relationship is non-linear at this stage, for practical purpose, this relationship is assumed to be linear during this stage.

8.3.1.3 Stiffness after yielding of reinforcement (Third stage)

For design purpose, it is conservative to use the yield strength throughout instead of ultimate strength after the yielding of steel reinforcement. The hybrid connection is

assumed to enter the plastic stage and the connection moment keeps constant. For hybrid connection under sagging moment, the three stages are defined follow the same principle. The complete moment-rotation curve under both hogging and sagging moment is shown in Figure 8.1.

8.3.2 Prediction of Moment Capacity

The method of prediction for the ultimate moment capacity is based on the stress-block analysis. Therefore, the basis of the method relies on the horizontal force equilibrium between the components contributing to the tensile and compressive regions. As discussed in Chapter 6, the maximum hogging moment capacity is governed by the ultimate tensile strength of tensile reinforcement and the maximum sagging moment capacity is determined by the maximum compressive strength of concrete.

8.3.2.1 Hogging moment capacity

When the hybrid connection is under hogging moment, the key components which are the top rebars, mesh reinforcements, steel connection and concrete will contribute to the hogging moment capacity of the hybrid connection. There are three possible cases in which the ultimate state of the hybrid connection may belong to. These three cases are classified according to the position of the neutral axis, which can be determined based on force equilibrium condition at the ultimate state.

- Neutral axis above the single plate connection.

In this case, the top rebars as well as the mesh reinforcements are fully in tension and the single plate connection is fully in compression. This case is unlikely to occur in practice unless very heavy top reinforcement is provided. This may result in over-reinforced section and connection may fail by crushing of the concrete before the reinforcements start to yield.

- Neutral axis within the single plate connection

The top rebars as well as the mesh reinforcements are fully in tension, while the single plate connection is in partial tension and compression. The tensile force is increased gradually by adding the contribution of single plate. The compressive force is increased gradually by adding the contribution of the concrete and bottom rebars. This will be done until the equilibrium is achieved.

- Neutral axis below the single plate connection

The top rebars, mesh reinforcements and single plate connection are fully in tension. The tensile force is equal to the compressive force provided by concrete in compression and bottom rebars. In general, the height of concrete stress block under compression is small and the neutral axis is always below the single plate connection.

In the case that the neutral axis is below the single plate connection, the position of neutral axis and compression centre of the hybrid connection under hogging moment is shown in Figure 8.2. The hogging moment is resisted by the top rebars, mesh reinforcements and steel connection in tension, and cast-in-place concrete and bottom rebars in compression. As indicated in Figure 6.34, all HSFG bolts have exceeded the slip resistance at the ultimate state and the interacting force between HSFG bolt and connected parts can be considered as slip resistance F_{slip} . When the maximum hogging moment is reached, the position of the compression centre can be determined. For design purpose, it is conservative to ignore the contribution of bottom rebars, which also simplifies the design calculation. Based on the BS8110 rectangular stress block approach, from the equilibrium of forces, the depth of neutral axis under hogging moment x_h can be given as:

$$x_h = \frac{f_y \cdot A_{s,top} + f_{ym} \cdot A_{sm} + nF_{slip}}{0.9 \times 0.67 f_{cu} \cdot b_p} \quad \text{Eq. 8-2}$$

$$d_h = 0.45x_h \quad \text{Eq. 8-3}$$

where f_y : Yield strength of steel;

f_{ym} : Yield strength of steel mesh;

f_{cu} : Cube strength of concrete;

$A_{s,top}$: Area of top rebars;

A_{sm} : Area of mesh reinforcements inside the effective breadth of the topping concrete;

F_{slip} : Slip resistance;

b_p : Width of precast beam;

n : Row number of bolts;

x_h : Depth of neutral axis under hogging moment;

d_{ch} : Depth of compression centre under hogging moment.

As shown in Figure 8.2, the hogging moment capacity M_{hog} can be given as:

$$M_{hog} = F_{rsm} \cdot d_{sm} + F_{r,top} \cdot d_{top} + \sum_{i=1}^n F_{slip} \cdot y_i - (b_p \cdot 2d_{ch} \cdot 0.67f_{cu}) \cdot d_{ch} \quad \text{Eq. 8-4}$$

where d_{sm} : Distance from bottom of connection to the centre of mesh;

d_{top} : Distance from bottom of connection to the centre of top rebars;

y_i : Distance from bottom of connection to the i th HSFG bolt;

b_p : Width of precast beam.

8.3.2.2 Sagging moment capacity

Similar to the hybrid connection under hogging moment, there are also three possible cases at the ultimate limit state of the hybrid connection may belong to. These three cases are classified according to the position of the neutral axis.

- Neutral axis above the single plate connection.

In this case, bottom rebars and single plate connection are fully in tension. The tensile force is equal to the compressive force provided by top rebars, mesh reinforcements, topping concrete and concrete in compression. In general, the height of the concrete stress block under compression is small and the neutral axis is always above the single plate connection.

- Neutral axis within the single plate connection

The bottom rebars are fully in tension, while the single plate connection is in partial tension and compression. The tensile force is increased gradually by adding the contribution from the single plate connection. The compressive force is increased gradually by adding the contribution from the concrete and top reinforcements. This will be done until the equilibrium is achieved.

- Neutral axis below single plate connection

This case is unlikely to occur unless very heavy bottom rebars are provided, which is unreasonable in practice.

In the case that neutral axis above the single plate connection, the position of neutral axis and the compression centre of the hybrid connection under sagging moment is shown in Figure 8.3. The sagging moment is resisted by the bottom rebars and the steel connection in tension, and top rebars, mesh reinforcements and concrete in compression. The contribution of reinforcements in compression is also

ignored due to the conservative reason. As indicated in Figure 6.36, all HSFG bolts have exceeded the slip resistance and entered the bearing stage at the ultimate state. Although single plate connection will continue to enhance the connection moment after the yielding of bottom tensile reinforcements, it is conservative to ignore this moment enhancement due to bearing and consider the interacting force between HSFG bolt and connected parts as slip resistance F_{slip} . The depth of neutral axis under sagging moment x_s can be given as:

$$x_s = \frac{f_y \cdot A_{s,bot} + nF_{slip}}{0.9 \times 0.67 f_{cu} \cdot b_{topping}} \quad \text{Eq. 8-5}$$

$$d_{cs} = 0.45x_s \quad \text{Eq. 8-6}$$

where f_y : Yield strength of steel;
 f_{cu} : Cube strength of concrete;
 $A_{s,bot}$: Area of bottom reinforcement;
 F_{slip} : Slip resistance;
 $b_{topping}$: Width of topping concrete;
 n : Row number of bolts;
 x_s : Depth of neutral axis under sagging moment;
 d_{cs} : Depth of compression centre under sagging moment.

As shown in Figure 8.3, the sagging moment capacity M_{sag} can be given as:

$$M_{sag} = F_{r,bot} \cdot d_{bottom} + \sum_{i=1}^n F_{slip} \cdot y_i - (b_{topping} \cdot 2d_{cs} \cdot 0.67 f_{cu}) \cdot d_{cs} \quad \text{Eq. 8-7}$$

where d_{bottom} : Distance from top of connection to the centre of bottom rebars;
 y_i : Distance from top of connection to the i th HSFG bolt;

$b_{topping}$: Width of topping concrete.

8.3.3 Rotation Capacity of Hybrid Connection

As discussed above, the moment capacity and depth of neutral axis of a hybrid connection under hogging and sagging moment can be determined. The rotation capacity can therefore be calculated based on the elongation of reinforcement at the yield strength and the position of neutral axis. The proposed method to predict the rotation capacity under hogging and sagging moment is shown in Figure 8.4.

8.3.3.1 Rotation capacity of hybrid connection under hogging moment

As shown in Figure 8.4 (a), based on the assumption that the beam cross section remains plane after deformation, therefore, the rotation capacity can be calculated based on the elongation of reinforcement at the yield strength. From the results of a tensile test, the yield strain of the reinforcement is about $3000 \mu\epsilon$. As discussed in Chapter 6, the length of reinforcement considered for calculating the steel strain may be taken as $h_c / 2 + 225$ mm (Eq.6-34). Therefore, the elongation of reinforcement Δ_r at the yield strength can be given as:

$$\Delta_r = 0.003 \left(\frac{h_c}{2} + 225 \right) \quad \text{Eq. 8-8}$$

where h_c : Depth of the column section.

Hence, the rotation capacity under hogging moment can be calculated as:

$$\theta_{hog} = \frac{\Delta_r}{d_h} \quad \text{Eq. 8-9}$$

$$d_h = d_{top} - x_h \quad \text{Eq. 8-10}$$

where d_h : Distance from neutral axis to the centre of top rebars;
 d_{top} : Distance from bottom of connection to the centre of top rebars;
 x_h : Depth of neutral axis under hogging moment

8.3.3.2 Rotation capacity of hybrid connection under sagging moment

Similarly, as shown in Figure 8.4 (b), for hybrid connection under sagging moment, the rotation capacity can be calculated as:

$$\theta_{sag} = \frac{\Delta_r}{d_s} \quad \text{Eq. 8-11}$$

$$d_s = d_{bottom} - x_s \quad \text{Eq. 8-12}$$

where d_s : Distance from neutral axis to the centre of bottom rebars;
 d_{bottom} : Distance from top of connection to the centre of bottom rebars;
 x_s : Depth of neutral axis under sagging moment.

8.4 Comparisons between Tri-linear Model, Component-based Mechanical Model and Test Results

Using simplified design method, the cracking moment, moment capacity and the corresponding rotations of hybrid connection can be calculated. As shown in Figure 8.5, the tri-linear model is compared with the test results for hybrid connection under both hogging and sagging moment. The component-based model is also included for comparison. Compared to component-based model, the simplified design method underestimates the sagging moment capacity by about 16% and the hogging moment capacity by about 23%. The reason is mainly due to the different

concrete design strength used in these two methods. The simplified design method was actually based on the BS8110 stress block approach. Based on BS8110, $0.67f_{cu}$ is adopted for the difference between the bending strength and the cube crushing strength of the concrete. The hogging and sagging moment capacities were calculated based on the concrete strength of $0.67f_{cu}$, which is a lower value than the concrete cylinder strength $0.8f_{cu}$ used in the component-based model. Therefore, the moment capacity calculated by simplified method is lower than the value predicted by component-based model.

Although the component-based model can give a better agreement with the test results, its modelling procedure is much more complicated compared to the simplified design method. Therefore, the proposed tri-linear model can yield a simple and conservative prediction which is more suitable for design purpose.

8.5 Summary

Single plate connection can be assumed to carry the shear load only in practice. The shear capacity of single plate connection can be calculated based on current design codes. A simplified design method is proposed to predict the moment capacity, rotation capacity as well as moment-rotation relationship for hybrid connection under both monotonic hogging and sagging moment. This approach gives a convenient and safe prediction for design purpose.

Figure 8.1 Proposed tri-linear model for hybrid connection under monotonic loading

Figure 8.2 Force distribution of hybrid connection under hogging moment

Figure 8.3 Force distribution of hybrid connection under sagging moment

Figure 8.4 Rotation capacity of hybrid connection

- (a) Rotation capacity under hogging moment
- (b) Rotation capacity under sagging moment

Figure 8.5 Tri-linear model predictions of hybrid connection under monotonic loading

Chapter 9: Analysis of Frames with Hybrid Connections

9.1 Introduction

As discussed in the literature review, the methods of frame analysis can be classified as simple design, rigid design and semi-rigid design. The simple design and rigid design assume the beam-column joint have zero moment capacity and zero rotational capacity, respectively. However, it is apparent that neither pinned nor rigid connections are actually obtained in real structures and the actual behaviour of all connections falls between these two extremes. Therefore, it would be necessary for structural engineers to adopt the semi-rigid methodology to reflect the true behaviour of connections rather than the assumed ideals of nil or full restraint in the connections. By introducing the connector element to represent the moment-rotation relationship of the hybrid connection in the finite element analysis, it is possible to predict the actual behaviour of frames with hybrid connections under vertical and horizontal loads. In practice, the frame can be designed as braced or unbraced frame. A column may be considered braced in a given plane if lateral stability (due to wind etc.) to the structure as a whole is provided by walls or bracing or buttressing designed to resist all lateral forces in that plane. Otherwise, it should be considered unbraced. In this chapter, the analyses of braced and unbraced frames with hybrid connections are discussed as follows.

9.2 Numerical Modelling of Frames with Hybrid Connections

The frame is selected based on a typical industrial building layout in Appendix A. The modelling of frame consists of two stages, which are the construction stage and service stage. At the construction stage, the precast beams are connected to the columns through single plate connections only. The beam-column joints are modelled as semi-rigid connections based on the moment-rotation relationship of single plate connection at the construction stage, in which the effect of shear force acting on the single plate connection is taken into account. At the service stage, the

hybrid connections are formed and therefore modelled as semi-rigid connections based on the moment-rotation relationship of hybrid connection at service stage. ABAQUS is used to model the behaviour of precast frame at the construction stage and service stage. The beam and column are modelled by the BEAM section elements (uncracked). For frames with semi-rigid connections, the beam-column connections can be modelled by connector elements based on their respective moment-rotation relationships. As shown in Figure 9.1, the node of the beam is connected to the node at the column via a connector element. The connector element is defined in such a way that there is no translational deformation and only rotational deformation is allowed.

The rotational property of the connector element is based on the predicted moment-rotation curves. To prepare the moment-rotation relationships under hogging and sagging moment, a three-step approach is taken as follows:

- Step 1: Obtain the predicted moment-rotation curve of connection under hogging moment;
- Step 2: Obtain the predicted moment-rotation curve of connection under sagging moment;
- Step 3: Combine these two curves to obtain a complete moment-rotation curve for loading in any direction.

The complete moment-rotation relationship is defined in the rotational property of the connector element to describe the variation of connection rotation under both hogging and sagging moment.

9.3 Stability Analysis of Precast Frame at Construction Stage

During the construction stage, the precast beams are connected to the columns through single plate connections. The single plate welded on the centre of column

flange may be susceptible to the out of plane bending. A stiffening arrangement is suggested to improve the stability of single plate during the construction stage (Figure 9.2). After the installation of the HSFG bolts on both ends of the precast beam, the single plate connections can achieve instant ability to undertake the self-weight of precast beam. Two triangle stiffeners can be welded on one side of the single plate to prevent the possible out of plan bending. In practice, vertical and horizontal slotted holes are usually used to cater for the vertical and horizontal tolerances (Figure 9.3).

As shown in Figure 9.5, precast frame with single plate connections is analysed to check the stability at the construction stage. Figure 9.4 shows the moment-rotation relationship of a single plate connection during the construction stage, which is incorporated into the rotational property of connector element. The factored uniform load and lateral load acting on the precast frame are calculated based on the case study in Appendix A. The analytical results are presented in Figure 9.6 and the maximum bending moment at column base is 143.3 kN.m. Based on the capacity check of embedded universal column given in Appendix A, the universal column section is satisfactory and therefore the structural system is stable at construction stage.

9.4 Analysis of Braced Frames with Hybrid Connections

The modelling of braced frame with hybrid connections is shown in Figure 9.8. Based on the case study in Appendix A, the factored uniform load on the beam is 145.7 kN/m. As shown in Figure 9.7, the predicted moment-rotation relationship of hybrid connection (HCS1 & HCS2) at service stage is incorporated into the rotational property of connector element. Beam-column connections of the frame are designed as pinned, rigid and semi-rigid respectively. Comparisons are carried out to study the behaviour of frames with these three different types of connections.

As shown in Figure 9.9, the beam-column connection is assumed to be pinned, the beam behaves like the simply supported beam and has the maximum bending moment of -792.5 kN.m at the mid-span of the beam. This ideal assumption neglects the contribution of the beam-column connection and may increase the beam size as well as the amount of reinforcements in beam. If the beam-column connection is assumed to be rigid (Figure 9.10), the frame will have a maximum hogging moment of 491.3 kN.m at the beam-column joint. This ideal assumption overestimates the hogging moment at the beam-column connection which may result in reinforcement congestion and possible honeycombed concrete inside the beam-column joint during construction. Without the introduction of semi-rigid design methodology, it is difficult for structural engineers to know the actual behaviour of semi-rigid frame and design the structural members accurately.

The introduction of the semi-rigid connection to the frame analysis provides the solution to the above problem. It is assumed that the external connection has the same moment-rotation relationship as the internal connection. As shown in Figure 9.11, the actual maximum hogging and sagging moment are 365.7 kN.m and -403.7 kN.m respectively. The semi-rigid methodology provides the insight into the influence of semi-rigid connection to the global behaviour of frame and also enables structural engineers to know the actual behaviour of structures with semi-rigid connections. The comparison of beam deflections with pinned, rigid and semi-rigid connections for braced frame under vertical loads is shown in Figure 9.12 and Table 9.1.

9.5 Analysis of Unbraced Frames with Hybrid Connections

The modelling of unbraced frames with hybrid connections is shown in Figure 9.13. For unbraced frame, the frame has to resist the moments caused by both vertical and horizontal loads. The factored lateral loads acting on the frame are determined based on the comparison between the notional load and wind load given in Appendix A. The analysis of unbraced frame may be separated into analysis under

vertical loads and analysis under horizontal loads. The analysis under vertical loads can be made in the same way as braced frames. The analysis of frames with pinned, rigid and semi-rigid connections under horizontal loads are discussed as follows.

As shown in Figure 9.14, the beam-column connection is assumed to be pinned, the column behaves like cantilever column and has the maximum bending moment of 248.1 kN.m at the bottom of the column. This ideal assumption neglects the contribution of beam-column connection and overestimates the moment in the column. For frame with rigid and semi-rigid connections, the moment distribution of frame under horizontal loads is shown in Figure 9.15 and 9.16 respectively. The initial stiffness under hogging and sagging moment was 2591 and 2342 kN.m/mrad respectively. Based on the observation of these two moment distribution diagrams, the frame with semi-rigid connections can resist the horizontal loads as effectively as frame with rigid connections. The comparison of lateral displacement for unbraced frame with pinned, rigid and semi-rigid connections under lateral loads is shown in Figure 9.17 and Table 9.2. The frame with semi-rigid connections has a similar lateral displacement as rigid frame.

9.6 Design Guide for the Analysis of Frames with Hybrid Connections

The connection behaviour may significantly affect the local behaviour of a structure. For beams, since reducing connection stiffness will directly reduce the hogging end-moments and increase sagging mid-span moments compared to the fully rigid method, the design moments are very sensitive to the semi-rigid connection behaviour. The connection behaviour may also significantly affect the global behaviour of a structure. The mid-span deflections of beams and lateral drift of a frame will increase due to the non-linear connection behaviour compared to the fully rigid method. The proposed design guide for the analysis of frames with hybrid connections can be given as follows (Figure 9.18):

1. Load application;

2. Preliminary member sizing;
3. Design of single plate connection;
4. Select top and bottom reinforcements of hybrid connection;
5. Determine the moment-rotation relationship of hybrid connection under both hogging and sagging moment (Use component-based mechanical model);
6. Carry out frame analysis incorporating the non-linear moment-rotation relationship into the connector element;
7. Check the serviceability limit states to ensure adequate stiffness for functionality at service loads;
8. Check the ultimate load carrying capacity;
9. Return to step 1 until all factored load combinations are satisfied. If member sizing is required, then return to step 2.

In the semi-rigid design method, the properties of connections are also treated as design variables, so that both the members and the connections can be optimised. Therefore, this approach is more general and provides greater freedom than both the traditional pinned and rigid methods.

9.7 Summary

The predicted moment-rotation relationships are incorporated into the connector elements in the finite element program for semi-rigid frame analysis. The structural system is found to be stable during the construction stage based on the stability analysis of semi-rigid frame. Design guidance for the analysis of frame with hybrid connections is also suggested. The two extreme assumptions of idealised connection behaviour (pinned and rigid condition) in the frame analysis may lead to substantial inaccuracies in the overall frame response assessment. The semi-rigid frame approach can realistically account for the connection behaviour and produces more appropriate results than the traditional methods.

Figure 9.1 Connector element

Figure 9.2 Stiffening arrangements of single plate connection during construction stage

Figure 9.3 Vertical and horizontal slotted holes used in practice

Figure 9.4 Moment-rotation relationship used in the connector element of single plate connection at construction stage

Figure 9.5 Stability analysis of unbraced precast frame at construction stage

Figure 9.6 Moment-distribution of precast frame at construction stage

Figure 9.7 Moment-rotation relationship used in the connector element of hybrid connection at service stage

Figure 9.8 Braced frame with hybrid connections under vertical loads

Figure 9.11 Moment-distribution of braced frame with semi-rigid connections

Figure 9.12 Deflection of braced frame under vertical loads

Figure 9.13 Unbraced frame with hybrid connections under horizontal loads

Figure 9.14 Moment-distribution of unbraced frame with pinned connections

Figure 9.15 Moment-distribution of unbraced frame with rigid connections

Figure 9.16 Moment-distribution of unbraced frame with semi-rigid connections

Figure 9.17 Lateral displacement of unbraced frame under horizontal loads

Figure 9.18 Design guide for the analysis of frame with hybrid connections

Table 9.1 Comparison of beam deflection for braced frame under vertical loads

Storey	Pinned connection	Rigid connection	Semi-rigid connection
	Beam deflection (mm)	Beam deflection (mm)	Beam deflection (mm)
2 nd	12.0	4.3	5.9
3 rd	13.7	5.3	7.3
4 th	16.1	6.1	9.0
5 th	19.3	6.9	11.1

Table 9.2 Comparison of lateral displacement for unbraced frame under horizontal loads

Storey	Pinned connection	Rigid connection	Semi-rigid connection
	Lateral displacement (mm)	Lateral displacement (mm)	Lateral displacement (mm)
2 nd	4.3	0.9	0.9
3 rd	12.3	1.6	1.6
4 th	22.0	2.1	2.1
5 th	32.2	2.3	2.3

Chapter 10: Conclusions

10.1 Summary of Present Research

In this study, three categories of research work have been carried out. Firstly, a total twelve of specimens have been tested to study the behaviour of hybrid connections under monotonic and cyclic loading. The experimental work undertaken in this research assists in the development of a better understanding of the behaviour of single plate connection and hybrid connection under different loading actions. The findings have laid the bases for developing the component-based mechanical model to predict the behaviour of hybrid connection under various loadings.

Secondly, a mechanical model and a component-based mechanical model have been proposed to predict the behaviour of single plate connection and hybrid connection respectively. The mechanical model is developed to analyse the moment-rotation relationship of single plate connection under both monotonic and cyclic loading. The predicted results have good agreement (within 20% differences) with the experimental results. This model can be incorporated into the component-based mechanical model, which is proposed for the analysis of the hybrid connection. In the component-based model, the non-linear force-displacement relationship of each component has been taken into account. For hybrid connections under monotonic loading, the effectiveness and accuracy of the component-based model has been demonstrated by the verifications against the test results. Moreover, the component-based model can be extended to predict the behaviour of hybrid connection under cyclic loading through the introduction a Damage Index into the concrete model.

Finally, the model prediction of hybrid connections can be incorporated into connector elements in the existing finite element program to analysis the frame with hybrid connections. A simplified and yet reliable design method of hybrid connection as well as the design guide for the analysis of frames with hybrid connections have also been developed.

The key conclusions and observations from the experimental and analytical studies are summarised below:

- (1) Review of the available work on semi-rigid steel and composite connections revealed only limited investigations were carried out on the behaviour of single plate connection. There is no existing composite connection similar to the proposed hybrid connection, which consists of the steel connection and concrete encasement.
- (2) Review of the existing models on semi-rigid composite connections suggested that the component-based mechanical model compares favourably with other analytical methods due to its accuracy and versatility. However, there is no attempt or model to predict the behaviour of the proposed hybrid connection and the existing modelling techniques are inadequate in the analysis of such hybrid connections.
- (3) Tests of specimens indicate that the proposed hybrid connection has certain results in the strength and ductility, which suggests that such hybrid connection may be suitable for the precast construction industry.
- (4) The experimental results indicate a typical three-stage behaviour for single plate connection under monotonic loading. Under cyclic loading, the hysteretic loops of force-lateral displacement ($P - \Delta$) and moment-rotation ($M - \theta$) relationship are stable and ductile. It is also observed that the cyclic behaviour of single plate connection reflects a kind of cyclic slip and bearing behaviour and the monotonic moment-rotation curve could be used as the skeleton curve for cyclic loading.
- (5) The performance of hybrid connection depends mainly on the contribution of each component. Adjustment of variables, such as amount of top and bottom reinforcements influences the response noticeably. Owing to the continuity of top and bottom reinforcements crossing the joint, hybrid connection can achieve significant moment resistance under both hogging and sagging loading. Cracks are developed along the critical sections which

are the interfaces between the cast-in-place concrete and the steel column flanges at the joint due to the weak bond between concrete and steel.

- (6) The hysteretic loops of hybrid connection are found to be ductile and stable under cyclic loading. However, the stiffness and strength of hybrid connection are much reduced and unable to achieve the same ultimate strength as those under monotonic loadings due to the fast development of cracks under the cyclic loading which may accelerate the damage of the concrete.
- (7) A mechanical model has been proposed to simulate the behaviour of single plate connection with HSFG bolts under monotonic loading. The force-displacement relationship between HSFG bolt and connected parts is presented by a three-stage analytical model. The mechanical model can be extended for cyclic loading by introducing the rules of cyclic force-displacement relationship of HSFG bolt. This model can be incorporated into the component-based mechanical model which is proposed for the analysis of hybrid connection.
- (8) The proposed component-based mechanical model is capable of predicting the hybrid connection behaviour under monotonic loading throughout the nonlinear range with good agreement. The adopted methodology is based on the assembling of contributions from various connection components, such as reinforcement, concrete, single plate connection, etc. The formulation takes into account the nonlinear behaviour of each component in the hybrid connection. Mander's concrete model has been modified to include the deterioration of concrete tensile strength.
- (9) Based on the moment contribution of the key components under hogging moment, single plate connection only contributes a small portion to the connection moment, however, the top tensile reinforcements actually govern

the connection behaviour. Therefore, increasing of top tensile reinforcement will effectively enhance the moment capacity.

- (10) Based on the moment contribution of the key components under sagging moment, single plate connection continues to enhance the connection moment even when the bottom tensile reinforcements reach the yielding strength. The connection behaviour is actually governed by the behaviour of single plate connection instead of bottom tensile reinforcements. Therefore, the increase of steel reinforcement contributes little to the overall connection behaviour after the bottom rebars exceeding the yield strength.
- (11) Based on the moment contribution of the key components under cyclic loading, the single plate connection does not contribute significantly to the connection behaviour. The top and bottom reinforcements actually govern the performance of the hybrid connection. Therefore, the increase of top and bottom reinforcements will effectively enhance the performance and capacity of hybrid connection under cyclic loading.
- (12) The proposed component-based model can be extended for hybrid connection under cyclic loading. The cyclic response of reinforcement, single plate connection and concrete are incorporated in the component-based model. A Damage Index (DI) is introduced into the model to characterise the reduction of concrete stiffness and decay of concrete strength under cyclic loading due to the damage of concrete.
- (13) A simplified design method is proposed to predict the moment capacity, rotation capacity as well as moment-rotation relationship of hybrid connection. This approach gives a convenient and safe prediction for design purpose.
- (14) The predicted moment-rotation relationships can be incorporated into the connector elements in the finite element program for semi-rigid frame analysis. The structural system is found to be stable during the construction

stage based on the stability analysis of semi-rigid frame. Design guide for the analysis of frame with hybrid connections is developed. The semi-rigid frame approach can realistically account for the connection behaviour and produces more appropriate results than the traditional methods.

10.2 Suggestions for Future Research

In this study, mechanical models to describe the behaviour of single plate connection and hybrid connection have been proposed. During the development of these models, a number of issues requiring further studies and some new topics have been identified. Suggestions for future researches are summarised as follows:

- a) For hybrid connection under cyclic loading, the determination of Damage Index is complicated and related to many factors. Therefore, advanced researches are necessary for the refinement of the Damage Index under cyclic loading in the future.
- b) Further refinement in connection modelling necessitates the evaluation of the effect of shear forces on the moment-rotation behaviour of hybrid connection.
- c) The behaviour of external hybrid connections needs to be studied.
- d) In order to achieve comprehensive design guidance for the frames with hybrid connections, extensive parametric studies focusing on the moment-rotation relationships of connections need to be undertaken.

Research directed towards the suggested topics aims at obtaining specific and complete design guidelines for the structures with hybrid connections. Only then structural engineers would be confident to utilise the advantages of proposed hybrid connection technology in practice.

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Appendix A: Case Study

A. 1 Typical Floor Plan of Precast Industrial Building

Figure A.1 Typical Floor Plan of Precast Industrial Building

A typical floor plan for three-storey industrial building is shown in Figure A.1. The slab span is 8 m and the beam span is 6.65 m. The floor-to-floor height is 3.2 m. The precast components consist of hollow core slabs, composite beams and precast columns. The hollow core slab is 265 mm thick with 75 mm topping concrete. The composite beam consists of two components, the 500 x 460 mm precast concrete beam and the 300 x 340 mm cast-in-place portion. The service imposed live load is 7.5 kN/m^2 and a construction live load is 1.5 kN/m^2 . The detail of the hybrid connection is the same as the specimen HCS1. Based on this layout plan, study is carried out on the behaviour of semi-rigid hybrid connection between the composite beam and the internal column.

Hybrid connection consists of single plate connection, precast beam with embedded steel section, cast-in-place concrete and precast column with embedded steel section at the joint position. The detailed design of each component is given as below.

A. 2 Design of Steel Section Embedded in the Precast Column

1) Design Loading at Construction Stage

- Floor loading:

Dead load:

Hollow core slabs:	=	4.5	kN/m ²
Topping concrete (75 mm):	=	1.8	kN/m ²
Total:	=	6.3	kN/m ²

Live load (Construction):	=	1.5	kN/m ²
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- Ultimate loading on the embedded steel section of precast column:

Dead Load:

Composite beam self-weight:	=	63.9	kN
Precast column self-weight:	=	13.8	kN
DL transferred from composite beam:	=	335.2	kN
Total:	=	412.9	kN

$$\text{Ultimate load per floor: } N = 1.4 \text{ DL} = 578.1 \text{ kN}$$

Live Load:

$$\text{Ultimate live load per floor: } Q = 1.6 \text{ LL} = 127.6 \text{ kN}$$

Ultimate load on embedded steel section (consider 3 floors are erected at each time):

$$P = 3 \times 705.7 = 2117 \text{ kN}$$

2) Design of Embedded Steel Section of Precast Column

Ultimate Loading on Steel Section: $P = 2117 \text{ kN}$

Try 356 x 171 UB 67,

$$p_y = 275 \text{ N/mm}^2$$

$$A = 85.5 \text{ cm}^2$$

$$P_c = 2351 \text{ kN} > P = 2117 \text{ kN}$$

The section selected is satisfactory.

3) Design of Shear Studs in Precast Column

Headed shear studs with 19 mm diameter are used. The ultimate tensile strength (nominal strength) of shear stud is $f_u = 450 \text{ N/mm}^2$. Along both sides of the steel universal column, they are placed uniformly with two shear connectors per trough at a spacing of 100 mm. According to clause 6.1.2 (2) (Eurocode 4, 1992), these shear connectors are classified as ductile connectors.

Shear studs are designed to ensure full shear connection. The ultimate compressive force in the embedded steel universal column is carried by the resistance of shear studs during the construction stage. According to Eurocode 4 (1992), the design shear resistance of an automatically welded headed stud can be determined as the lesser of followings:

$$P_{RD} = 0.8 f_u (\pi d^2 / 4) / \gamma_v \quad \text{Eq. A-1}$$

$$P_{RD} = 0.29 \alpha d^2 \sqrt{(f_{ck} E_{cm})} / \gamma_v \quad \text{Eq. A-2}$$

From Eq. A-1,

$$P_{RD} = 0.8 \times 450 \times (3.14 \times 19^2 / 4) / 1.25 = 81.6 \text{ kN}$$

From Eq. A-2,

$$P_{RD} = 0.29 \times 1 \times 19^2 \times (35 \times 32000)^{0.5} / 1.25 = 88.6 \text{ kN}$$

Therefore, $P_{RD} = 81.6 \text{ kN}$.

The number of shear connectors for full shear connection is given by:

$$N_f = P / P_{RD} = 26$$

Provide total 28 shear connectors, which is sufficient to ensure the full shear connection. However, in this experimental programme, since there is no axial load acting on the column, the shear studs were placed two numbers per trough at a spacing of 200 mm. Total 12 shear studs were used for each embedded steel column.

A. 3 Design of Single Plate Connection at Construction Stage

1) Design Loading Transferred by the Precast Concrete Beam

- Floor loading:

Dead load:

Hollow core slabs:	=	4.5	kN/m ²
Topping (75 mm):	=	1.8	kN/m ²
Total:	=	6.3	kN/m ²

Live load (Construction):	=	1.5	kN/m ²
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- Ultimate loading transferred by the precast concrete beam:

Dead load:

Composite beam self-weight:	=	25.3	kN
DL transferred from composite beam:	=	167.6	kN
Total:	=	192.9	kN

LL transferred from composite beam:	=	39.9	kN/m ²
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Ultimate load transferred: $V = 1.4 \text{ DL} + 1.6 \text{ LL} = 333 \text{ kN}$

2) Design of Single Plate Connection

During the construction stage, the self-weight of structural members and the construction live load are carried by the single plate connections only. Since the single plate connection is incapable to transmit significant moment, it is convenient to assume that the single plate connection carries the shear load only in practice. The shear capacity of single plate connection is governed by the following items:

1. Shear resistance of bolt group

$$P_{v-shear} = 0.6.N_{bw}.f_b.A_b = 0.6 \times 5 \times 970 \times 314 = 913.7 \text{ kN} > 333 \text{ kN} \quad \text{OK.}$$

2. Bearing strength of bolts against the steel plate or beam web

$$F_{v-bearing} = N_{bw} \cdot \min \begin{cases} 2.5.\alpha.f_{sp}.d.t_{sp} \text{ (steel plate)} \\ 2.5.\alpha.f_{bw}.d.t_{bw} \text{ (beam web)} \end{cases}$$

$$= 5 \times 2.5 \times 0.5 \times 460 \times 20 \times 6.9 = 397 \text{ kN} > 333 \text{ kN} \quad \text{OK.}$$

3. Weld resistance in shear

For 8-mm fillet weld, the weld strength: $F_{w.Rd} = 1.2 \text{ kN/mm}$

Weld resistance in shear:

$$P_{v-weld} = n.D_{fp}.F_{w.Rd} = 2 \times 300 \times 1.2 = 720 \text{ kN} > 333 \text{ kN} \quad \text{OK.}$$

4. Block shear resistance of steel plate

$$P_{v-block} = 0.6.C.f_{sp}.A_{vbc} = 0.6 \times 1 \times 460 \times 10 \times (300 - 5 \times 22)$$

$$= 524 \text{ kN} > 333 \text{ kN} \quad \text{OK.}$$

5. Block shear resistance of beam web

$$P_{v-block} = 0.6.f_{bw}.A_{vb} = 0.6 \times 460 \times 6.9 \times (352 - 5 \times 22)$$

$$= 461 \text{ kN} > 333 \text{ kN} \quad \text{OK.}$$

Therefore, the shear capacity of single plate connection is: $P_v = 397 \text{ kN}$

A.4 Moment-Rotation Relationship of Hybrid Connection under Hogging Moment

A.4.1 Properties of concrete and steel

Based on the configuration of specimen, the composite beam consists of the 500 x 460 mm precast concrete beam and the 300 x 340 mm cast-in-place portion. The topping concrete is 75 mm thick with D7 ($\phi 7 @ 100$, both ways) mesh inside. The top main bars are 2T16 and the bottom bars are 2T13. Based on the expression given in BS8110: Part 1, the short-term elastic modulus of concrete can be obtained

as: $E_c = 5.5 \sqrt{\frac{f_{cu}}{r_m}}$. Based on the tests results, the properties of concrete and steel are

shown in the followings:

$$f_{cu} = 41 \quad \text{N/mm}^2$$

$$f_y = 529 \quad \text{N/mm}^2$$

$$f_{ym} = 593 \quad \text{N/mm}^2$$

$$f_{rp} = 0.58\sqrt{f_c} \quad \text{N/mm}^2$$

$$E_c = 5.5\sqrt{\frac{41}{1.5}} = 28.8 \times 10^3 \quad \text{N/mm}^2$$

$$E_s = 216 \times 10^3 \quad \text{N/mm}^2$$

$$A_{sm} = 231 \quad \text{mm}^2$$

$$A_{s,top} = 402 \quad \text{mm}^2$$

$$A_{s,bot} = 530 \quad \text{mm}^2$$

A.4.2 Cracking moment

As shown in Figure A.2, the cracking moment is calculated using the transformed area method, which is based on the proportionality between the elastic modulus of the concrete and steel.

The modular ratio: $n = \frac{E_s}{E_c} = 7.5$

$$\begin{aligned} A &= 1502 + 2613 + 19500 + 3445 + 52500 + 102000 + 230000 \\ &= 411560 \text{ mm}^2 \end{aligned}$$

The centroid of the transformed section is given by taking moments of the areas about the bottom edge of the section:

$$\begin{aligned} \bar{y} &= \frac{1502 \times 775 + 2613 \times 750 + 19500 \times 230 + 3445 \times 50 + 52500 \times 762.5 + 102000 \times 630 + 230000 \times 230}{411560} \\ &= 400.8 \text{ mm} \end{aligned}$$

The moment of inertia is given by:

$$I = 2.552 \times 10^{10} \text{ mm}^4$$

Cracking will occur when the modulus of rupture $f_{rp} = 2.86 \text{ N/mm}^2$ is reached in the top fibre.

$$M_{cr-hog} = \frac{f_{rp} \cdot I}{(h - y)} = \frac{2.86 \times 2.552 \times 10^{10}}{399.2} = 182.9 \text{ kN.m}$$

In this study, the rotation of the hybrid connection is measured at the section 100 mm away from the column face. Therefore, the predicted rotation and the stiffness at this section will be:

$$\text{Curvature: } \varphi = \frac{M_{cr-hog}}{E_c \cdot I} = \frac{182.9 \times 10^6}{28.8 \times 10^3 \times 2.552 \times 10^{10}} = 2.448 \times 10^{-7} \text{ rad/mm}$$

$$\text{Rotation: } \theta_{cr-hog} = 2.448 \times 10^{-7} \times \frac{600}{2} = 0.075 \text{ mrad}$$

$$\text{Stiffness: } K_{cr-hog} = \frac{M_{cr-hog}}{\theta_{cr-hog}} = \frac{182.9}{0.075} = 2438 \text{ kN.m/mrad}$$

A.4.3 Hogging moment capacity

The method of prediction for the hogging moment capacity is based on the stress block analysis. Therefore, the basis of the method relies on the horizontal force equilibrium between the components contributing to the tensile and compressive regions. The effective breadth of the topping concrete is taken as 1/8 of the average beam spans. The hogging moment capacity M_{hog} is the moment when the top rebars and mesh reinforcements within the effective breadth reach the yielding strength. The tension components and compression components are illustrated as follows:

1) Top Reinforcements

$$F_{r,top} = f_y \cdot A_{s,top} = 529 \times 402 = 212.6 \text{ kN}$$

$$F_{rsm} = f_y \cdot A_{sm} = 593 \times 231 = 137 \text{ kN}$$

2) Steel Connection

At the construction stage, the slip resistance is exceeded due to the construction load. The interacting force between HSFG and connected parts is assumed to be slip resistance F_{slip} and the bearing behaviour is not considered for conservative reasons.

$$F_{slip} = k_0 \cdot n_0 \cdot \mu \cdot F_P = 1.0 \times 1.0 \times 0.45 \times 170 = 76.5 \text{ kN}$$

3) Concrete in compression

For the force equilibrium,

$$F_{rs} + F_{rsm} + 5F_{slip} - F_c = 0$$

$$F_c = 0.67 f_{cu} \cdot A_c = 0.67 \times 41 \times 500 \times d_c$$

$$d_c = \frac{212.6 + 137 + 5 \times 76.5}{0.67 \times 41 \times 500} = 53.3 \text{ mm}$$

As shown in Figure A.3, the hogging moment capacity is determined by taking moment about the bottom edge of precast beam. The maximum moment can be calculated as:

$$\begin{aligned} M_{hog} &= 593 \times 231 \times 775 + 529 \times 402 \times 750 + 76500 \times (330 + 280 + 230 + \\ &\quad 180 + 130) - 0.67 \times 41 \times 500 \times 53.3 \times 53.3/2 \\ &= 334.1 \text{ kN.m} \end{aligned}$$

A.4.4 Rotation capacity under hogging moment

$$\Delta_r = 0.003 \left(\frac{h_c}{2} + 225 \right) = 0.003 \times \left(\frac{600}{2} + 225 \right) = 1.575 \text{ mm}$$

$$d_h = d_{top} - x_h = 750 - \frac{53.3}{0.9} = 690.8 \text{ mm}$$

$$\theta_{hog} = \frac{\Delta_r}{d_h} = \frac{1.575}{690.8} = 2.28 \text{ mrad}$$

The comparison between the test result, component-based model and tri-linear model is shown in Figure A.5. The predicted hogging moment capacity is lower than the test result which gives a safe prediction for design purpose.

A.5 Moment-Rotation Relationship of Hybrid Connection under Sagging Moment

$$A = 411560 \text{ mm}^2$$

$$\bar{y} = 800 - 400.8 = 399.2 \text{ mm}$$

$$I = 2.552 \times 10^{10} \text{ mm}^4$$

A.5.1 Cracking moment

Cracking will occur when the modulus of rupture $f_{rp} = 2.86 \text{ N/mm}^2$ is reached in the bottom fibre.

$$M_{cr-sag} = \frac{f_{rp} \cdot I}{(h - \bar{y})} = \frac{2.86 \times 2.552 \times 10^{10}}{400.8} = 182.1 \text{ kN.m}$$

$$\text{Curvature: } \phi = \frac{M_{cr-sag}}{E_c \cdot I} = \frac{182.1 \times 10^6}{28.8 \times 10^3 \times 2.552 \times 10^{10}} = 2.48 \times 10^{-7} \text{ rad/mm}$$

$$\text{Rotation: } \theta_{cr-sag} = 2.448 \times 10^{-7} \times \frac{600}{2} = 0.075 \text{ mrad}$$

$$\text{Stiffness: } K_{cr-sag} = \frac{M_{cr-sag}}{\theta_{cr-sag}} = \frac{182.1}{0.075} = 2428 \text{ kN.m/mrad}$$

A.5.2 Sagging moment capacity

1) Bottom Reinforcements

$$F_{r,bot} = f_y \cdot A_{s,bot} = 529 \times 530 = 280.3 \text{ kN}$$

2) Steel Connection

$$F_{slip} = k_0 \cdot n_0 \cdot \mu \cdot F_p = 1.0 \times 1 \times 0.45 \times 170 = 76.5 \text{ kN}$$

3) Concrete in compression

For the force equilibrium,

$$F_{r,bot} + 5F_{slip} - F_c = 0$$

$$F_c = 0.67 f_{cu} \cdot A_c = 0.67 \times 41 \times 1000 \times d_c$$

$$d_c = 24.1 \text{ mm}$$

As shown in Figure A.4, the sagging moment capacity is determined by taking moment about the compression edge of the topping concrete. The maximum moment can be calculated as:

$$\begin{aligned} M_{sag} &= 529 \times 530 \times 750 + 76500 \times (670 + 620 + 570 + 520 + 470) - 0.67 \times 41 \times \\ &\quad 1000 \times 24.1 \times 24.1/2 \\ &= 420.3 \text{ kN.m} \end{aligned}$$

A.5.3 Rotation capacity under sagging moment

$$\Delta_r = 0.003 \left(\frac{h_c}{2} + 225 \right) = 0.003 \times \left(\frac{600}{2} + 225 \right) = 1.575 \text{ mm}$$

$$d_s = d_{bottom} - x_s = 750 - \frac{24.1}{0.9} = 723.3 \text{ mm}$$

$$\theta_{sag} = \frac{\Delta_r}{d_s} = \frac{1.575}{723.3} = 2.18 \text{ mrad}$$

The comparison between the test result, component-based model and tri-linear model is shown in Figure A.6. The predicted sagging moment capacity is lower than the test results which gives a safe prediction for design purpose.

A.6 Shear deformation of composite beam

The modulus of elasticity in shear can be given as:

$$G = \frac{E_c}{2(1+\mu)} = 0.38 \text{ N/mm}^2$$

The shear stiffness of an uncracked beam of unit length can be given as:

$$K_v' = \frac{Gbd}{f} = 2.19 \times 10^9 \text{ N/mm}^2$$

The magnitude of the shear force applied to the composite beam at unit length is:

$$V = 6.3 + 7.5 = 13.8 \text{ N/mm}$$

$$\text{Shear deformation at the beam end} = \frac{13.8 \times (6.65/2) \times 1000}{2.19 \times 10^9} = 2.1 \times 10^{-5} \text{ mm}$$

The shear deformation is negligible compared to the deformation under bending.

A.7 Horizontal loads at construction stage and service stage

A.7.1 Horizontal loads at construction stage

In BS8110, the notional load is taken to be 1.5% of the characteristic dead weight of the structure.

$$\text{Notional load} = 1.5\% \times (6.3 \times 8 \times 6.65 \times 2) = 10.1 \text{ kN}$$

The wind loads are calculated using CP3: Chapter 5: Part 2:

$$\text{Basic wind speed: } V_s = 45 \text{ m/s}$$

$$\text{Topography factor: } S_1 = 1.0$$

$$\text{Statistical factor: } S_3 = 1.0$$

Ground roughness 3

Building size Class B

$$\text{Force coefficient: } C_f = 1.13$$

Table A.1 Calculation of wind load at construction stage

Location	H (m)	S_2	$V_s = S_2 V$	$q = 0.613.V_s^2 / 10^3$	Wind load (kN)
4 th	9.6	0.78	35.1	0.76	15.6
3 rd	8.0	0.72	32.5	0.65	26.6
2 nd	4.8	0.63	28.3	0.5	20.5

The wind load is greater than notional load, therefore, the wind load will be used as horizontal load for frame analysis at construction stage.

A.7.2 Horizontal loads at service stage

The wind loads are calculated using CP3: Chapter 5: Part 2:

Table A.2 Calculation of wind load at service stage

Location	H (m)	S_2	$V_s = S_2 V$	$q = 0.613.V_s^2 / 10^3$	Wind load (kN)
5 th	13.7	0.8	36	0.79	16.0
4 th	12.1	0.78	35.1	0.76	30.8
3 rd	8.9	0.72	32.5	0.65	26.3
2 nd	5.7	0.63	28.3	0.5	23.5

The wind load is greater than notional load, therefore, the wind load will be used as horizontal load for frame analysis at service stage.

A.8 Capacity check of universal column at construction stage

Section properties for universal column 356 x 171 UB 67 are:

$$A = 85.4 \text{ cm}^2, M_{cx} = 333 \text{ kN.m}, p_y = 275 \text{ N/mm}^2$$

Vertical load on column: $N = 1159.6 \text{ kN}$

Moment at column base: $M = 143.3 \text{ kN.m}$

Interaction expression:
$$\frac{1159.6 \times 10^3}{85.4 \times 10^2 \times 275} + \frac{143.3 \times 10^6}{333 \times 10^6} = 0.49 + 0.40 = 0.89 < 1$$

The section is satisfactory with respect to local capacity.

Figure A.2 Section for the calculation of cracking moment
(a) Original section; (b) Transformed section

Figure A.3 Force distribution of hybrid connection under hogging moment

Figure A.4 Force distribution of hybrid connection under sagging moment

Figure A.5 Tri-linear moment-rotation relationship under hogging moment

Figure A.6 Tri-linear moment-rotation relationship under sagging moment

Appendix B: Material Properties

B.1 Concrete

The nominal concrete grade specified for test series was $f_{cu} = 35 \text{ N/mm}^2$. For each specimen of the test series, the cubes (150 x 150 x 150 mm) and cylinders (ϕ 150 x 300 mm) were made at the same time to monitor the concrete strength. The cubes and cylinders were tested at 7th, 28th day and the test day with three samples each as a batch each time. The summary of concrete strength development was shown in Table B.1.

B.2 Reinforcing Bars

The reinforcement bars used in the tests were high yield strength bars. The welded steel mesh used in the topping concrete was D7 at spacing of 100 mm. The longitudinal reinforcements in the cast-in-place beam were 13 mm, 16 mm and 25 mm diameter bars corresponding to the different reinforcement ratios and 10 mm bars were used for shear links. Three 600 mm long test bars were taken from the same batch of steel bars used. The steel bars were tested using a Universal Testing Machine of 3000 kN capacity (UTM-3000 kN). The tensile test results are given in the Table B.2 to Table B.6.

B.3 Structural Steel

Grade 43 steel components were used for the test specimens. These components consist of partly embedded universal beam 356 x 171 UB 45, partly embedded universal column 356 x 171 UB 67 and steel plate 130 x 300 x 10 mm. Three numbers of steel specimen coupons were cut from the parent material for each

component to determine the actual mechanical properties. The dimensions and testing of the steel coupon was carried out in accordance to Annex C of the standard BS EN 10 002-1: 1990. The tensile test results are given in the Table B.7 to Table B.9.

B.4 HSFG Bolts

All bolts used for the single plate steel connection were higher grade 20 mm High Strength Friction Grip (HSFG) bolts. The purpose of using HSFG bolts is to prevent the slip between the bolts and connected parts. The strength and properties of bolts were summarised in Table B.10. The installation of HSFG bolts was closely monitored to ensure that the proper pretension was applied to each bolt. To achieve this, a torque-pretension calibration was performed.

Several methods of pre-tensioning are available. They include the turn-of-the-nut method, calibrated wrench method, and use of direct tension indicators or tension control bolts. In this study, a load indicator washer (LW2588-50K) with capacity of 220 kN was used to determine the pretension of the HSFG bolts (Figure B.1) and the hand torque wrench with capacity of 560 N.m was used to tighten the HSFG bolts. Three HSFG bolts were tested to obtain the torque-pretension relationship (Figure B.2). As shown in Figure B.3, the torque-pretension relationship was found to be linear and can be expressed in a simple form:

$$N = \frac{T}{d_b \times K} \quad \text{Eq. B-1}$$

where N : The bolt pretension (kN);

T : The applied torque (N.m);

d_b : The bolt diameter (mm);

K : The torque coefficient.

The average value of K is taken as 0.112. During the installation, all HSFG bolts were pre-tensioned to 170 kN by applying a torque of 380 N.m on each bolt.

B.5 Shear Studs

In order to prevent slip between embedded universal column and precast column, 19 mm headed shear studs with 100 mm length were used in the experiments. The ultimate tensile strength (nominal strength) of shear stud is $f_u = 450 \text{ N/mm}^2$. Along both sides of the steel universal column, they are placed uniformly with two shear studs per trough at a uniform spacing. According to clause 6.1.2 (2) (Eurocode 4, 1992), these shear studs are classified as ductile connectors. Shear studs are designed to ensure full shear connection and effectively transfer the loads from steel column to precast column.

B.6 Hollow Core Slabs

The design of hollow core slabs (265 mm thick) was based on class 2 prestressed concrete structure with minimum 2 hours fire rating. The hollow core slabs were cast with C50 concrete. Each unit (1.2 m nominal width) was designed as simply supported with nominal 100 mm seating at the support. Resultant stresses were checked at serviceability and at prestress transfer. Design of the slab was carried out by the specialist supplier.

Table B.1 Concrete Cube and Cylinder Strengths

Specimen	Test Days	Cube Strength (N/mm ²)				Cylinder Strength (N/mm ²)			
		No.1	No.2	No.3	Average	No.1	No.2	No.3	Average
	7	25.69	25.32	25.53	25.51	18.34	19.31	18.65	18.77
	28	35.11	33.24	35.21	34.52	27.05	28.63	27.80	27.83
HCI	Test day	36.03	38.72	35.20	36.65	31.12	27.45	30.50	29.69
HC2	Test day	39.06	35.42	37.51	37.33	30.95	29.83	30.26	30.35
HC3	Test day	42.09	44.35	43.07	43.17	35.47	34.26	33.53	34.42
HCS1	Test day	42.17	40.01	41.07	41.08	33.75	31.81	33.62	33.06
HCS2	Test day	38.89	43.64	42.22	41.58	32.33	34.61	33.75	33.56
HCS3	Test day	40.90	43.06	40.13	41.36	31.61	32.02	34.31	32.64
HCS4	Test day	40.93	42.05	43.76	42.23	34.21	33.56	30.47	32.75
HCS5	Test day	44.30	42.15	43.21	43.22	33.47	34.08	35.53	34.36
HCS6	Test day	42.51	44.05	42.83	43.02	34.33	32.89	35.72	34.31

* Test day refers to the testing day of each specimen.

Table B.2 Tensile Test Results of T10 Steel Specimen Bars

Specimen	E-value (kN/mm ²)	Yield-stress (N/mm ²)	Ultimate tensile strength (N/mm ²)
Test Bar 1	212.7	596.2	643.6
Test Bar 2	220.3	592.5	640.8
Test Bar 3	223.5	594.9	658.5
Average	218.8	594.5	647.6

Table B.3 Tensile Test Results of T13 Steel Specimen Bars

Specimen	E-value (kN/mm ²)	Yield-stress (N/mm ²)	Ultimate tensile strength (N/mm ²)
Test Bar 1	215.6	535.0	612.9
Test Bar 2	210.3	546.3	616.3
Test Bar 3	220.6	540.6	610.6
Average	215.5	540.6	613.3

Table B.4 Tensile Test Results of T16 Steel Specimen Bars

Specimen	E-value (kN/mm ²)	Yield-stress (N/mm ²)	Ultimate tensile strength (N/mm ²)
Test Bar 1	217.2	544.8	596.1
Test Bar 2	210.5	523.9	596.0
Test Bar 3	220.3	517.9	597.0
Average	216.0	528.9	596.3

Table B.5 Tensile Test Results of T25 Steel Specimen Bars

Specimen	E-value (kN/mm ²)	Yield-stress (N/mm ²)	Ultimate tensile strength (N/mm ²)
Test Bar 1	211.1	505.0	589.6
Test Bar 2	220.7	506.0	587.7
Test Bar 3	215.9	521.7	633.2
Average	215.9	510.9	603.5

Table B.6 Tensile Test Results of D7 Steel Mesh

Specimen	E-value (kN/mm ²)	Yield-stress (N/mm ²)	Ultimate tensile strength (N/mm ²)
Test Bar 1	201.0	592.1	640.8
Test Bar 2	197.5	593.0	641.1
Average	199.3	592.6	642.0

Table B.7 Tensile Test Results of Steel Coupons (356 x 171 UB 45, Beam)

Specimen	E-value (kN/mm ²)	Yield-stress (N/mm ²)	Ultimate tensile strength (N/mm ²)
Web 1	201.5	413.5	569.6
Web 2	201.3	477.8	578.8
Web 3	201.4	478.1	590.0
Average	201.4	456.5	579.5

Table B.8 Tensile Test Results of Steel Coupons (356 x 171 UB 67, Column)

Specimen	E-value (kN/mm ²)	Yield-stress (N/mm ²)	Ultimate tensile strength (N/mm ²)
Flange 1	215.8	406.3	515.9
Flange 2	212	396.1	511.5
Flange 3	210.6	407.6	517.2
Average	212.8	403.3	514.8

Table B.9 Tensile Test Results of Steel Coupons (Steel plate)

Specimen	E-value (kN/mm ²)	Yield-stress (N/mm ²)	Ultimate tensile strength (N/mm ²)
Plate 1	224.2	307.0	473.5
Plate 2	218.1	337.6	478.2
Average	221.2	322.3	475.8

Table B.10 Higher Grade 20mm High Strength Friction Grip Bolts

HSFG Bolts	Diameter of bolt (mm)	Proof Load of Bolt (kN)	Shank Tension (kN)		Tensile Capacity (kN)
			Minimum (0.85 x Proof Load)	Maximum (1.15 x Proof Load)	
F10T	20	190.4	161.8	219.0	245

Figure B.1 Load indicator washer (LW2588-50K)

Figure B.2 Torque-pretension test of HSF bolts

Figure B.3 Torque-pretension relationship of HSFG bolts

Appendix C: Detailed Configurations of Specimens

C.1 Test Series A – BS1, BS2, BS3

Figure C.1 Specimens with bare steel connections under monotonic loading (BS1 & BS2)

Figure C.2 Specimen with bare steel connection under cyclic loading (BS3)

C.2 Test Series B – HC1, HC2, HC3

Figure C.3 Specimens with partial hybrid connections under monotonic loading (HC1 & HC2)

Figure C.4 Details of specimens with partial hybrid connections (HC1 & HC2)

Figure C.5 Section A-A & A0-A0 (HC1 & HC2)

Figure C.6 Specimen with partial hybrid connections under cyclic loading (HC3)

Figure C.7 Details of specimen with partial hybrid connections (HC3)

Figure C.8 End view, Section B-B & C-C (HC3)

C.3 Test Series C – HCS1, HCS2, HCS3, HCS4, HCS5, HCS6

Figure C.9 Specimens of complete hybrid connections under cyclic loading (HCS1, HCS2, HCS3, HCS4)

Figure C.10 Details of specimens with complete hybrid connection (HCS1, HCS2, HCS3, HCS4)

Figure C.11 Section A-A & A0 – A0 (HCS1, HCS2, HCS3, HCS4)

Figure C.12 Specimens with complete hybrid connections under cyclic loading (HCS5, HCS6)

Figure C.13 Details of specimens with complete hybrid connections (HCS5, HCS6)

Figure C.14 End view, Section B1-B1 & C1 - C1 (HCS5, HCS6)

Appendix D: Fabrication of Specimens

For the test series A of bare steel connection specimens, the steel connection was assembled in the NTU laboratory. The steel plates were first welded to the steel column, and then the steel beams were connected to the steel plate using the HSFG bolts. All HSFG bolts were pre-tensioned to the 170 kN by applying a torque of 380 N.m using a hand torque wrench.

For the test series B, and C, all the specimens were fabricated and assembled in the factory. The fabrication procedure adopted in the experiments is to simulate the actual construction sequence of hybrid connection structures in practice.

D.1 Actual Construction Sequence

- 1) In the actual construction sequence, the precast beams with embedded steel beams would be cast in the factory and then transported to the site.
- 2) Subsequently, these precast beams would be assembled on site through the single plate connections. Once assembled, the system could immediately withstand working loading within the serviceability limit. The hollow core slabs could then be placed on the edge of the precast beam. The top rebars of the cast-in-place beam as well as the mesh reinforcements in the topping concrete could be placed at this stage.
- 3) Finally, the exposed steel connection, the joints between the precast components, cast-in-place beam and topping concrete would be cast together. Therefore, the hybrid connection would be formed.

D.2 Fabrication Process for Series B and Series C

- 1) As shown in Figure D.1, at first stage, the universal steel column welded with shear studs was cast into the column. The universal beam and the rebars were cast into the precast beam. Two bottom rebars were protruding outside of the precast beam. One bottom rebar at each side was bent up for easy installation. The single plates were welded to the centre of column flanges.
- 2) It will be left to cure for one week after the casting of precast beams and precast columns. At stage two, precast beams were connected to the steel column via single plate connections using HSFG bolts. All HSFG bolts were pre-tensioned to 170 kN by applying a torque of 380 N.m on each bolt. As shown in Figure D.2, hollow core slabs were placed on the edge of the precast beam. The top rebars as well as D7 mesh were placed in position according to the construction drawings. Bottom rebars were straightened and lapped to each other at the beam-column joint.
- 3) As shown in Figure D.3, finally the hybrid connection portion was cast together with the cast-in-place beam as well as topping concrete. The cast-in-place beam and topping concrete were cast over the precast beam and continuous across the beam-column joint. A cold joint between precast beam and precast column was therefore formed through different casting sequence.

Figure D.1 Fabrication process – stage 1

Figure D.2 Fabrication process – stage 2

Figure D.3 Fabrication process – stage 3

Appendix E: Instrumentation

In order to record sufficient experimental data from each test, strains and rotations were measured at several sections of specimens. Strain gauges were used to measure the axial deformation at the beam flange, beam web and reinforcing bars. Strain rosettes were also used to measure the 2-dimensional strain condition. The layout of the internal instrumentation (strain gauge and rosette) and external instrumentation (LVDT) for test series A, B and C are shown in Figure E.1~E.14. For all the specimens, 5 mm long resistance strain gauges were used to measure the strains in rebars, mesh reinforcements and structural steels.

E.1 Series A

- **Internal Instrumentation (Figure E.1 & E.3)**

- 1) For specimen BS1 & BS2, three strain gauges were placed at one section along the length of the steel beam tension flange, beam web and compression flange to determine the flexural load distribution. The section was 150 away from the column flange.
- 2) For specimen BS3, strain gauges were placed at one section along the length of the steel beam. In addition, strain rosette was placed in the centre of steel column panel to determine the state of stress in this zone.

- **External Instrumentation (Figure E.2 & E.4)**

- 1) For specimen BS1 & BS2, LVDTs with 200 mm travel were placed along the length of the composite beams to determine the load-deflection profile of the steel beams.

- 2) Two pairs of LVDTs with 50 mm travel were used to obtain the average rotation of single plate connection. Each pair of LVDTs was mounted on the top and bottom of the beam flanges.

As shown in Figure E.2, a pair of LVDTs measured the connection rotation on each side of the column. The connection rotation was obtained by averaging the sum of the top left and right LVDT displacements and subtracting this value from the average of the sum of the bottom left and right LVDT displacements. This value was divided by the distance between two LVDTs to obtain the connection rotation value.

$$\theta = \frac{\delta_t - \delta_b}{h} \quad \text{Eq. E-1}$$

where θ : Connection rotation;
 δ_t : Top displacements measured by LVDT;
 δ_b : Bottom displacements measured by LVDT;
 h : Distance between top and bottom LVDT.

E.2 Series B

The internal and external instrumentation for HC1 and HC2 were shown in Figure E.5 & E.6 respectively and the internal and external instrumentation for HC3 was shown in Figure E.10 & E.11.

- **Internal Instrumentation**

- 1) Strain gauges were placed on rebars along the length of the bars. The resistance and insulation of the gauges were checked before and after waterproofing coating to ensure proper operation would occur after concreting. The gauges were intended to monitor the strains in the reinforcement in the composite beam to identify the extent of yielding.
- 2) Strain gauges were also placed at the steel beam top flange, beam web and beam bottom flange to determine the force taken by the steel component and flexural load distribution.

- **External Instrumentation**

- 1) The external instrumentation of HC1 & HC2 was similar to test series A;
- 2) The external instrumentation of HC3 was similar to BS3 in series A. Two LVDTs were installed to measure the displacement at the top and bottom of the column under cyclic loading.

E.3 Series C

The internal and external instrumentation of HCS1, HCS2, HCS3 and HCS4 were similar to those for HC1 and HC2. The internal instrumentation of HCS5 & HCS6 was similar to that of HC3. In addition, 5 mm strain gauges were placed on the welded mesh in the topping concrete for all specimens. The detailed arrangements of series C were shown in Figure E.7 ~ E.9 and Figure E.12 ~ E.14.

Figure E.1 Internal instrumentation – BS1 & BS2

Figure E.2 External instrumentation – BS1 & BS2

Figure E.3 Internal instrumentation – BS3

Figure E.4 External Instrumentation – BS3

Figure E.5 Internal instrumentation – HCl & HCl₂

Figure E.6 External instrumentation – HC1 & HC2

Figure E.7 Internal instrumentation - HCS1, HCS2, HCS3, HCS4

Figure E.8 Locations of strain gauges on top rebars and mesh reinforcements (HCS1, HCS2, HCS3, HCS4)

Figure E.9 External instrumentation - HCS1, HCS2, HCS3, HCS4

Figure E.10 Internal instrumentation – HC3

Figure E.11 External instrumentation – HC3

Figure E.12 Internal instrumentation – HCS5, HCS6

Figure E.13 Locations of strain gauges on topping concrete and cast-in-place beam (HCS5, HCS6)

Figure E.14 External instrumentation – HCS5 & HCS6

Appendix F: Computer Programs

The computer programs given in this Appendix were written in QBASIC and based on the mechanical models proposed in this thesis to predict the behaviour of single plate and hybrid connection. Emphasis was placed on the derivation of predicted $M - \theta$ relationship rather than transportability of the programs. Therefore, only programs to predict the single plate and hybrid connection under monotonic loading were given. The computer outputs were tabulated to verify the predictions against the test results.

Single Plate Connection - Single plate connection under monotonic loading

REM 'Single Plate Connection under Monotonic Loading' Unit: N, mm, rad

REM " SLIP MECHANISM

Fslip = 1! * .45 * 170000

Kslip = Fslip / .193

Dslip = .193

Dbearing = 1!

REM " BEARING MECHANISM

Kbr = 120 * tweb * fuweb * (db / 25.4) ^ .8

Kb = 32 * Es * tweb * (50 / db - .5) ^ 3

Kv = 6.67 * G * tweb * (50 / db - .5)

Kbi = 1 / (1 / Kbr + 1 / Kb + 1 / Kv)

Fbearing = 2.4 * fuweb * db * tweb

n = 5.73 * (Fbearing / Kbi) / (1 + .439 * (Fbearing / Kbi) * tweb ^ 2)

Disi1 = Fslip / (Kbi * (1 - (Fslip / Fbearing) ^ n))

Disi2 = Fslip / (Kbi * (1 - (Fslip / Fbearing) ^ n))

DR = .0001

DO UNTIL ROTATION > .06

ROTATION = ROTATION + DR

D1 = ROTATION * (hp / 2)

D2 = ROTATION * (hp / 4)

IF D1 < Dslip THEN

F1 = Kslip * D1

ELSEIF D1 < Dbearing + Disi1 THEN

F1 = Fslip

END IF

```

IF D2 < Dslip THEN
F2 = Kslip * D2
ELSEIF D2 < Dbearing + Disi2 THEN
F2 = Fslip
END IF
IF D1 > Dbearing + Disi1 THEN

Dis1 = D1 - Dbearing
DO UNTIL ABS(Dcon1 - Dis1) < .01
Dcon1 = F1 / (Kbi * (1 - (F1 / Fbearing) ^ n))
IF ABS(Dcon1 - Dis1) > .01 THEN
F1 = F1 + 10
Dcon1 = F1 / (Kbi * (1 - (F1 / Fbearing) ^ n))
END IF
LOOP
END IF
IF D2 > Dbearing + Disi2 THEN
Dis2 = D2 - Dbearing
DO UNTIL ABS(Dcon2 - Dis2) < .01
Dcon2 = F2 / (Kbi * (1 - (F2 / Fbearing) ^ n))
IF ABS(Dcon2 - Dis2) > .01 THEN
F2 = F2 + 10
Dcon2 = F2 / (Kbi * (1 - (F2 / Fbearing) ^ n))
END IF
LOOP
END IF

MOMENT = (F1 * (hp / 2) + F2 * (hp / 4)) * 2

PRINT ROTATION, MOMENT, Disi1, Dis1, D1, F1
WRITE #1, ROTATION * 1000, MOMENT / 1000000
LOOP

END

```

Hybrid Connection - Hybrid connection under monotonic loading

```

REM 'Hybrid Connection with Hollow Core Slab under Monotonic Hogging Moment
Unit: N, mm, radians

y = y0

DROTATION = .00005
FOR j = 1 TO 100
ROTATION = ROTATION + DROTATION
GOSUB MESH
GOSUB TOPBAR
GOSUB BOTTOMBAR
GOSUB STEELCONNECTION

```


GOSUB CONCRETESECTION

FORCE = FRMESH + FRTOP + FHSFG1 + FHSFG2 + FHSFG3 + FHSFG4 + FHSFG5 + FRBOT
+ FC

IF ABS(FORCE) > 1000 THEN
DO UNTIL ABS(FORCE) < 10000

y = y - .05

FORCE = FRMESH + FRTOP + FHSFG1 + FHSFG2 + FHSFG3 + FHSFG4 + FHSFG5 + FRBOT
+ FC

PRINT y, ROTATION, MC

LOOP

END IF

MOMENT = FRMESH * (h - 30) + FRTOP * d + FHSFG1 * DHSFG1 + FHSFG2 * DHSFG2 +
FHSFG3 * DHSFG3 + FHSFG4 * DHSFG4 + FHSFG5 * DHSFG5 + FRBOT * DBOTTOM + MC

M1 = FRMESH * (h - 30)

M2 = FRTOP * d

M3 = FHSFG1 * DHSFG1

M4 = FHSFG2 * DHSFG2

M5 = FHSFG3 * DHSFG3

M6 = FHSFG4 * DHSFG4

M7 = FHSFG5 * DHSFG5

M8 = FRBOT * DBOTTOM

M9 = FHSFG1 * DHSFG1 + FHSFG2 * DHSFG2 + FHSFG3 * DHSFG3 + FHSFG4 * DHSFG4 +
FHSFG5 * DHSFG5

M10 = MC

F1 = FRMESH

F2 = FRTOP

F3 = FHSFG1

F4 = FHSFG2

F5 = FHSFG3

F6 = FHSFG4

F7 = FHSFG5

F8 = FRBOT

F9 = FHSFG1 + FHSFG2 + FHSFG3 + FHSFG4 + FHSFG5

F10 = FC

WRITE #1, ROTATION * 1000, MOMENT / 1000000

NEXT j

END

MESH:

'Positive tension: Negative compression

DRMESH = (h - 30 - y) * ROTATION

FYMESH = fym * Asmesh

DYMESH = fym * Ls / Es

FRMESH = DRMESH * Asmesh * Es / Ls

IF ABS(DRMESH) > DYMESH THEN

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```
FRMESH = FYMESH + (DRMESH - DYMESH) * Asmesh * Es1 / Ls
END IF
```

```
RETURN
```

```
TOPBAR:                                     'Positive tension: Negative compression
```

```
DRTOP = (d - y) * ROTATION
FYTOP = fyT16 * Astop
DYTOP = fyT16 * Ls / Es
FRTOP = DRTOP * Astop * Es / Ls
IF ABS(DRTOP) > DYTOP THEN
FRTOP = FYTOP + (DRTOP - DYTOP) * Astop * Es1 / Ls
END IF
```

```
RETURN
```

```
BOTTOMBAR:                                'Positive tension: Negative compression
```

```
DRBOT = (DBOTTOM - y) * ROTATION
FYBOT = fyT13 * Asbot
DYBOT = fyT13 * Ls / Es
FRBOT = DRBOT * Asbot * Es / Ls
IF ABS(DRBOT) > DYBOT THEN
FRBOT = -FYBOT + (DRBOT + DYBOT) * Asbot * Es1 / Ls
END IF
```

```
RETURN
```

```
CONCRETESECTION:                          'Mander concrete model
```

```
Strainco = -.002                           'Unconfined concrete strain
Stressco = -fco
Stressl = 0                                'Lateral confinement
Stresscc = Stressco * (-1.254 + 2.254 * (1 + 7.94 * ABS(Stressl / Stressco)) ^ .5 - 2 * ABS(Stressl / Stressco))
Straincc = Strainco * (1 + 5 * (ABS(Stresscc / Stressco) - 1))
Esec = ABS(Stresscc / Straincc)
r = Ec / (Ec - Esec)
```

```
FC = 0
MC = 0
FOR i = 1 TO number
yi = y1 + (i - 1) * h / number
IF yi - y < 0 THEN
Strainc = (yi - y) * ROTATION / Lc
x = ABS(Strainc / Straincc)
Stressc = ABS(Stresscc * x * r / (r - 1 + x ^ r))
Econ = ABS(Stressc / Strainc)
KC1 = Econ * b3 * (h / number) / Lc
```

```

KC2 = Econ * b1 * (h / number) / Lc
KC3 = Econ * b2 * (h / number) / Lc
IF yi > h - (h3 + h4) THEN
FCi = KC1 * (yi - y) * ROTATION
ELSEIF yi > h2 THEN
FCi = KC2 * (yi - y) * ROTATION
ELSE
FCi = KC3 * (yi - y) * ROTATION
END IF

ELSE

Stresst0 = .1 * fco
Straint0 = .1 * fco / Ec
Straint = (yi - y) * ROTATION / Lc
Stresst = Straint * Ec
Econ = Ec
IF Straint > Straint0 THEN
Stresst = .33 * Stresst0 - .33 * Stresst0 * (Straint - Straint0) / (25 * Straint0)
Econ = Stresst / Straint
END IF

KC1 = Econ * b3 * (h / number) / Lc
KC2 = Econ * b1 * (h / number) / Lc
KC3 = Econ * b2 * (h / number) / Lc

IF yi > h - (h3 + h4) THEN
FCi = KC1 * (yi - y) * ROTATION
ELSEIF yi > h2 THEN
FCi = KC2 * (yi - y) * ROTATION
ELSE
FCi = KC3 * (yi - y) * ROTATION
END IF

END IF

FC = FC + FCi
MCi = FCi * yi
MC = MC + MCi
IF MC < 0 THEN
MC = 0
END IF
NEXT i

RETURN

STEELCONNECTION:                                'Positive tension: Negative compression

REM `SLIP MECHANISM

Fslip = 1! * .45 * 170000
Kslip = Fslip / .193

```

Dslip = .193
Dbearing = 1!

REM `BEARING MECHANISM

$G = E_s / (2 * (1 + .3))$
 $K_{br} = 120 * t_{web} * f_{uweb} * (d_b / 25.4) ^ .8$
 $K_b = 32 * E_s * t_{web} * (50 / d_b - .5) ^ 3$
 $K_v = 6.67 * G * t_{web} * (50 / d_b - .5)$
 $K_{bi} = 1 / (1 / K_{br} + 1 / K_b + 1 / K_v)$
 $F_{bearing} = 2.4 * f_{uweb} * d_b * t_{web}$
 $n = 5.73 * (F_{bearing} / K_{bi}) / (1 + .439 * (F_{bearing} / K_{bi}) * t_{web} ^ 2)$

DRHSFG1 = (DHSFG1 - y) * ROTATION

IF DRHSFG1 < Dslip THEN
 FHSFG1 = Kslip * DRHSFG1
 ELSEIF DRHSFG1 < Dbearing THEN
 FHSFG1 = Fslip
 END IF

IF DRHSFG1 > Dbearing THEN
 Dis1 = Fslip / (Kbi * (1 - (Fslip / Fbearing) ^ n))
 Dis1 = DRHSFG1 - Dbearing + Dis1
 DO UNTIL ABS(Dcon1 - Dis1) < .01
 Dcon1 = FHSFG1 / (Kbi * (1 - (FHSFG1 / Fbearing) ^ n))
 IF ABS(Dcon1 - Dis1) > .01 THEN
 FHSFG1 = FHSFG1 + 10
 Dcon1 = FHSFG1 / (Kbi * (1 - (FHSFG1 / Fbearing) ^ n))
 END IF
 LOOP
 END IF

DRHSFG2 = (DHSFG2 - y) * ROTATION

IF DRHSFG2 < Dslip THEN
 FHSFG2 = Kslip * DRHSFG2
 ELSEIF DRHSFG2 < Dbearing THEN
 FHSFG2 = Fslip
 END IF
 IF DRHSFG2 > Dbearing THEN
 Dis2 = Fslip / (Kbi * (1 - (Fslip / Fbearing) ^ n))
 Dis2 = DRHSFG2 - Dbearing + Dis2
 DO UNTIL ABS(Dcon2 - Dis2) < .01
 Dcon2 = FHSFG2 / (Kbi * (1 - (FHSFG2 / Fbearing) ^ n))
 IF ABS(Dcon2 - Dis2) > .01 THEN
 FHSFG2 = FHSFG2 + 10
 Dcon2 = FHSFG2 / (Kbi * (2 - (FHSFG2 / Fbearing) ^ n))
 END IF
 LOOP
 END IF

DRHSFG3 = (DHSFG3 - y) * ROTATION

IF DRHSFG3 < Dslip THEN
 FHSFG3 = Kslip * DRHSFG3
 ELSEIF DRHSFG3 < Dbearing THEN
 FHSFG3 = Fslip
 END IF
 IF DRHSFG3 > Dbearing THEN

Disi3 = Fslip / (Kbi * (1 - (Fslip / Fbearing) ^ n))
 Dis3 = DRHSFG3 - Dbearing + Disi3
 DO UNTIL ABS(Dcon3 - Dis3) < .01
 Dcon3 = FHSFG3 / (Kbi * (1 - (FHSFG3 / Fbearing) ^ n))
 IF ABS(Dcon3 - Dis3) > .01 THEN
 FHSFG3 = FHSFG3 + 10
 Dcon3 = FHSFG3 / (Kbi * (3 - (FHSFG3 / Fbearing) ^ n))
 END IF
 LOOP
 END IF

DRHSFG4 = (DHSFG4 - y) * ROTATION
 IF DRHSFG4 < Dslip THEN
 FHSFG4 = Kslip * DRHSFG4
 ELSEIF DRHSFG4 < Dbearing THEN
 FHSFG4 = Fslip
 END IF

IF DRHSFG4 > Dbearing THEN
 Disi4 = Fslip / (Kbi * (1 - (Fslip / Fbearing) ^ n))
 Dis4 = DRHSFG4 - Dbearing + Disi4
 DO UNTIL ABS(Dcon4 - Dis4) < .01
 Dcon4 = FHSFG4 / (Kbi * (1 - (FHSFG4 / Fbearing) ^ n))
 IF ABS(Dcon4 - Dis4) > .01 THEN
 FHSFG4 = FHSFG4 + 10
 Dcon4 = FHSFG4 / (Kbi * (4 - (FHSFG4 / Fbearing) ^ n))
 END IF
 LOOP
 END IF

DRHSFG5 = (DHSFG5 - y) * ROTATION
 IF DRHSFG5 < Dslip THEN
 FHSFG5 = Kslip * DRHSFG5
 ELSEIF DRHSFG5 < Dbearing THEN
 FHSFG5 = Fslip
 END IF
 IF DRHSFG5 > Dbearing THEN

Disi5 = Fslip / (Kbi * (1 - (Fslip / Fbearing) ^ n))
 Dis5 = DRHSFG5 - Dbearing + Disi5
 DO UNTIL ABS(Dcon5 - Dis5) < .01
 Dcon5 = FHSFG5 / (Kbi * (1 - (FHSFG5 / Fbearing) ^ n))
 IF ABS(Dcon5 - Dis5) > .01 THEN

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```

FHSFG5 = FHSFG5 + 10
Dcon5 = FHSFG5 / (Kbi * (5 - (FHSFG5 / Fbearing) ^ n))
END IF
LOOP
END IF

RETURN

```

Table F.1 Values of Predicted Moment-Rotation Relationship (BS1)

θ (mrad)	M (kN.m)	θ (mrad)	M (kN.m)	θ (mrad)	M (kN.m)	θ (mrad)	M (kN.m)	θ (mrad)	M (kN.m)	θ (mrad)	M (kN.m)
0.5	5.0	10.5	23.0	20.5	27.0	30.5	31.8	40.5	36.7	50.5	40.1
1.0	9.9	11.0	23.0	21.0	27.3	31.0	32.1	41.0	36.9	51.0	40.2
1.5	14.9	11.5	23.0	21.5	27.6	31.5	32.4	41.5	37.1	51.5	40.4
2.0	19.3	12.0	23.0	22.0	27.9	32.0	32.7	42.0	37.3	52.0	40.5
2.5	20.3	12.5	23.0	22.5	28.2	32.5	33.0	42.5	37.4	52.5	40.7
3.0	21.2	13.0	23.0	23.0	28.5	33.0	33.2	43.0	37.6	53.0	40.8
3.5	22.2	13.5	23.0	23.5	28.7	33.5	33.5	43.5	37.8	53.5	40.9
4.0	23.0	14.0	23.0	24.0	29.0	34.0	33.8	44.0	38.0	54.0	41.1
4.5	23.0	14.5	23.0	24.5	29.2	34.5	34.0	44.5	38.2	54.5	41.2
5.0	23.0	15.0	23.0	25.0	29.5	35.0	34.2	45.0	38.3	55.0	41.3
5.5	23.0	15.5	23.4	25.5	29.7	35.5	34.5	45.5	38.5	55.5	41.5
6.0	23.0	16.0	23.8	26.0	29.9	36.0	34.7	46.0	38.7	56.0	41.6
6.5	23.0	16.5	24.2	26.5	30.1	36.5	35.0	46.5	38.9	56.5	41.7
7.0	23.0	17.0	24.6	27.0	30.4	37.0	35.2	47.0	39.0	57.0	41.8
7.5	23.0	17.5	25.0	27.5	30.6	37.5	35.4	47.5	39.2	57.5	42.0
8.0	23.0	18.0	25.4	28.0	30.8	38.0	35.6	48.0	39.3	58.0	42.1
8.5	23.0	18.5	25.7	28.5	31.0	38.5	35.8	48.5	39.5	58.5	42.2
9.0	23.0	19.0	26.1	29.0	31.2	39.0	36.1	49.0	39.6	59.0	42.3
9.5	23.0	19.5	26.4	29.5	31.4	39.5	36.3	49.5	39.8	59.5	42.4
10.0	23.0	20.0	26.7	30.0	31.6	40.0	36.5	50.0	39.9	60.0	42.6

Table F.2 Values of Predicted Moment-Rotation Relationship (HCS1)

θ (mrad)	M (kN.m)	θ (mrad)	M (kN.m)	θ (mrad)	M (kN.m)	θ (mrad)	M (kN.m)	θ (mrad)	M (kN.m)
0.05	153.6	1.05	284.7	2.05	353.7	3.05	400.0	4.05	406.0
0.10	220.7	1.10	290.1	2.10	361.0	3.10	400.3	4.10	406.3
0.15	195.0	1.15	294.6	2.15	368.8	3.15	400.6	4.15	406.6
0.20	205.8	1.20	299.1	2.20	376.6	3.20	400.9	4.20	406.9
0.25	209.0	1.25	303.5	2.25	381.6	3.25	401.2	4.25	407.2
0.30	208.8	1.30	304.7	2.30	385.7	3.30	401.5	4.30	407.4
0.35	213.9	1.35	309.1	2.35	389.9	3.35	401.8	4.35	407.7
0.40	217.6	1.40	313.3	2.40	393.2	3.40	402.1	4.40	408.0
0.45	220.7	1.45	316.8	2.45	393.9	3.45	402.4	4.45	408.3
0.50	228.6	1.50	320.4	2.50	394.5	3.50	402.7	4.50	408.5
0.55	231.3	1.55	323.9	2.55	395.2	3.55	403.0	4.55	408.8
0.60	238.0	1.60	327.5	2.60	395.9	3.60	403.3	4.60	409.0
0.65	244.7	1.65	331.0	2.65	396.6	3.65	403.6	4.65	409.3
0.70	246.6	1.70	334.5	2.70	397.2	3.70	403.9	4.70	409.6
0.75	252.9	1.75	338.0	2.75	397.9	3.75	404.2	4.75	410.1
0.80	259.1	1.80	341.5	2.80	398.1	3.80	404.5	4.80	410.5
0.85	265.3	1.85	345.0	2.85	398.7	3.85	405.3	4.85	410.9
0.90	271.6	1.90	348.5	2.90	399.0	3.90	405.6	4.90	411.3
0.95	273.5	1.95	351.9	2.95	399.3	3.95	405.7	4.95	411.7
1.00	279.1	2.00	351.1	3.00	399.7	4.00	405.7	5.00	412.6

Table F.3 Values of Predicted Moment-Rotation Relationship (HCS2)

θ (mrad)	M (kN.m)	θ (mrad)	M (kN.m)	θ (mrad)	M (kN.m)	θ (mrad)	M (kN.m)	θ (mrad)	M (kN.m)
0.05	120.5	1.05	376.5	2.05	434.2	3.05	488.9	4.05	530.6
0.10	172.3	1.10	378.1	2.10	437.9	3.10	491.3	4.10	532.3
0.15	186.3	1.15	380.5	2.15	443.9	3.15	493.6	4.15	534.1
0.20	208.9	1.20	382.9	2.20	446.1	3.20	495.9	4.20	535.8
0.25	233.6	1.25	384.4	2.25	448.2	3.25	498.2	4.25	537.6
0.30	258.5	1.30	387.1	2.30	450.3	3.30	500.4	4.30	539.2
0.35	285.3	1.35	389.8	2.35	452.7	3.35	502.6	4.35	540.9
0.40	310.5	1.40	392.5	2.40	455.3	3.40	504.7	4.40	542.6
0.45	329.4	1.45	394.5	2.45	458.0	3.45	506.9	4.45	544.2
0.50	341.3	1.50	397.1	2.50	460.5	3.50	509.0	4.50	546.1
0.55	350.2	1.55	399.7	2.55	462.9	3.55	511.0	4.55	547.7
0.60	356.2	1.60	402.3	2.60	465.5	3.60	513.1	4.60	549.3
0.65	358.7	1.65	405.1	2.65	468.2	3.65	515.4	4.65	550.8
0.70	360.9	1.70	407.8	2.70	470.9	3.70	517.4	4.70	552.4
0.75	362.9	1.75	410.6	2.75	473.6	3.75	519.3	4.75	553.9
0.80	364.9	1.80	414.2	2.80	476.2	3.80	521.3	4.80	555.4
0.85	367.7	1.85	417.9	2.85	478.7	3.85	523.2	4.85	556.9
0.90	369.5	1.90	421.5	2.90	481.2	3.90	525.1	4.90	558.4
0.95	372.3	1.95	425.7	2.95	484.0	3.95	526.9	4.95	559.8
1.00	373.9	2.00	430.0	3.00	486.5	4.00	528.8	5.00	561.3

Figure F.1 Comparison between test results and model prediction

Figure F.2 Comparison between test results and model prediction

Figure F.3 Comparison between test results and model prediction