Behaviour of steel and composite beam-column joints under extreme loading conditions

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**Behaviour of Steel and Composite Beam-column
Joints under Extreme Loading Conditions**

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**SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING
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Behaviour of Steel and Composite Beam-column Joints under Extreme Loading Conditions

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ABSTRACT

Historical collapse incidents of buildings under extreme loadings have attracted academic and engineering interest to conduct research studies on the resistance of beam-column joints to mobilise alternate load path and develop catenary action, in order to bridge over lost structural members. The integrity of beam-column joints greatly influences the capability of structures to develop catenary action. Up to date, experimental tests have been conducted for various types of beam-column joints subjected to column removal scenarios, which are widely used as a threat-independent approach to represent structural damages caused by extreme loadings. However, experimental programmes on composite joints with fin plate (also referred to as shear tab) and welded unreinforced flange bolted web connections are very limited, although these two types of joints are quite common in industrial practice.

In the current study, experimental tests on steel and composite beam-column joints were conducted. Two types of connections, viz. fin plate and welded unreinforced flange bolted web connections were investigated under both quasi-static and impact loading scenarios. The load-resisting mechanism, failure mode, energy absorption and development of strain were presented based on test results. Tying and flexural resistances, as well as rotation capacities, were compared with design values. A comparison of quasi-static and impact tests was also conducted to quantify the contribution of strain rate effect. Furthermore, two connections, viz. fin plate connection with slotted holes and reduced beam section connection were conducted for comparison purposes. It was found that they contributed to better tying resistance and rotation capacity in comparison with conventional connections. In addition, numerical analyses on steel beam-column joints under impact loading scenarios were conducted. Three-dimensional finite element models in commercial software LS-DYNA were established and validated by test results with reasonably good accuracy. The models were used to compare two proposed evaluation indices by the author, namely, energy absorption ratio and deformation ratio for steel joints subjected to impact loads.

Based on test results, a component-based modelling approach for beam-column joints with fin plate and welded unreinforced flange bolted web connections was

proposed. The arrangement of nonlinear springs representing steel components was adopted from Eurocode and was combined with composite slab components. Furthermore, nonlinear spring properties were defined on the basis of component test results and design codes. The proposed models were able to simulate the behaviour of beam-column joints subjected to quasi-static and impact loads with adequate accuracy. Composite slab effect and strain rate were also considered by the proposed modelling approach.

In general, the current design calculation method was found to overestimate the tying resistance of both types of composite joints, especially when thicker slabs or fewer shear studs were used. The overestimation is less evident for WUF-B joints compared to FP joints. The novel FP joint was able to develop the design values of tying resistance in the test. The design values of flexural resistance and rotation capacity could be achieved by the test, especially when the beneficial effect of intermediate strain-rate was included. The aforementioned issues of the design method could be solved by using the proposed modelling approach, which could be used in design practice for engineers.

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LIST OF SYMBOLS

A_c	Area of the concrete slab
A_{nt}	Net area subjected to tension
A_{nv}	Net area subjected to shear
A_s	Stressed area of bolt
C	Cowper-Symonds strain-rate parameters
D	Design value of joint resistance rotation capacity
E	Modulus of elasticity
E_{c1}	Secant modulus from the origin to the peak compressive stress
E_{ci}	Secant modulus of concrete
E_p	Plastic hardening modulus
E_{tan}	Tangent modulus
F_{br}	Resultant bearing force of bolted connection
F_p	Peak impact force
F_{cm}	Peak compression force of the concrete slab
F_{ctm}	Peak tension force of the concrete slab
$F_{s,Rd}$	Friction force
$F(t)$	Impact force
G	Modulus of shear

I	Impact impulse
L_e	End distance from the centre of a bolt hole to the edge of the fin plate measured in the direction of load transfer
M	Bending moment of CHS members
M_i	Mass of the impactor
M_s	Equivalent lumped mass of the specimen
N	Axial force of CHS members
$N1, N2$	Respective axial forces at the left and the right pins
P	Applied vertical load
$R_{n,br}$	Ultimate strength of bolts in bearing
$R_{n,bs}$	Block tearing resistance
$R_{nv,bolt}$	Ultimate strength of bolts in single shear
V	Accompanying shear force
$V1, V2$	Respective shear forces at the left and the right pins
V_i	Initial velocity of the impactor
V_i'	Velocity of the impactor after impact
V_s	Initial velocity of the specimen
V_s'	Velocity of the specimen after impact
c	Ratio of velocity of the impactor normalised by velocity of the specimen after the impact

c_r	Coefficient of restitution
d	Distance from the pin to the strain gauged section in the CHS members
d_b	Nominal diameter of the bolt
f_{cm}	Mean value of compressive strength
f_{ctm}	Mean value of tensile strength
h_b	Beam depth
h_g	Spring gauge length
k	Energy absorption ratio
k_b	Stiffness of edge steel plate bending
k_{br}	Stiffness of bolt in bearing
k_i	Stiffness of bolted connection
k_s	Coefficient to account for the effect of the type of bolt holes
k_v	Stiffness of edge steel plate shearing
n	Number of friction surface
p	Cowper-Symonds strain-rate parameters
t	Thickness of the steel plate
v	Velocity
Δ	Displacement of bolted connection
Δt	Duration of the impact
$\bar{\Delta}$	Normalised displacement of bolted connection

$\alpha 1, \alpha 2$	Respective angles between the compressive arch and the original horizontal axis on the left and the right sides
$\beta 1, \beta 1$	Respective angles between the moving specimen axis and the original horizontal axis on the left and the right sides
δ	Residual displacement
δ_t	Time to reach a given displacement
ε	Strain
ε_p^{eff}	Effective plastic strain
ε_c	Compressive strain
ε_{c1}	Strain at maximum compressive stress
$\varepsilon_{c,lim}$	Ultimate compressive strain
ε_{ct}	Tensile strain
ε_{ctm}	Mean value of tensile strength
ε_{uk}	Ultimate strain
$\dot{\varepsilon}$	Strain rate
$\dot{\varepsilon}_c$	Strain rate in compression
$\dot{\varepsilon}_{ct}$	Strain rate in tension
η	Ratio of the compressive strain normalised by the strain at maximum compressive stress
μ	Coefficient of slip
σ	Stress

σ_0	Initial yield strength
σ_c	Compressive stress
σ_{ct}	Tensile stress
σ_y	Yield strength
σ_u	Ultimate strength
σ_{ub}	Ultimate strength of the bolt

CHAPTER 1: INTRODUCTION

1.1 Background

Extreme loading conditions include attacks from military weapons, vehicular and aircraft collisions, explosions, and the like. In general, these actions are not considered in routine design of buildings. When the size of the building actions arising from such conditions is large enough, a domino-type of collapse might occur if buildings are not designed properly. This domino-type of collapse is referred to as progressive collapse in ASCE (2010), which is “the spread of an initial local failure from element to element, resulting eventually in the collapse of an entire structure or a disproportionately large part of it”.

Fig. 1.1 Progressive collapse of Ronan point caused by a gas explosion in the 18th floor
(Krauthammer 2008)

The concept of progressive collapse can be illustrated by the collapse of Ronan point apartment building in 1968, as shown in Fig. 1.1 (Krauthammer 2008). A gas explosion at the 18th floor blew out the load-bearing flank walls, which had been supporting the four flats above. Due to the weaknesses in the joints connecting the vertical walls to the floor slabs, falling away of the flank walls left the floors above unsupported and caused the progressive collapse of the south-east corner of the building. This event first attracted academic interest in mitigating such tragic incidents. On September 11th of 2001, the collapse of the World Trade Centre

(Hamburger et al. 2002) in the US initiated a worldwide effort on researching measures to prevent progressive collapse (Fig. 1.2).

Fig. 1.2 Progressive collapse of World Trade Centre caused by aircraft collision (Hamburger et al. 2002)

Historically, the risk of progressive collapse is extremely low, but its catastrophic consequence is unacceptable to both the government and the public. To date, resistance against progressive collapse has been incorporated into many design codes and guidelines (BSI 2002a, Ellingwood et al. 2007, DOD 2013, GSA 2013). Design approaches against progressive collapse can be categorized into two groups, namely, indirect design and direct design (Ellingwood and Leyendecker 1978). Indirect design includes measures to maintain the integrity and continuity of structural members. Tie force (TF) method, which is aimed to provide horizontal and vertical ties through structural members and additional reinforcement, is one of the typical indirect design methods. By comparison, direct design refers to alternate path (AP) method and enhanced local resistance (ELR) method. The AP method aims to mobilise alternate load paths to bridge over damaged vertical structural elements, such as columns and walls. By contrast, the ELR method is used to provide sufficient strength for key elements to resist extreme loads.

1.2 Beam-column joint and progressive collapse

Among all the approaches to resist progressive collapse, the AP method provides the most straightforward resistance mechanism. As shown in Fig. 1.3, when vertical structural members are totally damaged by extreme loads on a frame, the AP method seeks to form an alternate load path in the remaining structure to bridge over the lost column and redistribute vertical loads. At large deformation stage, catenary action may be mobilised in the bridging beams over the damaged zone, as depicted by arrows in Fig. 1.3. With the development of catenary action, beam-column joints will be subjected to combined tension, shear force and bending moment, which is more complicated and unfavourable than normal load cases. Integrity of joints when subjected to these forces, ensures the development of catenary action, which helps form alternate load paths. Therefore, beam-column joints have a great impact on the resistance of structures against progressive collapse.

Fig. 1.3 Beam-column joints and catenary action in AP method

The resistance of beam-column joints has been incorporated in design guidelines against progressive collapse (DOD 2013, GSA 2013). Rotation capacities and acceptance criteria of both fully restrained and partially restrained joints are provided for analysis of AP method. However, provisions are largely adopted from the seismic design code (ASCE 2013), originated from seismic test results under cyclic loadings. This makes the criteria overly conservative for joints subjected to catenary action, which has been shown by previous test results on various types of joints (Demonceau 2008, Oosterhof 2013, Yang 2013, Weigand 2014). For commonly-used frame joints

with fin plate (FP, also referred to as shear tab) connections and welded unreinforced flange bolted web (WUF-B) connections, experimental research on their rotation capacity under column removal scenarios is limited. Hardly any tests (Wang et al. 2017) have incorporated the contribution of a composite slab.

1.3 Development of joint modelling method

A suitable joint model is necessary to conduct alternate load path analysis to evaluate the resistance of structures against progressive collapse. In the literature, a wide variety of modelling methods have been developed for beam-column joints, including finite element (FE) models using three-dimensional (3-D) solid elements, component-based models (also referred to as spring or fibre models) and plastic hinge models.

Three-dimensional solid element is available in many commercial software such as ABAQUS, ANSYS, LS-DYNA, etc. The configuration of beam-column joints can be well replicated using the solid element. With proper definition of failure criteria, the behaviour of each component of beam-column joints can be captured well by the 3-D FE model. However, this modelling technique requires a large number of elements, thereby substantially increasing the computational cost. Besides, convergence problems may exist in the analysis, which leads to unreliable results. These two limitations hinder the full applications of 3-D joint models in building structures.

Component-based model is aimed to discretise the geometry of beam-column joints into basic components or springs. In comparison with solid element models, component-based model neglects the subtle details of beam-column joints but maintains mechanical properties of components which dominate the joint behaviour. This makes the component-based model more computationally efficient than 3-D solid element model. Component-based model has been incorporated into Eurocode 3 Part 1-8 (2005b) for design of conventional joints. For joints subjected to catenary action, several models have been developed for specific types of joints to date (Del Savio et al. 2009, Bzdawka and Heinisuo 2010, Stylianidis 2011, Main and Sadek 2012, Piluso et al. 2012, Taib 2012, Oosterhof 2013, Yang and Tan 2013, Koduru and Driver 2014, Main and Sadek 2014, Yang et al. 2015). However, to the author's

knowledge, component-based model for commonly-used composite joints such as fin plate and WUF-B joints has not yet been developed. This is the novelty of the research programme.

1.4 Objectives and originality of the research work

The objectives of this research are:

- To investigate the resistance, load-resisting mechanism and failure mode of two types of commonly-used beam-column joints (FP and WUF-B) subjected to extreme loading conditions;
- To enhance the behaviour of bare steel and composite joints by modifying the joint detailing in the test programme;
- To identify governing factors on joint resistance and ductility;
- To compare resistance and rotation capacity of beam-column joints with current design methods;
- To investigate strain-rate effect on beam-column joints;
- To develop component-based models of bare steel and composite joints for analysis of progressive collapse resistance.

The originality of the current research lies in consideration of both composite slab effect and intermediate strain-rate effect on beam-column joints under middle column removal scenarios. Whether the current design method is still applicable for flexural, tying resistances and rotation capacities of beam-column joints will be investigated based on experiments, which is significant for design practice but has not been conducted in previous studies. To supplement the design calculation method, a new component-based modelling approach that is able to consider both effects will be provided.

1.5 Layout of the thesis

To fulfil the research objectives, both experimental and numerical studies were conducted and presented in the thesis, divided into 9 chapters. The contents of the chapters are summarised as follows:

Chapter 2 presents a literature review on previous research studies related to

behaviour of steel and composite beam-column joints. The review includes current design provisions on progressive collapse resistance of building structures, a widely-accepted design framework developed in the Imperial College London, and experimental and numerical studies on beam-column joints subjected to extreme loading conditions.

In Chapter 3, two types of bare steel beam-column joints (FP and WUF-B) were tested under quasi-static and impact loading scenarios. Numerical analyses on steel joints were conducted using three-dimensional finite element models in LS-DYNA under impact loading scenarios. Parametric study was conducted to investigate governing parameters including the impact mass, velocity, momentum and energy. Moreover, deformation ratio and energy ratio were proposed and compared for evaluating structural performance of bare steel joints.

Chapter 4 presents a test programme on composite joints with FP connections subjected to a column removal scenario and quasi-static load was applied to the beam-column joints. Load-resisting mechanism, failure mode, internal force, energy and development of strain were investigated. A comparison with code predictions was conducted.

Chapter 5 introduces a test programme on composite joints with moment-resisting connection subjected to a column removal scenario. Load-resisting mechanism, failure mode, energy and development of strain were investigated. Test results were compared with code predictions as well as those of FP connections.

In Chapters 6 and 7, composite joints with FP and WUF-B connections were investigated under impact loads, respectively. Structural response, failure model and development of strain were presented and compared with those of joints subjected to quasi-static loads. Resistance and rotation capacity of the composite joints were compared with design values. The strain rate effect was investigated.

In Chapter 8, a component-based modelling approach was proposed for steel and composite beam-column joints subjected to quasi-static and impact loads. Component-based models were validated against test data and good agreement was achieved.

Chapter 9 presents conclusions and recommendations for future work.

CHAPTER 2: LITERATURE REVIEW

2.1 Introduction

This chapter provides a literature review on the behaviour of steel and composite joints under extreme loading conditions. It mainly includes four parts, viz. current code provisions and guidelines to mitigate progressive collapse, assessment framework for structures under column removal scenarios, experimental tests and numerical simulations on beam-column joints. In the first part, current codes and guidelines on mitigating progressive collapse are reviewed. Their advantages and limitations are introduced. The second part introduces a design framework to assess the resistance of building structures under sudden column removal scenarios. Furthermore, experimental tests and numerical modelling of beam-column joints are reviewed.

2.2 Provisions on progressive collapse in current codes and guidelines

For the past few decades, especially after the disastrous terrorists' attack on the World Trade Centre on September 11th 2001, a few technical design documents have been released, among which UFC 4-023-03 (2013) and GSA (2013) guidelines are most commonly used.

2.2.1 UFC 4-023-03 Design of buildings to resist progressive collapse

Department of Defense of the US implements UFC 4-023-03 *Design of buildings to resist progressive collapse* (2013) for facilities required by UFC 4-010-01 *Minimum Antiterrorism Standards for Buildings*. In this document, buildings are divided into four occupancy categories (OC) and different design requirements are provided for each category as listed in Table 2.1. The occupancy categories are consistent with other design codes in the US such as ASCE (2010). Design methodology includes direct and indirect design approaches. For direct design approach, alternate path (AP) method and enhanced local resistance (ELR) method are provided. Among various indirect design approaches introduced in UFC 4-023-03, tie force (TF) method is the

major measure to provide continuity and integrity for building structures subjected to progressive collapse.

Table 2.1 Design approaches for buildings with different occupancy category (DoD 2013)

Occupancy category	Design requirement
I	No specific requirements
II	Option 1: Tie forces (TF) for the entire structure and enhanced local resistance (ELR) for the corner and penultimate columns or walls at the first story. OR Option 2: Alternate path (AP) for specified column and wall removal locations.
III	AP for specified column and wall removal locations and ELR for all perimeter first story columns or walls.
IV	TF and AP for specified column and wall removal locations and ELR for all perimeter first story columns or walls.

TF method is used to mechanically tie structural elements of buildings together to ensure a minimum level of continuity, ductility and integrity. TF method enables the development of tensile membrane action in the floor or roof system at large deformation stage. In this method, the contribution of structural members and connections should be neglected if they are not able to sustain a rotation of 0.20 radian (11.3 degrees), a value which is overly stringent in this author's opinion.

AP method is aimed to ensure buildings must be able to bridge across removed elements that may cause progressive collapse. Locations of removed vertical elements are prescribed in UFC 4-023-03. Three types of alternate path analysis methods are employed, namely, linear static, nonlinear static and nonlinear dynamic analysis.

In addition to TF and AP methods, ELR method is required for 3 occasions: OC II Option 1 (TF and ELR), OC III, and OC IV. This is to insure that columns or walls must be able to develop the maximum flexural strength without premature shear failure.

2.2.2 GSA 2013 Alternate path analysis and design guidelines for progressive collapse resistance

General Services Administration (GSA) of the US released the latest version of *Alternate path analysis and design guidelines for progressive collapse resistance* (GSA 2013) in 2013. It replaces the former design guideline *Analysis and design*

guidelines for new federal office buildings and major modernization projects (GSA 2003). It is intended to be applied to GSA owned and leased buildings. In this version, tie force and enhanced local resistance methods are abolished. Instead, only alternate path method is adopted.

Unlike UFC 4-023-03, GSA 2013 introduces the concept of facility security levels (FSL) to classify buildings. GSA 2013 incorporates redundancy requirements in the guidelines. According to the requirements, load redistribution systems should be provided at the exterior of structures. The locations and strength of these systems are specified.

2.2.3 ASCE 7 Minimum design loads for buildings and other structures

The American Society of Civil Engineers (ASCE) standard *Minimum design loads for buildings and other structures* (ASCE 2010) provides load combinations for extraordinary events to prevent disproportionate collapse. ELR and AP methods are also recommended as potential approaches to mitigate progressive collapse.

2.2.4 Eurocode 1 Actions on structures

Eurocode 1 *Actions on structures Part 1-7 General actions – Accidental actions* (BSI 2002) gives provisions on mitigating disproportionate collapse. Both direct methods (AP and ELR) and indirect methods (TF and integrity detailing) are provided. These methods can be applied to different categories of buildings that are classified based on consequence class, which is defined according to the acceptance of public society in Europe.

In the aforementioned documents, design criteria on the integrity of steel beam-column joints are provided based on previous research studies and findings on seismic design. However, it was pointed out that these criteria may not be suitable at all for structures subjected to progressive collapse (Yang and Tan 2013a).

2.3 Progressive collapse assessment method

A simplified framework to assess the progressive collapse resistance of multi-story frame buildings was developed at the Imperial College, London (Izzuddin et al. 2007, Vlassis 2007, Izzuddin et al. 2008, Vlassis et al. 2008, Vidalis and Nethercot 2013).

The assessment method was intended to determine the resistance of multi-story buildings subjected to sudden column loss, as shown in Fig. 2.1. The framework could be utilised at various levels of simplification, such as multiple floor level, single floor level and beam grillage level. Three steps were required in this framework as follows:

- To determine the nonlinear static response of subassembly
- To obtain the pseudo-static response through simplified dynamic assessment
- To assess the ductility of subassembly

Fig. 2.1 Multi-story building subjected to single column removal scenario (Izzuddin et al. 2008)

Figs. 2.2 and 2.3 show the simplified and detailed models to determine the nonlinear static response of a beam, which is the most simplified level for multi-story buildings. In the simplified beam model, response curves are defined by a series of functions based on an analytical model. Both compressive arching and tensile catenary action can be considered according to the strength of boundary restraints. In the detailed beam model, response curves are obtained by conducting an analysis of beams subjected to increasing gravity loads. Once nonlinear static curves are obtained at the beam level, the response of single or multiple floors can be evaluated accordingly by assembling the beams together, as illustrated in Figs. 2.4 and 2.5, respectively. By using virtual work (energy) method, the nonlinear static response of single or multiple floors can be derived in accordance with the relationships between the beam and floor displacements and corresponding loads.

Fig. 2.2 Simplified beam model (Izzudin el al. 2008): (a) Tensile catenary action; (b) Compressive arching and tensile catenary action

Fig. 2.3 Detailed beam model (Vlassis el al. 2008)

Fig. 2.4 Grillage approximation of single floor system with three beams (Izzudin el al. 2008)

Fig. 2.5 Multiple floor system with three stories (Izzudin el al. 2008)

Once the nonlinear static response is derived for that level in the form of beam grillage, or single floor or multiple floors, it can be transformed into the pseudo-static response by taking dynamic effect into consideration. It is assumed that the work done by static force (area under the nonlinear curve in Fig. 2.6) is equal to the work done by constant gravity force (rectangular area) at the same vertical displacement. Correspondingly, the pseudo-static load-displacement curve can be calculated according to the quasi-static response, as shown in Fig. 2.6.

The final step is to assess the ductility of the system. It can be represented by two indices: pseudo-static load capacity and ductility limit. These two indices are known for a given structure. To protect building structures against progressive collapse, the pseudo-static response of structures has to be smaller than these two indices. The assessment framework provides the first quantifiable approach for the design of buildings against progressive collapse.

Fig. 2.6 Simplified dynamic assessment and pseudo-static response: (a) Dynamic response ($P = \lambda_1 P_0$); (b) Dynamic response ($P = \lambda_2 P_0$); (c) Pseudo-static response

This approach was applied to evaluate the dynamic behaviour of steel structures subjected to sudden column removal scenarios (Fu et al. 2017). However, there are limitations of this framework. Firstly, the contribution of composite slabs to structural resistance is ignored and secondly, the failure mode is dominated by single

connection failure, which is not always applicable. Besides, it is unknown whether Izzuddin's approach is applicable to joints subjected to impact loads.

2.4 Experimental tests on beam-column joints

2.4.1 Bare steel joints

To date, bare steel FP connection has been intensively studied under column removal scenarios.

Guravich (2002) conducted one hundred tests on five types of shear connections, viz. header angle, knife angle, single angle, shear tab (fin plate), and end plate connections. Among them, six fin plate connections were tested under combined tension and vertical shear. It was found that the ductility of connections was provided by yielding in bearing at bolt holes and by shear yielding of plates. However, the specimens were not tested until final failure occurred.

Thompson (2009) tested a series of nine fin plate connections under simulated middle column removal scenario, using a test set-up shown in Fig. 2.7. Different numbers of A325 bolts with a diameter of 19 mm were used. Two pins were designed to simulate the inflection point at the middle span of beams under concentrated point load. It is notable that all the beam webs were stiffened by welding a 9.5 mm thick plate. Bolt shear and fin plate tear-out were the primary failure modes.

Fig. 2.7 Test set-up (Thompson 2009)

Yang and Tan (2013, 2013a) conducted two series of experimental tests on various semi-rigid connections. The test apparatus is shown in Fig. 2.8. Only one fin plate connection was included and its load versus displacement relationship is shown in Fig. 2.9. The vertical load increased smoothly and reached the peak point when bolt shear failure occurred. The post-peak behaviour was characterized by sequential brittle shear failure of bolts from the bottom row upwards. Fin plate connection had

very limited rotation capacity when compared with other connections, such as web cleat, end plate, top and seat angle, web cleat with top and seat angle (TSWA) connections.

Fig. 2.8 Configuration of test apparatus (Yang and Tan 2013a)

Fig. 2.9 Load vs displacement relationship (Yang and Tan 2013a)

To study the behaviour of shear connections under column removal scenarios, Oosterhof and Driver (2012, 2015) conducted thirty-five full-scale tests, in which nine fin plate connections and five welded single-angle connections were included. Fig. 2.10 shows the details of these specimens and Fig. 2.11 depicts the test set-up.

Force and displacement were calibrated at each step to match the values calculated through equivalent middle column removal scenario. Due to the short edge distance of the fin plate (29 and 35mm), all of the specimens exhibited tear-out failure of fin plates. By increasing the diameter of bolts and the thickness of fin plates, the resistance and rotation capacities of fin plate connections could be increased.

Fig. 2.10 Details of connections (Oosterhoof and Driver 2015): (a) Three bolts; (b) Five bolts

Fig. 2.11 Test set-up (Oosterhoof and Driver 2015)

Weigand and Berman (2014) investigated a series of fourteen sub-assemblages with fin plate connection under simulated middle column removal scenario (see Fig. 2.12). Governing parameters, including the number, diameter and grade of bolts, fin plate thickness, hole type, edge distance, location of beam axis, gap distance and connection type, were studied in this experimental programme. All the bolts were preloaded so that most of them failed in shear. It was concluded that short slotted holes improved the ductility of connections when compared with standard holes.

Fig. 2.12 Test set-up (Weigand and Berman 2014)

Cortés and Liu (2017) tested a series of twelve gravity connections subjected to a middle column removal scenario and seven fin plate connections were incorporated. It was found that all the six conventional fin plate connections were not able to achieve the required load demand by ASCE (2010): 1.2 Dead load + 0.5 Live load. However, after reinforcing the conventional connection by four additional steel plates (Fig. 2.13), the enhanced specimen could achieve the load demand.

Fig. 2.13 Reinforced fin plate connection (Cortés and Liu 2017)

Table 2.2 summarises the experimental tests on fin plate connections. It can be concluded that the 0.20 rad acceptance criteria for rotation capacity of fin plate joints in the design guidelines of DOD (2013) and GSA (2013) are overly conservative in comparison with test results. However, the resistance and ductility for fin plate joints are quite limited. There is an urgent need to enhance the joint behaviour through

special detailing. Furthermore, none of these studies considered the beneficial effect of composite action between the concrete slab and the bare steel FP connection underneath it.

Table 2.2 Summary of typical tests on fin plate connections under column removal scenarios

	Thompson 2009	Yang 2013	Oosterhof 2013	Weigand 2014	Cortés and Liu 2017
No. of tests	9	1	9	13	7
No. of bolts	3,4,5	3	3,5	3,4,5	4
Bolt type	A325	8.8 M20	A325	A325,A490	A325, J429
Bolt diameter(mm)	/	20	19,22	19,22	9.5, 12.7
Bolt hole	STD	STD	STD	STD,SSLT	STD
Plate thickness(mm)	/	8	6.4,9.5	6.4,9.5	6.4
Load arrangement	PL	PL	PL,UDL	PL	PL
Gap distance(mm)	/	10	25	38,6.4	6.4
Span length(m)	2	6	6,8,9,12	9.1,14.6	4.57
Beam	457×152×52	305×165×40	305×305×143 533×312×165	533×165×74 457×152×52	
Column	/	203×203×71	254×254×89	305×305×107 356×368×134	
Bolt shear	/	1	0	10	6
Plate tear-out	/	0	9	3	1
Catenary action (kN)	150-250	198	331-822	384-647	46.3-374
Rotation capacity	0.090-0.140	0.125	0.083-0.152	0.067-0.110	0.035-0.088
Rotation at peak	0.087-0.14	0.096	0.08-0.095	0.067-0.110	0.035-0.082

Note: STD is standard hole, SSLT is short slotted hole; PL means point load, UDL means uniform distributed load.

Bare steel joints with moment-resisting connections have been investigated intensively as well.

Karns (2009) investigated two types of full-scale moment connections under column removal scenarios, viz. WUF-B and SidePlate[®] connections. Fig. 2.14 shows the test set-up. One group of two specimens was subjected to blast loading while the other group was subjected to an artificial missing column scenario. Fig. 2.15 illustrates the post-blast configurations of test specimens. It was found that the middle column removal scenario served as a credible simulation of blast-induced initial damage. However, blast tests are extremely costly and difficult to repeat due to their sensitivity to a host of environmental factors such as relative humidity, ambient temperature, and wind velocity. Instead, drop-weight tests could be applied to

introduce dynamic loads to structural members.

Fig. 2.14 Test set-up for blast/progressive collapse scenario (Karns et al. 2009)

Fig. 2.15 Post-blast photo of test specimens (Karns 2009): (a) WUF-B; (b) SidePlate®

National Institute of Standards and Technology of the US conducted a series of full-scale tests on steel and concrete substructures under column removal scenarios (Lew et al. 2009, Sadek et al. 2010, Sadek et al. 2011, Lew et al. 2013). Among them, two steel moment frame sub-assemblages with WUF-B and reduced beam section (RBS) connections were tested. Fig. 2.16 shows the test set-up. With the initiation of fracture in the welded joints, the resistance dropped dramatically as shown in Fig. 2.17. The RBS specimen showed better resistance and ductility than the WUF-B specimen. Both specimens exhibited tensile fracture of bottom beam flanges as shown in Fig. 2.18. It was found that the rotation capacities were about twice of those required values (0.081 rad and 0.140 rad for WUF-B and RBS connections, respectively, versus requirements of 0.047 rad and 0.072 rad specified in FEMA 350

(2000)).

Fig. 2.16 Test set-up and instrumentation layout (Lew et al. 2009)

Fig. 2.17 Load vs displacement curves of moment frame sub-assemblages (Lew et al. 2013): (a) WUF-B connection (b) RBS connection

Fig. 2.18 Failure modes of moment frame sub-assemblages (Lew et al. 2013): (a) WUF-B connection; (b) RBS connection

Based on collapse resistance analysis and tests of several types of steel moment connections, Kim et al. (2009, 2012) concluded that WUF-B connection showed greater rotation capacity (0.092 rad) than the criteria provided by GSA (2013) and UFC 4-023-03 (2013).

In addition to moment-resisting connections, two three-story moment-resisting frames were tested horizontally by Tsitos (2010) under column removal scenarios as shown in Fig. 2.19. One of the frames was designed as a special moment-resisting frame (SMRF), while the other was strengthened using post-tensioned energy dissipating (PTED) method. Fig. 2.20 shows a comparison of the load versus displacement curves of both frames and design load required by UFC 4-023-03 (2013). It was concluded that both frames were able to resist the design load.

Fig. 2.19 Test set-up of multi-frame (Tsitos 2010)

Fig. 2.20 Global force versus displacement curves (Tsitos 2010)

Li et al. (2015) reported a test programme on two moment-resisting frame sub-assemblages with different bolt arrangements. The test set-up and schematic of specimens are illustrated in Figs. 2.20 and 2.21, respectively. Specimen SI-WB had one line of four bolts while the other had two lines of four bolts. Through experimental tests, vertical load versus middle joint displacement curves were obtained (see Fig. 2.23). It was concluded that one line of bolts had better strength and ductility than two lines of bolts. It was also found that moment connection could be robust enough to sustain the applied load after the middle column was removed.

Fig. 2.21 Schematic of test specimens (Li et al. 2015)

Fig. 2.22 Test set-up (Li et al. 2015)

Fig. 2.23 Load vs displacement curves (Li et al. 2015)

Table 2.3 shows the experimental tests on moment-resisting connections under column removal scenarios. Similar to the simple pin connection, rotation capacities of moment-resisting joints are considerably greater than the values provided in design guidelines against progressive collapse (DOD 2013, GSA 2013). However, the rotation of welded joints at peak load is still quite limited and a dramatic drop of resistance is observed to reach the maximum rotation. This greatly limits the rotation capacity of welded joints in design practice. Therefore, further modifications to welded connections are necessary to enhance the joint behaviour under column removal scenarios.

Table 2.3 Summary of tests on moment-resisting connections

	Karns 2009	NIST	Tsitos 2010	Li 2015
No. of tests	4	2	2	2
No. of bolts	/	3	3	4
Bolt type	/	A490	A325	/
Bolt diameter(mm)	/	25	20	19, 22
Plate thickness(mm)	/	/	/	6
Load arrangement	Blast, PL	PL	PL	PL
Span length(m)	5.5	6	1.2	4.5
Beam	457×152×52	533×210×109 610×229×140	203×102×19 203×102×15 152×102×13	300×150×6×8
Column	406×178×85	457×279×177 610×324×195	127×127×28	250×14
Vertical load (kN)	177-711	889, 2001	533, 689	225, 270
Rotation capacity	0.08-0.18	0.081, 0.14	/	0.1525, 0.173
Rotation at peak	0.043-0.148	0.06-0.14	0.06, 0.146	0.105, 0.108

Other than quasi-static experimental tests, researchers tend to focus on more realistic dynamic scenarios. Liu et al. (2015) conducted free-fall tests and numerical simulations on both flush end-plate and bolted angle joints subjected to a middle column removal scenario. Dynamic increase factors provided by design guidelines were reviewed based on these studies. Tyas et al. (2012) and Rahbari et al. (2014) developed a comprehensive test rig to study the behaviour of web cleat joints loaded by pneumatically-activated loading rams. Failure modes and different governing parameters were investigated. Zeinoddini et al. (2002, 2008) investigated axially preloaded tubular columns subjected to lateral impact and indicated that axial preloading affected the level of damage substantially. Cho et al. (2014) conducted drop-weight impact tests on four T-shaped steel beams at room and sub-zero temperatures. Permanent deflections were measured at room temperature and were smaller than those recorded at sub-zero temperature. Wu et al. (2016) conducted drop-weight tests on concrete beams prestressed with unbonded tendons and proposed mesoscale simulation methods. More specifically, researchers conducted drop-weight impact tests on structural steel and concrete joints. Qu et al. (2014) numerically investigated the dynamic behaviour of tubular T-joints subjected to impact loads from a drop-weight machine. The ratio of energy absorption to the total energy of the specimens was found to be consistent. Besides, respective energy absorbed by local and global deformations was also quantified and differentiated. Grimsmo et al. (2015, 2016) conducted experimental and numerical studies on extended end plate joints (without axial restraints) subjected to impact loads as shown in Fig. 2.24. Different failure modes were observed in different load directions. However, up to now, very few studies focused on fin plate (FP) and WUF-B joints subjected to impact loads.

Fig. 2.24 Set-up of low-speed impact test by Grimsmo (2015)

Besides, the contribution of composite slabs to the behaviour of steel joint was not considered in the aforementioned research studies.

2.4.2 Composite Joints

Currently, experimental studies on composite joints subjected to abnormal loads are much fewer than bare steel joints.

Demonceau (2008) investigated a full-scale composite substructure (see Fig. 2.25) with end plate connections. Prior to failure, the substructure experienced significant vertical deflections. The final failure was dominated by rupture of reinforcement in the concrete slab and crushing of concrete.

Fig. 2.25 Failure of composite frame (Demonceau 2008)

A series of tests on composite joints was also conducted at Stuttgart University (Kuhlmann et al. 2007, Demonceau 2008, Kuhlmann et al. 2009). To study the joint behaviour, composite joints with end plate connections were extracted from a substructure and tested under hogging and sagging moments, respectively. Two stages of loading scheme were employed for the joints, as shown in Fig. 2.26. The first stage was to apply a sagging or hogging moment to the joint. When joint attained the moment capacity, axial tension force was imposed while bending moment was kept constant. Before failure, composite joints exhibited desirable deformation capacities. Crushing of composite slabs under sagging moment and tensile cracking of slabs under hogging moment dominated the ultimate resistance of joints.

Fig. 2.26 Test set-up for composite joints (Kuhlmann et al. 2007): (a) The first stage of the composite testing procedure; (b) The second stage of the composite testing procedure

Guo et al. (2013) investigated a four-bay composite frame under middle column removal scenario, as shown in Fig. 2.27. Beams were fully welded to column flanges to form rigid beam-column connections. Fig. 2.28 shows the vertical load-displacement curve of the frame. After the initial peak resistance, the frame developed significant catenary action to prevent progressive collapse. Failure was characterized by fracture of bottom flange and beam web and crushing of concrete at the middle joint, and compressive buckling of bottom beam flange at the side joint. Further tests on beam-column joints subjected to combined sagging or hogging moment and axial tension were also carried out by Guo et al. (2014). It was

concluded that rigid composite joints could satisfy the requirements of developing catenary action under column removal scenarios, as specified in the DoD guidelines (DOD 2013).

Fig. 2.27 Tested composite frame (Guo et al. 2013)

Fig. 2.28 Load vs displacement curve of middle column (Guo et al. 2013)

Yang and Tan (2014) reported a series of five tests on composite joints with flush end plate and web cleat connections subjected to hogging and sagging moments. Experimental results demonstrated that composite slabs could increase load-carrying capacities at flexural action and catenary action stages compared to bare steel joints. Besides, side joints could develop much greater resistance than middle joints at flexural action stage due to greater lever arm. When using four additional high strength 16 mm diameter continuous reinforcing bars in composite slabs, it was found that the tying and flexural resistances of web cleat and flush end plate connections could be significantly strengthened.

Jamshidi and Driver (2014) presented a test programme on seventeen full-scale composite joints under column removal scenarios. Two types of composite connections, viz. FP connection and bolted double angle connection were included in the experimental programme. Fig. 2.10 shows the test set-up. It was found that arching action was initiated in the joint before the development of catenary action.

Similar to bare steel joints, failure of composite joints was dominated by tear-out of the fin plate. It was concluded that the rotation capacity of composite joints was significantly smaller than that of bare steel joints because the concrete slab considerably limited the rotation capacity of the FP connections at flexural action stage.

However, research studies on other types of joints (Wang et al. 2017) as shown in Fig. 2.29 concluded that composite joints could achieve good tensile capacity and ductility under column removal scenarios. Considering the contradictory findings in Wang et al. (2017) and Jamshidi and Driver (2014), it is important to study beam-column joint tests incorporating composite slab effect.

Fig. 2.29 Test set-up of composite joint (Wang et al. 2017)

Table 2.4 summarises the tests on composite joints under column removal scenarios. Even though experimental tests have been conducted on several types of composite joints, such as fin plate, web cleat, end plate and fully welded joints, test data on fin plate joints are very limited. Moreover, it is clear that behaviour of composite joints with WUF-B connection has not been explored yet.

Table 2.4 Summary of quasi-static tests on composite joints

	Demo 2008	Stuttgart	Guo 2013,2014	Yang 2014	Jamshidi 2014
No. of tests	1	5	6	5	1
Connection	Flush end plate	Flush end plate	Fully welded	Web cleat Flush end plate	Fin plate
Concrete	C25/30	C25/30	/	C25/30	C25/30
Slab type	Solid	Solid	Solid	Composite	Solid
Slab width (mm)	500	500	800	587	2040
Slab thickness (mm)	120	120	100	110	127
Reinforcement	6Φ8	6Φ8	12Φ12	4T16	10M@250 2 layers
Shear stud	19/75@150	19/75@150	16/75@100	16/75@90	19/115@200
Load arrangement	UDL	PL	PL	PL	UDL
Span length(m)	4	4	2	4	6
Beam	140×73×13	140×73×13	200×100×5.5×8	254×146×37	305×305×143
Column	152×160×30	152×160×30	200×200×8×12	203×203×71	254×254×89
Vertical load (kN)	120	/	400, 150-280	184-333	60
Rotation capacity	0.17	/	0.017-0.07	/	0.17
Rotation at peak	0.17	/	0.015-0.04	0.128-0.178	0.106

2.5 Numerical simulations on beam-column joints

Based on the test results, cost-effective numerical models were also developed to facilitate the analyses of beam-column joints subjected to progressive collapse scenario.

2.5.1 Finite element modelling of beam-column joints

Khandelwal and El-Tawil (2007) investigated collapse behaviour of steel joints with and without reduced beam section (RBS) using numerical simulations and concluded that moment-resisting connections demonstrated adequate rotation capacity and ability to mobilise catenary action. It was also found that transverse beams had no adverse effect on structural behaviour.

Numerical simulations conducted by Karns et al. (2009) showed that WUF-B connection with concrete slab had much smaller rotation capacity compared to bare steel connection, although its load-carrying capacity was not affected significantly.

In addition to experimental tests, National Institute of Standards and Technology of

the US (Sadek et al. 2010, Sadek et al. 2011, Sadek et al. 2013) carried out several numerical studies on WUF-B connections using LS-DYNA. Three-dimensional solid elements were used to model the two-bay moment frame. Material properties used in the simulations were validated by coupon tests, in which fracture of steel was defined when the ultimate strain was reached. Furthermore, contact relationships were defined between the contact surfaces. The force-displacement curves and failure modes of connections agreed well with test data. Subsequent numerical studies were extended to gravity frames (Main and Sadek 2012, Main and Sadek 2014).

Daneshvar and Driver (2011) conducted numerical simulations on fin plate connections through three-dimensional solid elements in ABAQUS. Material and geometric nonlinearity were considered in the numerical model. Contact-pair algorithms were also used in the studies and a friction coefficient of 0.3 was recommended. Moreover, hard-contact formulations with a penalty constraint enforcement method were applied to model the normal behaviour of contact. Additionally, mesh convergence analyses were conducted to find the minimum layer of elements in the thickness direction of the fin plate. Numerical results were verified by test data from Thompson (2009). In the latter's research, combined C3D6, C3D8, C3D8R and C3D8I elements were used to enhance computational efficiency (Daneshvar et al. 2013). Strain-based fracture criteria were defined for both fin plates and bolts to correctly capture failure modes of beam-column joints.

Yang and Tan (2012) investigated six types of beam-column joints numerically. Three-dimensional solid elements C3D8R provided by ABAQUS were chosen to simulate beam-column joints. Convergence difficulties caused by contact pairs were overcome by using the dynamic explicit solver in ABAQUS. Dynamic effects induced by the explicit solver were neglected as kinetic energy only accounted for less than 10% of total energy in the model. The standard solver was also utilised for comparison purpose. Satisfactory numerical results were obtained by explicit and standard solvers.

Jamshidi et al. (2012, 2013, 2014) simulated the fin plate connection under column removal scenarios using ABAQUS. General contact interactions were used to define

contact between various components. The welds of fin plate were simulated as tie constraint. Ductile damage criterion was chosen to model steel fracture. Element erosion was defined and corresponding energy-based damage evolution was adopted. To overcome convergence problems, explicit dynamic solver was utilised with appropriate mass scaling. A constant loading rate of 75mm/s was selected to minimise the dynamic effect and a displacement control process with smooth step was adopted. Good agreement between experimental and numerical results was achieved.

However, the solid element finite element models take a long computational time and convergency problems will arise when applying them to analyse full-scale building structures.

2.5.2 Component-based modelling of beam-column joints

Extensive research studies have been conducted to predict beam-column joint behaviour using component-based models.

Del Savio et al. (2009) proposed a generalised numerical model for semi-rigid joints, as shown in Fig. 2.30. Combined bending moment and axial force effects could be considered in the model. The constitutive law of each spring was simplified as a tri-linear curve, as shown in Fig. 2.31. The proposed component-based model was validated by six experimental tests on extended end plate joints.

Fig. 2.30 Generalised mechanical model for semi-rigid joints (Savio et al. 2009)

Fig. 2.31 Force vs displacement curves for components: (a) In tension; (b) In compression (Savio et al. 2009)

Bzdawka and Heinisuo (2010) introduced a component-based model for fin plate connections by using components defined in Eurocode 3 Part 1-8 (2005b). Fig. 2.32 depicts the arrangement of the components. It was assumed that all the components were subjected to tension. The model could only be used to predict the resistance rather than the load-rotation curves.

Fig. 2.32 Arrangement of components for fin plate connection (Bzdawka and Heinisuo 2010)

Stylianidis (2011) extended the mechanical model proposed by Del Savio et al. (2009) to composite joints, as shown in Fig. 2.33. Component properties were simplified as bi-linear and multi-linear curves (see Fig. 2.34). The initial stiffness and capacities of components were adopted from Eurocodes (BSI 2004a, BSI 2005b). Post-limit behaviour was defined by a hardening coefficient. The mechanical model was validated by the FE code ADAPTIC and empirical results on end plate connections.

Fig. 2.33 Connection modelling of composite joint: (I) Arrangement; (II) Mechanical model; (III) Component forces; (IV) Typical deformation mode (Stylianidis 2011)

Fig. 2.34 Component properties: (a) Bi-linear; (b) Multi-linear (Stylianidis 2011)

Piluso et al. (2012) developed a component-based model for composite joints with top and seat angle (TSWA) connections and end plate connections subjected to hogging and sagging moment. Property of each component was well defined. The model could replicate the moment-rotation curves of experiments for end plate connections. For TSWA connections, the model could only capture the initial stage of moment-rotation curves.

Taib (2012) introduced a component-based model for fin plate connection exposed to fire. Fig. 2.1 shows the arrangement of components. Property of each component was defined according to previous experimental results. The mechanical model was included in the FE code VULCAN.

Fig. 2.1 Mechanical model for fin plate connection (Taib 2012)

National Institute of Standards and Technology (Main and Sadek 2012, Main and Sadek 2014) developed a component-based model that could be incorporated in LS-DYNA. The constitutive relationship of bolt component was defined according to AISC (2010) and FEMA (2000), as shown in Fig. 2.2. Shear and tension deformations were coupled by limiting the sums of the two values to unity. The mechanical model was also extended to WUF-B connections by modelling welded flanges as beam elements in LS-DYNA (Sadek et al. 2010, Sadek et al. 2013).

Fig. 2.2 Axial load versus deformation curves for connection springs: (a) Gradual softening; (b) Sudden fracture

Yang and Tan (2013) proposed a component-based model for bolted angle (web cleat and TSWA) connections, as depicted in Fig. 2.35. Component tests were conducted to determine the properties of each spring. This model was calibrated by experimental results on steel joints under column removal scenarios. Yang et al. (2015) then extended the component-based model to composite connections (web cleat and end plate). The mechanical model was applied using ABAQUS and validated by test results on composite joints.

Fig. 2.35 Component-based model for bolted angle connections (Yang and Tan 2013b)

Oosterhof (2013) reported a component-based model for fin plate joints. Fig. 2.36 depicts the arrangement of components. Properties of springs were adopted from the Canadian code (CSA 2009) and component tests (Rex and Easterling 1996) as well. Failure criterion was approximately determined from experimental results. This model was applied by MATLAB code and validated by test results.

Fig. 2.36 Arrangement of components (Oosterhof 2013)

Koduru and Driver (2014) proposed a generic component-based model for fin plate connections (see Fig. 2.37). Each bolt row was modelled by a series of springs. Unloading and degradation behaviour was determined for the springs. This model was verified by test results on joints under combined tensile and moment loading

conditions.

Fig. 2.37 Component-based model for fin plate connection (Koduru and Driver 2014)

Weigand (2014) incorporated a component-based model for steel joints in OPENSEES. Springs of slip, bearing and friction were arranged in parallel or series based on their physical relationships. To simulate experimental tests on fin plate joints with pre-tensioned bolts, the constitutive relationship of bolt friction was also defined in the model. Good agreement was observed when comparisons were made between experimental and analytical results.

Table 2.5 summarises component-based models for FP and WUF-B joints. There are only a few analyses incorporating unloading, degradation and failure of springs in the models. Besides, composite slabs are not considered in most cases. Therefore, further improvement is necessary when applying the models to progressive collapse scenarios.

Table 2.5 Summary of component-based model on FP and WUF-B connections

		Bzdawka 2010	Taib 2012	NIST	Oosterhoof 2013	Koduru 2014	Weigand 2014
Type	Fin plate	√	√	√	√	√	√
	WUF-B			√			
	Slab						
	Bolt shear		√	√	√	√	√
	Bolt slippage		√		√	√	√
	Friction		√		√		√
	Failure criteria		√	√	√		
	Coupling of shear and tension			√			
	Unloading			√		√	√
	Degradation		√	√		√	

2.6 Concluding remarks

Based on the literature review, it can be concluded that the current design guidelines have provided acceptance criteria of beam-column joints originated from previous research studies and findings on seismic design. The design method needs to be investigated under progressive collapse scenarios. Currently, research studies on the behaviour of composite FP and WUF-B joints under progressive collapse scenarios are limited, especially under dynamic loading conditions. In addition, modified joint detailing is helpful to mitigate progressive collapse through the development of catenary action but relevant research studies are limited. Meanwhile, component-based models for composite joints are useful to facilitate analyses of beam-column joints subjected to progressive collapse scenarios. However, models considering both slab and strain-rate effect have not been proposed.

Therefore, the current research study aims to investigate the current design method for composite beam-column joints through experimental tests under both quasi-static and impact loading conditions. Enhanced connection detailing will be proposed. Based on the experimental study, a component-based modelling approach capable of simulating composite slab and the strain-rate effect will be proposed.

CHAPTER 3: BEHAVIOUR OF BARE STEEL BEAM-COLUMN JOINTS SUBJECTED TO QUASI-STATIC AND IMPACT LOADS

3.1 Introduction

The contribution of beam-column joints to global structural resistance is of significant importance when steel structures are subjected to extreme loads such as impact and explosion, which may lead to progressive collapse of the whole structure. This chapter describes an investigation of the behaviour of bare steel beam-column joints subjected to quasi-static and impact loads. A middle column removal scenario was chosen as a simplification of progressive collapse scenario based on UFC 4-023-03 (DOD 2013). A bare steel beam-column joint sub-structure was extracted from a prototype steel frame structure designed based on Eurocode 3 (2005a) and AISC 360-10 (AISC 2010). After removal of the middle column, the remaining beam-column sub-structures were tested under both quasi-static and impact loads. Two types of beam-column connection, namely, fin plate (FP) and welded unreinforced flange with bolted web (WUF-B) were considered. Structural behaviour of specimens with both connection types was investigated and compared under different loading conditions. To further understand the proportion of energy absorbed by the beam-column joints, parametric studies were conducted by finite element models built using LS-DYNA. Based on both experimental and numerical studies, two indices were proposed to evaluate structural performance of beam-column joints under impact loads.

3.2. Experimental study

3.2.1 Test specimens and material properties

In the experimental programme, a prototype steel structure was designed against gravity loads based on Eurocode 3 (2005a) and AISC 360-10 (AISC 2010) as shown Fig. 3.1. Typical dead load and live load of the building are 5.1 kN/m^2 and 3 kN/m^2 , respectively. The centre-to-centre distances of columns are 6 m and 9 m, in two

orthogonal directions. Beams and columns are UB 406×140×39 and UC 610×229×140, respectively. One middle column was ‘forcibly removed’ as prescribed by UFC 4-023-03 (DOD 2013). The prototype sub-structures were extracted and scaled down by half. Table 3.1 provides detailed information of the tested joint specimens. A total of five joint specimens were tested and categorised into two groups, namely, quasi-static and impact groups. Each group consisted of two types of connection, i.e. FP and WUFB connections. These two types of joints were selected due to their common applications in steel frames, viz. FP for conventional pinned joints and WUF-B for rigid or semi-rigid joints. To be compared systematically, the specimens were designed in such a way that the connection configurations, i.e. bolts, fin plates, and I-shaped beams and columns were kept the same. As shown in Table 3.1, the specimens were differentiated by connection types such as FP for fin plate or W for WUF-B. Static specimens ended with ‘static’ while impact specimens were identified by drop-weight M and drop-height H. For instance, FP6-M530H3 denotes that the specimen has fin plate connection, thickness of the fin plates is 6 mm, mass of the drop-weight hammer is 530 kg and drop-height is 3 m. Grade S355 steel was used for universal beams, columns, and other steel plates. For fin plates, mild steel Grade S275 was used to obtain a ductile failure mode. To prevent any brittle failure of bolts, Grade 10.9 M20 bolts with 280 kNm pre-torque were used for the web connection of the I-shaped beams. Compared to FP connection, full penetration butt welds were employed to connect the top and bottom beam flanges to the column flange of WUF-B connection (specimens W-static and W-M830H3). To facilitate the welding process, one-quarter circle holes with a radius of 18 mm were drilled in the beam web areas close to the flanges. Besides, four pieces of column web stiffeners were used in WUF-B connection to prevent local buckling of the column web. By keeping the same beam section and the web connection, contribution of welded beam flanges to the column flange can be investigated through comparing the behaviour of these two types of specimens. Fig. 3.2 shows the detailing of the FP and WUF-B specimens. Table 3.2 summarises the standard tensile coupon tests conducted on steel materials.

Fig. 3.1 Floor plan of prototype office building (unit: mm)

Table 3.1 Summary of test specimens

Loading scenario	ID	Thickness of fin plate (mm)	Drop-weight (kg)	Height (m)	Impact velocity (m/s)	Momentum (kgm/s)	Energy (kJ)
Quasi-static	FP-static	6	/	/	/	/	/
	W-static	6	/	/	/	/	/
	FP6-M530H3	6	530	3.015	7.389	3916	14.5
Impact	FP10-M530H3	10	530	3.015	7.305	3871	14.4
	W-M830H3	6	830	2.993	7.235	6005	21.7

Nomenclature: FP - Fin plate; W - Welded unreinforced flanges and bolted web (WUF-B); M - Mass of impact hammer, kg; H - Drop-height, m

Fig. 3.2 Detailing of specimens: (a) FP connection; (b) WUF-B connection

Table 3.2 Material properties of steel

Steel Grade	Material	Yield strength (MPa)	Modulus of elasticity (GPa)	Ultimate strength (MPa)	Fracture strain*
S355	Beam web	420	209	575	0.30
S355	Beam flange	427	199	586	0.24
S275	Fin plate	370	202	513	0.30

*Fracture strain was obtained from proportional coupon gauge length of $5.65\sqrt{S_0}$, where S_0 is the original cross-sectional area of coupons.

3.2.2 Test set-up

A hydraulic actuator with displacement control at 6 mm/min was employed to apply a quasi-static load to beam-column specimens as shown in Fig. 3.3. The actuator has a capacity of 500 kN. The quasi-static load was monotonically applied on the middle column joint for a ‘push-down’ test. On the left side, a strong A-frame was used to simulate a pinned support while on the right side the specimens were connected to a pinned support reacting against a strong wall. The two pinned supports were used to simulate the inflexion points located roughly at the middle span of each beam after the middle column was removed. The beam span of the prototype structure was 3668 mm, smaller than a typical full-scale steel frame, to fit within the limited space in the laboratory. In actual structures, the inflexion points would change during the load redistribution process. However, the purpose of this test was to investigate structural behaviour of beam-column joints subjected to combined axial and shear forces, and bending moment. The test set-up was validated by tests conducted by Yang and Tan (2013a).

Fig. 3.4 shows the front view of the impact test set-up. An MTS drop-weight test machine was used to apply impact loads in the test programme. The basic drop-weight of the hammer system was 530 kg including a load cell system. The drop-weight could be increased to 830 kg by adding 10 pieces of steel plates each weighing 30 kg. The free movement height of the hammer was up to 4 m, but the drop-height was limited to 3 m in this study. The impact hammer was centred to the axis of the middle column joint.

Fig. 3.3 Front view of quasi-static test set-up

Fig. 3.4 Front view of impact test set-up

Fig. 3.5 Impact test set-up in three-dimensional perspective

3.2.3 Instrumentation

During the quasi-static test, TML strain displacement transducers (LT) were used to capture displacements of the specimens. The locations of all transducers are shown in Fig. 3.6 and they were symmetrically placed about the middle joint. Two transducers were located at both sides of the bottom plate of the middle column joint to monitor the joint displacement and rotation. At each side of the beam, two transducers were placed to monitor vertical displacements. The layout of strain gauges is shown in Fig. 3.7. At each side of the beam, strain gauges were placed at two cross sections, viz. section 1-1 close to the middle joint and section 2-2 at 400 mm away from the end plate of pinned supports to monitor internal forces developed in the beam. The axial load from the actuator was also recorded by a load cell connected to the stroke of the 500 kN actuator. All the sensors were connected to a multifunctional TML data logger TDS-530 and recorded at an interval of 10 s.

In the impact test, to capture the rapidly-changing displacement, one laser sensor was placed at the location of the middle column joint. The layout of strain gauges was the same as that of the quasi-static tests and is shown in Fig. 3.7. Only the right half specimen corresponding to the rectangular zone in Fig. 3.6 is shown due to symmetry. The impact force generated by the collision of the hammer head with the specimen was obtained by a Kistler type 9393 load cell with 1000 kN capacity. Due to limited channels in the data logger system, two data acquisition systems were used simultaneously, viz. DEWE SIRIUS STG DSUB-9 and TML multi-recorder. Each system had 16 channels and the impact test data were recorded at a sampling rate of 100 kHz. To eliminate high-frequency environment noises, low pass filters at 300 Hz were applied in both systems.

Fig. 3.6 Layout of displacement transducers for quasi-static test

Fig. 3.7 Layout of strain gauges at the right side of specimens for quasi-static and impact tests

3.2.4 Test results and discussions

The main structural behaviour obtained from the tests included load, displacement, strain, strain rate, internal force and failure mode.

Fig. 3.8 shows the development of static load versus displacement of the middle column joint of specimens FP-static and W-static. Since W-static had a stronger beam-column connection, a greater load (Fig. 3.8(b)) could be resisted compared to FP-static (Fig. 3.8(a)). W-static was more ductile when comparing final beam chord rotation at the failure point (0.23 rad versus 0.17 rad for FP-static). The beam chord rotation was defined as the ratio of vertical displacement D over length of beam L as shown in Fig. 3.9. For W-static, the applied vertical load was resisted by flexural action at the beginning but catenary action at large deformation stage (Fig. 3.8(b)).

The proportion of these two parts is shown in Fig. 3.8 as well. For FP-static, catenary action started developing at a displacement of 75 mm. After 100 mm, catenary action increased to be the sole contributor to resist the vertical load. Vertical load of specimen W-static was resisted by flexural action at the beginning until a displacement of 154 mm. Then the bottom beam flange fractured and catenary action started to develop. After a displacement of 210 mm, catenary action became the sole contributor. When comparing specimens FP-static and W-static, it can be seen that through welding the bottom and top beam flanges to the column flange, W-static had a much greater load-carrying capacity and was more ductile. Furthermore, W-static could develop greater catenary action (Fig. 3.10(a)) and absorb much more energy (Fig. 3.10(b)) than FP-static. The strain energy stored in the WUF-B joint (21.5 kJ) at the bottom flange fracture point (corresponding to the first drop of load in Fig. 3.8(b)) was 2.4 times the maximum strain energy in the FP joint (8.9 kJ). At the final failure point, it was about 5 times more (48.9/8.9). For W-static as shown in Fig. 3.8(b), the load resisted by flexural action became negative after the bottom beam flange fractured, indicating that bending moment had reversed direction. This phenomenon can be explained by conducting a free-body analysis of W-static as shown in Fig. 3.11. Before fracture occurred, resultant force in the beam was in tension and acted at a point below the centroidal axis as shown in Fig. 3.11(a). The joint was resisting a sagging moment and the bottom beam flange was in tension. However, tension force provided by the bottom flange was lost due to fracture so that the resultant tension force moved upwards and acted at a point above the centroidal axis as shown in Fig. 3.11(b). Therefore, the joint was subjected to hogging moment, which was detrimental to resist the applied load.

Fig. 3.8 Load versus vertical displacement curves: (a) FP-static; (b) W-static

Fig. 3.9 Calculation of chord rotation

Fig. 3.10 Comparison between specimens FP-static and W-static: (a) Beam axial force; (b) Energy absorption

Fig. 3.11 Free-body analysis of W-static: (a) Before fracture of the bottom beam flange; (b) After fracture

Specimens FP6-M530H3 and FP10-M530H3 had FP connections of 6 mm and 10

mm plates, respectively, while specimen W-M830H3 had a WUF-B connection. After testing FP6-M530H3, it was found that failure only occurred in the fin plates while the bolts and the beam webs remained intact. Then the fin plates were sawn off and replaced by a pair of thicker 10 mm fin plates. The new specimen was named FP10-M530H3 and was tested under the same impact load. All the three specimens (Table 3.1) were subjected to impact loads. The impact forces- and displacement-time history are shown in Figs. 3.12(a) and 3.13, respectively. Since the mass and the height of the drop hammer for FP6-M530H3 and FP10-M530H3 were kept the same, the measured impact forces were generally the same as shown in Fig. 3.12(a). When the time axis was expanded to 5 ms, the impact force of the first collision was almost identical between FP6-M530H3 and FP10-M530H3, as shown in Fig. 3.12(b). It should be noted that each collision in the impact test consisted of three spikes. The first spike occurred when the hammer impacted the joint. The following two spikes were induced by stress waves due to horizontal restraint of the specimen. In the test conducted by Grimsmo et al. (2015), horizontal restraint was not applied so that only one spike was observed for each collision. After the second collision, the middle column joint of FP10-M530H3 moved downwards more slowly than that of FP6-M530H3, since the former had a stronger connection due to thicker fin plates. In Fig. 3.14, a greater beam axial force was developed in FP10-M530H3 as well.

Specimen W-M830H3 had a welded connection to the column flange and thus it was much stiffer and stronger than the fin plate specimens FP6-M530H3 and FP10-M530H3. A greater drop-weight of 830 kg was employed instead of 530 kg. A greater peak impact force (999 kN) and greater duration (1.49 ms) were observed as shown in Fig. 3.15. A stable period was observed between 22 ms and 36 ms for W-M830H3, which was also found in the impact test conducted by Fujikake et al. (2009). Due to the stable period, the impact momentum was much larger than that of the FP specimens. However, since the load-carrying capacity of the welded specimen was much greater, its structural response (in terms of displacement of middle column joint) was smaller than the FP specimens. Complete fracture of the connection was not observed in the welded specimen. The residual displacement caused by plastic deformation was nearly 110 mm. Since vertical displacement of the middle column in W-M830H3 was smaller than those of the two FP specimens, catenary action was

not fully developed as shown in Fig. 3.14. The peak beam axial force was smaller than those of the two FP specimens.

Fig. 3.12 Development of impact forces of FP specimens: (a) Complete curves; (b) Time axis expanded to 5 ms

Fig. 3.13 Vertical displacement of middle column versus time curves in the impact test

Fig. 3.14 Development of beam axial force in the impact test

Fig. 3.15 Development of impact force of WUF-B specimen

A comparison of the development of beam axial force for both quasi-static and impact tests is shown in Fig. 3.16. Because FP6-M530H3 failed by fracture of the fin plate, FP10-M530H3 was compared with FP-static as they both failed in the beam webs. For FP specimens, catenary action was fully mobilised so that the beam axial force developed completely until failure occurred at the middle joint. For both specimens subjected to quasi-static and impact loads, the peak axial forces were similar in magnitude as shown in Fig. 3.16(a). However, the slope for FP10-M530H3 was significantly greater, which means that when subjected to the impact load, the specimen tended to be stiffer. For W-M830H3 as shown in Fig. 3.16(b), the impact load was smaller than its capacity so that it did not fail and catenary action was not fully mobilised. However, the slope of the beam axial force in W-M830H3 subjected to impact load was significantly greater as shown in Fig. 3.16(b). A similar phenomenon was observed when comparing the beam bending moment in Fig. 3.17(b). In Fig. 3.17(a), the slope of FP-static cannot be compared because the beam bending moment was negligible. Bending moment of FP10-M530H3 (Fig. 3.17(a)) resulted from free vibration after the impact.

Fig. 3.16 Comparison of beam axial forces between specimens subjected to impact and quasi-static load: (a) FP connection; (b) WUF-B connection

Fig. 3.17 Comparison of beam bending moments between specimens subjected to impact and quasi-static load: (a) FP connection; (b) WUF-B connection

Figs. 3.18-22 show the failure modes of the five specimens. When subjected to quasi-static loads, FP-static failed in block shear of the right beam web as shown in Figs. 3.18(a) and (b). However, failure of the WUF-B joint in W-static was initiated by tensile fracture of the left bottom beam flange (Figs. 3.19(a) and (b)). Therefore, the applied load dropped dramatically as shown in Fig. 3.8(b) when attaining a peak load of 154 kN. After that, the beam web in W-static started carrying the load so that the applied load increased again. In this stage, catenary action played a key role in resisting the applied load. Final failure was caused by tensile fracture of the left top welded beam flange as shown in Fig. 3.19(b). It should be noted that the second peak in the load was even larger than the first one in Fig. 3.8(b), which means catenary action could resist more load than flexural action for W-static. By strengthening the FP connection through increasing the fin plate thickness, failure mode was changed

from tensile fracture of the fin plate (Figs. 3.20(a) and (b)) to block shear of the beam web (Figs. 3.20(c) and (d)) when comparing FP6-M530H3 with FP10-M530H3. Correspondingly, the beam axial force also proportionally increased in FP10-M530H3 as shown in Fig. 3.14. Although plastic deformation formed after the impact and a residual displacement of 113 mm was observed in Fig. 3.13, W-M830H3 was much stiffer and stronger so that its connection remained intact in Fig. 3.21(a). Only a small crack occurred at the left bottom beam flange as shown in Fig. 3.21(b), which was caused by tension from bending.

Fig. 3.18 Failure of specimen FP-static: (a) Beam-column joint; (b) Back view of right connection

Fig. 3.19 Failure of specimen W-static: (a) Beam-column joint; (b) Left connection

Fig. 3.20 Failure of FP specimens subjected to impact load: (a) Left beam of FP6-M530H3; (b) Left fin plate of FP6-M530H3; (c) Left beam of FP10-M530H3; (d) Left fin plate of FP10-M530H3

Fig. 3.21 Failure of specimen W-M830H3: (a) Beam-column joint; (b) Right bottom beam flange

In brief, the WUF-B joint had a greater load-carrying capacity and was more ductile than the FP joint when subjected to quasi-static loads (Figs. 3.8 and 3.9). Even if no fracture was allowed in the design of WUF-B joint, relying on flexural action alone could provide 2.1 times (174.6 kN versus 82.9 kN for FP joint) the load-carrying capacity of the FP joint (Fig. 3.8). The WUF-B joint could also store greater strain energy than the FP joint when subjected to the quasi-static load (Fig. 3.10(b)). Catenary action developed in the FP joints was similar in magnitude when subjected to either impact or quasi-static loads (Fig. 3.16(a)). For WUF-B joints, catenary action was only partially mobilised in the impact test due to a small displacement

caused by small initial impact energy (Fig. 3.16(b)). Besides, strengthening FP connection by increasing the fin plate thickness resulted in greater catenary action (Fig. 3.14), though only a marginal difference in the impact force was observed (Fig. 3.12).

3.3. Numerical study

To obtain a better understanding of structural behaviour of these two types of joints, finite element (FE) models were built and validated against test data.

3.3.1 Modelling techniques

The commercial software LS-DYNA (LSTC 2007) was chosen to build finite element models because it is commonly used in dynamic explicit analyses. True stress-strain constitutive curves for each material must be transformed from engineering stress-strain curves obtained from uni-axial material coupon tests. It is noteworthy that this transformation must be applied before necking occurs since the assumption of uniform extension along the steel coupons will be invalid beyond this point. After necking, failure criterion was defined as a linear increasing curve with a failure point. Typical values of failure strain for high strength bolts and web plates of I-shaped steel cross sections were 0.1 (10%) and 0.3 (30%), respectively. Elements with strain greater than the failure criterion were removed automatically.

Plastic kinematic model with isotropic hardening was used in the simulations. This material model adopts Cowper-Symonds model to consider the strain-rate effect of steel material, which scales the yield strength by the strain-rate dependent factor as shown below:

$$\sigma_Y = \left[1 + \left(\frac{\dot{\varepsilon}}{C}\right)^{\frac{1}{p}}\right](\sigma_0 + \beta E_p \varepsilon_p^{eff}) \quad (3-1)$$

where σ_0 is the initial yield strength, $\dot{\varepsilon}$ is the strain rate, C and p are the Cowper-Symonds strain-rate parameters, ε_p^{eff} is the effective plastic strain, and E_p is the plastic hardening modulus which is given by

$$E_p = \frac{E_{tan}E}{E - E_{tan}} = \frac{700 \times 205000}{205000 - 700} = 702MPa \quad (3-2)$$

According to the dynamic axial test of steel specimens under similar strain rate (Abramowicz and Jones 1984), C and P were set at 6844 and 3.91, respectively.

Eight-node solid element S164 was used in the three-dimensional model. One-point reduced integration was employed for faster element formulations. For thin-walled parts such as I-shaped columns and beams, fin plates and other steel plates, at least two layers of solid elements were used in the thickness direction. In other directions, the mesh size was generally kept the same to form cuboid shape elements. At locations of bolt holes, at least 16 divisions were used to form a near smooth circle. The thick plate connecting the I-beam and the pinned support was quite rigid with negligible deformation so it was meshed by tetrahedrons with diverse sizes at various locations. The typical element ranged from 5 to 60 mm. At locations where there were contact surfaces such as the hammer and the bracket holes (Fig. 3.22), mesh size was refined to be as small as 2 mm. Although high strength bolts were used to prevent any bolt failures, they were meshed with a fine size ranging from around 2 to 5 mm, to capture deformations of bolt shanks. Nominal diameter of the bolts was used so that threads were not modelled.

Automatic single surface contact was used to represent possible contact surfaces including: 1) bolt shanks and fin plates; 2) bolt shanks and beam webs; 3) fin plates and beam webs; 4) bolt heads and fin plates; 5) bolt nuts and beam webs; 6) pins and bracket holes. Automatic single surface contact is efficient for self-contacting problems or large deformation stage where general areas of contact are not known beforehand. However, contact forces could not be obtained. Therefore, surface-to-surface contact (automatic contact options) was used between the impact hammer and the top surface of the middle column joint. Surface-to-surface contact is the most general type of contact as it is commonly used for bodies that have arbitrary shapes and with relatively large contact areas. A friction coefficient of 0.3 was used for all contact options. Welds were simulated as surface-to-surface contact with tie option so that failure of welds was not considered. The reason is that failure of welds was not critical in this study and good agreement between simulations and test results

could be achieved without modelling the welds.

3.3.2 Validation

To validate the modelling techniques, specimens FP6-M530H3, FP10-M530H3, and W-M830H3 subjected to impact loads were modelled in LS-DYNA. Fig. 3.22 shows a typical FP model. The load and the displacement-time history from test and modelling results are shown in Figs. 3.23-25. For FP specimens, both impact collisions were captured well by simulations as shown in Figs. 3.23(a) and 3.27(a), respectively. The displacement-time history curves in the simulation agreed well with the tests as shown in Figs. 3.23(b) and 3.24(b). In numerical models, each pinned support in Fig. 3.4 was simplified as one pin and one bracket as shown in Fig. 3.22 to save computational resources so that the second and the third spikes of each collision in the test were not well simulated. Such simplification was reasonable because the structural behaviour, including the peak load and the displacement-time history could be well simulated by numerical models. Good agreement with experimental data was also obtained in WUF-B specimens (Figs. 3.25(a) and (b)). Compared with photographs taken from the test, numerical models could simulate tensile fracture of the fin plate (Fig. 3.26(a)) and block shear failure of the beam web (Fig. 3.26(b)). Since no failure occurred in specimen W-M830H3, only the load- and displacement-time history are compared in Fig. 3.25.

Fig. 3.22 Finite element model of FP specimen

Fig. 3.23 Comparison between test and numerical analysis results of specimen FP6-M530H3: (a) Load versus time; (b) Displacement of the middle column joint versus time

Fig. 3.24 Comparison between test and numerical analysis results of specimen FP10-M530H3: (a) Load versus time; (b) Displacement of the middle column joint versus time

Fig. 3.25 Comparison between test and numerical analysis results of specimen W-M830H3: (a) Load versus time; (b) Displacement of the middle column joint versus time

Fig. 3.26 Comparison between test and numerical analysis results in failure mode: (a) Left fin plate of FP6-M530H3; (b) Left beam web of FP10-M530H3

3.3.3 Parametric studies

Table 3.3 summarises all the numerical models built using LS-DYNA. For each type of connections, seven models were employed to study governing parameters including mass, velocity, momentum and energy. For each parameter, three variables based on the value used in the experiments were applied so that a general trend of the parameters could be observed. To differentiate finite element models from test specimens, notations of simulation models ended with an 's'. Results of all the models, including the first peak load, residual displacement, and strain energy absorption are summarised in Table 3.3. Models FP-M530H3s and W-M1660H3s failed completely so that their residual displacements could not be obtained for the corresponding drop-weight and the velocity.

Mass

To study the influence of the mass of the impactor, three mass levels were applied to each type of joint models, namely, 216.4, 265, and 530 kg for FP models, and 415, 830, and 1660 kg for WUF-B models while the height was kept as 3 m in both joint models. As shown in Table 3.3, when the mass was increased by 22.5% and 145% for FP-M265H3s and FP-M530H3s, respectively, compared to FP-M216H3s, marginal increases of the first peak load were observed (1.4% for FP-M265H3s and 5.4% for FP-M530H3s). Compared to FP-M216H3s, the impactor mass of FP-M265H3s was increased by 22.5% so that 10.4% increase of residual displacement was observed, which was much greater than the increase of the first peak load.

Therefore, the mass of the impactor has a greater influence on residual displacement than the first peak load of FP joints. A similar phenomenon was observed for WUF-B models. As shown in Table 3.3, by increasing the mass two- and four-folds, only 2.1% and 3.2% increases in the first peak load were observed for W-M830H3s and W-M1660H3s compared to W-M415H3s. However, the residual displacement increased significantly (2.2 times) when the mass was doubled for W-M830H3s compared to W-M415H3s. Thus, the mass of the impactor governs residual displacement of both FP and WUF-B joints but has little influence on the impact force. When four-fold mass was used for W-M1690H3s, the connection was damaged so that no residual displacement was obtained.

Velocity

Impact velocity is determined by drop-height so that they are regarded as the same parameter in this study. To study the influence of the velocity of the impactor, three different heights were applied to each type of joint models, namely, 0.5, 1.5, and 2 m for FP models and 0.75, 1.5, and 3 m for WUF-B models. Compared to FP-M530H0.5s (Table 3.3), while maintaining the same mass and increasing the velocity of the impactor by 73.2% and 100% for FP-M530H1.5s and FP-M530H2s, increases of the first peak load were 59.9% and 79.1%, respectively. Residual displacements increased 53.0% and 67.5% for FP-M530H1.5s and FP-M530H2s, respectively, compared to FP-M530H0.5s. Similarly, by increasing 41.4% and 100% of the velocity of the impactor for W-M1660H1.5s and W-M1660H3s (compared to W-M1660H0.7s), increases of the first peak load were 37.2% and 77.4%. Residual displacement increased by 109.9% for W-M1660H1.5s compared to W-M1660H0.7s. Therefore, increasing the velocity can result in a significant increase of both the first peak load and the residual displacement for both types of joints.

Compared to increases of the peak load and the residual displacement caused by the mass, increasing the velocity has a much greater influence on structural behaviour of both FP and WUF-B joints. The reason is that kinetic energy increases linearly with mass but increases quadratically with velocity. Therefore, varying the velocity of the impactor is more effective than controlling its mass to study the structural behaviour.

Momentum

From impact mechanics, momentum is frequently used as a parameter to represent the behaviour of two impact objects. Models FP-M153H6s, FP-M216H3s, and FP-M530H0.5s in Table 3.3 have the same impact momentum of 1659 kgm/s. Compared to FP-M153H6s, the impact energies of FP-M216H3s and FP-M530H0.5s decrease rapidly when the mass increases due to proportional decrease of impact velocity for the same momentum. Decreases of the first peak load were 16.7% and 56.7% for FP-M216H3s and FP-M530H0.5s compared to FP-M153H6s. Besides, decreases of residual displacement were 6.9% and 26.3%. For models W-M587H6s, W-M830H3s, and W-M1660H0.7s as shown in Table 3.3, the same phenomenon was observed. Decreases of the first peak load were 15.3% and 51.8% for W-M830H3s and W-M1660H0.7s compared to W-M587H6s. Besides, decreases of residual displacement were 30.7% and 65.9%. Therefore, imparting the same momentum cannot ensure the same structural behaviour.

Energy

Models FP-M265H3s, FP-M530H1.5s, and FP-M1060H0.8s as shown in Table 3.3 have the same initial impact energy of 7.8 kJ. Compared to FP-M265H3s, decreases of the first peak load for FP-M530H1.5s and FP-M1060H0.8s were 18.0% and 38.1%, respectively. In contrast, 8.8% and 20.2% increase of residual displacement were observed in FP-M530H1.5s and FP-M1060H0.8s compared to FP-M265H3. The same phenomenon was observed in W-M415H6s, W-M830H3, and W-M1660H1.5s as shown in Table 3.3. Compared to W-M415H6s, decreases of the first peak load for W-M830H3 and W-M1660H1.5s were 14.3% and 33.0%, respectively. In contrast, 11.0% and 14.8% increase of the residual displacement were observed. Compared to momentum, smaller differences of the peak load and residual displacement were observed when impact energy was kept the same. Besides, a greater impact energy resulted in a larger absorption of energy as shown in Table 3.3. Therefore, impact energy governs structural behaviour of all the joint models.

3.3.4 Mathematical explanations of governing parameters

To gain a better understanding of the four parameters, a simplified derivation of peak impact force and residual displacement of joint specimens is conducted as follows:

Fig. 3.27 Velocities of impactor and specimen before and after impact

As shown in Fig. 3.27, the specimen is simplified as a lumped mass M_s supported by a spring representing equivalent stiffness and deformation of the test specimen. Duration of the steel-to-steel impact Δt is around 1 ms from the test results and it is very short. Therefore, it can be assumed that the force of the spring cannot be activated and momentum of the impact is conserved. From momentum conservation, Equation (3-3) can be obtained as follows:

$$M_i V_i + M_s V_s = M_i V_i' + M_s V_s' \quad (3-3)$$

where M_i is the mass of the impactor; V_i is the initial velocity of the impactor; V_i' is the velocity of the impactor after impact; M_s is the equivalent lumped mass of the specimen; V_s is the initial velocity of the specimen and is equal to zero; V_s' is the velocity of the specimen after impact.

Coefficient of restitution c_r of the system is defined as follows:

$$c_r = \frac{V_s' - V_i'}{V_i} \quad (3-4)$$

Assuming $V_i' = c V_s'$ where c is a constant for each specimen, relationship between c and c_r can be written as:

$$c_r = (1 - c) \frac{V'_s}{V_i} \quad (3-5)$$

The ratio c represents the quantity of exchange of velocities immediately after impact. Based on the FE analysis, for FP models, c is about 0.6 while that for WUF-B models is about 0.7. Therefore, it is assumed to be constant for each specimen. It should be noted the derivation served as an explanation for the influence of parameters. Such an assumption will be invalid if one wants to calculate the value of energy and peak impact force using the equations proposed.

Shape of the impact impulse is assumed to be triangular and is equal to the change of the momentum of the impactor and the specimen as follows:

$$I = \int F(t)dt = \frac{1}{2}F_p\Delta t = M_i(V_i - V'_i) = M_sV'_s \quad (3-6)$$

where $F(t)$ and F_p are the impact force and the peak impact force, respectively; Δt is the duration of impact.

Combined with Equation (3-3), impact impulse I in Equation (3-6) can expressed as follows:

$$I = \frac{1}{2}F_p\Delta t = \frac{M_iM_s}{cM_i + M_s}V_i \quad (3-7)$$

Mass

When solving Equation (3-7), peak impact force F_p can be expressed as follows:

$$F_p = \frac{2}{\Delta t} \underbrace{\frac{M_s}{c + M_s/M_i}}_{mass} \underbrace{V_i}_{velocity} \quad (3-8)$$

It can be seen that F_p increases with an increase of the mass of the impactor M_i , which was observed in the parametric study in Chapter 3.3.

Residual displacement δ represents energy absorption of the joint through plastic deformation. Therefore, δ is assumed to be proportional to kinetic energy of the impactor and can be expressed as:

$$\delta = k \left(\frac{1}{2} \underbrace{M_i}_{\text{mass}} \underbrace{V_i^2}_{\text{velocity}} \right) \quad (3-9)$$

where k represents energy absorption ratio.

Residual displacement increases with an increase of the mass of the impactor M_i , which agrees with the parametric study.

Velocity

From Equations (3-8) and (3-9), it can be seen that both peak impact force F_p and residual displacement δ increase with an increase of the velocity of the impactor, which corroborates well with the parametric study.

Momentum

Equations (3-8) and (3-9) can also be expressed as Equations (3-10) and (3-11), respectively. It can be seen that although Equations (3-10) and (3-11) involve momentum, both the peak impact force and the residual displacement decrease with an increase of the mass of the impactor M_i , which was also observed in the parametric study.

$$F_p = \frac{2}{\Delta t} \frac{M_s}{\underbrace{cM_i + M_s}_{\text{mass}}} \underbrace{M_i V_i}_{\text{momentum}} \quad (3-10)$$

$$\delta = k \frac{1}{\underbrace{2M_i}_{\text{mass}}} \left(\underbrace{M_i V_i}_{\text{momentum}} \right)^2 \quad (3-11)$$

Energy

From Equation (3-8), it is clear that residual displacement increases with an increase of kinetic energy of the impactor, which was also observed in the parametric study.

3.3.5 Deformation and energy ratio

Dynamic increase factor for loads or displacement increase factor based on the energy method proposed by Izzuddin et al. (Vlassis et al. 2006, Izzuddin et al. 2008, Vlassis et al. 2008, Izzuddin and Nethercot 2009) was not applicable for impact test

since the impact force consisted of a series of spikes instead of a specific value as shown in Figs. 3.24(a) and 3.25(a). Jones (2010) proposed an energy absorption effectiveness factor for axially-loaded steel cross sections. Based on the parametric study, energy of impactor governs structural behaviour of the joints subjected to impact loads. Therefore, in current study, two indices are defined based on energy of impactor when evaluating the joint behaviour subjected to impact loads, viz. energy ratio and deformation ratio. Energy ratio is defined as the strain energy normalised by the total energy from the impactor, which is simply the kinetic energy when the hammer just impacts the joint. Energy ratio stands for the quantity of energy absorption in each model. The residual displacement, which is indicative of the strain energy absorbed by each beam-column joint, can be obtained from the numerical results and is listed in Table 3.3. For example, Fig. 3.28 shows the energy versus displacement curve of specimen W-static in the quasi-static test series. Energy absorption in model W-M830H3s can be obtained from Fig. 3.28 by referring to a specific residual displacement. If the procedure is reversed and assuming that total kinetic energy is absorbed in the form of strain energy by the model, then an equivalent displacement can be obtained as shown in Fig. 3.28. Deformation ratio is defined as the residual displacement obtained from the numerical analysis normalised by the equivalent displacement obtained from Fig. 3.28. Compared with energy ratio, deformation ratio is an index representing the resistance of each model to plastic deformations subjected to impact loads.

Fig. 3.28 Energy versus displacement curve of W-static

The average values of energy ratio for FP and WUF-B models are 0.268 and 0.516

(calculated from Table 3.3), respectively, showing that only 26.8% of kinetic energy can be absorbed by FP joints while 51.6% is absorbed by WUF-B joints. Therefore, WUF-B joints had better energy absorption capacity. The coefficients of variance were 0.338 and 0.201, respectively. When using deformation ratio, the average values of FP and WUF-B models are 0.672 and 0.514 (calculated from Table 3.3), respectively. Assuming total energy is absorbed and transformed to plastic deformation, then 67.2% of deformation is formed in FP joints, while 51.4% in WUF-B joints, showing that the latter have better resistance to plastic deformations than the former. The coefficients of variance were greatly reduced to 0.099 and 0.059, showing that deformation ratio was more helpful as an indicator of structural behaviour than energy ratio.

In brief, the velocity of the impactor has a great influence on structural behaviour of both FP and WUF-B joints compared to the mass. Furthermore, initial impact energy instead of momentum determines structural behaviour of all the joint models. To predict residual displacement of both FP and WUF-B joints, deformation ratio is more consistent than energy ratio. Besides, deformation ratio of FP models is greater than that of WUF-B models.

Table 3.3 Summary of numerical models

Type	ID	Mass (kg)	Height (m)	Impact velocity (m/s)	Momentum (kgm/s)	Initial impact energy (kJ)	First peak load (kN)	Residual displacement (mm)	Strain energy absorption (kJ)	Energy ratio	Equivalent displacement (mm)	Deformation ratio
FP	FP-M530H3s	530	3.000	7.668	4064.1	15.6	1083.8	/	/	/	/	/
	FP-M530H2s	530	2.000	6.261	3318.3	10.4	958.0	215.7	3.2	0.309	/	/
	FP-M530H1.5s	530	1.500	5.422	2873.8	7.8	855.0	197.0	2.3	0.301	282.3	0.698
	FP-M530H0.5s	530	0.500	3.130	1659.2	2.6	534.8	128.8	0.6	0.248	202.7	0.635
	FP-M265H3s	265	3.000	7.668	2032.1	7.8	1043.0	179.6	1.7	0.218	282.3	0.636
	FP-M216H3s	216.4	3.000	7.668	1659.4	6.4	1028.4	162.7	1.2	0.191	265.1	0.614
	FP-M153H6s	153	6.000	10.844	1659.2	9.0	1235.2	174.8	1.5	0.172	/	/
	FP-M1060H0.8s	1060	0.750	3.834	4064.1	7.8	645.3	219.3	3.4	0.436	282.3	0.777
	W-M830H3s	830	3	7.668	6364.5	24.4	1211.1	113.2	13.9	0.570	217	0.522
	W-M1660H3s	1660	3	7.668	12729.1	48.8	1223.5	/	/	/	401.5	/
WUF-B	W-M415H3s	415	3	7.668	3182.3	12.2	1185.8	50.4	4.4	0.364	102.4	0.492
	W-M1660H1.5s	1660	1.5	5.422	9000.8	24.4	946.1	117.1	14.5	0.595	217.1	0.539
	W-M1660H0.7s	1660	0.75	3.834	6364.5	12.2	689.8	55.8	5.2	0.434	101.1	0.552
	W-M587H6s	587	6	10.844	6365.6	34.5	1429.8	163.4	21.9	0.635	320.1	0.510
	W-M415H6s	415	6	10.844	4500.4	24.4	1413.1	102.0	12.1	0.497	217	0.470

3.4. Summary and conclusions

In this chapter, a series of five bare steel beam-column joints were tested under the middle column removal scenario. Two types of connections, namely, FP and WUF-B connections were investigated under both quasi-static and impact loads. Numerical models in LS-DYNA were built and validated against the three impact test results. Employing validated models, a parametric study was conducted to investigate the governing parameters including mass, velocity, momentum, and energy. Two indices, viz. energy ratio and deformation ratio, were defined to evaluate the behaviour of the beam-column joint subjected to impact load. Based on both experimental and numerical studies, the following conclusions can be drawn:

- (1) The WUF-B joint had a greater load-carrying capacity and was more ductile than the FP joint when subjected to quasi-static loads. The designer is recommended to use semi-rigid or rigid connections for impact loading scenarios.
- (2) Catenary action developed in the FP joints was similar in magnitude when subjected to impact and quasi-static loads. For WUF-B joints, catenary action was only partially mobilised due to smaller displacements caused by initial impact energy.
- (3) Increasing the velocity has a much greater influence on structural behaviour of both FP and WUF-B joints than increasing the mass of the impactor. Moreover, initial impact energy instead of momentum determines structural behaviour of all joint specimens.
- (4) Deformation ratio is a more consistent indicator than energy ratio for predicting the residual displacement for both FP and WUF-B joints. Besides, deformation ratio of FP specimens is greater than that of WUF-B specimens. Deformation ratio is recommended for evaluating performance of beam-column joints subjected to impact loads.

CHAPTER 4: EXPERIMENTAL TESTS OF COMPOSITE JOINTS WITH FIN PLATE CONNECTIONS UNDER A COLUMN REMOVAL SCENARIO

4.1 Introduction

FP connections are one of the most commonly found pin connections in steel beam-column joints and they are widely used in gravity-dominant frames due to its ample erection clearance and excellent safety (AISC 2011, BSCA/SCI 2011) under gravity loads. However, with heightened terrorist threats through the use of explosives or vehicular impacts, it is timely to assess structural adequacy of such connections in civilian buildings. GSA (2013) and UFC 4-023-03 (2013) advocate the alternate load path method based on column removal scenarios to assess rotation capacity of steel beam-column joints. However, the quantitative criteria are based on research studies and findings from seismic rehabilitation design (ASCE 2013). Such design criteria for FP connections need to be evaluated, especially as there is a potential benefit to be gained when incorporating composite slab effect into the FP connections. In this chapter, test results on composite joints with FP connection are presented and the beneficial effect of the composite slab is considered and discussed. A comparison with the bare steel joint FP-static in Chapter 3 is also conducted.

4.2. Test programme

4.2.1 Test specimens and material properties

Five half-scale beam-column joints with FP connections were designed based on Eurocode 3 Part 1-1 and Eurocode 4 Part 1-1 (BSI 2004a, BSI 2005b) and their detailed information is provided in Table 4.1. The specimens follow the nomenclature C75FP-M(S)(R/slot), where C indicates composite slab, 75 represents the slab thickness in mm, FP stands for fin plate connection, M means middle joint while S shows side joint, R represents reduced number of shear studs and slot for

slotted bolt hole. For instance, specimen C75FP-Mslot was a middle joint with 75 mm thick composite slab, fin plate connection and slotted bolt holes in the fin plate. For all the specimens, Grade S355 universal beams (UB 203×133×30) and columns (UC 203×203×71) were used and connected by Grade S275 fin plates and Grade 10.9 M20 bolts. The joint details for all five specimens can be found in Figs. 4.1(a) to (d). Specimen C75FP-M was a middle joint and was subjected to sagging moment as shown in Fig. 4.1(a). Two rows of shear studs with 16 mm diameter were placed at a spacing of 90 mm to connect slab sheeting to the I-beam to achieve full shear connection and composite action. Thickness and width of the slab in C75FP-M were 75 mm and 500 mm, respectively. In the composite slab, 1 mm thick Grade 550 re-entrant profiled steel sheeting was used. A mild steel mesh of $\Phi 6$ at a spacing of 170 mm in both longitudinal and transverse directions was placed on top of the steel sheeting without concrete spacers as an anti-crack steel mesh. The reinforcement served as an anti-crack steel mesh. No additional reinforcing bars were provided so that the slab effect using conventional design could be investigated. Side joint C75FP-S subjected to hogging moment is shown in Fig. 4.1(b). The composite slab was placed under the I-beam so that it was in tension. Specimen C75FP-MR was the same as C75FP-M (Fig. 4.1(a)) except that a reduced number of shear studs were used; one row of 16 mm diameter shear studs at 180 mm spacing was placed so that only partial composite action was obtained. A thicker 100 mm slab was used in C100FP-M as shown in Fig. 4.1(c). Slotted bolt holes (Fig. 4.1(d)) were used in C75FP-Mslot to enhance ductility of FP connection. The dimensions of the specimens are provided in Fig. 4.2 including a rectangular box of the joints as shown in Figs. 4.1(a) to (d).

Table 4.1 Summary of test specimens

ID	Beam, column, fin plate and bolt	Thickness of composite slab (mm)	Joint location	Bending moment	Shear studs
C75FP-M	S355 UC	75	Middle	Sagging	2 rows @ 90 mm
C75FP-S	203×203×71 column	75	Side	Hogging	2 rows @ 90 mm
C100FP-M	S355 UB	100	Middle	Sagging	2 rows @ 90 mm
C75FP-MR	203×133×30 beam	75	Middle	Sagging	1 row @ 180 mm
C75FP-Mslot	S275 150×70 plate	75	Middle	Sagging	1 row @ 180 mm
	Grade 10.9 M20 bolt	75	Middle	Sagging	2 rows @ 90 mm

Nomenclature: C - Composite; FP - Fin plate; M - Middle joint; S - Side joint; R – Reduced number of shear studs; slot - slotted holes

Fig. 4.1 Detailing of specimens: (a) C75FP-M and C75FP-MR; (b) C75FP-S; (c) C100FP-M; (d) C75FP-Mslot (slotted holes); (e) Detailing of steel sheeting

Twelve standard 150 mm diameter by 300 mm length concrete cylinders were tested and the average compressive strength was 36.7 MPa with a standard derivation of 2.8 MPa. For each steel part, including beams and fin plates, three coupons were cut from the parent material and standard tensile tests were conducted with the test

results shown in Table 4.2.

Table 4.2 Material properties of steel

Grade	Steel material	Yield strength (MPa)	Standard derivation of yield strength (MPa)	Ultimate strength (MPa)	Fracture strain*
S355	Beam web	397	9	544	31.1
S355	Beam flange	400	12	541	30.2
S275	Fin plate	394	20	523	30.6
550	Profiled sheeting	580	4	590	12.0
450	Shear Stud [†]	457	-	542	19.8
R	R6	416	6	650	34.0

*Fracture strain was obtained from proportional coupon gauge length of $5.65\sqrt{S_0}$, where S_0 is the original cross-sectional area of coupons.

[†]Data were obtained from mill certificate.

4.2.2 Test set-up

The front view of the test set-up is provided in Fig. 4.2 which is housed in the Protective Engineering Laboratory of Nanyang Technological University, Singapore. A three-dimensional view of the test set-up is shown in Fig. 4.3. One A-frame and one strong concrete reaction wall on opposite sides were used to provide horizontal support for the specimens. As shown in Fig. 4.2, two steel circular hollow section members (CHS 219×12.5) were bolted to two beam ends of the specimens. Two pinned supports were used to connect the A-frame and the reactional wall to these CHS members. The beam span measured between the two pins (Fig. 4.2) was 3668 mm. Only a column stub was placed at the middle beam-column joint. A 500 kN actuator was used to conduct the ‘push-down’ test with displacement control at 6 mm/min.

Fig. 4.2 Front view of the test set-up

Fig. 4.3 Three-dimensional view of the test set-up

4.2.3 Instrumentation

A 500 kN load cell and a displacement sensor were integrated into the actuator so that the applied load and displacement of the middle beam-column joint could be measured. Fig. 4.4(a) shows the detailing of the left CHS member (Fig. 4.2) since the two members were identical. Two cross-sections were mounted with strain gauges to measure internal forces. At each cross-section, four strain gauges were placed in each quadrant of the CHS as shown in Fig. 4.4(b). One steel plate with

holes was welded to each end of the CHS so that it could be connected by bolts to the pinned support (Fig. 4.2). A total of six linear transducers (LT) and two linear variable (LV) differential transducers were placed at various locations along the specimens to measure displacements as shown in Fig. 4.5 (four middle joints) and Fig. 4.6 (the side joint), respectively. Strains of surface concrete, profiled sheeting, reinforcing bars and I-shaped beams were measured at the critical sections. Strain gauge layout of the middle joints is shown in Fig. 4.7(a). Sections 1-1 and 2-2 were attached with strain gauges as shown in Figs. 4.7(b) and (c), respectively. In section 1-1, strain gauges MRR1(2,3) were for three reinforcing bars, MRP1 for steel profiled sheeting, MR1 for the restrained beam flange and C2 for concrete surface. In section 2-2, strain gauges RR1(2,3) were for three reinforcing bars, RP1(2) for steel profiled sheeting, R1(2) and R4(5) for the respective restrained and unrestrained beam flanges and R3 for beam web. The same layout was used for the side joint in Fig. 4.8(a). Layout of sections 1-1 and 2-2 is shown in Figs. 4.8(b) and (c), respectively. All the sensors, including the load cell, displacement transducers and strain gauges were connected to a TML data logger (model TDS-530). Data of the sensors were recorded at an interval of 10 s.

Fig. 4.5 Locations of displacement sensors in middle joints

Fig. 4.6 Locations of displacement sensors in side joint

Fig. 4.7 Layout of strain gauges of middle joint: (a) Front view; (b) Section 1-1; (c) Section 2-2

Fig. 4.8 Layout of strain gauges of side joint: (a) Front view; (b) Section 1-1; (c) Section 2-2

4.3. Test results and discussions

4.3.1 Load-resisting mechanism

Load applied to the specimens was resisted by three actions, viz. compressive arch action (CAA), flexural action (FA) and catenary action (CA). Based on the development of axial force in the specimens, the test consisted of two stages, namely, CAA and CA. At CAA stage, the compressive arch acted between the pins and the middle column as shown in Fig. 4.9(a). Vertical load P was balanced by CAA and shear forces at the two pins (Fig. 4.9(b)). At the right pin location as shown in Fig. 4.9(c), angle (α_2) between the compressive arch and the original horizontal axis was closing when the middle joint was pushed down by the actuator. However, angle (β_2) between the moving specimen axis and the original horizontal axis kept increasing as shown in Fig. 4.9(b). When α_2 became zero as shown in Fig. 4.10(a), CAA finished and CA commenced. At this point, axial force in the composite beam changed from compression to tension as shown in Fig. 4.10(b). It should be noted that FA co-existed at both stages.

Fig. 4.9 Force equilibrium at CAA stage: (a) Deformed geometry of the right side; (b) Schematic diagram; (c) Free body diagram at right pin

Fig. 4.10 Force equilibrium at CA stage: (a) Deformed geometry of the right side; (b) Schematic diagram; (c) Free body diagram at right pin

Actions at the right pin at CAA and CA stages are shown in Figs. 4.9(c) and 4.10(c), respectively. The dimension 620 mm was measured from the pin to the strain gauge section in the CHS members. Axial force N and bending moment M could be obtained from strain gauge readings of the CHS members in Fig. 4.4. Based on equilibrium, axial forces $N1$ and $N2$ and shear forces $V1$ and $V2$ at the two pins in Figs. 4.9(c) and 4.10(c) could be calculated. The shear force V from Equation (4-1) and Fig. 4.9(c) represents FA. At CAA stage, axial force and shear force ($N2$ and $V2$) were not in the same direction as the forces (N and V) calculated from the CHS members because the compressive arch axis did not coincide with the specimen axis as shown in Fig. 4.9(a). Therefore, N and V

should be transformed to the direction of the compressive arch as shown in Fig. 4.9(c). Axial force $N2$ and shear force $V2$ could be calculated from Equations (4-2) and (4-3), respectively. Similarly, at the left pin location, axial force $N1$ and shear force $V1$ could be calculated from Equations (4-4) and (4-5), respectively. When substituting the axial force and the shear force into Equation (4-6) (which was obtained from force equilibrium in Fig. 4.10(a)), Equation (4-7) was obtained. The first item in Equation (4-7) shows the load resisted by CAA while the second item by flexural action. At CA stage, CAA term disappeared so that the axial and the shear forces were the same as those obtained from the CHS members. Therefore, Equation (4-8) could be obtained.

$$V = M/d \quad (4-1)$$

Compressive arch action (CAA) stage:

$$N2 = N \cos(\alpha2 + \beta2) + V \sin(\alpha2 + \beta2) \quad (4-2)$$

$$V2 = -N \sin(\alpha2 + \beta2) + V \cos(\alpha2 + \beta2) \quad (4-3)$$

$$N1 = N \cos(\alpha1 + \beta1) + V \sin(\alpha1 + \beta1) \quad (4-4)$$

$$V1 = -N \sin(\alpha1 + \beta1) + V \cos(\alpha1 + \beta1) \quad (4-5)$$

$$P = \underbrace{(N1 \sin \alpha1 + N2 \sin \alpha2)}_{\text{Compressive arch action}} + \underbrace{(V1 \cos \alpha1 + V2 \cos \alpha2)}_{\text{Flexural action}} \quad (4-6)$$

$$P = \underbrace{(N \cos(\alpha1 + \beta1) + V \sin(\alpha1 + \beta1)) \sin \alpha1 + (N \cos(\alpha2 + \beta2) + V \sin(\alpha2 + \beta2)) \sin \alpha2}_{\text{Compressive arch action}} + \underbrace{(-N \sin(\alpha1 + \beta1) + V \cos(\alpha1 + \beta1)) \cos \alpha1 + (-N \sin(\alpha2 + \beta2) + V \cos(\alpha2 + \beta2)) \cos \alpha2}_{\text{Flexural action}} \quad (4-7)$$

Catenary action (CA) stage:

$$P = \underbrace{(N \sin \alpha1 + N \sin \alpha2)}_{\text{Catenary action}} + \underbrace{(V \cos \alpha1 + V \cos \alpha2)}_{\text{Flexural action}} \quad (4-8)$$

where N is the axial force of CHS members; M is the bending moment of CHS

members; V is the accompanying shear force for M ; d is the distance from the pin to the strain gauged section in the CHS members; $N1$ and $N2$ are respectively the axial forces at the left and the right pins; $V1$ and $V2$ are respectively the shear forces at the left and the right pins; P is the applied vertical load; $\alpha1$ and $\alpha2$ are respectively the angles between the compressive arch and the original horizontal axis on the left and the right sides; $\beta1$ and $\beta2$ are respectively the angles between the moving specimen axis and the original horizontal axis on the left and the right sides.

Fig. 4.11 Load versus displacement of the middle column curves of all the specimens: (a) C75FP-M; (b) C75FP-S; (c) C100FP-M; (d) C75FP-MR; (e) C75FP-Mslot

Specimen C75FP-M

Based on the load-resisting mechanism, load-versus-displacement curves of all the five specimens are shown in Fig. 4.11 with FA, CAA and CA clearly indicated. In Fig. 4.11(a), load applied on C75FP-M increased linearly at the initial stage and then it became nonlinear rapidly at a very small deformation due to plastic deformation of the FP connection and the composite slab. At 70 mm, the load reached the peak value of 45.0 kN. After that, concrete of the composite slab close to the middle column started crushing so that the load decreased slightly. At 150 mm, the load dropped suddenly due to fracture of the left fin plate at the bottom bolt row. The load was maintained until another fracture occurred at the upper bolt row. Then the left fin plate broke into two separate pieces so that the connection was severed. As shown in Fig. 4.11(a), the load was mainly resisted by FA although CAA increased before crushing of concrete occurred at the peak load. After the peak load, CAA decreased and ceased at the first fracture of the fin plate. The maximum load resisted by CAA was 8.0 kN while that by FA was 43.1 kN. At 154 mm, concrete in the composite slab completely crushed and spalled so that its contribution stopped entirely. Therefore, lever arm of the bending moment at the connection decreased so that FA started decreasing. However, CA was mobilised due to large deformation and it kept increasing until complete fracture of the fin plate occurred at 264 mm. The maximum load resisted by CA was 26.6 kN at 260 mm. It should be mentioned that a small drop of the load occurred at 221 mm due to fracture of the profiled steel sheeting. After around 200 mm, load resisted by FA became negative as shown in Fig. 4.11(a). The reason is that the location of resultant axial force in the composite beam moved from the point below the specimen axis (Fig. 4.12(a)) upwards gradually with the development of fracture in the fin plate. Therefore, the resultant bending moment taking from the centroid axis changed from sagging in Fig. 4.12(a) to hogging in Fig. 4.12(b). Correspondingly, load resisted by FA became negative with the development of fracture in the fin plate.

Fig. 4.12 Locations of resultant axial force in the composite beam: (a) Initial stage; (b) After fracture of fin plate

Specimen C75FP-S

Compared to C75FP-M, C75FP-S was a side joint subjected to hogging moment. The composite slab under the beam was subjected to tension. No CAA was observed as shown in Fig. 4.11(b). Similar to C75FP-M, C75FP-S experienced a small linear loading stage and then nonlinear loading occurred due to cracking of concrete. The load reached the first peak value of 28.0 kN at 71 mm. There was a gradual decrease in loading due to fracture of the profiled sheeting in the transverse direction. The applied load was mainly resisted by FA until the side reinforcing bars fractured in tension at 106 mm. Then CA developed with increasing deflection and became the major contributor to resist the applied load, which reached the peak value of 60.8 kN at 276 mm. After that, fracture of the left fin plate occurred and C75FP-S completely failed at 310 mm.

C100FP-M

C100FP-M had a thicker slab (100 mm) compared to C75FP-M (75 mm). No CA was observed as shown in Fig. 4.11(c) because the specimen failed at 160 mm, which was too small for the mobilisation of CA. Since the composite slab was thicker, the peak load of 49.0 kN was greater than that of C75FP-M (45.0 kN in Fig. 4.11(a)), and so were the contributions by CAA and FA. The first fracture of the fin plate occurred at 120 mm, much earlier than that of C75FP-M. Complete fracture of the

fin plate occurred before CA could be mobilised.

C75FP-MR

A reduced number of shear studs was used in C75FP-MR and thus the peak load (42.1 kN in Fig. 4.11(d)) was slightly smaller than that of C75FP-M (45.0 kN in Fig. 4.11(a)). At CAA stage, behaviour of C75FP-MR was generally the same as C75FP-M. However, due to weaker composite action, CA could not be fully mobilised as shown in Fig. 4.11(d). Fracture of the fin plate occurred at 175 mm and developed rapidly until final failure at 189 mm.

C75FP-Mslot

In C75FP-Mslot, slotted holes were used in the fin plates as shown in Fig. 4.11(d) so that bolts could cause plastic deformation of the plate along the slotted holes. The purpose of such design was to maintain resistance of the fin plate connection and to increase ductility. Due to the slotted holes, the applied load associated with initial crushing of concrete was 41.0 kN (in Fig. 4.11(e)), smaller than that of C75FP-M (45.0 kN in Fig. 4.11(a)). CAA was also smaller and ended earlier at 138 mm as shown in Fig. 4.11(e). It can be seen that there was not much difference between the two specimens at CAA stage. However, at CA stage, C75FP-Mslot could resist a much greater load (84 kN in Fig. 4.11(e)) than C75FP-M (45.0 kN in Fig. 4.11(a)) due to contribution of fin plates. The fin plates had not fractured at all at the end of CAA stage so that at CA stage, plastic deformation of the fin plates could develop and provide some resistance. Therefore, CA could be fully mobilised and thus a much greater peak load (83.8 kN) could be resisted. Finally, the applied load dropped rapidly when fracture of the left fin plate took place.

4.3.2 Failure mode

Fig. 4.13 shows the failure mode of C75FP-M typical of middle joints, viz. C100FP-M and C75FP-MR. Fig. 4.13(a) shows the front view of the damaged joint in C75FP-M. The failure mode included crushing and spalling of concrete in the slab and exposure of reinforcing bar (Fig. 4.13(b)), tensile fracture of the profiled steel sheeting from the soffit (Fig. 4.13(c)) and block shear failure of the left fin plate (Fig.

4.13(d)). The reinforcing bars did not fracture when the final failure of the joint occurred. Tensile fracture of the profiled sheeting indicates that the neutral axis of the connection lay within the concrete slab and the tension force provided by the bolted connections was small in comparison with the compression force provided by the top concrete slab. Fig. 4.13(e) shows the crack pattern of the composite slab. It can be seen that cracks mostly concentrated at the concrete crushing zone where the FP connection was located. Longitudinal cracks were also observed and they were induced by weakening of the concrete slab due to the re-entrant profile. Potential longitudinal shear failure surface in Fig. 4.14(a) agrees well with the longitudinal cracks observed in Fig. 4.13(e). Fig. 4.14(b) shows weakening of the concrete slab which created the longitudinal shear failure surface.

Fig. 4.13 Failures mode of C75FP-M (middle joint): (a) Front view; (b) Crushing of concrete and exposure of yielded reinforcing bar; (c) Fracture of profiled sheeting; (d) Block shear failure of fin plate; (e) Cracks of concrete slab

Fig. 4.14 Longitudinal shear failure surface: (a) Top view; (b) Section 1-1

Fig. 4.15(a) shows the failure mode of C75FP-S, a side joint subjected to hogging moment. All the slab components were subjected to tension so that the reinforcing bars, concrete and the steel sheeting fractured in tension as shown in Figs. 14(b) and (c), respectively. The bolt rows were also subjected to tension at large deformation stage so that they fractured individually (Fig. 4.15(d)). Although a transverse crack was observed (Fig. 4.15(e)), opening of the slab indicates that failure was concentrated in the connection as shown in Fig. 4.15(e).

Fig. 4.15 Failures mode of C75FP-S (side joint): (a) Front view; (b) Fracture of reinforcing bar; (c) Fracture of profiled sheeting; (d) Fracture of fin plate; (e) Cracks of concrete slab

Fig. 4.16 Fin plates in specimen C75FP-Mslot: (a) Fracture of left fin plate; (b) Sliding of bolts connected to right fin plate

Figs. 4.16(a) and (b) show the failure mode of the left and the right fin plates in C75FP-Mslot, respectively. Final failure - tensile fracture of the left fin plate - occurred after the bolts slid along the slotted holes as shown in Fig. 4.16(a). Sliding of the bolts was more clearly shown in Fig. 4.16(b). Since the slotted holes (10 mm) were smaller than the bolt shank (20 mm) in (d), plasticity developed in the plate during sliding so that the specimen could resist a greater load as shown in Fig. 4.11(e).

4.3.3 Axial force and bending moment

The axial forces obtained from the CHS members of all the specimens are compared in Fig. 4.17. The negative axial force is an index for CAA while the positive one is for CA. In Fig. 4.17(a), axial force development in the middle joint C75FP-M and the side joint C75FP-M was quite different at small deformation stage before 150 mm. Compression force was developed in C75FP-M due to CAA induced by the composite slab. By comparison, only CA was developed in C75FP-S because the composite slab was all the while in tension. Influence of the thickness of the composite slab was investigated in Fig. 4.17(b). FP-static was a bare steel joint tested so that the thickness of concrete slab was zero. It can be seen that only tension force was developed due to CA, similar to C75FP-S in Fig. 4.17(a). A thicker concrete section would induce greater CAA and greater compression. However, CA was significantly decreased. With weaker composite action in C75FP-MR (through reducing the number of shear studs), compression force was not affected whereas tension force could not fully develop as shown in Fig. 4.17(c). The composite slab

and the beam acted as two separate members due to reduced composite action. Therefore, at the same displacement, the fin plates in C75FP-MR had greater deformation than those of C75FP-M (with full composite action). The ductility of the fin plates in C75FP-MR was exhausted much earlier so that final failure occurred earlier as shown in Fig. 4.17(c). When comparing C75FP-Mslot to C75FP-M in Fig. 4.17(d), it can be seen that weaker initial CAA was developed due to the slotted bolt holes. However, much greater CA was developed due to better ductility of the fin plates with the slotted bolt holes.

Fig. 4.17 Comparison of axial force versus displacement curves: (a) Middle and side joints; (b) Three slab thicknesses; (c) Normal and fewer shear studs; (d) Normal and slotted bolt holes

Bending moment in the connection obtained from the CHS members of all the specimens is compared in Fig. 4.18 and it indicates FA. As shown in Fig. 4.18(a), much greater FA was developed in the middle joint C75FP-M. In the side joint C75FP-S, FA was negligible after the composite slab fractured in tension at 106 mm. In Fig. 4.18(b), a thicker slab contributed to greater FA. Without the slab, FA was negligible in FP-static, with only a simple bare steel pinned connection. Slightly smaller FA was developed in C75FP-MR compared to C75FP-M due to reduced

composite action as shown in Fig. 4.18(c). Compared to C75FP-M, smaller FA was developed in C75FP-Mslot due to sliding of bolts along the slotted holes. However, FA lasted longer since the fin-plate did not fracture at the CAA stage.

Fig. 4.18 Comparison of bending moment versus displacement curves: (a) Middle and side joints; (b) Three slab thicknesses; (c) Normal and fewer shear studs; (d) Normal and slotted bolt holes

4.3.4 Energy

Energy is an important index to evaluate the joint performance; it is equal to the work done by the applied load and could be obtained from integration of the area beneath the total-load-versus-displacement curve of each specimen as shown in Fig. 4.11. A greater energy at a greater final displacement indicates a better performance of the joint. As shown in Fig. 4.19(a), the initial energy absorbed by the middle joint C75FP-M was much greater than that of the side joint C75FP-S at the small deformation stage. However, due to the development of CA (Fig. 4.19(a)) at the large deformation stage, C75FP-S absorbed an equal energy at the final displacement compared with C75FP-M. When comparing C75FP-M and C100FP-M as shown in Fig. 4.19(b), the slab thickness did not affect the initial energy absorption at the small

deformation stage. However, due to a greater depth of the concrete slab, ductility of the fin plates in C100FP-M was exhausted earlier so that the final displacement was much smaller (168 mm versus 269 mm of C75FP-M). In the simple bare steel joint FP-static as shown in Fig. 4.19(b), absorbed energy was negligible initially when the joint rotated like a pin. At the large deformation stage, FP-static could absorb an equal energy (8.9 kJ) at the final displacement compared with C75FP-M (8.5 kJ). The behaviour of FP-static was similar to C75FP-S. In Fig. 4.19(c), the energy absorption of C75FP-MR was smaller than that of C75FP-M. However, failure of C75FP-MR occurred earlier (189 mm versus 269 mm of C75FP-M) due to weaker composite action so that a smaller energy (6.7 kJ versus 8.4 kJ of C75FP-M) was absorbed. In Fig. 4.19(d), although energy absorption of C75FP-Mslot was also slightly smaller than that of C75FP-M, it was much greater at the final displacement (11.0 kJ versus 8.4 kJ of C75FP-M) due to better ductility of the fin plates with slotted bolt holes.

Fig. 4.19 Comparison of energy versus displacement curves: (a) Middle and side joints; (b) Three slab thicknesses; (c) Normal and fewer shear studs; (d) Normal and slotted bolt holes

Deformations of the specimens at 4.0 kJ and 8.0 kJ are shown in Figs. 4.20(a) and (b), respectively. The two values are equal to one-half and close to the maximum energy of C75FP-M, respectively. The abscissa is the distance of each displacement measuring point (Figs. 4.5 and 4.6) to the centre line of the middle column. Each pin was 1834 mm away from the centre. In Fig. 4.20(a), the deformations of all the middle joints were similar while that of the side joint was much greater. In Fig. 4.20(b), performance of C75FP-Mslot was the best among the five specimens due to a much smaller deformation when absorbing 8.0 kJ of energy. Specimens C75FP-MR and C100FP-M were damaged so that they are not shown in Fig. 4.20(b).

Fig. 4.20 Comparison of vertical displacement of specimens along horizontal axis at two different energy levels: (a) 4.0 kJ; (b) 8.0 kJ

4.3.5 Development of strain

Typical strain gauge readings of various components, viz. concrete, reinforcing bar, profiled sheeting and steel beam in the middle and the side joints are shown in Figs. 4.21 and 4.22, respectively. Locations of the strain gauges are shown in Fig. 4.8. In Fig. 4.21(a), concrete at the centre line (C1) and the connection (C2) was subjected to compression at the initial stage and its strain kept increasing until crushing

occurred. Correspondingly, the applied load in Fig. 4.21(a) stopped increasing and went to a plateau. Due to spalling of concrete, the concrete strain gauges were damaged and removed. Strains of the middle and the side reinforcing bars are shown in Fig. 4.21(b). At section 1-1 (Fig. 4.8(a)), the middle reinforcing bar (MRR2) and the side reinforcing bar (MRR3) had the same compressive strain at the small deformation stage before 42 mm. Then the compressive strain of the middle reinforcing bar increased rapidly because the middle reinforcing bar started bearing on the column flange and buckling occurred. However, the side reinforcing bars were continuous across the joint so that compressive strain could develop more uniformly and no buckling of reinforcing bar was observed. At large deformation stage, strain of the side reinforcing bar (MRR2) changed from compression to tension as shown in Fig. 4.21(b) due to CA. At section 2-2 (Fig. 4.8(a)), strains of the middle (RR2) and the side (RR3) reinforcing bar were the same and changed from compression to tension. However, the strains at section 2-2 were much smaller than those at section 1-1 (connection). Strains of the steel profiled sheeting (MRP1) and the restrained beam flange (MR1) in section 1-1 were positive (tensile) as shown in Figs. 4.21(c) and (d), respectively, indicating that the neutral axis of the composite section lay within the concrete slab.

Fig. 4.21 Development of strain of different components in specimen C75FP-M (middle joint): (a) Concrete; (b) Reinforcing bar; (c) Profiled sheeting; (d) Steel beam

Concrete in the side joint was subjected to tension (positive strain) as shown in Fig. 4.22(a). Reading of strain gauge C2 (concrete in the connection) exceeded $150 \mu\epsilon$ (the maximum tensile strain given in fib model code (fib 2013)) whereas strain of concrete at the centre line (C1) was negligible since that tensile strain concentrated in the connection. Reinforcing bars were subjected to tension at both sections 1-1 and 2-2 as shown in Fig. 4.22(b). The side reinforcing bar (MRR3) was continuous so that its tensile strain was significant compared to that of the discontinuous middle reinforcing bar (MRR2). Profiled sheeting (MRP1) at section 1-1 was subjected to increasing tension force before fracture initiated at 71 mm as shown in Fig. 4.22(c). After that, strain (MRP1) decreased rapidly. Unrestrained beam flange (MR1) was subjected to compression (Fig. 4.22(d)). Compared with strains of other components such as concrete (C2), reinforcing bar (MRR2 and MRR3) and profiled sheeting (MRP1) at section 1-1, it can be concluded that the neutral axis always lay within the beam depth. However, at section 2-2, strain of unrestrained beam flange (R1) changed from compressive to tensile from 71 mm as shown in Fig. 4.22(d),

indicating that the cross section was subjected to tension at the large deformation stage.

Fig. 4.22 Development of strain of different components in specimen C75FP-S (side joint): (a) Concrete; (b) Reinforcing bar; (c) Profiled sheeting; (d) Steel beam

4.4. Comparison between design values and test results

Currently, FP connection with a composite slab is not considered as a composite connection in Eurocode 4 (BSI 2004a). Therefore, a combined method was used to calculate the design resistance in this study, namely, the connection was considered as a composite connection in Eurocode 4 (BSI 2004a), while the design resistance of steel components followed bolted connections in Eurocode 3 (BSI 2005b). Material properties in Table 4.2 were used without any partial safety factors when calculating the design resistance. Two failure modes of fin plates as shown in Fig. 4.23 were observed from test results. When calculating tying resistance, two side reinforcing bars were considered but not profiled steel sheeting due to its poor ductility (Yang and Tan 2014). Flexural and tying resistance of joints subjected to sagging and

hogging moment was calculated based on force distributions in Figs. 4.23(a) and (b), respectively. Locations of the neutral axis were obtained from strain gauge readings (Chapter 4.3.5) and force equilibrium of the composite cross-section in Fig. 4.24. Design values of joint rotation capacity were obtained from UFC 4-023-03 (2013). A ratio of test results to calculated design values (T/D) was used to evaluate the joint performance. A summary of design values and test results (Figs. 4.17 and 4.18) is shown in Table 4.3.

Fig. 4.23 Two failure modes in the test: (a) Case 1 block shear; (b) Case 2 tensile fracture

Fig. 4.24 Force distribution of composite joint: (a) Sagging moment; (b) Hogging moment

Table 4.3 Summary of design values and test results

ID	Block shear	Tying resistance			Flexural resistance			Rotation capacities		
		Design (kN)	Test (kN)	Ratio T/D	Design (kNm)	Test (kNm)	Ratio T/D	Design (rad)	Test (rad)	Ratio T/D
C75FP-M	Case 1	219.8	88.8	0.40	42.8	49.9	1.17	0.10	0.11	1.10
C75FP-S	Case 2	219.8	223.8	1.02	20.6	22.7	1.10	0.10	0.16	1.60
C100FP-M	Case 1	219.8	0	0	51.9	57.1	1.10	0.10	0.09	0.90
C75FP-MR	Case 2	219.8	22.9	0.10	42.8	48.9	1.14	0.10	0.10	1.00
C75FP-Mslot	Case 2	219.8	258.3	1.18	42.8	44.5	1.04	0.10	0.14	1.40

From Table 4.3, it can be seen that most of the joints cannot develop catenary action as the design tying resistance. With thicker slab (C100FP-M) or fewer shear studs (C75FP-MR), catenary action failed to develop. The reason is that, ductility of the FP connection was exhausted at FA stage. The side joint C75FP-S was an exception and it had greater T/D . This is probably because fin plates of C75FP-S were in compression at FA stage so that they were not so severely damaged. C75FP-Mslot had FP connection with better ductility so that it could reach the design tying resistance (Table 4.3). Compared to semi-rigid beam-column joints tested by Yang and Tan (2014), simple joints reported in this study had poor tying resistance. It should be noted that the design value of tie force (75 kN) specified in Eurocode 1 Part 1-7 (BSI 2002) (without rotation capacity) could be achieved for conventional joints with adequate composite action and the novel joint. However, specimens C100FP-M and C75FP-MR could not achieve the tie force requirement.

Flexural resistance of the joints is also compared in Table 4.3. In contrast to tying resistance, all of the joints could achieve design flexural resistance. Compared to bare steel joint FP-static as shown in Fig. 4.18(b), all the composite joints had greater flexural resistance.

It should be mentioned that with the exception of C100FP-M, rotation capacities of all the joints were greater than the design value of 0.10 rad provided by UFC 4-023-03 (2013). However, due to weakened tying resistance, rotation capacities of these simple joints may not ensure integrity at large deformation stage.

4.5. Summary and conclusions

Five simple joints with composite slab were tested under a middle column removal

scenario. Four parameters, viz. joint type, slab thickness, shear studs and bolt holes were investigated. Behaviour of the joints was compared with design values. The following conclusions can be made:

- (1) Resistance of the simple joint was provided by flexural action combined with compressive arch action or catenary action depending on the joint deflections. At small deformation stage, compressive arch action was dominant while catenary action was dominant at large deformation stage. Compared to the bare steel joint, composite joint could increase flexural action.
- (2) Increased slab thickness and reduced number of shear studs were detrimental to mobilisation of catenary action.
- (3) Middle joint absorbed more energy than side joint at the small deformation stage due to greater flexural action. However, similar energy was absorbed by both joints at the end because the side joint mobilised much greater catenary action.
- (4) FP connection with slotted bolt holes had better performance than conventional connection in terms of energy absorption and tying resistance.
- (5) Although tying resistance of the composite joints was reduced due to combined bending moment, tie force requirement from Eurocode 1 could be met for most of the composite joints. With a thicker concrete slab (100 mm) or a reduced number of shear studs, tie force requirement could not be met.

CHAPTER 5: EXPERIMENTAL TESTS OF COMPOSITE JOINTS WITH WUF-B CONNECTIONS SUBJECTED TO A COLUMN REMOVAL SCENARIO

5.1 Introduction

Compared to the pin connection in Chapter 4, WUF-B connection is one of the commonly-used moment-resisting connections. It is designed to transmit moment from the beams to supporting columns. If its integrity is lost, fatal loss such as progressive collapse will probably occur. Therefore, research studies on joints with WUF-B connection subjected to abnormal loads are useful in design practice. In this chapter, a test programme on composite joints with moment-resisting connections under a middle column removal scenario is presented. In the test programme, one series of five composite joints with moment-resisting connections were tested and four parameters including the joint type, slab thickness and number of shear studs were studied. One reduced beam section (RBS) connection was incorporated for comparison purpose. From the test results, load-resisting mechanism, failure mode, energy absorption capacity and development of strain were investigated. Furthermore, test results including tying and flexural resistances as well as rotation capacity of the composite joints were compared with design values from building codes. This chapter also presents a comparison between WUF-B connection and FP connection (Chapter 4).

5.2 Test programme

5.2.1 Test specimens and material properties

A total of five half-scale beam-column joints with moment-resisting connection were tested under a middle column removal scenario and the details are shown in Table 5.1 and Fig. 5.1. To identify each specimen, they are named based on the concrete slab thickness and connection detailing, such as C75 stands for 75 mm thick

composite slab, W for WUF-B connection, M for middle joint while S for side joint, R for reduced number of shear studs and rbs for reduced beam section connection. Due to symmetry, only the right half part of each specimen is shown in Figs. 5.1(a) to (e) while the test set-up is shown in Fig. 4.2. In Fig. 5.1(a), C75W-M was a middle joint subjected to sagging moment in which the middle column was ‘forcibly’ removed. Composite slab with 75 mm thickness was placed above the I-beam. Re-entrant steel profile sheeting with 1 mm thickness was used in the composite slab. Diameter 6 mm mild steel reinforcing bars at 170 mm spacing in both the longitudinal and the transverse directions were used as an anti-crack steel mesh. Additional high-strength 10 mm diameter reinforcing bars at 90 mm spacing were applied in the transverse direction to prevent longitudinal shear failure in the concrete slab. Two rows of mild steel shear studs of 16 mm diameter at 90 mm spacing were used for full shear connection between the composite slab and the I-beam. The WUF-B connection was designed based on AISC 360 (2010) and AISC 325 (2011). Recommendations in FEMA 350 (2000) were also considered. To prevent shear failure of bolts, Grade 10.9 M20 bolts were used. A pre-torque of 280 kNm was applied to the bolts. As shown in Fig. 5.1(b), C75W-S was a side joint subjected to hogging moment. Composite slab was placed underneath the I-beam and was subjected to tension during testing. Steel WUF-B connection in specimen C75W-S was the same as C75W-M. Compared with C75W-M (75 mm), a thicker slab (100 mm) was used in C100W-M as shown in Fig. 5.1(c). One row of mild steel shear studs of 16 mm diameter at 180 mm spacing was used in C75W-MR (Table) compare to two rows of shear studs at 90 mm spacing for C75W-M to achieve partial shear connection. In C75W-Mrbs, RBS connection was used and the front and top views are shown in Figs. 5.1(d) and (e), respectively.

Material properties of steel employed in the joints are listed in Table 4.2. Based on twelve standard 150 mm diameter by 300 mm length cylinder tests, concrete compressive strength was 37.4 MPa and the corresponding standard derivation was 1.4 MPa.

Table 5.1 Summary of test specimens

ID	Beam, column, Fin plate and bolt	Thickness of composite slab (mm)	Joint location	Bending moment	Shear studs
C75W-M	S355 UC 203×203×71 column	75	Middle	Sagging	2 rows @ 90 mm
C75W-S	S355 UB 203×133×30 beam	75	Side	Hogging	2 rows @ 90 mm
C100W-M	S275 150×70 plate	100	Middle	Sagging	2 rows @ 90 mm
C75W-MR	Grade 10.9 M20 bolt	75	Middle	Sagging	1 row @ 180 mm
C75W-Mrbs		75	Middle	Sagging	2 rows @ 90 mm

Nomenclature: C - Composite; W - WUF-B, welded; M - Middle joint; S - Side joint; R - Reduced number of shear studs; rbs - reduced beam section

The same set-up in Chapter 4 was used in this Chapter. To measure strains of different components of the joints, such as concrete, reinforcing bar, profiled sheeting and I-beam, strain gauges were attached to the middle joint in Fig. 5.2(a). Two cross-sections, namely, section 1-1 (Fig. 5.2(b)) close to the connection and section 2-2 (Fig. 5.2(c)) in the middle of I-beam were monitored. In section 1-1, strain gauges MRR1(2,3) were for three reinforcing bars, MRP1 for steel profiled sheeting, MR1 and MR2 for the respective restrained and unrestrained beam flanges, and C2 for concrete surface. In section 2-2, strain gauges RR1(2,3) were for three reinforcing bars, RP1(2) for steel profiled sheeting, R1(2) and R4(5) for the respective restrained and unrestrained beam flanges and R3 for beam web. The same strain gauge layout was used for the side joint and is shown in Figs. 5.3(a) to (c). For specimen C75W-

Mrbs, additional strain gauges (RBS1 and RBS2) were attached to the unrestrained beam flanges in the RBS as shown in Fig. 5.4. TML linear wire transducers (LT) and linear variable (LV) differential transformers were employed to record displacements of the specimens and their locations are shown in Fig. 4.5 for the middle joints and Fig. 4.6 for the side joint, respectively.

Fig. 5.2 Strain gauge layout of middle joint: (a) Front view; (b) Section 1-1; (c) Section 2-2

Fig. 5.3 Strain gauge layout of side joint: (a) Front view; (b) Section 1-1; (c) Section 2-2

Fig. 5.4 Additional strain gauges of specimen C75W-Mrbs

5.3 Test results and discussions

5.3.1 Load-resisting mechanism

Fig. 5.5 shows load-versus-displacement curves of the joints directly obtained from recorded data. Load-resisting mechanism, namely, respective load resisted by flexural action (FA), compressive arch action (CAA) and catenary action (CA) was quantified based on the method introduced in Chapter 4 and is shown in Fig. 5.5. In Fig. 5.5(a), load applied to C75W-M increased linearly at the initial stage before 22 mm of displacement. After that plastic deformation started developing and the applied load became nonlinear. At the initial stage, the applied load was mainly resisted by FA as load resisted by CAA was practically negligible (Fig. 5.5(a)). At around 49 mm, the applied load reached the first peak value of 200.4 kN. Then concrete in the composite slab started crushing. Therefore, lever arm of the connection section to resist bending moment started decreasing so that FA began to reduce. However, with the development of plastic deformation and more area of steel in the connection started yielding, CA started developing so that the applied load could still increase. At 152 mm, the unrestrained beam flange fractured, which caused a decrease of FA (Fig. 5.5(a)). Thereafter, the applied load dropped rapidly from 225.0 kN to 62.7 kN. After the fracture, load resisted by CA continued to increase due to contribution of the unrestrained beam flange and intact bolt rows. However, FA became unfavourable to resist the applied load. The reason is that the location of resultant tension force in the connection moved higher than neutral axis after the unrestrained beam flange fractured and hogging moment started to develop when the displacement reached 152 mm. Similar phenomenon was observed in previous tests (Yang and Tan 2014). Final failure of the joint was induced by fracture of the restrained beam flange at 293 mm. Due to unfavourable FA at large deformation stage, the peak load resisted by CA was reduced from 252.0 kN to 206.3 kN when the restrained beam flange also fractured at the joint.

Fig. 5.5(b) shows the load-resisting mechanism of the side joint C75W-S. Since composite slab was in tension, concrete started fracturing right from the beginning. Resistance provided by concrete tension force was negligible so that the applied load

could increase linearly at small deformation stage (before 21 mm) as shown in Fig. 5.5(b). After 21 mm, plastic deformations of steel components such as the restrained beam flange, reinforcing bars and profiled sheeting developed and applied load became nonlinear. The applied load reached the first peak value of 151.2 kN at 107 mm and then decreased after fracture of the restrained beam flange took place. Load was mainly resisted by FA before fracture occurred. As shown in Fig. 5.5(b), the applied load increased marginally because the increased CA was counteracted by unfavourable FA. At large deformation stage, CA was fully mobilised so that the applied load could increase until fracture of the unrestrained beam flange occurred at 375 mm. The applied load was smaller than that of the middle joint C75W-M, indicating that when subjected to tension, the composite slab contributed much less effectively to resisting applied load.

Fig. 5.5(c) shows the load-resisting mechanism of C100W-M. Linear load stage ended at 22 mm and the applied load was 157.4 kN. Due to a thicker slab (100 mm versus 75 mm of C75W-M), the applied load was greater (221.7 kN versus 200.4 kN of C75W-M) when crushing of concrete occurred. Unrestrained beam flange fractured at 116 mm, much smaller than 152 mm of C75W-M. Restrained beam flange fractured at 234 mm, also smaller than 293 mm of C75W-M. Therefore, CA was also weaker than that of C75W-M. When comparing Figs. 5.5(a) and (c), it can be concluded that a thicker slab could contribute to better FA but deformation capacity of joint exhausted much earlier and CA was significantly weaker.

Fewer shear studs were used and weaker composite action was formed in C75W-MR. Therefore, weaker FA developed when comparing with C75W-M (184.9 kN in Fig. 5.5(d) versus 200.4 kN in Fig. 5.5(a)). Unrestrained beam flange fractured at 98 mm, much earlier than 152 mm of C75W-M. However, the applied load at final failure was 204 kN in C75W-MR, similar to 206.3 kN of C75W-M. The reason is that the final displacement in C75W-MR was similar to C75W-M so that CA could develop. Although CA in C75W-MR was weaker than that in C75W-M, unfavourable FA was also slightly weaker at large deformation stage.

In the joint with reduced beam section (C75W-Mrbs), only FA and CA developed as shown in Fig. 5.5(e). Load resisted by FA (165.9 kN) was much smaller than all the

three middle joints (Fig. 5.5(e)). With the development of CA, the applied load could increase rapidly at CA stage until fracture of unrestrained beam flange occurred at 232 mm, later than the other middle joints. The applied load could reach 327.1 kN, which was much greater than the other middle joints. Then the fracture developed along the beam web and reached the restrained beam flange at 286 mm. For all the specimens, CAA was negligible even through both ends were adequately restrained.

Fig. 5.5 Load versus displacement curves of all the specimens: (a) C75W-M; (b) C75W-S; (c) C100W-M; (d) C75W-MR; (e) C75W-Mrbs

5.3.2 Failure mode

Front view of specimen C75W-M is shown in Fig. 5.6 while failures of concrete slab, reinforcing bar, profiled sheeting, bolt row and beam of the same specimen are shown in Fig. 5.7. Failure first occurred from crushing of slab concrete with buckling of reinforcing bar as shown in Fig. 5.7(a). Then the unrestrained beam flange fractured (Fig. 5.7(b)) with a sharp drop of applied load as shown in Fig. 5.5(a). This was followed by block shear failure of the left fin plate (Fig. 5.7 (c)). Finally, fracture of the left restrained beam flange occurred as shown in Fig. 5.7(d) and the joint could not sustain the load. Fig. 5.8 shows the crack pattern of the left slab of C75W-M where failure mainly concentrated in the crushing zone close to the connection. Longitudinal cracks were induced by longitudinal shear failure even though more than adequate number of shear studs were provided. Such shear cracks took place due to weakening of the slab section at the re-entrant profile.

Fig. 5.6 Front view of failure of C75W-M

Fig. 5.7 Failure mode of C75W-M: (a) Buckling of slab reinforcing bar and crushing of slab concrete; (b) Fracture of unrestrained beam flange; (c) Block shear failure of fin plate; (d) Fracture of restrained beam flange

Fig. 5.8 Failure of the left slab of C75W-M

Figs. 5.9-11 show the failure mode of the side joint C75W-S. Sequence of failure of various components in Fig. 5.9 were different from those of the middle joint C75W-M in Fig. 5.6. Fracture of reinforcing bar and profiled sheeting, and crushing of concrete occurred first as shown in Figs. 5.10(a) and (b), with fracture of the restrained beam flange (Fig. 5.10 (c)). Then local buckling occurred at the right unrestrained beam flange as shown in Fig. 5.10(c). With an increase of displacement,

block shear failure of the right fin plate occurred (Fig. 5.10(d)), followed by fracture of the right unrestrained beam flange (Fig. 5.10(e)). It should be noted that fracture of profiled sheeting and reinforcing bar in the side joint was not observed in the middle joint. Due to tension force acting on the slab, transverse cracks developed as shown in Fig. 5.11. A main crack opening that ran perpendicular to the beam was observed close to the connection. When transverse cracks were developing, they were not evenly distributed along the transverse direction so that eccentricity of the applied load occurred. Diagonal cracks were thus induced as shown in Fig. 5.11.

Fig. 5.9 Front view of failure of C75W-S

Fig. 5.10 Failure mode of C75W-S: (a) Fracture of concrete and profiled steel sheeting; (b) Fracture of reinforcing bar (c) Fracture of restrained beam flange and buckling of unrestrained beam flange; (d) Block shear failure of fin plate; (e) Fracture of unrestrained beam flange;

Fig. 5.11 Failure of the left slab of C75W-S

Figs. 5.12-14 show the failure mode of the middle joint C75W-Mrbs with RBS connection. Crushing of slab concrete and buckling of reinforcing bar as shown in Fig. 5.12 were similar to those of C75W-M. Failure of C75W-Mrbs was characterised by fracture of RBS connection as shown in Fig. 5.13(a). Tensile fracture of unrestrained beam flange as shown in Fig. 5.13(b) occurred first and then it slowly developed across the beam web as shown in Fig. 5.13(a). When the fracture reached the restrained beam flange, the joint was completely severed and no more load could be sustained.

Fig. 5.12 Front view of failure of C75W-Mrbs

Fig. 5.13 Fracture of the left RBS of C75W-Mrbs: (a) Front view; (b) Bottom view

Fig. 5.14 Failure of the left slab of C75W-Mrbs

5.3.3 Energy

Fig. 5.15 shows a comparison of energy absorption of all the joints, calculated from the applied load-versus-displacement curves in Fig. 5.5. If energy absorption capacity is defined by the amount of energy absorption at a given displacement, a specimen can achieve better energy absorption capacity through either absorbing more energy at the same displacement, or having a smaller displacement for the same amount of energy absorbed. Fig. 5.15(a) shows the energy absorption of the middle and the side joints. It can be seen that side joint C75W-S absorbed less energy than middle joint C75W-M although the former had better rotation capacity. When the beam flange first fractured, energy absorption of C75W-S was 18.6 kJ while that of C75W-M was 27.5 kJ. When the second beam flange fractured, energy absorption of

C75W-S was 31.2 kJ while that of C75W-M was 46.5 kJ. Clearly, the composite slab in compression could provide better energy absorption capacity than one in tension. Fig. 5.15(b) shows a comparison of energy absorption capacity of middle joints with three slab thicknesses: 0, 75 and 100 mm. It can be seen that the slab thickness had a great effect on energy absorption capacity of the middle joints. Energy absorption when the unrestrained beam flange fractured was increased when comparing C75W-M to W-static. However, energy absorption when the restrained beam flange fractured was reduced when comparing C75W-M and C100W-M to W-static. Reduction of energy absorption and failure displacement was clearly evident with the increase of slab thickness from 75 to 100 mm. As shown in Fig. 5.15(c), when fewer shear studs were used and weaker composite action was formed in C75W-MR, the nonlinear stage commenced much earlier so that energy absorption at unrestrained beam flange fracture point was reduced significantly (15.6 kJ versus 27.5 kJ in C75W-M). Although the final failure displacement was about the same as C75W-M, energy absorption was evidently smaller in C75W-MR. A comparison of middle joints with WUF-B and RBS connections is shown in Fig. 5.15(d). Generally, C75W-Mrbs had better energy absorption capacity than C75M-M. At unrestrained beam flange fracture point, C75W-Mrbs could absorb much greater energy (42.8 kJ) than that (27.5 kJ) of C75W-M. At final failure, greater energy absorption (52.3 kJ) could also be achieved in C75W-Mrbs while the final displacement was similar. For the four joints with WUF-B connection, two stages of energy absorption were observed: linear increasing stage until the beam flange first fractured and nonlinear increasing stage until the second beam flange also fractured. By comparison, for C75W-Mrbs, although the unstrained and restrained beam flanges also fractured sequentially, the two stages were nonlinear as shown in Fig. 5.15(d).

Fig. 5.15 Comparison of energy versus displacement curves between specimens: (a) Middle and side joints; (b) Three slab thicknesses; (c) Normal and fewer shear studs; (d) WUF-B and RBS connections

A comparison of vertical displacement profile of all the joints along horizontal axis is shown in Fig. 5.16 at two energy levels. Based on C75W-M, energy level at 27 kJ when unrestrained beam flange fractured and 46 kJ when restrained beam flange fractured were chosen for comparison purpose. In Fig. 5.16(a), among all the five joints, C75W-M and C100W-M had the best energy absorption capacities since their vertical displacements were the smallest. C75W-Mrbs had similar energy absorption capacity with C75W-M and C100W-M. Energy absorption capacities of C75W-MR and C75W-S were significantly weaker than the other three joints. Therefore, when using first beam flange fracture as the design criterion, ensuring enough shear studs and composite slab thickness is necessary for improving energy absorption capacity. As shown in Fig. 5.16(b), only C75W-M and C75W-Mrbs survived after absorbing 46 kJ of energy. The other three joints had already failed due to either weaker

catenary action (C100W-M and C75W-MR) or composite slab failure in tension (C75W-S). It should be noted that C75W-Mrbs had better energy absorption capacity than C75W-M in Fig. 5.16(b). In conclusion, C75W-Mrbs had the best energy absorption capacity among all the five joints when second beam flange fracture is allowed in design.

Fig. 5.16 Comparison of vertical displacement along horizontal axis at different energy levels: (a) 27 kJ; (b) 46 kJ

5.3.4 Development of strain

Fig. 5.17 shows strain gauge readings of different components in C75W-M and they represent typical results of the middle joint. The layout of strain gauges is shown in Fig. 5.2. As shown in Fig. 5.17(a), concrete in the middle joint was subjected to the same compressive strain at the linear loading stage before 22 mm. After that, strain of concrete (C1) increased more rapidly than that at the connection (C2). When crushing occurred at 49 mm, strain of concrete at both locations decreased significantly. Correspondingly, reinforcing bars in the composite slab had similar compressive strain at the linear loading stage before 22 mm as shown in Fig. 5.17(b). The compressive strain of middle reinforcing bar at the connection cross section

(MRR2) developed rapidly after that until buckling occurred. The side reinforcing bar was continuous across the joint so that compressive strain (MRR3) could uniformly develop until buckling occurred at the centreline as shown in Fig. 5.7(a). At large deformation stage, CA was mobilised so that strains of the reinforcing bars (RR2 and RR3) changed from compressive to tensile as shown in Fig. 5.17(b). Since the middle reinforcing bar was discontinuous, tensile strain was not observed (MRR2). Strains of the profile sheeting at the connection cross section (MRP1 in Fig. 5.17(c)) and the restrained beam flange (MR1 in Fig. 5.17(d)) were in tension and increased with the applied vertical displacement. Fluctuation of MRP1 was observed due to local buckling of the steel profiled sheeting. As shown in Fig. 5.17(d), tensile strain of the unrestrained beam flange increased linearly at initial loading stage. The unrestrained beam flange yielded after crushing of concrete occurred. It can be concluded that the neutral axis lay within the composite slab at flexure stage before unrestrained beam flange fractured.

Fig. 5.17 Development of strain of different components in specimen C75W-M (middle joint): (a) Concrete; (b) Reinforcing bar; (c) Profiled sheeting; (d) Steel beam

Fig. 5.18 shows strain gauge readings of different components in C75W-S and they

represent typical results of the side joint. All the components in the composite slab, including concrete, steel profiled sheeting and reinforcing bar were subjected to tension and thus tensile strains were observed. Strain of concrete at the centreline (C1 in Fig. 5.18(a)) was tensile and fracture strain ($150 \mu\epsilon$) was reached at 68 mm. The compressive strain in C2 was due to concrete bearing onto the middle column stub. In Fig. 5.18(b), tensile strains of the reinforcing bars were the same at linear loading stage. Strains of continuous side reinforcing bars (MRR3 and RR3) developed much faster than those of the discontinuous middle reinforcing bars (MRR2 and RR2) at nonlinear loading stage until complete fracture occurred at 159 mm. Peak strains of profiled sheeting (MRP1 in Fig. 5.18(c)) and concrete (C2) occurred at the same displacement (68 mm) when applied load was nonlinear. Compressive strain of unrestrained beam flange (MR1) and tensile strain of restrained beam flange (MR2) in Fig. 5.18(d) indicated that neutral axis lay within the beam web. Both flanges (MR1 and MR2) yielded at nonlinear loading stage.

Fig. 5.18 Development of strain of different components in specimen C75FP-S (side joint): (a) Concrete; (b) Reinforcing bar; (c) Profiled sheeting; (d) Steel beam

5.4 Comparison between design resistance and test results

To evaluate design resistance of the composite joints in current building codes, test results of the specimens are compared with design values as shown in Table 5.2. Design values of tying and flexural resistances of the specimens were calculated based on stress distribution of the composite joints as shown in Fig. 5.19. In Fig. 5.19, f_c is the average compressive strength of concrete; f_y is the yield strength of steel; f_e is the elastic stress of steel; N_{beam} is the beam resultant force; N_{bolt} is the resultant force of each bolt row; N_{con} is the resultant force of concrete slab; $N_{pl,Rd}$ is the tying resistance; $M_{pl,Rd}$ is the flexural resistance. Fig. 5.19(a) shows the stress distribution of middle joint for calculating flexural and tying resistances. Contribution of the steel profiled sheeting cannot be considered since it will buckle when subjected to compression and it only has small fracture strain when subjected to tension (Yang and Tan 2014). Steel components, including bolt rows, beam flanges and reinforcing bars were calculated based on AISC (2010). Contribution of the bolt rows to flexural resistance was not considered based on AISC (2010). Flexural resistance $M_{pl,Rd}$ was obtained by calculating bending moment of resultant force from each component. Lever arms of each component were based on connection geometry as shown in Fig. 5.19(a). Tying resistance $N_{pl,Rd}$ was obtained by simply summing up all the resultant forces. It should be noted that measurements of material properties without any partial safety factors were used when calculating the design values. The same method could be used for calculating flexural ($M_{pl,Rd}$) and tying ($N_{pl,Rd}$) resistances of side joint as shown in Fig. 5.19(b) as well as RBS joint as shown in Fig. 5.19(c). Design rotation capacities of the specimens were calculated based on UFC 4-023-03 (2013). Currently, UFC 4-023-03 design provisions on rotation capacities of beam-column joint are only applicable for bare steel joints.

Fig. 5.19 Design resistance based on stress distribution: (a) Middle joint; (b) Side joint; (c) RBS joint

From Table 5.2, design flexural resistance could be achieved in the test for all the specimens since the ratios T/D (test results to design values) for flexural resistance were greater than 1.0. At large deformation stage, design tying resistance could not be achieved as shown in Table 5.2 because all the connections were partially damaged during FA stage. Among the five specimens, C75W-Mrbs had the best tying resistance although the test result was also lower than design tying resistance. It should be noted that all the specimens could not function as ‘tie’ members since their rotation capacities were smaller than 0.2 rad based on UFC 4-023-03 (2013). However, tie force requirement of 75 kN in Eurocode 1 Part 1-7 (2002) could be met for all the joints. Compared to design rotation capacities provided in UFC 4-023-03 (2013), the test results of all the specimens achieved the design values for WUF-B

joints. It should be noted the rotation capacities from test results in this study were based on first fracture of the beam flanges. If second fracture of the beam flanges were allowed, even greater rotation capacities could be obtained (Table 5.2).

Table 5.2 Summary of design values and test results

ID	Tying resistance			Flexural resistance			Rotation capacities				
	Design (kN)	Test (kN)	Ratio T/D	Design (kNm)	Test (kNm)	Ratio T/D	Design (rad)	Test* (rad)	Ratio* T/D	Test† (rad)	Ratio† T/D
C75W-M	1240.3	797.4	0.64	139.5	190.	1.36	0.06	0.09	1.50	0.16	2.80
C75W-S	1240.3	743.2	0.60	102	147.6	1.45	0.06	0.09	1.53	0.20	3.61
C100W-M	1240.3	746	0.60	175.2	207.6	1.18	0.06	0.07	1.28	0.13	2.23
C75W-MR	1240.3	799.8	0.64	134.2	187.7	1.40	0.06	0.07	1.18	0.16	2.80
C75W-Mrbs	1006.3	805.9	0.80	113.4	150.2	1.32	0.08	0.13	1.59	0.16	1.96

*Based on first fracture of beam flanges.

†Based on second fracture of beam flange.

5.5 Comparison with composite joints with FP connection

In Chapter 4, composite joints with FP connection were also tested under middle column removal scenarios. Four specimens were chosen for comparison purpose: middle joints C75FP-M, C100FP-M and C75FP-MR; side joint C75FP-S. To evaluate the resistance of WUF-B and FP connections, a comparison is shown in Table 5.3. Joints with WUF-B connections had greater tying and flexural resistances than joints with FP connections in general due to welded beam flanges to the column flange. As shown in Table 5.3, the ratios T/D of tying resistance of both types of joints were smaller than 1.0, showing that design tying resistance could not be achieved. For both types of joints, partial damage of the connection occurred at FA stage. Therefore, the design values, which were based on pure tie force without bending moment, could not be achieved at large deformation stage. The ratios T/D of all the joints with WUF-B connections were greater than 0.60 while those of most of joints with FP connections were much smaller, indicating that the former had better tying performance. The side joint C75FP-S was an exception and it had greater T/D compared to C75W-S. This is probably because the unrestrained beam flange of C75FP-S was not welded to the column flange. At FA stage, lever arms of C75FP-S were smaller than those of C75W-S so that the former was not so severely damaged. By comparison, flexural resistance of both types of joints were greater than 1.0,

showing that design flexural resistance could be achieved. There was much better agreement for both types of joints based on T/D of flexural resistance.

When comparing rotation capacity, the ratios T/D of both types of joints were close to or greater than 1.0, showing the design rotation capacity could be achieved. Most of WUF-B connections had greater ratios T/D compared to FP connections. However, rotation capacities of WUF-B connections from test results were smaller than those of FP connections so that the latter had better rotation capacity.

Table 5.3 Comparison of WUF-B and FP connections

ID	Tying resistances			Flexural resistances			Rotation capacities		
	Design (kN)	Test (kN)	Ratio T/D	Design (kNm)	Test (kNm)	Ratio T/D	Design (rad)	Test (rad)	Ratio T/D
C75W-M	1240.3	797.4	0.64	139.5	190	1.36	0.06	0.09	1.50
C75FP-M	219.8	88.8	0.40	42.8	49.9	1.17	0.10	0.11	1.10
C75W-S	1240.3	743.2	0.60	102	147.6	1.45	0.06	0.09	1.53
C75FP-S	219.8	223.8	1.02	20.6	22.7	1.10	0.10	0.16	1.60
C100W-S	1240.3	746	0.60	175.2	207.6	1.18	0.06	0.07	1.28
C100FP-S	219.8	0	0	51.9	57.1	1.10	0.10	0.09	0.90
C75W-MR	1240.3	799.8	0.64	134.2	187.7	1.40	0.06	0.07	1.18
C75FP-MR	219.8	22.9	0.10	42.8	48.9	1.14	0.10	0.10	1.00

5.6 Summary and conclusions

Five composite joints with WUF-B and RBS connections were tested under a middle column removal scenario. Load-resisting mechanism, failure mode and energy absorption capacities of the joints were investigated. Besides, resistance and rotation capacities of the joints were compared with design values. The following conclusions can be drawn:

- (1) Applied load was sustained by flexural, compressive arch and catenary actions for composite joints with WUF-B connections under column removal scenario. The contribution of compressive arch action was negligible compared to that of flexural action. Before beam flange first fractured, applied load was sustained by flexural action. After that, it was sustained by catenary action at large deformation stage.

- (2) Compared to WUF-B joints, the RBS joint could resist greater load and had better rotation capacity when failure criterion was based on first fracture of beam flanges. Energy absorption of RBS joint was greater than WUF-B joint.
- (3) Failure of concrete initiated nonlinear load-resisting mechanism of composite joints with WUF-B connections. Failure mode was characterised by sequential fracture of beam flanges.
- (4) Middle joint had greater energy absorption capacity than side joint. With an increase of slab thickness, energy absorption of middle joint was reduced at large deformation stage.
- (5) Design flexural resistance and rotation capacity of composite joints with WUF-B and RBS connections could be achieved while design tying resistance could not be achieved since all the composite joints could not meet the 0.2 rad criterion based on UFC 4-023-03. However, tie force requirement of Eurocode 1 Part 1-7 (without any specification of rotation capacity) could be met.
- (6) Composite joints with WUF-B connection had better flexural and tying resistances than those with FP connection. However, the latter had better rotation performance.

CHAPTER 6: EXPERIMENTAL TESTS OF COMPOSITE JOINTS WITH FIN PLATE CONNECTIONS SUBJECTED TO IMPACT LOADS

6.1 Introduction

With an increase of abnormal loading conditions from terrorists' attacks, there is an urgent need to investigate beam-column joints subjected to dynamic loading. These dynamic actions could arise from falling debris from a damaged floor above a particular storey. The pertinent question is whether the beam-column joint could withstand the impact from falling debris. Therefore, composite joints in Chapter 4 are investigated under dynamic loading scenario. An increase of the resistance of beam-column joints from strain rate effect is introduced. This chapter presents a test programme of composite joints with FP connections subjected to impact loads from a low-velocity drop-weight hammer. The joints were kept the same as those in Chapter 4 for comparison purpose, although there were slight differences in material properties. Test results, including impact forces, axial force and bending moment at the joints, failure mode and development of strain were investigated. A comparison of design resistance with test results was presented. Structural performance of the joints under the impact loads was also compared with quasi-static tests in Chapter 4.

6.2 Test programme

6.2.1 Test specimens and material properties

As shown in Table 6.1, four composite joints were tested in this programme. The nomenclature is as follows: C stands for composite slab, FP for fin plate, M for mass, and H for height. For instance, specimen C75FP-M530H3 had a 75 mm thick composite slab and fin plate connections. It was subjected to an impact load from a 530 kg mass hammer dropping from 3 m height. Specimens C75FP-M530H3 and C75FP-M770H1.425 had the same design shown in Fig. 6.1(a). For comparison purpose, they also shared the same design as middle joint C75FP-M in Chapter 4. Fig. 6.1(b) shows side joint C75FP-M530H3-S, which had the same design as

C75FP-S. C100FP-M530H3 had a 100 mm thick slab, the same as C100FP-M as shown in Fig. 6.1(c). Impact height in Table 6.1 was measured by a laser rangefinder before each impact test and velocity was obtained from analysing video captured by high speed camera. It should be noted that the impact velocities provided in Table 6.1 were slightly smaller than those calculated from the height probably due to slight friction along the vertical guide rail in the impact test set-up.

Table 6.1 Summary of test specimens

Loading scenario	ID	Drop-weight (kg)	Height (m)	Impact velocity (m/s)	Momentum (kgm/s)	Energy (kJ)
Impact	C75FP-M530H3	530	3	7.518	3985	15.0
	C75FP-M770H1.425	770	1.425	5.020	3865	9.7
	C75FP-M530H3-S	530	2.994	7.388	3916	14.5
	C100FP-M530H3	530	2.995	7.469	3959	14.8

Nomenclature: C - Composite; FP - Fin plate; M - Mass, kg; H - Drop-height, m; S - Side joint

Fig. 6.1 Detailing of specimens: (a) C75FP-M530H3 and C75FP-M770H1.425; (b) C75FP-M530H3-S; (c) C100FP-M530H3

Steel grade used in the impact test was the same as that for the quasi-static test. Properties of steel material are listed in Table 6.2 based on standard coupon tests. The average concrete compressive strength was 37.0 MPa with a standard deviation of 3.6 MPa from the tests of ten concrete cylinders (150 mm diameter by 300 mm length).

Table 6.2 Material properties of steel

Material	Steel Grade	Yield strength (MPa)	Modulus of elasticity (GPa)	Ultimate strength (MPa)	Fracture strain*
S355	Beam web	420	209	575	0.30
S355	Beam flange	427	199	586	0.24
S275	Fin plate	370	202	513	0.30
550	Profiled sheeting	482	203	584	0.12
R	R6	372	204	533	0.28

*Fracture strain was obtained from proportional coupon gauge length of $5.65\sqrt{S_0}$, where S_0 is the original cross-sectional area of coupons.

6.2.2 Test set-up and instrumentation

Fig. 6.2 shows a three-dimensional view of the test set-up. The beam-column joint was restrained by two A-frames connected by two horizontal pinned supports. The A-frames were anchored to a strong floor. Boundary conditions from adjacent structures were represented by idealised pins located roughly at the mid-span, similar to the quasi-static test under column removal scenario in Chapter 4. The middle column stub of the beam-column joint was placed at the centre of the drop-weight test machine, similar to the impact test on bare steel joints in Chapter 3.

Fig. 6.2 Test set-up in three-dimensional perspective

Similar to the quasi-static tests in Chapter 4, two circular hollow section (CHS 219×12.5) members were used to measure the internal forces developed in the specimen. Due to symmetry, a front view of the left CHS member is shown in Fig. 6.3(a). In the impact tests, strain gauges were attached to only one cross section due to limited recording channels and Fig. 6.3(b) shows the layout of strain gauges. Similarly, a CHS member (Fig. 6.4(a)) was attached to the top of the middle column stub to measure the impact force, since the load applied was beyond the capacity of the Kistler load cell. Fifteen mm thick stiffeners were welded to the top plate of the CHS member to prevent buckling. The layout of strain gauges and the locations of CHS are shown in Fig. 6.5 for the middle joint and Fig. 6.6 for the side joint, respectively. For the middle joint, two cross-sections at the right composite beam were attached with strain gauges as shown in Fig. 6.7(a). Section 1-1 was close to the beam-column connection as shown in Fig. 6.7(b), while Section 2-2 (Fig. 6.7(c)) lay in the middle of the composite beam. Concrete strain gauge (C1) was attached to the centreline of the specimen. A similar layout for strain gauges was adopted for the side joint as shown in Fig. 6.8(a). The two cross-sections are shown in Figs. 6.9(b) and (c), respectively. The strain gauges were connected to DEWE SIRIUS STG

DSUB-9 and TML multi-recorder. Each datalogger system had 16 channels and data were recorded at a sampling rate of 100 kHz. Low pass filters at 1 kHz were applied to eliminate environmental noise. A Phantom V310 high-speed camera was used to capture the movement of the middle column stub and the deformations of the right connection. The video sampling rate was 4000 frames per second.

Fig. 6.3 Detailing of steel circular members CHS 219×12.5: (a) Front view; (b) Section 1-1

Fig. 6.4 Detailing of steel circular member CHS 168×14: (a) Front view; (b) Section 1-1

Fig. 6.5 Layout of steel circular hollow section members for the middle joint

Fig. 6.6 Layout of steel circular hollow section members for the side joint

Fig. 6.7 Layout of strain gauges of the middle joint: (a) Front view; (b) Section 1-1; (c) Section 2-2

Fig. 6.8 Layout of strain gauges of the side joint: (a) Front view; (b) Section 1-1; (c) Section 2-2

6.3 Test results and discussions

6.3.1 Structural response

Structural responses of each beam-column joint included the impact force, beam axial force, bending moment at the joint and displacement of the middle column stub. Structural responses of C75FP-M530H3 and C75FP-M770H1.425 are compared in

Fig. 6.9. As shown in Table 6.1, the major difference between the two specimens is the impact velocity and energy, while there is little difference in momentum. As shown in Fig. 6.9(a), the peak impact force of C75FP-M770H1.425 was smaller than that of C75FP-M530H3, although they had similar momentum (Table 6.1). A clearer view is shown in Fig. 6.9(b) when the horizontal axis (displacement) is reduced to 50 mm. The impact force of C75FP-M770H1.425 acted at a smaller displacement than that of C75FP-M530H3. The reason was that C75FP-M770H1.425 had a smaller impact velocity, although the impact mass was greater. It was validated that velocity has a greater influence on impact force than mass. Both impact loads (energy) were sufficient to sever the composite beam-column joints so that both specimens were completely damaged. Therefore, full developments of axial force and bending moment at the joints could be achieved. Fig. 6.9(c) shows a comparison of axial forces of the two specimens. Similar axial forces were observed at compressive arch action stage while at catenary action stage, peak axial force (as well as fracture of fin plate) of C75FP-M770H1.425 occurred at a smaller displacement (254 mm versus 292 mm of C75FP-M530H3). However, the two peak forces were similar in magnitude (104.9 kN versus 96.8 kN). A similar phenomenon was observed when comparing bending moments as shown in Fig. 6.9(d). Peak bending moment of C75FP-M770H1.425 occurred at a smaller displacement (58 mm versus 74 mm of C75FP-M530H3) while its value (146.1 kNm) was close to that (133.4 kNm) of C75FP-M530H3. The reason was that FP connections in C75FP-M770H1.425 were subjected to lower velocity. The increase in axial force and bending moment by strain rate effect was smaller than that of C75FP-M530H3 so that the peak resistances came slightly earlier. In the meantime, brittle failure caused by high strain rate was not so severe as that in C75FP-M530H3 so that peak axial force and bending moment could be maintained and were slightly greater than those of C75FP-M530H3. It should be noted that at the initial stage (roughly before 25 mm of displacement), the negative vibrations in axial forces and the spikes in bending moments were caused by stress waves at about 200 Hz frequency. The stress waves corresponded to the high vibration mode of the specimen and were triggered by the first strike of the impactor. A comparison of displacement of the middle column stub is shown in Fig. 6.10. Due

to lower impact velocity, velocity of the middle column stub in C75FP-M770H1.425 was also smaller (3.16 m/s versus 4.40 m/s in C75FP-M530H3) as obtained from Fig. 6.10.

Fig. 6.9 Comparison of structural responses of specimens subjected to different impact loads: (a) Impact force development; (b) Displacement reduced to 50 mm scale; (c) Beam axial force development; (d) Beam bending moment development

Fig. 6.10 Displacement of the middle column stub of each FP joint captured by high-speed camera

A comparison of the structural response of the middle and the side joints is shown in

Fig. 6.11. Since specimens C75FP-M530H3 and C75FP-M530H3-S had similar mass and inertia, the peak impact forces were generally similar as shown in Fig. 6.11(a). When reducing the scale of displacement to 50 mm (Fig. 6.11(b)), it can be seen that the peak impact force of middle joint C75FP-M530H3 was achieved at 0.9 mm displacement, smaller than 1.9 mm of side joint C75FP-M530H3-S. This was because the composite slab in the middle joint contributed to flexural resistance more effectively when subjected to compression so that the middle joint was stiffer than the side joint. Therefore, the first spike was sharper at a smaller displacement than the side joint as shown in Fig. 6.11(b). The side joint was unable to mobilise compressive arch action because stable compression force was not observed in Fig. 6.11(b). Tension force developed at large deformation stage for the side as shown in Fig. 6.11(c). The peak tension force was greater (105.1 kN versus 96.8 kN) but occurred at a smaller displacement (205 mm versus 292 mm) compared to that of the middle joint. Compression force at the initial stage was introduced by stress waves, which corresponded to the first strike of the impactor in Fig. 6.11(b). Greater bending moment was achieved in the stiffer middle joint as shown in Fig. 6.11(d). Besides, smaller velocity (4.40 m/s versus 4.58 m/s in the side joint) was observed in the middle joint.

Fig. 6.11 Comparison of structural responses of specimens with different joints: (a) Impact force development; (b) Displacement reduced to 50 mm scale; (c) Beam axial force development; (d) Beam bending moment development

Fig. 6.12 compares the three middle joints with different slab thicknesses. Without any slab (0 mm thickness), the middle column stub in FP6-M530H3 obtained greater velocity (6.52 m/s than 4.40 m/s in C75FP-M530H3 calculated from Fig. 6.10) due to smaller mass and stiffness. Therefore, the second impact for FP6-M530H3 occurred at a greater displacement as shown in Fig. 6.12(a). Similarly, the second collision for C100FP-M530H3 should have occurred at a smaller displacement compared to C75FP-M530H3. However, it occurred at a greater displacement, even greater than FP6-M530H3 instead. The reason was probably that the 100 mm thick concrete slab of C100FP-M530H3 absorbed more energy than the other two specimens so that it was completely damaged during the first collision. Therefore, velocity of the middle column stub (4.72 m/s) of C100FP-M530H3 could be slightly greater than that (4.40 m/s) of C75FP-M530H3, although it was smaller than that (6.52 m/s) of FP6-M530H3. Due to greater mass and inertia, a decrease in impactor velocity for C100FP-M530H3 was more significant than the other two specimens. As a result, it took more time for the second collision. When reducing the scale of

displacement to 50 mm (Fig. 6.12(b)), it can be seen that greater mass and inertia contributed to a greater impact force in general. Compared to C75FP-M530H3, FP6-M530H3 had a smaller stiffness so that the peak impact force occurred at a greater displacement (4.5 mm versus 0.9 mm). Although C100FP-M530H3 had a greater stiffness than C75FP-M530H3, the thicker composite slab was more severely damaged during the impact. Therefore, the peak impact force occurred at a greater displacement (2.5 mm versus 0.9 mm). Beam axial forces at the joints are compared in Fig. 6.12(c). Without the composite slab effect, development of axial force of FP6-M530H3 was different from those of composite joints. Compression stage was not observed for FP6-M530H3. When comparing the two composite joints, C100FP-M530H3 had a smaller peak impact force as well as a smaller displacement than C75FP-M530H3 due to more demand on fin plate deformation at the initial flexural action stage. However, it had smaller bending moment than C75FP-M530H3 due to more severely damaged composite slab as shown in Fig. 6.12(d). Without the composite slab effect, FP6-M530H3 was not able to resist bending moment compared to either C75FP-M530H3 or C100FP-M530H3.

Fig. 6.12 Comparison of structural responses of specimens with different slab thickness: (a) Impact force development; (b) Displacement reduced to 50 mm scale; (c) Beam axial force development; (d) Beam bending moment development

6.3.2 Failure mode

Failure mode of the four composite FP joints is shown in Fig. 6.13. As shown in Fig. 6.13(a), failure of middle joint C75FP-M530H3 included crushing of concrete slab, fracture of profiled sheeting and tensile fracture of the right fin plate. C75FP-M770H1.425 had a similar failure mode as shown in Fig. 6.13(b), indicating that the change of impact velocity did not affect the failure mode. Fig. 6.13(c) shows typical side joint C75FP-M530H3-S. Failure included fracture of concrete, reinforcing bar and profiled sheeting as well as tensile fracture of the right fin plate. With a thicker concrete slab, C100FP-M530H3 showed typical failure mode of the middle joint (Fig. 6.13(d)), the same as those of C75FP-M530H3 and C75FP-M770H1.425.

Concrete crack patterns of the four composite joints are shown in Fig. 6.14. As shown in Figs. 6.14(a) and (b), two crack patterns were observed for middle joints C75FP-M530H3 and C75FP-M770H1.425, i.e. longitudinal cracks from longitudinal shear and diagonal cracks from punching shear effect. By comparison, side joint C75FP-

M530H3-S sustained transverse cracks (Fig. 6.14(c)) since the concrete slab was in tension. Longitudinal shear cracks were also observed for the side joint. With a thicker slab, crushing of concrete close to the column stub was more severe for C100FP-M530H3 as shown in Fig. 6.14(d). Although longitudinal and diagonal cracks were observed, they were not as severe as those for C75FP-M530H3 and C75FP-M770H1.425.

Fig. 6.13 Failure mode of different specimens: (a) C75FP-M530H3; (b) C75FP-M770H1.425; (c) C75FP-M530H3-S; (d) C100FP-M530H3

Fig. 6.14 Concrete crack patterns of composite FP joints: (a) C75FP-M530H3; (b) C75FP-M770H1.425; (c) C75FP-M530H3-S; (d) C100FP-M530H3

6.3.3 Development of strain

Development of strains for typical middle joint C75FP-M530H3 is shown in Fig. 6.15. Locations of each strain gauge are shown in Fig. 6.7. From Fig. 6.15(a), concrete in the composite slab was subjected to compression based on strains at centreline (C1) and connection section (C2). Similarly, reinforcing bars in the composite slab at connection section (middle bar MRR2 and side bar MRR3) were subjected to compression as shown in Fig. 6.15(b). The compression force was enough to cause yielding of the reinforcing bars so that both strains were close to $3000 \mu\epsilon$ at a displacement of about 100 mm. By comparison, reinforcing bars at the right cross-section (section 2-2 in Fig. 6.7) were subjected to tension force induced by impact stress waves at the initial stage. After that, tensile strain started to decrease. Compressive strain was observed in the continuous side bar (RR3), indicating that the composite slab was subjected to compression. Fig. 6.15(c) shows the development of strain of the profiled sheeting and a tensile strain up to $3500 \mu\epsilon$ was observed. In addition, tensile strain of the restrained beam flange was also observed as shown in in Fig. 6.15(d), indicating that the neutral axis of bending moment at the connection section lay within the composite slab thickness.

Fig. 6.15 Development of strains of different components in specimen C75FP-M530H3 (middle joint): (a) Concrete; (b) Reinforcing bar; (c) Profiled sheeting; (d) Steel beam

Fig. 6.16 shows the development of strains for typical side joint C75FP-M530H3-S and the locations of each strain gauge are shown in Fig. 6.8. Concrete at the centreline (C1) and the connection section (C2) was subjected to tension as shown in Fig. 6.16(a). Continuous side reinforcing bars in the composite slab were also subjected to tension at connection section (MRR3) and right cross-section (RR3) as shown in Fig. 6.16(b). By comparison, tensile strains of the discontinuous middle reinforcing bar (MRR2 and RR2) were much smaller. Besides, the profiled sheeting was subjected to tension (Fig. 6.16(c)) and unrestrained beam flange was subjected to compression (Fig. 6.16(d)), indicating that the neutral axis of bending moment at the connection section lay in the beam web.

Fig. 6.16 Development of strains of different components in specimen C75FP-M530H3-S (side joint): (a) Concrete; (b) Reinforcing bar; (c) Profiled sheeting; (d) Steel beam

Strain rate was calculated from differentiating recorded strain with time. The peak strain rates at different locations for composite FP joints are summarised in Table 6.3. Dynamic increase factors (*DIFs*) for each material strength are also presented in Table 6.3. For concrete, *DIF* was calculated based on the *fib* Model Code (fib 2013) and *DIFs* of steel material were from the Cowper-Symonds model (Equation 3-1). For C75FP-M530H3, concrete strength can be increased by 28% in tension and by 18% in compression, respectively. The respective *DIFs* for tensile and compressive strength of steel components including profiled sheeting, reinforcing bar and I-beam can be up to 1.13 and 1.15. Middle joint C75FP-M770H1.425 was subjected to a greater impact mass but lower impact velocity compared to C75FP-M530H3. However, *DIFs* were generally similar to those of C75FP-M530H3 as shown in Table 6.3. For side joint C75FP-M530H3-S, 27% increase of tensile strength can be expected for concrete. For steel components, the maximum increase of tensile strength (16%) occurred in the side reinforcing bar at the right cross-section, corresponding to a strain rate of 5.60 s^{-1} . The maximum compressive strain rate was

0.97 s^{-1} occurring at right cross-section of the middle reinforcing bar. With a thicker slab, C100FP-M530H3 had no tensile strain rate in the concrete while the compressive strain rate was up to 2.51 s^{-1} with 17% increase of compressive strength. For steel material, the respective maximum tensile and compressive strain rates were 1.74 s^{-1} and 2.32 s^{-1} , inducing 12% and 13% increases of strength, respectively. For both concrete and steel materials, the order of magnitude of 1 s^{-1} strain rate was obtained from the low velocity impact test.

Table 6.3 Peak strain rates and *DIFs* at different locations for composite FP joints

Specimen ID	Material	Strain type	Strain gauge ID	Location	Strain rate (s ⁻¹)	<i>DIF</i>
C75FP-M530H3	Concrete	Tension	C2	Slab middle-right	0.77	1.28
		Compression	C2	Slab middle-right	-3.29	1.18
	Reinforcing bar	Tension	RR2	Reinforcing bar #2 right	2.32	1.13
		Compression	MRR2	Reinforcing bar #2 middle-right	-3.86	1.15
	Profiled sheeting	Tension	MRP1	Profiled sheeting middle-right	0.77	1.10
		Compression	MRP1	Profiled sheeting middle-right	-0.39	1.08
	I-beam	Tension	R2	Restrained beam flange	1.16	1.11
		Compression	R5	Unrestrained beam flange	-1.16	1.11
	Concrete	Tension	C2	Slab middle-right	0.58	1.27
		Compression	C2	Slab middle-right	-1.74	1.17
C75FP-M770H1.425	Reinforcing bar	Tension	RR3	Reinforcing bar #3 right	2.32	1.13
		Compression	MRR3	Reinforcing bar #3 middle-right	-3.87	1.15
	Profiled sheeting	Tension	MRP1	Profiled sheeting middle-right	1.16	1.11
		Compression	MRP1	Profiled sheeting middle-right	-0.58	1.09
	I-beam	Tension	R5	Unrestrained beam flange	0.58	1.09
		Compression	R5	Unrestrained beam flange	-0.77	1.10
	Concrete	Tension	C1	Slab centre	0.58	1.27
		Compression	C1	Slab centre	-0.97	1.16
C75FP-M530H3-S	Reinforcing bar	Tension	RR1	Reinforcing bar #1 right	5.60	1.16
		Compression	RR2	Reinforcing bar #2 right	-0.97	1.10
	Profiled sheeting	Tension	RP1	Profiled sheeting right	2.13	1.13
		Compression	RP1	Profiled sheeting right	-0.77	1.10
	I-beam	Tension	R2	Unrestrained beam flange	1.35	1.11
		Compression	R2	Unrestrained beam flange	-0.77	1.10
	Concrete	Tension	C2	Slab middle-right	0	0.00
		Compression	C2	Slab middle-right	-2.51	1.17
C100FP-M530H3	Reinforcing bar	Tension	RR2	Reinforcing bar #2 right	1.74	1.12
		Compression	MRR2	Reinforcing bar #2 middle-right	-2.32	1.13
	Profiled sheeting	Tension	MRP1	Profiled sheeting middle-right	0.97	1.10
		Compression	RP2	Profiled sheeting right	-0.58	1.09
	I-beam	Tension	R4	Unrestrained beam flange	0.77	1.10
		Compression	R5	Unrestrained beam flange	-1.16	1.11

6.4 Comparison of design resistance and test results

A comparison of design values and test results is shown in Table 6.4. The design resistances and rotation capacities were obtained based on the design calculations in Chapter 4. The test values were obtained from Figs. 6.18 to 6.20. Since vibrations of bending moment were observed, the average value of data between the peak value and the trough value was used to represent the measured flexural resistance. From Table 6.4, C75FP-M530H3 and C75FP-M770H1.425 had much greater flexural resistances than the design values due to strain rate effect. Their rotation capacities were also greater than the design values. Due to greater demand on deformation capacity of the fin plates at the initial stage for composite FP joints, test values of tying resistances were smaller than the design values. By comparison, side joint C75FP-M530H3-S had greater tying resistance. This was because the slab was in tension so that the composite slab effect was weaker than the middle joints. Demand on deformation capacity of the fin plates at the initial stage was smaller, so that the fin plates were not so severely damaged. Therefore, a greater tying resistance can be obtained at large deformation stage compared to the middle joints. C100FP-M530H3 had a thicker composite slab so that a greater flexural resistance and a smaller tying resistance were expected. However, more energy was absorbed by the thicker slab and it was severely damaged during the impact. As a result, a weaker flexural resistance and a tying resistance were obtained instead.

Table 6.4 Summary of design values and test results for composite FP joints

ID	Tying resistances			Flexural resistances			Rotation capacities		
	Design (kN)	Test (kN)	Ratio T/D	Design (kNm)	Test* (kNm)	Ratio T/D	Design (rad)	Test (rad)	Ratio T/D
C75FP-M530H3	211.5	96.8	0.46	41.1	104.8	2.55	0.10	0.18	1.80
C75FP-M770H3	211.5	104.9	0.50	41.1	102.5	2.49	0.10	0.16	1.60
C75FP-M530H3-S	211.5	150.1	0.71	17.4	69.2	3.98	0.10	0.12	1.20
C100FP-M530H3	211.5	64.1	0.30	48.9	64.1	1.31	0.10	0.16	1.60

*Average value of data between the peak value and the trough value

6.5 Comparison with quasi-static tests on composite FP joints

Fig. 6.17 shows a comparison of middle joints with 75 mm thick composite slabs subjected to quasi-static (Chapter 4) and impact loads. Developments of axial force at the beam-column joints were similar as shown in Fig. 6.17(a): compression force at small deformation stage due to compressive arch action (CAA) and tension force at large deformation stage due to catenary action (CA). When subjected to impact loads, it is clear that CAA of the middle joint was greater than that of quasi-static tests. By comparison, CA of the middle joint under impact loads was close to that of quasi-static tests although the failure displacements of the middle joint subjected to impact loads were greater than that of C75FP-M (quasi-static tests). From Fig. 6.17(b), flexural action (FA, based on bending moment) of middle joint was much greater when subjected to impact loads due to strain rate effect.

Fig. 6.18 shows a comparison of side joints with 75 mm thick composite slabs subjected to quasi-static (Chapter 4) and impact loads. Except for vibrations caused by the impact at the initial stage, developments of axial forces for the side joints subjected to both loads were similar and only CA developed as shown in Fig. 6.18(a). Unlike the middle joint, failure displacement of the side joint subjected to impact loads was smaller than that subjected to quasi-static loads. The reason was probably that the demand on compressive strength of the upper bolt row was greater and damage started accumulating at small deformation stage when subjected to impact loads. Thus, failure occurred earlier at large deformation stage. Compared to quasi-static tests, the side joint subjected to impact loads could develop much greater FA as shown in Fig. 6.18(b) due to strain rate effect.

A comparison of middle joints with 100 mm thick thicker slabs subjected to quasi-static and impact loads is shown in Fig. 6.19. Due to early damage of the thicker composite slab, the respective increase of CAA (Fig. 6.19(a)) and FA (Fig. 6.19(b)) at small deformation stage was not so evident as those of middle joints with 75 mm thick slab. By comparison, the increase of CA at large deformation was more evident as shown in Fig. 6.19(a).

Fig. 6.17 Comparison of middle FP joints subjected to quasi-static and impact loads: (a) Beam axial force development; (b) Beam bending moment development

Fig. 6.18 Comparison of side FP joints subjected to quasi-static and impact loads: (a) Beam axial force development; (b) Beam bending moment development

Fig. 6.19 Comparison of middle FP joints (thicker slab) subjected to quasi-static and impact loads: (a) Beam axial force development; (b) Beam bending moment development

Furthermore, a comparison of composite FP joints subjected to quasi-static and impact loads is shown in Table 6.5. The ratio T/D represents the ratio of test value normalised by the design value. Therefore, a greater T/D ratio represents a greater increase of resistance and rotation capacity. From Table 6.5, it can be seen that most of the joints cannot develop catenary action as the design tying resistance ($T/D < 1$)

since the deformation capacity of the FP connection was exhausted at FA stage. Due to strain rate effect, all the middle joints including C75FP-M530H3, C75FP-M770H1.425 and C100FP-M530H3 had greater T/D ratios than those of quasi-static test specimens in terms of tying and flexural resistances as well as rotation capacities. Similarly, side joint C75FP-M530H3-S had a greater T/D ratio than C75FP-S due to strain rate effect. By comparison, due to accumulation of compressive damage of the upper bolt rows during the impact, smaller T/D ratios of tying resistance and rotation capacity were observed.

Table 6.5 Comparison of composite FP joints subjected to quasi-static and impact loads

ID	Tying resistances			Flexural resistances			Rotation capacities		
	Design (kN)	Test (kN)	Ratio T/D	Design (kNm)	Test (kNm)	Ratio T/D	Design (rad)	Test (rad)	Ratio T/D
C75FP-M	219.8	88.8	0.40	42.8	49.9	1.17	0.10	0.11	1.10
C75FP-M530H3	211.5	96.8	0.46	41.1	104.8	2.55	0.10	0.18	1.80
C75FP-M770H1.425	211.5	104.9	0.50	41.1	102.5	2.49	0.10	0.16	1.60
C75FP-S	219.8	223.8	1.02	20.6	22.7	1.10	0.10	0.16	1.60
C75FP-M530H3-S	211.5	150.1	0.71	17.4	69.2	3.98	0.10	0.12	1.20
C100FP-M	219.8	0	0	51.9	57.1	1.10	0.10	0.09	0.90
C100FP-M530H3	211.5	64.1	0.30	48.9	64.1	1.31	0.10	0.16	1.60

6.6 Summary and conclusions

In this chapter, a test programme on composite FP joints subjected to impact loads is presented. Four beam-column joints were designed and tested in the programme. After that, structural response, failure mode and development of strain were investigated. Based on the test results, a comparison of test and design values was conducted. Such comparison was also extended to composite FP joints subjected to quasi-static loads. Based on the experimental study, the following conclusions can be drawn:

- (1) With the same impact momentum, a greater impact velocity contributed to a greater impact force. When subjected to the same impact load, the middle and the side joints experienced similar impact force during the first collision. For specimens with a thicker slab, a greater impact force was measured due to an increase of mass and inertia.

- (2) Compressive arch action and catenary action were mobilised for middle joints subjected to impact loads. Similar to bare steel joints, only catenary action was mobilised for side joints. Flexural action could develop for middle joints and was much greater than that developed for side joints when subjected to the same impact load.
- (3) Tear-out failure of fin plate, tensile fracture of profile sheeting, longitudinal and diagonal cracks in the composite slab and crushing of concrete close to the joint governed the failure mode of the middle joint. For side joints, tensile fracture of reinforcing bars was observed, together with fracture of profiled sheeting and concrete. Final tear-out failure of fin plate of side joint was observed. Different from those cracks of middle joint, longitudinal and transverse cracks developed in the composite slab of side joints since the slab was in tension.
- (4) An intermediate level of strain rate in the order of 1 s^{-1} was recorded, leading to respective maximum increase of 28% in concrete strength and 16% in steel strength.
- (5) Composite slab effect could ensure flexural action of both the middle and the side joints subjected to impact loads. Combined with strain rate effect, much greater flexural resistances were achieved compared to the design values. Rotation capacities were also greater compared to the design values provided by UFC 4-023-03 (2013). Due to a greater demand on deformation of the fin plates at the initial stage, tying resistances from the test results were smaller than the design values. However, tie force requirement from Eurocode 1 could be met for most of the composite joints. With a thicker concrete slab (100 mm), tie force requirement could not be met.
- (6) Compared to quasi-static tests, strain rate effect could increase compressive arch, catenary and flexural actions of composite FP joints in the impact tests. Side joint C75FP-M530H3-S was an exception due to accumulation of compressive damage of upper bolt rows during the impact, smaller T/D ratios of tying resistance and rotation capacity were observed.

CHAPTER 7: EXPERIMENTAL TESTS OF COMPOSITE JOINTS WITH WUF-B CONNECTIONS SUBJECTED TO IMPACT LOADS

7.1 Introduction

Similar to the composite joints with FP connection in Chapter 6, WUF-B connections were also investigated under impact loading scenario. This chapter describes a test programme of composite joints with WUF-B connections subjected to impact loading. The same joints as those designed in Chapter 5 were tested for comparison purpose. Through the tests, structural response of WUF-B connections was investigated. Failure mode and development of strain at the composite joints were presented as well. As one type of moment-resisting connections, WUF-B connections could resist the maximum impact load from the drop-weight hammer. Although plastic deformation was observed and flexural resistance was achieved, severing of beam-column joints was not observed in these specimens. Structural performance of the composite joints under the impact loads was also compared with the quasi-static tests in Chapter 5.

7.2 Test programme

7.2.1 Test specimens and material properties

A total of four composite WUF-B joints were tested in this programme as shown in Table 7.1. The velocity, momentum and energy of the drop-weight hammer used for each specimen were also provided. Three parameters including the impact velocity, joint type and slab thickness were investigated. In Table 7.1, the specimens were named as follows: C stands for composite slab, W for WUF-B connection, M for mass and H for height. Therefore, specimen C75W-M770H3 had a 75 mm thick composite slab with WUF-B connections and was subjected to an impact load of 770 kg mass hammer dropping from 3 m height. Fig. 7.1(a) shows a front view of C75W-M770H3 and C75W-M770H2, which shared the same design as C75W-M (Fig. 5.1(a)) in Chapter 5 for comparison purpose. The top plate of the middle column stub

was different from that of C75W-M to fit into the impact test set-up. C75W-M770H3-S was a side joint subjected to hogging moment as shown in Fig. 7.1(b), the same as C75W-S as shown in Fig. 5.1(b). Besides, detailing of the WUF-B connection was the same as the middle joints. A thicker 100 mm slab was used for C100W-M770H3 as shown in Fig. 7.1(c), the same as C100W-M (Fig. 5.1(c)). Due to symmetry, only one-half of each specimen was drawn in Fig. 7.1.

Steel properties are shown in Table 6.2 (Chapter 6). The average concrete compressive strength was 50.6 MPa with a standard derivation of 5.4 MPa based on the tests of ten concrete cylinders (300 mm length with 150 mm diameter).

Table 7.1 Summary of test specimens

Loading scenario	ID	Drop-weight (kg)	Height (m)	Impact velocity (m/s)	Momentum (kgm/s)	Energy (kJ)
Impact	C75W-M770H3	770	2.998	7.619	5867	22.3
	C75W-M770H2	770	2.005	6.230	4797	14.9
	C75W-M770H3-S	770	2.997	7.357	5665	20.8
	C100W-M770H3	770	2.996	7.357	5665	20.8

Nomenclature: C - Composite; W - WUF-B; M - Mass, kg; H - Drop-height, m; S - Side joint

Fig. 7.1 Detailing of specimens: (a) C75W-M770H3 and C75W-M770H2; (b) C75W-M770H3-S;
(c) C100W-M770H3

7.2.2 Test set-up and instrumentation

The test set-up and instrumentation used in this chapter were the same as those presented in Chapter 6, except that more strain gauges were used to record strains of welded beam flanges as shown in Fig. 7.2 for the middle joint and Fig. 7.3 for the side joint, respectively. Two cross-sections were attached with strain gauges for the middle joint as shown in Fig. 7.2(a). Section 1-1 (Fig. 7.2(b)) was the connection section and section 2-2 (Fig. 7.2(c)) lay roughly at the mid-span of the right composite beam. Compared to the FP joints in Chapter 6, both the top and bottom beam flanges of WUF-B joints were attached with strain gauges since their strains were significant due to welding of the beam flanges to the column flange. A similar layout of strain gauges was used for the side joint as shown in Fig. 7.3(a). The respective layout of strain gauges at Sections 1-1 and 2-2 is shown in Figs. 7.3(a) and (b).

Fig. 7.2 Layout of strain gauges of the middle joint: (a) Front view; (b) Section 1-1; (c) Section 2-2

Fig. 7.3 Layout of strain gauges of the side joint: (a) Front view; (b) Section 1-1; (c) Section 2-2

7.3 Test results and discussions

7.3.1 Structural response

Fig. 7.4 shows a comparison of structural responses of specimens C75W-M770H3 and C75W-M770H2 subjected to different impact loads. The respective drop-heights of C75W-M770H3 and C75W-M770H2 were 2.998 m and 2.005 m with velocities of 7.619 m/s and 6.230 m/s. As shown in Fig. 7.4(a), impact forces of the two specimens were generally similar. The peak impact force of C75W-M770H2 occurred earlier (0.6 ms) than that (0.8 ms) of C75W-M770H3. When integrating the impact force with time and reducing the time axis to 0.01 s (roughly duration of the first collision), the impulse of C75W-M770H2 (Fig. 7.4(b)) was smaller than that of C75W-M770H3 due to a smaller impact velocity. The difference of the two impulses was small so that they led to similar displacements at the initial stage as shown in Fig. 7.4(c). After that, due to greater impact energy, the respective peak and residual displacements of C75W-M770H3 were greater than those of C75W-M770H2. Beam axial forces developed at the two joints are compared in Fig. 7.4(d). Due to small displacement, axial forces were not fully mobilised so that only vibrations caused by the impactor were measured. By comparison, bending moments could fully develop for both specimens as shown in Fig. 7.4(e). It should be noted that axial force and bending moment at the joint are shown until the peak displacement was achieved.

Fig. 7.4 Comparison of structural responses of specimens subjected to different impact loads: (a) Impact force development; (b) Impulse development when reducing time to 0.01 s; (c) Displacement development of middle column stub; (d) Beam axial force development; (e) Bending moment development

A comparison of structural responses of the middle and the side joints is shown in Fig. 7.5. Peak impact force of side joint C75W-M770H3-S was greater (1307.8 kN versus 1188.9 kN of middle joint C75W-M770H3) and occurred earlier (0.6 ms versus 0.8 ms) as shown in Fig. 7.5(a). However, the following plateau of impact force was longer and smaller, probably because the side joint had smaller stiffness as the composite slab was in tension. Besides, the respective peak and residual

displacements of side joint C75W-M770H3-S were much greater than those of middle joint C75W-M770H3 due to the same reason as shown in Fig. 7.5(b). Although the peak displacement of C75W-M770H3-S was greater, axial force at the joint was not fully mobilised so that catenary action was not observed (Fig. 7.5(c)). Bending moment could fully develop and was smaller for the side joint as shown in Fig. 7.5(d).

Fig. 7.5 Comparison of structural responses of specimens with different joints: (a) Impact force development; (b) Displacement of middle column stub development; (c) Beam axial force development; (d) Bending moment development

Fig. 7.6 compares the structural responses of three middle joints with various composite slab thicknesses, including bare steel joint W-M830H3 presented in Chapter 3. It can be seen from Fig. 7.6(a) that a thicker slab contributed to a greater impact force due to its greater mass and inertia. By comparison, the ensuing plateau did not vary much with the slab thickness. A thicker slab contributed to a greater flexural resistance and stiffness so that the respective peak and residual displacements of C100W-M770H3 were the smallest among all the three middle joints as shown in Fig. 7.6(b). Correspondingly, axial force at the joint of C100W-

M770H3 was not fully mobilised (Fig. 7.6(c)). By comparison, catenary action of bare steel joint W-M830H3 could be partially mobilised. With the 100 mm thick slab, C100W-M770H3 had the greatest bending moment among all the three middle joints as shown in Fig. 7.6(d) due to the composite slab effect.

Fig. 7.6 Comparison of structural responses of specimens with different slab thickness: (a) Impact force; (b) Displacement of middle column stub; (c) Beam axial force; (d) Bending moment

7.3.2 Failure mode

Fig. 7.7 shows a typical failure mode of the middle joint. It can be seen from Fig. 7.7(a) that there was no critical failure at the beam-column joint. Debonding between the concrete slab and the profiled sheeting was observed. Plastic deformation of the unrestrained beam flange developed based on the residual displacement observed in Fig. 7.4(c). However, deformations on both the left and the right sides were not evident as shown in Figs. 7.7(b) and (c). Besides, fin plates, bolts and beam webs were almost intact on both sides.

Fig. 7.8 shows a typical failure mode of the side joint. Compared to the middle joint, failure of the side joint was more evident in the composite slab as shown in Fig.

7.8(a): fracture of the concrete slab in tension and debonding of profiled sheeting were observed. Similar to the middle joint, fin plates, bolts and beam webs were almost intact on both the left and the right sides. By comparison, local buckling occurred at the unrestrained beam flanges due to compression as shown in Figs. 7.8(b) and (c).

Fig. 7.7 Failure mode of specimen C75W-M770H3 (middle joint): (a) Front view; (b) Left connection; (c) Right connection

Fig. 7.8 Failure mode of specimen C75W-M770H3-S (side joint): (a) Front view; (b) Left connection; (c) Right connection

Crack patterns of all the composite WUF-B joints are compared in Fig. 7.9. Middle joints C75W-M770H3 (Fig. 7.9(a)) and C75W-M770H2 (Fig. 7.9(b)) shared similar crack patterns. Longitudinal shear cracks and diagonal cracks caused by punching shear effect were observed for both middle joints. However, transverse cracks due to tension forces were observed in side joint C75W-M770H3-S as shown in (Fig. 7.9(c)). Longitudinal shear cracks were also evident along the re-entrant profile. C100W-M770H3 had a 100 mm thick concrete slab and the WUF-B connections underneath could resist the impact load. In contrast to C100FP-M530H3 (Fig. 6.14(d)), crushing and spalling of concrete in C100W-M770H3 were not observed in Fig. 7.9(d), although the same longitudinal and diagonal cracks were observed.

Fig. 7.9 Concrete crack patterns of composite WUF-B joints: (a) C75W-M770H3; (b) C75W-M770H2; (c) C75W-M770H3-S; (d) C100W-M770H3

7.3.3 Development of strain

Development of strains for various components such as concrete, reinforcing bar, steel profiled sheeting and beam flange of typical middle joint C75W-M770H3 is shown in Fig. 7.10, where compressive strain was negative and tensile strain was positive. Locations of each strain gauge are shown in Fig. 7.2. Concrete at centreline and connection section was in compression as shown in Fig. 7.10(a). Similarly, reinforcing bars at connection section in the composite slab were also subjected to compression force and their strains exceeded $3000 \mu\epsilon$ (MRR2 for middle bar and MRR3 for side bar in Fig. 7.10(b)). Reinforcing bars at the right cross-section were also subjected to compression (RR2 for the middle bar and RR3 for the side bar). Tensile strains of RR2 and RR3 were caused by stress waves at the initial stage immediately after the impact. As shown in Fig. 7.10(c), vibrations of strain of the profiled sheeting were observed due to debonding between the concrete slab and the sheeting. The magnitude of compressive value was greater than that of the tensile one. By comparison, the restrained beam flange (MR1) was in compression while

the unrestrained flange (MR2) yielded in tension (tensile strain was greater than $3000 \mu\epsilon$), indicating that the neutral axis of bending moment at connection section lay within the beam web.

Development of strains for various components of typical side joint C75W-M770H3-S is shown in Fig. 7.10 and the locations of each strain gauge are shown in Fig. 7.3. Concrete at centreline (C1) was subjected to tension as shown in Fig. 7.10(a). However, concrete at connection section (C2 in Fig. 7.10(a)) and middle reinforcing bar at the same section (MRR2 in Fig. 7.10(b)) were subjected to compression because they were bearing on the column flange and subjected to shock waves after the impact. Side reinforcing bar at connection section (MRR3) was subjected to tension as shown in Fig. 7.10(b). Furthermore, middle (RR2) and side (RR3) reinforcing bars at right cross-section yielded in tension. Profiled sheeting was also subjected to tension at connection section as shown in Fig. 7.10(c). Similar to middle joint C75W-M770H3, unrestrained beam flange (MR1) yielded in compression while restrained flange (MR2) yielded in tension as shown in Fig. 7.10(d), indicating that the neutral axis of bending moment at connection section lay within the beam web.

Fig. 7.10 Development of strain of different components in specimen C75W-M770H3 (middle joint): (a) Concrete; (b) Reinforcing bar; (c) Profiled sheeting; (d) Steel beam

Fig. 7.11 Development of strain of different components in specimen C75W-M770H3-S (side joint):

(a) Concrete; (b) Reinforcing bar; (c) Profiled sheeting; (d) Steel beam

Table 7.4 shows the peak strain rates obtained by differentiating recorded strain with time, as well as corresponding dynamic increase factors (*DIFs*) for each material. For middle joint C75W-M770H3, tensile strain of concrete was not observed while the peak compressive strain rate was 2.32 s^{-1} , contributing to 17% increase of compressive strength. The maximum tensile strain rate of steel components (including profiled sheeting, reinforcing bar and I-beam) was 7.34 s^{-1} , occurring at unrestrained beam flange and inducing a 17% increase of tensile strength. Besides, the maximum compressive strain rate of steel was 4.83 s^{-1} observed in side reinforcing bar and the corresponding increase of compressive strength was 16%. C75W-M770H2 was subjected to lower impact velocity compared to C75W-M770H3. Therefore, the observed strain rates were generally smaller than those of C75W-M770H3. The peak compressive strain rate of concrete was 2.13 s^{-1} , inducing a 17% increase of compressive strength. Moreover, the respective maximum compressive and tensile strain rates of steel were 3.67 s^{-1} in the side reinforcing bar and 6.18 s^{-1} at the unrestrained beam flange. Side joint C75W-M770H3-S had the greatest peak tensile strain rate (0.77 s^{-1}) of concrete among the four specimens since

the composite slab was in tension. It also sustained the greatest peak compressive strain rate (6.38 s^{-1}) of steel and the corresponding *DIF* was 1.17. With a thicker slab, C100W-M770H3 was more rigid and had a greater mass and inertia so that it sustained slightly smaller compressive strain rate (1.93 s^{-1}) of concrete compared to C75W-M770H3. It also sustained marginally smaller maximum compressive (3.67 s^{-1}) and tensile (7.15 s^{-1}) strain rates of steel compared to C75W-M770H3 as shown in Table 7.4.

Table 7.2 Peak strain rates and *DIFs* at different locations of composite WUF-B joints

Specimen ID	Material	Strain type	Strain gauge ID	Location	Strain rate (s ⁻¹)	<i>DIF</i>
C75W-M770H3	Concrete	Tension	/	/	/	/
		Compression	C2	Slab middle-right	-2.32	1.17
	Reinforcing bar	Tension	RR3	Reinforcing bar #3 right	3.48	1.14
		Compression	RR3	Reinforcing bar #3 right	-4.83	1.16
	Profiled sheeting	Tension	MRP1	Profiled sheeting middle-right	0.97	1.10
		Compression	MRP1	Profiled sheeting middle-right	-1.35	1.11
	I-beam	Tension	MR2	Unrestrained beam flange	7.34	1.17
		Compression	MR1	Restrained beam flange	-1.55	1.12
C75W-M770H2	Concrete	Tension	C1	Slab centre	0.58	1.27
		Compression	C2	Slab middle-right	-2.13	1.17
	Reinforcing bar	Tension	MRR1	Reinforcing bar #1 middle-right	3.67	1.15
		Compression	MRR1	Reinforcing bar #1 middle-right	-3.09	1.14
	Profiled sheeting	Tension	RP1	Profiled sheeting right	1.39	1.11
		Compression	RP1	Profiled sheeting right	-0.86	1.10
	I-beam	Tension	MR2	Unrestrained beam flange	6.18	1.17
		Compression	MR1	Restrained beam flange	-1.55	1.12
C75W-M770H3-S	Concrete	Tension	C1	Slab centre	0.77	1.28
		Compression	C2	Slab middle-right	-1.74	1.17
	Reinforcing bar	Tension	MRR1	Reinforcing bar #1 middle-right	3.87	1.15
		Compression	MRR2	Reinforcing bar #2 middle-right	-2.13	1.13
	Profiled sheeting	Tension	RP1	Profiled sheeting right	2.28	1.13
		Compression	RP2	Profiled sheeting right	-0.59	1.09
	I-beam	Tension	MR2	Restrained beam flange	4.64	1.15
		Compression	MR1	Unrestrained beam flange	-6.38	1.17
C100W-M770H3	Concrete	Tension	C2	Slab middle-right	0.19	1.24
		Compression	C2	Slab middle-right	-1.93	1.17
	Reinforcing bar	Tension	RR1	Reinforcing bar #1 right	2.71	1.13
		Compression	RR1	Reinforcing bar #1 right	-3.67	1.15
	Profiled sheeting	Tension	RP1	Profiled sheeting right	2.18	1.13
		Compression	RP1	Profiled sheeting right	-2.50	1.13
	I-beam	Tension	MR2	Unrestrained beam flange	7.15	1.17
		Compression	R1	Restrained beam flange	-1.35	1.11

7.4 Comparison of design resistance and test results

Table 7.3 summarises design values and test results for composite WUF-B joints. The design values were calculated based on the method presented in Chapter 5. Test values of flexural resistances were obtained from Figs. 7.12-15 and the average value of data between the peak value and the trough value were adopted to represent the test results. Since all the four joints could resist applied impact loads, tying resistances and rotation capacities could not be obtained from the test results. From Table 7.3, it can be seen that design flexural resistances of all the joints could be achieved in the impact tests. When comparing C75W-M770H3 and C75W-M770H2, it can be seen that with a lower impact velocity, a smaller ratio of test value normalised by design value (T/D) was observed for C75W-M770H2. With a greater mass and inertia, a smaller ratio T/D was also observed for C100W-M770H3. The reason was probably that the obtained velocities of specimens C75W-M770H2 and C100W-M770H3 were lower so that the strain rate effect was less significant. Moreover, side joint C75W-M770H3-S had the greatest ratio among all the joints since it had a smaller stiffness when the composite slab was in tension compared with the middle joints.

Table 7.3 Summary of design values and test results for composite WUF-B joint

ID	Tying resistance			Flexural resistance			Rotation capacities		
	Design (kN)	Test (kN)	Ratio T/D	Design (kNm)	Test* (kNm)	Ratio T/D	Design (rad)	Test (rad)	Ratio T/D
C75W-M770H3	1309.3	/	/	159.4	275.4	1.73	0.06	/	/
C75W-M770H2	1309.3	/	/	159.4	226	1.42	0.06	/	/
C75W-M770H3-S	1309.3	/	/	109.5	201.7	1.84	0.06	/	/
C100W-M770H3	1309.3	/	/	196.0	258.7	1.32	0.06	/	/

*Average value of data between the peak value and the trough value

7.5 Comparison with quasi-static tests on composite WUF-B joints

A comparison of middle WUF-B joints with 75 mm thick slabs subjected to quasi-static and impact loads is shown in Fig. 7.12. Compared to C75W-M (quasi-static tests), catenary action of both joints subjected to impact loads was not fully mobilised

(Fig. 7.12(a)) since the displacement was small. By comparison, bending moments for joints subjected to impact loads could fully develop and were much greater than that of C75W-M as shown in Fig. 7.12(b) due to the strain rate effect. Similar phenomena were observed for the side joints (Fig. 7.13) and the middle joints with thicker slabs (Fig. 7.14). Beam axial forces were not fully mobilised for C75W-M770H3-S (Fig. 7.13(a)) and C100W-M770H3 ((Fig. 7.14(a)) due to small displacement while bending moments were much greater than the joints subjected to quasi-static loads due to the strain rate effect as shown in Figs. 7.13(b) and 7.14(b).

Fig. 7.12 Comparison of middle WUF-B joints from quasi-static and impact tests: (a) Beam axial force development; (b) Beam bending moment development

Fig. 7.13 Comparison of side WUF-B joints from quasi-static and impact tests: (a) Beam axial force development; (b) Beam bending moment development

Fig. 7.14 Comparison of middle WUF-B joints (thicker slab) from quasi-static and impact tests: (a)

Beam axial force development; (b) Beam bending moment development

To further quantify contribution of the strain rate effect and eliminate the difference caused by different material strengths, a comparison of T/D ratios of composite WUF-B joints subjected to quasi-static and impact loads was conducted in Table 7.4. Flexural resistances can be compared since bending moment could fully develop under the applied impact loads. Compared to the T/D ratio (1.36) of C75W-M (quasi-static tests), the T/D ratio (1.73) of C75W-M770H3 was much greater since the strain rate effect was significant. It can be further validated that when reducing the impact velocity (the strain rate effect as well), the T/D ratio (1.42) of C75W-M770H2 was greater than but closer to that (1.36) of C75W-M (quasi-static tests). Besides, when comparing T/D ratios of the side joints (1.84 for the impact tests versus 1.45 for the quasi-static tests) and the middle joints with thicker slabs (1.32 for the impact tests versus 1.18 for the quasi-static tests), it can be found that the strain rate effect contributes to a much greater flexural resistance.

Table 7.4 Comparison of WUF-B connections subjected to quasi-static and impact loads

ID	Tying resistances			Flexural resistances			Rotation capacities		
	Design (kN)	Test (kN)	Ratio T/D	Design (kNm)	Test (kNm)	Ratio T/D	Design (rad)	Test (rad)	Ratio T/D
C75W-M	1240.3	797.4	0.64	139.5	190	1.36	0.06	0.09	1.50
C75W-M770H3	1309.3	/	/	159.4	275.4	1.73	0.06	/	/
C75W-M770H2	1309.3	/	/	159.4	226.0	1.42	0.06	/	/
C75W-S	1240.3	743.2	0.60	102	147.6	1.45	0.06	0.09	1.53
C75W-M770H3-S	1309.3	/	/	109.5	201.7	1.84	0.06	/	/
C100W-M	1240.3	746	0.60	175.2	207.6	1.18	0.06	0.07	1.28
C100W-M770H3	1309.3	/	/	196.0	258.7	1.32	0.06	/	/

7.6 Summary and conclusions

This chapter presents a test programme on composite WUF-B joints subjected to impact loads from a drop-weight test machine. A total of four beam-column joints were designed and tested to investigate the governing parameters including the impact velocity, joint type and slab thickness. Structural responses such as the impact force, axial force and bending moment developed at the joints were presented. Typical failure mode for the respective middle and side joints was shown. Besides, development of strains at different locations of the joints, as well as the strain rate effect on material strength was investigated. Comparison of design and test values of tying resistance, flexural resistance and rotation capacities was conducted. In addition, a comparison with quasi-static tests presented in Chapter 5 was also conducted. Based on the test results and analyses, the following conclusions can be drawn:

- (1) All the four composite WUF-B joints could sustain the applied impact loads with small peak and residual displacements. Catenary action was not fully mobilised while flexural action could fully develop. When subjected to the same impact load, the middle joint could develop greater flexural action compared to the side joint. A thicker composite slab contributed to greater flexural action.
- (2) Yielding of the beam flanges in tension was observed for all the composite WUF-B joints contributing to residual displacements. Yielding and buckling of the unrestrained beam flanges in compression were only observed in the side joint. The same crack patterns as composite FP joints were observed: longitudinal and diagonal cracks for the middle joint while longitudinal and transverse cracks for the side joint. Failure of steel WUF-B connection was not observed.
- (3) The respective maximum increases of concrete and steel strength were 28% and 17%, resulting from strain rates in the order of 1 s^{-1} .
- (4) Composite slab effect could benefit flexural action of both the middle and the side joints subjected to impact loads. Together with the strain rate effect,

much greater flexural resistances were achieved in comparison with the design values.

- (5) When eliminating the difference in material strengths, the strain rate effect could enhance flexural resistance of the composite WUF-B joints subjected to impact loads than those subjected to quasi-static loads. Such enhancement increased with an increase of velocity of the joints after the impact, achieved by reducing the joint stiffness (the side joint) and inertia (a thinner slab), and increasing impact velocity.

CHAPTER 8: NUMERICAL MODEL OF BEAM-COLUMN JOINTS

8.1 Introduction

In general, numerical models with solid elements are able to provide reasonably accurate predictions of joint behaviour under column removal scenarios in Chapter 3. However, this modelling technique is only applicable to beam-column joints or sub-assemblies. When it comes to structural analyses, a huge amount of computational time is needed. Besides, convergence problems may become critical. Therefore, simplified joint models are needed for structural analyses. In this chapter, a simplified joint model, namely, component-based model, is introduced for FP and WUF-B joints. Interactions between beam and column are simplified as nonlinear components in the models. Such models are fast-to-build and highly efficient in terms of computational time. Meanwhile, their accuracy can be maintained. Therefore, they are suitable for structural analyses at the system level.

8.2 Development of component-based models

A component-based model consists of a group of basic springs. Each spring has its own constitutive relationship in terms of force and corresponding displacement curve. As for FP joints, two types of springs are included, viz. contact spring between the column flange and the beam flange to simulate gaps, and single-bolt connection spring between the column and the beam web. Similar to previous studies (Koduru and Driver 2014, Main and Sadek 2014, Oosterhof and Driver 2016), single-bolt connection spring consists of a series of components, namely, bolts in bearing between the fin plate and the beam web, and bolts in shear and friction between these components. In the beam (horizontal) direction, spring elements should be built for individual bolt row while in the column (vertical) direction, one shear spring element can be used to consider contribution of all bolt rows for simplification purpose. To simulate bolted connections between the fin plate and the beam web in the WUF-B joint, the same bolt row springs as FP joints can be used. Two different springs are

employed in the WUF-B joint to simulate the welded beam flanges to the column flange. When a composite slab is incorporated, a component-based model can be applied to the composite joint as shown in Fig. 8.1(a) for middle FP joint (sagging moment) and Fig. 8.1(b) for side FP joint (hogging moment), respectively. Each component in the composite slab is represented by individual springs, including concrete, steel profiled sheeting and reinforcing bars as shown in Fig. 8.1. Similarly, when adding these slab springs to the model for bare steel WUF-B joint, component-based modelling can be applied to composite joints with WUF-B connections (Fig. 8.2(a) for middle joint and Fig. 8.2(b) for side joint, respectively). Mechanical properties of the aforementioned springs, in terms of force-versus-displacement curves are defined in the next section.

Fig. 8.1 Component-based models for FP connections: (a) Middle joint; (b) Side joint

Fig. 8.2 Component-based models for WUF-B connections: (a) Middle joint; (b) Side joint

Novelties of current approach are strain-rate effect and modification of flange element.

8.2.1 Concrete slab

Concrete properties can be obtained from either codified models or concrete material tests. For instance, concrete stress-strain relationship in uniaxial compression from the *fib* Model Code (fib 2013) is shown in Equation 8-1.

$$\sigma_c = -f_{cm} \left(\frac{k\eta - \eta^2}{1 + (k-2)\eta} \right) \quad \text{for } |\varepsilon_c| < |\varepsilon_{c,lim}| \quad (8-1)$$

where $\eta = \frac{\varepsilon_c}{\varepsilon_{c1}}$; $k = \frac{E_c}{E_{c1}}$; σ_c is the compressive stress; ε_c is the compressive strain; ε_{c1} is the strain at maximum compressive stress; $\varepsilon_{c,lim}$ is the ultimate compressive strain; E_{ci} is the secant modulus of concrete; E_{c1} is the secant modulus from the origin to the peak compressive stress; f_{cm} is the mean value of compressive strength. It should be noted that the values of all the parameters can be obtained from Table 5.1-8 in the *fib* Model Code (fib 2013). The ultimate compressive strain of C30 concrete is 0.0035 accordingly. However, with the development of micro-cracks, softening of concrete can go up to a strain of 0.02 based on Eurocode 2 Part 1-2 (BSI 2004b). A linear degradation behaviour can be assumed between compressive strain values of 0.0035 and 0.02.

Based on the *fib* Model Code, concrete stress-strain relationship in uniaxial tension is shown in Equation 8-2. It should be noted that the contribution of concrete tension force may be negligible in most cases.

$$\sigma_{ct} = E_{ci}\varepsilon_{ct} \quad \text{for } \sigma_{ct} < 0.9f_{ctm} \quad (8-2)$$

$$\sigma_{ct} = f_{ctm} \left(1 - 0.1 \frac{0.00015 - \varepsilon_{ct}}{0.00015 - \frac{0.9f_{ctm}}{E_{ci}}} \right) E_{ci} \quad \text{for } 0.9f_{ctm} < \sigma_{ct} < f_{ctm} \quad (8-3)$$

where σ_{ct} is the tensile stress; ε_{ct} is the tensile strain; f_{ctm} is the mean value of tensile strength.

In the experimental tests, failure of concrete was observed in a region at a distance roughly equal to the beam depth (h_b) from the column flange. Therefore, gauge length (h_g) of the concrete spring is set as the beam depth plus half the column depth, which is calculated from the column centre line. The peak compression force (F_{cm}) of concrete spring is equal to the tension force provided by the steel components including the beam flange, bolt rows, and profiled sheeting. Therefore, for each connection type, individual concrete spring property has to be defined. It should be noted that F_{cm} should not exceed the maximum compressive resistance of the concrete slab, equal to area of concrete (A_c) multiply by compressive strength (f_{cm}). A schematic representation of concrete spring property is shown in Fig. 8.3.

Fig. 8.3 Schematic representation of concrete property

8.2.2 Reinforcing bar

Under compression, crushing and spalling of concrete surface can accelerate buckling of reinforcing bars, which was observed in the test. Therefore, compressive strength of reinforcing bars is negligible. A bilinear curve of a tensile spring representing the reinforcing bars is shown in Fig. 8.4, based on the yield strength σ_y , ultimate strength σ_u , elastic modulus E and nominal area of the bars. Only continuous reinforcing bars are considered. Gauge length (h_g) of reinforcing bar spring is the same as that of the concrete spring. It should be noted that compressive strength may be significant if the concrete provides sufficient restrained.

Fig. 8.4 Schematic representation of reinforcing bar property

8.2.3 Profiled sheeting

Since the thickness of the steel profiled sheeting is 1 mm, local buckling can substantially weaken its compressive resistance. Therefore, profiled sheeting in compression is negligible. Profiled sheeting in tension can be simplified as a bilinear curve (Yang et al. 2015) as shown in Fig. 8.5 based on coupon tests. The effective width of the profiled sheeting should be kept the same as that of the joint as shown in Fig. 8.6. The column width should be deducted from the total width based on the geometry. It should be noted that when there is overlapping of profiled sheeting at the joint in construction, contribution of the sheeting component is not considered.

Fig. 8.5 Schematic representation of profiled sheeting property

Fig. 8.6 Top view of joint dimension

8.2.4 Beam flange

For fin plate connections, gaps exist between the beam flange and the column flange.

The stiffness and resistance of the beam flange and the column flange in compression are much greater than those of a bolt row. Therefore, it is assumed that the stiffness and resistance of the beam flange and the column flange are infinite when the gap between them closes up.

The gap distance ranges from 10 to 25 mm according to different design guidelines and the depth of connected beams (AISC 2010, BSCA/SCI 2011). With a wider gap, contact between the beam flange and the column flange will not be mobilised at catenary action stage. But a narrower gap will trigger shifting of rotation centre, as shown in Fig. 8.7.

Fig. 8.7 Shifting of centre of rotation adopted (Taib (2012))

For WUF-B connections, top and bottom beam flanges are welded to the column flange. Beam flange spring can be simplified as a simply-supported column element. For the side joints under hogging moment, local buckling of the bottom beam flange may occur and the length of the buckled flange is assumed to be equal to the beam depth. However, for the middle joints, composite slab can prevent local buckling of the top beam flange. When calculating buckling resistance F_b of the column element based on Eurocode 3 (2005a), T-shaped cross-section extracted from the I-shaped beam is used as shown in Fig. 8.8. The column element in tension has the same rectangular cross-section as the beam flange. Schematic property of beam flange spring is shown in Fig. 8.9.

Fig. 8.8 Beam flange element of WUF-B connection

Fig. 8.9 Schematic representation of beam flange property

8.2.5 Bolted connection

When the fin plate connections are subjected to pure bending moment, each bolt row is under compression or tension depending on its vertical position from the centre of rotation. Similar results can be obtained when connections are subjected to combined axial tension force and bending moment at catenary action stage. Therefore, the behaviour of bolt rows in compression and tension has to be considered.

a) Bolts in bearing between fin plate and beam web

Several methods have been proposed to predict the ultimate strength $R_{n,br}$ of bolts in bearing in steel plates and included in national design codes such as Eurocode 3 Part 1-8 (2005b), AISC 360-10 (2010) and CSA S16-09 (2009). The equation in Eurocode 3 Part 1-8 (2005b) provided a more conservative strength prediction compared to the AISC and CSA codes. Therefore, for more accurate predictions of the joint behaviour, the equation in AISC 360-10 is adopted as follows:

$$R_{n,br} = 1.5(L_e - \frac{d_b}{2})t\sigma_u \leq 3td_b\sigma_u \quad (8-4)$$

where L_e is the end distance from the centre of a bolt hole to the edge of the fin plate measured in the direction of load transfer (horizontal direction), d_b is the nominal diameter of the bolt, t is the thickness of the plate, and σ_u is the ultimate strength of the steel plate.

Plate section may fail in block tearing mode prior to bolt bearing failure when the end distance is not adequate (Može and Beg 2014). In this instance, bolt in bearing resistance in Equation 8-4 cannot be achieved and block tearing resistance is used instead. Eurocode 3 Part 1-8 (2005b) provides block tearing resistance as follows:

$$R_{n,bs} = \sigma_u A_{nt} + (1/\sqrt{3})\sigma_y A_{nv} \quad (8-5)$$

where A_{nt} is net area subjected to tension and A_{nv} is net area subjected to shear.

The stiffness of bolt in bearing k_i is determined from Equation 8-6 proposed by Rex and Easterling (1996):

$$k_i = \frac{1}{\frac{1}{k_{br}} + \frac{1}{k_b} + \frac{1}{k_v}} \quad (8-6)$$

where k_{br} , k_b and k_v are the stiffness values of bolt bearing, edge steel plate bending and shearing, respectively. They are specified by Equations 8-7 to 8-9.

$$k_{br} = 120t\sigma_y d_b^{(4/5)} \quad (8-7)$$

$$k_b = 32Et\left(\frac{L_e}{d_b} - \frac{1}{2}\right)^3 \quad (8-8)$$

$$k_v = (20/3)Gt\left(\frac{L_e}{d_b} - \frac{1}{2}\right) \quad (8-9)$$

where σ_y is the yield strength, E and G are the respective moduli of elasticity and shear of the steel plate.

Rex and Easterling (2003) proposed force-displacement relationship of bolts in bearing based on experimental tests. The relationship is capable of predicting the behaviour of steel joints with reasonable accuracy (Taib 2012, Koduru and Driver 2014, Weigand 2014, Oosterhof and Driver 2016). Therefore, the method is used to represent the constitutive relationship for bolts in bearing, as expressed in Equation 8-10.

$$F_{br} = R_{n,br} \left[\frac{1.74\bar{\Delta}}{(1 + \bar{\Delta}^{0.5})^2} - 0.009\bar{\Delta} \right] \quad (8-10)$$

where F_{br} is the resultant force, $\bar{\Delta} = \frac{\Delta k_i}{R_{n,br}}$ is the normalised displacement, Δ is the displacement.

The main difference between the bolt rows in compression and tension arises from the bearing resistance at the bolt holes. In compression, the resistance of bolts in bearing can be calculated from Equation 8-11.

$$R_{n,br} = 3td_b\sigma_u \quad (8-11)$$

The stiffness of bolt rows in compression can be determined by Equation 8-7.

b) Bolts in shear

When shear failure of bolt shank governs failure mode of bolted connections, properties of bolts in shear should be used. A generalised force-versus-displacement curve for bolts in single shear suggested by Oosterhof and Driver (2016) is used in

the current study as shown in Fig. 8.10.

The ultimate strength of bolts in single shear is determined by Equation 8-12.

$$R_{nv,bolt} = 0.6 \frac{\pi d_b^2}{4} \sigma_{ub} \quad (8-12)$$

where σ_{ub} is the ultimate strength of the bolt.

This equation has been included in design codes such as Eurocode 3 Part 1-8 (2005b), AISC 360-10 (2010) and CSA S16-09 (2009). According to the test results of bolts in single shear (Moore 2007), a coefficient of 1.25 can be used to convert the nominal strength of steel to its ultimate strength. Besides, the predicted shear resistance is reduced by a factor of 0.7, if shear plane goes through bolt threads.

Fig. 8.10 Force-versus-displacement for bolts in single shear (Oosterhof and Driver (2016))

c) Influence of oversized bolt hole and friction

Typically, an oversized bolt hole is used in construction practice to facilitate the installation of bolts. In Europe, the diameter of bolt holes is generally 2 mm larger than that of bolts, while the value is 1.6 mm in North America. If a slotted hole is used, this value can vary with different design codes. It can be predicted that the bolt shank moves towards the gap before contacting the steel plate, as shown in Fig. 8.11. Movement of the bolt shank may vary from 0 to twice the gap distance. In simulations, it can be assumed that the axis of bolt shank is concentric with the centroid of plate holes for simplification.

Fig. 8.11 Direction of bolt movement: (a) Oversized hole; (b) Slotted hole (Taib (2012))

During the movement of bolt shank, only friction force exists and its magnitude depends on the surface treatment of the plate and the bolt type. An estimated constant value of 30 kN (before the gap closes) is suggested for non-preloaded bolts by Oosterhoof (2016) when snug-tight installation is used. For preloaded bolts, the value has to be determined according to relevant design codes. Friction force $F_{s,Rd}$ given by Eurocode 3 Part 1-8 (2005b) is expressed as follows:

$$F_{s,Rd} = 0.7k_s n \mu \sigma_{ub} A_s \quad (8-13)$$

where k_s is a coefficient to account for the effect of the type of bolt holes, n is the number of friction surface, μ is the coefficient of slip, A_s is the stressed area of bolt, usually taken as 75% of bolt gross area calculated using the nominal diameter.

Fig. 8.12 shows the friction force-slip curve given by Frank and Yura (1981). A constant value equal to $F_{s,Rd}$ can be assumed as a threshold force before the bolt starts to sustain bearing stress.

Fig. 8.12 Typical force-displacement curve (Frank and Yura (1981))

Although the influence of oversized holes is counteracted by frictional force before the start of bolt-in-bearing behaviour, it has a great impact on the ultimate deformation of bolt-in-bearing behaviour (Koduru and Driver 2014). This will increase the rotational ductility of the fin plate connection if no significant shear force exists and eliminates the gap in service stage.

d) Load reversal

Fig. 8.13 Load reversal of bolt row

Under column removal scenarios, the outermost bolt row experiences load reversal. At the initial stage, it is in compression as a result of bending moment. With an increase of axial tension force in the beam, tension resistance of the bolt row can be mobilised. Therefore, unloading and reloading paths have to be defined in the component-based model. It is assumed that the unloading path follows the initial stiffness under tension and compression. When the force reduces to zero, the bolt row moves freely in the opposite direction. Fig. 8.13 shows a schematic load reversal path of bolt rows in tension and compression.

e) Failure criteria

Failure of a bolt row is governed by its weakest component. Test results on fin plate connections subjected to catenary action (Yang 2013, Weigand 2014) indicate two possible failure modes, namely, shear failure of bolts and tear-out failure of steel

plates, depending on the relative resistance between the bolts and the steel plates.

In component-based models, deformation capacity of each bolt row is defined in tension and compression respectively. Oosterhoof (2016) provided the ultimate deformations of bolt rows in tension. The value is about 0.8 to 1.0 time of end distance. Since there are not sufficient test data on the ultimate deformations of bolt rows, it is recommended that 70% of end distance can be used. For bolt rows in compression, shear failure of bolts is dominant over tear-out failure of fin plates, and the ultimate deformation is around 0.23 times of bolt diameter.

8.2.6 Vertical shear

Vertical shear failure is not critical for joints subjected to column removal scenarios. An elastic spring can be used to model behaviour of joints subjected to shear force. In the vertical direction, bolts function in parallel. Therefore, stiffness of the elastic spring can be assumed to be the stiffness of bolts in bearing (k_i) multiplied by the number of bolts.

8.2.7 Strain rate effect

Material properties of steel and concrete can be affected by high strain rate, which leads to different behaviour of basic components under impact load. To consider the strain rate effect of steel and concrete materials, dynamic increase factors (*DIFs*) are used based on previous research studies (Abramowicz and Jones 1984, fib 2013).

For concrete material, the following *DIFs* at strain rate $\dot{\epsilon}_c$ from the *fib* Model Code (fib 2013) can be adopted.

Compressive strength:

$$DIF = (\dot{\epsilon}_c / \dot{\epsilon}_{c0})^{0.014} \quad \text{for } \dot{\epsilon}_c \leq 30s^{-1} \quad (8-14)$$

$$DIF = 0.012(\dot{\epsilon}_c / \dot{\epsilon}_{c0})^{1/3} \quad \text{for } \dot{\epsilon}_c > 30s^{-1} \quad (8-15)$$

$$\text{with } \dot{\epsilon}_{c0} = 30 \cdot 10^{-6}s^{-1}$$

Tensile strength:

$$DIF = (\dot{\epsilon}_{ct}/\dot{\epsilon}_{ct0})^{0.018} \quad \text{for } \dot{\epsilon}_{ct} \leq 10s^{-1} \quad (8-16)$$

$$DIF = 0.0062(\dot{\epsilon}_{ct}/\dot{\epsilon}_{ct0})^{1/3} \quad \text{for } \dot{\epsilon}_{ct} > 10s^{-1} \quad (8-17)$$

$$\text{with } \dot{\epsilon}_{ct0} = 1 \cdot 10^{-6}s^{-1}$$

Modulus of elasticity:

$$DIF = (\dot{\epsilon}_c/\dot{\epsilon}_{c0})^{0.026} \quad (8-18)$$

Strain at maximum stress:

$$DIF = (\dot{\epsilon}_c/\dot{\epsilon}_{c0})^{0.02} \quad (8-19)$$

For steel material, the Cowper and Symonds model is employed as follows:

$$DIF = 1 + \left(\frac{\dot{\epsilon}}{C}\right)^{\frac{1}{p}} \quad (8-20)$$

where 6844 and 3.91 are adopted for constants C and p (Abramowicz and Jones 1984); $\dot{\epsilon}$ is the strain rate.

Under impact loading scenario, the respective yield and ultimate strengths (σ_y and σ_u) of steel in Equations 8-4 to 8-13 should be modified by DIF obtained from Equation 8-20.

The relationship between strain rate $\dot{\epsilon}$ and displacement of each component Δ can be obtained from Equations 8-21 to 8-23, which are modified from the method by Stoddart et al. (2013, 2014).

$$\dot{\epsilon} = \frac{\epsilon}{\delta_t} \quad (8-21)$$

$$\delta_t = \frac{\Delta}{v} \quad (8-22)$$

$$\dot{\epsilon} = \frac{\epsilon}{\Delta} v \quad (8-23)$$

where v is the velocity and δ_t is the time to reach displacement Δ .

The displacement Δ and strain ε correspond to steel stress σ at the respective yield or ultimate strength when applying Equation 8-23.

More specifically, for components with uniform cross-section such as concrete slab, reinforcing bar, profiled sheeting and beam flange:

$$\varepsilon = \frac{\Delta}{h_g} \quad (8-24)$$

$$\dot{\varepsilon} = \frac{1}{h_g} v \quad (8-25)$$

where h_g is the gauge length of the component.

8.3 Model validation

Fig. 8.14 Component-based model of composite beam-column joint

To validate the modelling approach in section 2, finite element (FE) package ABAQUS was chosen and the springs were simulated by *CONNECTOR* elements (Dassault Systèmes 2011). After determination of the spring properties, nonlinear springs were assembled in the beam-column joint. Fig. 8.14 shows a typical composite beam-column joint model. In the component-based model, beam element was used to simulate linear members including the column, beam, shear stud, circular hollow section (CHS) and bracket support. Shell element was used to simulate two dimensional members including the concrete slab and steel profiled sheeting. Rigid elements were used to connect the springs. Due to symmetry, only one-half of the

joint was modelled. Boundary conditions and loads were applied based on the test procedure.

8.3.1 Joints subjected to quasi-static loads

Test results from Oosterhof and Driver (2015) were chosen for validation purpose. Based on the tests, eight specimens were simulated by component-based models using ABAQUS/Standard solver and they were loaded by displacement-control under column removal scenario. In the component-based models, two types of bolt row springs were used and the mechanical properties are shown in Fig. 8.15. Fig. 8.15(a) shows the property of the bolt row with 22 mm diameter bolt and 9.5 mm thick fin plate (type A) while Fig. 8.15(b) is for the bolt row with 19 mm diameter bolt and 6.4 mm thick fin plate (type B). Failure criteria of the springs were determined by the average deformation capacity of bolt rows, as listed in Table 8.1.

Fig. 8.15 Mechanical properties for bolt row in fin plate joints (Oosterhof and Driver (2015)): (a) Type A (22 mm diameter bolt and 9.5 mm thick fin plate); (b) Type B (19 mm diameter bolt and 6.4 mm thick fin plate)

Table 8.1 Failure criteria applied to component-based models

ID	Top bolt row (mm)	Second bolt row (mm)	Middle bolt row (mm)	Fourth bolt row (mm)	Bottom bot row (mm)
ST3A-1	35	35	35	35	35
ST3A-3	35	35	35	35	35
ST5A-1	32	32	32	32	32
ST5A-2	32	32	32	32	32
ST3B-1	27	27	27	27	27
ST3B-2	27	27	27	27	27
ST5B-1	27	27	27	27	27
ST5B-2	27	27	27	27	27

Fig. 8.16(a) shows a comparison of load-versus-rotation curves from component-based joint model and the test result for specimen ST3A-1 which had three bolt rows with type A property (22 mm diameter bolt and 9.5 mm thick fin plate in Fig. 8.15(a)). It can be seen that both the initial ascending and post peak descending curves (induced by sequential failure of the fin plate from the bottom bolt upwards) could be captured by simulations, indicating that the component-based model is capable of predicting the overall load-displacement responses with reasonably good accuracy. Component-based models also yielded good prediction for other type A specimens, including specimens ST3A-3 (longer beam span) in Fig. 8.16(b), ST5A-1 (one line of five bolt rows) in Fig. 8.16(c) and ST5A-2 (one line of five bolt rows and longer span) in Fig. 8.16(d). In addition to accuracy, computational time was substantially reduced compared to numerical models consisting of three dimensional solid elements.

Fig. 8.17(a) shows a comparison of load-versus-rotation curves from the component-based joint model and the test result for specimen ST3B-1. Type B property (19 mm diameter bolt and 6.4 mm thick fin plate in Fig. 8.15(b)) and one line of three bolt rows were used in ST3B-1. Comparisons of the other three specimens using type B properties are also included: specimens ST3B-2 (longer beam span) in Fig. 8.17(b), ST5B-1 (one line of five bolt rows) in Fig. 8.17(c) and ST5B-2 (one line of five bolt rows and longer span) in Fig. 8.17(d). Model predictions agree well with those from the tests, which means that mechanical properties in Fig. 8.15(b) and failure criteria in Table 8.1 are validated.

Fig. 8.16 Comparison of horizontal-load-versus-beam-rotation curves from component-based models and test results by Oosterhof and Driver (2015) (22 mm diameter bolt and 9.5 mm fin plate):

(a) ST3A-1; (b) ST3A-3; (c) ST5A-1; (d) ST5A-2

Fig. 8.17 Comparison of horizontal-load-versus-beam-rotation curves from component-based models and test results by Oosterhof and Driver (2015) (19 mm diameter bolt and 6.4 mm thick fin plate): (a) ST3B-1; (b) ST3B-2; (c) ST5B-1; (d) ST5B-2

From Chapters 3, 4 and 5, beam-column joint tests on a middle column removal scenario were conducted and quasi-static loads were applied in the tests. Using ABAQUS/Standard solver, six typical joint models were loaded by displacement control for comparison purpose, including two bare steel joints (FP-static and W-static) and four composite joints (C75FP-M, C75FP-S, C75W-M and C75W-S). Based on the modelling approach introduced in section 2.2, mechanical properties of each spring were obtained as shown in Fig. 8.18. Parabolic ascending curve and linear degradation were used for the concrete spring in Fig. 8.18(a). Bilinear curves were used for steel profiled sheeting (Fig. 8.18(b)) and reinforcing bars (Fig. 8.18(c)). Moreover, trilinear curves were used for beam flange in tension and compression as shown in Fig. 8.18(d). For bolt rows connecting the fin plate and the beam web, a parabolic curve was used for tensile behaviour while a trilinear curve was adopted for compression behaviour as shown in Fig. 8.18(e). To achieve good agreement between simulations and test results, different failure criteria were defined for steel springs in each specimen as shown in Table 8.2.

Fig. 8.18 Mechanical properties for each spring: (a) Concrete slab; (b) Profiled sheeting; (c) Reinforcing bar; (d) Beam flange; (e) Bolt row

Table 8.2 Failure criteria applied to component-based models

ID	Top bolt row (mm)	Middle bolt row (mm)	Bottom bot row (mm)	Restrained flange (mm)	Unrestrained flange (mm)
FP-static	21	21	25	/	/
W-static	10	10	10	18	10
C7FP-M	15	15	15	/	/
C75FP-S	4	12	19	/	/
C75W-M	10	10	10	10	17
C75W-S	10	10	10	8	11

Fig. 8.19(a) shows a comparison of load-versus-displacement curves from component-based model and experimental tests for specimen FP-static. Load applied to FP-static could be captured well by the simulation when applying mechanical properties in Fig. 8.18(e) and failure criteria in Table 8.2 for the three bolt rows. When incorporating mechanical properties (Fig. 8.18(d)) and failure criteria (Table 8.20) of beam flanges, load applied to W-static could also be captured well by simulation, except that the load obtained from simulation was slightly greater than that from experimental tests after the bottom beam flange fractured at around 150 mm. The reason is probably that only one-half of the specimen was modelled for simplification purpose and failure was assumed to take place at both sides simultaneously. However, in the test bottom beam flange fractured at only one side. Composite joints C75FP-M and C75FP-S are shown in Figs. 8.19(c) and (d), respectively. Although the absolute values of applied load from models and test results have slight differences, each failure including fractures of fin plate, profiled sheeting and reinforcing bars could be captured well by the simulations. Similarly, applied loads as well as failure of beam flanges of composite joints C75W-M (Fig. 8.19(e)) and C75W-S (Fig. 8.19(f)) could be captured by the simulations.

Beam axial forces from component-based models and experimental tests are compared in Fig. 8.20 and they represent the development of compressive arch action (CAA) and catenary action (CA). For bare steel joints FP-static (Fig. 8.20(a)) and WUF-B (Fig. 8.20(b)), component-based models could simulate the tension force (positive value) at large deformation stage, while the compression force (negative value) was negligible at small deformation stage. Similar to bare steel joints, component-based models could function well when simulating beam axial forces of composite joints C75FP-M (Fig. 8.20(c)), C75FP-S (Fig. 8.20(d)), C75W-M (Fig. 8.20(e)) and C75W-S (Fig. 8.20(f)), except that CAA (indicated by compression force) of C75FP-M was underestimated by the simulation while CA (indicated by tension force) was overestimated. The difference in compression force may be attributed to underestimation of compressive behaviour of bolt row spring, while overestimation of failure criterion may contribute to a difference in the tension force. Weaker boundary condition in the simulation may contribute to an underestimation

of compression force in Fig. 8.20(c).

A comparison of bending moment at the joints from component-based models and experimental test is shown in Fig. 8.21. Bending moment indicates the development of flexural action (FA). Although bending moment for simple joint FP-static is negligible in Fig. 8.21(a), it can be captured well by component-based model. Development of bending moment could also be simulated by component-based model for W-static in Fig. 8.21(b), although the absolute value of simulation was smaller. Good agreement between simulations and test results could be achieved for composite joints C75FP-M ((Fig. 8.20(b)), C75FP-S (Fig. 8.20(c)), C75W-M (Fig. 8.20(d)) and C75W-S (Fig. 8.20(e)). It should be noted that due to omission of concrete tensile strength, component-based model for side joint C75FP-S (Fig. 8.20(c) underestimated the tension force at small deformation stage (before fracture of profiled sheeting).

Fig. 8.19 Comparison of load-versus-displacement curves from component-based models and test results: (a) FP-static; (b) W-static; (c) C75FP-M; (d) C75FP-S; (e) C75W-M; (f) C75W-S

Fig. 8.20 Comparison of beam axial force-versus-displacement curves from component-based models and test results: (a) FP-static; (b) W-static; (c) C75FP-M; (d) C75FP-S; (e) C75W-M; (f) C75W-S

Fig. 8.21 Comparison of bending moment-versus-displacement curves from component-based models and test results: (a) FP-static; (b) W-static; (c) C75FP-M; (d) C75FP-S; (e) C75W-M; (f) C75W-S

8.3.2 Joints subjected to impact loads

Beam-column joint tests under impact loading scenarios were presented in Chapters 6 and 7. Seven typical beam-column joints were simulated by component-based models using ABAQUS/Implicit solver. Loads measured in the tests were applied to the models and structural responses were compared with model predictions. Strain rate effect was considered based on Section 2.2.7. Fig. 8.22(a) shows the mechanical properties that were used for the spring representing profiled sheeting. Compared to a single curve used in the quasi-static simulations (Fig. 8.18(b)), multiple curves

were used in impact simulations. Each curve represented one velocity of spring movement. Similarly, mechanical properties of reinforcing bars (Fig. 8.22(b)), beam flange (Fig. 8.22(c)) and bolt row (Fig. 8.22(d)) also consisted of multiple curves. For clarity, only the ascending part of bolt spring is shown in Fig. 8.22(d). Failure criteria for bolt rows and beam flanges are shown in Table 8.3. It should be noted that the typical velocities for each spring might fall in a range from 100 mm/s to 1000 mm/s for low speed impact test.

Fig. 8.22 Mechanical properties for components: (a) Profiled sheeting; (b) Reinforcing bar; (c) Beam flange; (d) Bolt row

Table 8.3 Failure criteria applied to component-based models

ID	Top bolt row (mm)	Middle bolt row (mm)	Bottom bot row (mm)	Restrained flange (mm)	Unrestrained flange (mm)
FP6-M530H3	10	8	12	/	/
FP10-M530H3	21	21	25	/	/
W-M830H3	5	8	12	18	10
C75FP-M530H3	4	5	8	/	/
C75FP-M530H3-S	6	8	12	/	/
C75W-M770H3	6	8	12	18	10
C75W-M770H3-S	6	8	12	10	18

Fig. 8.23(a) compares predicted displacements from the component-based model and experimental test for bare steel joint FP6-M530H3. Good agreement with test result was achieved by simulations. In comparison, displacements from component-based models for bare steel joints FP10-M530H3 (Fig. 8.23(a)) and W-M830H3 (Fig. 8.23(b)) were slightly greater than those from test results, indicating either the applied loads were greater or stiffness of the models was smaller. The differences were small so that the component-based models were acceptable. For composite joints C75FP-M530H3 (Fig. 8.23(c)) and C75FP-M530H3-S (Fig. 8.23(d)), displacements from component-based models were only slightly smaller than those from test results. Composite joint C75W-M770H3 had welded connection so that it was strong enough to withstand the impact load. After attaining the peak displacement, C75W-M770H3 recovered partially as shown in Fig. 8.23(f). Residual displacement in Fig. 8.23(f), an important indicator for structural behaviour, represents the impact energy absorption through plastic deformation of the joint. To capture the residual displacement, two simulations were conducted and are compared with the test result in Fig. 8.23(f). One of them used elastoplastic concrete properties for the shell elements of concrete slab (Fig. 8.14), while the other employed damage plasticity model for concrete material. It is clear that the latter had better agreement with test results although the peak displacement was slightly smaller. The reason was probably that the slab absorbed more energy through local deformation while the steel beam-column connection underneath absorbed less energy, leading to a smaller residual displacement. A similar procedure was applied to C75W-M770H3-S. Compared to the model using elastoplastic concrete material, the one using damage plasticity model gave better agreement with test results as shown in Fig. 8.23(g). The

peak displacements of both models were smaller than that from the test result, probably due to stronger boundary condition. In the test, the A-frames and connected pinned supports acted as an elastic spring in the horizontal direction but due to gaps, the restraint force may be overestimated by the simulation.

Fig. 8.24(a) shows a comparison of beam axial force from component-based model and experimental test for bare steel joint FP6-M530H3. It is clear that the model could capture both the ascending curve and the post peak curve (after fracture of fin plate initiated). For FP10-M530H3, since displacement from the model was greater as shown in Fig. 8.23(b), beam axial force curve from the model was also mobilised earlier (Fig. 8.24(b)). However, the peak axial force can be captured by the simulation. A similar phenomenon is observed when comparing the beam axial force for W-M830H3 in Fig. 8.24(c): the peak value of beam axial force from component-based model was greater due to greater displacement (Fig. 8.23(c)). For composite joints C75FP-M530H3 (Fig. 8.24(d)) and C75FP-M530H3-S (Fig. 8.24(e)), displacements from simulations were smaller so that the beam axial force was mobilised later than that from the test result. Due to small displacement, beam axial force was not fully developed for C75W-M770H3 (Fig. 8.24(f)) and C75W-M770H3-S (Fig. 8.24(g)).

Figs. 8.25(a) and (b) show a comparison of bending moments at the joint from component-based models and experimental tests for bare steel joints FP6-M530H3 and FP10-M530H3, respectively. As simple pinned joints, these two models were not able to resist bending moment. Bending moments observed in the test results were due to vibration of beams after the impact, which could not be captured by the models. For welded joint W-M830H3, the model could well capture the bending moment as shown in Fig. 8.25(b). Good agreement with test results was achieved by component-based models for all the four composite joints: C75FP-M530H3 (Fig. 8.25(c)) and C75FP-M530H3-S (Fig. 8.25(d)), C75W-M770H3 (Fig. 8.25(c)) and C75W-M770H3-S (Fig. 8.25(d)).

Fig. 8.23 Comparison of displacement-versus-time curves from component-based models and test results: (a) FP6-M530H3; (b) FP10-M530H3; (c) W-M830H3; (d) C75FP-M530H3; (e) C75FP-

M530H3-S; (f) C75W-M770H3; (g) C75W-M770H3-S

Fig. 8.24 Comparison of beam axial force-versus-time curves from component-based models and test results: (a) FP6-M530H3; (b) FP10-M530H3; (c) W-M830H3; (d) C75FP-M530H3; (e) C75FP-M530H3-S; (f) C75W-M770H3; (g) C75W-M770H3-S

Fig. 8.25 Comparison of bending moment-versus-time curves from component-based models and test results: (a) FP6-M530H3; (b) FP10-M530H3; (c) W-M830H3; (d) C75FP-M530H3; (e) C75FP-M530H3-S; (f) C75W-M770H3; (g) C75W-M770H3-S

8.4 Assumptions and limitations

Vertical shear behaviour in the proposed component-based modelling approach is

simplified by elastic springs. Block shear and tensile fracture failure of fin plates or beam webs are considered by adjusting failure criteria of spring displacement (Tables 8.2 and 8.3), rather than incorporating failure of shear spring explicitly. These assumptions are reasonable for column removal scenarios where vertical shear is not critical to initiating damage to the beam-column joint. Considerations should be carefully taken when extending the proposed approach to other loading scenarios.

Material or structural damping properties have not been included in the current modelling approach although in certain cases (Figs. 8.23(e) and (g)) vibrations of beam-column joint models are observed. Such simplification is reasonable since vibration is not observed in most validations.

Plastic deformation and damage are assumed to be concentrated at the joint and represented by mechanical properties of each spring. It should be noted that composite joints C75W-M770H3 and C75W-M770H3-S are two exceptions since more accurate simulations are achieved through applying concrete damage plasticity to shell elements for slabs.

Load reversal is introduced in the current approach. However, under column removal scenarios, only one load reversal is observed in beam flanges and bolt rows. Special considerations must be taken when energy dissipation under cyclic loads is critical.

The proposed model is validated by low speed impact tests, where the speed of each spring is limited to 1000 mm/s and the strain rate of each material is limited to 10 s^{-1} . Although a higher speed can be incorporated in the proposed approach, more validations are necessary when it is applied to a higher loading speed such as pneumatic shocks or even explosions. It should be noted that during validation, the applied load is measured from the experimental test. Explicit modelling of impact surfaces between drop-weight hammer and middle column stub was not used because the objective of the current study is to validate beam-column joint models rather than contact forces.

8.5 Summary and conclusion

A component-based modelling approach has been proposed for steel and composite

beam-column joints in this study. In the proposed component-based models, beam-column joints are discretised into individual springs, including the concrete slab, reinforcing bar, profiled sheeting, beam flange and bolted connection. Mechanical property of each spring is determined by material and geometry of individual component. Failure criteria are determined accordingly. Strain rate effect is considered through transforming strain rate to velocity of movement of each spring when applying the models to impact loading scenarios. The models are validated against fourteen quasi-static joint tests and thirteen impact joint tests. It is found that the proposed models could capture structural behaviour of joints under both scenarios, including load, axial force and bending moment for quasi-static loads, as well as displacement, axial force and bending moment for impact loads. Furthermore, to obtain better agreement of residual displacements with test results, concrete damage plasticity was applied in two of the ABAQUS models under impact loads. It can be concluded that the proposed modelling approach performs well for steel and composite beam-column joints under both quasi-static and impact loads.

CHAPTER 9: CONCLUSIONS AND FUTURE WORK

9.1 Conclusions

The current research study focuses on the behaviour of steel and composite beam-column joints under abnormal loading conditions. In this research programme, two types of commonly-used connections were investigated subjected to both quasi-static and impact loads and a notional middle column scenario was applied to represent the initial damage that probably leads to progressive collapse of building structures. Structural response, load-resisting mechanism, failure mode, energy absorption and development of strain were investigated.

A comparison of test results and design resistances was conducted to further understand the behaviour of steel and composite beam-column joints under the applied loads. In general, the current design calculation method was found to overestimate the tying resistance of both types of composite joints, especially when thicker slabs or fewer shear studs were used. The overestimation is less evident for WUF-B joints compared to FP joints. The novel FP joint was able to develop the design value of tying resistance in the test.

Moreover, comparison of beam-column joints subjected to quasi-static and impact loads was conducted to investigate the dynamic effect. Finally, a new component-based modelling approach considering both composite slab and strain-rate effects was proposed and validated against test results.

More details are provided in the following sections.

Beam-column joints with FP connections subjected to quasi-static loads

Resistance of the simple joint was provided by flexural action combined with compressive arch action or catenary action depending on the joint deflections. At small deformation stage, compressive arch action was dominant while catenary action was dominant at large deformation stage. Compared to the bare steel joint, the composite joint had greater flexural action. However, increased slab thickness and reduced number of shear studs were detrimental to mobilisation of catenary action.

Middle joint absorbed more energy than side joint at the small deformation stage due to greater flexural action. However, similar energy was absorbed by both middle and side joints finally because the side joint could mobilise much greater catenary action. Although tying resistance of the composite joints was reduced due to combined bending moment, tie force requirement (75 kN) from Eurocode 1 could be met for most of the composite FP joints. However, with a thicker concrete slab (100 mm) or a reduced number of shear studs, tie force requirement could not be met. In comparison to conventional connections, FP connection with slotted bolt holes had better performance than conventional connection in terms of energy absorption and tying resistance

Beam-column joints with WUF-B connections subjected to quasi-static loads

Similar to joints with FP connections, applied load was sustained by flexural, compressive arch and catenary actions for composite joints with WUF-B connections under column removal scenario. Before beam flanges first fractured, applied load was sustained by flexural action. After that, it was sustained by catenary action at large deformation stage. Failure of concrete initiated nonlinear load-resisting mechanism of composite joints with WUF-B connections. With an increase of slab thickness, energy absorption of middle joint was reduced at large deformation stage. Furthermore, design flexural resistance and rotation capacity of composite joints with WUF-B could be achieved while design tying resistance could not be achieved since all the composite joints could not meet the 0.2 rad criterion based on UFC 4-023-03. However, tie force requirement of Eurocode 1 Part 1-7 (without any specification of rotation capacity) could be met.

In contrast to joints with FP connections, the contribution of compressive arch action was negligible compared to that of flexural action. Composite joints with WUF-B connection had better flexural and tying resistances than those with FP connection. However, the latter had better rotation performance. Failure mode was characterised by sequential fracture of beam flanges. Middle joints had greater energy absorption capacity than side joints.

More importantly, compared to WUF-B joints, RBS joint could resist greater load

and had better rotation capacity when failure criterion was based on first fracture of beam flange. Furthermore, energy absorption of RBS joint was greater than WUF-B joint.

Beam-column joints with FP connections subjected to impact loads

With the same impact momentum, greater impact velocity contributed to a greater impact force. When subjected to the same impact load, the middle and the side joints had a similar impact force during the first collision. For specimens with a thicker slab, a greater impact force was observed due to an increase of mass and inertia. Besides, an intermediate level of strain rate in the order of 1 s^{-1} was observed, leading to respective maximum increase of 28% in concrete strength and 16% in steel strength.

Similar to joints subjected to quasi-static loads, compressive arch action and catenary action were mobilised for middle joint subjected to impact loads and only catenary action was mobilised for side joint and bare steel joint. Flexural action could develop for middle joint and was much greater than that developed for side joint when subjected to the same impact load. Tear-out failure of fin plate, tensile fracture of profiled sheeting, longitudinal and diagonal cracks in the composite slab and crushing of concrete close to the joint governed the failure mode of middle FP joint. For side joints, tensile fracture of reinforcing bars was observed, together with fracture of profile sheeting and concrete. Final tear-out failure of fin plate of side joint was observed. Different from those cracks of middle joints, longitudinal and transverse cracks developed in composite slab of side joint since the slab was in tension. Composite slab effect could ensure flexural action of both the middle and the side joints subjected to impact loads. Combined with strain rate effect, much greater flexural resistances were achieved compared to the design values. Rotation capacities were also greater compared to the design values provided by UFC 4-023-03 (2013). Due to greater demand on deformation of the fin plates at the initial stage, tying resistances from the test results were smaller than the design values. However, tie force requirement from Eurocode 1 could be met for most of the composite joints. With a thicker concrete slab (100 mm), tie force requirement could not be met.

In contrast to quasi-static tests, strain rate effect could increase compressive arch action, catenary action and flexural action of composite FP joints in the impact tests. Side joint C75FP-M530H3-S was an exception due to accumulation of compressive damage of upper bolt rows during the impact. It gave a smaller T/D ratios of tying resistance and rotation capacity.

Beam-column joints with WUF-B connections subjected to impact loads

All the WUF-B joints could sustain the applied impact loads with small peak and residual displacements. Therefore, catenary action was not fully mobilised while flexural action could fully develop. When subjected to the same impact load, the middle joint could develop greater flexural action compared to the side joint. Moreover, a thicker composite slab contributed to greater flexural action. The respective maximum increases of concrete and steel strength were 28% and 17%, resulting from strain rates in the order of 1 s^{-1} , similar to FP joints although WUF-B joints were much stiffer.

Although failure of steel WUF-B connection was not observed, large deformation occurred at the same location as joints were subjected to quasi-static loads. Yielding of the beam flanges in tension was observed for all the WUF-B joints contributing to the residual displacement. Besides, yielding and buckling of the unrestrained beam flanges in compression were only observed in the side joint. Furthermore, the same crack patterns of composite slab occurred as the joints were subjected to quasi-static loads: longitudinal and diagonal cracks for the middle joint while longitudinal and transverse cracks for the side joint. Similar to joints subjected to quasi-static loads, composite slab effect could benefit flexural action of both the middle and the side WUF-B joints subjected to impact loads.

However, when eliminating the difference in material strengths, the strain rate effect could enhance the flexural resistance of composite WUF-B joints subjected to impact loads than those subjected to quasi-static loads. The enhancement increased with an increase of velocity of the joints after the impact, achieved by reducing joint stiffness (the side joint) and inertia (a thinner slab), and increasing impact velocity.

Component-based modelling approach

A component-based modelling approach has been proposed for steel and composite beam-column joints subjected to quasi-static and impact loads. In the proposed component-based models, beam-column joints were discretised into individual springs, including the concrete slab, reinforcing bar, profiled sheeting, beam flange and bolted connection with mechanical properties determined by material and geometry of individual component. Strain rate effect was incorporated through transforming the strain rate to velocity of movement of each spring when applying the models to impact loading scenarios. The proposed models could capture structural behaviour of joints under both quasi-static and impact loading regimes, including load, axial force and bending moment for quasi-static loads, as well as displacement, axial force and bending moment for impact loads.

9.2 Recommendations for future work

Based on the current research study, there are several promising aspects for future investigation as follows:

- (1) Improvement methods for the current joints studied is recommended. In the improvement method, the effect of position of the bolts needs to be investigated.
- (2) All the beam-column joints in the current study were investigated under point load to accommodate the validated set-up. Such investigation may be extended to uniformly-distributed loading as well. In real building structures, load-resisting frames are subjected to these two loads combined together depending on the floor layout.
- (3) An intermediate level of strain rate was triggered by the impact test set-up. In the future, higher strain rates introduced by pneumatic test rig and real explosives are recommended to further investigate the behaviour of the beam-column joints. Moreover, lower strain rate domain such as that introduced by free-fall scenario is promising. Behaviour of the beam-column joints under high temperature such as in a fire is also recommended since that building structures under terrorists' attack will probably be subjected to a

combination of high strain rate and temperature. The proposed component-modelling approach needs to be validated under aforementioned loading scenarios.

- (4) The current study focuses on two types of beam-column connections, namely, FP and WUF-B. It can be extended to more connection types such as end plate, cover plate, bolted angle and fully welded connections. Composite beam-column connections involving hollow section members are also recommended. Furthermore, abnormal loading scenarios are so complicated that they demand novel connection types with enhanced performance than conventional connections.
- (5) The proposed component-based modelling approach in the current study was validated by beam-column joint tests. It is more efficient than solid element modelling. To maximise the usefulness of what has been developed thus far, the proposed approach is more promising to be applied to simulations of large scale building structures, such as multi-bay and multi-storey frames. Currently, there are quite few quasi-static tests on large scale building frames with FP and WUF-B connection for validation purpose. Under dynamic scenario, such tests are even fewer.

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