The behaviour of steel and composite structures under a middle-column-removal scenario

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**THE BEHAVIOUR OF STEEL AND
COMPOSITE STRUCTURES UNDER A
MIDDLE-COLUMN-REMOVAL SCENARIO**

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**SCHOOL OF CIVIL AND ENVIRONMENTAL
ENGINEERING**

2013

**The Behaviour of Steel and Composite
Structures under a Middle-column-removal
Scenario**

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SUMMARY

The objective of this research is to study the behaviour of steel and composite structures under a middle-column-removal scenario. Firstly, the behaviour of different types of steel bolted beam-column joints under a middle-column-removal scenario has been studied by experimental tests. A group of numerical models was developed and validated by the conducted experimental tests. As a result of the success of applying finite element analyses, parametric studies have been conducted using these numerical models to obtain the rotation capacities of various types of connections under catenary action. In addition, from the test results of the first series, it is found that bolted-angle connections normally performed better than other types of connections in the development of catenary action. In order to achieve a better understanding on the performance of bolted-angle connections under a middle-column-removal scenario, nine experimental tests were carried out on different types of bolted-angle beam-column joints. A new mechanical model has also been developed to predict the behaviour of bolted-angle connections under pure tension. This new mechanical model is proposed based on a series of component tests. In this model, the deformation capacities of bolted-angle connection components can be predicted. Based on this mechanical model, component-based models of bolted-angle connections have been proposed to represent the key responses of beam-column joints under a middle-column-removal scenario, including the formation of flexural action at small deformation stage, the development of catenary action at large deformation stage and fractures of the connection components at the last stage.

Finally, a series of experimental tests was conducted in order to investigate the behaviour of composite beam-column joints under a middle-column-removal scenario and identify the contribution of composite slab to resist progressive collapse. The experimental tests consisted of two types of tests, namely, middle joints under sagging moment and side joints under hogging moment. A total of five composite beam-column joints with re-entrant steel profile decking were tested. The failure modes, the contribution of composite slab to resist progressive collapse and the key components which control the failure of the composite joints, were

discussed in this study. In addition, a component-based model was developed to predict the responses of composite beam-column joints subjected to a middle-column-removal scenario.

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LIST OF SYMBOLS

$b_{eff,a}$	Effective width of the bolted angles
c	Half of the steel beam flange width
c_f	Friction coefficient of bolts
d	Rebar diameter
d_b	Bolt diameter
d_{bh}	Bolt head diameter
$d_{b,hole}$	Bolt hole diameter
f_y	Yield strength of steel
$f_{y,b}$	Yield strength of bolts
f_u	Ultimate strength of steel
g	Gap between the beam web and the column flange (Figure 5.12)
h_{bg}	Depth of bolt group
h_B	Depth of beam
h_c	Middle column section depth
l	Effective length of the yield zone in beam flange and web
m	Gauge length between the centrelines of horizontal bolts and the vertical loading plate, as shown in Figure 5.1 (mm)
m_a	Distance between two plastic hinges
$m_{a,EC3}$	The value of m_a recommended by Eurocode 3-1-8 (2005)
$m_{a, Faella}$	The value of m_a recommended by Faella et al. (2000)
m^*	Distance between the two plastic hinges at the horizontal legs of angles
n	Number of bolts
n_a	Distance between the bolt centrelines and the prying force position
p_{pf}	Bolt pretension force during the tightening process
r_a	Radius of the angle fillet
t	Nominal steel angle or plate thickness (mm),
t_a	Angle thickness
u_d	Maximum dynamic response

A_b	Tensile stress area of bolts
$B_{T,Rd}$	Ultimate loading resistance of bolts
F_A	Ultimate loads predicted by Astaneh-Asl's model
$F_{A,l}$	Load limits due to bolt fracture predicted by Astaneh-Asl's model
F_b	Axial resistance of bolts
$F_{B,l}$	Load limits due to bolt fracture predicted by Blass's model
F_E	Ultimate loads predicted by Eurocode 3-1-8
$F_{E,l}$	Load limits due to bolt fracture predicted by Eurocode 3-1-8
F_F	Ultimate loads predicted by Faella's model
F_g	Yield strength of angle legs
F_i	Force at step i
F_P	Ultimate loads predicted by the proposed model
$F_{P,l}$	Load limits due to bolt fracture predicted by the proposed model
F_s	Bolt slippage friction resistance
F_T	Ultimate loads from experimental tests
$F_{T, Rd}$	Design resistance of T-stub in tension
F_u	Load-carrying capacity of bolted-angle connections
F_l	Elastic loading resistance F_1 of steel beam flange and web in compression
I_1	Inertia moment of angle leg 1 shown in Figure 5.13
I_2	Inertia moment of angle leg 2 shown in Figure 5.13
K_a	Initial stiffness of angles in bending
K_h	Horizontal restraint stiffness
K_c	Elastic stiffness of one beam-column connection under tension
K_l	Elastic stiffness of bolted-angle connections
K_2	Stiffness of bolted-angle connections at transition stage
K_3	Stiffness of bolted-angle connections at large deformation stage
L	Length of the horizontal angle leg
L_1	Length of angle leg 1 shown in Figure 5.13
L_2	Length of angle leg 2 shown in Figure 5.13

M_a	Plastic moment capacity of the T-stub flange section
N_a	Tensile forces of angles at large deformation
N_B	Maximum axial force of the beam
P	Applied load
P_E	Load capacity from experimental results (kN)
P_C	Load capacity from calculated values (kN)
$P.u_s$	A given nonlinear static response
Q	Prying action
S_0	Original cross-section area of coupons
T	Tensile resistance of connection
T_D	Design values of tying resistances (kN)
T_E	Tying requirements in (Eurocode 1-1-7 2006)
T_T	Test results of tying resistances (kN)
T_θ	Beam-column joint tensile resistance after undergoing rotation
θ	
σ_{eng}	Engineering strain
σ_{true}	True stress
ε_{eng}	Engineering strain
ε_{true}	True strain
ε_u	Fracture strain of steel
ε_y	Yield strain of the steel material
θ	Rotation angle
θ_{pra}	Plastic rotation angle
θ_y	Yield rotation angle
λP_0	Sudden application of gravity load
δ	Displacement
δ_i	Displacement at step i
δ_u	Deformation capacity of bolted-angle connections
$\delta_{u,A}$	Deformation capacity predicted by Astaneh-Asl's model
$\delta_{u,B}$	Deformation capacity predicted by BCSA/SCI model
$\delta_{u,P}$	Deformation capacity predicted by the proposed model

$\delta_{u,T}$	Deformation capacity from experimental tests
δ_1	Yield displacement of steel beam flange and web in compression
δ_2	Deformation limits of steel beam flange and web in compression shown in Figure 7.32
γ	normalized stiffness ratio
γ_a	Angle stiffness coefficient
η	Coefficient related to the strength ratio between bolts and angles
β	Coefficient to determine the dimensions of yield lines and plastic zones
ξ	Coefficient to determine the dimensions of yield lines and plastic zones
ψ_1	A coefficient given by Equation (5.7)
Δ	Displacement increase at each incremental step
ΔF	Force increase at each incremental step

CHAPTER 1 INTRODUCTION

1.1 Background

After the partial collapse of the Ronan Point apartment tower in 1968 (Figure 1.1), engineers begin to realise the importance of structural resistance against progressive collapse. More and more research works and design efforts have been directed to this area, especially after the World Trade Centre disaster on 11 September 2001 (Figure 1.2).

Figure 1.1 Ronan Point building after 16 May 1968 collapse

The General Services Administration (GSA 2003) offered a somewhat more specific description of this phenomenon: “Progressive collapse is a situation where

local failure of a primary structural component leads to the collapse of adjoining members which, in turn, leads to additional collapse.”

Figure 1.2 World Trade Centre 1 and 2 on 11 September 2001

The alternate load path method, an important design approach to mitigate progressive collapse, has been included by a number of design codes including GSA (2003) and DoD (2009). This approach allows a local failure to occur when subjected to an extreme load, but seeks to provide alternate load paths so that the initial damage can be contained and major collapse be averted. A typical example is shown in Figure 1.3 under the scenario when an interior column has been removed by blast and an alternate load path can take place through adjacent structural assemblage including beams, columns and joints.

One of the key mechanisms to mitigate the spread of *domino* effect is to redistribute the applied load on damaged members through *catenary action*. As shown in Figure 1.3, the term *catenary action* refers to the ability of beams to resist vertical loads through the formation of a string-like mechanism. Catenary action implies the development of sufficiently large deformations, such that associated debris impact

loads are mainly resisted by the vertical components of axial forces developed in the beams through catenary action. Under such condition, beams are unable to resist the applied loads through flexural action alone and seek supplementary resistance through the formation of catenary configuration. The structural system must be sufficiently ductile to allow the new configuration to be achieved while maintaining adequate integrity to support the developed catenary forces (Khandelwal and El-Tawil 2007).

Figure 1.3 Typical example of alternate load path

Besides catenary action, another mechanism that can be mobilised to prevent progressive collapse is tensile membrane action of three dimensional (3D) floor system. Many research works (Li and El-Tawil 2011, Sadek et al. 2008, Vlassis et al. 2008) have indicated that tensile membrane action of 3D floor system may play an important role against progressive collapse. The numerical analytical results of Sadek et al. (2008) indicate that the formation of tensile membrane action mainly relies on the reinforcement mesh and profile decking in composite floor system.

According to Eurocode 1-1-7 (2006) and some other different national design codes (GSA 2003, DoD 2009), structural integrity should be ensured through appropriate measures but in most cases, no precise practical guidelines on how to achieve this goal is provided. Even the basic requirements to fulfil are generally not clearly stated (Demonceau 2008).

The main issues that require an urgent study currently include: i) the structural behaviour of beam-column joints under column-removal scenarios, including the formation of flexural action at small deformation stage, the development of catenary action at large deformation stage and fractures of the connection components at the last stage, ii) the performance of 3D floor system to develop tensile membrane action under column-removal scenarios, and iii) the design guidelines that can provide an analytical tool to predict structural responses up to total failure.

1.2 The behaviour of steel beam-column joints under column-removal scenarios

Figure 1.4 Beam-column joint failure mode under catenary action (Karns et al. 2009)

It is noteworthy that the beam-column joints are the most critical elements in steel and composite structure systems and they usually control the extent of catenary action because of the limited resistance and rotation capacity of joints. Figure 1.4 shows a failure mode of a welded beam-column joint subjected to catenary action under a column-removal scenario. However, although there have been extensive research studies on different types of joint behaviour under *normal loads*, which have led to the codification of component-based approach to joint design (Eurocode 3-1-8 2005), to date, there are relatively very few research studies of the joint behaviour under column-removal scenarios, especially for bolted steel connections.

Only if large deformations are developed could catenary action be mobilised and in turn redistribute the vertical loading to other columns and beams. Large rotations will occur in the joints whereas significant tensile forces will be formed in the beams and joints. This indicates that bending moments and tensile forces will be transferred through the beam-column joints. However, this loading condition has never been considered in the Eurocodes and other design codes, which means no design method is available to calculate the resistance and stiffness of joints under this extreme loading condition. At the same time, the beam-column joints must be able to transfer large tensile forces if alternate load path forms and catenary action occurs. Thus, experimental and analytical works are required to find out how the internal forces of joints change from flexure to tensile force so as to insure this procedure could be carried out smoothly.

According to the author's knowledge, there are very limited analytical and experimental works on the influence of the joint configuration on the extent of catenary action, especially on bolted steel and composite joints, which are the most popular connection types in Europe. Thus, it is necessary to conduct further research works on this topic. In this study, the behaviour of steel bolted beam-column joints under column-removal scenarios will be studied. It should be noted that the scope of this research is focused on a middle-column-removal scenario, under which condition both ends of the beam elements have sufficient horizontal restraints to provide catenary forces in the affected beams.

1.3 The behaviour of composite steel beam-column joints under column-removal scenarios

In the alternate load path method, the middle joint above the removed column shown in Figure 1.5, is subjected to a sagging moment while the side joints are under hogging moments. Due to the presence of composite slabs above steel joints, the structural behaviour of side and middle composite joints, including flexural and catenary resistances and rotation capacities, is different from each other. Therefore,

to obtain a better understanding about the composite structural behaviour under column-removal scenarios, the middle and side beam-column joints should be tested individually. The isolated joint tests are also necessary because these joint test data can be used directly to validate the analytical models.

Figure 1.5 Composite frame subjected to a middle-column-removal scenario

Whilst many experiments have been conducted in the past few decades to study the behaviour of composite joints under gravity and cyclic loads or under fire conditions (Anderson and Najafi 1994; Fu and Lam 2006; Wald, da Silva et al. 2006; Shi, Li et al. 2007; Wang and Li 2007; Fu, Lam et al. 2008; Wang and Li 2008; Fu, Lam et al. 2010; Yuan, Tan et al. 2011; Yuan, Tan et al. 2011), there are limited experimental data of composite beam-column joints subjected to column-removal scenarios. Analytical and numerical studies (Vlassis et al. 2008, Alashker et al. 2010) have demonstrated that composite slab systems have significant beneficial effects to enhance the structural resistances against progressive collapse. The analytical works of Vlassis et al. (2008) indicate that the provision of additional reinforcement in the slab over the hogging moment regions has a beneficial effect on both the dynamic load-carrying and deformation capacities. Alashker, El-Tawil et al. (2010) stated that composite floors could develop significant tensile forces in steel beams, steel profile decking, and slab reinforcement. The steel profile decking was identified as the main source of the structural resistance of the slab. In DoD

(2009), the tie force (TF) method is applied by specifying the minimum tensile forces that must be designed to tie the structures together. In the TF method, loads from the damaged portion of the structure are redistributed to the remainder of the structure through catenary action or tensile membrane action. Unless structural members (beams, girders or spandrels) and their connections are capable of carrying the required longitudinal, transverse, or peripheral tie forces while undergoing rotations of 0.20-rad (11.3-deg), otherwise these tie forces have to be carried by the floor and roof system (DoD 2009). From the previous experimental test results (Yang and Tan 2011, Demonceau 2008, Sadek et al. 2011, Karns et al. 2009), it is found that most steel connections do not achieve a rotation capacity of 0.20-rad (11.3-deg). This indicates that in DoD (2009) horizontal tie forces are no longer permitted to be carried by beams and girders for steel and composite structures. Instead, the horizontal tie forces should be carried by slabs alone. In this study, experimental tests will be carried out to validate these findings and recommendations for 2-D frame analyses.

1.4 Development of design guidelines against progressive collapse

1.4.1 Rotation capacities of steel connections considering catenary action

Beam-column joints sustain substantial tensile forces at large rotation stage and fractures of the connection components determine the ultimate resistances of joints. Therefore, the connection rotation capacities determine the joint capacities to form catenary action and govern structural resistances against progressive collapse. A series of rotation capacity criteria of different types of steel connections subjected to progressive collapse have been proposed by DoD (2009). However, these nonlinear acceptance criteria are based on the experimental tests of beam-column joints under seismic loads. But the rotation capacities of beam-column joints under monotonic loads are much greater than the rotation capacities obtained from cyclic tests (Kim et al. 2011). In addition, only pure flexural action contributes to the connection resistances against seismic loading, while under a middle-column-removal scenario, flexural action as well as catenary action could resist the applied monotonic load.

Therefore, it is necessary to propose a new series of acceptance criteria of connection rotation capacities in order to consider the contribution from catenary action.

1.4.2 Improvement of the component-based method

Figure 1.6 Detailed model of connection using FEM

Due to the complexity of progressive collapse analyses, finite element method (FEM) is usually used to simulate the complex loading and structural response. For the analyses of beam-column connections, the detailed FEM model (Figure 1.6) is needed to obtain the accurate responses of joints under this complex loading. However, due to the complexity of using detailed joint models and significant computational resources, it is not feasible to use this method for design although it is quite popular in research. A more practical and simplified method of modelling joints is needed for analyzing the behaviour of the whole structure against progressive collapse.

In Eurocode 3-1-8 (2005) and Eurocode 4-1-1 (2004), the component-based method is recommended for the analysis and design of steel connections. In this method, the connection is divided into several individual components. The behaviour of connection can be predicted by the assembly of components. Figure 1.7 shows a component-based model of a bolted-angle connection. In the two design codes

(Eurocode 3-1-8 2005, Eurocode 4-1-1 2004), the component-based method for calculating the stiffness and resistance of every component in the joint is given under the condition that only bending moment is applied on the joint, and at the same time the ductility of components is not given in these codes. As shown in Figure 1.7, the values of F_{Rd} and K have been given by the codes while limited information is provided about the component deformation capacity δ . Nevertheless, during the process of progressive collapse, if one column is removed and large deformations are developed, the joints will sustain large tensile forces and the ductility of the most critical component governs the rotation capacities of joints. The research works of Yu et al. (2009a and 2009b) demonstrated that at the stage of large deformation, fracture of the connection components determined the ultimate resistances of joints. Thus, the behaviour of beam-column joints under tension governs the structural resistances against progressive collapse. Therefore, the behaviour of beam-column joints under tension should be well understood in order to assess the robustness of steel buildings against progressive collapse. In addition, the failure criteria to determine the deformation capacities of beam-column joints under tension should be articulated and quantified to predict the ultimate resistances of joints at large deformation stage. However, traditional design of connections is limited to moment-rotation characteristics and the information about the deformation capacities of connection components is quite limited. Thus, it is necessary to improve the component-based method in order to predict the fracture of each connection component and the development of catenary action.

Figure 1.7 Component-based model of a bolted-angle connection

1.5 Objectives and scope of the research work

The objective of this research is to study the behaviour of steel and composite structures under a middle-column-removal scenario. Both the steel and the composite steel joint behaviour will be studied. The aims are:

- 1) To investigate the behaviour of different types of steel bolted beam-column joints subjected to catenary action under a middle-column-removal scenario, including failure modes, development of forces and rotation capacities;
- 2) To study the behaviour of bolted-angle connections subjected to catenary action and to develop a component-based model, which could predict the behaviour of bolted-angle beam-column joints up to total failure under column-removal scenarios;
- 3) To investigate the behaviour of composite beam-column joints under a middle-column-removal scenario.

The first objective was achieved by a series of experimental tests as well as numerical simulations. In total, seven experimental tests have been carried out on different types of steel beam-column joints, including simple and semi-rigid connections. In the group of simple connections, web cleat, top and seat angle, top and seat with web angle (TSWA) (8 mm thickness angles) and fin plate connections were investigated while flush end plate, extended end plate and TSWA (12 mm thickness angles) constituted the group of semi-rigid connections. This study provides the behaviour and failure modes of different connections, including their abilities to deform in catenary mode. The experimental results of this series were used to calibrate the numerical simulation models. Both static and explicit dynamic solvers were employed to overcome the problems of convergence, contact, large deformation and fracture simulations. It is demonstrated that the finite element analyses give reasonable accuracy compared to the test results. In addition, an extensive parametric study was undertaken using these validated models to obtain the rotation capacities of various types of connections under catenary action. Finally, some practical design implications have been drawn up from the parametric study and four new connection acceptance criteria of rotation capacities have been

proposed to consider catenary action under a middle-column-removal scenario. The experimental programme will be presented in Chapter 3, followed by the numerical simulations and parametric studies in Chapter 4.

In order to achieve the second objective, an experimental investigation of bolted-angle connections under tension was conducted. In total, fourteen specimens were tested. The parameters studied in the experiments included bolt hole positions, angle thickness, bolt size and material properties. Based on the experimental results, a new mechanical model of bolted-angle connections was developed. In the proposed model, the following issues were addressed: (i) the interaction between angles and bolts, (ii) failure criteria to determine the deformation capacities of connection components, and (iii) load limits due to bolt fracture. These experimental and analytical works will be presented in Chapter 5. In addition, Chapter 6 will present another series of experimental tests of bolted-angle beam-column joints under a middle-column-removal scenario. Three types of connections, including (1) web cleat, (2) top and seat angle and (3) top and seat with web angles connections were investigated while three thicknesses (8, 10 and 12 mm) of angles were tested. Nine experimental tests have been conducted. The experimental results demonstrate the ductility and load-carrying capacities of three connection types with different angle thicknesses in catenary action mode. When the angle thickness increases, the failure mode changes from angle fracture to bolt fracture. A component-based model is also developed to predict the behaviour of bolted-angle beam-column joints up to total failure. The validation study indicates that the proposed models can represent the key responses of bolted-angle beam-column joints under a middle-column-removal scenario, including the formation of flexural action at small deformation stage, the development of catenary action at large deformation stage, and fractures of the connection components at the last stage. Based on the proposed joint model, frame analyses are conducted. The analytical results of the frame models demonstrate that the joint tests could represent the behaviour of prototype steel frames against progressive collapse. The effect of horizontal restraint stiffness, which is critical to the development of catenary action, is also investigated.

A series of experimental tests of composite beam-column joints was conducted in order to achieve the third objective. It consisted of two types of tests, namely, middle joints under sagging moment and side joints under hogging moment. A total of five composite beam-column joints with re-entrant steel profile decking were tested. The contributions from two types of mechanisms, namely, flexural and catenary actions, have been identified. The failure modes, the contribution of composite slab to resist progressive collapse and the key components which control the failure of the composite joints, were discussed in this study. Finally, a component-based model was developed to predict the responses of composite beam-column joints subjected to a middle-column-removal scenario. Chapter 7 will present this experimental and analytical works about this topic.

CHAPTER 2 LITERATURE REVIEW

2.1 Overview

A literature review is carried out to determine the research objectives and the scope of study for the entire programme. Firstly, the existing robustness design provisions and various new design guidelines for progressive collapse mitigation are reviewed. This is followed by a more critical review of the behaviour of steel, composite and concrete structures subjected to catenary action, based on the experimental perspective. After that, a discussion on the ductility analyses of component-based joint modelling is presented. The chapter ends with a discussion on the dynamic effect under sudden-column-removal scenarios.

2.2 Design approaches to prevent progressive collapse

Methods of mitigating progressive collapse are incorporated in many building codes or standards, especially in Europe and North America. Both direct and indirect approaches are included in these codes. The indirect design approach requires the development of minimum ties, at specific locations of a structure. Alternate load path and key element methods are included in the direct design approach.

2.2.1 Indirect approach—tying capacity design

In the indirect approach—tying capacity design, an entire building is mechanically tied together, enhancing continuity, ductility, and residual strength through catenary action in the structure. In BS 5950-1 (2000), Eurocode 1-1-7 (2006) and DoD (2009), the indirect method is expressed in terms of tie requirements, including horizontal and vertical ties.

The provision of ties increases structural continuity, thereby creating a structure with a high degree of redundancy, should part of the structure be removed by an

accidental action. Generally, ties are provided by steel members or steel reinforcement. Besides, the beam-column joints can aid in the transfer of tying forces. Ties are intended to provide a minimum level of tensile capacities in members and connections in the event of extreme loads.

Figure 2.1 Tie force method (DoD 2009)

In DoD (2009), as shown in Figure 2.1, the tie force (TF) method is used by specifying the minimum tensile forces that must be designed to tie the structure together. In the TF method, loads from the damaged portion of the structure are redistributed to the rest of the structure through catenary action or tensile membrane action. In DoD (2009), it is stated that “*unless structural members (beams, girders, spandrels) and their connections are capable of carrying the required longitudinal, transverse, or peripheral tie force magnitudes while undergoing rotations of 0.20-rad (11.3-deg), the longitudinal, transverse and peripheral tie forces are to be carried by the floor and roof system*”. However, most steel connections do not have a rotation capacity of 0.20-rad (11.3-deg). This indicates that in DoD (2009) horizontal tie forces are no longer permitted to be carried by beams and girders for steel structures. Instead, the horizontal tie forces should only be carried by the slab system. However, in BS 5950-1 (2000) and Eurocode 1-1-7 (2006), horizontal tie

forces are to be carried by steel connections, which requires tying capacities. Therefore, further studies should be conducted to validate these findings and recommendations.

2.2.2 Direct approach—alternate path method

The alternate path method shown in Figure 2.2, an important design approach to mitigate progressive collapse, has been included by a number of design codes including BS 5950-1 (2000), Eurocode 1-1-7 (2006), GSA (2003) and DoD (2009). It is an approach that allows local failure to occur when subjected to an extreme load, but seeks to provide alternate load paths so that the structure is capable of bridging over a removed structural element and major collapse can be averted. The portion of building at risk of collapse should not exceed 15% of the floor or roof area, or 70m^2 (which is less) at the relevant level and at one immediately adjoining floor or roof level. If the notional removal of a column, or of an element of a system providing resistance to horizontal forces, would risk the collapse of a greater area, that column or element should be designed as a key element (BS 5950-1 2000).

Figure 2.2 Typical example of alternate load path

In GSA (2003) and DoD (2009), only one column is removed each time during the analysis. The types of analyses permitted include elastic static, inelastic static and inelastic dynamic analyses, in which the acceptance criteria are different for each type of analysis. A series of rotation capacity criteria of different types of steel

connections subjected to progressive collapse has been proposed by DoD (2009). However, since there are limited test data under column-removal scenarios, these nonlinear acceptance criteria are based on the experimental tests of beam-column joints under seismic loads. The behaviour of beam-column joints under column-removal scenarios may be quite different from the performance under seismic loads. Therefore, it is necessary to conduct further investigations on the rotation capacities of beam-column joints under column-removal scenarios.

2.2.3 Direct approach—key element method

For any critical structural elements over which the building cannot bridge, the elements must be designed as "key" or "protected" elements, capable of carrying a static pressure loading of 34 kN/m^2 (Eurocode 1-1-7 2006). The accidental load should be applied to the member from horizontal and vertical directions, in one direction at a time, together with the reactions from other building components attached to the member that are subject to the same accidental load, but limited to the maximum reactions that could reasonably be transmitted (BS5950-1 2000).

2.3 Previous research studies of steel, composite and concrete structures subjected to catenary action

Following the World Trade Centre disaster, some researchers have identified joint integrity as a key parameter to maintain structural integrity under catenary action and have conducted some research works.

Khandelwal and EI-Tawil (2007) used structural simulation to investigate a number of key design variables that influence the formation of catenary action in special steel moment resisting frame sub-assemblages. Welded joints with and without reduced steel beam sections were considered. Fahim et al. (2009) conducted an experimental and analytical assessment of the performance of steel beam-column assemblies with two types of moment-resisting connections similar to the ones investigated by Khandelwal and EI-Tawil (2007) under a middle-column-removal

scenario. The test set-up and instrumentation layout are shown in Figure 2.3. The failure modes and load history curves are given in Figures 2.4 to 2.6. It was observed that reduced beam section connections had greater resistances, rotation capacities and better performance to form catenary action compared to welded unreinforced flange-bolted web connections.

Figure 2.3 Fahim's test set-up and instrumentation layout (Fahim et al. 2009)

Figure 2.4 Failure modes of two types of joints (Fahim et al. 2009)

In 2009, Karns et al. (2009) conducted a test programme consisting of a steel frame subjected to a blast. Conventional welded moment and side-plate moment connections were investigated. Progressive collapse loading conditions were

simulated by using double-span steel frame assemblies, and both blast-damaged and non-blast-damaged specimens were tested under monotonic vertical loads as shown in Figure 2.7.

Figure 2.5 (a) Applied vertical load and (b) beam axial force versus vertical displacement at centre column of the WUF-B specimen (Fahim et al. 2009)

Figure 2.6 (a) Applied vertical load and (b) beam axial force versus vertical displacement at centre column of the RBS specimen (Fahim et al. 2009)

Demonceau (2008) conducted a substructure experimental test and five beam-column joint tests in order to observe the development of catenary action and its effect on the joint behaviour. The $M-N$ interaction curves of composite joints (under hogging and sagging moments) were included in his work (2008). The substructure test configuration and failure deflection are shown in Figure 2.8. Large deformations and catenary action have been observed in his test.

Figure 2.7 Karns's test (Karns et al. 2009)

(a) Test configuration

(b) Failure deflection

Figure 2.8 Substructure test (Demonceau 2008)

Tsitos (2009) conducted experimental tests of steel frames under a column-removal scenario. Two 1/3 scale three-storey, two-bay steel frames were tested. One was a special moment resisting frame (WMRF) and the other one a posttensioned energy dissipating frame (PTED) designed and previously tested for seismic performance on earthquake simulators. These two frame specimens were tested under quasi-

static collapse loading by simulating the structural response after the sudden failure of a column. The WMRF specimen provided further resistance through wide-spread yielding at members' ends and connection panel zones, until a series of fractures compromised its strength. On the other hand, the collapse resistance of the PTED specimen was primarily dependent on the ultimate ductility and failure mode of the wire strands that provided the post-tensioning force (Tsitos 2009).

Figure 2.9 WMRF specimen of Tsitos (2009)

Figure 2.10 PTED specimen of Tsitos (2009)

Yi et al. (2008) performed an experimental test of a four-bay and three-storey one-third scale reinforced concrete frame under a bottom-column-removal scenario. A constant vertical load was applied to the top of the middle column by a servo-hydraulic actuator to simulate the gravity load of the upper floors and the failure of the middle column of the first storey was simulated by unloading a mechanical jacking system. The frame collapse, defined in this study as the rupture of tension steel bars in the floor beams, occurred at a vertical unloading displacement of 456 mm (18 in.) that corresponds to a beam drift angle (rotation with respect to the

horizontal) of 10.3 degrees. The frame specimen after the test is shown in Figure 2.11. The measured load-displacement is given in Figure 2.12, from which it can be seen that at large deformation stage, catenary action could be involved to sustain the vertical loads.

Figure 2.11 RC frame specimen at the collapse limit state (Yi, He et al. 2008)

Figure 2.12 Load-displacement curve of the RC frame specimen (Yi et al. 2008)

Yu and Tan (2012) conducted a series of experimental tests on reinforced concrete (RC) beam column sub-assemblages under a middle-column-removal scenario. The tests were performed under static loads. One of the tested specimen is shown in Figure 2.13. It is found from the tests that with adequate axial restraints, both compressive arch action (CAA) and catenary action could be mobilized,

significantly increasing the structural resistance beyond the beam flexural capacity. Three of the measured load-displacement curves are given in Figure 2.14, which shows that at large deformation stage, catenary action formed well to increase the load-carrying capacities.

Figure 2.13 Failure modes of specimen S5 (Yu and Tan 2012)

Figure 2.14 Load-displacement curves of Yu and Tan's tests (Yu and Tan 2012)

Many experimental studies have focused on the behaviour of steel beam-column joints under catenary action in fire. Ding and Wang (2007), Dai et al. (2010),

Elsawaf et al. (2011) and Wang et al. (2011) conducted experimental tests and numerical simulations of structural fire behaviour of steel beam to column assemblies using different types of joints. Ding and Wang (2007) conducted 10 tests of structural fire behaviour of steel beam to concrete filled tubular (CFT) column assemblies using different types of joints. The principal aim of this study was to present experimental results of joint behaviour in fire (which until now is lacking) so as to achieve a better understanding of rational design methods for robust construction of joints to resist fire. The results of the experiments indicate that even the relatively simple joints used in this study were able to allow the beams to develop substantial catenary action, so that the final failure times and beam temperatures of the assemblies were much higher than those obtained by assuming the beams in pure bending. One of the test deformed specimens is shown in Figure 2.15, which shows large deformations of the beam and development of catenary action.

Figure 2.15 Overall deformed shape of one test specimen (Ding and Wang 2007)

Yu et al. (2008a, 2008b, 2009a, 2009b) conducted a series of experimental tests to investigate the robustness of common types of steel connections when subjected to fire. Fin plate, flexible end plate, flush end plate and web cleat connections were tested under fire conditions. One typical test set-up is shown in Figure 2.16. The key objective was to include major aspects of joint behaviour into global structural analysis for performance-based design of steel and composite structures by means of the component-based method.

Figure 2.16 Test set-up (Yu et al. 2009a)

The experimental tests and analytical studies mentioned above were conducted until total structural collapse took place at large deflection. Most of these investigations demonstrated that connection failures controlled the final collapse. In the small deformation stage, flexural action dominates the behaviour of joints, while at the large deformation stage, beam-column joints are subjected to catenary action and tensile failure controls the final failure modes of joints.

So far, under column-removal scenarios, most of the reported experimental tests focus on welded moment connections (Khandelwal and El-Tawil 2007, Fahim et al. 2009, Karns et al. 2009) and only very limited experimental investigations have been conducted on bolted steel connections subjected to catenary action. Compared with welded moment connections, bolted steel beam-column joints, which are usually classified as simple or semi-rigid connections, can be more vulnerable under extreme loading conditions. But the behaviour of bolted steel connections under column-removal scenarios, in which the connections are subjected to monotonically increasing combined bending and tension, has not been experimentally investigated well. In Europe and in some commonwealth areas such as Singapore and Hong Kong, bolted steel connections such as fin plate, flush end plate, web cleat and extended end plate, are very popular. Therefore, the evaluation of these kinds of joints subjected to catenary action is important and timely.

2.4 Joint modelling under column-removal scenarios using the component-based method

Under column-removal scenarios or fire conditions, at large deformation stage, catenary action could develop. This is normally associated with excessive ductility demands, which can often be localized in the joint regions of the members involved. Thus, mitigation of progressive collapse hinges on the ability of joints to accommodate large ductility demands. The main aim of this section is to present a literature review of mechanical models for evaluating the rotation capacity and ability to form catenary action of bolted-angle connections.

The capacity of members to form catenary action under column-removal scenarios is directly related to the rotation capacities of connections. The weakest component in the mechanical model controls the ductility of connections, and governs the joint resistance to progressive collapse. If connections have sufficient ductility, enormous catenary action could develop and prevent progressive collapse. On the other hand, if brittle failure governs the rotation capacities of connections, catenary action could hardly contribute to structural resistance against progressive collapse. Therefore, the ductility assessment of connections is a crucial part in the evaluation of the extent of catenary action.

The evaluations of stiffness and moment resistance of joints are provided by Eurocode 3-1-8 (2005). However, concerning the connection ductility, only some qualitative principles are given in this code. These rules include: if the bending deformation of the flanges governs the plastic mechanism, the rotation capacity is sufficient for plastic analysis. Thus, a beam-column joint in which the design moment resistance of the joint is governed by the design resistance of the column web panel in shear, may be assumed to have adequate rotation capacity for plastic global analysis. These rules are, as yet insufficient.

In the past, the behaviour of bolted-angle connections has been studied by many researchers. Azizinamini and Radzinski (1987, 1989) performed an experimental

study of bolted-angle connections under static and cyclic loading conditions. An analytical procedure was also developed to predict the initial stiffness of moment-rotation curves. Chen and his colleagues (Kishi and Chen 1987, Chen et al. 1988a, Chen et al. 1988b) worked extensively to study the behaviour of bolted angle connections. A mechanical model to predict moment-rotation relationships was proposed. Faella et al. (2000) proposed a component-based model of bolted-angle connections, which produced good estimates of moment-rotation relationships. Eurocode 3-1-8 (2005) also uses the component-based method to predict the behaviour of bolted-angle connections. In this code, the component of angles in bending is treated as an equivalent bolted T-stub. An elastic-plastic model is proposed for this component. Elghazouli et al. (2009) performed an experimental investigation of blind-bolted angle connections between open beams and tubular columns and developed a component-based mechanical model (Málaga-Chuquitaype and Elghazouli 2010) to predict the monotonic and cyclic responses of blind-bolted angle connections. Yuan and Tan (2011a, 2011b) performed experimental and analytical studies on composite steel top-and-seat-and-web angle joints at ambient and elevated temperatures. The main objective of this study was to ascertain the moment-rotation characteristics of composite angle joints. The above studies covered only moderate deformations and were limited to moment-rotation relationships. Most previous tests of bolted-angle connections stopped once a connection component yielded or the connection rotation angle reached certain values without any fracture. In those mechanical models, only the strength and stiffness of the connection components were given and the ductility (deformation capacity) was not introduced into the models. Currently, the deformation capacity of each component has not been addressed and studied well (Wang 2011). However, under column-removal scenarios, catenary action will be formed at large deformation stage and rotation capacities and tensile resistances of beam-column joints usually control the final failures and govern the contribution from catenary action. Therefore, it is necessary to conduct further investigations on the deformation capacities of bolted-angle connections at the last stage prior to fracture of joints

Under other extreme loading conditions such as earthquake and fire, some research works have been conducted to develop the mechanical models of bolted-angle connections, which could predict fractures of connection components. Shen and Astaneh-Asl (1999, 2000) performed an experimental investigation and proposed a hysteretic model of bolted-angle connections under seismic loading conditions. In this model, large deformation effect was considered and the deformation capacities of bolted-angle connections were also proposed. Yu et al. (2009a, 2009b) performed an experimental investigation and proposed a component-based model of web cleat connections subjected to tying forces at elevated temperatures. In this model, large deformation and component fracture were included. It was assumed that the ultimate capacity of the whole web cleat was reached when the angle was in pure tension. This failure criterion may not be accurate enough to predict the deformation capacities of bolted-angle connections. In the ambient tests of this research work, only one failure model (angle fracture) was observed and considered in the mechanical model. In addition, this model was too complicated to be used for hand calculations. Under column-removal scenarios, the loading condition is different from seismic or fire loads. Further studies are needed to develop a new mechanical model of bolted-angle connections subjected to catenary action under column-removal scenarios.

2.5 Dynamic effect

Table 2.1 Dynamic increase factors for nonlinear static analysis (DoD 2009)

Material	Structure Type	DIF
Steel	Framed	$1.08+0.76/(\theta_{pra}/\theta_y+0.83)$
Reinforced Concrete	Framed	$1.04+0.45/(\theta_{pra}/\theta_y+0.48)$
	Load-Bearing Wall	2
Masonry	Load-Bearing Wall	2
Wood	Load-Bearing Wall	2
Cold-formed Steel	Load-Bearing Wall	2

Under blast or impact loading conditions, sudden-column-removal scenarios involve dynamic effects. In GSA (2003), the dynamic effects of sudden column loss are considered using an equivalent static load multiplied with a constant Dynamic

Increase Factor (DIF) of 2.0. DIFs for nonlinear static analysis have also been suggested by DoD (2009). As shown in Table 2.1, DoD (2009) incorporates the expressions of the DIF in terms of ductility for different types of structures. In Table 2.1, θ_{pra} is the plastic rotation angle and θ_y is the yield rotation.

According to the expressions in Table 2.1, as an example, the results of this procedure for the DIF, used for nonlinear static analyses of steel structures, are shown in Figure 2.17, where the DIF is shown as a function of the normalized rotation (allowable plastic rotation divided by the rotation at yield of the cross section). Figure 2.17 shows that the DIF reduces monotonically with the normalized rotation.

Figure 2.17 DIF for structural steel (DoD 2009)

Izzuddin et al. (2008) and Vlassis et al. (2008) proposed a novel simplified framework for progressive collapse assessment of multi-storey buildings, considering sudden column loss as a design scenario. Three main stages are utilised in the proposed assessment framework, including the determination of the nonlinear static response, dynamic assessment using a novel simplified approach, and ductility assessment. In the simplified dynamic assessment, for a given nonlinear static response ($P.u_s$), which is considered at the appropriate level of structural idealisation, the maximum dynamic response (u_d) associated with the sudden

application of gravity load (λP_0) can be determined from energy balance between the work done by the load and the internal energy stored, as shown in Figure 2.18.

Figure 2.18 Simplified dynamic assessment of Izzuddin's model (Izzuddin et al. 2008)

Based on this simplified dynamic assessment approach (Izzuddin et al. 2008), further analyses were conducted by Izzuddin (2010). As shown in Figure 2.19, for an elastic-plastic system, the DIF reduces monotonically with available ductility, which corresponds well with DoD (2009). However, as shown in Figure 2.20, for an elastic-plastic hardening system, the DIF can increase with ductility to high values, which are much greater than those adopted in DoD (2009).

Figure 2.19 Characteristic responses and DIF for elastic-plastic system (Izzuddin 2010)

Figure 2.20 Characteristic responses and DIF for elastic-plastic hardening system (Izzuddin 2010)

Figure 2.21 Dynamic test set-up (Liu and Tan 2012)

Liu and Tan (2012) conducted dynamic tests of web cleat connections under a sudden-column-removal scenario. In the test, different uniformly distributed loads were applied at the two beam spans, which were connected to a middle column with double angle web cleat connections. The test set-up is shown in Figure 2.21. Both dynamic and static tests were conducted for a better understanding of the dynamic effects on the structural performance. As shown in Figure 2.22, the test results show that the maximum displacement of the web cleat connection under sudden column removal is significantly increased compared with the one under static loading condition. Both the force and displacement based DIFs were evaluated, and the resistance of the web cleat connections to progressive collapse considering dynamic effects was discussed in this study. From this study, it is concluded that measured

DIFs can be predicted well by Izzuddin's model (2008). In the research works of the current thesis, Izzuddin's model will be used to assess the dynamic effect.

Figure 2.22 Comparison of test results on force-displacement relationship response
(Liu and Tan 2012)

2.6 Summary

Currently, some building codes have incorporated design guidelines to mitigate progressive collapse, including tying capacity design, alternate load path and key element methods. Nevertheless, due to the limited research works that have been conducted on this topic, there are still many areas that need to be improved. The behaviour of steel and composite structures under column-removal scenarios has not been well studied. The previous experimental works focus on welded moment connections or in fire conditions. However, in Europe, bolted steel connections such as fin plate, flush end plate, web cleat and extended end plate, are very popular and these simple or semi-continuous joints seem to be vulnerable to progressive collapse compared with the welded moment joints. Therefore, the evaluation of these kinds of joints subjected to catenary action under column removal scenarios is important and timely. For the purpose of evaluating the joint ability to form catenary action in practical application, the component-based method is also reviewed. It can be found that more analysis works are needed to address the ductility issue of critical components of beam-column joints.

CHAPTER 3 EXPERIMENTAL TESTS OF DIFFERENT TYPES OF BOLTED STEEL BEAM-COLUMN JOINTS

3.1 Introduction

In this chapter, an experimental programme of bare steel beam-column joints under a middle-column-removal scenario will be presented. Seven experimental tests have been carried out on different types of steel beam-column joints, including simple and semi-rigid connections, at the Protective Engineering Laboratory of Nanyang Technological University. In the group of simple connections, web cleat, top and seat angle, top and seat with web angle (TSWA) (8 mm thickness angles) and fin plate connections were investigated while flush end plate, extended end plate and TSWA (12 mm thickness angles) constituted the group of semi-rigid connections. The principal aim of this chapter is to provide the experimental results of bolted steel beam-column joint behaviour, including failure modes, development of forces and deflections in the beams under a middle-column-removal scenario. The experimental results could be used to validate the numerical models. In addition, the robustness of different types of connections will also be assessed.

3.2 Test set-up and specimens

3.2.1 Test set-up

The hypothetical beam-column joint considered for experimental tests is located above the storey where an internal column has been forcibly removed. As shown in Figure 3.1, after the removal of the middle column, the internal forces and deflection of the middle and end connections are anti-symmetric. Thus, the inflection point locates at the middle of the beam span during the deflection process. Therefore, only half of the beam span is simulated using pin conditions, as shown in Figure 3.1. The behaviour of the middle and side joints, including load-carrying and rotation capacities, could be represented by the tested specimens. A numerical

simulation conducted by the author demonstrates that the simplified test could provide equivalent performance to a sub-frame test. Although the vertical deflection capacities of the simplified and sub-frame tests are different, the rotation angles and internal forces experienced by the connections are identical. The simplified specimen is representative of any other upper floors above the zone of damage, since the whole column experiences a downward rigid body displacement and the axial forces in the column above the damaged storey are very small indeed and can be neglected. Further discussions about the pin support assumption will be presented in Section 6.4.1 for bare steel joint tests and Section 7.6.1 for composite steel joint tests.

Figure 3.1 Prototype bare steel beam-column joint

It should also be noted that if the contribution of slabs is considered, the internal forces and deflection of the middle and end connections will be different. Nevertheless, the objectives of the current research work are to compare the performance of different types of connections and to validate numerical and

component models. The relative performance of different types of connections will still be valid. The validated numerical and component models could be used in the sub-frame simulations. The slab effect will be studied in the following works.

(a) Aerial view

(b) Elevation view

Figure 3.2 Test set-up of bare steel beam-column joints (unit: mm)

The test set-up is shown in Figure 3.2. Compared with other joint tests under normal loading conditions, an additional horizontal restraint was provided by an A-frame and a strong reaction wall to consider the restraint from surrounding structural elements. In order to consider the rotational restraint to beam-column joints from the continuous column of upper storeys, the test rig included a rotational restraint system at mid-span, as shown in Figures 3.2 and 3.3. The column rotation was restrained by two steel rods, which would bear against the flanges of two steel columns during testing. In addition, the beams were restrained from lateral movement by two lateral restraint systems, as shown in Figures 3.2 and 3.3.

A displacement-controlled point load was applied to the middle column using an actuator, which was attached to a strong H-frame. Load was applied under displacement control at a rate of 6 mm/min. It should be mentioned that the loading condition in reality under progressive collapse is uniformly distributed load rather than a point load used in the experimental tests. Under the point load condition, the fracture of beam-column joint indicates the collapse of the building structures. However, in reality, after the joint failure, 3D floor slab may still be able to resist the vertical loads by tensile membrane action. It means the joint fracture may be just the initiation of the structural failure and the 3D floor system still has the potential resistance to prevent progressive collapse. Therefore, the point load method can be used to investigate the behaviour of beam-column joint. However, it may not be a proper way to investigate the total structural resistance against progressive collapse, if the 3D floor resistance of tensile membrane action is also studied. Further discussions about the difference of these two loading methods will also be presented in Section 6.4.2.

(a) Rotational restraint system

(b) Lateral restraint system

Figure 3.3 Rotational and lateral restraint systems of bare steel beam-column joint tests

3.2.2 Instrumentation

Measurements of internal forces were obtained through attaching strain gauges to four sections of steel beams, as shown in Figure 3.4. The beam axial forces were estimated based on the measured strains across the beam section. Rosette strain gauges were attached onto the beam web to measure shear strains, so as to compute the internal shear forces of beams. Actuator load was also measured by an external load cell so as to verify the external and internal forces in the test assembly.

The instrumentation included linear variable differential transducers (LVDTs) and line transducers for vertical deflection and joint rotation measurements. Ten LVDTs and four line transducers were placed in each specimen as shown in Figure 3.5. One

pair of LVDTs was placed horizontally at each side of the middle beam-column joint to capture the joint rotation. The remaining LVDTs and line transducers were placed along the beam length to measure in-plane vertical deflections.

Figure 3.4 Locations of strain gauges on bare steel beam-column joint test specimen
(unit: mm)

Figure 3.5 Locations of LVDTs and line transducers on bare steel beam-column joint test specimen

3.2.3 Test specimens

A prototype steel framed building was designed for the purpose of examining its resistance against progressive collapse under column-removal scenarios. The multi-storey office building had a plan dimension of 45 m by 30 m and a bracing system to resist lateral loads. Simple and semi-rigid beam-column joints were planned to study the effectiveness of different connection details in resisting progressive collapse under predominantly gravity load based on EC1-1-1 (2002). The dead loads and live loads for a typical bay of the building consisted of 5.1 kN/m² and 3 kN/m², respectively. The design standards of members and their connections were in accordance to EC3-1-1 (2005), EC4-1-1 (2004) and some UK design recommendations (BCSA/SCI 2002, BCSA/SCI 1995, BCSA/SCI 1998). The highlighted area in Figure 3.6 indicates a sub-assembly of beams and connections that is typical of the experimental tests reported herein. In the test programme, there were two series of specimens, viz. simple and semi-rigid bare steel beam-column joints and composite steel joints with profiled decking. Due to space limitation in the laboratory, the test specimens were scaled down to 2/3 of their original size. Although the tested specimens were scaled down to be two thirds, the connection configurations including angle thickness, bolt size, bolt positions are still commonly used in practice. In the frame level, there may be some difference between the tested specimens and prototype structures. However, in the connection component level, the connection components have similar configurations with prototype beam-column joints. Thus, these test data can be used to validate the component models, which can be used in practice to predict the prototype joint performance under column-removal scenarios. This chapter only reports the behaviour of simple and semi-rigid bare steel joints subjected to catenary action.

Figure 3.6 Plan layout of the designed multi-storey office building (unit: mm)

Seven tests were carried out as shown in Table 3.1. Shear resistances and tying capacities, which were calculated based on UK design recommendations (BCSA/SCI 2002, BCSA/SCI 1995), are also listed in Table 3.1. For semi-rigid connections, moment resistance values were included as well. For both simple and semi-rigid jointed frames, two-dimensional linear elastic analysis of one slice of the building in Figure 3.6 was carried out. Only gravity load case was considered. In the design of simple connection specimens, beams were treated as simply-supported and beam-column joints could only transfer shear forces. However in the design of semi-rigid connection specimens, beams were treated as partial-continuous and beam-column joints could transfer shear forces as well as moments. Therefore, in the group of simple connection specimens, a deeper beam section of UB305×165×40 was used while in the group of semi-rigid connection specimens, the beam section changed to be UB254×146×37.

All specimens have the same length of 4208 mm with the distance between two pin supports as 4868 mm. Figure 3.7 shows the construction details of the seven test specimens. Each specimen consisted of two steel beams and a steel column. The

column cross-section in all the tests was the same, viz. Grade S355 UC203×203×71. In all the specimens, the beams and columns were strengthened by some stiffeners or welded thick plates to limit the influence of beam and column deformations on the connection behaviour. As shown in Figure 3.7, four types of joints were investigated in the simple connections: web cleat, top and seat angle, TSWA (8 mm angle) and fin plate, while in the group of semi-rigid connections, three types of connection were studied: flush end plate, extended end plate and TSWA (12 mm angle). Since the specimen of TSWA (8 mm angle) has a relatively low flexural stiffness, it was treated as a simple connection and a deeper beam section was adopted whereas the specimen of TSWA (12 mm angle) has a higher flexural stiffness, so it was treated as a semi-rigid connection. In all the tests the steel material of columns and beams was of grade S355 whereas the steel material of angles and welded plates was S275. Grade 8.8 M20 bolts were used for all specimens.

The double lines of bolts were used to avoid the failure mode of bolt fracture in shear for the specimen of TSWA (12 mm angle). The test objective of this specimen is to estimate the performance of TSWA connections. If the final failure is controlled by bolt fracture in shear, the structural behaviour of TSWA connections cannot be demonstrated. Therefore, this specimen was designed to be failed by angle fracture or bolt fracture in tension, which is the most critical component for TSWA connections. In addition, according to BCSA/SCI (2002), double lines of bolts are commonly used to connect a larger beam with a heavier vertical reaction.

Table 3.1 Summary of bare steel beam-column joint test specimens

Specimen ID	Connection type		Beam section	Column section	Bolt	End plate/angle	Shear/moment resistance *	Tying capacity *
Specimen 1	Simple connections	Web cleat	305×165×40 UB S355	203×203×71 UC S355	Grade 8.8 M20	L90×8	Shear: 124 kN	177 kN
Specimen 2		Top and seat angle	305×165×40 UB S355	203×203×71 UC S355	Grade 8.8 M20	L90×8	Shear: 152 kN	153 kN
Specimen 3		TSWA (8 mm angle)	305×165×40 UB S355	203×203×71 UC S355	Grade 8.8 M20	L90×8	Shear: 276 kN	330 kN
Specimen 4		Fin plate	305×165×40 UB S355	203×203×71 UC S355	Grade 8.8 M20	100×8	Shear: 124 kN	198 kN
Specimen 5	Semi-rigid connections	Flush end plate	254×146×37 UB S355	203×203×71 UC S355	Grade 8.8 M20	200×12	M: 53 kNm S: 332 kN	480 kN
Specimen 6		Extended end plate	254×146×37 UB S355	203×203×71 UC S355	Grade 8.8 M20	200×12	M: 76 kNm S: 516 kN	546 kN
Specimen 7		TSWA (12 mm angle)	254×146×37 UB S355	203×203×71 UC S355	Grade 8.8 M20	L150×90×12	M: 48 kNm S: 480 kN	495 kN

* Design values obtained from (Eurocode 3 2005, BCSA/SCI 2002, BCSA/SCI 1995)

(c) Specimen 3 (TSWA-8 mm angle)

(d) Specimen 4 (fin plate)

Technical drawing of the cross-section of a reinforced concrete slab with an S275JR steel plate. The drawing shows a central vertical steel plate (436x200x12) with horizontal flanges. It is reinforced with M20 8.8 bolts. Dimensions are given in mm: total width 200, plate width 136, plate thickness 12, and various spacing dimensions (55, 45, 45, 55). Vertical dimensions include 50, 40, 60, 136, 60, 40, 50.

- 45 -

(g) Specimen 7 (TSWA-12 mm angle)

Figure 3.7 Details of bare steel beam-column joint test specimens (unit: mm)

3.2.4 Material properties

Material tests were conducted to obtain the representative stress-strain curves for analysis purpose. These include tensile testing of steel angles, end plates and steel beams. The tensile test results of structural steel including 8 mm thick and 12 mm thick steel angles, 12 mm thick flush end plate, 8 mm thick fin plate and beam flange and web, are presented in Table 3.2.

Table 3.2 Tensile test results of bare steel beam-column joint test specimens

Coupon plate	Yield strength (MPa)	Ultimate strength (MPa)	Ultimate strain*
L 90×8 angle	300	455	0.27
L150×90×12 angle	280	430	0.31
12 mm thick end plate	235	409	0.36
8 mm thick fin plate	398	552	0.22
Beam flange	320	482	0.243
Beam web	343	484	0.305

* Fracture strain is based on the proportional coupon gauge length of $5.65\sqrt{S_0}$,

where S_0 is the original cross-section area of coupons.

3.3 Test results

A summary of the test results can be found in Table 3.3, from which the maximum vertical loads, the corresponding middle column displacement and rotation angles at the ends are given. The maximum horizontal reaction forces, the maximum moments and axial forces of beams and the failure modes are also included in this table. It should be mentioned that the rotation capacities of joints correspond to the maximum loads and these rotation angles are obtained by dividing the centre column displacement at the maximum load by the beam span of 2.326 m. This simplification is reasonable because plastic hinges are formed in the beam-column joints and the beam deflection profiles can be approximated by straight lines, as shown in Figures 3.10, 3.20 and 3.25. In subsequent sections, the experimental results and observations from the seven connection tests, summarized in Table 3.3, are presented.

3.3.1 Simple connections

3.3.1.1 Specimen 1—web cleat

The specimen was held at horizontal position, and a vertical load was gradually applied to the middle column. As the connection had limited capacity to resist moment, the specimen rotated at both ends with increasing deflection at the mid-span. Catenary action soon developed until fracture occurred in the central connection. Figure 3.8 shows the vertical force-middle column displacement relationship for Specimen 1. At the initial loading stage, it could not resist any load until axial tensile force was mobilised in the beam due to large deflection, which marks the beginning of catenary action. As shown in Figure 3.8, the load resisted by flexural action is quite limited. During the whole loading process, most of the load applied was resisted by catenary action. At a displacement of 367 mm, one of the web angles fractured close to the heel, which was immediately followed by the fracture of the other web angle. Figure 3.9 shows the failure mode of the web cleat connection. Two observations are evident from the failure modes and action-

vertical deformation curves: (1) catenary action was formed prior to failure in the web cleat connection and (2) the beam segments remained straight and localized large strain occurred at the connection. As shown in Figure 3.9 (c), the web angle experienced large deformation and localized yielding before fracture took place.

Table 3.3 List of test results of bare steel beam-column joints

Specimen ID	Connection type		Maximum vertical load (kN)	Displacement* (mm)	Joint rotation** (degree)	Maximum horizontal reaction (kN)	Maximum moment (kNm)	Maximum axial force of the beam N_B (kN)	Failure mode
Specimen 1	Simple connections	Web cleat	119	367	8.97	349	5.9	369	Angle fracture
Specimen 2		Top and seat angle	44.8	243	5.96	115	49.8	120	Angle fracture
Specimen 3		TSWA (8 mm angle)	113	366.4	8.95	376	60.3	380.5	Angle fracture
Specimen 4		Fin plate	77.5	227	5.57	363	10.3	365	Bolt fracture in shear
Specimen 5	Semi-rigid connections	Flush end plate	161.1	350	8.56	567	79.2	574	Bolt thread stripping
Specimen 6		Extended end plate	125.6	142	3.49	405.6	143.5	412.8	Weld fracture
Specimen 7		TSWA (12 mm angle)	229.8	398	9.71	670.7	91.4	680.4	Bolt fracture

* Displacement corresponding to the maximum load

** Joint rotation corresponding to the maximum load

Figure 3.8 Vertical force-middle column displacement curve of Specimen 1 (web cleat)

(a) Left web angle 1

(b) Left web angle 2

(c) Fractured angle

Figure 3.9 Failure mode of Specimen 1 (web cleat)

3.3.1.2 Specimen 2—top and seat angle

Significant flexural action is observed in Figure 3.10 and the fracture mode of angles is similar to the web cleat test, as shown in Figure 3.11. Top and seat angle connections are usually designed as simple connections. In conventional design, the top angle provides lateral support to the compression flange of the connecting beam, and the seat angle can only transfer vertical reaction of the beam to the column with minimal moment. However, this connection was able to transfer not only vertical reaction but also some end moment of the beam to the column. Figure 3.10 shows the force-displacement relationship for Specimen 2. At the initial loading stage, applied load did not increase linearly. This is because when shear force exceeded friction forces in the tightened bolts between the adjoining beam bottom flange and the seat angle, the bolts loosened and slipped. After yielding of the bottom angles, the stiffness of the connection increased again due to plastic deformation of seat angles. As shown in Figure 3.10 (b), at this stage, since the beam ends were horizontally restrained, the beams were in compression and bending. Figure 3.10 (c) shows that significant moment was formed in the joint at this stage, which demonstrates flexural behaviour. Flexural action developed until the left bottom angle fractured close to the heel at a displacement of 260 mm. There was a dip followed by a rise in the reaction-displacement curve in Figure 3.10 (a) until fracture of the right bottom angle occurred. It is noteworthy that catenary action was partially mobilised after fracture of the left bottom angle. Figure 3.10 shows that at the stage of catenary action, large tensile forces developed in the beams and the bending moments reversed in sign since the moment calculations were based on the beam neutral axis. Due to limited tying capacity at the second stage, the applied load could not exceed the peak load attained at the first stage. With regard to the capacity to form large deformation and catenary action, this connection has relatively limited rotation capacity compared to the web cleat connection. As shown in Figure 3.11 and Table 3.3, catenary action could not be fully mobilised and the maximum load was controlled by flexure rather than catenary action.

(a) Vertical force-middle column displacement curve

(b) Left beam axial force-middle column displacement curve

(c) Left connection moment-middle column displacement curve

Figure 3.10 Action-middle column displacement curve of Specimen 2 (top and seat angle)

(a) Failed connection

(b) Fractured left bottom angle

Figure 3.11 Failure mode of Specimen 2 (top and seat angle)

3.3.1.3 Specimen 3—TSWA (8 mm angle)

The TSWA (8 mm angle) connection has a similar behaviour in load and deformation capacities with the web cleat connection, as shown in Figure 3.12 and Table 3.3. With regard to the force-displacement history shown in Figure 3.12, there is no obvious point of demarcation between flexural and tensile phases. The end of

flexural action corresponds to the fracture of left bottom angle, as shown in Figure 3.12(c) at a displacement of 230 mm, but the onset of catenary action is defined at the instant when the beam axial force shows tension, as indicated in Figure 3.12(b). In Figure 3.12(a), the two bottom angles fractured first, after which load continued to increase to the maximum value until one of the right web angles fractured at a displacement of 366.4 mm. Ultimately, the other right web angle also fractured. When the two bottom angles fractured, bending moment could not be sustained by the joint, which indicates the end of flexural action stage, as shown in Figure 3.12(c). Different from Specimen 2, compression was not observed in the beams. Instead, a significant tensile force was formed in the beams before the fracture of bottom angles, which means catenary action co-existed with flexural action when vertical displacement ranged between 100 mm and 230 mm. As shown in Figure 3.13, the same fracture pattern of web angles with the web cleat connection in Figure 3.9 is observed.

As shown in Figure 3.7, Specimen 3 has addition top and seat angles than Specimen 1. However, Table 3.3 indicates that Specimen 3 has a slightly lower load resistance than Specimen 1. This is because for Specimen 3, at the maximum loading point, the top and seat angles cannot help to sustain the applied vertical load. As shown in Figure 3.12 (a), for Specimen 3, when the maximum load was reached, the bottom angles had fractured and only the web angles and top angles were still connected onto the middle column. Nevertheless, since this specimen has a higher beam section, at the maximum load point, most of the tensile forces were still transferred by web angles. The top angles of this specimen had little contribution and might be still in compression, which may reduce the connection load-carrying capacity. This is probably the reason why the load resistance of Specimen 3 is slightly lower than Specimen 1.

(a) Vertical force-mid-column displacement curve

(b) Left beam axial force-mid-column displacement curve

(c) Left connection moment-middle column displacement curve

Figure 3.12 Action-middle column displacement curve of Specimen 3

(TSA—8 mm angle)

(a) Failed connection

(b) Fractured web angle

Figure 3.13 Failure mode of Specimen 3 (TSA—8 mm angle)

3.3.1.4 Specimen 4—fin plate

Compared with the web cleat connection, the fin plate connection has a smaller rotation capacity. As shown in Figures 3.14 and 3.15, this connection failed by

shear fracture of bolts. The three bolts at the left connection failed one by one but there was no fracture of bolts at the right connection. At failure, the fin plate has undergone significant bearing deformations around the bolt holes. As a typical example of simple connection, at the initial loading stage, this connection could not sustain any load until flexural and catenary actions were formed at the large deflection stage. To be different with the web cleat connection, this connection has a greater bending moment resistance. However, due to limited rotation capacity of this connection, it has low capacity against vertical load.

Figure 3.14 Action-middle column displacement curve of Specimen 4 (fin plate)

(a) Failed connection (b) Deformation of fin plate (c) Bolt fractured in shear

Figure 3.15 Failure mode of Specimen 4 (fin plate)

3.3.1.5 Comparison of the simple connections

Table 3.4 shows that, if catenary action is considered under a middle-column-removal scenario, load capacity could increase significantly, especially for Specimens 1, 3 and 4. If only flexural action was considered, and if there was no horizontal restraint in the experimental tests, Specimens 1 and 4 could only sustain limited vertical loads due to their poor moment resistances. When catenary action is mobilised, large increase of load capacity is observed. Therefore, catenary action plays an important role for beam-column joints under the middle-column-removal scenario, in which sufficient horizontal restraints is provided by surrounding structural elements. It should be mentioned that the load capacities of beam-column joints without catenary action are calculated based on the bending moments measured in the steel beams. Although these are calculated values, there should be little difference with actual tests, particularly so as the connections achieved maximum moments at small displacement, with little tension developed in the beams.

The responses of simple connections subjected to catenary action are compared in Table 3.4. It is clear from Tables 3.3 and 3.4 that Specimen 1 (web cleat) has the best performance in the development of catenary action as it attains the largest vertical displacement. This type of connection could resist the highest vertical load, generate the largest vertical displacement and joint rotation, as it has an adequate rotation capacity. Another observation from Table 3.3 is that, the horizontal reaction of Specimen 1 (web cleat) is slightly less than Specimen 4 (fin plate) although the vertical load capacity of the web cleat connection is much higher. This relatively low horizontal reaction could place a lower demand on adjacent columns. Specimen 3 has a similar behaviour in load and deformation capacities with Specimen 1 although this joint used up more angles and bolts. At the first stage, this specimen sustained a much higher load than Specimen 1 due to its higher flexural stiffness. However, when bottom angles fractured, similar vertical load and axial force curves were observed in these two tests, which means the top angles were not effective at catenary action stage. Specimen 4 formed large tensile forces in beams, but due to

its limited rotation capacity, it had a low vertical load resistance. Specimen 2 has the poorest performance under catenary action. Hence, Specimen 1 (web cleat) has the best performance in resisting vertical loads. These benefits are due to its large rotation capacity, which could in turn mobilise catenary action.

Table 3.4 Comparison with and without catenary action

Specimen ID	Connection type		Load capacity considering catenary action (kN)	Load capacity without catenary action * (kN)	Load capacity increase caused by catenary action
Specimen 1	Simple connections	Web cleat	119	4.46	2568%
Specimen 2		Top and seat angle	44.8	42.53	5%
Specimen 3		TSWA (8 mm angle)	113	51.64	119%
Specimen 4		Fin plate	77.5	10.78	619%
Specimen 5	Semi-rigid connections	Flush end plate	161.1	74.16	117%
Specimen 6		Extended end plate	125.6	122.86	2%
Specimen 7		TSWA (12 mm angle)	229.8	78.32	193%

* The load capacities without catenary action are calculated based on the maximum moments of joints listed in Table 3.3.

3.3.2 Semi-rigid connections

3.3.2.1 Specimen 5—flush end plate

As shown in Figure 3.16, the force-displacement history can be divided into two stages: flexural action and catenary action. After local buckling of the top flange of the left beam, flexural moment decreased gradually and then the bottom bolt-row failed by thread stripping. Finally, failure propagated to the top two bolt-rows and subsequently, the left connection was completely severed. It should be noted that flush end plate connection is a typical type of semi-rigid connection, and therefore at the initial loading stage, vertical load increased due to its high flexural stiffness. After local buckling of the top beam flange, applied load could not increase any more until catenary action kicked in and resisted the vertical load. Thread stripping failure of top bolt-row shown in Figure 3.17(b) defines the ultimate capacity for this connection. Although this type of joint finally failed due to thread stripping of bolts, as shown in Figure 3.17, large displacement has developed due to significant bending of the end plate before failure.

Figure 3.16 Vertical force-middle column displacement curve of Specimen 5 (flush end plate)

Figure 3.18 depicts the displacement profiles of the beams at different load levels. It shows that before plastic hinge formation, the whole specimen deflected like a simply-supported beam. After local buckling had occurred, a slight middle column rotation was observed and plastic hinge became more pronounced in the deflection profiles. Beyond 69.9 kN, the displacement profile of each beam can be approximated by a straight line, which indicates the formation of plastic hinges at the beam-column connections.

(a) Failed connection

(b) Bolt thread stripping

Figure 3.17 Failure mode of Specimen 5 (flush end plate)

Figure 3.18 Middle column displacement profiles of beams corresponding to indicated vertical forces for Specimen 5 (flush end plate)

3.3.2.2 Specimen 6—extended end plate

Figure 3.19 shows the force-displacement history of Specimen 6. Extended end plate connection has the greatest flexural stiffness and moment strength among these seven specimens. Under normal loads, the top beam flange is under tension while the bottom beam flange is in compression. However, under column-removal scenarios, the moment reversed in sign. Therefore, the original compression zone is under tension. Partial-strength weld was used in the original compression zone, viz. bottom beam flanges. Hence, under positive bending moment, the bottom flange partial-strength weld at the right connection sustained a large tensile force and fractured first, as shown in Figure 3.20. After that, there was a drop in the applied load. It should be noted that the middle column rotated due to unbalanced flexural actions at two sides of the joint. Ultimately, the specimen failed completely due to bolt thread stripping failure at the left connection, as shown in Figure 3.20. This connection sustained substantial bending moment at the initial loading stage whereas at the large displacement stage, there was no further increase of loading in catenary action, unlike the flush end plate case.

Extended end plate connections have higher stiffness and flexural resistances than flush end plate connection. This phenomenon has been observed in the experimental tests. As shown in Table 3.3, the maximum moment formed by Specimen 6 (extended end plate connection) is much higher than Specimen 5 (flush end plate connection). However, Table 3.3 also indicates that Specimen 6 has a lower load resistance than Specimen 5. This is because the failure mode of Specimen 6 is weld fracture, which is a typical type of brittle failure. This brittle failure reduced the ductility of this specimen. Table 3.3 shows that the joint rotation at maximum load of Specimen 6 is much smaller than Specimen 5. Therefore, the contribution from catenary action is limited for Specimen 6, as shown in Figure 3.19.

Figure 3.19 Vertical force-middle column displacement curve of Specimen 6 (extended end plate)

Figure 3.20 Failure mode of Specimen 6 (extended end plate)

3.3.2.3 Specimen 7—TSWA (12 mm angle)

Figure 3.21 Vertical force-middle column displacement curve of Specimen 7 (TSWA-12 mm angle)

(a) Failed connection

(b) Fracture bolt

Figure 3.22 Failure mode of Specimen 7 (TSWA-12 mm angle)

In the test of Specimen 7, as shown in Figure 3.21, there is no obvious point of demarcation between flexural and tensile phases. Figure 3.21 shows that load could increase significantly before the two bottom bolts at the left connection fractured. Then the two bottom bolts at the right connection fractured, after which the load increased to the maximum value until the bolts connecting the web angles started to fracture one by one at a displacement of 398 mm. As shown in Figure 3.22 (b), a 45 degree full-slant fracture surface indicates that the bolts were in tension as well as in shear. Although this specimen failed by bolt fracture, the rotation capacity (9.71°) is

still higher than Specimen 3 (8.95°). This is due to a higher span-beam depth ratio (9.1) of this specimen compared with Specimen 3 (7.33). This conclusion demonstrates that for TSWA connections, span-beam depth ratio has a great influence on joint rotation capacity.

3.3.2.4 Comparison of the semi-rigid connections

Table 3.4 shows for semi-rigid beam-column joints, if catenary action is considered under a middle-column-removal scenario, the load capacities could increase significantly, especially for Specimens 5 and 7. If only flexural action is considered, Specimen 6 could still achieve its load capacity, which indicates that catenary action has little influence on this joint. In contrast, under catenary action the load resistances of Specimens 5 and 7 could increase more than 100% of their capacities without catenary action.

The responses of the semi-rigid connections subjected to catenary action are compared in Table 3.4 and Figure 3.23. The steel beam-column joints in these three tests went through two phases. During the first stage, the joints sustained vertical applied loads by flexural action. Specimen 6 has the greatest flexural strength. Therefore, at this stage, this specimen reached the largest loads among these three tests, as shown in Figure 3.23 (b). At the second stage when the deflection was very large, the specimens went into catenary action. Specimens 5 (flush end plate) and 7 (TSWA-12 mm) performed well under catenary action. Figure 3.23 (a) shows that at the stage of flexural action, no axial forces were developed in beams and at the second stage (catenary action), tensile forces were formed rapidly. Among these three tests, Specimen 7 had the largest load and rotation capacities. In the tests of Specimens 5 and 7, although finally the failure modes are due to bolt fracture or bolt thread stripping, there was a ductile failure process. Before failure of bolts, end plate or angles yielded and experienced high local plastic deformation and rotation, which introduced plastic hinges into the beam-column joints and consequently mobilised catenary action. Nevertheless, brittle failure was observed in Specimen 6 and this failure mode severely limited the formation of catenary action.

(a) Beam axial force-middle column displacement curves

(b) Connection moment-middle column displacement curves

Figure 3.23 Comparison of the behaviour of different semi-rigid connections

3.4 Practical implications of the experimental results

The experimental results demonstrate that under a middle-column-removal scenario, beam-column joints sustain a combination of shear, tensile forces and bending moments. However, according to the measured internal forces of beams at the maximum load values, for those joints which could develop catenary action well, tensile forces usually dominate the behaviour and the failure modes of joints. The behaviour of Specimens 1, 3, 4, 5 and 7, which could form catenary action well, is analogous to a truss system, as shown in Figure 3.24. In this system, rotation value θ and tensile resistance of connection T are obtained from the experimental results and the applied load P can be calculated based on equilibrium:

$$P = 2T\sin\theta \quad (3.1)$$

Figure 3.24 Simplified truss system

The comparison between the experimental and calculated results is shown in Table 3.5. Clearly, the calculated values agree very well with the test results with a mean of 99.8% and a small standard deviation of 6%. This indicates that after large rotations have occurred, beam-column joints are subjected to pure tension and tensile failure controls the final failure modes of these joints. Based on the experimental tests and numerical simulations, four new connection acceptance criteria of rotation capacities (θ) will be proposed to consider catenary action under a middle-column-removal scenario in Chapter 4. These connection acceptance criteria could be used in Equation (3.1) to calculate the load resistances of connections under catenary action.

Table 3.5 Comparison between experimental results and calculated values based on the simplified models

Specimen ID	Load capacity from experimental results P_E (kN)	Load capacity from calculated values P_C (kN)	P_C/P_E
Specimen 1	119	115.1	96.7%
Specimen 3	113	118.4	104.8%
Specimen 4	77.5	70.9	91.5%
Specimen 5	161.1	170.9	106.1%
Specimen 7	229.8	229.5	99.9%

Mean $P_C/P_E = 99.8\%$

Standard deviation $P_C/P_E = 0.06$

Table 3.6 Comparison of tying resistances and rotation capacities of bare steel beam-column joints between the design values and the test results

Specimen ID	Connection type		Tying resistances			Rotation capacities (degree)	
			Design values T_D	Test results T_T	T_T / T_D	Design values (DoD 2009, ASCE 2006)	Test results
Specimen 1	Simple connections	Web cleat	177 kN	369 kN	2.08	5.71	8.97
Specimen 2		Top and seat angle	153 kN	120 kN	0.78	2.01	5.96
Specimen 3		TSWA (8 mm angle)	330 kN	380.5 kN	1.15	2.01	8.95
Specimen 4		Fin plate	198 kN	365 kN	1.84	2.47	5.57
Specimen 5	Semi-rigid connections	Flush end plate	480 kN	574 kN	1.20	0.86	8.56
Specimen 6		Extended end plate	546 kN	412.8 kN	0.76	0.57	3.49
Specimen 7		TSWA (12 mm angle)	495 kN	680.4 kN	1.37	0.74	9.71

Table 3.6 indicates a comparison of tying resistance between the design values from Table 3.1 and the test results from Table 3.3. From Table 3.6, it can be concluded that certain types of connections could not develop the designed tying capacities based on (Eurocode 3-1-8 2005, BCSA/SCI 2002). In the design stage, only pure shear or pure tying forces are considered, whereas under column-removal scenarios beam-column joints sustain pure tensile forces *after undergoing large rotations*, which means the load resistances of beam-column joints are related to tying resistances as well as rotation capacities. This implies that when tying capacity design method (Eurocode 1-1-7 2006) is used to mitigate progressive collapse, tying resistances of beam-column joints should still satisfy certain values after large rotations have occurred. Therefore, enhancing tensile resistances of beam-column joints *after undergoing large rotations* is one of the most effective means to mitigate progressive collapse. In current Eurocode 1-1-7 (2006), only tying force magnitudes are required. However, the beam-column joints may not have the required rotation capacity to develop catenary action. Thus, certain types of joints have failed to achieve the design tying resistance, as shown in Table 3.6 (Specimens 2 and 6). It should also be mentioned that although the design tying resistances of Specimens 3, 5 and 7 have been met in the experiments, the ratios of P_T / P_D exceed unity by a relatively small margin compared to those of Specimens 1 and 4. Therefore, for these types of connections, the influence of rotations should be considered in the design stage.

Based on the discussion, Equation (3.2) is proposed to replace the expression of tying resistance in the design code (Eurocode 1-1-7, 2006):

$$T_E \leq T_\theta \sin \theta \quad (3.2)$$

where T_E is the tying requirements in (Eurocode 1-1-7 2006),

T_θ is beam-column joint tensile resistance after undergoing rotation θ ,

θ is the rotation capacity of connection.

Table 3.6 also summarizes the rotation capacities based on the tests conducted in this study and on the recommended values by (DoD 2009, ASCE 2006). The design values of the rotation capacities are obtained from Table 5-2 (DoD 2009) and Table

5-6 (ASCE 2006), as well as the failure modes observed in the tests. As Table 3.6 indicates, the rotations at peak loads based on the experimental results in this study were much higher than the recommended values by (DoD 2009, ASCE 2006). In the current design codes (GSA 2003, DoD 2009) against progressive collapse, there are no experimental results available for bolted steel beam-column joints, and the recommended values are based on seismic testing data. However, under a middle-column-removal scenario, connections sustain monotonic loads rather than seismic actions. Under seismic loading, the connections are under pure flexure and cyclic loading leading to significant degradation in the strength and ductility. In contrast, under catenary action, the connections are subjected to pure tension after undergoing large rotations. Hence, seismic testing data may not be appropriate for column-removal scenarios and these experimental test results may be adopted as the failure criteria in the design codes.

3.5 Summary and conclusions

This chapter presents the descriptions and experimental results of seven tests on steel beam-column joints subjected to catenary action. The following conclusions can be drawn:

- (1) The test specimens revealed various modes of fracture in the connection regions, ductility of different failure modes and rotation capacities of various types of connections under a middle-column-removal scenario;
- (2) The test results demonstrate the contribution of catenary action to the load resistances of various types of steel beam-column joints and conclude that under the middle-column-removal scenario, catenary action can be considered to increase the load capacity;
- (3) For web cleat and fin plate connections, under the middle-column-removal scenario, only a limited load could be applied onto the joints at the initial loading stage and the applied load increased significantly at the large deformation stage due to catenary action. Therefore, the behaviour of these two joints was dominated by catenary action. For top and seat angle and extended end plate connections, flexural action dominates the behaviour of the joints in

the early stage of the response. There was little load contribution from catenary action. For flush end plate and TSWA (8 and 12 mm angle) connections, significant flexural actions were observed and catenary action could also develop well at large deformation stage;

- (4) In the group of simple connections, the web cleat connection has the best combination of desirable features: ability to develop catenary action and extremely high ductility (rotational capacity) through deformation of the web angles. The TSWA (8 mm angle) connection has a similar behaviour with the web cleat connection. In the group of semi-rigid connections, TSWA (12 mm angle) achieved the highest load and rotation capacities whereas flush end plate could also mobilize catenary action well;
- (5) By comparing Specimens 3 and 7, it can be seen that for the connection type of TSWA, span-beam depth ratio has a great influence on the rotation capacities of connections;
- (6) It is worthy to note that tensile capacities of beam-column joints after undergoing large rotations usually control the failure mode and the formation of catenary action. This implies that engineers should adopt high tensile resistances of beam-column joints after undergoing large rotations rather than pure tying resistance. A new tying resistance expression is proposed to consider the effect of large rotation. If large rotation is not considered in the design stage, the joints with poor rotation capacities would fail to achieve the design tying resistances;
- (7) The rotations of beam-column joints at peak loads based on the experimental results in this study were much higher than the recommended values by (DoD 2009, ASCE 2006). Since the recommended values are based on seismic testing data, the experimental test results may be adopted as the failure criteria in the analyses of progressive collapse.

CHAPTER 4 NUMERICAL ANALYSES OF STEEL BEAM-COLUMN JOINTS

4.1 Introduction

The beam-column joint tests presented in Chapter 3 provide experimental data to calibrate numerical simulation models, which can then be used to perform parametric study so as to develop more effective joint details to reduce the risk of progressive collapse. These tests have been modelled using the general finite element (FE) package ABAQUS. Both static and explicit dynamic solvers were employed to overcome the problems of convergence, contact, large deformation and fracture simulations. In this chapter, the key aspects of applying 3D solid continuous elements are presented and further parametric studies are conducted to obtain the rotation capacities of various types of connections under catenary action.

4.2 Finite element simulation

Several simulations of steel connections subjected to catenary action under fire or column-removal scenarios have been reported previously. Yu et al. (2008c) explored the use of an explicit dynamic solver to analyse bolted steel connections in order to overcome convergence difficulties in static solvers. By controlling the time step, it was shown to be possible to produce quasi-static responses. Yu et al. (2011, 2009b) used this method to model flush end plate and web cleat connections under tying forces in fire. The explicit dynamic solver has also been adopted by Sadek et al. (2010), Karns et al. (2009) and Khandelwal and El-Tawil (2007) to investigate the behaviour of steel beam-column welded joints subjected to catenary action under column-removal scenarios. The great challenge in modelling progressive collapse is to simulate extensive fractures in various structural components. Material fractures have been successfully simulated by Sadek et al. (2010), Karns et al. (2009) and Khandelwal and El-Tawil (2007) by introducing fracture criterion. However, the explicit dynamic solver requires huge computational resources and

often produces unstable structural behaviour (Dai et al. 2010). Dai et al. (2010) and Elsayaf et al. (2011) employed an implicit static solver to study the behaviour of bolted steel beam-column joints in fire. Great strides have been made to overcome convergence problem and to achieve numerical stability. However, material fractures were not simulated in their numerical models.

Clearly, the behaviour of beam-column joints subjected to catenary action is three dimensional in nature. Modelling such a behaviour involves material and geometric non-linearities, large deformations, contact phenomena and fracture simulations. Firstly, six numerical models were created for six types of conventional connections using ABAQUS (2009) finite element software and validated by the experimental results presented in Chapter 3. Furthermore, parametric studies were conducted based on these validated models. Solid element type C3D8R, which has the capability of representing large deformations, geometric and material nonlinearities and fracture simulations, was employed for steel joints.

4.2.1 Solution strategy

When running simulation analysis, numerical convergence difficulty usually occurs when there are too many contact pairs. However, in bolted steel beam-column joints, the contact phenomenon must be addressed. The convergence problem becomes even more acute at the stage of fracture simulation.

In this research, both static and explicit dynamic solvers were used to conduct numerical simulations. A static solver is commonly used in structural analyses. However, significant efforts are required to make the procedure convergent when contact phenomenon is simulated in the model (Yu et al. 2008c). It becomes even more challenging for the static solver to converge when modelling fracture simulations as well as contact phenomenon. An explicit dynamic solver was also used in the following simulations because it can overcome numerical convergence difficulties associated with complex contact problems and fracture simulations. Compared with the static analyses, the explicit dynamic solver allows easy pre-

processing and provides an easy solution procedure (Yu et al. 2008c). Although the explicit dynamic solver is unable to provide the static response, the dynamic effect could be ignored if the kinetic energy of the deformed structure is only a fraction (typically 5% and 10%) of its internal energy throughout most of the structural response (ABAQUS 2009). Therefore, a quasi-static simulation was also used in the following analyses. However, for some cases since huge computational resources are required by the explicit dynamic solver (Dai et al. 2010), only static analyses are conducted.

A “surface-to-surface” contact interaction was developed for the numerical models. The friction condition was represented by isotropic Coulomb friction model. In the tangential direction, a friction coefficient of 0.3 is used. Since the bolt shank diameter is smaller than the bolt hole diameter, bolt slip can also be simulated in the numerical analyses. In this contact interaction, master-and-slave surfaces need to be defined, and the surface of the stiffer body or with the coarser mesh (if the two surfaces are on members with comparable stiffness) would be chosen as the master surface (ABAQUS 2009). Half of the central column section should be modelled, but in this simulation, only the outer column flange bolted to the connection is modelled, assuming rigid column web panel at the centre. This simplification is reasonable because no column web deformation was observed after the experimental tests. This is justified on the basis of the relatively low stiffness of the connection compared to the high in-plane stiffness of the column web.

4.2.2 Material properties

A study of the material properties with single angle members has been carried out by Madugula et al. (1997). In this study, it was observed that yield strength of the steel within the heel region of the angle tended to be substantially higher than that the steel in the remainder of the angle. In the current study, material properties of the steel close to the heel region and the remainder of the angle have been experimentally determined and the obtained engineering stress-strain curves are shown in Figure 4.1. Figure 4.1 shows that the yield strength of Coupon 1 located at

the angle heel region is 350 MPa while the yield strength of Coupon 2 located close to the angle edge is 300 MPa only. In addition, the fracture strain of Coupon 1 is 0.23 while the fracture strain of Coupon 2 is 0.27. It can be seen that the angle steel within the heel region has a higher yield strength and lower ductility than the remainder region. The tested material properties have been employed in the numerical models.

Figure 4.1 Engineering stress-strain curves of the tested angles

Most available material test data are in the form of engineering stress and strain, but to simulate the connection behaviour more realistically, the material model has to be defined by true stress and strain relationship in the finite element analyses. Therefore, it is necessary to convert material data from engineering stress and strain to true stress and strain. The following equations can be used for the conversion:

$$\sigma_{true} = \sigma_{eng} (1 + \epsilon_{eng}) ; \quad \epsilon_{true} = \ln(1 + \epsilon_{eng}) \quad (4.1)$$

In ductile crystalline materials like steel, fracture initiation can be triggered off by plasticity. Ductile fracture will occur after the material undergoes extensive necking and plastic deformation in the necked region.

The ductile criterion in ABAQUS (2009) is a phenomenological model for predicting the onset of damage due to nucleation, growth, and coalescence of voids. The model assumes the damage starts when the specified equivalent plastic strain at the onset of damage is reached (ABAQUS 2009). The damage evolution, which is characterized by the progressive degradation of the material stiffness, was also defined. Once the fracture strain is reached, the damaged elements will be deleted from the mesh. Since triaxial test data for current steel materials are not available, the triaxial effect could not be captured by the current fracture criterion. Therefore, the fracture strain defined in the ductile criterion is independent of hydrostatic stress, although many research works (Hopperstad et al. 2003, Brünig et al. 2008, Mirone 2007) have shown that the ductility of steel depends considerably on the stress triaxiality. Thus, model parameters required for ductile criterion that have been calibrated to a particular experiment cannot be considered as material properties and may not be able to accurately simulate ductile fracture in situations which differ largely from the original experiment used for model calibration. Therefore for different experimental tests, different model parameters are used. It should be mentioned that this fracture criterion is dependent on mesh size as well (ABAQUS 2009).

4.2.3 Finite element modelling

Only one quarter model is simulated due to symmetry for all the steel bolted connections except for the fin plate connections, which are simulated by one-half models. The column section is represented by half of the flange only, as shown in Figure 4.2. The other end of the beam is supported by a pinned connection. In order to reduce the number of elements and nodes in the finite element models, the steel beam was divided into two parts and only the central part connected to the joint was actually modelled using solid elements, as shown in Figure 4.2 (a). The other part away from the joint zone was modelled using general beam elements. The ABAQUS “Coupling” function was used to join the two parts (Dai et al. 2010). During the analysis procedure, the column flange was “pushed” vertically downwards using displacement control until failure occurred at the joint. A series of

mesh studies was conducted to assess the sensitivity of simulation results of different FE mesh sizes. The result of this sensitivity study suggested that the appropriate mesh size should be 5 mm for bolts, angles or end plates, and 10 mm for the beam and column elements. Bolt thread was not modelled directly because its modelling would be extremely time consuming. Due to difficulty in measuring the initial stress in bolts, no pre-stress was applied to the bolts. An M20 bolt was simulated by an equivalent diameter based on the effective cross-sectional area of one bolt.

(a) FE model

(b) Connection part

Figure 4.2 Typical FE model adopted in numerical simulations

In static simulations, in order to overcome the convergence problem caused by temporary instabilities of rigid body motions in contact and fracture problems, an artificial contact damping was specified by “contact controls”. In order to ensure

that an appropriate contact damping is used, the ratio of the dissipated energy by damping (parameter ALLSD in ABAQUS) to the total strain energy (parameter ALLIE in ABAQUS) should be less than the specified dissipated energy fraction (Dai et al. 2010).

Quasi-static analyses could be performed by the explicit dynamic solver for complicated contact conditions with fracture simulations. In the current models, in order to minimize the dynamic effect, a constant velocity was applied onto the middle column by using “smooth step” (ABAQUS 2009).

4.2.4 Comparison between finite element models and test results

4.2.4.1 Web cleat connection test

In the experimental test of this specimen, the failure mode is the fracture of angle close to the heel. In this connection simulation, both the static and explicit dynamic solvers have been used to simulate the test procedure. Comparison of the predicted failure modes with the experimental observation is shown in Figure 4.3. It is found that the explicit dynamic solver could simulate the fracture initiation and evolution until complete failure of the connected angle, while the static solver could only simulate the fracture initiation and certain crack propagation. Nevertheless, the solver was able to attain the peak load when the analysis was terminated due to excessive cracking. Thus, the static solver could also predict the failure mode and load-deformation curve well. As shown in Figure 4.4, the predicted load-displacement curves agree well with the experimental curve. It is also worthy to note that the explicit dynamic solver gives almost the same curve with the static solver, although the latter is more numerically stable than the former. A numerical analysis without fracture criteria has also been conducted and the results shown in Figure 4.4 indicate that this model predicted a much higher load-carrying capacity than the experimental data.

Figure 4.3 Failure modes of FE simulations and experiment of the web cleat connection

Figure 4.4 Comparison of the FE simulations to the test data of web cleat connection

4.2.4.2 Fin plate connection test

In the experimental test, this connection failed by shear fracture of bolts. In the numerical simulations using either the static or explicit dynamic solvers, the same failure mode was observed, as shown in Figure 4.5. Figure 4.6 compares simulated and measured vertical force versus middle column displacement until the bottom bolt fractured. Clearly, it demonstrates close agreement between the simulation and the test result.

Figure 4.5 Failure modes of FE simulations and experiment of the fin plate connection

4.2.4.3 TSWA (8 mm) connection test

The TSWA (8 mm) connection has a similar behaviour in load-carrying and deformation capacities and the same failure mode with the web cleat connection. Since the static solver could not capture the complete fracture of angles due to convergence problem, only the explicit dynamic simulation was conducted for this connection. Figure 4.7 indicates the same failure pattern between the simulation and

the experimental test. As shown in Figure 4.8, after the fracture of the bottom angle, dynamic effect is observed. However when the middle column was further displaced downwards, the dynamic effect vanished due to increased stiffness. Figure 4.8 shows close agreement between simulation and the test results.

Figure 4.6 Comparison of the FE simulations to the test data of fin plate connection

(a) Explicit dynamic solver (b) Experimental test

Figure 4.7 Failure modes of FE simulation and experiment of the TSWA (8 mm) connection

Figure 4.8 Comparison of the FE simulations to the test data of TSWA (8 mm) connection

4.2.4.4 Flush end plate connection test

(a) Explicit dynamic solver (b) Static solver (c) Experimental test

Figure 4.9 Failure modes of FE simulation and experiment of the flush end plate connection

This connection failed by thread stripping of bolts, as shown in Figure 4.9. Both the static and explicit dynamic solvers have been used to conduct the numerical simulations. It should be mentioned that the two ends of bolts were connected by springs with limited stiffness to constrain free rigid body movement after bolts have fractured. Otherwise, non-convergence would occur. The predicted vertical force-middle column displacement curve by the static solver matches the experimental

results very well as shown in Figure 4.10. It could also be seen that the explicit dynamic solver predicted a slightly smaller elastic stiffness and a greater peak load than the experimental result and the static solver.

Figure 4.10 Comparison of the FE simulations to the test data of flush end plate connection

4.2.4.5 Extended end plate connection test

This connection failed by weld fracture in the test. Since the heat affected zone (HAZ) near the welds can significantly alter local material properties resulting in a local reduction in ductility, the HAZ effect is considered by locally reducing the damage initiation strain and damage evolution process. As shown in Figure 4.11, the explicit dynamic solver could simulate weld fracture process while in the static simulation, no weld fracture was observed due to convergence problem. Figure 4.12 shows that some discrepancies exist between the predicted and measured load-displacement curves. The predicted elastic stiffness of this test is greater than the experimental test. This is probably because in the numerical model, residual stress was not considered, which may change the elastic responses significantly. However, residual stress does not affect the ultimate bending moment resistance. Thus, at

plastic response stage, the prediction agrees well with the test data, as shown in Figure 4.12. In addition, the static solver predicted the load-displacement curve better than the explicit dynamic solver.

(a) Explicit dynamic solver (b) Static solver (c) Experimental test

Figure 4.11 Failure modes of FE simulations and experiment of the extended end plate connection

Figure 4.12 Comparison of the FE simulations to the test data of extended end plate connection

4.2.4.6 TSWA (12 mm) connection test

This connection failed by bolt fracture in the experimental test. Due to the huge computation resources required by the explicit dynamic solver for the model, only the static solver was employed. As shown in Figure 4.13, the same failure mode

was observed in the numerical simulation. Figure 4.14 shows that the predicted results were slightly different from the experimental observation at the initial loading stage. This may be attributed to the pre-stress in the bolts, which was not considered in the numerical model. In addition, the numerical model does not fully reflect actual configuration. The effective diameter of bolts (16.67 mm in diameter) used in the numerical model may affect the initial response of the system since bolt holes were modelled using a standard hole size of 22 mm in diameter. In other words, there was a nearly 3 mm gap instead of the usual 1 mm tolerance.

Figure 4.13 Failure modes of FE simulation and experiment of the TSWA (12 mm)

Figure 4.14 Comparison of the FE simulations to the test data of TSWA (12 mm) connection

Figure 4.15 Effect of connection depth for web cleat connections

4.2.5.2 Fin plate connections

The static solver was used to conduct the parametric study for this type of connections. Figure 4.16 quantifies the effect of connection depth on the performance of fin plate connections. It is clear from Figure 4.16 that the load-carrying capacity increases by using more bolts. However if the bolt rows were to increase from 5 to 6, loading could only increase by 4%; if the bolt rows were to increase from 3 to 4, the load-carrying capacity could increase by 28%. It indicates that the effect of connection depth decreases when too many bolt rows are used. This is probably because with the increase of connection depth, the contribution of catenary action becomes less due to the decreased rotation capacities. As shown in Figure 4.16, the vertical displacement corresponding to peak loads decreases with an increase in connection depth, which means the rotation capacities of connections reduce when connection depth increases.

Figure 4.16 Effect of connection depth for fin plate connections

4.2.5.3 Flush end plate connections

Figure 4.17 Effect of bolt rows for flush end plate connections

Figure 4.17 compares the effect of bolt rows. In the model with 4 bolt rows, a deeper beam section was adopted. It is very clear that when the bolt rows increase

from 3 to 4, the increase of the first peak load is due to the enhanced flexural resistance. Nevertheless, at the stage of large deformation (catenary action), when 4 bolt rows are used, the second peak load could only increase by 21% compared with the first peak load, whereas when 3 bolt rows are used, the second peak load could increase by 106%. This is caused by the higher rotation capacity of the 3 bolt rows connection compared to 4 bolt rows. In the model with 2 bolt rows, the same beam section and connection configuration were adopted with 3 bolt rows and the middle bolt was not used in this model. Figure 4.17 indicates that under the condition of the same beam section, the bolt located at the centroid of beam section will improve the resistances of beam-column joints under catenary action.

Figure 4.18 Effect of bolt arrangement for flush end plate connections

The connection configuration plays an important role under normal loading conditions. In order to investigate the effect of this parameter under the loading condition of catenary action, two additional models have been established. In the first model, the bolts were moved 20 mm towards the beam flanges, whereas in the second model, the bolts were moved 20 mm towards the centroid of beam section. The results of both simulations are shown in Figure 4.18, along with the response of

the original configuration. In the second case, all the bolts were close to the centroid of the beam section, resulting in a more ductile and stronger connection at the stage of large deformation.

4.2.5.4 TSWA (12 mm) connections

For this type of connections, more bolt rows may be used in the web angles. Therefore four cases with 2, 3, 4 and 5 bolt rows connecting web angles are considered. Figure 4.19 shows that deeper connections are substantially less ductile than shallower connections. In addition, deeper connections have higher load-carrying capacities before bottom bolts fractured. After the bottom bolts have fractured, the connections with 4 and 5 bolt rows cannot sustain further loads. It is interesting to notice that the connection with 2 bolt rows connecting web angles could sustain a higher load than the one with 4 bolt rows. This indicates that a deeper connection may not perform better under column-removal scenarios.

Figure 4.19 Effect of connection depth for TSWA (12 mm) connections

4.3 Practical implications of the simulation results

The simulation results demonstrate that under a middle-column-removal scenario, increasing the number of bolt rows increases the load-carrying capacity and rotation stiffness. However, it is also observed that connection ductility is adversely affected by the increase of bolt rows, which means this effect is not always beneficial. In addition, the simulation results indicate that connection configurations play an important role to form catenary action. For example, for flush end plate connections, if the bolts were moved towards the centroid of beam section, more ductile and higher load-carrying capacity of connections at large deformation stage was observed. However, this also caused a significant reduction of moment capacity.

Figure 4.20 Rotation capacity comparison between the current simulation results and DoD (2009) for web cleat connections

Figure 4.20 shows the rotation capacity comparison between the current simulation results and DoD (2009) for web cleat connections. The numerical simulation results show much higher rotation capacities of web cleat connections than the rotation capacities specified by steel connection nonlinear acceptance criteria in (DoD 2009). Table 4.1 summarizes the rotation capacities of web cleat, fin plate, flush end plate and TSWA (12 mm) connections. For flush end plate and TSWA (12 mm) connections, the rotation capacities are not given in DoD (2009), therefore the specification of ASCE 41 (2006) is adopted in the table for these two types of connections. It should be mentioned that all the rotation criteria from (DoD 2009)

and (ASCE 41 2006) are plastic rotations while the current simulation results include elastic rotations. However it is believed that elastic rotation angles are quite small compared with final rotation capacities. It can be seen that for these three types of connections, the rotation capacities obtained from the current numerical simulations are much higher than those from (DoD 2009) and (ASCE 41 2006). Thus, these code values are probably too conservative.

Similar conclusions have been drawn by other researchers (Sadek et al. 2010, Kim et al. 2011). The nonlinear acceptance criteria of connections specified in DoD (2009) and ASCE 41 (2006) are based on the experimental tests of beam-column joints under seismic loads. Thus, the rotation capacities of beam-column joints obtained from monotonic loading tests are much greater than the rotation capacities obtained from cyclic tests (Kim et al. 2011). In addition, only pure flexural action contributes to the connection resistances against seismic loading, while under a middle-column-removal scenario, flexural action as well as catenary action could resist the applied monotonic loading. Figure 4.21 shows the force-joint rotation angle curves of the flush end plate connection. It can be found that flexural action dominates the applied load at the initial loading stage. However when the joint rotates further, catenary action starts to be formed and flexural action decreases significantly. Therefore without catenary action, the rotation capacity of this connection is only about 0.05 radian while under catenary action, the joint could reach as high as 0.15 radian, a three-fold increase in rotation capacity, as shown in Figure 4.21.

Table 4.1 Comparison of connection rotation capacities between the current simulation results and DoD (2009) /ASCE 41 (2006)

Connection type	DoD (2009) /ASCE 41 (2006) (radians)	Current simulation results (radians)
Web cleat	$\theta = 0.1125 - 0.0027h_{bg}^{(1)}$	$\theta = 0.1745 - 0.0033h_{bg}$
Fin plate	$\theta = 0.0502 - 0.0015h_{bg}$	$\theta = 0.1172 - 0.0034h_{bg}$
Flush end plate	0.015	0.15
TSWA (12 mm)	0.035	$\theta = 0.214 - 0.0077h_B^{(2)}$

(1) h_{bg} = depth of bolt group, inch

(2) h_B = depth of beam, inch

Figure 4.21 Force-joint rotation angle curves of the flush end plate connection

Thus, from Figure 4.21, if the connection acceptance criteria of rotation capacities from ASCE 41 (2006) is adopted, the load-carrying capacity of this connection is only about 62 kN; whereas if the proposed connection acceptance criteria is used, the load-carrying capacity could reach as high as 160 kN. The increased load resistance is mainly contributed by catenary action. This demonstrates that the proposed connection acceptance criteria of rotation capacities should consider the contribution of catenary action.

4.4 Summary and conclusions

This chapter describes the development of a group of numerical models to investigate the behaviour of bolted steel beam-column joints subjected to catenary action under a middle-column-removal scenario. In this research, both the static and explicit dynamic solvers were used to conduct numerical simulations. The predictions of the finite element models have been validated with experimental results. It is shown that the finite element models correlate well with test results including the observed failure modes. As a result of the success of applying finite element analyses, parametric studies have been conducted using these numerical models to obtain the rotation capacities of various types of connections under catenary action. The following conclusions are drawn:

- (1) A static solver could usually predict more accurate simulation results than an explicit dynamic solver, although for some cases almost the same predictions have been observed for both solvers;
- (2) The problem of numerical non-convergence will occur when the static solver is employed and certain fracture simulations could not run to the end while complete fracture simulations could be conducted by the explicit dynamic solver. However, huge computation resources are required by the explicit dynamic solver for complicated models;
- (3) Connection depth has a significant influence on the behaviour of beam-column joints subjected to catenary action under column-removal scenarios;
- (4) By improving the arrangement of bolts in configurations of flush end plate connections, significant increase of load-carrying and rotation capacities has been achieved;
- (5) Under middle-column-removal scenarios, current acceptance criteria of rotation capacities for steel joints such as web cleat, fin plate, flush end plate and TSWA connections, are probably too conservative as they only consider pure flexural resistance;
- (6) Four new connection acceptance criterion of rotation capacities have been proposed to consider catenary action under a middle-column-removal scenario.

CHAPTER 5 MECHANICAL MODELLING OF BOLTED- ANGLE CONNECTIONS UNDER TENSION

5.1 Introduction

The experimental tests presented in Chapter 3 indicated that bolted-angle connections normally performed better than other types of connections in the development of catenary action. This is due to extremely high ductility (rotational capacity) through deformation of the connected angles. The experimental tests have also shown that after large rotations have occurred, beam-column joints are subjected to pure tension and tensile failure controls the final failure modes of these joints. The research works of Yu et al. (2009a and 2009b) also demonstrated that at the stage of large deformation, fracture of the connection components determined the ultimate resistances of joints. Thus, it can be concluded that the behaviour of beam-column joints under pure tension governs the structural resistances against progressive collapse. Therefore, the behaviour of beam-column joints under pure tension should be well understood in order to assess the robustness of steel buildings against progressive collapse. In addition, the failure criteria to determine the deformation capacities of beam-column joints under tension should be articulated and quantified to predict the ultimate resistances of joints at large deformation stage. However, traditional design of connections is limited to moment-rotation characteristics and the information about the deformation capacities of connection components is quite limited. Thus, it is necessary to develop a new mechanical model, which could predict the fracture of each connection component and the development of catenary action. In this chapter, a component-based model will be developed to predict the behaviour of bolted-angle connections under pure tension. In this model, the deformation capacities of connection components can be predicted. Although in reality, beam-column joints were subjected to pure tension after large rotation, it is believed that the most critical connection component, which has the largest axial deformation, will control

the final failure. The deformation capacity of this critical component can be predicted by the proposed model.

An experimental investigation of bolted-angle connections under tension will be presented first and a new mechanical model will be developed. The experimental tests have been conducted until the complete fracture of the connections occurs. The new mechanical model includes the interaction between the angles and the bolts, and considers the failure mode of bolt fracture. In addition, a new failure criterion to determine the deformation capacities of bolted-angle connections is proposed. Finally, this developed model will be validated by the conducted experimental tests.

5.2 Test programme

5.2.1 Test specimens

Type A

Figure 5.1 Component test specimen configurations

In total, fourteen bolted-angle connections were tested under tension. A summary of the test specimens is given in Figure 5.1 and Table 5.1. In the specimen design, the parameters studied include gauge length m , angle thickness t , bolt size and material properties. Figure 5.1 illustrates the three connection configurations utilised (A, B and C). In Type A and B, only Grade 8.8 M20 bolts were used while in Type C,

Grade 8.8 M24 bolts were used in horizontal legs and Grade 8.8 M20 bolts in vertical legs. Type B and C specimens were designed to avoid the failure mode of bolts in shear. This is because if this failure mode occurs, the interaction between the angles in bending and the bolts in tension could not be observed or measured in the experiments. Type B differs from Type C in the bolt size and gauge length m . The dimensions of m and t shown in Figure 5.1, are given in Table 5.1. The other geometric properties are also given in Table 5.1. The average values of the angle thickness measured by digital callipers are also summarized in Table 5.1. The test specimens consisted of four angles. The bolt holes in the angles and thick steel plates were 2 mm oversize. To focus on the behaviour of angles and bolts, very thick steel plates were used to connect the bolted angles. The reference used for the specimens follows the nomenclature $XL-t-m-e$ for equal angles and $XL_1-L_2-t-m-e$ for unequal angles, where X represents the configuration type (A, B and C with reference to Figure 5.1), L is the equal angle leg length in mm, L_1 is the longer leg length of unequal angle in mm, L_2 is the shorter leg length of unequal angle in mm, t is the nominal angle thickness in mm, m is the gauge length between the centrelines of horizontal bolts and the vertical loading plate in mm, and e represents the material type of steel angles. For instance, in Specimen A90-8-50- I , the configuration is Type A, the equal angle leg length is 90 mm, the nominal angle thickness is 8 mm, the gauge length between the centrelines of horizontal bolts and the vertical loading plate is 50 mm and the material type is I . The material properties of steel angles obtained from coupon tests are given in Table 5.2. All the bolts were hand-tightened. The strength grade of all the bolts was Grade 8.8 and came from one supplier. Therefore it is assumed that all the bolts have the same material properties.

Figure 5.2 Test set-up of bolted-angle component tests

5.2.2 Test procedure

The specimens were subjected to a monotonic tensile force, which was applied to vertical steel plates that were clamped to the testing machine, as shown in Figure 5.2. The tests were carried out under displacement control at a speed of 0.025 mm/s up to failure of the specimens.

Table 5.1 Geometrical properties of component test specimens

Specimen	Connection type	Angle size	Angle thickness t (mm)	Dimension m (mm)	Material Type*
A90-8-50- I	A	90×8	7.88	50	I
A90-8-60- I	A	90×8	7.88	60	I
A90-8-40- I	A	90×8	7.88	40	I
A90-9-50- II	A	90×9	8.79	50	II
A90-9-50-III	A	90×9	8.98	50	III
A90-9-60-III	A	90×9	8.98	60	III
A90-9-40-III	A	90×9	8.98	40	III
A90-10-50-IV	A	90×10	9.85	50	IV
A90-10-60-IV	A	90×10	9.85	60	IV
B150-90-11-50- V	B	150×90×11	10.62	50	V
B150-90-11-60- V	B	150×90×11	10.62	60	V
B150-90-11-40- V	B	150×90×11	10.62	40	V
C125-11-75-VI	C	125×11	10.84	75	VI
C125-11-60-VI	C	125×11	10.84	60	VI

* The material properties are given in Table 5.2.

Table 5.2 Material properties of the component test angles

Material Type	Yield strength f_y (MPa)	Ultimate strength f_u (MPa)	Fracture strain ε_u^*
I	323	473	0.317
II	345	496	0.253
III	261	426	0.321
IV	328	430	0.31
V	281	425	0.305
VI	280	430	0.304

* Fracture strain is based on the proportional coupon gauge length of $5.65\sqrt{S_0}$, where S_0 is the original cross-section area of coupons.

5.2.3 Experimental results

The experimental results are summarized in Table 5.3 and discussions will focus on: deformation history, failure modes and the effects of some design parameters, such as gauge length m , angle thickness t , bolt size and material properties.

5.2.3.1 Deformation history

Figure 5.3 Load-displacement curve of Specimen A90-8-50- I

Table 5.3 Summary of the component test results

Specimen	Maximum load (kN)	Maximum displacement (mm)	Failure mode
A90-8-50- I	546	64	Angle fracture at bolt holes (Figure 5.4)
A90-8-60- I	520	82	Angle fracture at bolt holes (Figure 5.4)
A90-8-40- I	629	58	Angle fracture at bolt holes with yielded bolts (Figure 5.6)
A90-9-50- II	549	52	Angle fracture close to heel with yielded bolts (Figure 5.7)
A90-9-50-III	513	51	Angle fracture close to heel (Figure 5.5)
A90-9-60-III	498	67	Angle fracture close to heel (Figure 5.5)
A90-9-40-III	576	39	Angle fracture close to heel with yielded bolts (Figure 5.7)
A90-10-50-IV	696	66	Bolt fracture with yielded angles (Figure 5.8)
A90-10-60-IV	620	75	Angle fracture close to heel with yielded bolts (Figure 5.7)
B150-90-11-50- V	688	59	Angle fracture close to heel with yielded bolts (Figure 5.7)
B150-90-11-60- V	699	80	Angle fracture close to heel with yielded bolts (Figure 5.7)
B150-90-11-40- V	729	44	Angle fracture close to heel with yielded bolts (Figure 5.7)
C125-11-75-VI	533	66	Angle fracture close to heel (Figure 5.5)
C125-11-60-VI	733	65	Angle fracture close to heel (Figure 5.5)

The deformation history of all specimens had five stages: (1) elastic stage before bolt slippage, (2) bolt slippage stage, (3) elastic stage after bolt slippage, (4) transition stage, and (5) mechanism deformation stage. Figure 5.3 shows the load-displacement curve of Specimen A90-8-50- I . The curve from Point *0* to *A* represents the first stage. Pretension load was applied onto the bolts during the

hand-tightening process. During the first stage, applied load was transferred through friction caused by pretension of bolts at the vertical legs. When the applied load overcame the bolt friction slip resistance, bolt slippage occurred and the deformation curve went into the second stage, from Point *A* to *B* as shown in Figure 5.3. During the second stage, the deformation continued to increase without any significant increase in load. When the gap between the bolt shanks and the holes was closed, the deformation history went into the third stage, from *B* to *C* as shown in Figure 5.3. During this stage, the applied load was transferred directly through the contact between the bolt shanks and the holes rather than through friction. At Point *C* of Figure 5.3, the bolted angles yielded and the load-displacement curve went into the fourth stage (transition stage). At Point *D*, a mechanism was formed. At the last stage (mechanism deformation stage), the increased load was due to material hardening and geometric nonlinearity. It can be seen from Figure 5.3 that the load increase at the last stage is much higher than the yield strength, which was reached at Point *C* on Figure 5.3. From this observation, it can be concluded that the load-displacement responses of bolted-angle connections were dominated by large deformation rather than the yield strength of angles.

5.2.3.2 Failure modes

The failure mode of every specimen is given in Table 5.3. Totally, there are five types of failure modes observed in the tests. All five failure modes are shown in Figures 5.4, 5.5, 5.6, 5.7 and 5.8, respectively. The first failure mode (angle fracture at bolt holes) was observed in Specimens A90-8-50- I and A90-8-60- I . The same failure mode was observed in the experimental tests of Yu et al. (2009a). Failure mode of angle fracture close to the heel (shown in Figure 5.5) was observed in many tests, as indicated in Table 5.3. It is found that these two failure modes would occur when thin angles or larger diameter bolts were used. Figures 5.6 and 5.7 show the failure modes of angle fracture with yielded bolts. When the angles became slightly thicker or the bolt diameters were smaller, significant plastic deformations of the bolts took place. It means that the failure mode also depends on the strength

ratio between the angles and the bolts. Figure 5.8 shows bolt fracture with yielded angles in Specimen A90-10-50-IV.

Figure 5.4 Angle fracture at bolt holes (A90-8-50- I)

Figure 5.5 Angle fracture close to heel (A90-9-60-III)

Figure 5.6 Angle fracture at bolt holes with yielded bolts (A90-8-40- I)

Figure 5.7 Angle fracture close to heel with yielded bolts (A90-10-60-IV)

Figure 5.8 Bolt fracture with yielded angles (A90-10-50-IV)

In the tests, it was also observed that the strength ratio between angles and bolts would also influence the plastic hinge locations formed in angles, as shown in Figure 5.9. For flexible angle legs, plastic hinges moved closer to angle heel as shown in Figure 5.9 (a), while for relatively stiff angles, plastic hinges tended to move to angle edge as shown in Figure 5.9 (c). For those angles between these two, the plastic hinges were formed around the bolt centrelines, as shown in Figure 5.9 (b). This effect will be discussed in Section 5.3.2.3.

Figure 5.9 Interaction between angles and bolts

5.2.3.3 Design parameters

The design parameters in the experiments included gauge length m , angle thickness t , bolt size and material properties. The effects of these parameters on the connection behaviour will be discussed below. The deformation stiffness, loading and deformation capacities and failure modes, will be addressed.

5.2.3.3.1 Gauge length m

Since Material types I and IV had similar properties, the load-displacement curves of the specimens using these two materials, are compared in Figure 5.10 (a). This is also the reason that Material types III and V are compared together in Figure 5.10 (b). The effect of gauge length m is shown in Figures 5.10 (a), (b) and (c). An increase of gauge length m decreased the loading capacity and stiffness at large deformation stage. In addition, failure modes were also affected by gauge length m . For instance, for Specimens A90-10-50-IV and A90-10-60-IV, when m increased from 50 mm to 60 mm, the failure mode changed from bolt fracture with yielded angles to angle fracture close to heel with yielded bolts. The same phenomenon was observed for Material types I and III. In terms of ductility, Figures 5.10 (a) and (b) show that an increase of gauge length m increased the deformation capacities. Thus, it can be concluded that the deformation capacities of bolted-angle connections are affected by gauge length m .

5.2.3.3.2 Angle thickness t

As expected, the angle thickness had a direct influence on the response of connections, which can be observed from Figures 5.10 (a) and (b) by comparing the curves which have the same gauge length m . An increase in the angle thickness t results in a higher stiffness and loading capacity. In addition, similar to the effect of gauge length m , an increase of angle thickness t also changes the failure mode. Specimen A90-8-60- I had similar material properties with Specimen A90-10-60-IV. The only geometric difference between these two specimens was the angle

thickness. However, the failure modes of these two tests were quite different. When the angle thickness increased from 8 to 10 mm, plastic deformation of the bolts was observed after the test. Figures 5.10 (a) and (b) show that deformation capacities of bolt-angle connections are affected by angle thickness t . It is found from Figure 5.10 (b) that an increase of angle thickness raised the deformation capacities. The same phenomenon was observed in the comparison between Specimens A90-8-50-I and A90-10-50-IV, as shown in Figure 5.10 (a). On the other hand, as shown in Figure 5.10 (a), for Specimens A90-8-60-I and A90-10-60-IV, an increase in the angle thickness actually decreased the deformation capacity, which is probably due to the interaction between angles and bolts. It means that angle thickness as well as the interaction between angles and bolts could both affect the deformation capacities of bolted-angle connections.

5.2.3.3.3 Bolt size

Specimens B150-90-11-60-V and C125-11-60-VI demonstrated the effect of bolt size on the behaviour of bolted-angle connections. Two bolt diameters were considered, namely, M20 and M24. Figure 5.10 (c) shows that an increase of bolt diameter increased the deformation stiffness, but the overall deformation capacity actually decreased. Due to a greater deformation capacity of Specimen B150-90-11-60-V, contribution from geometric nonlinearity to the loading resistance is greater. This is the reason that there is only a slight increase of loading resistance when the bolt diameter changed from M20 to M24.

5.2.3.3.4 Material properties

Specimens A90-9-50-II and A90-9-50-III had the same geometric configurations. The only difference was the material type. As indicated in Table 5.2, Material type II used in Specimen A90-9-50-II had different properties from Material type III, which was used in Specimen A90-9-50-III. Material type II had higher strength while type III had higher ductility. However, as shown in Figure 5.10 (d), the obtained load-displacement curves of these two tests are quite similar. This

indicates that ductile material could delay failure and enhance the overall performance of bolted-angle connections at large deformation stage.

(a)

(b)

(c)

(d)

Figure 5.10 The load-displacement relationships of the tested specimens

5.3 Development of a mechanical model of bolted-angle connections

The experimental results presented in the Section 5.2 provide essential data for the development and validation of a new mechanical model of bolted-angle connections, which will be discussed in this section.

5.3.1 General

The experimental results indicate that the load-displacement responses of bolted-angle connections were governed by large deformation. The increase in load after angle yielding was much larger than the yield strength. This is due to the developed tensile forces in angles and material hardening effect. In addition, the fracture of angles or bolts determined the ultimate load capacities of bolted-angle connections. The test results demonstrated that the strength ratio between angles and bolts affected the failure mode. Therefore, the developed component-based model should consider the following effects: (i) tensile membrane action and material hardening effect of angles at large deformation stage, (ii) failure criteria to determine the deformation capacities of bolted-angle connections, and (iii) interaction between angles and bolts.

5.3.2 Development of the mechanical model

5.3.2.1 Small deformation response

Many research works have been conducted to study the stiffness and loading capacities of bolted-angle connections at small deformation stage. Eurocode 3-1-8 (2005) treats the component of angles in bending as an equivalent bolted T-stub. An elastic-plastic model is proposed for this component. The design resistance of T-stub in tension $F_{T,Rd}$ can be computed as the smallest value among three possible failure modes, as shown in Figure 5.11.

Figure 5.11 Three failure modes of T-stubs in tension

Mode 1: Complete yielding of T-stub flange

$$F_{T,1,Rd} = \frac{4M_a}{m_a} \quad (5.1)$$

Mode 2: Bolt failure with yielding of T-stub flange

$$F_{T,2,Rd} = \frac{2M_a + n_a \sum B_{T,Rd}}{m_a + n_a} \quad (5.2)$$

Mode 3: Bolt failure

$$F_{T,3,Rd} = \sum B_{T,Rd} \quad (5.3)$$

where M_a is the plastic moment capacity of the T-stub flange section, $B_{T,Rd}$ is the bolt design resistance, m_a is the distance between two plastic hinges, and n_a is the distance between the bolt centrelines and the prying force position.

The value of m_a recommended by Eurocode 3-1-8 (2005) is given as

$$m_{a,EC3} = m - t_a - 0.8r_a \text{ for } g \leq 0.4t_a \quad (5.4)$$

$$m_{a,EC3} = m - 0.5r_a \text{ for } g > 0.4t_a \quad (5.5)$$

where m is the gauge length shown in Figure 5.1, t_a is the angle thickness, r_a is the radius of the angle fillet, and g is the gap between the beam web and the column flange (Figure 5.12). These two equations consider the plastic hinge locations which are affected by the gap g .

Figure 5.12 Gap between the beam web and the column flange

From the experimental tests carried out by Azizinamini (1987, 1989), Faella et al. (2000) proposed the following equation to calculate the value of m_a

$$m_{a,Faella} = m_{a,EC3} - \psi_1 \left(\frac{d_{bh}}{2} + \frac{t_a}{2} + 0.2r_a \right) \quad (5.6)$$

where d_{bh} is the bolt head diameter and ψ_1 is a coefficient given by

$$0 \leq \psi_1 = 1.89 - 3.22 \left(\frac{t_a}{d_b \sqrt{m_{a,EC3} / d_b}} \right) \leq 1 \quad (5.7)$$

where d_b is the bolt diameter.

Equations (5.6) and (5.7) have been adopted in the proposed mechanical model to calculate the resistances of bolted-angle connections at small deformation stage.

Faella et al. (2000) proposed the following equation to calculate the initial stiffness of angles in bending

$$K_a = E \frac{0.5b_{eff,a}t_a^3}{(m-t_a-0.8r_a)^3} \left(\frac{4\gamma_a}{4\gamma_a+3} \right) \quad (5.8)$$

where $b_{eff,a}$ is the effective width of the bolted angles and γ_a is calculated by

$$\gamma_a = \frac{I_2 / L_2}{I_1 / L_1} \quad (5.9)$$

where I_1 , I_2 , L_1 , and L_2 are shown in Figure 5.13. The criterion to calculate the value of $b_{eff,a}$ has been proposed by Faella et al. (2000).

Figure 5.13 Mechanical model of angles in bending

Equations (5.8) and (5.9) are used to calculate the initial stiffness of bolted-angle connections in the proposed model.

5.3.2.2 Large deformation response

The experimental tests indicate that once a mechanism was formed, the load-displacement curves went into large deformation stage. At this stage, the applied load increased significantly due to material hardening and geometric nonlinearity. During this stage, tensile membrane action may develop gradually in angles. However, it is very difficult to identify the accurate tensile force formed at each step. Thus, a constant value of the tensile force was assumed in the proposed model. In addition, in order to include the effect of material hardening, the ultimate strength of angles rather than the yield strength was used in the proposed model. As shown in Figures 5.4 and 5.6, the experimental results demonstrate the net cross-section area controlled the tensile capacities of angles. Therefore, the tensile forces of angles at large deformation stage are calculated as

$$N_a = (b_{eff,a} - nd_{b,hole})t_a f_u \quad (5.10)$$

where n is the number of bolts, $d_{b,hole}$ is the bolt hole diameter and f_u is the ultimate strength of angles.

Figure 5.14 Mechanical model of bolted-angles at large deformation stage

In reality the tensile forces increase gradually rather than maintaining a constant value as proposed in Equation (5.10). Hence, the proposed tensile force will only be used to calculate the load increment to reduce the calculation error due to this assumption. Therefore, at large deformation stage, the load resistance will be calculated based on an incremental method. Displacement δ is increased by a small amount Δ in each step. Each step uses the value of F from the previous step, and this value is updated again at the end of this step for subsequent steps. For instance, as shown in Figure 5.14, the increment of the applied load from Step 1 to Step 2 can be calculated as

$$\Delta F = F_2 - F_1 = N_a (\sin \theta_2 - \sin \theta_1) = N_a \left(\frac{\delta_1 + \Delta}{\sqrt{m^2 + (\delta_1 + \Delta)^2}} - \frac{\delta_1}{\sqrt{m^2 + \delta_1^2}} \right) \quad (5.11)$$

Figure 5.15 Outward movement of the angle heel

In this model a horizontal restraint is considered at the angle heel to simplify the mechanical model. However, the experimental tests demonstrated that outward movement occurred at the angle heel, as shown in Figure 5.15. In order to consider this effect, m is taken as the horizontal length of angle leg instead of L_l . The validations presented below also indicate that the use of m can give a better fit with test results.

5.3.2.3 Deformation capacities of bolted-angle connections

BCSA/SCI. (2002) proposed the following equation to determine the deformation capacity δ_u of bolted-angle connections

$$\delta_u = \frac{2.6L}{t_a} \times \frac{m}{60} \leq 30mm \quad (5.12)$$

where L is the length of the horizontal leg. This equation is too simple to give an accurate prediction. As indicated in the experimental results, many parameters, such as gauge length m , bolt size and material properties, are not considered in this model. Shen and Astanteh-Asl (2000) proposed the following equation to calculate the ultimate deformation of bolted-angle connections:

$$\delta_u = 2(m-t_a)\varepsilon_u \sqrt{\frac{t_a}{(m-t_a)\varepsilon_u}} \quad (5.13)$$

$$\delta_u = \frac{(L+m)}{2} \tan(\varepsilon_u) \quad (5.14)$$

Equation (5.13) applies to the failure mode of angle fracture and Equation (5.14) applies to the failure mode of bolt fracture with yielded angles. Nevertheless, these two equations had been derived based on cyclic tests. They may not predict the response of bolted-angle connections well under monotonic loading conditions.

The strength ratio between angles and bolts will influence the plastic hinge locations formed in angles, which has been shown in Figure 5.9. Normally it is assumed that the plastic hinges of angles formed around the bolt centrelines (Eurocode 3-1-8 2005, BCSCA/SCI. 2002, Shen and Astanteh-Asl 2000). For more flexible angle legs, plastic hinges may be located closer to the angle heel (Yu et al. 2009b, Málaga-Chuquitaype and Elghazouli 2010), while for relatively stiff angles, plastic hinges tend to move to the angle edge (Shen and Astanteh-Asl 2000). This phenomenon was observed in the experimental tests presented above. In order to consider the interaction between angles and bolts, the following relationship has been obtained to calculate the distance between the two plastic hinges at the horizontal legs of angles based on this series of experimental results:

$$m^* = m - t_a - 0.8r_a + \eta d_b \quad (5.15)$$

where η is a coefficient related to the strength ratio between the axial resistance of bolts F_b and the yield strength of angle legs F_g .

$$\begin{aligned} \text{If } \frac{F_b}{F_g} < 560, \quad \eta &= 1.1335 - 0.00242 \frac{F_b}{F_g} \\ \text{If } \frac{F_b}{F_g} \geq 560, \quad \eta &= -0.22 \end{aligned} \quad (5.16)$$

$$\text{and } F_b = f_{y,b} A_b, \quad F_g = \frac{F_{T,Rd}}{b_{eff,a}} \quad (5.17)$$

where $f_{y,b}$ is the yield strength of bolts and A_b is the tensile stress area of bolts. $F_{T,Rd}$ should be calculated according to Faella's model (Equation (5.6)).

At the failure stage of the specimens, the deformation of angle horizontal legs is shown in Figure 5.16. Since at this moment the angle is under very large tensile strain, the tensile elongation of the horizontal leg is assumed to be $m^* \times \varepsilon_u$. Therefore, at ultimate state, total length of the deformed horizontal leg is $m^*(1 + \varepsilon_u)$, as shown in Figure 5.16. The ultimate displacement δ_u can be calculated as

$$\delta_u = m^*(1 + \varepsilon_u) \sin \theta = m^*(1 + \varepsilon_u) \sin \left(\frac{\varepsilon_u m^*}{2t_a} \right) \quad (5.18)$$

The expression of θ in Equation (5.18) is simplified from a complex formula (Piluso et al. 2001) and the validations presented below demonstrate that no great errors are created by this simplification.

Figure 5.16 Ultimate deformation status of the angle horizontal leg

Once the ultimate displacement δ_u is obtained, the ultimate resistance can be determined in the incremental process, which is presented in Section 5.3.2.2.

5.3.2.4 Bolt slippage

Bolt slippage was observed in the tests. Therefore, a special bolt slippage component is included in the developed model. No deformation is assumed until the applied force overcomes the slip friction resistance in the bolt slippage component. The bolt slippage friction resistance is calculated as

$$F_s = nc_f p_{pf} \quad (5.19)$$

where c_f is the friction coefficient and p_{pf} is the bolt pretension force applied during the tightening process. The two values ($c_f=0.2$, $p_{pf}=40 \text{ kN}$) recommended by Málaga-Chuquitaype and Elghazouli (2010), are used in this model. Since the bolt holes in the angles and thick steel plates were 2 mm oversize, the gap between bolt shank and hole is assumed to be 1 mm in one bolt hole. There are two bolt holes in a bolted-angle connection. One is at the angle and one is at the connected element. Therefore, the bolt slippage model shown in Figure 5.17 is suggested in the proposed model. When the applied force is smaller than the friction resistance, the stiffness of this component is infinite. Once the force overcomes the friction resistance, the stiffness reduces to zero. However, when the gap between the bolt shanks and the holes is close, the stiffness of this component will become infinite again.

Figure 5.17 Bolt slippage model

5.3.2.5 Load limits due to bolt fracture

As observed in the test of Specimen A90-10-50-4, bolt fracture may control the loading resistances of bolted-angle connections. However, all the previous equations are based on the assumption that the final failure is controlled by angle fracture. Therefore, it is necessary to propose a load limit which can provide the loading resistance of the bolt fracture failure mode. As shown in Figures 5.11 (a) and (b), due to prying action Q , the applied load is always smaller than the ultimate loading resistance of bolts $B_{T,Rd}$. In the current model, the ultimate loads should be limited by

$$F_u \leq \frac{2t_a}{(2t_a + 0.4m^*)} \times \frac{b_{eff,a}}{66.67} \times B_{T,Rd} \quad (5.20)$$

Blass (2011) proposed a calculation equation, which was similar to Equation (5.20), to consider prying action on bolted-angle connections. However, in Equation (5.20), the interaction effect between angles and bolts, has been included.

5.3.2.6 Development of the mechanical model

Figure 5.18 Proposed mechanical model of bolted-angle connections under tension

A general load-displacement relationship of bolted-angle connections predicted by the proposed mechanical model, is shown in Figure 5.18. The value of $F_{T, Rd}$ and K_1 can be calculated based on the discussion in Section 5.3.2.1. According to Faella et al. (2000), the value of K_2 was recommended to be $1/7K_1$. Equation (5.19) determines the resistance F_S in the bolt slippage model. The load-displacement curve at large deformation stage ($F_{T, Rd} < F < F_u$) can be calculated according to the discussion in Section 5.3.2.2 and the ultimate deformation can be calculated by Equation (5.18). An incremental method is used for the calculation at this stage. It should be mentioned that the stiffness value (K_3) is not constant and may change with increasing deformation. Finally, the obtained ultimate load should satisfy the requirement of Equation (5.20), which defines the failure mode of bolt fracture.

Figure 5.19 presents the comparisons of the load-displacement curves predicted by the proposed mechanical model and the experimental results. In addition, the predictions of Astaneh-Asl's model (Shen and Astaneh-Asl 2000) are also shown in these figures. It can be seen that the proposed model predictions are in close agreement with the experimental results. The loading resistances predicted by the proposed model, Eurocode 3-1-8 (2005), Faella's model (Faella et al. 2000) and Astaneh-Asl's model (Shen and Astaneh-Asl 2000) are compared in Table 5.4. It should be mentioned that the loading resistances predicted by Eurocode 3-1-8 (2005) were only about 18% of the experimental results. This is because Eurocode 3-1-8 (2005) calculates the connection resistances based on the angle yield strength. However, for bolted-angle connections, post-yielding behaviour contributes much greater resistance than the yield strength because of tensile membrane action formed in the angles at large deflection. The deformation capacities predicted by the proposed model, BCSA/SCI model (BCSA/SCI 2002) and Astaneh-Asl's model (Shen and Astaneh-Asl 2000) are summarized in Table 5.5. Overall speaking, the proposed model gives the best prediction among all the models. The predicted loading resistances and deformation capacities of Astaneh-Asl's model were always smaller than the test results. This is probably because Astaneh-Asl's model was derived based on the cyclic loading tests. Due to the effect of fatigue, ductility of connections is not as good as the connections under monotonic loading condition.

The proposed mechanical model was developed based on the assumption that the final failure mode was controlled by angle fracture. Thus, a load limit was set to consider the failure mode of bolt fracture. Equation (5.20) presents this limit. The calculated values from this equation are given in Table 5.6. In addition, the predicted loading resistances controlled by bolt fracture with yielded angles, which were calculated by Eurocode 3-1-8 (2005), Astaneh-Asl's model (Shen and Astaneh-Asl 2000) and Blass's model (Blass 2011), are also summarized in Table 5.6. It indicates that the predicted load limits ($F_{P,l}$) by Equation (5.20) are always larger than the test results (T). This conclusion agrees well with the test observations. In the experiments, Specimen A90-10-50-IV failed by bolt fracture and the value of $F_{P,l}/T$ is 1.05, which means the prediction was very close to the test result. The loading resistances calculated by Eurocode 3-1-8 (2005), Astaneh-Asl's model (Shen and Astaneh-Asl 2000) and Blass's model (Blass 2011) do not agree well with the test results, as shown in Table 5.6.

The failure modes predicted by the proposed mechanical model are also compared with the test observations in Table 5.7. In the mechanical model, the failure modes depend on the strength ratio between angles and bolts, which is related to the coefficient of η . If the value of η calculated by Equation (5.16) is less than 0.2, the failure mode is angle fracture. However, if the η is greater than 0.2, it is assumed that the failure mode changes to angle fracture with yielded bolts. If the ultimate loads are determined by Equation (5.20), the failure mode should also change to bolt fracture with yielded angles. It can be found from Table 5.7 that in most cases, the proposed mechanical model could predict correct failure modes.

(1) A90-8-50- I

(2) A90-8-60- I

(3) A90-8-40- I

(4) A90-9-50- II

(5) A90-9-50-III

(6) A90-9-60-III

(7) A90-9-40-III

(8) A90-10-50-IV

(9) A90-10-60-IV

(10) B150-90-11-50- V

(11) B150-90-11-60- V

(12) B150-90-11-40- V

(13) C125-11-75-VI

(14) C125-11-60-VI

Figure 5.19 Comparison of the proposed mechanical model and Astaneh-Asl's model against the test results

Table 5.4 Comparison between the predicted ultimate loads and the test results

Specimen	Test results F_T (kN)	Proposed model		Eurocode 3-1-8		Faella's model		Astaneh-Asl's model	
		F_P (kN)	F_P / F_T	F_E (kN)	F_E / F_T	F_F (kN)	F_F / F_T	F_A (kN)	F_A / F_T
A90-8-50- I	546	580	1.06	87	0.16	164	0.30	446	0.82
A90-8-60- I	520	500	0.96	72	0.14	116	0.22	408	0.79
A90-8-40- I	629	697	1.11	111	0.18	251	0.40	496	0.79
A90-9-50- II	549	646	1.18	117	0.21	220	0.40	511	0.93
A90-9-50-III	513	575	1.12	92	0.18	238	0.46	439	0.86
A90-9-60-III	498	515	1.03	76	0.15	243	0.49	403	0.81
A90-9-40-III	576	627	1.09	119	0.21	187	0.32	488	0.85
A90-10-50-IV	696	745	1.07	141	0.20	169	0.24	624	0.90
A90-10-60-IV	620	721	1.16	116	0.19	126	0.20	572	0.92
B150-90-11-50- V	688	685	1.00	142	0.21	226	0.33	594	0.86
B150-90-11-60- V	699	692	0.99	116	0.17	178	0.25	544	0.78
B150-90-11-40- V	729	677	0.93	183	0.25	296	0.41	660	0.91
C125-11-75-VI	533	564	1.06	95	0.18	157	0.29	501	0.94
C125-11-60-VI	733	662	0.90	121	0.16	213	0.29	558	0.76
Mean			1.05		0.18		0.33		0.85
Standard deviation			0.08		0.03		0.09		0.06

Table 5.5 Comparison between the predicted deformation capacities and the test results

Specimen	Test results $\delta_{u,T}$ (mm)	Proposed model		BCSA/SCI model		Astaneh-Asl's model	
		$\delta_{u,P}$ (mm)	$\delta_{u,P}/\delta_{u,T}$	$\delta_{u,B}$ (mm)	$\delta_{u,B}/\delta_{u,T}$	$\delta_{u,A}$ (mm)	$\delta_{u,A}/\delta_{u,T}$
A90-8-50- I	64	63.3	0.94	49.5	0.77	41.0	0.64
A90-8-60- I	82	68.6	0.88	59.4	0.72	45.6	0.56
A90-8-40- I	58	56.5	0.90	39.6	0.68	35.8	0.62
A90-9-50- II	52	55.9	1.02	44.4	0.85	38.3	0.74
A90-9-50-III	51	57.2	1.05	43.4	0.85	43.5	0.85
A90-9-60-III	67	64.7	0.94	52.1	0.78	48.5	0.72
A90-9-40-III	39	45.7	1.07	34.7	0.89	37.8	0.97
A90-10-50-IV	66	66.4	0.93	39.6	0.60	44.3	0.67
A90-10-60-IV	75	84.2	1.06	47.5	0.63	49.5	0.66
B150-90-11-50- V	59	52.7	0.86	36.7	0.62	45.2	0.77
B150-90-11-60- V	80	70.8	0.84	44.1	0.55	50.6	0.63
B150-90-11-40- V	44	36.7	0.80	29.4	0.67	39.0	0.89
C125-11-75-VI	66	68.1	1.20	60.0	0.91	58.2	0.88
C125-11-60-VI	65	58.6	0.85	60.0	0.92	50.9	0.78
Mean			0.95		0.75		0.74
Standard deviation			0.11		0.12		0.12

Table 5.6 Comparison between the predicted load limits due to bolt fracture and the test results

Specimen	Test results F_T (kN)	Proposed model		Eurocode 3-1-8		Astaneh-Asl's model		Blass's model	
		$F_{P,l}$ (kN)	$F_{P,l}/F_T$	$F_{E,l}$ (kN)	$F_{E,l}/F_T$	$F_{A,l}$ (kN)	$F_{A,l}/F_T$	$F_{B,l}$ (kN)	$F_{B,l}/F_T$
A90-8-50- I	546	698	1.28	609	1.12	1119	2.05	611	1.12
A90-8-60- I	520	664	1.28	462	0.89	1053	2.02	532	1.02
A90-8-40- I	629	722	1.15	756	1.20	1160	1.84	716	1.14
A90-9-50- II	549	696	1.27	616	1.12	1053	1.92	655	1.19
A90-9-50-III	513	744	1.45	610	1.19	1110	2.16	664	1.29
A90-9-60-III	498	716	1.44	463	0.93	1039	2.09	582	1.17
A90-9-40-III	576	786	1.36	757	1.31	1152	2.00	772	1.34
A90-10-50-IV	696	729	1.05	622	0.89	1002	1.44	702	1.01
A90-10-60-IV	620	683	1.10	475	0.77	896	1.44	619	1.00
B150-90-11-50- V	688	772	1.12	622	0.90	1003	1.46	739	1.07
B150-90-11-60- V	699	724	1.04	475	0.68	897	1.28	653	0.93
B150-90-11-40- V	729	832	1.14	769	1.05	1066	1.46	851	1.17
C125-11-75-VI	533	993	1.86	770	1.44	1057	1.98	540	1.01
C125-11-60-VI	733	1082	1.48	990	1.35	1118	1.52	629	0.86

Table 5.7 Comparison between the predicted failure modes and the test observations

Specimen	Test observations	Proposed model
A90-8-50- I	Angle fracture	Angle fracture
A90-8-60- I	Angle fracture	Angle fracture
A90-8-40- I	Angle fracture with yielded bolts	Angle fracture with yielded bolts
A90-9-50- II	Angle fracture with yielded bolts	Angle fracture with yielded bolts
A90-9-50-III	Angle fracture	Angle fracture
A90-9-60-III	Angle fracture	Angle fracture
A90-9-40-III	Angle fracture with yielded bolts	Angle fracture with yielded bolts
A90-10-50-IV	Bolt fracture with yielded angles	Angle fracture with yielded bolts
A90-10-60-IV	Angle fracture with yielded bolts	Angle fracture with yielded bolts
B150-90-11-50- V	Angle fracture with yielded bolts	Angle fracture with yielded bolts
B150-90-11-60- V	Angle fracture with yielded bolts	Angle fracture with yielded bolts
B150-90-11-40- V	Angle fracture with yielded bolts	Angle fracture with yielded bolts
C125-11-75-VI	Angle fracture	Angle fracture
C125-11-60-VI	Angle fracture	Angle fracture

5.4 Conclusions

A mechanical model of bolted-angle connections has been developed based on the experimental tests. The test results were used to validate the proposed mechanical model. The model has included the effect of geometric nonlinearity. It is found that the predictions of this model agree well with the fourteen bolted-angle connection test results. The following conclusions are drawn from this study:

- (1) The effects of some design parameters, such as gauge length m , angle thickness t , bolt size and material properties, are identified from the experimental results;
- (2) The test results indicate that the ultimate strength of bolted-angle connections is much greater than the yield strength and the load-displacement responses of bolted-angle connections were dominated by large deformation rather than by yield capacities of bolts or angles;
- (3) Depending on the strength ratio between angles and bolts, five types of failure modes were observed in the experimental tests;
- (4) A new relationship has been concluded to calculate the distance between the two plastic hinges at the horizontal legs of the angles, which was affected by the interaction between angles and bolts;

- (5) A new equation has been proposed to calculate the deformation capacities of bolted-angle connections under tension, and the validations indicate that the predicted values have better agreement than other models against the test results;
- (6) Based on the proposed distance between the two plastic hinges at the horizontal legs of the angles, a new equation of load limits due to bolt fracture is proposed, and a better agreement than other models was found in the validations;
- (7) Overall speaking, the proposed mechanical model gives the best prediction among all the considered models.

CHAPTER 6 EXPERIMENTAL TESTS OF BOLTED-ANGLE BEAM-COLUMN JOINTS AND DEVELOPMENT OF COMPONENT-BASED MODELS

6.1 Introduction

Most previous tests of bolted-angle connections stopped once a connection component yielded or the connection rotation angle reached certain values without any fracture (Azizinamini and Radziminski 1987, Azizinamini and Radziminski 1989, Kishi and Chen 1987, Chen et al. 1988a, Chen et al. 1988b, Elghazouli et al. 2009, Yuan and Tan 2011a, Yuan and Tan 2011b). However, the test results presented in Chapter 5 indicated that the behaviour of bolted-angle connections was dominated by large deformation. The load increase at large deformation stage was much higher than the yield strength. The experimental tests presented in Chapter 3 indicated that bolted-angle connections normally performed better than other types of connections in the development of catenary action. The experimental tests presented in Chapter 3 have also shown that bolted-angle connections could sustain substantial tensile forces at large deformation stage and fractures of the connection components determined the ultimate resistances of joints.

In order to achieve a better understanding on the performance of bolted-angle connections subjected to catenary action under a middle-column-removal scenario, nine experimental tests were carried out on bolted-angle beam-column joints. Three types of connections, including web cleat, top and seat angle and TSWA connections were investigated using three angle thicknesses (8, 10 and 12 mm). One of the principal aims of this chapter is to provide the experimental results of bolted-angle beam-column joints under middle-column-removal scenarios, including the failure modes, development of forces and deflections in the beams. In addition, based on the mechanical model presented in Chapter 5, a component-based model is developed to predict the behaviour of bolted-angle beam-column joints up to total

failure under middle-column-removal scenarios. Finally, frame analyses are conducted based on the developed component-based model.

6.2 Test set-up and specimens

6.2.1 Test set-up

Figure 6.1 Prototype bolted-angle beam-column joint

A detailed description of the test set-up and instrumentation is given in Chapter 3, so only a brief overview will be presented here. As shown in Figure 6.1, after the removal of the middle column, the internal forces and deflections of the *middle* and the *side* connections are anti-symmetric about the middle of the beam span. Thus, the inflection point is located at the middle of the beam span during the deflection process. Therefore, only one-half of the beam span on both sides is simulated using pin conditions, as shown in Figure 6.1. The behaviour of the *middle* and the *side* connections, including load-carrying and rotation capacities, can be represented by

the specimens. The test set-up is shown in Figure 6.2. Further discussions about the pin support assumption will be presented in Section 6.4.1 for bare steel joint tests and Section 7.6.1 for composite steel joint tests. Compared with the other joint tests under normal loading conditions, an additional horizontal restraint was provided by an A-frame on one end and a strong reaction wall on the other end to simulate the restraint from surrounding structural elements. It is found from the experimental tests that the axial restraint stiffness that can be provided by the A-frame and the strong reaction wall is 117 kN/mm. It is also found that for a typical building, when a middle column is removed, the axial restraint stiffness provided by the surrounding beams, columns and slabs can be as large as 400 kN/mm (Yu 2012). Certainly, if a edge or corner column is removed, the axial restraint stiffness can be zero. Thus, it can be seen that the axial restraint stiffness provided by the A-frame and the strong reaction wall is a typical restraint stiffness under a middle-column-removal scenario. A displacement-controlled point load was applied to the middle column using an actuator, which was attached to a rectangular portal frame. Load was applied under displacement control at a rate of 6 mm/min. Measurements of internal forces were obtained through attaching strain gauges to four sections of the steel beams, as shown in Figure 6.2 (c). The deflection instrumentation included linear variable differential transducers and line transducers for vertical deflection and joint rotation measurements.

6.2.2 Test specimens

Nine tests were carried out as shown in Table 6.1. The connection type, angle size and material type, are listed in this table. The reference used for the specimens follows the nomenclature $X-t$ where X represents the connection type (W represents web cleat, TS represents top and seat and $TSWA$ represents top and seat with web angles) and t is the angle thickness in mm. For instance, in Specimen W-8, the connection type is *web cleat*, and the angle thickness is 8 mm. All the connection details are shown in Figure 6.3. It should be mentioned that the test of Specimen TSWA-12 has been reported in Chapter 3. For convenience of comparison, this test

is also listed in this table. A UB254×146×37 section was used for beams and a UC203×203×71 section for columns.

(a) Aerial view

(b) Elevation view

(c) Locations of four sections to be attached strain gauges on test specimen

Figure 6.2 Test set-up of bolted-angle beam-column joints (unit: mm)

Table 6.1 Summary of bolted-angle joint specimens

Specimen	Connection type	Angle size	Angle material type*
W-8	Web cleat	L90×8	I
W-10	Web cleat	L90×10	II
W-12	Web cleat	L150×90×12	III
TS-8	Top and seat angle	L90×8	I
TS-10	Top and seat angle	L90×10	II
TS-12	Top and seat angle	L150×90×12	III
TSWA-8	TSWA	L90×8	I
TSWA-10	TSWA	L90×10	II
TSWA-12	TSWA	L150×90×12	III

* The material properties are given in Table 6.2.

Table 6.2 Material properties of the tested angles

Material Type	Yield strength f_y (MPa)	Ultimate strength f_u (MPa)	Fracture strain ϵ_u *
I	331	484	0.31
II	308	455	0.25
III	280	430	0.31

* Fracture strain is based on the proportional coupon gauge length of $5.65 \sqrt{S_0}$,

where S_0 is the original cross-sectional area of coupons.

(a) W-8(10)

(b) TS-8(10)

(c) TSWA-8(10)

(d) W-12

(e) TS-12

(f) TSWA-12

Figure 6.3 Bolted-angle joint specimen configurations

All specimens have the same length of 4208 mm with the distance between the two pin supports maintained at 4868 mm (Figure 6.1). Figure 6.3 shows the connection details of the nine test specimens. Three types of bolted-angle connections were investigated: web cleat, top and seat angle and TSWA. In all the tests the steel

material of columns and beams was of grade S355, whereas the steel material of angles was S275. Grade 8.8 M20 bolts were used for all specimens. The material properties obtained from coupon tests are given in Table 6.2. All the bolts were hand-tightened.

6.2.3 Test results

A summary of the test results is shown in Table 6.3, from which the maximum vertical loads, the corresponding middle column displacement and the rotation angles are given. The maximum horizontal reaction forces, the maximum moments and axial forces of beams and the failure modes are also included in this table. It should be mentioned that the rotation capacities of joints correspond to the maximum loads and these rotation angles are obtained by dividing the centre column displacement at the maximum load by the beam net span of 2.326m shown in Figure 6.2 (b). This simplification is reasonable because plastic hinges are formed in the beam-column joints and the beam deflection profiles can be approximated by straight lines.

As shown in Table 6.3, there are three types of failure mode. The test results indicate that an increase of angle thickness changes the failure mode. When the angle thickness was 8 mm, failure mode of angle fracture at the heel was observed, as shown in Figure 6.4. When the angle thickness was increased to 10 mm, the failure mode also changed to angle fracture with yielded bolts (Figure 6.5). But when the angle thickness was further increased to 12 mm, the failure mode of bolt fracture with yielded angles was observed, as shown in Figure 6.6.

Figure 6.4 Failure mode of angle fracture at the heel

Figure 6.5 Failure mode of angle fracture with yielded bolts

Figure 6.6 Failure mode of bolt fracture with yielded angles

Table 6.3 List of test results of bolted-angle joint specimens

Specimen	Connection type	Maximum load (kN)	Displacement* (mm)	Joint rotation** (degree)	Maximum horizontal reaction (kN)	Maximum moment (kNm)	Maximum axial force of beams N_B (kN)	Failure mode
W-8	Web cleat	183	443	10.78	473	11.8	470	Angle fracture
W-10	Web cleat	193	419	10.2	517	15.6	524	Angle fracture with yielded bolts
W-12	Web cleat	185	365	8.9	566	24.4	571	Bolt fracture with yielded angles
TS-8	Top and seat angle	87	319	7.8	229	31.4	229	Angle fracture
TS-10	Top and seat angle	87	286	7.0	245	43.1	247	Angle fracture with yielded bolts
TS-12	Top and seat angle	114	294	7.2	290	68	289	Bolt fracture with yielded angles
TSWA-8	TSWA	204	454	11.04	545	36.2	552	Angle fracture
TSWA-10	TSWA	218	423	10.3	594	59.4	599	Angle fracture with yielded bolts
TSWA-12	TSWA	230	398	9.71	677	91.4	680	Bolt fracture with yielded angles

* Displacement corresponding to the maximum load

** Joint rotation corresponding to the maximum load

6.2.3.1 Web cleat connections

(a) Vertical force versus vertical displacement of the middle column

(b) Axial force of beams versus vertical displacement of the middle column

Figure 6.7 Test results of web cleat connections

The specimen was held at the horizontal position, and a vertical load was gradually applied to the middle column until total failure of beam-column joint occurred. As this type of connections had no capacity to resist moment, the specimen rotated at

both ends with increasing deflection at the mid-span. Catenary action soon developed until fracture occurred at the middle beam-column joint. Figure 6.7 (a) shows the vertical force-middle column displacement relationships for this type of connection. It can be found that an increase of angle thickness increased the connection axial stiffness at catenary action stage. However, the increased connection axial stiffness decreased the connection deformation capacities (the displacement corresponding to the maximum load). This is the reason why there are no significant increases of loading resistances with an increase of angle thickness.

6.2.3.2 Top and seat angle connections

Significant flexural actions are observed in Figure 6.8. It is noteworthy that catenary action was mobilised before the fracture of the bottom angles. Due to catenary action, the load increased rapidly until bottom angle fractured, which determined the load-carrying capacities of this type of connections. Figure 6.8 also shows that an increase of angle thickness enhanced the flexural action and tensile forces formed in the beams.

6.2.3.3 TSWA connections

Figure 6.9 shows that an increase of angle thickness enhanced the connection performance, including the load resistance, stiffness and tensile force for TSWA. Similar to the previous tests, deformation capacities decreased with an increase of angle thickness. It should be mentioned that the load-carrying capacity of Specimen TSWA-10 (218 kN) listed in Table 6.3, corresponded to the peak point of 423 mm displacement. Although at a displacement of 373 mm, the load is 230 kN, which is higher than the chosen value, at the stage before this point the whole joint was not symmetric (at one middle connection the bottom angle fractured while at the other middle connection the angle was still intact). The load-displacement curve is affected by the un-symmetric boundary condition. Therefore at this point, the obtained value is not representative. After the peak point of 373 mm displacement, the whole joint became symmetric again, and the obtained curve at the following

stage is valuable. This is why the peak point of 423 mm displacement was chosen for this specimen. For the other eight specimens, the maximum loads were reached at the symmetric loading stage, which could represent the real joint behaviour well.

(a) Vertical force versus vertical displacement of the middle column

(b) Axial force of beams versus vertical displacement of the middle column

Figure 6.8 Test results of top and seat angle connections

(a) Vertical force versus vertical displacement of the middle column

(b) Axial force of beams versus vertical displacement of the middle column

Figure 6.9 Test results of TSWA connections

The observations about flexural action and catenary action of this type of connections are similar to top and seat angle connections. However, because of the presence of web angles, the load-carrying capacities, the maximum moments and the axial forces of beams of TSWA connections are much greater than top and seat

angle connections. For TSWA connections, the load-carrying capacities were reached when the web angle fractured.

6.3 Development of a component-based model of bolted-angle connections

In this section, a component-based model of bolted-angle connections will be developed. The behaviour of beam-column joints under a middle-column-removal scenario can be predicted by this model. In addition, the fracture of each component, which determines the ultimate loads, can be predicted as well. The experimental results presented in the previous section provide essential data for the development and validation of the proposed component-based model.

Figure 6.10 Component-based model of bolted-angle connections

6.3.1 General

In the component-based approach, connections are divided into a series of components. The assembly of these components can predict the performance of joints under different loading conditions. This method has been successfully used by many researchers (Tschemmerneegg 1982, Tschemmerneegg et al. 1987, Faella et

al. 2000, Eurocode 3-1-8 2005, Lemonis and Gantes 2009, Málaga-Chuquitaype and Elghazouli 2010, Wang et al. 2007, Park and Wang 2012, Wang 2011). Lemonis and Gantes (2009) suggested that for bolted-angle connections, aligned models are more effective overall. In aligned models, all the springs of the first row are aligned with the bolt axis while in non-aligned models the springs corresponding to components of the connection are aligned with the beam flange or the angle horizontal leg. The proposed component-based model of bolted-angle connections shown in Figure 6.10, is an aligned model. The basic components include bolt in tension (bt), top, web and seat angle in bending (tab , wab , sab), angle in bearing (abb), bolt in shear (bs), beam flange in bearing (bfb), bolt slippage model (bsm) and beam web in bearing (bwb). The mechanical model of bolted-angle connections under tension presented in Chapter 5, has been used in this component-based model. In this model, it is assumed that the components of tab , wab and sab have the same mechanical properties. As shown in Figure 6.10, the component-based model has been simplified by combining all the components at one bolt row as one equivalent spring. For example, all the components located at the bottom bolt row including bt , sab , bsm , bfb , bs and abb have been combined as one spring $k1$. In this model it is assumed that the springs of $k1$, $k3-1$, $k3-2$ and $k3-3$ can sustain tensile forces while all the compression forces are sustained by the spring of $k2$. This assumption is used by most of the proposed component-based models (Faella et al. 2000, Eurocode 3-1-8 2005, Lemonis and Gantes 2009, Elghazouli et al. 2010, Yuan and Tan 2011a and Yuan and Tan 2011b, Wang et al. 2007, Park and Wang 2012). In this set of nine experimental tests on three types of bolted-angle connections, no shear failure was observed. Thus, in the component-based model, a rigid shear spring to transfer vertical shear forces is used.

The failure model of bolt fracture was considered in the mechanical model of bolted-angle connections under tension presented in Chapter 5, and bolt slippage model was also incorporated into this model. It means three components including tab (wab , sab), bt and bsm have been considered by this model. The component properties of plate in bearing and bolt in shear were proposed by Yu et al. (2009a,

2009b) and the components such as abb , bs , bfb and $bwbb$, were calculated based on Yu's model.

It should be mentioned that the deformation capacities of top and seat angles in bending is based on the deformation at the beam flange level. This is because in the component-based joint model shown in Figure 6.10, the top and seat angles are assumed to be rigid and the deformation of angles is located along the top and bottom bolt rows. However, in the mechanical model of bolted-angles in bending presented in Chapter 5, the deformation capacities are estimated based on the deformation of the angle leg connected to the beam flange, which is located at the beam flange level. Therefore, in this model it is assumed that once the deformation at the beam flange level reaches the deformation capacities of bolted-angles in bending, this component is deemed to have failed. It should be noted that the group effects are not considered in the component method. However, in the web angle connections, the three components of web angles ($k3-1$, $k3-2$ and $k3-3$) are working together and they influence with each other. In the experimental tests, it was observed that the two springs corresponding to the bottom two bolt rows of web cleat angles ($k3-2$ and $k3-3$) failed simultaneously. Therefore, in the proposed component-based model, failure of these two springs is dependent on the component of $k3-2$. The deformation capacity of $k3-2$ can be predicted by the proposed mechanical model of bolted-angle connections under tension in Chapter 5. Once the component of $k3-3$ reaches the deformation capacity predicted by the proposed model in Chapter 5, it is assumed that this spring goes into a plastic plateau stage without any increase of loads. When the component of $k3-2$ reaches the deformation capacity, those two springs are defined to fail at the same moment. The validation works presented below also show this assumption can produce a closer agreement with the test results.

In the test set-up system, as shown in Figure 6.2, there were some gaps between the two support pins and the pin holes. These connection gaps have great influence on the formation of compressive arch action and catenary action. Therefore, the measured horizontal movement of support pins should be introduced into the

component-based model in order to simulate actual boundary conditions. A simplified horizontal restraint model is shown in Figure 6.11. This model is obtained from the experimental tests and is used in the component-based model simulations.

Figure 6.11 Horizontal restraint model

6.3.2 Comparison with test results

The component-based model proposed in Section 6.3.1 is validated against the results of the experimental programme described in Section 6.2. The vertical force-middle column displacement relationships from the component-based models are compared with the test results in Figures 6.12 - 6.20. The relationships between beam axial forces and middle column displacements are also compared in these figures. It can be observed that the component-based model predictions are in close agreement with the experimental results. The models can simulate the key responses of bolted-angle beam-column joints under a middle-column-removal scenario, including the formation of flexural action at small deformation stage, the development of catenary action at large deformation stage and fractures of the connection components at the last stage. Table 6.4 shows the comparisons between the test results and the calculated results predicted by the component-based models. From this table, it can also be found the predictions agree well with the test results.

(a) Vertical force versus vertical displacement of the middle column

(b) Axial forces of beams versus vertical displacement of the middle column

Figure 6.12 Comparison of the analytical predictions against the test data of W-8

(a) Vertical force versus vertical displacement of the middle column

(b) Axial forces of beams versus vertical displacement of the middle column

Figure 6.13 Comparison of the analytical predictions against the test data of W-10

(a) Vertical force versus vertical displacement of the middle column

(b) Axial forces of beams versus vertical displacement of the middle column

Figure 6.14 Comparison of the analytical predictions against the test data of W-12

(a) Vertical force versus vertical displacement of the middle column

(b) Axial forces of beams versus vertical displacement of the middle column

Figure 6.15 Comparison of the analytical predictions against the test data of TS-8

(a) Vertical force versus vertical displacement of the middle column

(b) Axial forces of beams versus vertical displacement of the middle column

Figure 6.16 Comparison of the analytical predictions against the test data of TS-10

(a) Vertical force versus vertical displacement of the middle column

(b) Axial forces of beams versus vertical displacement of the middle column

Figure 6.17 Comparison of the analytical predictions against the test data of TS-12

(a) Vertical force versus vertical displacement of the middle column

(b) Axial forces of beams versus vertical displacement of the middle column

Figure 6.18 Comparison of the analytical predictions against the test data of TSWA-

(a) Vertical force versus vertical displacement of the middle column

(b) Axial forces of beams versus vertical displacement of the middle column

Figure 6.19 Comparison of the analytical predictions against the test data of TSWA-

(a) Vertical force versus vertical displacement of the middle column

(b) Axial forces of beams versus vertical displacement of the middle column

Figure 6.20 Comparison of the analytical predictions against the test data of TSWA-

Figures 6.15, 6.16 and 6.17 indicate that the spring kl at the top bolt row always predicts higher resistances than the test results. This is because after the seat and web angles have fractured, the failure mechanism of the top angle was different from the other angle components. Although for this type of connection, this component prediction is poor, normally the loading resistances of bolted-angle connections are not determined by this component alone. In general, the proposed component-based model captures the main features of bolted-angle beam-column joints up to total failure under a middle-column-removal scenario.

Table 6.4 Comparison between the component-based models and the test results at ultimate load points

Specimen	Vertical load (kN)			Displacement (mm)			Beam axial force (kN)		
	Test	Model	Model/Test	Test	Model	Model/Test	Test	Model	Model/Test
W-8	183	203	1.11	443	429	0.97	470	542	1.15
W-10	193	223	1.16	419	416	0.99	524	613	1.17
W-12	185	197	1.06	365	372	1.02	571	598	1.05
TS-8	87	101	1.16	319	308	0.97	229	230	1.00
TS-10	87	99	1.14	286	279	0.98	247	220	0.89
TS-12	114	93	0.82	294	264	0.90	289	169	0.58
TSWA-8 (first peak load)	136	197	1.45	311	319	1.03	408	547	1.34
TSWA-10 (first peak load)	192	181	0.94	318	288	0.91	549	464	0.85
TSWA-12 (first peak load)	218	176	0.81	312	276	0.88	598	422	0.71
TSWA-8 (ultimate load)	204	201	0.99	454	418	0.92	552	551	1.00
TSWA-10 (ultimate load)	218	211	0.97	423	396	0.94	599	610	1.02
TSWA-12 (ultimate load)	230	191	0.83	398	360	0.90	680	601	0.88
Mean			1.04			0.95			0.97
Standard deviation			0.18			0.05			0.21

6.4 Frame analyses

Having gained the confidence in the reliability of the component-based model, frame analyses are carried out by incorporating the proposed joint model. One of the objectives of the frame analyses is to identify the difference in structural performances between simplified joint and frame models under middle-column-removal scenarios. In addition, the behaviour of steel frames under concentrated point load (CPL) and uniformly distributed load (UDL) conditions will be investigated. Finally in the section, the effect of horizontal restraint stiffness, which plays a critical role in the development of catenary action, is also studied.

6.4.1 Discussion about frame and joint models

The prototype steel frame is shown in Figure 6.21 (a). In the *frame model* shown in Figure 6.21 (b), the whole two beam spans and four connections at these two spans are modelled. The two columns at the two sides are restrained by surrounding structure elements. The connections are simulated by the developed component-based model, which was presented in Section 6.3. In the experimental tests and the *joint model*, an inflection point was assumed to be located at the middle of the beam span and only one-half of the beam span at both sides of the beam-column joint was used, as shown in Figure 6.21 (c). As stated in Section 6.2.1, the experimental tests and the *joint model* could represent the behaviour of both the middle and the side connections. In the analyses of this section, two types of connections are considered. The first type has the same configuration with Specimen TSWA-12 and the second one is identical with TSWA-8. The experimental and joint model analytical results of these two connections have been presented in Sections 6.2.3 and 6.3.2, and these results will be used to compare with the frame model shown in Figure 6.21 (b).

Figure 6.21 Frame model and joint model with bolted-angle beam-column joints

The numerical results of the frame models are compared with the joint experimental results and joint models in Figure 6.22. In this figure, the joint models represent the analytical results presented in Section 6.3.2. It should be mentioned that the vertical displacements of the middle columns in the frame models shown in Figure 6.22 are only one-half of the actual displacement from anti-symmetric condition about Point E (F) shown in Figure 6.21. This is for convenience to compare the results against the experimental tests and joint models. It can be found from Figure 6.22 that the frame models have almost the same predictions with the joint models. Although the vertical displacements of the middle columns in the joint models are only one-half of the frame models, the connection rotation angles are identical. This indicates that the conducted joint tests can represent the behaviour of the prototype steel frame.

The obtained connection rotation angles, loading resistances and internal beam axial forces from the joint tests are in close agreement with the frame model.

(a) TSWA-12

(b) TSWA-8

Figure 6.22 Comparison of the frame models against the test results and joint models

6.4.2 Discussion about loading methods

Figure 6.23 Frame models under CPL and UDL

In the experimental tests and previous analytical works, CPLs were applied onto the middle columns until total structural collapse occurred. However, the loading condition in reality under progressive collapse is UDL. The structural performance of steel frames under UDL should be analysed and compared with CPL. Thus, CPL and UDL are applied onto the frame models, respectively, as shown in Figures 6.23 (a) and (b), and the two frame model results are compared in Figure 6.24.

In Figure 6.24, the vertical displacements of the middle columns are the actual values obtained from the analyses. The UDLs were applied onto the frame models under the load-controlled algorithm. Thus, only the load-increasing part of the load-displacement curves is obtained, as shown in Figure 6.24. Nevertheless, the CPLs were applied under displacement control, and the whole load-displacement curves could be obtained. In the frame models under UDL, the vertical forces shown in Figure 6.24 are only one-half of the total vertical loads. As shown in Figure 6.25, the UDL close to the middle column can be converted into an equivalent CPL. Nevertheless, the UDL close to the two side joints will be transferred onto the adjacent structural members directly. Therefore, the equivalent CPL at the middle column is equal to one-half of the total UDL. It is evident from Figure 6.24 that the analytical results under CPL are identical with the results under UDL. It indicates

that for frame analyses and tests, the behaviour of structural members under CPL can represent the performance of structures under UDL and the obtained data can be used to assess the structural resistance against progressive collapse. In addition, the analytical results also demonstrate that the steel frames under CPL have the same performance with the frames under UDL, if the CPL is equal to one-half of the total vertical forces of UDL.

(a) TSWA-12

(b) TSWA-8

Figure 6.24 Comparison of the frame models under UDL against the frame models under CPL

Figure 6.25 Equivalent CPL of UDL in a frame model

As stated in Section 3.2.1, under CPL, the fracture of beam-column joint indicates the collapse of the building structures. However, in reality, after the joint failure, 3D floor slab may still be able to resist the vertical loads by tensile membrane action. Therefore, CPL can be used to investigate the behaviour of beam-column joint, but it may not be a proper way to investigate the total structural resistance against progressive collapse, if the 3D floor resistance of tensile membrane action is also studied.

6.4.3 Discussion about the effect of horizontal restraint stiffness

The stiffness of the horizontal restraint has a significant influence on the mobilisation of catenary action. In order to investigate this effect, a parametric study is conducted based on the frame models under CPL. Two connections including W-12 and TSWA-12 are considered in this study. To illustrate the effect of horizontal restraint stiffness, the stiffness term is normalized by the elastic stiffness of the connections under pure tension, as follows:

$$\gamma = \frac{K_h}{K_c / 2} \quad (6.1)$$

where K_h is the horizontal restraint stiffness, K_c is the elastic stiffness of one beam-column connection under tension and γ is the normalized stiffness ratio. In the frame model, there are two connections in one beam span. These two connections

are placed in series and under tension, the stiffness of these two connections becomes $K_c/2$. Therefore, only half of K_c is used to normalize the stiffness.

Figures 6.26 and 6.27 show the simulation results under different horizontal restraint stiffnesses for W-12 and TSWA-12, respectively. Both the vertical loads and axial forces of beams are compared. It can be seen that the responses of the frames are not sensitive if γ is greater than one. It means if the horizontal stiffness is greater than the frame stiffness under tension, the performance of the frame is only affected by the beam-column connections. However, if γ is less than one, the influence of the horizontal restraint stiffness on the behaviour of steel frames is very significant. It is interesting that for W-12 and TSWA-12, with increasing horizontal restraint stiffness, the behaviour of the two frame models is different. From Figure 6.26 (b), it can be found that for W-12, if $\gamma \geq 0.04$, the formed maximum axial forces of beams are kept as a constant value of 600 kN. Figure 6.26 (b) also shows that an increase of the horizontal restraint stiffness reduces the connection rotation capacity. Figure 6.26 (a) shows that if $\gamma \geq 0.04$, an increase of the horizontal stiffness will cause a decrease of load resistance. This is because the contribution from catenary action is reduced when the connections can only form smaller rotation angles. Meanwhile, if $\gamma \leq 0.04$, a decrease of γ will cause reducing load resistance and beam axial force. This is because if $\gamma \leq 0.04$, beam flange will come into contact with column flange directly and transfer compression forces. As shown in Figure 6.3 (d), the gap between beam flange and column flange is 10 mm. If $\gamma \geq 0.04$, due to this 10 mm gap, beam flange could not contact with column flange directly. Under this condition, a decrease of γ increases the connection rotation capacity and load resistance. Nevertheless, if $\gamma \leq 0.04$, the horizontal movement of the beam flange would be greater than 10 mm and beam flange could transfer compressive forces through the direct contact with column flange. The contact effect will decrease the contribution from catenary action. This is because the rotation capacities of the connections are limited by this compressive spring. Once the rotation capacities are limited, the contribution from catenary action is limited as well. Thus, a special compressive spring should be included to consider the effect of the contact between beam flange and column flange. In this model, this effect was simulated by a

compressive spring shown in Figure 6.28. The stiffness of this spring K_f depends on the stiffnesses of beam flange in compression and column web in compression. This spring is located at the same level with spring k_2 shown in Figure 6.10. From Figure 6.26 (a) it can be concluded that for W-12, if there is no horizontal restraint, the load resistance is only 67 kN. In contrast, if a horizontal restraint is provided, the load resistance can be increased by 310% at most.

(a) Vertical force versus vertical displacement of the middle column

(b) Axial forces of beams versus vertical displacement of the middle column

Figure 6.26 Effect of horizontal restraint stiffness for W-12

(a) Vertical force versus vertical displacement of the middle column

(b) Axial forces of beams versus vertical displacement of the middle column

Figure 6.27 Effect of horizontal restraint stiffness for TSWA-12

For TSWA-12, the simulation results are compared in Figure 6.27. In order to simplify the curves, only the results up to the failure of one component are presented. It is very clear that if γ is less than one, an increase of the horizontal restraint stiffness increases the load resistances and beam axial forces. In this model, the spring k_2 shown in Figure 6.10 could transfer the compressive forces from the initial loading stage to the final failure. It can be found that this spring has a similar effect with the special compressive spring shown in Figure 6.28. Due to the

presence of the compressive spring k_2 , a decrease of γ reduces the load resistances and beam axial forces. Figure 6.27 (a) also shows that the load resistance of TSWA-12 can be increased by 100% at most if there are sufficient horizontal restraints.

Figure 6.28 Special compressive spring to consider the contact effect between beam flange and column flange

Figure 3.6 shows a middle-column-removal scenario. According to Yu (2012), the axial restraint stiffness of a middle-column-removal scenario may range from 356.7 kN/mm to 438 kN/mm. To be conservative, it is assumed in Figure 3.6, the axial restraint stiffness is 356.7 kN/mm. In the parametric study of Section 6.4.3, K_c is equal to 314.6 kN/mm for Connection W-12 and K_c is equal to 533.45 kN/mm for Connection TSWA-12. Therefore, the value of γ for Connection W-12 is 2.27 and the value of γ for Connection TSWA-12 is 1.34. As shown in Figures 6.26 and 6.27, if the value of γ is greater than one, the responses of the frames are not sensitive to the parameter γ . It means under a middle-column-removal scenario, as shown in Figure 3.6, the performance of the frame is only affected by the behaviour of beam-column connections.

6.5 Summary and conclusions

This chapter presents the descriptions and experimental results of nine tests on bolted-angle beam-column joints under a middle-column-removal scenario. A component-based model is also developed to simulate the test results. The following conclusions can be drawn:

- (1) For bolted-angle connections, the test results indicate an increase of angle thickness will alter the failure mode. With increasing angle thickness, there are three corresponding failure modes, namely, angle fracture at the heel, angle fracture with yielded bolts and bolt fracture with yielded angles;
- (2) For web cleat connections, an increase of angle thickness increases the connection axial stiffness at catenary action stage. However, an increased connection axial stiffness reduces the connection deformation capacities;
- (3) For top and seat angle connections, an increase of angle thickness enhances the flexural action and also the formation of tensile forces in the beams;
- (4) For TSWA connections, an increase of angle thickness enhances the connection performance, including loading resistance, stiffness, tensile forces of beams formed. Similar to the other two types of connections, the deformation capacities decrease with an increase of angle thickness;
- (5) The proposed component-based models can represent the key responses of bolted-angle beam-column joints under a middle-column-removal scenario, including the formation of flexural action at small deformation stage, the development of catenary action at large deformation stage and fractures of the connection components at the last stage;
- (6) By using the developed component-based beam-column joint models, frame analyses indicate that the conducted joint tests can represent the behaviour of a prototype steel frame. The obtained connection rotation angles, loading resistances and internal beam axial forces from the joint tests are in close agreement with the frame model;
- (7) Frame analyses demonstrate that the behaviour of structural members under CPL can represent the performance of structures under UDL. The obtained data under CPL can be used to assess the structural resistance against progressive collapse. The frame analytical results also demonstrate that the steel frames

under CPL have the same performance with the frames under UDL, if the CPL is equal to one-half of the total vertical forces of UDL;

- (8) A parametric study indicates that the influence of the horizontal restraint stiffness on the behaviour of steel frames is very significant if the horizontal restraint stiffness is smaller than the frame stiffness under tension.

CHAPTER 7 EXPERIMENTAL TESTS AND COMPONENT- BASED MODELLING OF COMPOSITE BEAM-COLUMN JOINTS

7.1 Introduction

Chapters 3 and 6 presented the behaviour of bare steel beam-column joints under middle-column-removal scenarios. However, the behaviour of composite joints has not been investigated well. The performance of composite joints is different from bare steel joints at both flexural action stage and catenary action stage.

In this chapter, the behaviour of composite beam-column joints under a middle-column-removal scenario will be presented. In addition, the contribution of composite slab to resist progressive collapse will be discussed quantitatively.

In the past, numerical and analytical models have been developed to simulate the behaviour of composite beam-column joints by many researchers (Anderson and Najafi 1994; Fu, Lam et al. 2008; Wang and Li 2008; Fu, Lam et al. 2010; Piluso, Rizzano et al. 2012). However, these studies only included moderate deformations and were limited to moment-rotation relationships. In this chapter, component-based models will be developed to predict the behaviour of composite beam-column joints up to total failure under a middle-column-removal scenario. The experimental results obtained from the tests will be used to validate the proposed component models. Finally, frame analyses are conducted based on the developed component-based models.

7.2 Test set-up and specimens

7.2.1 Test set-up

(a) Middle joint

(b) Side joint

Figure 7.1 Prototype composite beam-column joint

The hypothetical beam-column joints considered for the experimental tests are located above the storey where an internal column has been forcibly removed. As shown in Figure 7.1, an inflection point is assumed to be located at the middle of the beam span. Therefore, only one-half of the beam span was simulated using pin conditions. Two types of joints were tested, viz. the middle joint located above the removed column and the side joint at the adjacent connections of the damaged beam spans. They are both shown in Figures 7.1 (a) and (b), respectively. The behaviour of the middle and the side joints, including the load-carrying and rotation capacities, can be represented by the test specimens. Due to space limitation in the laboratory, the test specimens were scaled down to 2/3 of their original size. The simplified specimen is representative of any other upper floors above the zone of damage, since the whole column experiences a downward rigid body displacement and the axial forces in the column above the damaged storey are very small and can be neglected.

It should be noted that during the deformation process, the actual inflection point may change its location. However, the objective of this study is to investigate the behaviour of composite joints under a middle-column-removal scenario. The obtained experimental results will be used to validate the component-based analytical models, which can be used to study the interaction between the middle and the side joints and the load redistribution after the removal of the middle column. It is believed that the assumption of the inflection point will not affect the formation of catenary action and the rotation capacities of joints under column-removal scenarios. Further discussions about the pin support assumption will be presented in Section 7.6.1 for composite steel joint tests.

The test set-up is shown in Figure 7.1. An additional horizontal restraint was provided by an A-frame and a strong reaction wall to simulate restraint from

surrounding structural elements. In order to consider the rotational restraint to beam-column joints from the continuous column of upper storeys, the test rig included a rotational restraint system at the mid-span, as shown in Figure 7.2. The column rotation was restrained by sixteen steel rollers (Roller 2 shown in Figure 7.2), which would bear against the flanges of four steel columns during testing. In addition, the beam-column joints were restrained from lateral movement by a lateral restraint system (Roller 1 shown in Figure 7.2). A displacement-controlled point load was applied to the middle column using an actuator, which was reacted against a strong H-frame. Load was applied under displacement control at a rate of 6 mm/min. A pair of secondary loading beams shown in Figure 7.2, was also connected to the middle column, to ensure the line load could be applied onto the composite slabs directly.

7.2.2 Instrumentation

Every specimen consisted of two parts, namely, the composite beam-column joint and the two horizontal steel hollow circular members. The internal forces of the specimens were measured through strain gauges attached to four sections onto the steel hollow section members, as shown in Figure 7.3. The strains of reinforcement, profile decking and steel beams were also measured. The locations of strain gauges are shown in Figure 7.3. Actuator load was measured by an external load cell so as to verify the external and internal forces in the test assembly.

The instrumentation included linear variable differential transducers (LVDTs) and line transducers for vertical deflection and joint rotation measurements. The LVDTs and line transducers were placed along the beam length to measure in-plane vertical deflections.

(a)

(b)

Figure 7.2 Restraint and loading systems of composite beam-column joint tests

(a) Strain gauge layout of the two steel circular hollow members

(b) Strain gauge layout of the tested specimen

Figure 7.3 Locations of strain gauges of composite beam-column joint tests

(a) Middle joint specimens (flush end plate connection)

(b) Side joint specimens (web cleat connection)

Figure 7.4 Layout of the composite joint specimens

(a) Flush end plate

(b) Web cleat

(c) Strengthened web cleat

Figure 7.5 Details and dimensions of the composite joint specimens

7.2.3 Test specimens

All specimens had the same total length of 2,990 mm, with the distance between two pin supports of 4849 mm. The length of concrete slab was kept at only 2,930 mm. The configurations of the specimens are illustrated in Figure 7.4, which shows the elevation views of the middle and the side joint specimens. In this experimental

programme, three types of connections including web cleat, strengthened web cleat and flush end plate connections, were tested. All the connection details of these three types of connections are shown in Figure 7.5. The beam and column cross-sections of all the specimens were the same, viz. UB254×146×37 and UC203×203×71. The beams and columns were strengthened by some stiffeners or welded thick plates to limit the influence of beam and column deformations on the connection behaviour. As shown in Figure 7.5, two rows of shear stubs with a displacement of 90mm were welded along the steel beams in order to ensure the full shear connection between steel beam and composite slab. Composite action is greatly affected by shear connection provided by shear stubs. One of the objectives of this chapter is to investigate the effect of composite action, therefore full shear connection has been used in specimen design. The details of the five composite joint specimens are also presented in Table 7.1. The reference used for the specimens follows the nomenclature $M(S)-W(SW)(F)-t$ where M represents the middle joint and S the side joint, W represents web cleat connection, SW represents strengthened web cleat connection, F represents flush end plate connection and t is the angle or end plate thickness in mm. For instance, in M-W-9, the specimen represents the *middle joint*, the connection type is *web cleat*, and the angle thickness is *9 mm*.

Table 7.1 Summary of composite joint test specimens

Specimen ID	Loading condition	Connection type	Bolt	End plate/angle	Rebar
M-W-9	Middle joint	Web cleat	Grade 8.8 M20	L90×9	4 T16
M-SW-11		Strengthened web cleat	Grade 8.8 M24	L125×11	4 T16
M-F-12		Flush end plate	Grade 8.8 M20	200×12	4 T16
S-W-9	Side joint	Web cleat	Grade 8.8 M20	L90×9	4 T16
S-F-12		Flush end plate	Grade 8.8 M20	200×12	4 T16

7.2.4 Material properties

Material tests were conducted to obtain the representative stress-strain curves for analysis purpose. These include tensile testing of steel angles, flush end plate, steel profile decking, steel beams and reinforcement, and compressive and tensile tests of concrete.

The tensile test results of structural steel including steel angles, 12 mm thick flush end plate, profile decking, beam flange and web, and T16 reinforcement, are presented in Table 7.2. Both compressive test and indirect splitting test were conducted to obtain the compressive and tensile strength of concrete. The average concrete compressive and tensile strengths were 31.8MPa and 2.71MPa, respectively. One of the concrete compressive cylinder test curve is shown in Figure 7.6.

Table 7.2 Tensile test results of structural steel from composite joint test specimens

Coupon plate	Yield strength (MPa)	Ultimate strength (MPa)	Ultimate strain*
L 90×9 angle	345	496	0.253
L125×11 angle	280	430	0.304
12 mm thick end plate	235	409	0.36
Profile decking	580	590	0.12
Beam flange	320	482	0.243
Beam web	343	484	0.305
T16 reinforcement	525	611	0.203

* Fracture strain is based on the proportional coupon gauge length of $5.65\sqrt{S_0}$,

where S_0 is the original cross-section area of coupons.

7.3 Experimental results

In this section, the experimental results of five composite beam-column joints are presented. A summary of the test results is included in Table 7.3, from which the maximum vertical loads, the corresponding middle column displacement and rotation angles at the ends are given. The maximum horizontal reaction forces, the

maximum moments and axial forces of beams are also included in this table. It should be mentioned that the rotation capacities of joints correspond to the maximum loads and these rotation angles are obtained by dividing the centre column displacement at the maximum load by the beam span of 2.317m, as shown in Figure 7.1. This simplification is reasonable because the beam deflection profiles in between the plastic hinges can be approximated by straight lines. In addition, the maximum horizontal reaction forces and the maximum axial forces of beams listed in Table 7.3 correspond to the maximum load points. In subsequent sections, the experimental results and observations from the five connection tests, summarised in Table 7.3, are presented.

Figure 7.6 Stress-strain curve obtained from concrete compressive cylinder tests

Table 7.3 List of composite joint test results

Specimen ID	Connection type		Maximum vertical load (kN)	Displacement* (mm)	Joint rotation** (degree)	Maximum horizontal reaction (kN)	Maximum moment (kNm)	Maximum axial force of the beam N_B (kN)
M-W-9	Middle joint	Web cleat	184	320	7.86	654.5	81.36	664
M-SW-11		Strengthened web cleat	284	382	9.36	933	116.2	946
M-F-12		Flush end plate	333	419	10.25	1056	143.2	1088
S-W-9	Side joint	Web cleat	241	392	9.6	653.6	170.5	658
S-F-12		Flush end plate	238	298	7.33	581	191.7	580

* Displacement corresponding to the maximum load

** Joint rotation corresponding to the maximum load

7.3.1 Specimen M-W-9

Figure 7.7 (a) shows the vertical force-middle column displacement relationship for Specimen M-W-9. The beam axial forces and bending moments of the left connection are given in Figures 7.7 (b) and (c), respectively. It can be seen that at initial loading stage, the applied load was resisted by flexural action while at large deformation stage, the load was resisted by catenary action. Figure 7.7 (b) shows that catenary action could not be mobilized until the vertical displacement of the middle column was greater than 200 mm. Through the development of catenary action, the bending moment of the left connection started to decrease, as shown in Figure 7.7 (c). At the maximum load point, the bending moment reduced 89% to 8.6 kNm, as shown in Figure 7.7 (c). It means the composite beam-column connection was almost under pure tension at the maximum load point. Figure 7.7 (a) shows that at this moment, 98% of the total loads were resisted by catenary action. The failure mode of this specimen is shown in Figure 7.8. Profile decking fractured at a displacement of 262 mm. Figure 7.7 (a) shows that there is no significant decrease of total applied load after even this fracture. Concrete crushed severely in compression during testing, as shown in Figure 7.8 (b). One of the web angles fractured at a displacement of 320 mm, which caused a significant reduction of applied load (Figure 8 (c)). Finally, the two web angles at the right-hand side of the beam-column joint fractured completely at a displacement of 370 mm.

(a) Vertical force-middle column displacement curve

(b) Left beam axial force-middle column displacement curve

(c) Left connection moment-middle column displacement curve

Figure 7.7 Vertical force-middle column displacement curve of Specimen M-W-9

7.3.2 Specimen M-SW-11

Figure 7.10 (a) shows the force-displacement history of Specimen M-SW-11. Similar to Specimen M-W-9, the force-displacement history can be divided into two stages: flexural action and catenary action. The beam axial forces and bending moments of the left connection are given in Figures 7.10 (b) and (c), respectively. Figure 7.11 shows the failure mode of this specimen. At a displacement of 128 mm, fracture of profile decking was observed. Final failure was controlled by web angle fracture at a displacement of 382 mm, which corresponded to the maximum load.

(a) Fractured profile decking

(b) Crushed concrete

(c) Fractured angle

Figure 7.8 Failure mode of Specimen M-W-9

Figures 7.9 (a) and (b) show the overall deformations of Specimen M-W-9 before and after the test, respectively. From these two figures it can be found that the displacement profile of each beam can be approximated by a straight line, which indicates the formation of plastic hinges at the beam-column connections.

(a) Before the test

(b) After the test

Figure 7.9 Overall deformation of Specimen M-W-9

(a) Vertical force-middle column displacement curve

(b) Left beam axial force-middle column displacement curve

(c) Left connection moment-middle column displacement curve

Figure 7.10 Vertical force-middle column displacement curve of Specimen M-SW-

11

7.3.3 Specimen M-F-12

The force-middle column displacement relationship of Specimen M-F-12 is given in Figure 7.12. As shown in Figure 7.13, fracture of profile decking and concrete crushing in compression were observed in this test. In the steel part of this specimen, failure was initiated by beam web welding fracture, followed by thread stripping of the bottom row of bolts. Finally, the maximum load was reached when the welded end plate fractured close to the beam flange welding zone at a displacement of 419 mm.

(a) Fractured profile decking

(b) Crushed concrete

(c) Fractured angle

Figure 7.11 Failure mode of Specimen M-SW-11

(a) Vertical force-midddle column displacement curve

(b) Left beam axial force-midddle column displacement curve

(c) Left connection moment-middle column displacement curve

Figure 7.12 Vertical force-middle column displacement curve of Specimen M-F-12

(a) Fractured profile decking

(b) Crushed concrete

(c) Fractured flush end plate connection

Figure 7.13 Failure mode of Specimen M-F-12

7.3.4 Specimen S-W-9

The vertical force-middle column displacement relationship of Specimen S-W-9 is given in Figure 7.14 (a). At a displacement of 45 mm, T16 reinforcing bars yielded, followed by the fracture of profile decking at a displacement of 64 mm, as shown in Figure 7.15 (b). Figure 7.14 (a) shows that this fracture caused a slight decrease of the vertical load. It was also noticed that due to the fracture of profile decking, the beam axial force changed from tension to compression. Local buckling of the beam flange shown in Figure 7.15 (a), was also observed before the maximum load was reached. It can be found from Figure 7.14 that, at side joints, catenary action could be mobilised to resist the vertical load. The load resistance of this specimen was governed by T16 reinforcement that fractured at a displacement of 392 mm, followed by web angle fracture at a displacement of 440 mm. This specimen was initially in tension, as shown in Figure 7.14 (b). At the beam-column connections of this specimen, one of the most critical components was the steel beam flange in compression before the fracture of profile decking . At the initial loading stage, the

steel beam flange attained the compressive resistance of this component quickly and went into plastic stage. Therefore, compressive arch action could not be formed initially. Nevertheless, once the profile decking fractured, the compressive force sustained by the steel beam flange was reduced significantly. Under this condition, the specimen had the potential to develop compressive arch action. This is the reason that in the following stage, compressive forces were formed in this specimen, as shown in Figure 7.14 (b).

(a) Vertical force-middle column displacement curve

(b) Left beam axial force-middle column displacement curve

(c) Left connection moment-middle column displacement curve

Figure 7.14 Vertical force-middle column displacement curve of Specimen S-W-9

(a) Local buckling of the beam flange

(b) Fractured profile decking

(c) T16 reinforcement fracture

(d) Fractured angle

(e) Cracked concrete slab

Figure 7.15 Failure mode of Specimen S-W-9

Similar to the previous tests, for Specimen S-W-9, Figure 7.16 shows the overall deformations of each beam can be approximated by a straight line. It indicates the formation of plastic hinges at the beam-column connections.

(a) Before the test

(b) After the test

Figure 7.16 Overall deformation of Specimen S-W-9

7.3.5 Specimen S-F-12

Figure 7.17 (a) shows that for Specimen S-F-12, the load increase caused by catenary action was not significant. At a displacement of 32 mm, T16 reinforcing bars yielded, followed by the fracture of profile decking at a displacement of 60 mm, as shown in Figure 7.18 (b). Similar to Specimen S-W-9, fracture of profile decking

caused a slight decrease in the vertical load, as shown in Figure 7.17 (a). Local buckling of the beam flange shown in Figure 7.18 (a), was also observed before the formation of catenary action. It can be seen from this figure that the local buckling phenomenon of this specimen is more significant than Specimen S-W-9. This is because in this specimen, due to a greater bending moment resistance, the steel beam flange sustained a higher compression force. The load resistance of this specimen was governed by T16 reinforcement fracture at a displacement of 298 mm, followed by bolt thread stripping at a displacement of 418 mm. As shown in Figure 7.17 (b), different from the other tested specimens, during the whole loading process, no compression force was formed in this specimen. Catenary action started forming at a displacement of 100 mm, earlier than the other specimens. This is because at the flexural action stage, local buckling of steel beam flange and web in compression occurred. It means the compressive resistance of the connection has been reached quickly and this connection could not sustain further compressive forces. Thus, compressive arch action could not be formed. In addition, the gap between the support pins and pin holes also affected the development of compressive arch action. Figure 7.19 shows the relationship between the horizontal restraint movement at the left-hand side and the middle column vertical displacement for Specimen S-F-12. It can be found that the left pin was pushed outwards at the initial loading stage. However, due to the presence of the gap, no compression forces developed. It is noticed that in Specimen S-W-9, fracture of profile decking promoted the formation of compressive arch action. However, in Specimen S-F-12, after the fracture of profile decking, the tensile force sustained by profile decking transferred onto the bottom bolt row in the steel flush end plate connection. Therefore, the compressive forces of beam flange and web did not reduce. Thus, the compressive component in this specimen did not develop compressive arch action.

(a) Vertical force-middle column displacement curve

(b) Left beam axial force-middle column displacement curve

(c) Left connection moment-middle column displacement curve

Figure 7.17 Vertical force-middle column displacement curve of Specimen S-F-12

(a) Local buckling of the beam flange

(b) Fractured profile decking

(c) Reinforcement fracture

(d) Bolt thread stripping

(e) Cracked concrete slab

Figure 7.18 Failure mode of Specimen S-F-12

Figure 7.19 Horizontal restraint movement at left-hand side-middle column
displacement curve of Specimen S-F-12

7.4 Discussion about the experimental results

7.4.1 Comparison between web cleat and strengthened web cleat connections

A comparison between web cleat and strengthened web cleat connections is shown in Figure 7.20. Compared with Specimen M-W-9, a greater load-carrying capacity was observed in Specimen M-SW-11. In addition, the joint rotation capacity and maximum beam axial force of Specimen M-SW-11 are also greater than Specimen M-W-9, as shown in Table 7.3. This is due to the greater deformation capacity of the strengthened web cleat connection. In the flexural action stage, the load-carrying capacity of Specimen M-SW-11 is also higher than Specimen M-W-9. This is due to the formation of higher compressive arch action in Specimen M-SW-11, as shown in Figures 7.7 (b) and 7.10 (b).

Figure 7.20 Effect of the strengthened web cleat connection

7.4.2 Effect of composite slab

Chapter 3 presented seven bare steel experimental tests on the performance of common types of bolted steel beam-column joints subjected to catenary action. In this experimental programme, Specimens 1 (web cleat connection) and 5 (flush end plate connection) (Yang and Tan 2011) have similar configurations with the tested composite specimens in the present study. Thus, the effect of composite slab can be elucidated through the comparisons of bare steel specimens against the current composite tests. Figures 7.21 and 7.22 show the comparison of web cleat connections and flush end plate connections, respectively. It can be concluded from these two figures that composite slabs could increase the load-carrying capacities of beam-column joints at both flexural action and catenary action stages. In addition, for middle and side joints, the obtained load-displacement curves are quite different. For side joints, the composite slabs were in tension and the steel connections in compression. In the composite specimens, four T16 reinforcing bars were used in the composite slabs, therefore the slabs had sufficient tensile resistances. For side joints, the compression forces were mainly transferred by the steel beam flanges, as shown in Figures 7.15 (a) and 7.18 (a). However, for middle joints, composite slabs were in compression and steel connections were in tension. As shown in Figure 7.23 (a), at middle joint of web cleat connections, the tensile force is sustained by the steel angle and the compressive force is sustained by concrete. Figure 7.23 (b) shows that at side joint, the tensile force is mainly sustained by the reinforcing bars in the slab and the compressive force is sustained by the steel beam flange. Based on this, it can be found from Figure 7.23 that the lever arm of the side joint is much larger than the middle joint. This is the reason that at flexural action stage, the load-carrying capacities of side joints were much greater than middle joints.

Figure 7.21 Effect of composite slab for web cleat connections

Figure 7.22 Effect of composite slab for flush end plate connections

(a) Middle joint

(b) Side joint

Figure 7.23 Lever arms of middle and side joints for web cleat connections

7.4.3 Strain measurements

Specimen M-SW-11 is a typical example to show the strain variations of the specimens representing middle joints, including Specimens M-W-9, M-SW-11 and M-F-12. The measured strains of T16 reinforcing bars, profile decking and steel beam flanges are shown in Figures 7.24 (a), (b) and (c), respectively. It can be

found from Figure 7.24 (a) that the reinforcing bars located at Section L-C were in tension at the initial loading stage. As shown in Figure 7.24 (b), the profile decking at Section L-C was also in tension at small deformation. It demonstrates that for the tested composite joints under sagging moment, at small deformation stage, the neutral axes were located above the T16 reinforcing bars in composite slabs. At a vertical displacement of 100 mm, longitudinal strain in the profile decking decreased due to fracture of profile decking itself, which is shown in Figure 7.11 (a). At Section L-C, when the concrete crushed in compression, the T16 reinforcing bars changed from tension to compression, as shown in Figure 7.24 (a). It means after concrete crushing, the neutral axes moved downwards below the T16 reinforcing bars and the reinforcing bars sustained compression forces to resist bending moment at the beam-column connection. At Section L-M, the reinforcing bars were always in compression before the formation of catenary action. It means at Section L-M, the neutral axis was always located below the T16 reinforcing bars at flexural action stage. This is because at this section, most of the tensile forces have been transferred onto the steel beams through composite effect. Figure 7.24 (c) shows that the steel beam bottom flange was in tension during the whole loading process and the top steel beam flange was in compression before the development of catenary action. This indicates that the neutral axis of Section L-M was located in the steel beam web at flexural action stage. Finally, at catenary action stage, the reinforcing bars at Sections L-C and L-M as well as the steel beam top flange at Section L-M, changed from compression to tension.

(a) T16 reinforcing bars

(b) Profile decking

(c) Steel beam flange

Figure 7.24 Strain measurements in Specimen M-SW-11

Specimen S-W-9 is a typical example to show the strain variations of the specimens representing side joints, including Specimens S-W-9 and S-F-12. The strains of T16 reinforcing bars, profile decking and steel beam flanges are shown in Figures 7.25 (a), (b) and (c), respectively. It can be found from Figures 7.25 (a) and (b) that, at Section L-C the reinforcing bars and profile decking quickly yielded at the initial loading stage. This is because in this specimen, the beam-column joint was subjected to a hogging moment and the composite slab was in tension under this loading condition. It can be observed from Figure 7.25 (c) that the steel beam bottom flange was in tension at Section L-M during the whole loading process while the steel beam top flange was in compression at the initial loading stage. However, at catenary action stage, the bottom flange changed from compression to tension. This indicates that under catenary action, the whole section of the composite connection was in tension.

(a) T16 reinforcement

(b) Profile decking

(c) Steel beam flange

Figure 7.25 Strain measurements in Specimen S-W-9

7.5 Component-based modelling of composite beam-column joints

In this section, component-based modelling of composite beam-column joints will be presented. The behaviour of composite beam-column joints under a middle-column-removal scenario can be predicted by these models. The experimental results presented in the previous sections provide essential data for the development and validation of the proposed component-based models.

7.5.1 Introduction to the component properties

The proposed component-based models of composite beam-column joints are shown in Figure 7.26. The components considered include bolted-angles in tension, T-stubs in tension, profile decking in tension, beam flange and web in compression,

concrete in compression, and reinforcing bars in tension and in compression. In this set of five experimental tests on composite beam-column joints, no shear failure was observed. Thus, in the component-based models, a rigid shear spring to transfer vertical shear forces is used. In order to predict total failure of composite connections, deformation capacity of each connection component should be identified and introduced into the component-based models.

Figure 7.26 Component-based models of composite beam-column joints

a. Bolted-angles in tension

To obtain the real behaviour of bolted-angle connections in tension, the two types of bolted-angle connections used in Specimens M-W-9, M-SW-11 and S-W-9 have

been tested under pure tension. These two component test specimens were named by W-9 and SW-11. The materials and configurations of the component tests were identical to the composite specimens presented in Section 7.2.3. The component test specimens were subjected to a monotonic tensile force, which was applied to vertical steel plates that were clamped to the testing machine, as shown in Figure 7.27. The tests were carried out under displacement control at a speed of 0.025 mm/s up to failure of the specimens.

Figure 7.27 Component test set-up

The obtained load-displacement curves of these two component test specimens are shown in Figure 7.28. It can be found from this figure that these two connections have almost the same load-carrying resistance. However, Specimen SW-11 has a much higher deformation capacity compared to Specimen W-9. The greater deformation capacity is beneficial to the formation of catenary action in beam-column joint tests. This is the reason that Specimen M-SW-11 has a much higher load-carrying capacity than Specimen M-W-9 at catenary action stage, which is shown in Figure 7.20. The failure modes of these two component tests are shown in

Figure 7.29. Angle fractures close to the heel governed the final failure, which is identical to that of the composite specimen tests.

Figure 7.28 Load-displacement curves of the two component tests and Yang's model (2012b)

A mechanical model of bolted-angle connections under tension was proposed in Chapter 5 (Yang and Tan 2012b). This model has been used to simulate the component tests. The predictions of this model are shown in Figure 7.28. In general, this mechanical model gives acceptable predictions of the test behaviour. The predictions of this model will also be used in the component-based simulations of the composite beam-column joint tests.

(a) W-9

(b) SW-11

Figure 7.29 Failure modes of the two component tests

b. T-stubs in tension

Flush end plate connections are represented by equivalent T-stubs in tension in the component-based models. Piluso et al. (2001) proposed a mechanical model to

predict the deformation capacities of bolted T-stubs in tension. Piluso's model is used in this study.

c. Profile decking in tension

Figure 7.30 Stress-strain curve of the profile decking

In the composite joint specimens, the profile decking ribs were parallel to the steel beams. It was observed in the tests that profile decking sustained large tensile forces and finally fractured in tension. Thus, in the proposed component-based models, profile decking is included as a tensile component. The tensile resistance of this component is determined by the tensile capacity of the profile decking at beam-column interface. It should be mentioned that in the profile decking floor system, there was a rectangular opening for the vertical column. Therefore, the tensile resistance of the profile decking is reduced due to the presence of this opening. Figure 7.30 shows the stress-strain curve of the profile decking, obtained from a coupon test. Clearly, this material behaves in an elastic-perfectly plastic manner. Therefore, the component of profile decking in tension is represented by an elasto-plastic element. The elastic stiffness of this component is determined by the

Young's modulus of the profile decking. Once the resistance of this component is reached, it goes into a perfectly plastic stage. The experimental results demonstrate that the deformation capacity of this component is 2 mm. When the deformation of this component is greater than 2 mm, the resistance is defined to be zero.

d. Beam flange and web in compression

As shown in Figures 7.15 (a) and 7.18 (a), local buckling of beam flange and web was observed in Specimens S-W-9 and S-F-12. Therefore, beam flange and web in compression is one of the most critical components in composite beam-column joints. EC3-1-8 (2005) and Faella et al. (2000) proposed a rigid-plastic element to represent the behaviour of this component. In their models, the resistance of this component is governed by the bending moment resistance of the beam section. However, no information was given about the deformation capacity of this component in their models. Gioncu and Petcu (1997) proposed a plastic collapse mechanism in order to determine the ultimate plastic rotations and the available rotational capacities of steel and composite sections, as shown in Figure 7.31.

Figure 7.31 Post-Critical Curve of Plastic Collapse Mechanism (Dharma 2007)

Figure 7.32 Component model of beam flange and web in compression

In this study, Gioncu and Petcu's model has been used to predict the post-buckling curve of the beam flange and web in compression. The proposed mechanical model of beam flange and web in compression is given in Figure 7.32. This mechanical model can be divided into three stages, namely, elastic stage, strain hardening stage and local buckling stage. At elastic stage, the loading resistance F_l is determined by the yield strength of the steel beam and the yield displacement δ_l is determined by

$$\delta_l = \varepsilon_y \times l \quad (7.1)$$

where ε_y is the yield strain of the steel material and l is the effective length of the yield zone in beam flange and web. As shown in Figure 7.33, in Gioncu & Petcu's model, the effective length of the yield zone of beam flange is proposed as

$$l = \beta c + \zeta c \quad (7.2)$$

where c is one-half of the steel beam flange width, and β and ζ are two coefficients to determine the dimensions of yield lines and plastic zones.

Figure 7.33 Plastic Collapse Mechanism Model (Gioncu and Petcu 1997)

At strain hardening stage, the deformation is $\delta_1 \leq \delta \leq \delta_2$. As shown in Figure 7.33, only the plastic zones go into strain hardening stage. Thus, the increase in resistance will only consider the contribution from plastic zones. In addition, the deformation limit of this stage (δ_2) is determined based on the plastic deformation of the plastic zones. At local buckling stage, the load-displacement curve follows Gioncu and Petcu's model. The moment-rotation curve could be predicted by this model. In this study, it is assumed at local buckling stage, the rotation centre is located at the level of the reinforcing bars. This assumption is reasonable since at this stage, the moment resistance is governed by the compressive component, and the compressive resistance of this component would decrease slowly at the local buckling stage. Based on this assumption, the bending moment-rotation curve obtained from Gioncu and Petcu's model can be converted as a force-displacement curve. In order to simplify the model, a bi-linear curve is used at the post-peak final stage, as shown in Figure 7.32.

e. Concrete in compression

The compressive forces sustained by concrete balance the tensile forces at beam-column connections. Thus, the resistance of concrete in compression should be determined by the resistances of tensile components. During the whole loading process, the internal forces of the tensile components at beam-column connections do not remain constant. Therefore, the compressive forces sustained by concrete are also changing. It means the position of the concrete component will shift. This phenomenon has been observed by strain gauge readings. In this study, it is assumed that the compressive resistance of concrete in compression depends on the tensile resistances of steel connection components and profile decking. The contributions from reinforcing bars are ignored since the position of the reinforcement is too close to the neutral axis. Based on this assumption, the height of the concrete compression zone is determined. Thus, the position of this component can also be predicted.

From concrete cylinder tests, as shown in Figure 7.6, it is found that the maximum stress is reached when the concrete compressive strain is about 0.002. It was observed from the experimental tests that the length of the crushed concrete in beam-column joints was about $3h_c$, where h_c is the middle column section depth. Therefore, in this study, it is assumed that the effective length of the concrete in compression is equal to $1.5h_c$ in one beam-column connection. If the deformation of this component is greater than $0.003h_c$, concrete starts to crush. EC2-1-2 (2004) recommends that the ultimate compressive strain of concrete is 0.02 at ambient temperature. Thus, it is assumed that the concrete component loses the resistance completely when the deformation of this component is greater than $0.03h_c$. The component model of concrete in compression is shown in Figure 7.34.

Figure 7.34 Component properties of concrete in compression

f. Reinforcing bars in tension and in compression

As shown in Figures 7.15 (e) and 7.18 (e), major cracks were observed at beam-column connections when composite slabs were in tension. These major cracks caused concrete splitting and spalling, resulting in bond deterioration in the vicinity of the crack zones. The bond deterioration zone is shown in Figure 7.35. The bond deterioration length is assumed to be equal to $h_c/2 + 5d$, where h_c is the column depth and d the rebar diameter. This assumption is based on Maekawa's model (Maekawa et al., 2003). Since in this zone, concrete bond stress has limited influence on the behaviour of reinforcement bars, the rebar deformation capacity within the bond deterioration length can be derived from the rebar material tests. Beyond the bond deterioration zone, the deformation of rebar in tension is based on a bond-slip model proposed by Yu (2012). This bar stress-slip model is based on the assumption of respective uniform bond stress over the elastic and inelastic development lengths of a reinforcing bar. Therefore, the component deformation of reinforcing bars in tension is the total deformation in the bond deterioration zone and normal bar bond-slip zone. The component resistance of rebar in tension is based on the rebar material test results.

Figure 7.35 Bond deterioration close to cracks

For the component of reinforcing bars in compression, the deformation is based on Yu's model (2012) and the component resistance is also obtained from rebar material test results.

7.5.2 Simulation of the tests using the component-based models

The proposed models have been validated through comparisons of the simulated and observed responses of composite beam-column joint tests (Section 7.3). The horizontal restraint stiffness should be introduced into the component-based models in order to simulate the actual boundary conditions. It is found from the experimental tests that the horizontal restraint stiffness is about 1.1675×10^5 kN/m. A spring with a stiffness of 1.1675×10^5 kN/m has been used at the beam end in the

developed component-based models in order to represent the real restraint in the test set-up.

Figures 7.36, 7.37, 7.38, 7.39 and 7.40 show the simulated responses of five composite beam-column joints under a middle-column-removal scenario with the experimental results in terms of (a) vertical forces, (b) beam axial forces and (c) connection bending moments versus middle column displacement relationship. For web cleat and strengthened web cleat connections, two bolted-angle connection models, namely, component test model and Yang's model (Yang and Tan 2012b), have been incorporated into the component-based models and compared with the joint test results. The component test model is based on the load-displacement curves obtained from component tests. The component properties of Yang's model (Yang and Tan 2012b) are based on the predicted load-displacement relationship, as shown in Figure 7.28. From the comparisons it can be found that the proposed component-based models can capture the essential structural behaviour with satisfactory accuracy. Yang's model (Yang and Tan 2012b) can predict similar responses of composite joints. This indicates that this component model of bolted-angle connections can be used in composite steel structures. It should be pointed out that there are some differences between the simulation results of Specimen M-F-12 and the test observations. This is because the actual failure mode of this specimen was welding fracture of steel beam web. However, in the component-based model, this failure mode was not considered (Piluso et al. 2001). Another main reason for the differences is the existing gaps in the test set-up, as shown in Figure 7.1. These connection gaps have great influence on the formation of compressive arch action. As shown in Figures 7.36 (b) and 7.37 (b), the predicted compressive forces in beams are much smaller than the measured values. Fortunately, in the conducted joint tests, the influence of compressive arch action is not critical and the failures of the specimens always occurred at catenary action stage. In addition, for each test,

the gaps between the pins and holes varied from specimen to specimen. For the convenience of explanations, these gaps were not included in the simulations. In general, these component-based models yield acceptable predictions of the test behaviour.

(a) Vertical force-middle column displacement curves

(b) Beam axial force-middle column displacement curves

(c) Left connection moment-middle column displacement curves

Figure 7.36 Comparison of the analytical predictions against the test data of
Specimen M-W-9

(a) Vertical force-middle column displacement curves

(b) Beam axial force-middle column displacement curves

(c) Left connection moment-middle column displacement curves

Figure 7.37 Comparison of the analytical predictions against the test data of
Specimen M-SW-11

(a) Vertical force-middle column displacement curves

(b) Beam axial force-middle column displacement curves

(c) Left connection moment-middle column displacement curves

Figure 7.38 Comparison of the analytical predictions against the test data of
Specimen M-F-12

(a) Vertical force-middle column displacement curves

(b) Beam axial force-middle column displacement curves

(c) Left connection moment-middle column displacement curves

Figure 7.39 Comparison of the analytical predictions against the test data of
Specimen S-W-9

(a) Vertical force-middle column displacement curves

(b) Beam axial force-middle column displacement curves

(c) Left connection moment-middle column displacement curves

Figure 7.40 Comparison of the analytical predictions against the test data of
Specimen S-F-12

7.6 Frame analyses

Having gained the confidence in the reliability of the component-based models, frame analyses are carried out by incorporating the proposed joint models into the frame analyses. One of the main objectives of the frame analyses is to identify the difference in structural performances between isolated joint and frame models under middle-column-removal scenarios. In addition, parametric studies have been carried out to investigate the effects of reinforcement ratios, profile decking and composite slabs. Finally, based on Izzuddin's model (Izzuddin, Vlassis et al. 2008), dynamic effect is considered and dynamic load-carrying capacities of composite frames are obtained, which can be used directly in the design of composite steel structures to prevent progressive collapse.

7.6.1 Discussion about frame and joint models

The prototype composite frame is shown in Figure 7.41 (a). In the *frame model* shown in Figure 7.41 (b), the entire two beam spans and four connections at these two spans are modelled. The two columns at the two sides are restrained by surrounding structures. The connections are simulated by the developed component-based models, presented in Section 7.5. In the experimental tests and the *joint models*, an inflection point was assumed to be located at the middle of the beam span and only one-half of the beam span at both sides of the beam-column joint was used, as shown in Figure 7.41 (c). However, due to the presence of composite slabs, the inflection point may not be located at the beam middle-span. The developed frame model will be used to study the interaction between the middle and the side joints and the load redistribution after the removal of the middle column. In the analyses of this section, one type of connection has been chosen as an example to demonstrate these effects. The chosen connection type has the same configuration with the web cleat connection in Figure 7.5 (b). For this type of connections, analytical results of the joint model have been presented in Section 7.5, and these results will be used to compare with the frame model.

Figure 7.41 Frame model and joint model with composite beam-column joints

The numerical results of the frame model are compared with the joint models in Figure 7.42. It should be mentioned that the vertical displacements of the middle columns in the frame model are only one-half of the actual displacement from anti-symmetric assumption about Point E (F) as shown in Figure 7.41, for convenience in comparison. It can be found from Figure 7.42 (a) that at flexural action stage, the frame model has different predictions with the middle and side joint models. This is because at this stage, the inflection point is not located at the beam mid-span, which has been explicitly assumed in the joint models. The flexural resistance of the frame model is smaller than the side joint model and larger than the middle joint model. However, it can be found from Figure 7.42 (c) that in the frame model, the bending

moments of middle and side joints are similar to the joint models. This indicates that the flexural resistances of middle and side joints will not affect each other in the frame model and the inflection point assumption does not affect the flexural resistances of the isolated joints. The analytical results of the middle and side joints indicate that the flexural resistance of the middle joint is only half of the side joint. Therefore, in the frame model, the inflection point should be located at one-third of the beam span at the flexural stage. At catenary action stage, the load-carrying capacity is close to the middle joint model. This is probably because at this stage, in the frame model, the tensile forces developed at the middle and side joints are equal to each other. Thus, in the frame model, the increase of load-carrying capacity contributed by catenary action will be governed by the smaller tensile resistance of the middle or the side joints.

(a) Vertical force-middle column displacement curves

(b) Beam axial force-middle column displacement curves

(c) Connection moment-middle column displacement curves

Figure 7.42 Comparison of the composite frame models against the test results and joint models

7.6.2 Effects of reinforcement ratios, profile decking and composite slabs

Based on the frame model presented in Section 7.6.1, further parametric studies have been conducted to investigate the effects of reinforcement ratio, profile decking and composite slab. The beneficial effect of additional reinforcement to increasing the frame load-carrying capacities is illustrated in Figure 7.43. It should be noted that the 1.3% reinforcement ratio corresponds to the composite specimens presented in Section 7.3. Figure 7.43 shows that the increase of the reinforcement ratio causes higher load resistances at flexural action stage as well as catenary action stage.

Figure 7.43 Effect of reinforcement ratio in the composite frame model

In order to study the effect of profile decking on the behaviour of composite frames, a frame analysis without profile decking has been conducted and the results are

shown in Figure 7.44. It can be found that the presence of profile decking can increase the load-carrying capacity at flexural action stage. However, at large deformation stage, no difference is observed between the two frame models with or without the profile decking. This is because the profile decking has limited deformation capacity and fractured at small deformation stage. As shown in Figure 7.44, at a displacement of 180 mm, profile decking fractured at side joints. In addition, the effect of composite slabs has been studied by conducting a bare steel frame simulation. As shown in Figure 7.44, the steel frame can develop catenary action fully. Thus, the ultimate load resistance of the steel frame is close to the composite frame. However, at flexural action stage, the composite frame could form much higher load-carrying capacity than the steel frame. This is due to beneficial effect of the composite slab.

Figure 7.44 Effects of profile decking and composite slabs in the composite frame model

7.6.3 Dynamic effect

It should be noted that the previous experimental and numerical tests are all based on nonlinear static responses of steel and composite structures. This assumption is only reasonable when the column is damaged slowly. However, under a sudden-column-removal scenario like blast loading conditions, the building structures exhibit dynamic responses. Izzuddin (Izzuddin, Vlassis et al. 2008) proposed a simplified approach to assess the dynamic effect of building structures based on nonlinear static responses. In this study, Izzuddin's model has been used to obtain the dynamic resistances of steel and composite frames under a sudden-column-removal scenario.

Figure 7.45 Dynamic effects of steel and composite frames under a sudden-column-removal scenario

Figure 7.45 shows the dynamic effects of steel and composite frames. It can be found that the composite frame can sustain much higher dynamic resistance than the steel frame although the difference in the static resistances of steel and composite frame is not so great. This is because the composite frame could form much higher plastic deformation and absorb more energy. Thus, the associated dynamic increase factor of the composite frame is much lower than that in the steel frame.

Under slow-column-removal scenarios, the static resistances of steel and composite frames can be used to design a building structure to prevent progressive collapse. Under sudden-column-removal scenarios like blast and impact loads, the dynamic resistances should be applied to design the building against progressive collapse.

7.7 Conclusions

In this chapter, five experimental tests were conducted to investigate the behaviour of composite beam-column joints under a middle-column-removal scenario. Composite web cleat and flush end plate connections were studied. Two types of joints including middle and side joints were both tested to failure. Component-based models are also developed to simulate the test results. Finally, further parametric studies were conducted. The following conclusions can be drawn:

- (1) The test results demonstrated the ductility and load resistances of these five specimens in catenary action mode. The test results indicate that at initial loading stage, the applied load was resisted by flexural action while at large deformation stage, the load was resisted by catenary action;
- (2) The strengthened web cleat connection has a much higher load-carrying capacity than the normal web cleat connection because the former could sustain greater deformation;
- (3) Composite slabs could increase the load-carrying capacities of beam-column

joints at both flexural action and catenary action stages. For middle and side joints, the obtained load-displacement curves are quite different. The lever arms of middle joints are much smaller than side joints. This is the reason that at flexural action stage, the load-carrying capacities of side joints were much greater than middle joints;

- (4) Component-based models of composite beam-column joints under a middle-column-removal scenario have been proposed. Failure criteria have been introduced for most connection components, which enable the component-based models to accurately predict the total failures of the composite joints. In general, these component-based models yield acceptable predictions of the composite beam-column joint behaviour under a middle-column-removal scenario;
- (5) By using the developed component-based beam-column joint models, frame analyses indicate that the increase of the reinforcement ratio causes higher load resistances at flexural action stage as well as catenary action stage;
- (6) It can be found that the presence of profile decking can increase the load-carrying capacity at flexural action stage. However, at large deformation stage, no difference is observed at the two frame models with and without profile decking;
- (7) Further parametric study also shows that the ultimate load resistance of the steel frame is close to the composite frame. However, at flexural action stage, the composite frame could form much higher load-carrying capacity than the steel frame;
- (8) Based on Izzuddin's model, it is found that the composite frame can sustain much higher dynamic resistance than the steel frame although the difference of the static resistances of steel and composite frame is not so great;
- (9) It should be mentioned that the experimental and analytical studies of the composite joint tests were conducted at a 2-D frame level. However, under a

column-removal scenario, composite slabs may contribute to prevent progressive collapse through the development of tensile membrane action. As a continuation to this research, it is recommended that the behaviour of composite slabs should be estimated by further experimental tests and numerical simulations.

CHAPTER 8 CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

This research investigated the behaviour of steel and composite beam-column joints under middle-column-removal scenarios and proposed component-based models to simulate the performance of beam-column joints.

Firstly, an experimental tests of bare steel beam-column joints under a middle-column-removal scenario was conducted. Seven experimental tests have been carried out on different types of steel beam-column joints, including simple and semi-rigid connections. In the group of simple connections, web cleat, top and seat angle, top and seat with web angle (TSWA) (8 mm thickness angles) and fin plate connections were investigated while flush end plate, extended end plate and TSWA (12 mm thickness angles) constituted the group of semi-rigid connections. The test results demonstrate the contribution of catenary action to the load resistances of various types of steel beam-column joints and conclude that under the middle-column-removal scenario, catenary action can be considered to increase the load capacity. For web cleat and fin plate connections, under the middle-column-removal scenario, only a limited load could be applied onto the joints at the initial loading stage and the applied load increased significantly at the large deformation stage due to catenary action. Therefore, the behaviour of these two joints was dominated by catenary action. For top and seat angle and extended end plate connections, flexural action dominates the behaviour of the joints in the early stage of the response. There was little load contribution from catenary action. For flush end plate and TSWA (8 and 12 mm angle) connections, significant flexural actions were observed and catenary action could also develop well at large deformation stage. In the group of simple connections, the web cleat connection has the best combination of desirable features: ability to develop catenary action and extremely high ductility (rotational capacity) through deformation of the web angles. The TSWA (8 mm angle) connection has a similar behaviour with the web cleat connection. In the group of

semi-rigid connections, TSWA (12 mm angle) achieved the highest load and rotation capacities whereas flush end plate could also mobilize catenary action well. It is worthy to note that tensile capacities of beam-column joints after undergoing large rotations usually control the failure mode and the formation of catenary action. This implies that engineers should adopt high tensile resistances of beam-column joints after undergoing large rotations rather than pure tying resistance. A new tying resistance expression is proposed to consider the effect of large rotation. If large rotation is not considered in the design stage, the joints with poor rotation capacities would fail to achieve the design tying resistances.

Then these tests have been modelled using the general finite element (FE) package ABAQUS (2009). Both static and explicit dynamic solvers were employed to overcome the problems of convergence, contact, large deformation and fracture simulations. From the numerical analyses it is found that a static solver could usually predict more accurate simulation results than an explicit dynamic solver, although for some cases almost the same predictions have been observed for both solvers. The problem of numerical non-convergence will occur when the static solver is employed and certain fracture simulations could not run to the end while complete fracture simulations could be conducted by the explicit dynamic solver. However, huge computation resources are required by the explicit dynamic solver for complicated models. The predictions of the finite element models have been validated with experimental results. It is shown that the finite element models correlate well with test results including the observed failure modes. As a result of the success of applying finite element analyses, parametric studies have been conducted using these numerical models to obtain the rotation capacities of various types of connections under catenary action. Current acceptance criteria of rotation capacities for steel joints such as web cleat, fin plate, flush end plate and TSWA connections, are probably too conservative as they only consider pure flexural resistance. Four new connection acceptance criterion of rotation capacities have been proposed to consider catenary action under a middle-column-removal scenario.

An experimental investigation of bolted-angle connections under tension was conducted in order to investigate the deformation capacities of this type of connections. The test results indicate that the ultimate strength of bolted-angle connections is much greater than the yield strength and the load-displacement responses of bolted-angle connections were dominated by large deformation rather than by yield capacities of bolts or angles. Based on the experimental data, a component-based model has been developed to predict the behaviour of bolted-angle connections under pure tension. In this model, the deformation capacities of connection components can be predicted. Overall speaking, the proposed mechanical model gives the best prediction among all the considered models.

In order to achieve a better understanding on the performance of bolted-angle connections subjected to catenary action under a middle-column-removal scenario, nine experimental tests were carried out on bolted-angle beam-column joints. Three types of connections, including web cleat, top and seat angle and TSWA connections were investigated using three angle thicknesses (8, 10 and 12 mm). For bolted-angle connections, the test results indicate an increase of angle thickness will alter the failure mode. With increasing angle thickness, there are three corresponding failure modes, namely, angle fracture at the heel, angle fracture with yielded bolts and bolt fracture with yielded angles. A component-based model is also developed to simulate the test results. The proposed component-based models can represent the key responses of bolted-angle beam-column joints under a middle-column-removal scenario, including the formation of flexural action at small deformation stage, the development of catenary action at large deformation stage and fractures of the connection components at the last stage. By using the developed component-based beam-column joint models, frame analyses indicate that the conducted joint tests can represent the behaviour of a prototype steel frame. The obtained connection rotation angles, loading resistances and internal beam axial forces from the joint tests are in close agreement with the frame model. A parametric study indicates that the influence of the horizontal restraint stiffness on the behaviour of steel frames is very significant if the horizontal restraint stiffness is smaller than the frame stiffness under tension.

Finally, five experimental tests were conducted to investigate the behaviour of composite beam-column joints under a middle-column-removal scenario. Composite web cleat and flush end plate connections were studied. Two types of joints including middle and side joints were both tested. The test results demonstrate that composite slabs could increase the load-carrying capacities of beam-column joints at both flexural action and catenary action stages. For middle and side joints, the obtained load-displacement curves are quite different. The lever arms of middle joints are much smaller than side joints. This is the reason that at flexural action stage, the load-carrying capacities of side joints were much greater than middle joints. Component-based models are also developed to simulate the test results. Failure criteria have been introduced for most connection components, which enable the component-based models to predict the total failures of the composite joints. In general, these component-based models have given acceptable predictions of the composite beam-column joint behaviour under a middle-column-removal scenario. By using the developed component-based beam-column joint models, frame analyses indicate that the increase of the reinforcement ratio causes higher load resistances at flexural action stage as well as catenary action stage. Further parametric study also shows that the ultimate load resistance of the steel frame is close to the composite frame. However, at flexural action stage, the composite frame could form much higher load-carrying capacity than the steel frame. In addition, dynamic effect was considered. Based on Izzuddin's model, it is found that the composite frame can sustain much higher dynamic resistance than the steel frame although the difference of the static resistances of steel and composite frame is not so great.

8.2 Recommendations for future work

The followings are some recommendations for future research:

- (1) The numerical predictions indicate that composite frame behaviour is different from isolated composite joints. Therefore, it is necessary to conduct composite frame tests under column-removal scenarios to validate the numerical simulations;

- (2) The steel and composite joint tests that were carried out in the present research have been limited to middle-column-removal scenarios. Therefore, it is proposed that additional tests be conducted in the future which includes other column-removal scenarios like corner column;
- (3) The connection types studied in this thesis are limited. Further investigations are needed to develop the component-based models for other types of connections, which have not been assessed, such as welded connections;
- (4) In this thesis, the experimental and analytical studies were conducted at a 2-D frame level. However, under a column-removal scenario, composite slabs may contribute to prevent progressive collapse through the development of tensile membrane action. As a continuation to this research, it is recommended that the behaviour of composite slabs should be estimated by further experimental tests and numerical simulations;
- (5) In the research works of this thesis, only a short discussion is presented in Section 7.6.3 about the dynamic effect under sudden-column-removal scenarios. In the experimental tests conducted in this thesis, quasi-static loading rate was used and the dynamic effect was not considered. Further experimental and analytical works are needed to study the dynamic effect in detail.

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