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STRUCTURAL BEHAVIOUR OF COMPOSITE FLOOR SYSTEMS UNDER COLUMN REMOVAL SCENARIO

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**STRUCTURAL BEHAVIOUR OF COMPOSITE
FLOOR SYSTEMS UNDER COLUMN REMOVAL
SCENARIO**

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SUMMARY

Civilian buildings have been increasingly threatened by abnormal loads, such as terrorist attack and accidental gas explosion. Robustness of buildings should be ensured to prevent local failure from spreading to avoid disproportional casualties and economic losses. Alternate Path (AP) method, also known as the column removal scenario, is a feasible and conservative approach to assess the robustness of buildings.

In this study, four three-dimensional-steel-frame-composite-floor (3D composite floor) specimens were quasi-statically tested to failure under an internal column removal scenario. The specimens were all two-by-two-bay sub-structures, differentiating in terms of slab aspect ratio, degree of composite action between beams and slabs and column removal location (or boundary condition). Two principal objectives were achieved accompanied by some secondary but valuable findings through these system-level tests. Firstly, load-resisting mechanisms of 3D composite floor systems subjected to an internal column removal scenario were elucidated, through separating contributions of the double-span girder, double-span secondary beam and composite slabs, separating flexure and catenary action (CA), as well as evaluating the load redistribution among the remaining surrounding columns. The second primary achievement was the reveal of effects from slab aspect ratio, degree of composite action and boundary condition. The discussions of the effects were carried out on load-carrying capacity, ductility, failure mode, deformation characteristics, load redistribution among the remaining columns and reaction forces from surrounding structures.

Due to prohibitively high cost of system-level tests, numerical models were employed to study inter-dependence among load-resisting mechanisms in 3D composite floor systems. It is found that dominant flexure would suppress the development of CA, since they are the two complimentary components in the double-span girder and double-span beam. The slab contribution at the final stage involves TMA remains constant if steel reinforcement, profiled decking and dimensions of the slab are the same. Furthermore, the verified modelling method was applied to a full-scale 3D

composite floor system subjected to sudden column loss, yielding the dynamic behaviour. A series of dynamic results can be summarised into a pseudo-static response, which was compared with the corresponding static response to obtain the dynamic increase factors (DIFs). The obtained DIFs were then compared with those specified in DoD (2013) and those of 2D steel frames with the same connection types.

Last but not least, a mechanical model was proposed to estimate the entire load-deflection response of 3D composite floor systems subjected to an internal column loss. Compared with actual test results and numerical simulations, the model shows reasonable accuracy in predicting load-deflection responses and energy quantities stored in individual structural members including girders, secondary beams and slabs. Besides this novelty and distinct feature from other studies, the model can capture the effects of key parameters, such as slab aspect ratio, joint type, number of joint bolts, slab thickness, reinforcement ratio in the slab and thickness of steel decking. Most importantly, the procedure of the model can be implemented by a spreadsheet method, which provides a simple and numerical robust tool for engineers to calculate progressive collapse resistance of structures for a missing column scenario. Using this model, twelve composite floor systems were analysed to investigate the effects of joint type and slab aspect ratio on structural robustness. Some valuable conclusions were made accompanied by some recommendations for engineering practice.

LIST OF PUBLICATIONS

Journal papers

Fu, Q.N., Tan, K.H., Zhou, X.H. and Yang, B. (2017). “Numerical simulations on three-dimensional composite structural systems against progressive collapse”. *Journal of Constructional Steel Research*, **135**, 125-136.

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Conference papers

Fu, Q. N., Tan, K. H., and Yang, B. “3D slab effect of composite floor systems under internal column loss. 15th Academic Conference of Chinese Steel Structure Association on Steel-Concrete Composite Structures”, Chongqing University, Chongqing, China, October 16-18, 2015.

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Fu, Q. N., Tan, K. H., and Yang, B. “Experimental tests on 3D composite floor

systems after removal of an internal column", Eighth International Conference on Steel and Aluminium Structures. Hong Kong, China, December 7 – 9, 2016.

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LIST OF SYMBOLS

A_D	area of bottom horizontal legs of steel decking per unit width
A_s	tension steel area per unit width of slabs
$A_{s,i}$	area of the i -th steel layer (steel reinforcement or decking) per unit width
a	slab width
D	flexural rigidity of a plate
d_{br}	distance between the br -th component and centre line of the connected beam
d_s	effective depth of RC slab cross-section
d_{sp}	spacing of bars along crack direction
E	Young's modulus
E_c	elastic modulus of concrete
E_s	elastic modulus of steel
F_D	yield force in steel decking per unit width
F_R	yield force in reinforcing mesh per unit width
F_r	bolt-row force corresponding to the r -th bolt row
f_c'	compressive cylinder strength of concrete
$f_{ck,cube}$	compressive cube strength of concrete
f_u	ultimate strength of steel material
f_y	yield strength

f_{yR}	yield strength of steel reinforcement
$f_{y,b}$	yield strength of the secondary beam
$f_{y,D}$	yield strength of steel decking
h_b	depth of composite beam including slab
H_c	distance between the two measured sections
h_c	thickness of concrete topping
$h_{D,g}$	distance between steel decking layer and centre line of the girder
h_e	equivalent thickness of composite slab
h_g	depth of composite girder including slab
h_p	depth of profile sheeting
$h_{R,b}$	distance between reinforcement layer and centre line of the beam
$h_{R,g}$	distance between reinforcement layer and centre line of the girder
h_r	distance from the r -th bolt row to the compression centre at small deformation stage
h_s	overall composite slab thickness
$h_{s,i}$	distance between the i -th steel layer and N.A.
I_b	moment of inertia of steel beam
I_c	moment of inertia of slab cross-section per unit width
I_s	moment of inertia of the steel reinforcement and decking

$K_{\text{eff}, r}$	effective stiffness coefficient for the r -th bolt row
l	the shorter dimension of a slab panel
l_b	length of a beam
$M_{1,\text{ma}}, M_{1,\text{mi}}$	the respective bending moments about major and minor axes at Section 1 of a column
$M_{2,\text{ma}}, M_{2,\text{mi}}$	the respective bending moments about major and minor axes at Section 2 of a column
M_C	column foot bending moment acting in the analysed plane
M_p	plastic bending moment resistance of the secondary beam cross-section
m_s	positive bending moment resistance of composite slabs per unit width
m_{sx}, m_{sy}	positive bending moment resistance of composite slabs per unit width in the respective x - and y -directions
N_1, N_2	the respective axial forces at Section 1 and 2
N_B	axial force in a beam
N_c	average value of N_1 and N_2
N_R	axial force in a restrained CHS
n	ratio of steel-to-concrete elastic modulus
P_{dy}	dynamic load
P_i	magnitude of the i -th point load
P_{st}	static load

R_H, R_V	horizontal and vertical reaction forces, respectively
r	slab aspect ratio (≥ 1)
S_j	rotational stiffness of the j -th joint
u_{dy}, u_{st}	maximum dynamic and static displacements, respectively
V_B	shear force in a beam
V_C	shear force in a column
$V(x), V(y)$	respective shape functions of plates OGH and $OEFG$
v_i	vertical displacement at point i
ν_c	Poisson's ratio for concrete
$W_{c,ma}, W_{c,mi}$	the respective sectional modulus about the major and minor axes of a column section
W_{jc}	energy stored in all the joint components
W_{ex}	total external work
W_{in}	total internal strain energy
$W_{OH}, W_{OE}, W_{EF}, W_{FH}$	energy stored in steel reinforcement and decking at deformed regions along <u>OH</u> , OE , EF , and FH respectively
$W_{OE,R}, W_{OE,D}$	respective energy stored in reinforcing mesh and steel decking at deformed region along OE
$W_{OH,R}, W_{OH,D}$	respective energy stored in reinforcing mesh and steel decking deformed region along OH
W_p	plastic section modulus
W_{ph}	energy stored in all the plastic hinges

W_y	elastic section modulus
W_{yl}	energy stored in one yield line
w	magnitude of uniformly distributed load (UDL)
w_{slab}, w_{frame}	UDL carried by slab and skeletal frame, respectively
w_1, w_2	modal frequencies corresponding to the top two largest modal participation factors in loading direction
$x_{0,R}$	length of reinforcing mesh in compression in FH
$x_{0,D}$	length of steel decking in compression in FH
x_{lim}	the length of intact reinforcing mesh in FH
$y_{0,R}$	length of reinforcing mesh in compression in EF
y_{lim}	the length of intact reinforcing mesh in EF
ΔS_r	deformation corresponding to r -th bolt row
α	mass proportional damping coefficient
β	stiffness proportional damping coefficient
$\Delta_{hog,br}$	extension of the br -th joint component at the hogging end of girder, beam or inner beam
Δ_{lim}	extension limit of reinforcement
$\Delta_{OE,D}^y, \Delta_{FH,D}^y$	y -extensions of steel decking at deformed regions OE and FH respectively
$\Delta_{OE,R}^y, \Delta_{EF,R}^x, \Delta_{FH,R}^y, \Delta_{OH,R}^x$	extensions of reinforcement at deformed regions OE, EF, FH and OH respectively

$\Delta_{\text{sag},br}$	extension of the br -th joint component at the sagging end of girder (B1)
$\Delta_{\text{sag},B2}, \Delta_{\text{sag},B3}$	extension of joint components at the sagging end of beam (B2) and inner beam (B3) respectively
δ	the maximum vertical deflection of a slab
ε_{ave}	the average strain of $\varepsilon_a, \varepsilon_b, \varepsilon_c$ and ε_d
ε_i	strain at location I ($i=1,2,\dots,7,8, a,b,c,d$)
ε_u	uniform strain corresponding to the ultimate tensile strength of tested steel material
η	coefficient related to slab aspect ratio
θ	rotation of plate segments $OEFG$ and OGH ; rotation of B1, B2 and B3
θ_{pra}	plastic chord rotation of a steel beam
θ_x, θ_y	slab rotations about x - and y - directions at the yield line
θ_{yb}	yield rotation of a steel beam
ξ_1, ξ_2	the respective damping ratios of the first two modes
φ_j	rotation of the j -th joint
Ψ	percentage of yielded decking area

CHAPTER 1 INTRODUCTION

1.1 Background

Progressive collapse is defined as “the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or a disproportionately large part of it” in American Society of Civil Engineers Standard 7 (ASCE7-05 2005). GSA (2013) defines progressive collapse as an extent of damage or collapse that is disproportionate to the magnitude of an initiating event. Although various codes do not explain progressive collapse in the same way, all of them include two essential aspects. One is the initial local damage on structures caused by accidental loads, and the other is final large-scale failure, which is disproportionate to the magnitude of the triggering source and the initial damage.

After partial collapse of the Ronan Point apartment caused by gas explosion in 1968 (Fig. 1.1 (a)), researchers and engineers in Britain took the lead in the investigation on structural resistance against progressive collapse. Besides this tragic accident, there were two more symbolic progressive collapse events drawing great attention from researchers world-wide on this issue. As shown in Fig. 1.1 (b), the federal government office building in Alfred P. Murrah was attacked by a car bomb in 1995. This serious terrorist attack killed 168 people and wounded more than 800 persons in the US. Fig. 1.1 (c) shows the World Trade Centre attacked by two planes on 11 September 2001. Similar events happened around the world with different triggers. Once progressive collapse happened, economic losses and casualties could not be avoided. It may not be possible to prevent accidents from happening. However, structures can be designed to resist collapse triggered by local failure. Therefore, relevant design guides are currently being developed by structural engineers to increase robustness of buildings to avoid progressive collapse under abnormal loads. This calls for research studies on structural behaviour under such circumstances.

(a) Ronan Point apartment

(https://en.wikipedia.org/wiki/Ronan_Point)

(b) Alfred P. Murrah

(https://en.wikipedia.org/wiki/Oklahoma_City_bombing)

(c) World Trade Centre

(https://en.wikipedia.org/wiki/September_11_attacks)

Fig. 1.1: Iconic events of progressive collapse

1.2 Column Removal Scenario

GSA (2013) and DoD (2013) are the world-wide recognised engineering guides for anti-progressive collapse design. DoD (2013) incorporates tie force (TF) method, enhanced local resistance (ELR) method and alternate path (AP) method. TF method is an indirect reinforcement measure through specifying the minimum levels of strength, continuity and ductility to resist progressive collapse. ELR method aims to ensure the safety of key elements, including peripheral columns and walls on the first and second storeys, under a specific blast load scenario. However, it is difficult to accurately determine the required strength for key elements due to the random nature of terrorist attack. In AP method, it is assumed that one vertical structural member is removed, followed by redistribution of gravity loads, as shown in Fig. 1.2, and the remaining structure system is re-analysed under the updated internal forces. In the latest version of GSA (2013), TF and ELR methods have been removed and only AP method is employed. Therefore, AP method, also known as column removal scenario, becomes a conventional methodology to study robustness of buildings against progressive collapse.

It should be noted that column removal scenario can only be suitably applied in the buildings subjected to small charges of explosions (Jahromi et al. 2012, Dinu et al. 2018). For large weights, even under the same scaled distance, the direct damages (due to blast) may involve more structural elements besides the target column, such as the above joint, beams and slabs, thus can initiate progressive collapse. Besides, blast pressure can intensify dynamic increase effects when floors are lifted upward, which usually occurs in case of large charge weights and lower gravity loads. However, the difficulties and risk in developing real blast tests transferred most of the research to column loss tests (or similar) under static or dynamic conditions. Afresh, difficulties in considering blast effects for practical design lead the current anti-progressive design guides (GSA 2013 and DoD 2013) to adopt a column removal scenario for analyses.

GSA (2013) and DoD (2013) indicate locations of internal and external column removal to consider extreme events and consequences (Fig. 1.3).

Fig. 1.2: Typical alternate load paths

Fig. 1.3: Location of column removal (DoD 2013)

1.3 Previous Studies

There have been an increasing number of studies on progressive collapse, encompassing reinforced concrete (RC) structures, steel structures and composite steel-concrete structures. It should be mentioned that only steel structures and composite steel structures will be considered in this study. Under a column removal scenario with large deformation, upper-bound mechanisms such as Catenary Action (CA) in beams and Tensile Membrane Action (TMA) in slabs could be mobilised to transfer loads to surrounding structures to mitigate collapse, which are not considered in conventional routine structural design. Besides, structural mechanisms in a beam-slab system are so complex that they cannot be analysed in one step. Consequently, researchers investigate structural behaviour from different levels of analysis including joint components, two-dimensional (2D) frames, three-dimensional (3D) skeletal frames and 3D steel-frame-composite-floor systems (composite floor systems).

A few studies (Piluso et al. 2001a, Taib 2012, Yang and Tan 2013c) have proposed component models for commonly-used steel joints subjected to large deformation, which form very useful bases for higher levels of structural analysis. There are a great number of studies on 2D steel frames, including Khandelwal and El-Tawil

(2007), Khandelwal et al. (2008), Demonceau (2008), Lee et al. (2009), Yang and Tan (2012), Guo et al. (2013), Li et al. (2013), Lew et al. (2013), Liu et al. (2013), Yang and Tan (2013a), Yang and Tan (2014), Liu et al. (2015b), Yang et al. (2015), Fu et al. (2016), Liu et al. (2016), Yang et al. (2016) and Mashhadi and Saffari (2017). A few publications were on 3D steel skeletal frames (Dinu et al. 2016, Li et al. 2017). Recent research works indicate that the slab effect, especially TMA at large deflection stage, can influence the load redistribution mechanism and enhance progressive collapse resistance. To gain deeper insight into realistic structural behaviour, current research studies should focus more on 3D floor systems rather than simplified 2D or 3D skeletal frames which exclude the composite floor effect.

For 3D composite floor systems, there were a few numerical studies (Sadek et al. 2008, Alashker et al. 2010, Fu 2010, Alashker et al. 2011, Hoffman and Fahnestock 2011, Jahromi et al. 2013, Li and El-Tawil 2014, Main 2014, Song et al. 2014) and limited experimental tests (Tan and Astaneh-Asl 2003, Chen et al. 2012, Hull 2013, Song and Sezen 2013, Zandonini et al. 2014, Johnson et al. 2016). However, these numerical models of 3D composite floor systems had not been verified comprehensively by actual tests under column loss scenarios due to unavailable test data. As a result, these models could not identify failure modes so that they could not ensure conservatism or accuracy of simplifications on beam-to-column joints, steel decking, and shear stud connections between composite slabs and steel beams. Hence, experimental tests are essential for the research studies. Astaneh-Asl et al. (2001), Chen et al. (2012) and Johnson et al. (2016) have successfully conducted experimental tests under an edge column removal scenario and given valuable analyses based on test results. However, the only test specimen under a corner column removal scenario by Johnson et al. (2016) collapsed under an unexpectedly-low load so that detailed data has not been obtained. Hull (2013) and Johnson et al. (2016) have tested 3D composite floor systems under an internal column removal scenario, but the results were not satisfying for further analyses. Zandonini et al. (2014) have also tested a 3D steel-concrete sub-structure under an internal column removal scenario. However, the sub-structure was constructed with concrete solid slabs rather than commonly-used steel-decking-with-concrete slabs, which may result in different load transfer paths and crack patterns in slabs. Hence, it is necessary to conduct tests

on 3D composite floor systems under internal and corner column removal scenarios. This research just focuses on an internal (internal and penultimate internal) column removal scenario to achieve in-depth appreciations of structural behaviour of 3D composite floor systems.

Previously, although it is known that composite slabs can influence load redistribution and collapse resistance, the contribution of slabs to overall structural resistance has not been quantified, and load-resisting mechanisms such as CA and TMA have not been clearly identified. Furthermore, the effects of some governing parameters remain unclear, such as slab aspect ratio, boundary restraint condition and degree of composite action between slabs and steel beams, which may significantly affect the behaviour of 3D composite floor systems under column loss events. Hence, there are very limited insightful research outcomes on 3D composite floor systems subjected to an internal column loss scenario and further studies especially experimental tests are urgently needed to shed light on 3D structural behaviour.

In view of the significant differences in structural behaviour between 3D floor systems and simplified 2D frames, clause 3-2.11.2 of DoD (2013) specifies that *“To model, analyse, and evaluate a building, employ a three-dimensional assembly of elements and components. Two-dimensional models are not permitted.”* To date, finite element (FE) software is the only tool can be used to realistically capture the behaviour of 3D composite floor systems. However, 3D modelling of slab action is impractical for office design environment due to high cost of software and requisites of expertise in simulations. Therefore, it is expedient to propose an analytical model for calculating structural resistance of floor systems against progressive collapse. A design-oriented method was proposed by Alashker and El-Tawil (2011). However, their model cannot predict the load-displacement response at small deformation stage. Besides, their model was only verified by FE simulations, so further verifications by actual tests are needed.

The systematic reviews in Chapter 2 show that research on 3D steel composite floor systems subjected to an internal column loss scenario still has some technical gaps, including:

- (1) Comprehensive validations for FE models, due to very limited data base of 3D experimental tests;
- (2) Load-resisting mechanisms and quantifications of individual structural member contributions such as slabs, girders and beams;
- (3) Parametric studies on governing factors such as slab aspect ratio, degree of composite action between slabs and steel beams, and boundary condition;
- (4) Studies on force-based and displacement-based Dynamic Increase Factors (DIFs);
- (5) Reliable and practical models for calculating the entire load-deflection response.

Therefore, this study aims to fill these knowledge gaps to shed light on the behaviour of 3D composite floor systems subjected to an internal column removal scenario.

1.4 Research Objectives

The specific objectives of this study are as follows:

- (1) To investigate failure modes and load-resisting mechanisms of 3D composite floor systems under an internal column loss scenario through 4 large realistic scale experimental tests and to quantify the contributions of respective mechanisms in floor systems including flexure and CA in the double-span girder and the double-span beam above the missing column, as well as the slab effect.
- (2) To study the effects of slab aspect ratio, composite action and boundary condition.
- (3) To investigate parametric influences on load-resisting mechanisms and identify the inter-relationship among these mechanisms using validated numerical models.
- (4) To study dynamic responses and failure modes of 3D composite floor systems subjected to a sudden column loss scenario and to study DIF.
- (5) To propose a mechanical model for assessing the robustness of 3D

composite floor systems subjected to loss of an internal column.

1.5 Layout of Thesis

This chapter introduces the background and research gaps of 3D composite floor systems against progressive collapse, addressing the objectives of this study.

Chapter 2 firstly reviews the design approaches against progressive collapse in the current design guides. Subsequently, Chapter 2 reviews previous studies on component-based model for steel joints subjected to large deformation as well as experimental and numerical studies on 2D steel frames under column removal scenarios. At the end of Chapter 2, numerical, experimental and analytical studies on 3D composite floor systems are reviewed and research gaps are identified, leading to the research scope of this study.

Chapter 3 presents the specimen design, the loading scheme, the set-up, the instrumentation for each test and the material properties. Based on the comprehensive measurements in one test, Chapter 4 clarifies the failure mechanisms, load-resisting mechanisms and load redistribution among the remaining columns in 3D composite floor systems at each deformation stage under an internal column removal scenario. Through comparing test results of the four specimens, Chapter 5 reveals the parametric influences of slab aspect ratio, composite action and boundary condition.

Chapter 6 verifies the numerical models with the test results. The models are then used to evaluate the role played by slabs and investigate parametric influences on load-resisting mechanisms. Additionally, Chapter 6 studies the dynamic response and the failure mode of a 3D composite floor system subjected to a sudden column-loss scenario. Lastly, DIFs for 3D composite floor systems are investigated and compared with those for 2D frames and with the factors recommended by DoD (2013).

Chapter 7 shows the development of a mechanical model to capture the entire static load-displacement curve of 3D composite floor systems under an internal column removal scenario. The model is verified by actual test and numerical results. On top of that, a detailed worked example is provided in Chapter 7 to show how the mechanical model works. Based on this model, robustness of eight sub-structures

with different joint designs is evaluated and compared. Four sub-structures with different slab aspect ratios are also compared in terms of structural resistance. The comparisons yield some meaningful findings and recommendations for structural steel design against progressive collapse.

Chapter 8 summarises the research outcomes and proposes recommendations for the future work.

CHAPTER 2 LITERATURE REVIEW

2.1 Overview

Chapter 1 has introduced the background and research works on progressive collapse, and established the objectives for filling the knowledge gaps. This chapter further reviews the valuable approaches proposed in current anti-progressive collapse design guides and some questionable issues. Moreover, previous studies on steel and composite structures are systematically reviewed in Sections 2.3, 2.4 and 2.5. The end of this chapter summarises deficiencies of existing studies, leading to the scope of this research.

2.2 Design Approaches against Progressive Collapse

After the collapse of Ronan Point apartment, engineers and authorities recognised the importance of structural building robustness. In the following decades (1970s-1990s), the studies on progressive collapse were not so sophisticated. Thus, the indirect method was the main approach to ensure structural integrity by specifying the minimum level of tie forces in horizontal and vertical directions at critical locations. The British code (BS8110 1997) was the authoritative guide during that period, and was subsequently inherited by Eurocode (EC1-1-7 2006). However, up to now, the design guides proposed by the U.S. Department of Defense (DoD) and the U.S. General Services Administration have (GSA) become the de facto standard worldwide for anti-progressive collapse design.

2.2.1 Structural analytical approaches

As mentioned in Section 1.2, it is globally acknowledged that AP method is well-founded and unambiguous. With AP method, structures can be analysed with one of the three approaches: linear static analysis, nonlinear static analysis and nonlinear dynamic analysis. To consider the dynamic and nonlinear effects caused by sudden column loss, DoD (2013) and GSA (2013) allow linear static analysis by amplifying loads with Load Increase Factor (LIF) or allow nonlinear static analysis with Dynamic Increase Factor (DIF). This idea is rather convenient for practical design environment. However, the linear static analysis approach is too conservative, while

the specifications on DIF in DoD (2013) and GSA (2013) are debatable, as discussed in the subsequent section.

2.2.2 Dynamic effect

The old versions of the guides (GSA 2003, DoD 2005) used a fixed factor of 2 for considering nonlinear and inertial effects. After years of exploring, DoD (2013) and GSA (2013) use Eq. (2.1) to calculate DIFs for steel structures under nonlinear static analysis. It can be seen that DIF monotonically decreases from 2 with increasing normalised rotation ($\theta_{\text{pra}}/\theta_{\text{yb}}$), as shown in Fig. 2.1.

$$\text{DIF} = 1.08 + \frac{0.76}{\theta_{\text{pra}} / \theta_{\text{yb}} + 0.83} \quad (2.1)$$

where θ_{pra} is the plastic rotation angle (Eq. (2.2)).

$$\theta_{\text{pra}} = \theta - \theta_{\text{yb}} \quad (2.2)$$

where θ is the total rotation of a beam and θ_{yb} is the yield rotation of the beam. For steel beams, θ_{yb} is determined as follows (ASCE41-06 2007):

$$\theta_{\text{yb}} = \frac{W_y f_y l_b}{6EI_b} \quad (2.3)$$

where the inflection point is assumed to be at the mid-length of the beam, W_y is the elastic section modulus, f_y is the yield strength, l_b is the length of the beam, E is Young's modulus, and I_b is the moment of inertia. It should be mentioned that $6EI_b/l_b^2$ is the value of bending moment at either end of a fixed beam with the unit deflection at one end.

Fig. 2.1: DIF for structural steel (DoD 2013)

To achieve higher accuracy, structures can be dynamically analysed under a sudden column removal scenario (viz. nonlinear dynamic analysis) without amplifying imposed or dead loads. This method is usually employed by researchers to directly assess structural safety or study DIF in DoD (2013) in return. DIF for a given structure can be obtained through comparing the pseudo-static response with corresponding quasi-static push-down load-displacement curve (Fig. 2.2). The pseudo-static response denotes the relationship between the free-fall load and the corresponding maximum dynamic deflection. By means of experimental tests or numerical simulations, it is feasible to conduct nonlinear dynamic analysis for a structure under a load subjected to a sudden column loss scenario, yielding the corresponding maximum deflection. In the same way, numerical analyses can be conducted under increasing magnitudes of load until the structure just collapses, leading to the pseudo-static response (Fig. 2.2).

Fig. 2.2: The static push-down response and pseudo-static response of a structure under a column removal scenario

Fig. 2.3: Dynamic assessment based on energy balance method

From Fig. 2.2, DIF is assumed to be a ratio of the static load P_{st} to the dynamic load (P_{dy}) under the same displacement, as shown in Eq. (2.4). This is the method to provide data for curve fitting in Fig. 2.1. Hence, DIF in DoD (2013) is force-based (DIF_p) to evaluate the dynamic magnification of load within the allowable deformation. *Using this approach, engineers can ascertain for a given deflection if the structure has sufficient dynamic strength for a missing column scenario.* Besides the force-based DIF, displacement-based Dynamic Increase Factor (DIF_u) is used to consider the dynamic effect of sudden column loss from the perspective of ductility of joints. According to Biggs (1964), DIF_u can be calculated based on Eq. (2.5) for a given load.

$$DIF_p = P_{st} / P_{dy} \quad (2.4)$$

$$DIF_u = u_{dy} / u_{st} \quad (2.5)$$

where u_{dy} and u_{st} are the maximum dynamic and maximum static displacements under the same applied load, respectively. For a given load, DIF_u reflects how much the static displacement is magnified under the dynamic effect. *Therefore, DIF_u can be applied to assess if a structure has adequate ductility under a sudden column removal scenario.*

As a whole, pseudo-static response is a wise choice to assess robustness of a building

subjected to sudden column loss associated with inertial and nonlinear effects. According to the definition, a pseudo-static curve is established on a series of nonlinear dynamic analyses, which is tedious and may not be necessary for engineering design. Alternatively, a pseudo-static response can be converted from the corresponding push-down nonlinear static response, using the simplified framework proposed by Izzuddin et al. (2008). The framework is based on energy balance between external work and internal energy with the assumption of single degree of freedom. As shown in Fig. 2.3, each point on the nonlinear static load-displacement curve corresponds to a free-fall load on the pseudo-static curve at the same displacement. Izzuddin (2010) pointed out that the derivation of Eq. (2.1) is based on elastic-plastic systems, so DIF decreases monotonically with allowable ductility (Fig. 2.4). However, for elastic-plastic hardening and softening-hardening systems, DIFs increase to great values at large deformation stage when hardening occurs in the nonlinear static response (Fig. 2.5 and Fig. 2.6). For this reason, DIF in DoD (2013) is not safe for these two types of systems. It should be noted that the dynamic responses in Fig. 2.4 to Fig. 2.6 are nonlinear dynamic responses rather than linear dynamic responses, since they are converted from nonlinear static analyse and involve the dynamic effect.

Fig. 2.4: Characteristic responses and DIF for elastic-plastic system (Izzuddin 2010)

Liu et al. (2013, 2015b) and Liu et al. (2016) performed dynamic tests on web-cleat, flush-end-plate and top-and-seat-with-web-angle beam-to-column connections

subjected to sudden loss of a middle column. Both displacement-based and force-based DIFs were presented and discussed. They found that DIF could be greater than 2.0, unlike conventional knowledge that the DIF should be less than 2.0. They further concluded that DIFs depend on the connection type, deformation capacity and failure mechanism. Also, Liu et al. (2013, 2015b) and Liu et al. (2016) proposed different DIFs for different types of connections. They also suggested that more specific DIF values should be provided in the design guide. Fu et al. (2016) used the component-based model (Yang and Tan 2013b) to simulate two types of beam-to-column connections, i.e. web-cleat and top-and-seat-with-web- angle connections. The models were verified by the test data (Liu et al. 2013). Based on numerical simulations, Fu et al. (2016) found that the force-based DIFs of these two types of connections were very different from the recommendations in DoD (2013). However, their conclusions (Liu et al. 2013, 2015b, Fu et al. 2016, Liu et al. 2016) were drawn solely based on *2D steel frames*. Further studies are needed to investigate what force-based DIFs of *3D composite floor systems with the same types of connections* would be.

Fig. 2.5: Characteristic responses and DIF for elastic-plastic hardening system (Izzuddin 2010)

Fig. 2.6: Characteristic responses and DIF for softening-hardening system (Izzuddin 2010)

In summary, nonlinear static analysis approach with DIF is applicable with high accuracy. However, the values of DIF specified in DoD (2013) and GSA (2013) are debatable, and even incorrect for elastic-plastic hardening systems (Fig. 2.5 and Fig. 2.6). Besides, DoD (2013) and GSA (2013) do not consider the magnified ductility demand due to dynamic effect. Therefore, both force- and displacement-based DIFs

will be studied in this research in the context of 3D steel structures with composite floor action.

2.3 Component-Based Model for Bolted Steel Joints Subjected to Large Deformation

For steel structures, joints are usually more vulnerable than members, so joint behaviour governs the robustness of steel structures. Hence, to accurately study the progressive collapse resistance of steel-concrete composite structures, the ultimate joint behaviour subjected to large deformation must be disclosed first. Thankfully, there are existing analytical models of commonly-used connections.

EC3-1-8 (2005) has provided component-based models through evaluating individual flexibility of different zones. This method can be used to predict the behaviour of end-plate and bolted-angle connections, treating the component of end plate or angles in bending as an equivalent T-stub. Nevertheless, EC3-1-8 (2005) cannot predict behaviour of components subjected to large deformation.

Piluso et al. (2001a) proposed a theoretical model to predict plastic deformation capacity of T-stubs in semi-rigid bolted connections (such as flush-end-plate). They analysed the failure mechanisms and presented the formulations for predicting ultimate plastic displacements of bolted T-stubs. Besides, the entire force-displacement curves of T-stubs can be approximately evaluated based on this model which was further verified in Piluso et al. (2001b). Besides the behaviour of T-stubs in tension, local plastic buckling behaviour of girder flanges in compression is also critical, which was quantified in Gioncu and Mazzolani (2002) and Petcu and Gioncu (2003).

Taib (2012) developed an equivalent bolt-row component for a single bolted lap joint in tension and compression. Both types of components failed by either ductile plate bearing failure or brittle bolt shearing failure. The force-displacement relationship of each bolt-row component can be assembled into the overall response of a fin-plate joint subjected to tension or bending.

Yang and Tan (2013b) and Yang and Tan (2013c) developed a component-based

model to simulate the behaviour of bolted angle connections subjected to tension under large deformation. The accuracy was validated by the static tests. The proposed component-based model of bolted-angle connections is shown in Fig. 2.7. 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 ($bwbb$). As shown in Fig. 2.7, all the basic components corresponding to one bolt row were combined as one equivalent spring at that bolt row. For example, all the components located at the bottom bolt row including bt , sab , bsm , bfb , bs and abb were combined as one spring k_1 . In this model, it is assumed that the springs of k_1 , k_3-1 , k_3-2 and k_3-3 can sustain tensile forces while all the compression forces are sustained by the springs of k_2 . This kind of component-based method can be used for macro-model simulations as well as practical applications for progressive collapse analysis.

Fig. 2.7: Component-based model of bolted-angle connections of beam-column joints (Yang and Tan 2013b)

2.4 Studies on Steel Skeletal Frames under Column Removal Scenarios

2.4.1 Two-dimensional steel skeletal frames

In the past two decades, there have been some valuable research outcomes including actual tests and numerical simulations on the behaviour of 2D beam-to-column

assemblies under missing column scenarios. It can be confirmed that CA can enhance progressive collapse resistance of 2D frames (Demonceau 2008, Yang and Tan 2012, Guo et al. 2013, Yang and Tan 2013a). However, CA can be only fully developed in joints with sufficient strength and ductility.

Khandelwal and El-Tawil (2007) used numerical simulations to study the effect of steel moment-resisting connections with seismic detailing on developing CA. They concluded that the steel frames with Reduced Beam Sections (RBS) are stronger and more ductile than steel frames with Welded Unreinforced Flange, Bolted web (WUF-B) connections. Before long, Khandelwal et al. (2008) developed computationally-efficient macro-models to compare the progressive collapse resistance of two 2D steel moment frames using the respective WUF-B and RBS connections for moderate and high seismic risk. They pointed out that ductility demands associated with column loss in the moment connections of both buildings are rather small, so the benefit of ductile seismic detailing for anti-progressive collapse design is not evident. They also found that although shear tab connections have reasonable ductility to develop CA, they do not have sufficient strength to resist the load after one column is suddenly removed.

Lew et al. (2013) revealed that WUF-B and the RBS connections possessed twice the rotational capacities under a column removal scenario (monotonic vertical displacement) compared with those under seismic action. According to their findings (Khandelwal et al. 2008, Lew et al. 2013), it is not necessary to improve ductility of moment joints subjected to a column removal scenario.

Lee et al. (2009) conducted nonlinear FE analyses of the double-span beams in welded steel moment frames. Based on the numerical results, they have proposed two simplified methods to assess the resistance of a double-span beam in welded frames. They conducted ideal plastic analyses without considering the influence of welded sections subjected to large deformation. However, the actual practice shows that fully welded moment connections have poor performance (Krauthammer 1999, Brunesi et al. 2015).

Yang and Tan (2013a) studied the behaviour of various types of beam-to-column

connections, through connection tests on web-cleat, top-and-seat-angle, top-and-seat-with-web-angle, fin-plate, bolted flush-end-plate and bolted extended-end-plate, under a column removal scenario. They found that flush-end-plate connections can develop both flexural action and CA well (Fig. 2.8), while web cleat connections had the best rotational capacity to resist progressive collapse (Fig. 2.9).

Fig. 2.8: Test result of flush-end-plate connection (Yang and Tan 2013a)

Fig. 2.9: Test result of web-cleat connection (Yang and Tan 2013a)

Besides the experimental and numerical studies on 2D frames, an analytical approach was proposed by Stylianidis et al. (2015) to predict the response of steel and composite beams subjected to an internal column removal scenario. The model can capture the distinct characteristics of response and deliver in-depth appreciations in the structural behaviour of beam grillages after losing one column. Most likely, the double-span beam including the supporting connections are the principal alternate load paths. Using the proposed method (Stylianidis et al. 2015), they (Stylianidis et al. 2016b) further studied the response of the double-span beam and the corresponding joint components in detail, providing a thorough view on development

of compressive arching action, flexure and catenary action.

2.4.2 Three-dimensional steel skeletal frames

On top of 2D frames, Stylianidis et al. (2016a) evaluated the robustness of 3D frame structures under various column removal scenarios using the simplified method developed in Stylianidis et al. (2016b). One of the aims was to investigate the 3D effects on structural behaviour. As a result, they concluded that integrating the basic characteristics of individual beam responses may accurately predict the behaviour of a grillage structure. Basically, one shortcoming is that the slab effect was not involved, which can significantly affect structural behaviour such as load-redistribution, load-resisting mechanisms and failure modes.

Dinu et al. (2016) experimentally investigated the behaviour of a 2×2-bay steel skeletal frame with end-plate bolted joints subjected to a central column removal scenario (Fig. 2.10). They found that end-plate bolted connections had sufficient strength and ductility to develop CA in the double-span beams. The ultimate chord rotation of the double-span beam reached 0.206 rad. Meanwhile, great axial forces in the beams caused the perimeter columns to deform significantly.

(a) Overview of specimen and set-up (b) Load-central displacement curve

Fig. 2.10: Tests on 3D steel skeletal frame by Dinu et al. (2016)

Li et al. (2017) also conducted a push-down test on a 3D bare steel frame with welded moment-resisting connections (Fig. 2.11 (a)). According to the test results, they pointed out that flexure played an important role in the collapse resistance during the entire loading process, while CA in the double-span beams above the removed

column became dominant at the final stage. Ultimately, the specimen failed by the fracture of the bottom flange at the welded beam-to-removed column connection (Fig. 2.11 (b)). Lastly, they emphasised that DIF could be underestimated when the static resistance is significantly enhanced by CA, which confirms the findings of Izzuddin (2010), Liu et al. (2013, 2015b), Fu et al. (2016) and Liu et al. (2016).

However, this kind of 3D skeletal frame tests could not deliver more meaningful results than 2D frames, since the composite slab action was not involved. Besides, only one point load was applied at the removed column stub; this may result in a different failure mode compared with that from distributed loads which is more realistic. For instance, the connections at the removed column location would prematurely fail due to a concentrated load above the removed column stub.

(a) Overview of specimen

(b) failure mode

Fig. 2.11: Tests on 3D steel skeletal frame by Li et al. (2017)

2.4.3 Load-resisting mechanisms in sub-structures

The previous studies (Sections 2.4.1 and 2.4.2) revealed the common load-resisting mechanism in 2D beam-column assemblies and 3D skeletal frames subjected to an internal column loss scenario. It is noted that flexure initially contributes to resisting the vertical loads, followed by the dominant CA at large deformation stage. CA is usually mobilised at a central deflection of one beam depth, signified by initiation of tensile resultant axial force in the beam. Some researchers have further quantified the individual contributions of flexure and CA for 2D assemblies (Yang and Tan 2013a)

and 3D skeletal frames (Li et al. 2017).

Similar to CA in beams, TMA is an ultimate mechanism in slabs to resist vertical loads. As the name suggests, TMA involves the formation of a tensile net of steel reinforcement in the slab central region when the slab deflects to a certain extent (Fig. 2.12). Development of the outer compressive ring (Fig. 2.12) is affected by the boundary conditions along the slab edges, since the purpose of the compressive ring is to equilibrate the internal tension.

However, for 3D steel frames with composite slabs under a column removal scenario, load-resisting mechanisms have never been unveiled. They are far more complicated than mechanisms in skeletal frames or mere slabs, since steel skeletons and composite slabs actually interact with each other. For instance, composite slabs can enhance the flexural strength of steel beams due to composite action. In return, steel beams can affect stress distribution in slabs. Nevertheless, the unknown mechanisms can be hypothesised based on the knowledge of skeletal frames and slabs, as shown in Fig. 2.13. Specifically, the load is redistributed through two double-span beams above the missing column, as well as the composite slab. Generally, at small deformation stage, flexure should be the dominant mechanism in both the double-span beams and the slab. In subsequent stages, CA in the double-span beams and TMA in the slab may become dominant. Certainly, these hypotheses should be supported by actual tests or reliable numerical simulations. It is meaningful to identify the relative importance of these mechanisms through quantifying individual contributions of them, so that engineers can have better appreciation of each alternate load path in the 3D structural system.

Fig. 2.12: TMA in slabs

Fig. 2.13: Hypothetical load-resisting mechanisms in composite floor systems under an internal column removal scenario

2.5 Studies on 3D Composite Floor Systems under Column Removal Scenarios

Up to now, there are a few numerical studies and very limited experimental studies on 3D composite floor systems under column removal scenarios. These studies are carefully reviewed to identify preliminary findings and deficiencies. In particular, the

deficiencies in the previous tests can help to improve experimental design in this research.

2.5.1 Numerical studies on 3D composite floor systems

Sadek et al. (2008) were the first to conduct numerical simulations on 3D composite floor systems subjected to a column removal scenario using commercial FE software (LS-DYNA). They aimed to evaluate robustness of composite floor systems with fin-plate connections. Therefore, the first part of their work was to study the behaviour of a fin-plate joint in a beam-column assembly under push-down analysis, as shown in Fig. 2.14 (a). The joint was modelled under three levels of fidelity (Fig. 2.14 (b)). All the three models could capture the behaviour of the joint although they were only verified by very limited test data. Finally, they adopted a coarse shell connection model when simulating a floor system under an internal column removal scenario (Fig. 2.15). Simulation results of the floor system showed that the composite slab significantly contributed to structural robustness through two mechanisms. Firstly, the composite slab functioned as a rigid diaphragm preventing the surrounding columns from being pulled inward. Secondly, the reinforcing mesh and steel decking in the slab provided membrane action. Their findings were to be confirmed in actual tests, since their FE models lacked more convincing verifications.

Fu (2009) employed ABAQUS package to simulate the structural behaviour of a 20-storey steel composite building under a sudden column loss scenario. The beams and columns were simulated using 2D beam elements, composite slabs were simulated by shell elements and the beam-to-column connections were fully pinned. The model was simply verified by the published composite-frame test (Wang and Li 2007) in terms of moment-rotation relationship of a composite joint. Based on this model, Fu (2010) conducted further parametric analyses and found that reinforcement ratio in composite slabs slightly influences the behaviour of composite steel buildings under one column removal scenario. He is also one of the pioneers who studied robustness of 3D composite buildings against progressive collapse using commercial FE software and gave some opinions on global behaviour of the building. However, he simplified all the connections as pins so that rotational stiffness of the joints could only rely on continuous slabs above the joints. Thus, the rotational limits of the joints

were also governed by the failure of slabs above the corresponding joints. In fact, bolt-row components in steel connections usually govern the stiffness and ductility of the composite joints. Hence, both the rotational stiffness and limits of the joints in his model could be quite different from those in reality. From his numerical study on 3D composite floor systems, clearly, researchers should adopt more refined joint models, since the joint behaviour can significantly influence the overall performance of a 3D structure.

(a) Beam-column subassembly subjected to push-down analysis

(b) High Fidelity
Connection Model

(c) Reduced Component
Connection Model

(d) Reduced Coarse
Shell Connection Model

Fig. 2.14: Modelling of shear connection (Sadek et al. 2008)

Fig. 2.15: Modelling of composite floor system (Sadek et al. 2008)

Alashker et al. (2010) simulated composite floor systems subjected to removal of a centre column with macro-based numerical models verified by published test data (Kim et al. 2001, Foster et al. 2004). With this model, they conducted parametric studies on steel decking thickness, slab reinforcement ratio and numbers of bolts in the fin plate connection under both the force-controlled Uniform Load (UL) and displacement-controlled Concentrated Load (CL) above the removed column (Fig. 2.16). They have some interesting conclusions:

- (1) The steel decking can contribute as much as 60% of the load at ultimate state. Besides, thickening the steel decking can significantly increase the overall load-carrying capacity.
- (2) The load-based DIF for the simulated structure is 1.29, so the DIF of 2.0 recommended in the old version design guide (GSA 2003) could be relaxed.
- (3) The simulation results show that applying UL can obtain more realistic behaviour but it is more difficult to implement UL in a laboratory. In comparison, CL can be easily applied but may result in premature failure of joints at the removed column location.

It is meaningful to quantify steel decking contribution at large deformation stage. This would be a useful test. Except for the design guides, Alashker et al. (2010) was the first to study DIF of 3D composite floor systems, showing the opposite comment on the DIF in design guides compared with the comments from DIF studies on 2D frames (Liu et al. 2013, 2015b, Fu et al. 2016, Liu et al. 2016). Based on conclusion (3), a three-tier multi-point loading system will be used in the author's test programme (Chapter 3), since loads can be applied through distributed beams in displacement-control mode.

Fig. 2.16: Load-displacement relationships for both UL and CL (Alashker et al. 2010)

Hoffman and Fahnestock (2011) studied the numerical behaviour of three-storey and ten-storey steel composite buildings subjected to individual column removal scenarios. Hoffman and Fahnestock (2011) pointed out that, for the structural type they studied, each floor can be considered independently in terms of gravity load redistribution. Therefore, it is reasonable to test a representative floor to study the load-redistribution mechanism considering the extremely high cost of system-level tests. Furthermore, they found that removal of a column with only shear connections was more vulnerable, resulting in localised collapse. The reason was that shear connections were subjected to significant deformation capacity demands after the column loss. They failed due to lack of sufficient strength, similar to the findings by Khandelwal et al. (2008) (Section 2.4.1). However, Hoffman and Fahnestock (2011) concluded that moment-resisting connections to the removed column also experienced considerable flexural and axial deformation demands, contradicting to the findings by Khandelwal et al. (2008). These two opposing opinions arise from their different models. The analyses in Hoffman and Fahnestock (2011) were based on a 3D steel composite building, while those of Khandelwal et al. (2008) were based on a 2D steel frame. Therefore, the experimental behaviour of moment-resisting and simple joints in 3D composite floor systems subjected to a column loss scenario should be investigated, since there are very limited test data.

Based on the simplified grillage model (Fig. 2.17) in Vlassis et al. (2008), Jahromi et al. (2013) introduced a more sophisticated grillage model to capture the nonlinear

response of connections and consider the nonlinearity of secondary beams. The improved model incorporated effective composite-beam flanges (Fig. 2.18). Besides, a composite floor system model (Fig. 2.19) was established to investigate the influence of membrane action. They compared the results of three models, showing that the ultimate load-carrying capacity of the composite floor system model was 37.8% greater than that of the simplified grillage model. Although the sophisticated grillage model was developed to improve individual beam model, the ultimate load-carrying capacity of the improved model was still smaller than that of the composite floor system model by 10%, due to neglect of TMA. These quantifications of slab contributions are rather valuable to identify the 3D slab effect on structural resistance. These findings were drawn from very limited numerical simulations, which need to be verified by actual tests before they can be widely accepted. However, no one has used published tests on 3D composite floor systems to quantify the slab contribution numerically.

Fig. 2.17: Deformation mode for the assembled floor system (Vlassis et al. 2008)

Fig. 2.18: Grillage systems with effective slab width (Jahromi et al. 2013)

Fig. 2.19: Composite ribbed slab model (Jahromi et al. 2013)

Song and Sezen (2013) conducted a field experimental programme on an existing steel building by removing four first-storey columns one after another. The scarce test data was mainly used to validate the 2D and 3D SAP2000 models constructed based on the tested steel frame building. The 2D and 3D models were analysed with linear static and nonlinear dynamic approaches as recommended in GSA guidelines. In 2D linear static analyses, increases in demand-to-capacity ratios (DCR) for columns were much greater than those for beams, indicating columns were more vulnerable than beams. In comparison, the DCR values in 3D model showed an opposite trend due to participation of transverse beams, so that 2D linear static analysis may cause underestimation of demands for beams. However, DCR values of structural members in 3D model were generally lower than those in the 2D model, due to contributions from transverse beams. Similarly, in nonlinear dynamic analyses, the vertical displacements at the removed column locations in the 3D model were smaller than the counterparts in the 2D model. Furthermore, vertical displacements calculated in linear static analysis with the amplified loads were greater than corresponding displacements in nonlinear dynamic analyses for both the 2D and 3D models. Hence, the load amplification factor of 2.0 for linear static analysis is very conservative.

The limited experimental results helped researchers to understand some behaviour of full-scale steel frame buildings subjected to column loss scenarios, but were not sufficient to shed light on load-resisting mechanisms or failure modes since the structure was not significantly damaged. For numerical modelling, the author views that it is important to simulate composite slab action and detailed joint behaviour to

obtain more realistic models for more accurate investigation.

Li and El-Tawil (2014) constructed numerical models of a 10-storey prototype composite structure building to investigate their collapse resistance. Firstly, Li and El-Tawil (2014) pointed out that after losing a column, loads were mostly redistributed in moment bays through frame action. Specifically, axial forces in columns connected to the removed column with simply supported beams were hardly affected. Instead, the columns outside of the immediately-affected bays and connected with moment-resisting connections may sustain a significant increase in vertical loads. This finding does not support the provision in DoD (2013) and GSA (2013) in terms of applied loads in static analysis, which recommends the amplification of loads on the bays directly above a removed column. Secondly, it was found that removal of an internal column with simple connections was more vulnerable than removal of a column with moment-resisting connections, which was also found by Hoffman and Fahnestock (2011). Based on their common finding that joint behaviour can significantly affect the robustness of a structure, the effect of joint design in a structure is worthy to be studied. In addition, the simulation results showed that slabs could mobilise 3D action to arrest or slow down collapse. However, presence of slabs might trigger progressive collapse in certain situations of loss of columns with simple connections. Also, flexural composite action of beam-slab system could play an important role in resisting collapse. Nevertheless, composite action would be lost since slabs would be damaged at large deformation stage. After that, TMA would become a dominant load-transfer mechanism.

All these findings expanded the knowledge on the behaviour of a global 3D composite floor building, in particular, the discussion on load redistribution associated with global framing action. However, this opinion needs justifications from more studies especially from experimental tests on 3D sub-structures. Besides, the finding that “slabs might trigger progressive collapse in certain situations” requires further experimental studies since it seems to be counter-intuitive. Furthermore, whether the loss of flexural composite action at large deformation stage would decrease the load-carrying capacity should be experimentally studied.

Main (2014) built reduced model (or macro-model) using LS-DYNA (2007) to assess

resistance of composite floor systems against collapse. In the reduced model, the actual profiled cross-sections were replaced by the respective strong and weak equivalent rectangular cross-sections (Fig. 2.20) simulated by shell elements, while the behaviour of beam-to-column connections and shear studs were simulated with component-based approach (Fig. 2.21). The reduced model was validated by the detailed model results. Afterwards, they found that the boundary conditions of the affected bays above the missing column had significant influence on ultimate capacity of the floor system. As shown in Fig. 2.22, for the 4×4-bay floor, the slab continuity beyond the affected bays facilitated the development of CA in beams and TMA in slabs compared to the free boundary condition in 2×2-bay floor. They concluded that the load-carrying capacity of the former was 71% higher than that of the latter. Besides, they found longer span lengths caused a floor system to be more susceptible to collapse. According to these conclusions, the effects of boundary condition and span length are worthy to be studied in actual tests. Besides, their reduced modelling method has the potential to be widely applied to numerical simulations for 3D steel sub-structures, since it can save huge computational cost while maintaining rather accurate predictions compared with detailed 3D continuum modelling method.

Table 2.1 summarises some features and achievements of the mentioned numerical studies.

Fig. 2.20: Reduced model for composite slabs (Main 2014)

(a) actual profile (b) simplified strong and weak strips

Fig. 2.21: Reduced model for shear connectors and beam-to-column joints (Main 2014)

Fig. 2.22: Axial forces at beam ends (arrows) and tensile force per length normal to slab edges (filled areas) (1/4 model) (Main 2014)

(a) 2×2-bay floor system at the ultimate state (b) 4×4-bay floor system at the ultimate state.

Table 2.1: Summary of current numerical studies on 3D composite structures subjected to column removal scenarios

Publications	Realistic joint behaviour	Validation at system level	Investigation on DIF
Sadek et al. (2008)	√	×	×
Fu (2009)	×	√ (Limited validation, test was not under column removal scenarios)	×
Alashker et al. (2010)	√	×	√ (Opposite opinion compared with 2D frames)
Hoffman and Fahnestock (2011)	√	×	×
Jahromi et al. (2013)	√	×	×
Song and Sezen (2013)	×	√ (No slab involved)	×
Li and El-Tawil (2014)	√	×	×
Main (2014)	√	×	×

2.5.2 Experimental studies on 3D composite floor systems

The first experimental study on composite floor systems against progressive collapse was conducted at University of California at Berkeley by Astaneh-Asl et al. (2001) (Fig. 2.23). The researchers found that a composite floor could survive even when it was subjected to loss of a middle perimeter column. This study mainly focused on the contribution of cable retrofitting to arrest progressive collapse. However, the interest of this research work is the structural behaviour in commonly-designed composite floor systems subjected to column loss, to provide suggestions for new construction. Hence, additional devices such as cables are certainly not used in buildings.

Fig. 2.23: Test at University of California at Berkeley (Astaneh-Asl et al. 2001)

Chen et al. (2012) launched an experimental programme on a two-storey steel frame composite floor system to investigate the progressive collapse resistance subjected to sudden removal of an edge column. They observed that the slab contributed significantly to load redistribution and confirmed that the slab contribution would be underestimated if composite-beam action with effective flange width is considered. However, they did not quantify the slab contribution. After instantaneously removing the column, the structure did not collapse and the strains of remaining members were far smaller than the yield strains. That is to say, the structural behaviour at large deformation stage could not be observed or studied. Chen et al. (2012) designed the experimental programme carefully and obtained valuable test data to advance the cutting-edge knowledge on progressive collapse. Similar tests are timely needed to explore the performance of steel composite structures under column removal scenarios. For example, besides the perimeter column, other column removal scenarios should be tested as well. Chen et al. (2012) used welded unreinforced flange-bolted web connections for beam-to-column connections in both directions, which is mainly used in seismic design. Future experimental studies could shed light on structures with bolted connections such as extended-end-plate, flush-end-plate, fin-plate and web-cleat joints.

Fig. 2.24: Test conducted by Chen et al. (2012)

An experimental test on a composite floor system under an internal column loss scenario was conducted at the University of Texas (Hull 2013). The specimen was a 2×2 -bay steel gravity frame with composite slabs (Fig. 2.25 (a)). The surrounding beams were designed as an oblate box section, which was relatively stiff to ensure mobilisation of in-plane membrane action in the slabs. The researchers intended to gradually fill water into buckets on the floor to model quasi-static loading. However, the testing was not carried out smoothly, since load-carrying capacity was significantly underestimated and the loading was difficult to be controlled. The floor system was tested by four rounds to collapse eventually. In the first round of tests, the specimen sustained a greater load than that prescribed by design guide (DoD 2010), with practically no damage to the structure. At the end of the first round, it was observed that some buckets were leaking excessively due to inferior quality of the buckets (Fig. 2.25 (b)). The test was then suspended when the buckets were retrofitted with plywood. The second test was performed 50 days later to repeat the first testing procedure to obtain more reliable data. In the third test, they deliberately weakened the connections to the removed column by removing the nuts of all the bolts except two of them for transferring shear only. Even then, no physical damage was observed. Eventually, in the fourth test, they removed the nuts and bolts from all connections to the removed column so that the load could be transferred to the framing members through the steel decking. As a result, collapse was initiated by

tearing out at a seam of the decking.

Although they encountered very serious loading issues, the test results still gave some useful observations. Firstly, the strain gauge readings from the surrounding box-section beams showed the presence of in-plane lateral forces caused by the loading on the affected bays. This showed that TMA in slabs could provide resistance. Secondly, steel gravity frame buildings had sufficient robustness to guard against internal column loss. Unfortunately, the test results were too limited to investigate the load-resisting mechanisms in the composite floor systems. Most importantly, the collapse was caused by artificial action so that they failed to unveil the realistic failure mode of the sub-structure subjected to an internal column loss scenario.

(a) Overview of the test

(b) Leakage of loading buckets

Fig. 2.25: Test at the University of Texas (Hull 2013)

Johnson et al. (2016) conducted a half-scale test on a composite floor system under column removal scenarios. As shown in Fig. 2.26 (a), the floor system was successively subjected to the removal of a corner column (CC), an edge column (EC-B) in the north-south direction, an edge column (EC-G) in the east-west direction and an internal column (IC) in each of the four testing scenarios. In each scenario, only the bays adjacent to the removed column were loaded. Uniformly distributed load (UDL) was applied by a circular water pond applied on the floor. Water was pumped into the pool while the floor was held by a crane which was slowly lowered down during the test, as shown in Fig. 2.26 (b). For CC removal scenario, the load-carrying capacity was unexpectedly low and equilibrium was not achieved even at the first

load level. It means that load-deflection response was not obtained for the CC removal scenario. Similarly, the load-carrying capacity under the IC scenario was much lower than expected. The reason may be that the neighbouring bays outside of IC test bays were already damaged by the previous tests so that the surrounding restraints and tie forces were weakened significantly. Additionally, the edge column tests caused some degree of debonding and loss of composite action in the IC test bays.

Based on the test results, they concluded that commercial buildings designed for gravity loads were not robust enough to resist the prescribed (by DoD 2013) load combination in extreme events. In contrast, Hull (2013) indicated that the same type of floor system was capable of carrying around 1.5 times the prescribed progressive collapse loads (DoD 2013) without sustaining significant damage. Johnson et al. (2016) gave totally opposite conclusions but his reasons were not clearly stated. The test results showed that composite slabs played an important role in developing an alternate load path for load redistribution under the CC removal scenario. Besides, they identified the load-resisting mechanisms under EC-B and EC-G removal scenarios. For EC-B removal scenario, load was transferred to surrounding sub-structures through flexure in the cantilever girder along Axis 2 (Fig. 2.26 (a)), whereas for EC-G removal scenario, load was redistributed through flexure and tension in the double-span girder along Axis 4 (Fig. 2.26 (a)). Nevertheless, Johnson et al. (2016) did not evaluate individual contributions of flexure and tension in girders, and slabs to overall structural resistance. Besides, load-resisting mechanisms under the IC scenario were not clearly investigated. They have conducted tests under internal, edge and corner column removal scenarios, although the tests under IC and CC removal scenarios were out of their expectations. Studies of column removal location could be extended to penultimate internal column (or next-to-edge column in Fig. 1.3 (a)), so that behaviour of 3D composite floor systems under all types of column removal scenarios could be revealed through actual tests.

Referring to their (Hull 2013, Johnson et al. 2016) improvised loading procedures and sometimes unavailable test results, further experimental tests should be cautiously designed with careful attention to details. Displacement-controlled loading

method should be used to obtain the full load-displacement curve including the descending branch. Water loading is definitely not recommended even though the load is uniformly distributed, since it is challenging to control the loading and leaked water may ruin instrumentation.

(a) Structural layout and removal scenarios

(b) The specimen and set-up

Fig. 2.26: Quasi-static tests on composite floor systems (Johnson et al. 2016)

Zandonini et al. (2014) quasi-statically tested a full-scale steel sub-frame with solid concrete slabs under an internal column removal scenario in the University of Trento (Fig. 2.27). The specimen was applied with sand bags weighing 8.8 kN/m^2 in accordance with the design load (EC1-1-7 2006). Subsequently, the pressure of the hydraulic jack (Fig. 2.27 (b)) was reduced to zero, and the substructure successfully sustained the sand bags without significant damage. To estimate the residual safety margin of the specimen, a downward force was applied at the central column to cause failure of the bottom-row bolts and concrete at the removed column location. Nearly 50% of load-carrying capacity margin was observed according to the evaluation of the vertical load sustained by the removed column stub. Zandonini et al. (2014) confirmed that the composite floor system had sufficient robustness and pointed out that ductility of beam-to-column joints plays an important role against progressive collapse.

Compared to Hull (2013) and Johnson et al. (2016), the loading procedure and instrumentation were carefully designed so the expected results were obtained in Zandonini's test (Zandonini et al. (2014)). However, the load was only applied up to the failure of bottom-row components at the removed column location for safety concern, when the load-displacement curve was still in ascending stage. Thus, the post-failure resistance could not be ascertained. Besides, the concentrated load at the last phase may cause ambiguity, since the failure modes and load transfer mechanisms in the floor are different under UDL and CL. Furthermore, the specimen in Zandonini et al. (2014) had only concrete solid slabs without any beams, whereas in common practice, it should be profiled-decking-concrete slabs supported by secondary beams. In such a case, the load transfer paths and crack patterns of the two types of flooring system are quite different. Besides, the steel decking itself could provide considerable structural resistance.

(a) Top view

(b) Bottom View

Fig. 2.27: Test on steel sub-frames with solid concrete slabs (Zandonini et al. 2014)

Table 2.2 summarises some achievements of the aforementioned experimental tests.

Table 2.2: Summary of current experimental studies on 3D composite structures subjected to column removal scenarios

Research objects	Astaneh-Asl et al. (2001)	Chen et al. (2012)	Hull (2013)	Zandonini et al. (2014)	Johnson et al. (2016)
External column removal	√	√	×		√
Internal column removal	×	×	√ (Artificial collapse)	√ (Mixed loading form, no decking or filler bam)	√ (Initial damage introduced)
Load-resisting mechanisms	×	×	×	×	√ (Only for external removal)
Quantification of individual contributions of mechanisms	×	×	×	×	×
Study on degree of composite action or slab aspect ratio	×	×	×	×	×

2.5.3 Analytical models

Dusenberry and Hamburger (2006) proposed two energy-based practical approaches to assess disproportionate collapse potential. The two approaches were based on push-down linear elastic analysis and elastic-plastic analysis involving flexural and catenary behaviour. The first approach can be applied to obtain the analysis for a simple sub-frame by hand calculations, or for a frame using common structural analysis software. However, linear elastic design against progressive collapse is too conservative. The second approach evaluated the energy reserved by both flexural and catenary actions to incorporate material nonlinearity. This evaluation was based on the assumption that plastic hinges were successively formed in beam sections without considering flexibility of joints. In fact, joints are usually the most vulnerable parts in a frame especially for a steel frame. Most importantly, the slab effect was not considered. Nevertheless, their work provided a valuable notion that the energy-based approach is a rational way to consider nonlinear and dynamic behaviour. More intuitively, one can assess collapse potential through comparing the energy absorbed by the structure and the change in potential energy, as shown in Fig. 2.28.

Fig. 2.28: Energy balance as collapsing portion of structure falls (Dusenberry and Hamburger 2006)

Izzuddin et al. (2008) proposed a novel and comprehensive framework to assess the robustness of multi-storey frames under a sudden column loss scenario. According to this framework, a frame is assessed through three steps, including the determination

of nonlinear static response, dynamic assessment and ductility assessment. To obtain the nonlinear static response, both detailed and simplified models can be accommodated by the framework. The application of detailed model is based on the use of FE software. The advantage is that slabs can be simulated so that the 3D slab effect is considered. The disadvantage is that there is greater level of expertise in FE simulations. For structural design in practice, it is better to implement the design procedure with simple tools such as Excel, or even by hand calculations. Hence, simplified models are more practical. When applying the simplified model, the nonlinear response of individual beams is obtained first, and then assembled into the response of individual floors. Certainly, the disadvantage is the neglect of slab effect. Further, they proposed the energy balance method to convert a nonlinear static response into the corresponding pseudo-static response for dynamic assessment, and recommended the pseudo-static capacity for ductility assessment. They pointed out that the pseudo-static capacity is a rational measure to estimate structural robustness, since the former encompasses three conventional indicators including the energy absorption capacity, redundancy and ductility, none of which is suitable on its own as a measure of robustness.

Alashker and El-Tawil (2011) proposed a design method to calculate the resistance of 3D composite floor systems subject to an internal column loss scenario (Fig. 2.29). Based on equilibrium of forces in transverse, longitudinal and vertical directions, the model was carried out in the form of incremental procedures that could track evolution of loading up to failure. Ultimate loads and displacements calculated by this method agreed well with the corresponding numerical simulation results, whereas the load-displacement relationship at small deformation stage could not be predicted. The reason was that the model was developed on the premise that the load was resisted by membrane action in slabs and CA in steel beams. A few assumptions were made in this method to simplify calculations. However, the assumption that steel reinforcement and decking yielded throughout the floor system should be further examined. This assumption simply ignored localised strain concentration of reinforcement at cracks and fracture of reinforcement before attaining global failure criterion when central deflection exceeds 10% of the shorter span length. Finally, it is better that the model can be verified by test data besides numerical simulation

results.

This model was developed to calculate static push-down resistance so that it could not be directly applied to situations when the dynamic effect was involved. What is less ideal was that the pseudo-static response could not be obtained based on energy balance method, since the load-displacement relationship at small deformation stage has not been ascertained. Hence, it is timely to develop an analytical model for calculating the entire push-down load-deflection response, to evaluate the actual robustness of 3D composite floor systems subjected to a sudden column loss scenario based on energy balance method. The future model should attempt to capture the detailed structural behaviour as much as possible on the premise that the model can be implemented in common office tools such as Excel.

Fig. 2.29: Assumed deformed shape for floor subjected to column loss (Alashker and El-Tawil 2011)

2.6 Summary of Literature Review and Research Scope of Work

From the literature review, development of component-based model for bolted joints is sophisticated for direct applications in numerical and analytical studies. The research outcomes on joint components and 2D frames cannot satisfy the requirements of a building design under column removal scenarios, since the slab effect can significantly influence structural behaviour. Up to now, there are a few numerical studies and very limited experimental studies for internal column removal scenarios. These are summarised in Table 2.1 and Table 2.2, respectively. Due to

unavailability of test data, most numerical models lack comprehensive verifications such as failure sequence and failure mode of structural components in a system. Therefore, it is important to conduct a carefully planned programme of 3D floor systems subjected to column removal scenarios. Among the five tests (Table 2.2), Astaneh-Asl et al. (2001), Chen et al. (2012) and Johnson et al. (2016) involved external column removal scenarios and provided valuable analyses based on test results. Hull (2013), Johnson et al. (2016) and Zandonini et al. (2014) attempted to study structural behaviour of the floor system subjected to an internal column removal scenario. However, the first test under an internal column removal scenario involved deliberate weakening of the joint above the missing column, leading to an artificial collapse. The second test introduced initial damage which significantly decreased the collapse resistance. The third test did not adopt steel-decking-with-concrete slabs. Besides, UDL was applied first followed by CL to failure of the joint. This change in loading regime resulted in an ambiguous mechanism at the final stage. Clearly, structural behaviour of 3D composite floor systems under an internal column removal scenario has not been successfully or clearly studied. In addition, an in-depth knowledge including of load-resisting mechanisms is missing and the effects of governing factors such as degree of composite action and slab aspect ratio are not yet studied. This motivates the author to develop an experimental programme comprising four 3D composite floor systems with three variables, viz. slab aspect ratio, degree of composite action and boundary condition, under an internal column loss scenario. Details of the experimental work are described in Chapter 3. Based on the test result of one substructure, load-resisting mechanisms and individual contributions of them are clearly presented in Chapter 4. Comparisons of test results among the four large-scale test specimens are presented in Chapter 5. Due to cost constraint, only four floor systems were tested, supplemented by numerical simulations for further parametric studies in Chapter 6. The numerical models are comprehensively validated by the four tests to ensure reliability.

For anti-progressive collapse design, AP method is the main stream approach, upon which nonlinear static analysis with DIF is recommended. It is believed that DIF in DoD (2013) is not safe for elastic-plastic hardening and softening-hardening systems. 3D composite floor systems can be quasi-statically tested to failure (Chapters 3, 4

and 5) to inspect if they belong to elastic-plastic hardening and softening-hardening systems. Furthermore, previous studies on 2D steel frames showed that the formula for calculating DIF in the current design guides is unsafe. However, this conclusion should not be generalised to 3D composite floor systems, of which dynamic behaviour needs further investigations. It is prohibitively expensive to conduct 3D experimental tests to study DIF for a sub-structure, so numerical simulations are the best choice. Unfortunately, as shown in Table 2.1, only one study had involved DIF estimation for 3D composite floor systems, showing the opposite trend compared to conclusions in 2D frames (Liu et al. 2013, 2015b, Fu et al. 2016, Liu et al. 2016). Hence, the DIF for 3D composite floor systems should be further evaluated. In Chapter 6, the author uses a reliable numerical model to study DIFs of a 3D sub-structure, which are further compared with those of 2D assemblies (Liu et al. 2013, Liu et al. 2016) and with that in DoD (2013). Besides, the current guides do not consider the dynamic effect on ductility demand. Also, there is no publication on dynamic magnification of deflections for 3D composite floor systems. This provides the impetus for the author to evaluate displacement-based DIF in Chapter 6.

For practical applications, there are very few analytical models for steel composite structures. Dusenberry and Hamburger (2006)'s model does not consider joint behaviour and the slab effect for steel composite structures. Izzuddin et al. (2008)'s framework either neglects the slab effect when using simplified grillage models or needs high expertise in using FE software. The ideal model should capture the detailed structural behaviour as much as possible, but at the same time, the method can be implemented in common office tools through Excel spreadsheet. Therefore, the above two models are not accurate or practical in office design. Alashker and El-Tawil (2011)'s design-oriented model cannot predict the load-deflection response at small deformation stage, so that it cannot be converted into the pseudo-static response based on energy balance method to enable assessment of robustness in dynamic environment. To overcome these shortcomings, in Chapter 7, the author develops a mechanical model for calculating the entire push-down load-deflection response of composite floor systems subjected to an internal column loss scenario. The model can capture the effects of key parameters, such as slab aspect ratio, joint type, number of bolts in the joint, slab thickness, reinforcement ratio in the slab and thickness of steel

decking. Most importantly, it can be implemented by a spreadsheet.

The following chapter introduces the experimental programme conducted by the author at Chongqing University. The author was jointly supervised by Professor Yang Bo from Chongqing University and Professor Tan Kang Hai on her conduct of the four experimental tests.

CHAPTER 3 EXPERIMENTAL PROGRAMME ON 3D COMPOSITE FLOOR SYSTEMS SUBJECTED TO AN INTERNAL COLUMN REMOVAL SCENARIO

3.1 Introduction

Technical gaps in existing research necessitate experimental studies on 3D composite floor systems subjected to a column removal scenario to realistically investigate the structural behaviour incorporating the slabs. Besides, the test results can be used to comprehensively validate numerical models and analytical approaches, and can enrich the test data base of 3D composite structures.

This chapter introduces the design of four one-third scaled sub-structures, followed by descriptions of the test set-up and the loading scheme. Considering the loading issues encountered by the previous tests (Hull 2013, Johnson et al. 2016), the loading in this experimental programme was in displacement control to obtain complete structural response up to failure. Specially designed distributed beam transfer system applied a concentrated load to the slabs through 12 points, to simulate distributed dead and imposed loads. The set-up also simulated restraints from surrounding structures. The reaction forces in the test set-up were measured by instrumentation so that the entire system was statically determinate and the load-resisting mechanisms in the floor systems could be studied in detail. Besides, instrumentation was also arranged for each specimen to measure strain, vertical and horizontal deflections, and rotations associated with the floor. This chapter ends with the measured mechanical properties of materials used in the test programme.

3.2 Design of Specimens

According to specifications on load amplification associated with AP method, both DoD (2013) and GSA (2013) indicate that load redistribution areas are the bays just above the removed column. DoD (2013) does not allow the areas to fail, while GSA (2013) allows them to fail in a proportionately manner relative to the magnitude of the source. In this chapter, the research focuses on the behaviour of these floors directly affected by the column loss. The current studies are based on the assumption

that each affected floor behaves similarly since columns above the damaged level experience an identical rigid body displacement, as shown in Fig. 3.1. Thus, 3D specimens could be designed as a representative one-storey floor system. Since virendeel action exists in multi-storey buildings after losing a column and each floor would not absolutely behave similarly. It is an approximate assumption that makes the possibility for experimental studies in the constraint lab condition. The assumption is conservative for upper storeys and may not for lower storeys. The effect of virendeel action will be further studied using numerical models in the near future. This programme of tests aimed to study structural behaviour of 3D composite floor systems under an internal column removal scenario (Fig. 1.3 (a)). Accordingly, the test specimens were designed to be one-storey, two-by-two-bay sub-structures (Fig. 3.2), which were extracted from a typical steel-frame-composite-floor building subjected to a dead load of 4.0 kN/m^2 and a live load of 3.0 kN/m^2 . The composite floors including slabs and steel beams were extended by $1/4$ of their neighbouring spans to simulate adjacent spans from surrounding structures in the prototype building. Firstly, continuity of composite slabs around the specimens was ensured. Secondly, the floor was extended to coincide with inflection points, which were restrained vertically and horizontally without any rotational restraints (refer to Section 3.3). Considering the laboratory space constraint, the test specimens were scaled down from the full-scale substructure to $1/3$ models. The structural layout of a typical specimen is shown in Fig. 3.2, where “girder” indicates main beam and “beam” and “inner beam” belong to secondary beam. It should be noted that that, in this thesis, “secondary beams” are used to include both beams and inner beams, and “steel beams” include girders, beams and inner beams. The overall height of specimens was 1.4 m, scaled down from a storey height of 4.2 m in the prototype building.

Fig. 3.1: Isolation of the studied sub-structure

*H 200×100×8×12—H represents the “H” shape wide-flange section and the numbers are the depth, flange width, web thickness, and flange thickness in mm, respectively.

All the dimensions are in mm.

Fig. 3.2: Structural layout of the specimens

As reviewed in Chapter 2, the welded moment-resisting beam-to-column connections are costly and have limited rotational capacities (Krauthammer 1999, Khandelwal et

al. 2008, Lew et al. 2013, Brunesi et al. 2015) to develop CA. Dinu et al. (2016) found that end-plate bolted connections had sufficient strength to develop CA in beams. The test results (Yang and Tan 2013a) revealed that flush-end-plate joints can develop both flexural action and CA well. Hence, the girder-to-column connections in the present specimens adopted bolted flush-end-plate joints. Simple joints, such as web-cleats and fin-plates have enough ductility to mobilise CA at large deformation stage (Yang and Tan 2013a). However, fin-plate connections do not have sufficient strength to resist the load shifted from the removed column (Khandelwal et al. 2008, Johnson et al. 2016). Thus, the beam-to-column and inner beam-to-girder connections in the present specimens employed double web-cleat connections. The good load-carrying and rotational capacities of flush-end-plate and double web-cleat joints could be relied on for load redistribution in the slab system.

The composite slabs consisted of a 40 mm deep by 0.9 mm thick profiled steel decking, 65 mm thick concrete slab, and $\phi 6$ reinforcing mesh at 100 mm spacing in both directions. Each piece of profiled decking was 740 mm in width, so the steel decking throughout a floor was not continuous in short-side direction. Pieces of profiled decking were overlapped at the adjacent ribs, locked by preformed shapes. The reinforcing mesh was directly placed on the steel decking. When typical hot rolled deformed bars of 10 mm were scaled down to roughly one-third, only mild steel round bars of $\phi 6$ could be used, with greater ductility but weaker bond strength. Fig. 3.3 shows the detailed cross-sections of the composite slabs involving two types of shear stud arrangement. The purpose of this design will be introduced in the following paragraph. It could be noticed that the slabs (65 mm thick) may be slightly thicker than the ideal 1/3-scale slabs. However, 40 mm deep profiled sheeting was the shallowest type can be purchased from suppliers.

(a) Strong composite action (Full composite action)

(b) Weak composite action (Degree of partial connection was 34%)

* All the dimensions are in mm

Fig. 3.3: Details of the composite slabs

The similar situation was met when choosing steel sections. Wide-flange section H 200×100×5.5×8 was the smallest section for Q345B with nominal yield strength of 345 MPa (GB 50017 2013), and H 150×75×5×7 was the smallest section for Q235B with nominal yield strength of 235 MPa. The term H represents the “H” shape and the numbers are the depth, flange width, web thickness, and flange thickness in mm, respectively. H 200×100×5.5×8 and H 150×75×5×7 were slightly larger than expected sections (viz. H 150×75×5×7 for girders and H125×60×6×8 for secondary beams), but just matched with the floor slabs in terms of resistance. Accordingly, the joints were designed based on selected beam sections. For instance, angle section L 90×56×5 with longer leg length of 90 mm, shorter leg length of 56 mm and thickness of 5 mm was used to connect beams and column webs considering the sizes of beam and column sections. Determination of the bolt size and positions of bolt holes should consider strength and ductility of bolt-row components (Fig. 3.4), feasibility of installation, and design guide specifications (EC 3-1-8 2005). All the detailed geometry information of the joints is shown in Fig. 3.4. The inner beam-to-girder connection is the same as the beam-to-column connection (Fig. 3.4 (a)) The material properties of steel materials are presented in Section 3.5. To ensure the robustness of joints subjected to large deformation, force-displacement relationships of individual components in web-cleat connections (WC in Fig. 3.4 (a)) were predicted beforehand based on Yang and Tan (2013c)’s model, and those in flush-end-plate connections (Fig. 3.4 (b)) were based on the models proposed by Gioncu and Petcu (1997) and Faella et al. (2000). These joint component properties are plotted in Fig. 3.5.

As a whole, these four specimens could be regarded as small-scale prototype structures with enhanced slabs, members and connections. Therefore, the uniformly distributed load (UDL)-carrying capacities of these specimens should be greater than those of commonly designed prototype structures. Nevertheless, the relative strength of different structural members including slabs, steel beams and joints was maintained in the reduced models, as demonstrated in Section 5.6, so sequence of damage, load-resisting mechanisms and parametric influences obtained from these reduced model tests can be applied to full-scale floor systems.

(a) Beam-to-column connection

(b) Girder-to-column connection

* All the dimensions are in mm

Fig. 3.4: Details of the connections and individual components in joints

Fig. 3.5: Force-displacement relationships of joint components

In total, four specimens (Fig. 3.6 (a)~(d)) were labelled as 2×3-S-IC, 2×2-S-IC, 2×3-W-IC and 2×3-S-PI, as shown in Table 3.1, with “IC” denoting internal column removal and “PI” penultimate internal column removal (Fig. 1.3). Specimen 2×3-S-IC was the control specimen. For comparison, specimens 2×2-S-IC, 2×3-W-IC, and 2×3-S-PI varied in terms of aspect ratio (span length), degree of composite action between slabs and steel beams, and boundary condition, respectively, with “S” implying strong (full) composite action and “W” weak (34%) composite action (column removal location); 2×3 and 2×2 indicate the dimensions of slab panels in m (Fig. 3.2). For specimens with strong composite action, shear studs were placed at 75 mm spacing on girders and 92 mm on secondary beams (Fig. 3.3 (a)). In contrast, shear studs in 2×3-W-IC were placed at 225 mm spacing on girders and 370 mm spacing on secondary beams (Fig. 3.3 (b)), which satisfied the maximum spacing requirements in EC4-1-1 (2004). In the actual tests, IC denoted the specimens were restrained at four edges (Fig. 3.6 (a), (b) and (c)), while PI indicated the specimen was restrained only at three edges (Fig. 3.6 (d)). The geometry and details of these specimens are included in Table 3.1. Comparing the test results of 2×2-S-IC, 2×3-W-IC and 2×3-S-PI with that of 2×3-S-IC, it can be obtained the effects of slab aspect ratio, degree of composite action, and boundary condition, respectively. It should be

noted that the actual slab thickness was measured with a leveling instrument and a ruler. The cast slab thickness of 2×3-S-IC, 2×2-S-IC, 2×3-W-IC was 65 mm, while that of 2×3-S-PI was 70 mm due to 5 mm constructional error. This should be noted when comparing the test results among the four specimens in Chapter 5.

(a) Specimen 2×3-S-IC

(b) Specimen 2×2-S-IC

(c) Specimen 2x3-W-IC

(d) Specimen 2x3-S-PI

*LVTD—Linear Variable Differential Transducer

Fig. 3.6: Test specimens

Table 3.1: Parameters of the four specimens

Specimen ID	Slab panel dimensions	Slab thickness	Aspect ratio	Boundary condition	Composite action
2×3-S-IC (Control)	2 m×3 m	65 mm	1:1.5	IC	Strong
2×2-S-IC	2 m×2 m	65 mm	<u>1:1</u>	IC	Strong
2×3-W-IC	2 m×3 m	65 mm	1:1.5	IC	<u>Weak</u>
2×3-S-PI	2 m×3 m	70 mm*	1:1.5	<u>PI</u>	Strong

*The intended thickness is 65 mm.

3.3 Loading Scheme and Set-up

In practice, if a column in a building is removed, the immediate floor above the removed column would deform under dead and imposed loads. Twelve-point loading was utilised to represent dead and imposed loads, which were quasi-statically applied under displacement control to obtain the full load-deflection curve. Thus, push-down loading condition was applied to study structural response. To apply displacement control, a special loading system was designed with three tiers of steel beams culminating in 12 loading points, as shown in Fig. 3.7. Specifically, the first-tier beam was subjected to a 200-ton jack, vertically mounted onto a strong A-frame. The first-tier beam transferred the single point load onto the second-tier beams, which in turn applied four point-loads to four Y-shape third-tier beams through ball-and-socket bearings, capable of large rotations to ensure verticality of applied loads as shown in Fig. 3.8 (a). The four Y-shape beams transferred the vertical loads to the composite slabs via 12 loading points (Fig. 3.7). The three loading points from the Y-shape loading beam corresponded to the vertices of a triangle and each ball-and-socket bearing was placed at the centroid of each Y-shape beam to ensure that the loads passing through each loading point were equal. It should be mentioned that the jack could apply the load under displacement-control through an intelligent system. Specifically, there was a displacement transducer to measure the jack extension; the transducer reading was then fed back to the intelligent system, which controlled the jack through analysing the target displacement and a feedback loop.

Specimens 2×3-S-IC, 2×2-S-IC and 2×3-W-IC were restrained by one set of vertical and diagonal Circular Hollow Section (CHS) (Fig. 3.8 (a)) braces at the ends of the cantilever beams (Fig. 3.9) via pins at three edges and by one set of vertical and horizontal CHS at one edge. The ends of vertical and diagonal CHS were connected to the strong floor, while the ends of horizontal CHS to the reaction wall via pin connections. For 2×3-S-PI, vertical and diagonal CHS were removed from one edge, as shown in Fig. 3.8 (b).

*250/750, 2000/3000, 5000/7000 — the contents before and after the slashes were used for the slab panels with dimensions of 2m×2m and 2m×3m, respectively.

* R is short for restraint.

Fig. 3.7: Numbering of columns, beams, restraints and loading positions

(a) For the specimens under IC scenario (2×3 -S-IC, 2×2 -S-IC and 2×3 -W-IC)

(b) For the specimens under PI scenario (2×3 -S-PI)

Fig. 3.8: Test set-up

3.4 Instrumentation

3.4.1 Measurements of loads and reaction forces

*CHS-D and -V represent diagonal and vertical Circular Hollow Sections, respectively.

Fig. 3.9: Restrained condition and arrangement of strain gauges

The applied load could be measured by a load cell placed underneath the hydraulic jack. To obtain reaction forces in the columns for investigating load redistribution among columns and load-resisting mechanisms in the composite floors, the strains at two cross-sections (Fig. 3.9) of each H-shaped column were measured. Four strain gauges were attached onto the outer flange tips of each column cross-section. The axial forces, shear forces and moments could be calculated as follows:

$$N_c = \frac{1}{2} \times (N_1 + N_2) = \frac{1}{8} \times E \times A_c \times \sum_{i=1}^8 \varepsilon_i \quad (3.1)$$

$$M_{1,ma} = \frac{1}{2} \times E \times W_{c,ma} \times \left(\left(\frac{\varepsilon_1 + \varepsilon_2}{2} - \frac{N_1}{E \cdot A_c} \right) - \left(\frac{\varepsilon_3 + \varepsilon_4}{2} - \frac{N_1}{E \cdot A_c} \right) \right) \quad (3.2)$$

$$M_{1,mi} = \frac{1}{2} \times E \times W_{c,mi} \times \left(\left(\frac{\varepsilon_1 + \varepsilon_3}{2} - \frac{N_1}{E \cdot A_c} \right) - \left(\frac{\varepsilon_2 + \varepsilon_4}{2} - \frac{N_1}{E \cdot A_c} \right) \right) \quad (3.3)$$

$$V_{ma} = \frac{M_{1,ma} - M_{2,ma}}{H_c}; V_{mi} = \frac{M_{1,mi} - M_{2,mi}}{H_c} \quad (3.4)$$

where N_1 and N_2 are the respective axial forces at Section 1 and 2 in Fig. 3.9; N_c is the average value of N_1 and N_2 , and used to represent the axial force in a column. The term E is Young's modulus of steel columns; A_c is the area of column cross-section; ε_i is the strain at location i ($i=1,2,\dots,7,8$ as shown in Fig. 3.9). The terms $M_{1,ma}$ and $M_{1,mi}$ are the respective bending moments about major and minor axes at Section 1 of a column, while $M_{2,ma}$ and $M_{2,mi}$ are the moments at Section 2. The terms $W_{c,ma}$ and $W_{c,mi}$ are the respective sectional modulus about the major and minor axes of the column section. The term H_c is the distance between the two measured sections.

As shown in Fig. 3.9, four strain gauges were attached on each restrained CHS. The axial force in a CHS could be obtained through multiplying average strain (ε_{ave}) by the product of Young's modulus of the restraint (E_R) and area of the cross section (A_R), as follows:

$$N_R = E_R \times A_R \times \varepsilon_{ave} = E_R \times A_R \times \frac{\varepsilon_a + \varepsilon_b + \varepsilon_c + \varepsilon_d}{4} \quad (3.5)$$

where ε_a , ε_b , ε_c and ε_d is the strain at locations a, b, c, and d in a CHS (Fig. 3.9). The reaction forces in a diagonal and a vertical CHS can be decomposed into a vertical (R_V) and a horizontal (R_H) reaction forces acting on the specimens (Fig. 3.9).

For the set of vertical and horizontal CHS on one edge, the reaction force in each CHS can be obtained in the same way. The vertical (R_V) and horizontal (R_H) components of reaction force can also be similarly obtained.

3.4.2 Measurements of vertical deformations and rotations

Fig. 3.10 shows the arrangements of transducers and inclinometers. Vertical displacements were measured by LVDTs or Line Transducers (for V-2, V-3a, V-3b, and V-5), while horizontal displacements were measured by LVDTs. Among the vertical measurements, V-3 (the average value of V-3a and V-3b) was a vital indicator for the floor centre deformation. V-1,2,4,5 were the vertical displacements corresponding to the loading points (Fig. 3.7 and Fig. 3.10), V-6 was the central deflection of girder B2, and V-7, 8 were the respective vertical displacements at

middle span of peripheral girder B10 and peripheral beam B15. All these vertical measurements could be used to plot the general deformed shape of each floor system. To obtain the horizontal stiffness at the ends of the double-span girder and the double-span beam, horizontal displacements were measured at R1 and R9 (*H-R1* and *H-R9* in Fig. 3.10) and R5 and R11 (*H-R5* and *H-R11* in Fig. 3.10) for all the four specimens. Since R1, R2, and R2' (see Fig. 3.7 and Fig. 3.8 (b)) were removed for specimen 2×3-S-PI. The horizontal displacements of all four specimens were measured at R1, R2, and R2' (*H-R1*, *H-R2* and *H-R2'* in Fig. 3.10) for comparison purpose. To observe local rotations at the ends of steel beams, inclinometers were attached on the ends of girder B1 (*I-1* and *I-2*), beam B4 (*I-7* and *I-8*) and inner beam B5 (*I-11* and *I-12*), as shown in Fig. 3.10.

3.4.3 Measurements of strain

Fig. 3.11 (a) shows that the end- and middle sections of beam B4 (B4-1, B4-2 and B4-3) and inner beam B3 (B3-1, B3-2 and B3-3) were measured, while only the end-sections of girder B1 (B1-1 and B1-2) were measured considering stress concentration at the middle section. Fig. 3.11 (b) shows the strain gauges were adhered to four bottom tips of each measured section. The strain gauges on the steel decking (D1 through D15) were arranged along the secondary beams (Fig. 3.11 (a)). It should be mentioned that the vertical LVDTs and strain gauges were only placed on one-quarter of specimens 2×3-S-IC, 2×2-S-IC and 2×3-W-IC taking advantage of double symmetry. For 2×3-S-PI, vertical LVDTs were also placed under one-quarter of the slab. However, strain gauges were placed on the bottom section of column C4 (Fig. 3.11 (c)) to monitor the behaviour of C4, since it was most vulnerable due to the absence of lateral restraints.

Fig. 3.10: Arrangements of instrumentation for all the four specimens

(a) Measured locations on girders, beams, inner beams and steel decking for all the four specimens

(b) Arrangement of strain gauges on the sections of girder, beam, inner beam for all the four specimens

(c) Additional strain gauges on C4 of 2×3-S-PI

Fig. 3.11: Arrangement of strain gauges

3.5 Material Properties

Grade 8.8S bolts in GB50017-2003 (2013) with the diameter of 16 mm were used for all joints. The nominal yield strength and ultimate tensile strength are 640 MPa and 800 MPa, respectively. Steel coupon tests were carried out to obtain material properties of steel beam sections, column sections, restrained CHS braces, angles and end plates used in joints, reinforcement and profile sheeting. Since the CHS braces would remain elastic throughout the test, only Young's modulus was measured. The Young's modulus, yield strength, ultimate strength and ultimate strain of all the other

materials are given in Table 3.2. Besides, the 150 mm concrete cube strength was 54 MPa obtained from four samples.

Table 3.2: Material properties for steel members

Steel material	f_y (MPa)	f_u (MPa)	E (GPa)	Ultimate strain ϵ_u^*
H150×75×5×7	283.7	426	195.0	0.290
H200×100×5.5×8	378.9	531.5	206.5	0.245
H200×200×8×12	353.2	507.2	202.5	0.253
End plate	450.7	638.6	212.6	0.200
L90×56×5	371.7	540.5	207.1	0.244
Profile sheeting	333.5	396.0	208.4	0.189
Reinforcement	340.3	503.1	220.4	0.227
CHS	\	\	209.6	\

*Uniform strain corresponding to the ultimate tensile strength of the tested structural steel.

3.6 Summary

This chapter describes an experimental programme on four two-by-two-bay composite floor systems. The following aspects are included:

- (1) One-storey representative sub-structures were isolated and scaled down from a steel building subjected to either internal or penultimate internal column loss. Accordingly, the set-up could be used to provide appropriate restraint from surrounding structures.
- (2) Arrangements of instrumentation on specimens and restrained set-up were clearly described.
- (3) The reaction forces acting on the floor systems could be quantified based on the strain gauge readings. The formula to calculate shear force, axial force and bending moment for each column and axial force for each restrained CHS were listed in this chapter.
- (4) The properties of all the materials used in the specimens were presented at the end of this chapter.

The test results will be discussed in the following two chapters.

CHAPTER 4 LOAD-RESISTING MECHANISMS OF 3D COMPOSITE FLOOR SYSTEMS UNDER AN INTERNAL COLUMN REMOVAL SCENARIO

4.1 Overview

In this chapter, based on the test result of one specimen (2×3-W-IC), the author classifies the structural response of 3D composite floor systems under an internal column removal scenario into three stages. At the first stage, flexure in slabs and the double-span girder above the missing column dominates the structural resistance. In the second stage, TMA instead of flexure in the slabs becomes dominant; the double-span girder gradually failed, while the double-span beam was gradually mobilised. At the final stage, TMA in slabs and the double-span-beam were the major contributors to vertical loads. Stress states and deformations of the floor components including girders, beams and steel decking are also presented. Most importantly, the author attempts to clarify load-resisting mechanisms (alternate load paths) for 2×3-W-IC through analysing reaction forces acting on the statically-determinate floor system. It is confirmed that the vertical load was redistributed through the double-span girder and the double-span beam above the missing column, and composite slabs. The individual contributions of these three paths are quantified. After that, the relative importance of each alternate load path at different loading stages is quantified to allow engineers to gain a deeper appreciation of each load path.

4.2 Experimental Results of Specimen 2×3-W-IC

4.2.1 Load-deflection curve

The curve marked “Load cell” in Fig. 4.1 is the load cell reading versus the average slab central displacement V-3 (Fig. 3.10). The other load-deflection curve is the measured total vertical reaction forces from the column supports and restraints. Clearly, excellent agreement between applied loads and measured reaction forces demonstrates that the system of load measurements was adequate and could be used for detailed evaluations. Fig. 4.1 indicates that the maximum load and the corresponding deflection were 1243 kN and 537 mm, respectively. The ultimate state

was defined as the maximum load of the 3D composite floor system, when all joint components at the removed column connection failed and the slab along the interior sides of the beams and girders also formed negative yield lines and fractured at the perimeter. The entire structural response can be classified into three stages as demarcated by the two vertical dash lines in Fig. 4.1. The first stage is defined between the central displacement of 0 mm and 166 mm ($0.63h_g$), when the first bolt at girder B2-to-removed column connection (Fig. 3.7) fractured, as shown in Fig. 4.2 (a). In Stage 1, the load increased rapidly and reached a plateau. Then Stage 2 began and ended with complete bolt failure of the same girder-to-column connection (Fig. 4.2 (b)), after which Stage 3 ensued. The load remained constant during Stage 2 and then began to increase moderately during the first phase of Stage 3. The load-deflection response shows descending branch at the end of Stage 3.

In stage 1, flexure in slabs and double-span girder B1-B2 (Fig. 3.7) were the dominant mechanisms accompanied by Compression Arch Action (CAA) in B1-B2. In stage 2, CAA in B1-B2 and B4-B7 vanished and CA in B1-B2 and B4-B7 was mobilised. TMA was mobilised from the end of Stage 1. The related evidence is shown in Sections 4.2.3.2 and 4.3.1.

Fig. 4.1: Three stages of the load-deflection response

(a) First bolt failure

(b) Last bolt failure

*S indicates the sagging end of a steel beam.

Fig. 4.2: Failure of the girder-to-column connection at the removed column location

4.2.2 Failure modes

Local failure modes of the 3D composite floor system were quite complex since there were so many elements, including steel columns, girders, beams, concrete slabs, bolts, steel angles, end plates, steel decking, shear studs and reinforcement. Similar to the three loading stages shown in Fig. 4.1, failure of the specimen could also be described in three stages. In Stage 1, the composite slab cracked along the top-flange edges of double-span girder B1-B2 and double-span beam B4-B7 bridging over the missing column (Fig. 4.3 (a)), and the cracks further spread along the top-flange edges of perimeter girders and beams due to negative bending moment. At the end of Stage 1, the concrete between loading points 1, 2 and 3 and peripheral girders B9 and B10, and between loading points 10, 11 and 12 and peripheral girders B11 and B12 spalled off (Fig. 4.3 (a) and Fig. 4.4 (a)) due to the failure of shear studs near the hogging ends of secondary beams, as shown in Fig. 4.4 (b).

Stage 2 showed the whole process of bolt fracture in girder B2-to-removed column connection as shown in Fig. 4.2. The reason that the bolt fracture only occurred in one connection was that the first bolt failure caused the shifting of forces to the remaining top two rows of bolts (compared with the intact connection by girder B1), which led to successive failure of the connection on one side only. Meanwhile, concrete cracks along the girders and the beams opened more widely, with concrete spalling around columns C4 and C5, and concrete crushing around the removed

column and along the paths from loading points 1, 3, 10 and 12 to the respective corner columns, as shown in Fig. 4.3 (b) and Fig. 4.5 (a). Concrete spalling and crushing was caused by negative and positive bending moments at the corresponding regions, respectively. Besides, more shear studs fractured and secondary beams yielded at the sections under the loading positions 1, 2, 3, 10, 11 and 12, as shown in Fig. 4.5 (b). The load increased slowly during this stage.

In the first phase of Stage 3 (Fig. 4.1), the applied load grew moderately, which happened after the complete failure of girder B2-to-removed column connection. The increased load was redistributed through two means. Firstly, the double-span beam (B4-B7) supported the separated double-span girder (B1-B2). Secondly, the composite slab worked like a net and transferred the load to the peripheral girders and beams. The structural elements continued to deform gradually with increasing load. As shown in Fig. 4.3 (c) and Fig. 4.6 (a), the crushed and spalled concrete regions expanded quickly. Steel decking fractured around the removed column, peripheral columns (C4 and C5) and under the loading points (Fig. 4.6 (b) and (c)). Some reinforcement fractured around the removed column and peripheral columns (C4 and C5) (Fig. 4.6 (d)). Some bolts connected to columns C4 and C5 fractured (Fig. 4.6 (c)) while plastic hinges were formed in secondary beams (Fig. 4.6 (e)). Most of the shear studs fractured (Fig. 4.6 (e)) and the connection angles to support secondary beams fractured or became highly deformed (Fig. 4.6 (f)). All these failures caused the descending branch at the last phase of Stage 3. It also can be seen that the connections had adequate rotation capacity to allow the slab system to fail in a ductile way.

Additionally, an interesting phenomenon was that some small cracks developed along the re-entrants of decking and spread throughout the whole floor, as shown in Fig. 4.3 (c) and Fig. 4.6 (a). The reason may be that most shear studs had failed at the final loading stage, causing separation of the slab from the girders and beams. In other words, at the final stage the slabs worked as a pure tensile membrane without any beams or girders. Under this circumstance, such kind of cracks appeared since the concrete thickness above the re-entrant profile decking was small (25 mm).

Fig. 4.3: Failure modes

*H indicates the hogging end of a steel beam.

Fig. 4.4: Failure at Stage 1

(a) Concrete spalling and crushing

(b) Yield of secondary beams

Fig. 4.5: Failure at Stage 2

(a) Overview of final failure

(b) Failure of decking

(c) Failure of bolts connecting girders to peripheral columns

(d) Failure of reinforcement at middle and ends of the double-span girder

(e) Failure of shear studs

(f) Failure of the angles

Fig. 4.6: Failures at Stage 3

4.2.3 Measurements of strain in the floor

4.2.3.1 Strain in steel beams

Fig. 4.7 shows the strain development in girder B1 and beam B4 (Fig. 4.3) during the whole loading process. The positions of these beams and the measured sections are given in Fig. 3.11 (a). Fig. 4.7 (a) indicates that at girder section B1-1 (near the removed column), initial tensile strain in the bottom flange was larger than that of the top flange, showing that Section B1-1 was under a sagging moment. After the middle joint reached a displacement of 150 mm ($0.57h_g$), the strain in the bottom flange of section B1-1 dropped dramatically due to failure of bottom bolts in girder B2-to-removed column connection. As shown in Fig. 4.7 (b), section B1-2 (near peripheral column C4) was under a hogging moment. The bottom flange yielded at a displacement of 100 mm ($0.38h_g$) and then the strain was maintained until the end of the test, demonstrating that the adopted flush-end-plate connection could provide moment resistance and rotation capacity. At the small deformation stage, the bending moment diagram of the double-span girder (B1-B2) is shown in Fig. 4.8 (a), which is consistent with conventional understanding of the behaviour of the girders under a middle column removal scenario.

Fig. 4.7: Strains in steel beams

However, strain readings on the double-span beam (B4-B7) were rather surprising. For secondary beam B4, it worked like a simply supported beam, with all the sections

under two-point sagging moments as shown in Fig. 4.8 (b). As demonstrated in Fig. 4.7 (c), (d) and (e), all the top flanges of B4 were in compression and all the bottom flanges were in tension. At the beginning, the magnitudes of the strain on both the top and bottom flanges increased, caused by the increased bending moment. This phase did not exceed the displacement of 200 mm (0.93 times the depth of composite beam including the slab, or $0.93h_b$). After that, the magnitude of compressive strain in the top flange reduced while that of tensile strain in the bottom flange increased, showing the development of axial force in the beams. From then on, CA kicked in the double-span beam (B4-B7). It was interesting to observe that the bending moment at section B4-2 (at the middle span of beam B4) was much larger than that at section B4-1 (close to the removed column) when comparing the strains in the flanges of both sections in Fig. 4.7 (c) and (d), which means that B4-B7 did not work as a double-span beam. Instead, beam B4 functioned more like a simply supported beam with one end supported by column C2 while the other end supported by the double-span girder B1-B2 (Fig. 4.8 (b)). The behaviour of beam B4-B7 was similar to those of inner beams B3-B6 and B5-B8. Therefore, it can be concluded that the double-span girder transferred the loading from the secondary beams to peripheral columns C4 and C5 when the girder-to-column joints were still fully effective, i.e. at Stage 1.

Fig. 4.8: Bending moment diagram of the double-span girder/beam

4.2.3.2 Strain in steel decking

As shown in Fig. 3.11 (a), strain in steel decking was measured along its supports (beams B3, B4 and B13), with D referring to steel decking followed by the gauge numbers. Fig. 4.9 (a), (b) and (c) show the strain development of the steel decking. The steel decking along secondary beams B3 and B4 were always in tension throughout the whole loading process, while the strains of steel decking along peripheral beam B13 were in very small compression initially and changed to tension at a displacement of about 70 mm (1.1 times the slab thickness, or $1.1h_s$). These phenomena revealed that the composite slab around the peripheral beams was subjected to hogging moments initially and changed to tension subsequently, while the inner slab was always under tension. This indicated the expansion of inner tensile region caused by TMA.

Fig. 4.9: Strain in steel decking

4.2.4 Measurements of displacements and inclinations

As mentioned in Section 3.4.2, the deflections of beams B2, B4 and B5 were measured at locations corresponding to the loading positions (Fig. 3.10) to capture the deformed profile of the composite slab. The 3D deformed shapes of one-quarter of the floor system at different loading stages are shown in Fig. 4.10. Girder B2 deformed linearly throughout the whole loading process, while secondary beams B4 and B5 deformed in a multi-linear manner due to multi-point loading. This type of deformed shape is similar to a polynomial deflection curve caused by uniform loads. The difference in deflected shape between the girder and the secondary beam was caused by two reasons. Firstly, the missing internal column released the restraint at one end of the girder. Secondly and more importantly, the flexural rigidity of girder B2 was much greater than that of secondary beams (B4 and B5) to resist applied loads. As a result, the bending deformation of the girder was much smaller than those of secondary beams. It should be noted that after the central displacement was over $1.12h_g$ (296 mm), girder B2 was separated from the column stub (Fig. 4.10 (c)). After that, the contribution of double-span beam (B4-B7) increased, while that of the separated double-span girder (B1-B2) decreased rapidly.

(a) At a central deflection of 100 mm ($0.38h_g$, Stage 1)

(b) At a central deflection of 250 mm ($0.94h_g$, Stage 2)

(c) At a central deflection of 500 mm ($1.89h_g$, Stage 3)

Fig. 4.10: Deformations of the floor at different central deflections

Fig. 4.11 shows the inclination angles of girder B1 and beam B4 were measured by *I-1*, *I-2*, *I-7* and *I-8* (Fig. 3.10). They represented local rotations at the ends of girder B1 and beam B4. These angles are respectively compared with rotations of the girder and the beam segments, which are calculated based on vertical displacements measured by *V1*, *V2* and *V3* (Fig. 4.11). It is found that the local angles agree well with the corresponding girder and beam segment rotations, confirming that the

girders had a linear deformed profile while the beams deformed in a multi-linear shape (Fig. 4.10). The inner secondary beams also behaved like beams B4 and B7 and they had multi-linear shape.

Fig. 4.11: Comparisons between the member rigid rotations and the local inclinations

4.2.5 Measurements of reaction forces

Under a column loss scenario, horizontal restraint is very crucial to the load-transfer mechanisms, such as the development of CAA, CA and TMA, which would influence the load-carrying capacity and ductility of a structure. In this study, sufficient horizontal restraints were provided and all the reaction forces in the restrained CHS were measured. Herein, Fig. 4.12 shows the horizontal reaction forces in R1, R5, R9 and R11 (Fig. 3.7), which correspond to the restraints of double-span girder B1-B2 and beam B4-B7. These reaction forces can demonstrate the mobilisation of CA in the double-span girder B1-B2 and the double-span beam B4-B7 bridging over the missing column. As the horizontal CHS of R9 was connected to the reaction wall (Fig. 3.7 and Fig. 3.8), the horizontal stiffness of R9 (230 kN/mm) was much greater than those of R1 (45 kN/mm), R5 (30 kN/mm) and R11 (40 kN/mm). Fig. 4.12 shows that the high horizontal stiffness of R9 kept the double-span girder in compression for the longest time, which caused the double-span girder to change into tension at the very late stage so that tensile force was only developed prior to the ultimate state. The reason that the tensile horizontal force in R9 dropped to zero and further changed

into compression again (Fig. 4.12) was that the double-span girder B1-B2 had completely failed in a joint connecting to the removed column stub, causing the structure to lose symmetry and to lean towards the reaction wall. Horizontal reaction forces at the other restraint locations (R1, R5 and R11) changed from compression to tension at a displacement smaller than one depth of the composite girder (h_g) including the slab ($h_g = 265$ mm in this case). As expected, all the reaction forces experienced compression initially and then changed into tension, showing the presence of CAA and CA, respectively. But this transition could not be used to demarcate the boundary between CAA and CA for the floor system due to different stiffness values of horizontal reaction forces. In the next section, the occurrence and contribution of CA towards collapse load will be discussed.

Fig. 4.12: Horizontal reaction forces at ends of double-span girder and double-span beam

4.3 Discussions on Load-Resisting Mechanisms

4.3.1 Individual contributions of the floor system members

After an internal column removal, the floor system transferred the vertical loads to the surrounding intact frames through the double-span girder B1-B2, the double-span beam B4-B7 and the composite slabs. The inner beams (B3, B6, B5 and B8) transferred loads to the double-span girder B1-B2 and peripheral beams. Therefore, the former and latter parts of loads should be attributed to the contributions of double-span girder B1-B2 and the composite slabs, respectively. In other words, the sum of vertical loads carried by the (a) double-span composite girder, (b) double-span

composite beam and (c) composite slabs with inner beams should be equal to the total applied loads. Furthermore, the loads carried by the double-span composite girder and beam could be attributed to flexural action and CA. As shown in Fig. 4.13, $V_B \cdot \cos\theta$ and $N_B \cdot \sin\theta$ represent the contributions of flexural action and CA to resist the vertical loads, respectively. Angle θ can be calculated based on the vertical displacements measured by the corresponding LVTDs. The terms V_B and N_B can be calculated based on force equilibrium, as shown in Eqs. (4.1) and (4.2).

$$N_B = (R_H - V_C) \cdot \cos\theta - (N_C + R_V) \cdot \sin\theta \quad (4.1)$$

$$V_B = (V_C - R_H) \cdot \sin\theta - (N_C + R_V) \cdot \cos\theta \quad (4.2)$$

where R_H , R_V , V_C , N_C and M_C could be obtained from the strain gauge readings in the tests.

Fig. 4.13: Force analysis of one end of the double-span girder/beam

Based on the obtained axial forces (Eq. (4.1)) and shear forces (Eq. (4.2)) in the double-span girder and the double-span beam, the respective contributions of flexural action and CA in the girder and the beam can be quantified and are shown in Fig. 4.14. Fig. 4.14 (a) shows that in the double-span girder, flexural action was the major contribution until the central deflection reached 440 mm ($1.66h_g$), after which CA took the lead. The contribution of flexural action increased very fast to the peak point when the central deflection was about 130 mm ($0.49h_g$), and then it monotonically

decreased till the end. On the other hand, CA was mobilised at a displacement of 240 mm, approximately the depth of the composite girder ($h_g=265$ mm). At the last moment, the contribution of flexural action almost reduced to 0 and the contribution of CA also dropped due to complete failure of the girder-to-peripheral column (C4 and C5) connections in the double-span girder.

Fig. 4.14 (b) shows that in the double-span beam, the contribution of flexural action increased until the central deflection reached about 160 mm ($0.74h_b$). After that, it fluctuated with a certain decrease till the displacement reached about 400 mm ($1.86h_b$), beyond which it gradually rebounded to the ultimate value that was almost twice the first peak value. The rebound of the flexural contribution was achieved by two factors. Firstly, the web-cleat connections have very good rotation capacity. Secondly, the tensile forces and bending moments sustained by web-cleat connections can have significant increase after large rotation (Yang and Tan 2013a, Jiang et al. 2016). CA was mobilised at a displacement of 210 mm, which was about the depth of the composite beam ($h_b=215$ mm). CA also increased rapidly between the displacements of 400 mm and 540 mm, which was consistent with the rebounding stage of flexural contribution. It can be concluded that the sharp increase in contribution from the double-span beam allowed for a further increase in total load at large deformation stage (Stage 3), as shown in Fig. 4.1.

Moreover, comparing Fig. 4.14 (a) with Fig. 4.14 (b), it is easy to find that CAA in the double-span beam was not as significant as that in the double-span girder. This was because the double-span girder had higher axial stiffness and bending rigidity and was subjected to much stiffer horizontal restraint from the reaction wall.

Fig. 4.14: Separation of contributions in the double-span composite girder/beam

It is assumed that the contribution of CA in the floor system comprises the total contribution of CA in the double-span composite girder and beam. The contribution of flexural action is assumed in the same way. Besides the contributions of flexural action and CA in the double-span girder and beam, the remaining part of the vertical load was sustained by the composite slabs. The three load-resisting mechanisms can be separately ascertained, as shown in Fig. 4.15. There are three points to be noted. Firstly, the contribution of the composite slabs increased quickly at the beginning due to flexural action in the slabs. It kept increasing steadily until the central deflection attained about 160 mm ($0.60h_g$), when the contribution of flexural action in the double-span composite girder and beam significantly dropped, and the contribution of the composite slabs clearly increased. This was due to failure of the bottom bolts (at a displacement of 166 mm ($0.63h_g$)), causing part of the loads to be shifted from the girder to the composite slabs. Secondly, CA was mobilised at a displacement of 230 mm, which was in between the depths of composite girder (h_g) and composite beam (h_b). Lastly, at the peak load point, the total load, the loads carried by flexure in B1-B2 and B4-B7, the loads carried by CA in B1-B2 and B4-B7, and composite slabs were 1243 kN, 452 kN, 331.8 kN and 459.2 kN, respectively. That is to say at the ultimate state, the contributions of flexural action, CA and composite slabs accounted for 36.4%, 26.7% and 36.9%, respectively. The slab contribution would even be greater if the slabs were to be subjected to uniform loads instead of 12-point loads. In this study, the loading points were located above the steel beams, causing the double-span girder and the double-span beam to directly transfer loads to the supporting columns. Besides, the double-span composite girder and double-span

composite beam contributed significantly to flexure due to composite action of the slab with the girders and beams.

All these actions, including flexural action and CA in the double-span composite girder and beam, and TMA in the composite slabs, were transferred to the peripheral columns. The load redistribution among these columns will be discussed in the following section.

Fig. 4.15: Contributions of the composite slabs and the double-span composite girder/beam in the overall floor system

4.3.2 Load redistribution among columns

4.3.2.1 Vertical load redistribution

To analyse the vertical load redistribution among the surrounding columns after the internal column removal, the surrounding columns can be classified into three groups, viz. corner columns (C1, C3, C6 and C8, as shown in Fig. 3.7), supporting columns of double-span girder B1-B2 (C4 and C5 in Fig. 3.7) and supporting columns of double-span beam B4-B7 (C2 and C7 in Fig. 3.7). Fig. 4.16 (a) shows the trend of the vertical load redistribution among these three groups of columns with increasing slab central deflection. Initially, the loads sustained by C4 and C5 kept increasing until the displacement of 160 mm ($0.60h_g$), which corresponded to the first bolt failure in the double-span girder. It can also be found that C4 and C5 sustained greater loads than the other two groups of columns before the displacement of 245 mm ($0.92h_g$). Thereafter, the loads carried by C4 and C5 continued to decrease, while the loads carried by C2 and C7 continued to increase. The decreasing part of load was shifted

onto C2 and C7, so that the total load could still increase until the ultimate state (537 mm or $2.03 h_g$).

In addition, the increase of vertical reaction forces in C2, C7, C1, C3, C6 and C8 from the displacement of 300 mm ($1.13h_g$) to 537 mm ($2.03h_g$) corresponded to the increasing trend of the “load-deflection” curve in Fig. 4.1. The increase in vertical reaction forces in C2, C7, C1, C3, C6 and C8 were due to the loads transferred from the double-span beam and composite slabs. Thus, it can be concluded that the double-span beam and the composite slabs contributed to the load increase at large deformation stage (Stage 3 in Fig. 4.1).

4.3.2.2 Moment redistribution

In Fig. 4.1 (b), $M_{4,5\text{-ma}}$ means the average major-axis bending moment at the bottom sections (top level of column feet) of C4 and C5, and $M_{2,7\text{-mi}}$ represents the average minor-axis bending moment at the bottom sections of C2 and C7. Similar meanings are extended to $M_{1,3,6,8\text{-ma}}$ and $M_{1,3,6,8\text{-mi}}$. Positive bending moment indicates external fibre of a column was in tension, and negative bending moment indicates external fibre of a column was in compression. Fig. 4.1 (b) shows that after an internal column removal, the largest bending moment occurred at the bottom of supporting columns (C4 and C5) of double-span girder B1-B2. The terms $M_{4,5\text{-ma}}$, $M_{2,7\text{-mi}}$ and $M_{1,3,6,8\text{-mi}}$ were initially negative but changed into positive at a displacement of 220 mm ($0.83h_g$), indicating that these columns were initially pushed outwards at Stage 1 but subsequently pulled inwards at Stage 2. The change in the sign of moments also showed the transition from CAA to CA in the double-span girder, double-span beam and peripheral beams (B13, B14, B15 and B16). It is also found that $M_{1,3,6,8\text{-ma}}$ was always negative, indicating that there was no CA in peripheral girders B9, B10, B11 and B12. The linearly increased $M_{1,3,6,8\text{-ma}}$ was caused by the applied loads. Lastly, the sharp increase of $M_{2,7\text{-w}}$ and $M_{1,3,6,8\text{-mi}}$ between the displacements of 300 mm ($1.13h_g$) and 537 mm ($2.03h_g$) corresponded to the rapid increase in CA in the double-span beam and peripheral beams, respectively.

Fig. 4.16: Load redistribution among columns

4.4 Summary and Conclusions

In this chapter, a 3D composite floor system was quasi-statically tested to failure under an internal column removal scenario. The load-deflection response curve was obtained and the behaviour could be categorised into three stages. The objective of this chapter is to investigate the load-resisting mechanisms of a 3D composite floor system under internal column loss scenario. The contributions of respective mechanisms in the floor system were separately quantified, and the load redistribution among the eight columns was measured. Conclusions can be drawn as follows:

- (1) At small deformation stage, the vertical loads were mainly transferred by flexure and CAA in the double-span girder and flexure in composite slabs to the surrounding intact frame. Most of the loads shift from the removed column were redistributed to adjacent columns with moment connections through the double-span girder. At large deformation stage, the double-span beam and the composite slabs with inner beams were the major contributors to the vertical load increase. Hence, the redistributed vertical loads were transferred from the columns connected with the double-span girder to remaining surrounding columns.

- (2) CAA in the double-span beam was not as significant as that in the double-span girder. Catenary actions in the double-span composite girder and double-span composite beam were mobilised at about one girder or one beam depth.
- (3) TMA was present in the composite slabs and the tensile zone propagated outwards towards the peripheral at very early stage (1.1 times slab thickness).
- (4) The contributions of composite slabs with inner beams at ultimate state accounted for at least $1/3$ of the total vertical load.
- (5) Failure modes of the 3D composite floor system developed in a ductile way if the beam-to-column connections had enough rotation capacity.

CHAPTER 5 EXPERIMENTAL ANALYSES OF PARAMETRIC INFLUENCE ON BEHAVIOUR OF 3D COMPOSITE FLOOR SYSTEMS UNDER A COLUMN REMOVAL SCENARIO

5.1 Overview

As reviewed in Section 2.5.1, span length and boundary condition may significantly influence robustness of 3D composite floor systems (Main 2014). Loss of flexural composite action between steel beams and slabs would lead to transformation of load-resisting mechanisms in slabs (Li and El-Tawil 2014). Therefore, the author was keen to investigate the effects of these three parameters in actual tests. Accordingly, the four specimens (2×3-S-IC, 2×2-S-IC, 2×3-W-IC and 2×3-S-PI) in Chapter 3 varied in terms of the slab aspect ratio (span length), degree of composite action and boundary condition. The objective of this chapter is to identify the effects of these three parameters through comparing experimental results of the four specimens. The comparisons revealed the effects on load-carrying capacity, ductility, failure mode, deformation characteristics, load redistribution among the remaining columns and reaction forces from surrounding structures.

5.2 Load-deflection Response

To intuitively understand the load-carrying capacities of the four specimens, the applied total loads were converted to UDL. The UDL versus central deflection ($V-3$ in Fig. 3.10) curves of the four specimens are shown in Fig. 5.1. Similar to dealing with 2×3-W-IC (Fig. 4.1), each curve could be classified into three stages by two short vertical lines (Fig. 5.1), corresponding to failure of the first and the last bolts in the girder-to-removed-column connection (Fig. 4.2). The structures suffered varying degree of damage and mobilised different load-transfer paths in each stage. The first stage was dominated by flexure, in which double-span girder B1-B2 and composite slabs bridged over the removed column and transferred the vertical loads to surrounding intact structure. In the second stage, the components in the girder-to-removed-column connection failed successively, leading to gradual transfer of loads

from B1-B2 to B4-B7 and composite slabs. In the third stage, the applied loads were carried by B4-B7 and composite slabs. To facilitate discussion, the respective load-carrying and deformation capacities of a specimen were defined as the load and deflection at the peak points of the load-deflection curve.

*F and L denote the respective first and last bolts failure at the girder-to-removed-column connection, e.g. the first and last bolts at the girder-to-removed-column connection in 2×3-S-IC failed at a displacement of 140 mm and 353 mm, respectively.

* h_g denotes the overall composite slab thickness.

Fig. 5.1: Load-deflection responses of the four specimens

Table 5.1: The maximum loads and corresponding central deflections for the four specimens

Specimen ID	Load-carrying capacity	Deformation capacity (mm)
2×3-S-IC (Control)	51.2	470
2×2-S-IC	96.3	253
2×3-W-IC	51.8	537
2×3-S-PI	53.1	470

Table 5.1 shows the load-carrying and deformation capacities for all the specimens. Comparing the load-deflection response (Fig. 5.1) and the results at the ultimate state

(Table 5.1) of the control specimen (2×3-S-IC) with those of the other specimens, the effects of aspect ratio, degree of composite action and boundary condition on the capacities of specimens could be obtained.

Firstly, the load-carrying capacity of the specimen with square slabs (2×2-S-IC) was twice that of the control specimen (2×3-S-IC), while deformation capacity of the former was only 54% of the latter. The significant difference in UDL capacity can be explained from the following aspects. Compared to 2×3-S-IC, the higher flexural stiffness of secondary beams and slabs in 2×2-S-IC due to a smaller span length enhanced load-carrying capacity at small deformation stage. However, this enhancement from flexure gradually reduced when the floor system went into large deformation stage, instead, more prominent geometry nonlinearity in 2×2-S-IC took over and contributed to difference in load relative to 2×3-S-IC. More straightforward, for a given central deflection, the slabs of 2×2-S-IC experienced a greater rotation due to a smaller span length compared to 2×3-S-IC, promoting the development of TMA in slabs. Another non-negligible fact was that the multi-point loading system which was designed for 2×3-S-IC cannot simulate UDL for 2×2-S-IC. The loading points 1, 2, 3, 10, 11 and 12 were too close to the perimeter girders B9, B10, B11 and B12 in the test of 2×2-S-IC (Fig. 3.7), resulting in a greater load-carrying capacity. Compared to the other specimens, failure of the first bolt in girder-to-removed-column connection in 2×2-S-I was postponed (at $0.73h_g$ in Fig. 5.1) due to the equal span of slabs in both directions.

Secondly, the specimen with weak composite action (2×3-W-IC) could carry lower loads in the first and second stages, but greater loads in the last stage than the control specimen (2×3-S-IC). As a consequence, the load-carrying capacity of 2×3-W-IC (51.8 kN/m^2) was even slightly greater than that of 2×3-S-IC (51.2 kN/m^2), while the deformation capacity of 2×3-W-IC was 14% higher than that of 2×3-S-IC (see Table 5.1). Therefore, composite action between slabs and steel beams could influence the development of flexure, TMA and deformation capacities of 3D composite floor systems, but only had marginal influence on the load-carrying capacity.

Lastly, 2×3-S-PI was expected to have smaller load-carrying capacity than 2×3-S-IC. However, 2×3-S-PI could sustain greater loads than 2×3-S-IC at the early stage, since

2×3-S-PI had a thicker slab (70 mm) due to constructional error. Nevertheless, 2×3-S-PI had a smaller increase in load after the stage dominated by flexure compared with 2×3-S-IC (Fig. 5.1), due to the lack of horizontal restraints in the former. As a result, the load-carrying capacity of Specimen 2×3-S-PI was only 3.7% greater than that of 2×3-S-IC.

5.3 Failure Modes

Similar to the failure modes of 2×3-W-IC discussed in Chapter 4, the general damage sequence of structural elements in the four specimens can be summarised as follows:

In the first stage, concrete slabs cracked along the edges of the double-span girder, the double-span beam, and perimeter girders and beams with a crack width of 1~3 mm on the slab surface. In the second stage, the cracks opened more widely around the perimeters. The girder-to-removed column connections failed and the secondary beams started yielding under loading points 1, 2, 3, 10, 11 and 12 (Fig. 3.7). Concrete was crushed around the removed column locations and along the paths connecting the corner columns to the nearest loading points (1, 3, 10 and 12). The concrete spalled off around columns C4 and C5. In the third stage, some bolts connecting to columns C4 and C5 fractured. The connection angles to support secondary beams fractured or became highly deformed, and plastic hinges were formed or shear failure occurred in the secondary beams. The crushed and spalled regions expanded. At the last phrase of the third stage, reinforcing mesh and steel decking fractured at the central region (refer to Fig. 4.3).

Most of the failure modes in the four specimens were similar, except for some differences caused by a particular design in each one, as discussed below.

The typical final failure mode of composite slabs of the control specimen (2×3-S-IC) was that cracks of concrete opened widely (10 mm~30 mm crack width) along the perimeters, and concrete was crushed severely along double-span girder B1-B2, double-span beam B4-B7 and the paths connecting the corner columns to the nearest loading points (1, 3, 10 and 12), as shown in Fig. 3.7 and Fig. 5.2 (a). In other words, negative yield lines (solid bold lines) were formed along the perimeters and positive

yield regions (circled by dash line) were formed along double-span-girder B1-B2 connecting to C4 and C5, double-span-beam B4-B7 connecting to C2 and C7, and paths connecting the four corner columns to the nearest loading points (Fig. 3.7 and Fig. 5.2 (a)). Besides, cracks at the central region were caused by TMA.

As specimen 2×2-S-IC shared the same loading system used for 2×3-S-IC, loading points 1, 2, 3, 10, 11, and 12 were very close to the peripheral girders (B9, B10, B11 and B12) in 2×2-S-IC, as shown in Fig. 5.2 (b). Hence, in 2×2-S-IC with a lower shear span-to-depth ratio of composite slabs, severe shear failure occurred along peripheral girders B9, B10, B11 and B12 instead of positive yield lines at the corners, as shown in Fig. 5.2 (b). Besides, shear failures of the web of secondary beams (B3, B4, B5, B6, B7 and B8) (Fig. 3.7) in 2×2-S-IC were also observed, as shown in Fig. 5.3 (a). Similarly, the shear span-to-depth ratio of secondary beams was 1.67, while the width-to-thickness ratio of web was 27.2. Consequently, shear failures in the webs of secondary beams (B3, B4, B5 and B6 in Fig. 3.7) were also observed and accompanied by kinks (Fig. 5.3 (a)) which were caused by concentrated loads. In contrast, yield of secondary beams in the other three specimens was caused by bending moment, similar to that of specimen 2×3-S-IC in Fig. 5.3 (b). In these three tests, stiffeners were welded to successfully avoid occurrence of kinks. Lastly, as shown in Fig. 5.2 (b), cracks appeared throughout the entire floor rather than just at the central region as in 2×3-S-IC, indicating TMA was fully developed in square slabs.

For Specimen 2×3-W-IC, three differences should be noted. Firstly, most of the shear studs failed so that the composite slabs were separated from the steel beams at the final stage, as shown in Fig. 5.4 (a). However, the shear studs in the other three specimens had not fractured except at the peripheral ends of secondary beams due to high shear forces between secondary beams and slabs, as shown in Fig. 5.4 (b) and (c). Secondly, for 2×3-W-IC, small cracks developed along the re-entrants of the decking throughout the entire floor (Fig. 5.2 (c)), which was not found in the other three specimens. As discussed in Section 4.2.2, this could be due to separation of composite slabs from beams and girders causing the composite slabs to function as a pure slab. Under this circumstance, such kind of cracks appeared since the concrete

thickness above the re-entrant profile decking was small (25 mm). Thirdly, two positive yield lines extended to the floor edges rather than towards the corner columns (C1 and C6), as shown in Fig. 5.2 (c). The reason was that the corresponding longitudinal seam between two sheets of steel decking opened due to inadequate number of shear studs, as shown in Fig. 5.5. The weakened composite slabs induced unusual positive yield lines (Fig. 5.2 (c)).

For 2×3-S-PI, the crack pattern and failure mode were similar to those of 2×3-S-IC, as shown in Fig. 5.2 (d). Nevertheless, column C4 yielded at an early stage due to the absence of horizontal restraints. As shown in Fig. 5.6, the outer flange of the bottom section of C4 yielded under compression caused by CAA at a displacement of 35 mm ($0.13h_g$), while the inner flange yielded later at a displacement of 320 mm ($1.18h_g$) caused by the combination of vertical loads and CA. Afterwards, the compressive strain in the inner flange of C4 increased to 0.03 eventually.

(a) Control Specimen 2×3-S-IC

(b) Specimen 2×2-S-IC

(c) Specimen 2×3-W-IC

(d) Specimen 2x3-S-PI

Fig. 5.2: Failure modes of composite slabs

(a) Specimen 2x2-S-IC

(b) Control Specimen 2x3-S-IC

Fig. 5.3: Failure of secondary beams

(a) Specimen 2x3-W-IC

(b) Specimen 2x3-S-PI

(c) Control Specimen 2x3-S-IC

Fig. 5.4: Failure of shear studs

Fig. 5.5: Opening of the seam in Specimen 2x3-W-IC

Fig. 5.6: Strain in the bottom section of C4 of Specimen 2x3-S-PI

5.4 Measurements of Displacements and Rotations

As shown in Fig. 3.10, the deflections of beams B4 (V-1 and V-2) and B5 (V-4 and V-5) were measured just under the loading points. The central deflections of double-span-girder B1-B2 (V-3a and V-3b), beam B2 (V-6), as well as peripheral beams B10 (V-7) and B15 (V-8) were also measured. According to these data, the deformation profiles of a quarter of the control specimen (2x3-S-IC) at different loading stages are plotted in Fig. 5.7. Similar to 2x3-W-IC (Fig. 4.10 and Fig. 5.4 (a)), the girders deformed linearly, while the secondary beams (B4 and B5) deformed in a multi-linear

way due to the multi-point loading, as shown in Fig. 5.7. This can be demonstrated through comparisons between the local angles and the corresponding segment rotation angles of the beams, as shown in Fig. 5.8. Specifically, $I-1$ and $I-2$ represent the local rotation angles at the ends of girder B1. The terms $I-7$ and $I-8$ represent the local rotation angles at the ends of beam B4 (Fig. 3.10). $\text{Arctan}(V3/1800)$ represents the rotation of girder B1, $\text{Arctan}(V1/750)$ and $\text{Arctan}((V3-V2)/750)$ represent the rotations of segments in beam B4. The good agreement shows that girders (B1 and B2) had a linear deflection profile while secondary beams (B4 and B7) deformed in a multi-linear way. Similar observation was obtained for the inner beams (B3, B5, B6 and B8).

The deformed shape of 2×3-W-IC was similar to that of the control specimen. However, the composite slabs of 2×3-W-IC separated from the girders and beams with increasing loads. This delamination weakened the composite action between the slabs and steel beams, causing more deformation of secondary beams in 2×3-W-IC compared to those of the control specimen (2×3-S-IC) at a given slab central deflection, as shown in Table 5.2. On top of that, the ultimate central deflection of 2×3-W-IC (Table 5.1) was the largest among all four specimens. For comparison purpose, the measured vertical displacements of all the four specimens at the central displacement of 100 mm (Stage 1), 250 mm (Stage 2) and 500 mm (Stage 3) are listed in Table 5.2.

As expected, the deformation 2×2-S-IC was the smallest compared to the other three specimens at each loading stage (Table 5.2) due to smaller span in secondary beam direction. Besides, the deformed shape of 2×2-S-IC was also different. Specifically, Specimen 2×2-S-IC deformed approximately linearly in both directions, as shown in Fig. 5.9 and Fig. 5.10. It should be mentioned that only two deformed profiles of 2×2-S-IC in Stages 1 and 2 are depicted in Fig. 5.9, since deflections $V-1$ and $V-5$ were not successfully measured at the final stage. Fig. 5.11 (a) shows girder B1 deformed linearly. Fig. 5.11 (b) shows that inclinations measured by $I-7$ and $I-8$ were approximately equal to each other and they agreed well with segment rotation of beam B4, demonstrating linear deformation of secondary beam B4.

Specimen 2×3-S-PI deformed similarly to 2×3-S-IC, so its deformed profiles are not

plotted here. Instead, the specific deflection values at particular loading stages are listed in Table 5.2, which show that the deflections of beams and girders in 2×3-S-PI were smaller than the counterparts in 2×3-S-IC for a given deflection. This was not due to the more flexible boundary conditions. In reality, the actual thicker slab of 2×3-S-PI could lead to this fact. As shown in Fig. 5.12 (a), the top of column C4 in Specimen 2×3-S-PI experienced much greater lateral deformations compared with that in control specimen 2×3-S-IC. The lateral deformation was only concentrated at the free end of the double-span girder at C4 location (Fig. 5.13 (a) and (b)), causing detrimental bending moment at the bottom of the peripheral column C4 (Fig. 5.12 (b)). It should be mentioned that the positive bending moment indicated the outside fibre of a column was in tension, and negative bending moment indicated the outside fibre of a column was in compression. The bending moment in column C4 attained the plastic bending resistance of the column section (around 160 kN·m) at the last stage, as shown in Fig. 5.12 (b).

(a) At a displacement of 100 mm ($0.38h_g$, Stage 1)

(b) At a displacement of 250 mm ($0.94h_g$, Stage 2)

(c) At a displacement of 500 mm ($1.89h_g$, Stage 3)

Fig. 5.7: Deformations of Control Specimen 2×3-S-IC at different loading stages

Fig. 5.8: Comparisons between the segment rotations and the corresponding local rotations for Control Specimen 2×3-S-IC

(a) At a displacement of 110 mm ($0.42h_g$, Stage 1)

(b) At a displacement of 250 mm ($0.94h_g$, Stage 2)

Fig. 5.9: Deformations of Specimen 2×2-S-IC at different loading stages

Fig. 5.10: Final deformation of Specimen 2×2-S-IC

Fig. 5.11: Comparisons between the member rotations and the local rotations for Specimen 2×2-S-IC

(a) Lateral displacement at the top of column C4

(b) Bending moment at the bottom section of column C4

Fig. 5.12: Comparison between Specimens 2×3-S-IC and 2×3-S-PI

(a) Deformation of C4

(b) Comparison of lateral displacements at free edge

Fig. 5.13: Lateral displacements at the free edge of Specimen 2×3-S-PI

Table 5.2: Comparison of vertical displacements of the four specimens

Specimen ID	V3 (Central displacement)	V1	V2	V4	V5	V6	V7	V8
(mm)								
2×3-S-IC	100	54.7	90.9	32.0	62.8	52.0	1.8	7.3
2×3-W-IC		74.1	102.9	53.2	75.2	51.5	1.2	16.3
2×3-S-PI		44.7	86.3	28.4	50.0	50.5	1.4	6.9
2×2-S-IC	109	11.7	98.8	8.3	59.9	58.1	1.0	2.0
2×3-S-IC	250	136.2	222.1	68.7	145.8	129.9	2.9	12.4
2×3-W-IC		171.1	223	121.9	157.9	128	1.7	34.8
2×3-S-PI		104.3	213.6	56.1	102.3	121.5	2.2	10.7
2×2-S-IC		51	217.5	13.6	123	125.6	2.4	4.3
2×3-S-IC	500	283.4	432.9	142.8	306.3	338.1	4.6	28.4
2×3-W-IC		347.9	455.5	194.6	360.2	354.9	2.5	93.5
2×3-S-PI		229.7	425.7	121.1	217.4	243.8	3.4	14.3
2×2-S-IC		N.A.*	483.1	33.9	N.A.	240.6	3.5	8.4

* N.A. indicate the deflection had not been effectively measured, since the deflection exceeded the measuring range of LVDT and the LVDT was not adjusted in time.

5.5 Reaction Forces

5.5.1 Horizontal reaction forces in restraints

As demonstrated in Chapter 4, under an internal column removal scenario, the double-span beam and the double-span girder played very important roles in transferring the vertical loads to the surrounding structure. Hence, the reaction forces acting on the ends of the double-span girder and the double-span beam were obtained to investigate the structural mechanisms. With this notion, horizontal reaction forces acting on the ends of double-span girder B1-B2 (R1 and R9 in Fig. 3.7) and double-span beam B4-B7 (R5 and R11 in Fig. 3.7) are shown in Fig. 5.14. As expected, the horizontal reaction forces were in compression initially and then changed into tension, showing the transformation from CAA to CA. Arising from external restraint, the horizontal compression force acting on girder B1-B2 was greater than that of beam B4-B7 (Fig. 5.14), indicating CAA in the girder developed more

significantly than that in secondary B4-B7 due to two reasons. Firstly, double-span-girder B1-B2 had much higher axial stiffness than double-span-secondary-beam B4-B7, viz. 639.4 versus 122.2 kN/mm, respectively. Secondly, horizontal stiffness at the ends of girder B1-B2 were higher, viz. 45 kN/mm for R1 and 230 kN/mm for R9, compared to 30 kN/mm (R5) and 40 kN/mm (R11) at the ends of B4-B7.

However, higher horizontal stiffness of restraint does not necessarily impart additional strength at CAA stage when the demand of horizontal stiffness can be provided by columns. For instance, in the tests, columns themselves could provide adequate flexural rigidity (15.5 kN/mm) to resist outward lateral deformation at the column top or bending moment at the column base caused by CAA. This can be observed through comparing 2×3-S-PI with 2×3-S-IC in terms of lateral displacement at the top of column C4 (Fig. 3.7 and Fig. 5.12 (a)) and bending moment at the bottom section of C4 (Fig. 5.12 (b)). The maximum lateral negative displacements of the two curves in Fig. 5.12 (a) are close to each other (around 2 mm) during the CAA stage, while the negative bending moments of the two curves in Fig. 5.12 (b) (around 50 kN·m) are similar, showing that column C4 itself had enough flexural rigidity to develop CAA. It should be noted that, whether columns themselves can provide sufficient horizontal stiffness to develop CAA depends on the relative strength between the columns and the connected double-span beam. If the flexural rigidity of steel columns is not sufficient to support the development of CAA, in interior bays, adjacent floors will assist to resist outward movements so nothing detrimental will happen to the column adjacent to the removed column. However, if a penultimate column was removed, the peripheral column adjacent to the removed column could yield at CAA stage by outward pushing.

In 2×3-S-PI (Fig. 5.12 (b)), once column C4 returned to its original position, it was subjected to increasing positive bending moment at the column base, attaining 160.9 kN·m (Fig. 5.12 (b)); at this point the entire column section at the base had yielded. Hence, horizontal stiffness (R9 and R1 in Fig. 3.7) only governed the behaviour of the floor systems at CA stage.

At CA stage, Fig. 5.14 (a) shows that the maximum tensile reaction force acting on B1-B2 of 2×2-S-IC (283 kN) was smaller than that of Control Specimen 2×3-S-IC

(429 kN). Fig. 5.14 (b) shows that 2×2-S-IC developed the smallest tensile reaction force at both ends of beam B4-B7 during the whole CA stage compared with the other three specimens. However, Specimen 2×2-S-IC sustained the greatest load-carrying capacity (96.3 kN/m² in Table 5.1) among the four specimens. Hence, smaller tensile horizontal reaction forces from external restraints do not necessarily imply smaller CA or load-carrying capacity; it only indicates fewer demands of tie force required from surrounding structures. With this notion, it is reasonably found that 2×3-S-PI developed greatest tensile reaction force (308 kN) at the ends of B4-B7 (R5 and R11 in Fig. 5.14 (b)), since there was no R1 restraint (Fig. 3.8).

(a) At the ends of double-span-girder B1-B2

(b) At the ends of double-span-beam B4-B7

Fig. 5.14: Horizontal reaction forces acting on the ends of double-span girder and beam

5.5.2 Reaction forces in columns

To investigate the load redistribution, the columns were classified into three groups as in Section 4.3.2. The first group consisted of columns C4 and C5 connecting to the removed-column stub with girders B1 and B2 (Fig. 3.7). The second group included columns C2 and C7 connecting to the removed-column stub with secondary beams B4 and B7. The third group comprised corner columns C1, C3, C6 and C8 of the affected bays. If the central column was not removed, the respective three groups and the central column would sustain 25% of the total vertical load assuming that the load was uniformly distributed. After removal of the central column, Fig. 5.15 shows percentages of axial forces in the three groups of columns accounting for the total axial forces in eight columns. For comparison purpose, Fig. 5.16 shows the load redistribution among columns for internal column (IC) removal scenario in the test by Johnson et al. (2016).

(a) 2×2-S-IC

(b) 2×3-S-IC

(c) 2×3-W-IC

(d) 2×3-S-PI

Fig. 5.15: Axial force percentages in different groups of columns

Fig. 5.16: Vertical load redistribution for IC removal (Johnson et al. 2016)

It can be seen that the load redistribution pattern of 2×2-S-IC (Fig. 5.15 (a)) was similar to that observed in the previous test (Fig. 5.16), since both specimens had square floor slabs. Johnson et al. (2016) found that the girder end columns (corresponding to C4 and C5 in this study), beam end columns (corresponding to C2 and C7 in this study) and corner columns (corresponding to C1, C3, C6 and C8) sustained the respective 37%, 38% and 25% of the total load prior to failure of joint components. In comparison, groups C4+C5, C2+C7 and C1+C3+C6+C8 carried 35%, 40% and 25% of the total load before the first component failure (at a central deflection of 193 mm) (Fig. 5.1). In Johnson et al. (2016)'s test, the load percentage of girder end columns slightly increased, while that of beam end columns slightly decreased due to the failure of beam-to-removed column connections. It was similarly observed that in the test of 2×2-S-IC, group C2+C7 carried slightly increasingly percentages of total load while group C4+C5 showed the opposite trend (Fig. 5.15 (a)) due to successive failures of joint components in the double-span girder B1-B2.

However, the load redistribution pattern in specimens with slab panels of 2m×3m was quite different. At an early stage, group C4+C5 sustained a significantly greater percentage of the total load than group C2+C7 (Fig. 5.15 (b), (c) and (d)). However, the load carried by C4 and C5 significantly decreased with increasing deflection. Specifically, at the initial state, C4 and C5 sustained around 45%, C2 and C7 around 30%, and C1, C3, C6 and C8 around 25% of the total vertical load. With applying loads, the load percentage of group C4+C5 increased while those of the remaining columns decreased within a temporary period, when the central deflection did not

exceed 100 mm. This period corresponded to the phrase of the most rapid increase in load, as shown in Fig. 5.1. Subsequently, it is observed that the load percentage of group C4+C5 began to significantly decrease due to concrete spalling around C4 and C5, while the remaining columns sustained increasingly more loads. That is to say, after failure of slabs around C4 and C5, vertical loads were transferred from group C4+C5 to groups C2+C7 and C1+C3+C6+C8, due to the weakening of flexural action in double-span girder B1-B2. Such transfer continued to go on due to successive failures of joint components in the double-span girder B1-B2. The load carried by columns C2 and C7 surpassed that of C4 and C5 at a central deflection of around 350 mm. In comparison, as shown in Fig. 5.15 (a), the load transfer from group C4+C5 to groups C2+C7 was not evident in 2×2-S-IC, since they sustained similar quantities of loads from the beginning due to equal span in both directions. Therefore, vertical load redistribution is significantly affected by slab aspect ratio and connection ductility.

Fig. 5.17: Comparisons among the four specimens in terms of vertical load

redistribution

Fig. 5.17 shows comparisons among the four specimens in terms of contribution percentages of different groups of columns. It can be evidently seen that smaller span length in beam direction weakened the load contribution of C4 and C5, but increased those of the remaining columns. Compared with the control specimen, weak composite action hardly affected vertical load redistribution, while removal of restraints at one edge only slightly influenced the vertical load redistribution until the central deflection arrived around 400 mm (Fig. 5.17 (b)). After that, the contributions of group C2+C7 in both 2×3-W-IC and 2×3-S-PI became more significant compared with the control specimen, which may stem from more dominant CA in the double-span beams.

5.6 Comparisons with DoD (2013) and Previous Tests

As the four tests were conducted quasi-statically, the nonlinear static load-deflection curves are converted to the corresponding pseudo-static responses (Fig. 5.18) to assess robustness of floor systems subjected to sudden column loss associated with inertial and nonlinear effects, using the simplified framework proposed by Izzuddin et al. (2008) as introduced in Section 2.2.2. The pseudo-static responses are compared with the extreme event load combination, $1.5 \times \text{Dead Load} + 0.5 \times \text{Live Load}$, according to DoD (2013), showing that the test specimens with enhanced members, joints and slabs could sustain much greater loads than that originally demanded (6.3 kN/m^2 in this study).

Fig. 5.18: Pseudo-static responses of test specimens

Specifically, the load-carrying capacities of specimens with slab panel dimensions of 2m×3m and 2m×2m were around 6.8 and 12.7 times the required load (6.3 kN/m²), respectively. However, previous tests conducted by Johnson et al. (2016) showed that their specimen could only carry around 4.0 kN/m² static load which was smaller than the extreme event load combination. The floor system in Jahromi et al. (2012) was successfully applied with a load over 1.5 times the load combination prescribed by DoD (2013) without damage. Unfortunately, they failed to obtain the marginal capacity beyond this load level due to the loading issue (Hull 2013). It is known that ultimate load-carrying capacity of a floor system is mostly affected by span length, joint and slab strength in tension. Hence, from these aspects, the test specimens in Johnson et al. (2016) and Jahromi et al. (2012) are quantitatively compared with the specimens in this study to reveal the differences among them, as shown in Table 5.3. It should be noted that tensile strength of slabs is the summation of tensile yield strength of reinforcing mesh and steel decking only accounting for the horizontal legs.

Table 5.3 shows that the floor systems in Jahromi et al. (2012) and Johnson et al. (2016) had similar span length, joint and slab strength, whereas the former carried more than twice the load of the latter. The reason was the different joint configurations. In Jahromi et al. (2012), the joints were designed with two 12.7 mm-diameter-bolt rows. In comparison, the joints in Johnson et al. (2016) were devised with three 9.5 mm-diameter-bolt rows. Consequently, each bolt-row strength in the former, e.g. 59 kN for beam-to-column connection (Table 5.3), was much greater than the counterpart of the latter (26.5 kN). In reality, the bolt-row components in a joint do not equally sustain a load. Failure of joint components always starts from the bottom row of girder (beam)-to-removed column location, followed by successive failure of the bottom most row upwards. As a result, in Johnson et al. (2016), the joint components at the removed column locations were destroyed one after another by a fairly low load incremental. Another important reason of significant difference in load-carrying capacity between Jahromi et al. (2012) and Johnson et al. (2016) was that the horizontal stiffness of restraints of the former floor system was much greater than that of the latter. Besides, the difference of slab detailing may contribute to the difference in load-carrying capacity.

In comparison, the enhanced joint and slab strength in this study were around two times as great as those in previous studies, while the span lengths of the former were smaller than half of the counterparts. Besides, joints in this study failed in more ductile manners relative to previous studies (Table 5.3). Those were the reasons that the specimens in this paper were much more robust compared with the previous tests. Nevertheless, the steel beams, joints and slabs were proportionally enhanced (Table 5.3) so that load redistribution (Section 5.5.2) and failure criteria were similar between the present study and Johnson et al. (2016). Specifically, for interior column removal in Johnson et al. (2016), collapse was precipitated by failure of ties in the double-span girder and the double-span beam. In this study, separation of girder- and beam-to-removed column connections led to the descending branch of the load-deflection curve. The failure modes in composite slabs of the two tests were also similar, viz. cracks formed along girders and beams, and at 45° angles to the perimeters in Johnson et al. (2016), while slabs failed along the perimeter beams and girders, double-span-girder and -beam and the paths connecting the corner columns to the nearest loading points. Hence, the structural behaviour, except for load-carrying capacity, obtained from the small-scale enhanced models in this thesis could be applied to full-scale structures.

Table 5.3: Comparisons of structural component strength between previous and present studies

	Static load-carrying capacity (kN/m ²)	Span length (m×m)	Joint strength in tension (kN)			Tensile strength of slabs per meter (kN/m)
			Girder-to-column	Beam-to-column	Inner beam-to-girder	
Jahromi et al. (2012)	> 8.0	4.864×4.864	118.0 (angle tear up, 2 bolt rows)	118.0 (angle tear up, 2 bolt rows)	91.2 (bolt shear, 2 bolt rows)	213.7
Johnson et al. (2016)	3.2 (Initial damage);4.0	4.5×4.5	133 (web tear out*, 3 bolt rows)	79.5 (bolt shear, 3 bolt rows)	79.5 (bolt shear, 3 bolt rows)	240
This study	53.0; 96.0	2×3;2×2	320 (end plate fracture and bolt yield, 2 bolt rows)	234 (angle tear up, 2 bolt rows)	234 (angle tear up, 2 bolt rows)	396.4

*The predicted failure mode was girder web tear out, while the actual failure was governed by bolt shear.

5.7 Summary and Conclusions

This chapter presents in-depth analyses based on the test results of four 3D composite floor systems with differences in slab aspect ratio (span length), composite action between girders and beams and boundary condition. Firstly, the load-displacement curves, failure modes and deformed shapes of the floors were reported. Besides, the reaction forces in columns and restrained CHS were calculated based on strain readings to shed light on load redistribution among the remaining columns after the loss of an internal column. Furthermore, the effects of aspect ratio, composite action and boundary condition, were investigated through comparisons. Lastly, the present test results are compared with the previous similar tests. The similarities and differences among them are discussed, giving an in-depth appreciation on robustness of 3D composite floor systems subjected to column removal scenarios. The following conclusions can be drawn:

- (1) The load-carrying capacity of specimen 2×2-S-IC was much greater than those of the other three specimens due to a smaller span length in beam-direction and a smaller aspect ratio which increased flexural capacity at small deformation stage and promoted development of TMA at large deformation stage, while the deformation capacity of 2×2-S-IC was the smallest among the four specimens. The equal span of slabs in both directions delayed the failure of girder-to-removed-column connection. The columns at the ends of the double-span girder and the double-span beam sustained similar percentages of total load in the specimen with square slab before failure of joint components. Loads sustained by the double-span-girder end columns were slightly transferred to the other columns due to failure of joint components in the double-span girder.
- (2) Vertical load redistribution is significantly affected by slab aspect ratio and connection ductility. For composite floor systems with rectangular slabs, the load supposed to be sustained by the removed column was mostly redistributed to the adjacent columns in short-span direction before complete failure of the girder-to-removed column connection. Afterwards, most of the

load was sustained by the columns at the ends of double-span beam, followed by the columns at the corners of the affected bays.

- (3) Composite action between slabs and steel beams had marginal influence on ultimate load-carrying capacity. Although weaker composite action initially decreased flexural resistance at small deformation stage, it improved ductility significantly which is beneficial for the development of CA and TMA in the subsequent stages. Load redistribution pattern in 2×3-W-IC was similar to that in 2×3-S-IC until the final stage, when the columns at the ends of the double-span beam in 2×3-W-IC sustained a greater percentage of the total load compared with those in 2×3-S-IC. This was due to more significant CA developed in the double-span beam in 2×3-W-IC.
- (4) More flexible boundary conditions could not increase the applied load after the flexural stage. Along the free edge, lateral deformation was concentrated at the end of the double-span girder. The catenary action generated in the double-span girder caused yielding of the peripheral column base. This kind of column could easily buckle if they are also subjected to axial compression forces from upper storeys. Therefore, special attention should be paid to the stability of peripheral columns when designing for catenary action in double-span beams. For this specimen, the boundary condition (with Penultimate Internal column) only slightly influenced the vertical load redistribution among the remaining columns.
- (5) For composite floor systems, the flexural rigidity of steel columns was sufficient to support the development of CAA so that additional horizontal restraints from surrounding structural members were not so beneficial at CAA stage. However, the stiffness of horizontal restraints dominated the floor system at CA stage.
- (6) Strength and ductility of girder- and beam-to-column joints are crucial to robustness of 3D composite floor systems. Enhancing strength and ductility of each bolt-row component can increase ultimate load-carrying capacity and postpone collapse of the floor system, respectively.

- (7) Typical yield pattern of composite slabs consisted of negative yield lines along the perimeters and positive yield lines along the double-span girders and beams, and paths connecting corner columns to the nearest loading points.

Since the cost of conducting a system-level 3D sub-structural test is prohibitively high, only four tests were performed. However, the comprehensive test data could be used to validate FE models for further parametric studies in Chapter 6.

CHAPTER 6 NUMERICAL STUDIES ON ROBUSTNESS OF 3D COMPOSITE FLOOR SYSTEMS UNDER A COLUMN REMOVAL SCENARIO

6.1 Overview

This chapter carries out parametric studies on 3D composite floor systems subjected to column loss scenarios using macro-based FE models. The modelling method has been verified by the author using six pure slabs (Cashell et al. 2011) and four system-level tests (Chapter 3). The verified FE models are used to investigate the effects of governing parameters on load-resisting mechanisms, such as flexure and CA in the double-span girder and the double-span beam over the missing column, and composite slabs with inner beams.

The author has gained substantial knowledge of static behaviour of 3D composite floor systems under a column removal scenario through analysing the test results in Chapters 4 and 5 and conducting numerical simulations. Another attention is drawn to dynamic behaviour of a 3D sub-structure when exposed to free-fall events. With the reliable FE modelling method, DIF of 3D composite floor systems can be studied to fill the corresponding technical gap summarised in Chapter 2.

Hence, a full-scale model is constructed to study the dynamic behaviour of a 3D sub-structure subjected to sudden removal of an internal column. Firstly, the model is statically analysed under UDL to obtain the quasi-static load-displacement curve of the sub-structure. Secondly, the author simulates the sub-structure subjected to a range of UDLs under sudden removal of an internal column, to obtain dynamic displacement-time curves and the pseudo-static load-displacement curve. DIFs can be calculated through comparing the quasi-static and pseudo-static load-displacement curves. Furthermore, force-based DIF_p for the 3D composite floor systems is compared with those for corresponding 2D steel frames (Liu et al. 2013, Liu et al. 2016) and with that recommended by DoD (2013). The comparison results reveal different dynamic behaviour between 3D composite floor systems and 2D steel frames and show that DIF_p in DoD (2013) tends to be unsafe at large deformation stage.

6.2 Material Models

Since material models can significantly influence predictions of numerical models, material models should be carefully defined. This section describes the adopted material models when using ABAQUS (2011) package.

6.2.1 Steel material model

The steel reinforcement and profiled decking adopt bilinear stress-strain relationship to represent uniaxial material property (Fig. 6.1). The density and Poisson's ratio for both steel reinforcement and decking are assumed to be 7850 kg/m^3 and 0.3, respectively. As shown in Fig. 6.1, Young's modulus (E), yield stress (f_y), ultimate strength (f_u) and ultimate strain (ϵ_u) can be obtained from steel coupon tests, e.g. material properties for the present tests are included in Table 3.2. Reinforcement and steel decking are modelled by ductile damage criteria. Steel fracture is modelled by ductile fracture, which involves the process of nucleation, growth and coalescence of voids. Damage evolution assumes that the damage is characterised by progressive degradation of material stiffness, leading to material fracture.

It should be noted that the ultimate strain (ϵ_u) cannot govern the failure of steel in the numerical models. Instead, "fracture strain" is defined for damage initiation. It is the equivalent plastic strain at failure, of which the value also depends on the characteristic length of the element. Hence, "fracture strain" should be calibrated by experimental tests first and the calibrated "fracture strain" can only be used to similar meshed models with the similar material. Examples of calibrating "fracture strain" are given in Section 6.3.1.3.

Steel beams and columns in the numerical model are simulated by one-dimensional beam elements. No damage criteria are introduced into the beam elements because beams and columns are much stronger than slabs and connections and it is assumed that no fracture will occur in them. Connections are simplified using joint components simulated by connector elements in ABAQUS. The full force-displacement relationships of joint components are presented in Appendix.

Fig. 6.1: Bilinear stress-strain curves for steel material

6.2.2 Concrete material model

The density and Poisson's ratio of concrete are assumed to be 2450 kg/m^3 and 0.2, respectively. The FE models in this study incorporate concrete with damaged plasticity model, which assumes concrete with scalar (isotropic) damage. This model is suitable for concrete structures subject to arbitrary loading conditions (ABAQUS 2011). For both tension and compression loading conditions, the model takes account of degradation of elastic stiffness due to plastic straining. In ABAQUS (2011), the stress-plastic strain relationship can be automatically converted from the uniaxial stress-strain curve. Hence, the uniaxial stress-strain curve of concrete is required for numerical analysis. In this study, the uniaxial stress-strain curve is defined based on formulae given by GB50010 (2010). For instance, Fig. 6.2 shows the response of concrete used in the experimental tests (Chapter 3) under uniaxial loading.

In this study, RC slabs are simulated by shell elements with composite materials, so "rebar" is defined as one part of shell element section. The interactions between reinforcement and concrete, such as bond slip and dowel action, can be simulated because tension stiffening of concrete is specified by means of a post-failure stress-strain relation.

Fig. 6.2: The response of concrete under uniaxial loading

6.3 Verification of FE Modelling

6.3.1 Verification of slab modelling

6.3.1.1 Referenced experimental tests

In 2010s, Cashell et al. (2011) studied the behaviour of reinforced concrete slabs under extreme loading conditions, so as to identify the failure conditions of composite floors in a fire considering the early strength loss of steel deck. They focused on the responses of slabs with four parameters, which could be inferred by the names of the specimens. This study simulates six specimens of Cashell et al. (2011) covering all four parameters. They are R-F60-D6-A, R-F60-M6-A, R-F60-M6-A (2), S-F60-M6-A, S-P120-M6-A, S-P120-D8-D. The first letter R/S represents a rectangular or a square slab. F60 and P120 denote flat 60mm-thick and profiled 120 mm-thick slabs without steel decking. The third field (D6, D8 or M6) describes the type of rebar, and D6 and D8 indicate deformed bars with 6 mm and 8 mm of diameter, respectively, and M6 is welded mesh A142 which consists of 6 mm diameter bars at 200 mm spacing. Alphabets A, B, C and D represent different reinforcement arrangements. All the details of the six specimens can be found in Cashell et al. (2011).

6.3.1.2 FE modelling

Since the six slabs were symmetrically loaded and supported in the tests (Fig. 6.3 (a)),

only one-half of specimens are simulated. As shown in Fig. 6.3 (b), the slabs are vertically supported at perimeters and statically loaded through 6 points to failure. Besides constraint conditions and loading schemes, model details used in the simulations are consistent with the test specimens. It should be noted that the cross-sections of the profiled slabs are replaced by the respective “strong” and “weak” equivalent rectangular cross-sections with effective widths (Fig. 6.3 (c)) so that quadrilateral finite-membrane-strain shell elements (S4R) can be used to model slabs.

(a) The dimensions and loading positions of slabs

(b) Overview of a FE model

(c) Simplification for profiled cross-section of concrete slabs

Fig. 6.3: Information of the slab modelling

6.3.1.3 Comparisons between FE modelling and test results

As mentioned in Section 6.2.1, “fracture strain” of reinforcement should be calibrated first. Specimens S-F60-M6-A (Fig. 6.4 (a)) and R-F60-D6-A (Fig. 6.4 (b)) are modelled based on different “fracture strains”. The simulation results of Specimen S-F60-M6-A indicate the modelling result with “fracture strain” value of 0.017 matches the test data best for welded mesh reinforcement. Meanwhile, the simulation results of Specimen R-F60-D6-A show that a value of 0.020 fits best for deformed reinforcement bars. Similarly, for the specimens using rebar D8, fracture strain value of 0.030 is determined. Thus, the fracture strains of all types of reinforcement have been validated, and these values will be used for other specimens.

Fig. 6.4 shows that the FE models can capture major features of the experimental curves, especially at large deformation stage, such as the ultimate load-carrying capacity, ultimate deflection, and stiffness at large deformation stage. However, in the simulation curves of concrete slab (Fig. 6.4), a peak is observed at elastic stage, followed by a significant drop in load. This result is caused by the occurrence of concrete cracking at the bottom layer of the elements. Thus, the value of the peak load and the moment of drop are strongly influenced by the thickness of the slab and the uniaxial tensile stress-strain curve of concrete. In actual tests, uniformity of slab thickness is very difficult to control, resulting the inevitable difference in slab thickness between FE models and actual slabs. Besides, the adopted uniaxial tensile stress-strain curve in FE models are determined from the given tensile strength based on the average of three sample tests. Hence, it is also impossible to input the exact

concrete tensile behaviour as the test results. These two aspects of difference cause the discrepancy between FE predictions and test results at the end of the elastic stage. In spite of this discrepancy, the FE models can predict the loads and displacements at the ultimate limit state well. As shown in Table 6.1, the average ratios of FE-to-test results for the six slabs are close to unity with a standard deviation of 0.08.

(a) S-F60-M6-A

(b) R-F60-D6-A

(c) R-F60-M6-A

(d) R-F60-M6-A (2)

(e) S-P120-M6-A

(f) S-P120-D8-D

Fig. 6.4:. Comparison of the load-deflection response of six specimens between tests and FE results

Table 6.1: Comparison of the ultimate loads and displacements of six specimens between tests and FE results

Specimen ID	Load (kN)			Displacement (mm)		
	FE	Test	FE/Test	FE	Test	FE/Test
S-F60-M6-A	83.7	80.2	1.04	61.4	64	0.96
R-F60-D6-A	82.3	70.2	1.17	71.6	71.7	1
R-F60-M6-A	70.7	71.7	0.99	68.6	67.1	1.02
R-F60-M6-A (2)	78.5	72.8	1.08	74.3	72.7	1.02
S-P120-M6-A	88.9	89.0	1	49.4	51.9	0.95
S-P60-D8-D	168.7	174.7	0.97	62.7	58.7	1.07
Mean			1.04			1.00
Standard deviation			0.08			0.04

6.3.2 Verification of 3D composite floor system modelling

The capability of shell element S4R simulating RC slabs has been verified by the isolated six slab tests. Subsequently, the reliability of the 3D composite floor system

modelling with ABAQUS (2011) is ensured by comparing the simulation predictions and the test results (Chapter 5) in terms of overall load-displacement curve and failure mode.

6.3.2.1 FE modelling

The 3D composite floor systems are simulated by macro-models, with one-dimensional beam elements simulating beams and columns, shell elements for composite slabs and connector elements (similar to spring) for joint components, as shown in Fig. 6.5 (a). These spring components only represent the steel connection without involving composite slabs which are simulated by shell elements. The spring properties of these components are determined from the actual tests (Jiang et al. 2016) in the author's research group (refer to Appendix). It should be mentioned that steel decking delaminates from RC slabs at early loading stage, so that bond strength between them can be neglected. To consider this effect, two separate shells are incorporated for simulating the respective steel decking and RC slab, as shown in Fig. 6.5 (a). Additionally, beam elements (B31) are used to connect composite slabs to steel beams (Fig. 6.5 (a)) like shear studs in test specimens. The spacing, cross-section and material property of the "shear studs" in the model are adjusted to simulate the experimental shear resistance between the composite slabs and the steel beams from the four push-out tests (Jiang et al. 2016). Fig. 6.6 shows the comparisons of normalised load-slip curves of push-out tests (Section A.2.3) and the corresponding push-out simulations, indicating the adjusted "shear studs" in the FE model is valid to represent the actual shear studs. In Fig. 6.6, "P" and "T" are the respective loading directions parallel and transverse to the rib of profile decking.

Due to symmetry of loading beams in one-direction, half-models are constructed and loaded through 6 points corresponding to the specimens, as shown in Fig. 6.5 (b). CHS sections attached to the extended girders and beams (Fig. 3.8) are simulated by axial springs with very high stiffness values in horizontal direction and rigid restraints in vertical direction. Horizontal stiffness value of horizontal CHS sections connecting to the strong wall (on one edge) is 230 kN/mm, while that of diagonal CHS surrounding the three edges of the specimen (Fig. 3.8 (a)) is assigned with the average stiffness value (36 kN/mm).

Fig. 6.5: Information of a macro-model

* P and T denote parallel and transverse directions, respectively.

Fig. 6.6: Validation of shear stud model

6.3.2.2 Comparisons between FE modelling and test results for 2×3-S-IC

This section presents comparisons between the test of 2×3-S-IC and the corresponding simulation in terms of load-deflection curve and failure mode. Fig. 6.7 shows that the model can predict the overall load-deflection response very well despite the fact that the first drop in load is more significant and it occurs later compared with the test behaviour. The first drop in load is caused by the failure of the bottom most row of components in the girder-to-removed-column connection. In the FE model, one component failure indicates that two bolts in the same row fracture together. However, in the actual test, the two bolts actually fractured one after another. This is the reason that the first drop in load in the numerical model is more significant. Besides, the actual sub-structure leaned slightly towards the more vulnerable half when it was under loading, instead of deflecting vertically and symmetrically as assumed in the model. This assumption causes a delay in the failure of the first component in the connection model.

Shell elements cannot distinguish spalling, crushing or crack development of concrete slab. Hence, only the failure modes in steel members are compared with the simulation results. Fig. 6.8 (a) and Fig. 6.8 (b) show the photographs of the fractured components in the girder-to-removed-column connection subjected to sagging moment, while Fig. 6.8 (c) plots the component force versus displacement of the removed column. In the test, the bottom- and top-row components of the sagging moment connections (denoted by “S”) fractured at the respective displacements of 150 mm (0.57 times depth of composite girder including slab thickness, or $0.57h_g$ where $h_g = 265$ mm) and 340 mm ($1.27h_g$). In comparison, the corresponding components failed at the respective displacements of 206 mm ($0.78h_g$) and 319 mm ($1.2h_g$) in the FE model. From Fig. 6.9 (a), at the hogging end of the double-span girder (denoted by “H”), yielding of bottom flange was visible at a displacement of 100 mm ($0.38h_g$), in agreement with the behaviour of the bottom compressive component in the FE model (Fig. 6.9 (b)). Similarly, the model can also capture failure of the top- and middle-row bolts at the hogging end, as shown in Fig. 6.9 (a) and (b).

When the central displacement increased from 353 mm (1.64 times depth of

composite beam including slab thickness, or $1.64h_b$ where $h_b=215$ mm) to 530 mm ($2.46h_b$), the web angles in the beam-to-removed column connection were gradually torn up, as shown in Fig. 6.10 (a). The components in the model can capture this behaviour, as shown in Fig. 6.10 (b). At the final loading stage, the model can capture the fracture of steel decking around the removed column and along the double-span beam, as shown in Fig. 6.10 (c).

Fig. 6.7: Load-displacement curve of 2×3-S-IC

(a) The bottom-row component fracture ($0.57h_g$)

(b) The top-row component fracture (at $1.27h_g$)

*S indicates the sagging end of a beam.

(c) Force development of components in the girder-to-removed-column connection

Fig. 6.8: Failure of components in the girder-to-removed-column connection in 2×3-S-IC

(a) Failures at B2-H

(b) Force development of components at B2-H

*H indicates the hogging end of a beam.

Fig. 6.9: Failure of components in the hogging end of double-span girder in 2×3-S-IC

(a) Failure of angles and steel decking in the test

(b) Force development of components in the beam-to-removed column connection

(c) Fracture of steel decking

Fig. 6.10: Failure of beam-to-removed column connection and steel decking in 2×3-S-IC

6.3.2.3 Comparisons between FE modelling and test results for 2×2-S-IC

This section presents comparisons between the test of 2×2-S-IC and the corresponding simulation in terms of load-deflection curve and failure mode. Fig. 6.11 shows that the FE model can predict the overall load-deflection response well. Similar to 2×3-S-IC, failure of components in the girder-to-removed-column connection can be captured with acceptable tolerance (Fig. 6.12). The model also can predict the failure of components at the hogging end of the double-span girder at a correct range of displacement (Fig. 6.13). Moreover, the tearing process of angles in the beam-to-removed column connection (from 300 mm ($1.4h_b$) to 400 mm ($4.86h_b$)) in the test is predicted by descending forces of the joint components (from 256 mm ($1.19h_b$) to 407 mm ($1.89h_b$)) in the model, as shown in Fig. 6.14 (a) and (b). Lastly, fracture of the steel decking is also captured by the model (Fig. 6.14 (c)).

Fig. 6.11: Load-displacement curve of 2×2-S-IC

(a) The first bolt fracture at $0.73h_g$

(b) The last bolt fracture at $1.16h_g$

(c) Force development of components in the girder-to-removed-column connection

Fig. 6.12: Failure of components in the girder-to-removed-column connection in 2×2 -S-IC

(a) Failures at B1-H

(b) Force development of components at B1-H

Fig. 6.13: Failure of components in the hogging end of double-span girder in 2×2 -S-IC

Fig. 6.14: Failure of beam-to-removed column connection and steel decking in 2x2-S-IC

6.3.2.4 Comparisons between FE modelling and test results for 2x3-W-IC and 2x3-S-PI

Fig. 6.15 show that the simulation results provide good predictions of overall load-displacement responses for 2x3-W-IC and 2x3-S-PI. The critical failure modes are similar to those observed in 2x3-S-IC and 2x2-S-IC, which can be captured by the FE model (Table 6.2 and Table 6.3). Specifically, the experimental behaviour of crucial joints, including girder- and beam-to-removed column joints, and the hogging end of girder-to-column joint, can be captured by connector (spring) elements in FE

models. Decking fracture can be simulated by shell elements failure.

Fig. 6.15: Comparisons of load-displacement curves between simulations and tests for 2×3-W-IC and 2×3-S-PI

Table 6.2 : Comparison of failure sequence between test and simulation results for 2×3-W-IC

Location	Test results		Simulation results	
	Actual failure	Central displacement (mm)	Element failure	Central displacement (mm)
B2-S	First bolt fracture	166 (0.63 h_g)	First component failure	224 mm (0.84 h_g)
	Last bolt fracture	296 (1.12 h_g)	Last component failure	370 mm (1.4 h_g)
B2-H	Flange buckling	100 (3.85 h_g)	Compressive component yielding	89 (0.34 h_g)
	Top bolt fracture	370 (1.4 h_g)	Top component failure	357 (1.35 h_g)
	Middle bolt fracture	485 (1.83 h_g)	Middle component failure	480 (1.81 h_g)
B7-S	Angles torn up	422(1.96 h_b) ~ 550(2.56 h_b)	Descending component force	399(1.85 h_b) ~ 554(2.58 h_b)
Around removed column	Decking fracture	Final stage	Shell elements failure	Final stage

Table 6.3: Comparison of failure sequence between test and simulation results for
 2×3 -S-PI

Location	Test results		Simulation results	
	Actual failure	Central displacement (mm)	Element failure	Central displacement (mm)
B1-S	First bolt fracture	159 ($0.59 h_g$)	First component failure	204 ($0.76 h_g$)
	Last bolt fracture	308 ($1.14 h_g$)	Last component failure	274 ($1.01 h_g$)
B2-H	Flange buckling	100 ($0.37 h_g$)	Compressive component yielding	71 ($0.26 h_g$)
	Top bolt fracture	396 ($1.47 h_g$)	Top component failure	398 ($1.47 h_g$)
	Middle bolt fracture	534 ($1.98 h_g$)	Middle component approaching failure	487 ($1.8 h_g$)
B7-S	Angles torn up	424 ($1.93 h_b$) ~ 554 ($2.52 h_b$)	Descending component force	399 ($1.81 h_b$) ~ end
Around removed column	Decking fracture	Final stage	Shell elements failure	Final stage

After the comparisons, the 3D floor system modelling method shows the capability to predict the overall load-displacement response and precisely capture the failure mode of structural components. The verified models are used to conduct parametric analyses on aspect ratio, composite action, boundary condition and slab thickness in the subsequent section.

6.4 Further Discussions and Parametric Studies

6.4.1 Individual contributions of the load-resisting mechanisms

The 3D composite floor systems are very robust so that any slight changes in the slab thickness or varying degree of composite action would not affect the ultimate load-carrying capacity significantly. As discussed in Section 5.2, Specimens 2×3 -S-IC, 2×3 -W-IC and 2×3 -S-PI shared similar ultimate loads, which are underlined in Table 6.4 accompanied with the simulation results. The reason is that, different load-resisting mechanisms (or contributors) in a structural system are complementary

depending on structural features of the system. Hence, it is necessary to study these mechanisms separately to understand how these parameters can affect the structural behaviour.

Table 6.4 : The maximum loads of the four specimens

Specimen ID	Peak load (kN)	
	Model	Test
2×3-S-IC	<u>1193</u>	<u>1228</u>
2×2-S-IC	1598	1541
2×3-W-IC	<u>1170</u>	<u>1243</u>
2×3-S-PI	<u>1135</u>	<u>1275</u>

At small deformation stage, vertical loads applied on composite floor systems are resisted by flexure in composite girders, beams and slabs. When the floor system goes into large deformation stage, CA in double-span girders and double-span beams and TMA in slabs are mobilised. Hence, to elucidate the shift in load-resistance among these mechanisms, the respective load contributions from flexure and CA in the double-span girder and double-span beam, and composite slabs have to be separately ascertained. Flexure contribution (Fig. 6.16, Fig. 6.17 and Table 6.5) stems from plastic action of the double-span girder and the double-span beam with their corresponding joint resistance. Catenary action (Fig. 6.16, Fig. 6.17 and Table 6.5) arises from the double-span girder and the double-span beam with their corresponding joint resistance. Finally, the slab resistance (Fig. 6.16, Fig. 6.17 and Table 6.5) includes flexure and tensile membrane action of the slab and resistance of inner beams (excluding the perimeter beams).

The method of individual mechanism separation based on test results was clearly presented in Section 4.3.1. In the actual test, large amount of data acquisition channels was connected to strain gauges for obtaining the necessary reaction forces, which could then be used to calculate individual contributions. Due to some faulty strain gauges, the author failed to measure all the reaction forces for 2×3-S-IC, 2×2-S-IC and 2×3-S-PI. To accomplish this purpose, FE models are conducted to obtain the reaction forces. Fortunately, all the reaction forces of 2×3-W-IC were individually

measured and the contributions of different mechanisms were separated (Fig. 6.16) to verify the reliability of the FE models in this regard. Fig. 6.16 shows that the FE modelling can generally capture the development of contributions from flexure and CA in the double-span girder and the double-span beam, and contribution from composite slabs. Load contributions at the ultimate state in the respective test and FE results are 328 vs. 354 kN for flexure, 330 vs. 256 kN for CA and 572 vs. 560 kN for composite slabs. Therefore, it is clear that the FE modelling method is reliable to predict the individual contributions in spite of a slight underestimation for CA.

Fig. 6.16: Comparisons of individual contributions between the test and FE simulation results for 2×3-W-IC

Firstly, in the specimen with weak composite action (2×3-W-IC), the contributions of CA and flexure are comparable at the final stage (Fig. 6.16). However, at the final stage, CA in the other three specimens cannot develop as much significantly (Fig. 6.17 (a), (b) and (c)). The load contributions of CA at ultimate state are smaller in 2×3-S-IC (123.6 kN) and 2×2-S-IC (58.4 kN) compared with that in 2×3-W-IC (256.1 kN). That is to say weaker composite action can promote greater development of CA. In contrast, partial composite action would weaken flexure. As shown in Table 6.5, the load contribution of flexure in 2×3-W-IC (354.0 kN) is the smallest among all the specimens. The load contribution of the slab (560.1 kN) at the final stage is hardly affected by weak composite action compared with 2×3-S-IC (567 kN), since the slab contribution arises from TMA provided by reinforcing mesh and steel decking. As a result, the slab contribution (560.1 kN) is much greater than flexural contribution

(354.0 kN).

Secondly, the smaller span of 2×2-S-IC with higher double-span beam flexural rigidity enhances flexural action compared with 2×3-S-IC (viz. 851.8 kN for 2×2-S-IC and 502.6 kN for 2×3-S-IC in Table 6.5). The dominant flexure mode in 2×2-S-IC hinders the development of CA, as shown in Table 6.5. Besides, the slab contribution of 2×2-S-IC (687.8 kN) is greater than that of 2×3-S-IC (567 kN) due to more significant development of TMA. For a given central deflection, the slope (or rotation) of slabs in 2×2-S-IC is greater than that of 2×3-S-IC due to the shorter span, so TMA in the former is more dominant in the square slab.

Thirdly, as shown in Fig. 6.17 (c) and Table 6.5, minimum CA is developed in 2×3-S-PI due to the absence of restraint and presence of dominant flexure (578.1 kN) provided by the thicker slab ($h_s = 70$ mm compared to 65 mm) due to a construction error. However, the thicker slab does not increase the slab contribution at the ultimate state. In contrast, the slab contribution in 2×3-S-PI is slightly smaller than those in 2×3-S-IC and 2×3-W-IC (Table 6.5) due to its more flexible boundary condition. Lastly, for 2×2-S-IC and 2×3-S-PI, the respective flexural contribution is greater than slab contribution at the ultimate state (Table 6.5), and vice versa for 2×3-S-IC and 2×3-W-IC. The reason is that either the smaller span or the thicker slab would increase flexure more significantly than TMA. All the specimens eventually failed by the complete separation of the double-span girder and the double-span beam at the removed column location, when most of the joint components at the hogging ends of the double-span girder and double-span beam also fractured. Thus, the total load sustained from the 3 specimens with the same aspect ratio are fairly similar.

(a) 2x3-S-IC

(b) 2x2-S-IC

(c) 2x3-S-PI

Fig. 6.17: Development of individual load-resisting mechanisms based on FE results

Table 6.5: Contributions of individual mechanisms at the maximum load based on
FE results

Specimens	Individual contributions (kN) & percentages of total load (%)			Total load (kN)
	By flexure [*]	By CA [*]	By slab [*]	
2×3-S-IC (Control)	502.6(42.1)	123.6(10.4)	567(47.5)	1193
2×2-S-IC	851.8(53.3)	58.4(3.7)	687.8(43.0)	1598
2×3-W-IC	354.0(30.2)	256.1(21.9)	560.1(47.9)	1170
2×3-S-PI	578.1(50.9)	19.9(1.8)	537(47.3)	1135

^{*}Flexure and CA stems from the double-span girder and the double-span beam with their corresponding joint resistance, while the slab resistance includes flexure and tensile membrane action of the slab and resistance of inner beams.

6.4.2 Extended parametric studies

Besides the simulations for the test specimens, three numerical cases with three slab thickness values are investigated, viz. ST-85 and ST-100 represent the respective slab thickness of 85 mm and 100 mm, with all the other parameters remaining the same as the control specimen 2×3-S-IC. Additionally, a steel skeletal frame is also analysed to compare with the floor system, to study the influence of slabs on the robustness of structures subjected to an internal column loss scenario.

6.4.2.1 Slab thickness

As shown in Fig. 6.18, increasing the slab thickness would enhance the stiffness and load-carrying capacity, especially at small deformation stage. Thinner slabs would postpone the occurrence of the first peak load in load-displacement curves (Fig. 6.18). Surpassing the peak points, the gaps between the three curves gradually narrow down with increasing displacement until the final drop at around 500 mm, when the individual contributions are listed in Table 6.6. Firstly, thicker slabs can enhance flexure in the double-span girders, double-span beams, and composite slabs, thereby increasing the corresponding contributions as shown in Table 6.6. However, thicker slabs of ST-85 and ST-100 would impede the development of CA (governed by

dominant flexure) and postpone the mobilisation of CA. Besides, the slab contribution at the final stage is hardly affected by slab thickness as shown in Table 6.6, which is consistent with the finding from 2×3-S-PI with 70 mm thick slab in Section 6.4.1. Eventually, the respective load-carrying capacities of ST-85 and ST-100 are only 2.2% and 7.6% greater than that of 2×3-S-IC.

From the discussions in Sections 6.4.1 and 6.4.2, it is found that prominent flexure, such as that found in 2×2-S-IC, 2×3-S-PI and ST-85 and ST-100, would suppress development of CA, since they are the two complimentary components in the double-span girder and beam. The slab contributions in 2×3-S-IC, 2×3-W-IC, ST-85 and ST-100 are similar since they possess the same steel decking and reinforcement, and slab dimensions.

Fig. 6.18: Load-displacement curves of cases with different slab thicknesses

Table 6.6: Contributions of individual mechanisms in 2×3-S-IC, ST-85 and ST-100 at the ultimate state

Specimens	Individual contributions (kN)			Total load (kN)
	By flexure	By CA	By slab	
2×3-S-IC (65)	502.6	123.5	567.0	1193.0
ST-85	602.6	72.1	545.0	1219.6
ST-100	629.2	49.5	603.5	1282.2

6.4.2.2 Steel skeletal frame

To identify the role played by composite slabs in a floor system, a steel skeletal frame

which is exactly the same as the frame of 2×3-S-IC is loaded to failure. The loaded beams in the skeletal frame are laterally restrained to avoid lateral torsional buckling. Thus, the skeletal frame and the composite floor system share the similar failure mode. Specifically, in the skeletal frame, the components at girder-to-removed column joints fracture first starting from 300 mm of central deflection (Fig. 6.19 (a)), followed by the joint components failure at the hogging ends of the double-span girder between central deflections of 320 mm and 530 mm (Fig. 6.19 (b)), then the failure of joint components at the sagging and hogging ends of the double-span beam at 610 mm of central deflection (Fig. 6.19 (c) and (d)), afterwards load-carrying capacity experience a steep decrease (Fig. 6.20). The final failures in both the floor system and skeletal frame are indicated by the failure of joint components at the sagging and hogging ends of the double-span beam.

(a) Sagging ends of double-span girder

(a) Hogging ends of double-span girder

(a) Sagging ends of double-span beam

(a) Hogging ends of double-span beam

Fig. 6.19: Force development in crucial joint components in skeletal frame

Fig. 6.20: Comparison of load-deflection curve between skeletal frame and composite floor system

Comparing the load-displacement responses of the skeletal frame and the floor system (Fig. 6.20), it is found that the slab is the major contributor in the floor system to resist vertical load, especially at small deformation stage. At the first peak around 200 mm, the load-carrying capacity of the frame (273.3 kN) is only 23.4% of that of the floor system (1166.8 kN). However, the load-carrying capacity of the skeletal frame increases by 80.4% from 273.3 kN to 493 kN due to catenary action in the double-span girder and the double-span beams and inner beams. This significant increase in load at the final stage is not observed in the composite floor system. The reason is that the flexure in slabs already increase the load-carrying capacity from the beginning, and the slabs promote flexure in girders and beams through composite action but hinder the development of CA at large deformation stage. Besides, slabs would decrease the deformation capacity, viz. 511 mm and 620 mm for the respective floor system and frame.

From the scaled-model tests in Chapters 4 and 5, as well as the simulations in Sections 6.3 and 6.4, the author gains an appreciation of static behaviour of 3D composite floor systems subjected to a column removal scenario. Further, subsequent focus is placed on dynamic behaviour of prototype composite sub-structures subjected to a sudden column loss, which is more realistic for a blast event.

6.5 Study on a Full-scale 3D Composite Floor System

6.5.1 Geometry and material information

The full-scale sub-structure is extracted from a typical composite floor building, of which plan layout and configurations of connections are shown in Fig. 6.21 and Fig. 6.22, respectively. Girders are connected to columns by flush-end-plate connections, while beams are simply supported by web-cleat connections. The composite slabs of the model consist of 76 mm deep by 1.21 mm thick profiled steel decking, 64 mm thick concrete topping and A142 ($\phi 7@200$) reinforcing mesh placed at the centre of topping cross-section. Shear studs with diameter of 19 mm at 76 mm spacing are used to achieve full composite action between the slabs and the steel beams. Grade S275 steel is assigned to web angle with a yield strength of 275 MPa and an ultimate strength of 430 MPa. S355 with a yield strength of 355 MPa and an ultimate strength of 510 MPa is used for beams, columns and flush end plates. The steel decking and rebar are assigned with a yield strength of 314 MPa and 500 MPa, respectively, and an ultimate strength of 406 MPa and 675 MPa, respectively. The cylinder strength of concrete material is 30 MPa.

The slabs and beams are extended by one-quarter of the corresponding adjacent span lengths. The numerical model is vertically and horizontally restrained at the four surrounding edges to represent the boundary conditions of internal bays. The force-displacement relationships of components in the flush-end-plate joint can be obtained from EC3-1-8 (2005) and Piluso et al. (2001a), while those for the web-cleat joint from Yang and Tan (2013c). Tie function is applied between composite slabs and steel beams to simulate full composite action. Uniform loads are statically applied on the floors. Due to double symmetry of the sub-structure, loading and boundary conditions, only one-quarter model is built (Fig. 6.23).

Fig. 6.21: Structural layout of the full-scale substructure

Fig. 6.22: Details of connections in the full-scale substructure

6.5.2 Static analyses

The objective of this section is to statically analyse the model as the comparison basis for dynamic analyses. Fig. 6.24 shows the static load-displacement response of the model. Uniform loads are applied on the floor until ultimate failure occurs, corresponding to the fracture of all the joint components (G-1~G-4 and B-1~B-3) at the removed column location (Fig. 6.25 (a)). Failure of joint components at the removed column location develops from the bottom-most row of bolts upwards. After failure of the last component (top-row component B-3), the slab reinforcing mesh along the interior sides of the beams and girders also fractures severely, as shown in Fig. 6.26. Beyond this point, the system is not stable. The failure mode of the floor system under static loads exhibits ductile characteristics due to successive failures of the joint components (Fig. 6.25 (b)). The same finding was also observed in the tests (Section 4.2.2).

Comparing the failure mode in the full-scale sub-structure model with the actual test results, two critical points need to be addressed. Firstly, the common observation is that failure of the last joint component at the removed column location indicates final failure of the structure system. In the tests, failure of angles at the removed column location corresponds to the descending branch of Stage 3 (Section 4.2.2). Secondly, the reinforcing mesh in the full-scale sub-structure fractures severely (Fig. 6.26), while that in the test rarely fractured. The reason is that A193 reinforcing mesh with limited ultimate strain ($\epsilon_u=0.05$) is used in the model while mild steel round bars with inherent ductility ($\epsilon_u=0.23$) are adopted in the test.

Fig. 6.23: One-quarter model of the prototype sub-structure

Fig. 6.24: Static load-displacement curve of the prototype sub-structure

(a) Joint components at the removed column

(b) Forces development of joint components at the removed-column location

Fig. 6.25: Failure of joint components at the removed-column location

(a) Stress in y-direction reinforcement

(b) Stress in x -direction reinforcement

Fig. 6.26: Stress distributions of reinforcement at the ultimate state

6.5.3 Dynamic analyses

6.5.3.1 FE modelling

To simulate the sub-structure behaviour when subjected to sudden removal of an internal column, ABAQUS/standard solver is the optimal choice in this case. The accuracy of ABAQUS/standard simulating dynamic structural behaviour based on simplified macro-model has been validated by a number of studies (Fu 2009, Liu et al. 2015a, Fu et al. 2016).

(1) Material properties

In dynamic analyses, there are two important parameters to be considered, viz. material enhancement due to strain-rate effect and structural damping. The previous tests results (Liu et al. 2013) showed that the maximum strain rate during the testing is about 5 s^{-1} . Material tests under various high strain rates conducted by Marais et al. (2004) showed that strain rate effects usually become important for steel at strain rates ranging from 50 to 1000 s^{-1} . Thus, the material enhancement due to strain-rate effect is negligible and not considered in the present numerical models. For structural damping, this study adopts Rayleigh damping matrix assuming a linear combination of mass and stiffness matrix, as given below:

$$C = \alpha M + \beta K$$

$$\alpha = \frac{2w_1w_2(\xi_1w_2 - \xi_2w_1)}{w_2^2 - w_1^2} ; \beta = \frac{2(\xi_2w_2 - \xi_1w_1)}{w_2^2 - w_1^2} \quad (6.1)$$

where α and β are the mass and stiffness proportional damping coefficients, respectively. The terms w_1 and w_2 are the modal frequencies corresponding to the top two largest modal participation factors in loading direction (herein is vertical), which can be obtained through modal analysis. In this case, w_1 and w_2 are equal to 37.16 and 81.56, respectively. The terms ξ_1 and ξ_2 are the respective damping ratios of these two modes. Specifically, a damping ratio of 0.04 is widely used in steel structural design or analysis (Davison and Owens 2012), so this value is adopted in all the models in this study. The other input data is the same as that in static models.

(2) Loading method

Three loading steps are involved in the analyses as shown in Fig. 6.27. Firstly, uniform loads are applied onto the slabs in 1 second, which can be regarded as a quasi-static process. During this time, the internal column, which is about to be removed, is still intact. At the end of Step 1, the vertical reaction force at the bottom of the internal column can be obtained. Secondly, the restraint at the bottom of the internal column is replaced by the vertical external force which is equal to the vertical reaction force obtained at the end of Step 1. Then the vertical external force is reduced to zero in 0.01 second, which is one tenth of the natural period of the structure, as recommended in DoD (2013). Lastly, the remaining structure is analysed until 2 second, which is long enough to acquire the maximum displacement under sudden column loss.

Fig. 6.27: Loading scheme for free-fall analysis

6.5.3.2 Discussion on dynamic responses

In this study, the pseudo-static response of the sub-structure is defined as the relationship between the uniform floor loads and the corresponding maximum dynamic displacements. Usually, a pseudo-static response is compared to its corresponding quasi-static response to study the influence from dynamic effects. To obtain the full pseudo-static responses, extensive numerical simulations on the sub-structure subjected to sudden column loss need to be conducted with different magnitudes of UDL for each analysis representing the combined values of gravity and imposed loads.

Fig. 6.28 shows the displacement-time histories at the slab centre (above the removed column) when subjected to a range of UDLs under a sudden column loss scenario. As shown in Fig. 6.28, the largest and permanent deflections corresponding to a load of 3 kN/m^2 are 20 mm and 11 mm, respectively. The counterparts under a load of 13 kN/m^2 are 419 mm and 416 mm, respectively. Therefore, the ratios of rebound amplitude-to-the maximum dynamic displacement corresponding to these two loads are $9/20$ (viz. $(20-11)/20$) and $3/419$ (viz. $(419-416)/419$). It can be seen that the smaller the load, the larger is the ratio of rebound. In other words, the sub-structure model is dominated by elasticity under a lower level of load, while by plasticity under a higher level of load. When the applied UDL is below 13 kN/m^2 , the joint components still remain intact, and the floor system can rebound but undergoes permanent deformation smaller than 419 mm at a stable state. But if the UDL is greater than 13 kN/m^2 , all the joint components linking to the removed column would fracture successively, and the whole system will lose stability indicated by a steep gradient of central deflection (non-converging) at the end of 2 s in Fig. 6.28. Therefore, 13 kN/m^2 is deemed to be the ultimate dynamic load-carrying capacity of the sub-structure. The corresponding ultimate dynamic deformation capacity is 419 mm and the ultimate deflection contour is shown in Fig. 6.29.

Fig. 6.28: Displacement-time histories under different floor loads

Fig. 6.29: Deflection contours of the sub-structure under the ultimate load

Fig. 6.30 shows the full pseudo-static response compared with the corresponding quasi-static response. Each dot in the pseudo-static curve represents the maximum displacement and the corresponding UDL in Fig. 6.28. Fig. 6.30 shows that for a given displacement, the corresponding magnitude of UDL under static loading is greater than that under dynamic loading. For the numerical model investigated, the maximum static and dynamic load-carrying capacities are 19.3 kN/m^2 and 13.0 kN/m^2 , respectively. Besides, Fig. 6.30 shows that the dynamic ultimate state does not correspond to the static ultimate state as both the deformations and the loads at these two states are quite different from each other. On the whole, the sub-structure under dynamic loading fails earlier than under static loading. Collapse of the floor system under dynamic analysis is governed by the first joint component failure (G-1

in Fig. 6.25 (a)), while the floor system under static analysis can survive until the last component (B-3 in Fig. 6.25 (a)) failure at the removed column.

Additionally, the pseudo-static response calculated based on energy balance (Izzuddin et al. 2008) is applied to validate the pseudo-static curve obtained through dynamic analyses with the FE model. It can be concluded from Fig. 6.30 that the pseudo-static response from the numerical model agrees well with the pseudo-static response calculated by energy method before the fracture of the first joint component. Afterwards, the pseudo-static response is meaningless since the structure has already collapsed, as demonstrated by full dynamic analyses with the numerical model. However, the energy method cannot predict collapse. Hence, the energy method can only be adopted if the dynamic failure criterion is known, since this method cannot give any indications of dynamic ultimate loads or deformations.

*EM represents energy method.

Fig. 6.30: Comparison between static and dynamic responses

6.5.3.3 Discussion on dynamic increase factors (DIFs)

As mentioned in Section 2.2.2, there are force-based DIF_p and displacement-based DIF_u to be considered in the dynamic analyses of sudden column loss. DIF_p is used to evaluate the dynamic increase in load within the allowable deformation, while DIF_u reflects how much the static displacement is enlarged under dynamic effect to assess if a structure has adequate ductility under sudden loss of a column. A few researchers (Izzuddin 2010, Liu et al. 2013, 2015b, Liu et al. 2016) pointed out that force-based

DIF specified in DoD (2013) needs to be improved. Besides, DoD (2013) and GSA (2013) do not consider magnified ductility demand due to dynamic effect. Therefore, both force-and displacement-based DIFs are worthy to be further studied in this research.

(1) Comparisons between dynamic analyses and DoD (2013)

For the 3D composite floor system discussed in this section, both DIF_u and DIF_p can be obtained from the pseudo-static and quasi-static responses in Fig. 6.30. The curves of DIF_u versus floor load and DIF_p versus central displacement are plotted in Fig. 6.31 (a) and Fig. 6.31 (b), respectively. Assuming that the double-span girder deforms linearly, total rotation of the girders can be calculated based on central displacement. Hence, with Eqs. (2.2) and (2.3), the central displacement can be normalised as θ_{pra}/θ_{yb} of the girder for comparison with the DIF recommended in DoD (2013) (Fig. 6.31 (b)). The reason is that the DIF in DoD (2013) is determined based on the smallest normalised rotation θ_{pra}/θ_{yb} of the primary beams. For instance, if girders are treated as primary members of a 3D structure, the DIF value is a function of the normalised rotation θ_{pra}/θ_{yb} of the girders.

From Fig. 6.31 (a), the value of DIF_u gradually rises with increasing loads until the structure collapses, which can be inferred from the evolution of u_{dy}/u_{st} (Eq. (2.5)) in Fig. 6.30. This is to say, for 3D sub-structures, the difference between dynamic ductility demand and static ductility demand increases with increasing load. Ultimately, the value of DIF_u corresponding to the final failure is about 2.8. However, the value of DIF_p decreases with increasing central displacement of the slab, as shown in Fig. 6.31 (b). When the floor system is in the elastic stage of deformations, the value of DIF_p is close to 2.0. DIF_p will reduce to around 1.4 at normalised rotation θ_{pra}/θ_{yb} of 2.4 then remain constant thereafter. The trend of DIF_p is similar to that given in DoD (2013) at small deformation stage but different from that at large deformation stage (Fig. 6.31 (b)). The DIF_p value determined by DoD (2013) tends to be unsafe compared to the dynamic analyses at final stage. It is not observed that DIF_p will increase significantly at large ratio of θ_{pra}/θ_{yb} , as stated in Izzuddin (2010). The reason is that, at large deformation stage, TMA and CA in 3D composite floor system cannot significantly enhance the quasi-static resistance (Fig. 6.30).

Fig. 6.31: Dynamic increase factors of 3D sub-structure

Fig. 6.32: Force-based DIF_p of 2D frames

The trend of DIF_p is totally different from those of web-cleat and flush-end-plate connections as proposed by Liu et al. (2013) and Liu et al. (2016), as shown in Fig. 6.32. The reason of difference between the 3D sub-structure and the 2D frames are further discussed in the next paragraph.

(2) Comparisons between 3D composite floor system and corresponding 2D steel frames

The 3D composite floor system discussed here incorporates web-cleat and flush-end-

plate connections, of which both the static and pseudo-static (dynamic) load-displacement curves have been experimentally obtained by Liu et al. (2013) and Liu et al. (2016). The developments of DIF_p with θ_{pra}/θ_{yb} for both types of connections were determined as shown in Fig. 6.32. They found that the DIF_p value of the web-cleat connection increases with increasing displacement until failure occurs (Fig. 6.32 (a)). This is caused by the mechanical behaviour of web-cleat connections, which have no flexural resistance, and vertical load can only be resisted by catenary action, as shown in Fig. 6.33 (a). Liu et al. (2016) also found that the DIF_p value of flush-end-plate connections decreases at flexural stage but increases at CA stage (Fig. 6.32 (b)), but they did not point out the inherent reason. Actually, the reason can be inferred from the evolution of UDL_{st}/UDL_{dy} in Fig. 6.33 (b), which shows both the static and the dynamic responses of 2D steel frames with flush-end-plate connections. However, the DIF_p -versus-displacement history of the 3D composite floor system is distinctly different from both the 2D steel frames with either web-cleat connections or flush-end-plate connections. This is essentially because the 3D slab effect and both types of joints are involved in the 3D composite floor system. The DIF_p value of the 3D composite floor system will not significantly increase at large deformation stage since TMA and CA in 3D composite floor system cannot significantly enhance the quasi-static resistance compared to CA in 2D frames. Hence, it can be concluded that the evolution of DIF_p value depends on the characteristics of load-displacement curves under static loading.

(a) 2D steel frames with web-cleat connections

(b) 2D steel frames with flush-end-plate connections

(c) 3D floor systems

Fig. 6.33: Different forms of load-displacement curves for different structural systems

Liu et al. (2013) and Liu et al. (2016) pointed out that the DIFs are dependent on connection type and presented the DIF_u and DIF_p values for web-cleat and flush-end-plate connections when they reach the final failure. Since the numerical models in this chapter employ web-cleat and flush-end-plate connections, the dynamic increase factors at the final failure of the 3D composite floor system are compared with their counterparts of these two types of joints in Table 6.7. It can be seen that the displacement-based DIF_u of the 3D floor system is twice that of corresponding 2D steel frames due to the combined TMA action in slabs and CA in beams. Hence, the dynamic effects will enlarge the ultimate displacements of 3D floor systems more significantly than those of 2D steel frames. That is to say, if the ductility of a 3D

structure is designed based on simplified 2D analyses, the 3D structure will be unsafe when it is subjected to a sudden column removal scenario. Furthermore, the force-based DIF_p of the 3D floor system is smaller than that of either web-cleat or flush-end-plate connections. This means that a 3D structure will be over-conservative in terms of resistance if it is designed based on 2D analyses. All in all, compared with 2D frames, 3D structures should be designed with greater ductility but lower resistance to resist sudden column loss. These features are determined by the forms of load-displacement curves, as shown in Fig. 6.33 (c).

Table 6.7: Comparisons of the DIFs at the final failure between the 3D composite floor system and the 2D steel frames

	3D floor system	2D Web-cleat	2D Flush-end-plate
Displacement-based DIF_u	2.8	1.4	1.6
Force-based DIF_p	1.4	2.8	1.8

6.6 Summary and Conclusions

At the beginning of this chapter, FE models are verified at slab- and system-levels, showing the capability to capture major structural behaviour of 3D floor systems. The verified models are used to study the load-resisting mechanisms in 3D floor systems and identify the role played by slabs. It is found that as long as girder- and beam-to-column joints have sufficient ductility, a 3D composite floor system can be very robust due to the presence of slabs, due to two reasons. Firstly, the slab can increase the load-carrying capacity of skeletal frame by around three times. Secondly, different load-resisting mechanisms (viz. flexure and CA in double-span girder and double-span beam, and flexure and TMA in slabs) in the floor system are complementary depending on structural features of the system. As a result, a slight change in the slab thickness or different degree of composite action, would not affect the ultimate capacity significantly. However, slabs would decrease the deformation capacity of the structure compared with the skeletal frame. In between the skeletal frame and full-composite floor system, the floor system with partial composite action has good

ductility without compromising load-carrying capacity.

Inter-dependence among the three load-resisting mechanisms is revealed. Firstly, dominant flexure would suppress the development of CA, since they are the two complimentary components in the double-span girder and double-span beam. Shorter span, stronger composite action and a thicker slab can promote flexure but suppress CA, and vice versa. The slab contribution at the final stage involves TMA which is fairly independent of flexure and CA in the double-span girder and double-span beam. TMA remains constant if steel reinforcement, profiled decking and dimensions of the slab are the same.

Furthermore, the verified modelling method is used to simulate a full-scale 3D composite floor system, confirming the static behaviour in the prototype substructure subjected to a column removal scenario. Subsequently, the model is analysed under sudden column loss and the pseudo-static response is compared with the corresponding quasi-static response to obtain the dynamic increase factors (DIFs). The obtained DIFs are then compared with those recommended by DoD (2013) and those of 2D steel frames with the same connection types. Some conclusions are summarised as follows:

- (1) The dynamic displacement is greater than the corresponding quasi-static displacement under the same load, and the difference will rise with increasing loads. The displacement-based DIF_u at the ultimate dynamic load-carrying capacity is about 2.8.
- (2) The resistance of the 3D composite floor system under sudden column loss decreases by 31.6%. Specifically, the static and dynamic load-carrying capacities are around 19.3 kN/m^2 and 13.0 kN/m^2 , respectively. The dynamic ultimate state does not correspond to the static ultimate state in terms of load and deformation.
- (3) The pseudo-static response from numerical results agree well with the pseudo-static response calculated by energy method. However, the energy method does not give any indication of ultimate limit state or failure mode.
- (4) The trend of DIF_p -versus-deformation curve for a structure depends on the

characteristics of the static load-displacement curve.

(5) In 3D composite floor systems, the force-based DIF_p decreases with increasing central displacement, and will not increase significantly during CA or TMA stage. Even so, the DIF_p recommended in DoD (2013) tends to be unsafe at large displacement stage.

(6) The 3D composite floor system requires more ductility than 2D steel frames with the same connection types, and the DIF_p for the 3D composite floor system is smaller than those for 2D steel frames.

CHAPTER 7 A MECHANICAL MODEL OF COMPOSITE FLOOR SYSTEMS UNDER AN INTERNAL COLUMN REMOVAL SCENARIO

7.1 Introduction

As elucidated in Chapter 6, presence of composite slabs can enhance robustness of 3D steel frames subjected to a column removal scenario. When it comes to dynamic behaviour, the 3D composite floor system requires more ductility than 2D steel frames with the same connection types, and the force-based DIF_p for the 3D composite floor system is smaller than those for 2D steel frames. Hence, to evaluate a building, a 3D assembly of elements and components are recommended, while simplified skeletal models are not permitted (DoD 2013). However, there is no practical model which can accurately predict robustness of 3D composite floor systems under a column removal scenario. This provides the impetus for the author to fill the technical gap by developing a mechanical model for calculating the entire push-down load-deflection response.

The model incorporates three branches to capture the response at small deformation, transitional and large deformation stages. For ease of hand calculations, the load carried by a 3D composite floor is assumed to be the summation of the respective contributions from the composite slabs and the steel sub-frame. At small deformation stage, the load is resisted through flexural action. But at large deformation stage, structural capacity is mostly contributed by membrane action in the slabs and catenary action in the steel beams. The assumptions and failure criteria in the last stage are based on the results of actual experimental tests in Chapters 4 and 5. The derivations are discussed in detail, followed by experimental and numerical verifications of the model, showing reasonable accuracy in predicting load-displacement curves of 3D composite floor systems under an internal column removal scenario. Besides, the model can capture the effects of key parameters, such as slab aspect ratio, joint type, number of joint bolts, slab thickness, reinforcement ratio in the slab and thickness of steel decking. Most importantly, the procedure of the model can be implemented by a spreadsheet method, which provides a simple and numerical robust tool for

engineers to calculate progressive collapse resistance of structures for a missing column scenario.

Based on this model, eight sub-structures with various types of joints are investigated to figure out the robust structural design. Besides, four sub-structures with different slab aspect ratios are compared to identify how aspect ratio affect load resistance. Lastly, the analytical results are further verified by numerical simulations from the perspective of energy quantities stored in individual structural members.

7.2 Proposed Mechanical Model

As shown in Fig. 7.1, load-displacement curves at small deformation and transitional stages are assumed to be linear, while large deformation stage involves nonlinear response. The terms w and δ in Fig. 7.1 denote the respective uniformly distributed load and central deflection. The three stages are demarcated by two respective indicators of vertical displacement of the removed column, viz. effective slab thickness (h_e) and depth of composite girder including the slab (h_g). At small deformation stage ($0 \sim h_e$), vertical load is resisted by flexure in slabs, secondary steel beams and girders. In this paper, main beams are termed as girders while secondary beams are simply denoted as beams. At transitional stage ($h_e \sim h_g$), flexure is considerably weakened and Compressive Arch Action (CAA) kicks in. For conservatism, the proposed model does not focus on CAA as this development is governed by surrounding in-plane restraint stiffness. After the central deflection (δ) exceeds h_g , CA in beams and girders over the missing column is mobilised in two directions and TMA in slabs becomes dominant. In Section 4.3.1, it is found that when central displacement δ reached around $2h_g$, the load-deflection response went into descending branch. In other words, the slab system could not sustain a further increase in UDL. Hence, in the proposed mechanical model, $\delta > 2h_g$ is defined as the failure criterion, which is equivalent to the failure criterion of 10% of the span as assumed in Park and Gamble (2000). Between h_g and $2h_g$ (Fig. 7.1), large deformation governs structural behaviour of 3D composite floor systems. The model explicitly considers the contribution from structural members based on virtual work principle. The theory and derivations of structural behaviour in three stages are introduced in Sections 7.2.1, 7.2.2 and 7.2.3, respectively.

Fig. 7.1: Three stages of the proposed model

7.2.1 Small deformation stage ($\delta \leq h_e$)

To simplify the model for hand calculations or excel spreadsheet applications, the load applied on the composite floor is assumed to be sustained by composite slabs and the steel sub-frame independently. The UDL-deflection response of a composite slab is dealt with by applying small-deformation thin plate theory (Timoshenko and Woinowsky-Krieger 1987) that assumes the maximum deflection δ of the composite slab is smaller than its equivalent thickness (h_e). For steel frames at small deformation stage, work done is mostly concentrated at the joints in double-span girders and beams, while the girders and beams remain straight and elastic between the deformed joints. With known component properties of the connections and an assumed central deflection δ for the composite slab, the load carried by the sub-frame can be calculated based on virtual work principle.

At small deformation stage, the slabs contribute most towards load-carrying capacity, while the steel joints play a minor role. The degradation of joint stiffness with load does not affect overall stiffness of the composite floor. In contrast, any variations in the slab thickness can significantly affect load-carrying capacity of the composite floor. Hence, the vertical deflection of one slab thickness (h_e) is defined as the upper limit of small deformation stage.

7.2.1.1 Load contribution of composite slabs

To calculate the load-deflection response of a rectangular plate, the applied load and the boundary conditions around the plate are required. For 3D composite floors under an internal column removal scenario, the slab is under UDL with all edges fixed. The equation of deflection for a uniformly loaded rectangular plate with all-round fixed edges is given by Timoshenko and Woinowsky-Krieger (1987):

$$\delta = \eta w a^4 / D \quad (7.1)$$

where η is a coefficient related to the aspect ratio of the composite slab. For instance, η is equal to 0.0022 and 0.00126 for the aspect ratio of 1.5 and 1 (Timoshenko and Woinowsky-Krieger 1987), respectively. The term w is the magnitude of UDL, a is the slab width with a smaller dimension and D is the flexural rigidity. To simplify calculations, flexural rigidity of a composite slab can be obtained based on equations for isotropic plates. The current code (EC4-1-1 2004) does not have any specifications on simplifying a composite slab as an equivalent isotropic plate. However, EC4-1-2 (2005) recommends a method to calculate the effective thickness of a composite slab for fire resistance assessment. Based on this recommendation, the authors propose a simplified approach to calculate the equivalent thickness (h_e) of a composite slab. For re-entrant composite slabs, h_e is calculated as Eq. (7.2).

$$h_e = h_c + h_p / 2 \quad (7.2)$$

where h_c is the thickness of concrete topping and h_p is the depth of profile sheeting, as shown in Fig. 7.2 (a). For composite slabs with trapezoidal decking, h_e is equal to h_c (Eq. (7.3)), as shown in Fig. 7.2 (b). Additionally, it is more appropriate that h_e is ranged between $a/80$ and $a/220$ for structural slabs (Timoshenko and Woinowsky-Krieger, 1987).

$$h_e = h_c \quad (7.3)$$

The flexural rigidity (D) of an isotropically reinforced concrete slab is calculated by Eq. (7.4).

Fig. 7.2: Simplification of the slab cross-section

$$D = \frac{E_c}{1 - \nu_c^2} [I_c + (n - 1)I_s] \quad (7.4)$$

where $n = E_s/E_c$, E_s and E_c are respective elastic modulus of steel and concrete, and ν_c is Poisson's ratio for concrete. According to Park and Gamble (2000), ν_c has minor influence on the stiffness of reinforced concrete slabs. Besides, ν_c approaches zero when the slabs crack. Hence, the term ν_c^2 in Eq. (7.4) can be neglected. The term I_c is the moment of inertia of cross-section per unit width of the slab ($I_c = 1 \cdot h_c^3 / 12$), I_s is the moment of inertia of the steel reinforcement and decking per unit width about the neutral axis (N.A.), which coincides with centroidal axis of the equivalent cross-section. This assumption gives acceptable error, since the steel reinforcement and decking are minor contributors to the flexural rigidity of a cross-section. The term I_s is equal to $\sum_i A_{s,i} \cdot h_{s,i}^2$, where $A_{s,i}$ is the area of i -th steel layer (steel decking or reinforcement) per unit width, $h_{s,i}$ is the distance between the i -th steel layer and N.A.

7.2.1.2 Load contribution of sub-frame

For steel frames under an internal column removal scenario, it is assumed that rotations of joints in the affected bays (Fig. 7.3) proportionally increase with

increasing vertical deflection, while the girders, beams in between the deformed joints remain straight. Vertical load is resisted by flexure at small deformation stage. The deviation caused by this assumption is negligible, since the flexural contribution of the sub-frame is much smaller than that of the slabs, which can be seen in the worked example (Section 7.4.1). Since the contribution of simple joints to flexural stiffness is negligible, only the moment-resisting joints are considered. To calculate internal energy stored in a joint, the property (or effective stiffness coefficient $K_{\text{eff},r}$) of each component in a connection should be quantified. The term $K_{\text{eff},r}$ is the combination of stiffness coefficients of basic components in series, such as end-plate in bending, beam web in tension, plate in tension, bolt in tension, bolt in shearing and so on, which can be determined by EC3-1-8 (2005). Alternatively, the force-displacement response of each bolt row ($F_r-\Delta S_r$) (Fig. 7.4) can be obtained through individual component tests. The term h_r is the distance from the r -th bolt row to the compression centre, as shown in Fig. 7.4. At small deformation stage, for bolted end-plate connections, the compression (rotation) centre is assumed at the centre of the compression flange of the connected girder (EC3-1-8 2005), while neglecting deformations of components in compression (Fig. 7.4). It should be mentioned that Fig. 7.4 shows the commonly used moment-resisting joint. Besides, the proposed model can be applied to other types of joints.

The maximum vertical deflection (δ) of a composite slab at the removed column location is used to determine the rotations of joints (ϕ_j) and elongations of components ($\Delta S_r = \phi_j \cdot h_r$). Once ΔS_r and $F_r-\Delta S_r$ curve of the j -th joint are obtained, internal work stored in the j -th joint is calculated as $\sum_r \int F_r(\Delta S_r) d\Delta S_r$. Total internal work done by all the moment-resisting joints is $\sum_j \sum_r \int F_r(\Delta S_r) d\Delta S_r$. On the other hand, the external work done by UDL sustained by the sub-frame can be calculated using integral as long as the function of the deformed slab surface is quantified. However, it is very difficult but not necessary to determine the exact deformed function of the slab, since flexural resistance of moment-resisting joints is much smaller than that of the slab which can be seen in the worked example (Section 7.4.1.2). Alternatively, UDL is represented by multiple point loads applied on the

sub-frame, as shown in an example (Fig. 7.3). Vertical displacement (v_i) at point i can be expressed in terms of δ based on the simplified assumption that the beams and girders deflect linearly in the respective x - and y - directions. The magnitude of each point load (P_i) can be determined based on its tributary area as shaded in Fig. 7.3. Hence, external work done by the applied loads is $\sum_i P_i \cdot v_i$. Equating external work with internal work, the applied loads $\sum_i P_i$ can be computed according to Eq. (7.5).

$$\sum_j \sum_r \int F_r(\Delta S_r) d\Delta S_r = \sum_i P_i \cdot v_i \quad (7.5)$$

Fig. 7.3: Deformed joints considered, and numbering, tributary area and vertical displacement of multiple loading points

* This rotation centre is applied for small deformation stage

Fig. 7.4: Components in flush-end-plate joint

7.2.2 Transitional stage ($h_e < \delta \leq h_g$)

The response up to h_e has been obtained in Section 7.2.1. Between h_e and h_g , a straight line is assumed as transitional stage (Fig. 7.1), whose gradient is determined by the load enhancement between $\delta=h_e$ and $\delta=h_g$. The load at h_g can be calculated using equations in large deformation stage in Section 7.2.3. The load enhancement arises from strength hardening of joints and nonlinear large deformations of all structural components. It should be mentioned that rigid-plastic material model is assumed for steel decking, beam sections and reinforcement at large deformation stage (Section 7.2.3.2).

7.2.3 Large deformation stage ($h_g < \delta \leq 2h_g$)

At large deformation stage, the proposed model is based on conservation of energy which means that internal strain energy (W_{in}) is equated with external work (W_{ex}) by applied loads. Strain energy stored in deformed structural components is based on assumed failure modes (Section 7.2.3.1), including deformed shaded regions and 45° -direction yield lines in composite slabs, plastic hinges in beams (Fig. 7.5 (a)), and joint deformations (Fig. 7.6 and Fig. 7.7). To quantify total strain energy in the

structural system, material properties and deformations of individual structural components are required. For the analysis of a sub-structure, the first term is known, while the second term can be determined based on geometric assumptions (Section 7.2.3.2) and compatibility conditions (Section 7.2.3.3) for an assumed central deflection δ . Total strain energy can be calculated from:

$$W_{in} = 4(W_{OE} + W_{EF} + W_{FH} + W_{OH} + W_{yl}) + W_{jc} + W_{ph} \quad (7.6)$$

where W_{OE} , W_{EF} , W_{FH} and W_{OH} denote the respective strain energy stored in the deformed shaded regions of one-quarter of the structural system along lines OE , EF , FH and OH (Fig. 7.5 (a)) and F is the location of column removal; W_{yl} is the strain energy stored in one 45°-direction yield line along OG ; W_{jc} and W_{ph} represent the respective strain energy stored in all the joint components and plastic hinges in the beams.

One the other hand, external work done by the applied load is calculated based on assumed shape function ($V(x,y)$) of the slab (geometric assumptions in Section 7.2.3.2) and UDL value (w) (Eq. (7.7)). Equating internal strain energy (W_{in}) with external work (W_{ex}), one can obtain w as a function of δ . In this way, the static load-displacement curve at large deformation can be obtained by implementing step-by-step procedure until the failure criterion ($\delta > 2h_g$) is met. It should be mentioned that the failure mode and geometric assumptions are obtained from observations of actual test results (Chapters 4 and 5).

$$W_{ex} = \int w dA \cdot V(x, y) \quad (7.7)$$

where dA is the infinitesimal area of the slab.

7.2.3.1 Failure mode

Sections 4.2.2 and 5.3 show that the 3D composite floors failed in a ductile way as long as the joints had sufficient ductility. Cracking of the concrete slab developed along the inner edges of the four perimeters; the concrete in compression zone was crushed severely along the double-span girder, double-span beam and 45°- direction towards the corner columns. Based on these findings from the actual tests, the failure

mode of a composite slab subject to an internal column loss is shown in Fig. 7.5 (a).

Negative yield lines are mobilised along perimeters OE and OH while positive yield lines are developed along double-span girder (EF), double-span beam (FH) and 45° -direction (OG). Failure of the joint components always starts from the bottom row of girder-to-removed-column joint, followed by successive failure of the bottom most row upwards. Good ductility and strength of simple joints allows plastic hinges to be formed in the beams. In comparison, ductility of moment-resisting joints is not as good as that of simple joints, while the flexural rigidity of girders far exceeds that of beams, so no plastic hinge is formed in girders due to the premature failure of moment-resisting joints.

Since the floor system is double symmetric, subsequent discussions will focus on one-quarter composite slab. The dimensions of the slab panel are l by rl with an aspect ratio of r ($r \geq 1$). As shown in Fig. 7.5 (a), the plan view of the undeformed slab surface is $OEF GH$, while the corresponding deformed shape is $O'E'F'G'H'$ allowing for horizontal displacements towards F . The alphabets in brackets (M and N) denote the respective centres of rotation for girder B1 together with the rigid plate $OEF G$ (Fig. 7.5 (a) and (b)) and for beam B2 together with inner beam B3 and the rigid plate OGH (Fig. 7.5 (a) and (c)).

Rotation centre N

B2

In tension

Rotation centre M

B1

In compression

Fig. 7.5: Failure mode of the floor

*All the symbols will be referred and explained in the subsequent sections.

Fig. 7.7: Side view of Section II-II

7.2.3.2 Assumptions

The following assumptions are proposed based on the failure mode (Section 7.2.3.1), aiming to quantify the deformations of all the structural components in a deformed floor system.

- (1) The slab is fully fixed at all the perimeter edges due to the presence of adjacent

bays. Thus, negative yield lines are formed along supporting edges OE and OH , and positive yield lines are developed along double-span girder (EF), double-span beam (FH) and internal bisector (OG) of the slab corner (Fig. 7.5 (a)).

- (2) Rigid plates OGH and $OEFG$ remain flat and compatibility is only maintained along yield line OG (45° -direction) so that plates OGH and $OEFG$ share the same rotation θ . Rigid plate OGH rotate at an angle θ about the supporting edge (y -direction) together with girder B2. The rotation line passes through the centre of girder B2 (Point N directly under point H in Fig. 7.5 (b) and Fig. 7.6 (a)). Similarly, rigid plate $OEFG$ and girder B1 rotate at an angle θ about the line in x -direction passing through the centre of girder B1 (Point M directly under point E in Fig. 7.5 (c) and Fig. 7.7 (a)). The deformation of inner beam B3 is compatible with those of plates $OEFG$ and OGH , forming a plastic hinge in B3 at the intersection with internal bisector OG (Fig. 7.5 (a)).
- (3) Elongations of steel reinforcement and decking run perpendicular to the deformed shaded regions OE , EF , FH and OH and concentrate in these regions. Since concrete spalls off or crushes severely in these regions at large deformation stage, the contribution of concrete is not considered in regions OE , EF , FH and OH . However, moment resistance of the slab along yield line OG is taken into account. It is also assumed that steel reinforcement and decking cannot sustain compression.
- (4) Both the steel reinforcement and steel decking adopt rigid-plastic material model.
- (5) Connections are much more flexible than steel girders and beams, so only the deformations of joint components are considered while axial elongations of girders and beams are ignored.
- (6) The connection behaviour is simulated by a component-based model to calculate strain energy stored in the components of the joints. Both girder-to-column moment-resisting, beam-to-column and beam-to-beam simple joints at both the sagging and hogging ends of the members are considered at large deformation stage.

- (7) The deflections of supporting peripheral girders and beams are negligible compared with the slab centre deflection δ .

7.2.3.3 Deformations of deformed structural components

Conservation of energy is applied to the proposed model at large deformation stage. Strain energy is stored in the reinforcement in the four deformed regions, y-direction steel decking in regions OE and FH, deformed joints, plastic hinges in the beams and yield lines in the slabs (Eq. (7.6)). Therefore, extensions of the joint components, reinforcement, steel decking, rotations of plastic hinges and yield lines must be obtained first. It should be noted that when the extension of a joint component or a rebar exceeds the respective limiting value, it fractures and no longer sustains load or stores energy. In this model, the fracture of joint components at girder and beam ends and steel reinforcement in deformed regions OH , OE , EF and FH is considered, while fracture of steel decking is not taken into account. The limiting extensions of joint components can be determined based on component-based models (Piluso et al. 2001a, Taib 2012, Yang and Tan 2013b) or component tests. The limiting extension of reinforcement (Δ_{lim} in Eq. (7.8)) is related to the fracture strain (ϵ_u) and ‘free length’ of a rebar, which is assumed to be the spacing (d_{sp}) of transverse bars in the composite slab (Burgess 2017). This assumption is reasonable since the transverse bars act as anchors on either side of a crack. It assumes the bond strength between these two bars is zero, which is an extreme case. Burgess (2017) mentioned that fracture strain of the rebar can be scaled down to represent a particular bond strength model or obtained from pull-out tests.

$$\Delta_{\text{lim}} = d_{\text{sp}} \epsilon_u \quad (7.8)$$

For region OH , the x -direction reinforcement is subjected to tension and deforms equally from the beginning of large deformation stage (assumption (2)), so it fractures if the extension $\Delta_{\text{OH,R}}^x > \Delta_{\text{lim}}$. The situation is the same as the y -direction reinforcement in region OE . For regions EF and FH , fracturing of reinforcement always starts from the slab centre (F) and progresses towards the edges along FE and FH , respectively (Fig. 7.5 (a)).

To describe the stress state in x -direction reinforcement in region EF at a deflection δ , two variables ($y_{0,R}$ and y_{lim}) are introduced (Fig. 7.8 (a)). The term $y_{0,R}$ indicates that the x -direction reinforcement between $y=0$ and $y= y_{0,R}$ is in compression, so the corresponding deformation ($\Delta_{EF,R}^x$) is assumed to be zero according to assumption (3). The term y_{lim} denotes that the x -direction reinforcement between $y=0$ and $y= y_{lim}$ remains intact. As a result, only the x -direction reinforcement between $y= y_{0,R}$ and y_{lim} is effective for storing strain energy. The same concept is extended to the y -direction reinforcement in region FH . As shown in Fig. 7.8 (b), $x_{0,R}$ and x_{lim} denote the respective lengths of reinforcing mesh in compression and prior to fracture. For region FH , one additional term $x_{0,D}$ represents the length of steel decking in compression. All these lengths, $y_{0,R}$, y_{lim} , $x_{0,R}$, x_{lim} and $x_{0,D}$, will reduce with increasing slab deflection. It should be mentioned that the steel decking only contributes in the rib direction (y -direction in Fig. 7.5 (a)) in regions OE and FH .

(a) x -direction reinforcement in EF

(b) y-direction reinforcement in FH

Fig. 7.8: Progress of reinforcement fracture

All the reinforcement extensions in regions OH ($\Delta_{OH,R}^x$), OE ($\Delta_{OE,R}^y$), EF ($\Delta_{EF,R}^x$) and FH ($\Delta_{FH,R}^y$), the steel decking extension in regions OE ($\Delta_{OE,D}^y$) and FH ($\Delta_{FH,D}^y$), and the critical variables ($x_{0,R}$, x_{lim} , $x_{0,D}$, $y_{0,R}$ and y_{lim}) can be quantified according to assumption (2) and geometry compatibility (Fig. 7.6 and Fig. 7.7). These terms will be discussed in detail in subsequent section.

(1) Extension of x-direction reinforcement in region OH — $\Delta_{OH,R}^x$

Plate OGH (Fig. 7.5 (a) and Fig. 7.6 (a)) rotates about the y-direction edge passing through point N directly below point H in Fig. 7.6 (a). Hence, the extension of x-direction reinforcement in region OH ($\Delta_{OH,R}^x$) is calculated according to Eq. (7.9).

$$\Delta_{OH,R}^x = \theta \cdot h_{R,b} \leq \Delta_{lim} \quad (0 \leq y \leq l) \quad (7.9)$$

where $h_{R,b}$ is the distance between reinforcement layer and the centre line of beam B2 (Fig. 7.6 (b)).

With increasing slab rotation θ , the reinforcement fractures when $\Delta_{OH,R}^x > \Delta_{lim}$.

(2) Extension of y-direction reinforcement and steel decking in region OE — $\Delta_{OE,R}^y$ and

$\Delta_{OE,D}^y$

Plate $O E F G$ (Fig. 7.5 (a)) rotates about the x -direction edge passing through point M (Fig. 7.7 (a)). Hence, $\Delta_{OE,R}^y$ and $\Delta_{OE,D}^y$ are calculated according to Eq. (7.10).

$$\Delta_{OE,R}^y = \theta \cdot h_{R,g} \leq \Delta_{lim}; \quad \Delta_{OE,D}^y = \theta \cdot h_{D,g} \quad (0 \leq x \leq l) \quad (7.10)$$

where $h_{R,g}$ is the distance between reinforcing mesh layer and the centre line of girder B1, while $h_{D,g}$ is the distance between steel decking and the centre line of girder B1 (Fig. 7.7 (c)).

With increasing deflection, the reinforcement fractures when $\Delta_{OE,R}^y > \Delta_{lim}$.

(3) *Extension of x -direction reinforcement in region EF — $\Delta_{EF,R}^x$, $y_{0,R}$ and y_{lim}*

As shown in Fig. 7.6 (a), for section I-I, $G'F'$ remains horizontally straight and the removed column stub displaces vertically downward due to double symmetry. Hence, the extension of x -direction reinforcement at the slab centre F ($\Delta_{EF,R}^x (y=l)$) is caused by the rotation of segment GH , leading to the result of $l/\cos\theta - l - h_{R,b} \cdot \theta$, as shown in Fig. 7.6 (a) and (c). Similar to section I-I in Fig. 7.5 (a) and Fig. 7.6 (a), all the extensions of x -direction reinforcement in region EF can be obtained as $y/\cos\theta - y - h_{R,b} \cdot \theta$, since the length of rotational segment L_x (Fig. 7.5 (a)) varies from 0 to l along y -axis.

Given a value of θ , by setting $y/\cos\theta - y - h_{R,b} \cdot \theta = 0$, one can obtain the length of reinforcing mesh in compression ($y_{0,R}$ in Fig. 7.5 (a) and Fig. 7.8 (a)) as Eq. (7.11); by setting $y/\cos\theta - y - h_{R,b} \cdot \theta = \Delta_{lim}$, one can also calculate the length of intact reinforcing mesh as Eq. (7.12).

$$y_{0,R} = h_{R,b} \cdot \theta (1 / \cos \theta - 1) \leq l \quad (7.11)$$

$$y_{lim} = \frac{\Delta_{lim} + h_{R,b} \cdot \theta}{1 / \cos \theta - 1} \leq l \quad (7.12)$$

In summary, the extension of x -direction reinforcement in region EF ($\Delta_{EF,R}^x$) is

calculated as follows:

$$\Delta_{EF,R}^x(y) = \begin{cases} 0 & 0 \leq y \leq y_{0,R} \\ y / \cos \theta - y - h_{R,b} \cdot \theta & y_{0,R} \leq y \leq y_{lim} \end{cases} \quad (7.13)$$

(4) *Extension of y-direction reinforcement and steel decking in region FH— $\Delta_{FH,R}^y$, $\Delta_{FH,D}^y$, $x_{0,R}$, x_{lim} and $x_{0,D}$*

As shown in Fig. 7.7 (a), the removed column stub displaces vertically downward due to symmetry, so $\Delta_{FH,R}^y$ and $\Delta_{FH,D}^y$ are caused by rotation of plate *OFG*. Since the length of rotation segment between $x=l$ and $x=rl$ is equal to l (Fig. 7.5 (a)), the respective extensions of y -reinforcement ($\Delta_{FH,R}^y$ ($l \leq x \leq rl$)) and steel decking ($\Delta_{FH,D}^y$ ($l \leq x \leq rl$)) in region *FG* (Fig. 7.5 (a) and Fig. 7.7 (b)) are equal to $l/\cos\theta - l - h_{R,g} \cdot \theta$ and $l/\cos\theta - l - h_{D,g} \cdot \theta$. In comparison, the length of rotational segment (L_y in Fig. 7.5 (a)) between $x=0$ and $x=l$ varies linearly from 0 to l along x -axis, so $\Delta_{FH,R}^y$ ($0 \leq x \leq l$) and $\Delta_{FH,D}^y$ ($0 \leq x \leq l$) are equal to $x/\cos\theta - x - h_{R,g} \cdot \theta$ and $x/\cos\theta - x - h_{D,g} \cdot \theta$, respectively. It should be reminded that reinforcement may be in compression or fractured and steel decking may also be in compression, which are ineffective for calculating internal energy. Similar to region *EF*, the lengths of y -direction reinforcing mesh ($x_{0,R}$) and steel decking ($x_{0,D}$) in region *GH* in compression can be calculated by Eq. (7.14) and Eq. (7.15), respectively. Besides, the length of intact y -direction reinforcing mesh (x_{lim} in Fig. 7.8 (b)) in region *GH* can be obtained with Eq. (7.16).

$$x_{0,R} = h_{R,g} \cdot \theta \cdot (1 / \cos \theta - 1) \quad (7.14)$$

$$x_{0,D} = h_{D,g} \cdot \theta \cdot (1 / \cos \theta - 1) \quad (7.15)$$

$$x_{lim} = \frac{\Delta_{lim} + h_{R,g} \cdot \theta}{1 / \cos \theta - 1} \quad (7.16)$$

With increasing central deflection δ , stress state of y -direction reinforcing mesh in

region FH can be classified into three stages as follows:

if $x_{0,R} \geq l$, then all the y -reinforcement in FH is in compression,

$$\Delta_{FH,R}^y = 0 \quad (0 \leq x \leq rl) \quad (7.17)$$

if $x_{0,R} < l$ and $x_{lim} \geq l$, then all the y -reinforcement in FG is intact,

$$\Delta_{FH,R}^y(x) = \begin{cases} 0 & (0 \leq x \leq x_{0,R}) \\ x / \cos(\theta) - x - h_{R,g} \cdot \theta & (x_{0,R} \leq x \leq l) \\ l / \cos(\theta) - l - h_{R,g} \cdot \theta & (l \leq x \leq rl) \end{cases} \quad (7.18)$$

if $x_{0,R} < l$ and $x_{lim} < l$, then y -reinforcement in FG has fractured and the fracturing has progressed to a location in GH , such that

$$\Delta_{FH,R}^y(x) = \begin{cases} 0 & (0 \leq x \leq x_{0,R}) \\ x / \cos(\theta) - x - h_{R,g} \cdot \theta & (x_{0,R} \leq x \leq x_{lim}) \end{cases} \quad (7.19)$$

Since fracture of steel decking is not considered, stress state of the steel decking in region FH can be classified into two stages as follows:

if $x_{0,D} \geq l$, then all the steel decking in FH is in compression,

$$\Delta_{FH,D}^y = 0 \quad (0 \leq x \leq rl) \quad (7.20)$$

if $x_{0,D} < l$, then the steel decking between $x = x_{0,D}$ and $x = rl$ is in tension,

$$\Delta_{FH,D}^y(x) = \begin{cases} 0 & (0 \leq x \leq x_{0,D}) \\ x / \cos(\theta) - x - h_{D,g} \cdot \theta & (x_{0,D} \leq x \leq l) \\ l / \cos(\theta) - l - h_{D,g} \cdot \theta & (l \leq x \leq rl) \end{cases} \quad (7.21)$$

After obtaining the extensions of the steel decking and the deformed but intact reinforcing mesh, energy stored in them can be calculated (Section 7.2.3.4). Subsequently, energy stored in the joints can also be determined based on the respective deformations of the joint components at the girder and beam ends.

(5) Extensions of joint components

The extensions of all the components at both ends can be determined based on geometric compatibility as follows:

At the hogging end of girder B1 (Fig. 7.7 (c)), the extensions of joint components ($\Delta_{\text{hog,br}}$) can be computed by Eq. (7.22).

$$\Delta_{\text{hog,br}} = \pm d_{\text{br}} \theta \quad (7.22)$$

where subscripts “hog” and “br” indicate the respective hogging end and bolt row number. The term d_{br} is the distance between the br -th component and the rotation centre M . The signs “+” and “-” are used for the components above and below rotation centre M (Fig. 7.7 (c)), respectively.

As shown in Fig. 7.7 (a), B1 rotates about point M and the removed column stub only displaces vertically, so the elongation of centre line of girder B1 is $l/\cos\theta - l$. According to assumption (5), only the joint deformations are considered, while the axial elongations of girders and beams are neglected. Hence, at the sagging end of B1 (Fig. 7.7 (b)), the extensions of joint components ($\Delta_{\text{sag,br}}$) are determined by Eq. (7.23).

$$\Delta_{\text{sag,br}} = l / \cos \theta - l \pm d_{\text{br}} \theta \quad (7.23)$$

where the subscript “sag” indicates the sagging end of the girder.

According to assumption (2), the rotations of B2 and B3 are equal to θ ($\theta = \delta/l$). At the hogging ends of B2 and B3 (Fig. 7.6 (b)), the extensions of joint components can be calculated according to Eq. (7.22).

As shown in Fig. 7.6 (a), beam B2 and inner beam B3 deflect as a piece-wise bilinear profile and only the joint deformations are considered. At the sagging end of B2 (Fig. 7.6 (a)), α_1 is equal to α_2 since they are the corresponding angles of two sets of parallel lines. Besides, α_2 is equal to half of the radius angle θ due to Alternate Segment Theorem. Thus, α_1 is equal to $\theta/2$. Extensions of joint components at the sagging end of B2 ($\Delta_{\text{sag,B2}}$ in Fig. 7.6 (c)) can be determined by Eq. (7.24). Similarly, Extensions of joint components at the sagging end of B3 ($\Delta_{\text{sag,B3}}$) is calculated as Eq. (7.25), since the maximum deflection of B3 is half of that of B2 for the same rotation θ .

$$\Delta_{\text{sag,B2}} = \delta \cdot \theta / 2 \quad (7.24)$$

$$\Delta_{\text{sag,B3}} = \delta \cdot \theta / 4 \quad (7.25)$$

(6) Rotations of plastic hinges in beams and yield lines in slabs

As shown in Fig. 7.6 (a), the rotations of plastic hinges in beams and yield lines in the slabs are equal to θ ($\theta = \delta/l$).

7.2.3.4 Internal Energy

(1) Internal energy stored in reinforcement and steel decking

Internal energy stored in the steel reinforcement and decking is a function of their extensions and yield forces. The extensions in all the deformed regions have been determined in Section 7.2.3.3. The yield forces in steel decking and reinforcing mesh per unit width are F_R and F_D , respectively. Yield force in decking F_D is equal to $\psi f_{y,D} A_D$, where $f_{y,D}$ is the yield strength of steel decking, A_D is the area of bottom horizontal legs of steel decking per unit width, and ψ represents the percentage of yielded decking area. ψ is assigned with 0.45, as reported by Alashker and El-Tawil (2011).

The energy quantity stored in x-direction reinforcing mesh in region OH is $W_{OH,R}$ as shown in Eq. (7.26). For region OE , the respective energy quantities stored in the y-direction reinforcing mesh and steel decking are $W_{OE,R}$ and $W_{OE,D}$ (Eq. (7.27)). For EF and FH , energy is stored in the deformed but intact steel reinforcement and decking, as shown in Eqs. (7.28), (7.29) and (7.30).

Region OH :

$$W_{OH} = \begin{cases} F_R \cdot \Delta_{OH,R}^x \cdot l & (\Delta_{OH,R}^x \leq \Delta_{\text{lim}}) \\ 0 & (\Delta_{OH,R}^x > \Delta_{\text{lim}}) \end{cases} \quad (7.26)$$

Region OE :

$$W_{OE} = \begin{cases} W_{OE,R} + W_{OE,D} = F_R \cdot \Delta_{OE,R}^y \cdot rl + F_D \cdot \Delta_{OE,D}^y \cdot rl & (\Delta_{OE,R}^y \leq \Delta_{lim}) \\ W_{OE,D} = F_D \cdot \Delta_{OE,D}^y \cdot rl & (\Delta_{OE,R}^y > \Delta_{lim}) \end{cases} \quad (7.27)$$

Region EF :

$$\begin{aligned} W_{EF} &= \int_{y_{0,R}}^{y_{lim}} F_R \Delta_{EF,R}^x(y) dy = \int_{y_{0,R}}^{y_{lim}} F_R \cdot (y / \cos \theta - y - h_{R,b} \cdot \theta) dy \\ &= F_R (1 / \cos \theta - 1) \cdot (y_{lim}^2 - y_{0,R}^2) / 2 - F_R \cdot h_{R,b} \cdot \theta \cdot (y - y_{0,R}) \end{aligned} \quad (7.28)$$

Region FH :

If the extension of y -direction reinforcement at G ($x=l$, as shown in Fig. 7.8 (b)) does not exceed Δ_{lim} , the reinforcement in the entire segment FH remains intact. Energy stored in region FH stems from the y -direction reinforcement between $x=x_{0,R}$ and point F , as well as steel decking between $x=x_{0,D}$ and point F . Hence,

For $\Delta_{FH,R}^y (x=l) < \Delta_{lim}$:

$$\begin{aligned} W_{FH} &= \int_{x_{0,R}}^l (F_R \cdot (x / \cos \theta - x - h_{R,g} \cdot \theta) dx + \int_{x_{0,D}}^l (F_D \cdot (x / \cos \theta - x - h_{D,g} \cdot \theta) dx \\ &\quad + (F_R (l / \cos \theta - l - h_{R,g} \cdot \theta) + F_D \cdot (l / \cos \theta - l - h_{D,g} \cdot \theta)) \cdot (rl - l) \\ &= F_R \cdot (1 / \cos \theta - 1) \cdot \frac{l^2 - x_{0,R}^2}{2} + F_D \cdot (1 / \cos \theta - 1) \cdot \frac{l^2 - x_{0,D}^2}{2} \\ &\quad - F_R h_{R,g} \cdot \theta (l - x_{0,R}) - F_D h_{D,g} \cdot \theta (l - x_{0,D}) \\ &\quad + (F_R (l / \cos \theta - l - h_{R,g} \cdot \theta) + F_D \cdot (l / \cos \theta - l - h_{D,g} \cdot \theta)) \cdot (rl - l) \end{aligned} \quad (7.29)$$

However, if y -direction reinforcement extension at G exceeds Δ_{lim} , fracture of reinforcement has developed from FG to a location at HG (Fig. 7.8 (b)). Energy stored in region FH comes from reinforcement between $x=x_{0,R}$ and $x=x_{lim}$, as well as steel decking between $x=x_{0,D}$ and point F . Therefore,

For $\Delta_{FH,R}^y (x=l) > \Delta_{lim}$:

$$\begin{aligned}
 W_{FH} &= \int_{x_{0,R}}^{x_{lim}} (F_R \cdot (x / \cos \theta - x - h_{R,g} \cdot \theta) dx + \int_{x_{0,D}}^l (F_D \cdot (x / \cos \theta - x - h_{D,g} \cdot \theta)) dx \\
 &\quad + F_D \cdot (l / \cos \theta - l - h_{D,g} \cdot \theta) \cdot (rl - l) \\
 &= F_R \cdot (1 / \cos \theta - 1) \cdot \frac{x_{lim}^2 - x_{0,R}^2}{2} + F_D \cdot (1 / \cos \theta - 1) \cdot \frac{l^2 - x_{0,D}^2}{2} \\
 &\quad - F_R h_{R,g} \cdot \theta (x_{lim} - x_{0,R}) - F_D h_{D,g} \cdot \theta (l - x_{0,D}) \\
 &\quad + F_D \cdot (l / \cos \theta - l - h_{D,g} \cdot \theta) \cdot (rl - l)
 \end{aligned} \tag{7.30}$$

(2) Internal energy stored in joint components

The spring property of each component needs to be calculated based on the joint configuration. Piluso et al. (2001a), Taib (2012) and Yang and Tan (2013b) and presented their own methods to calculate the spring properties at large deformation stage for various types of connections. Alternatively, the component properties can be determined through component tests. In the worked example of this paper, the component properties were quantified by component tests (Jiang et al. 2016).

The obtained extensions (Eqs. (7.22), (7.23), (7.24) and (7.25)) combined with spring property of each component can be used to compute the energy stored in all the joint components (W_{jc}).

(3) Internal energy stored in plastic hinges

$$W_{ph} = \sum_i M_p \cdot \theta = \sum_i W_p \cdot f_{y,b} \cdot \theta \tag{7.31}$$

where M_p is the plastic bending moment resistance of secondary beam cross-section, W_p is the plastic sectional modulus, and $f_{y,b}$ is the yield strength of beams. It should be noted that the strain energy stored in a plastic hinge will stop increasing once all the joint components at one end of the corresponding beam fracture, since this beam will have a rigid rotation without plastic deformation anymore.

(4) Internal energy stored in yield lines

The deformed slab forms positive yield line OG with rotation θ ($\theta_x = \theta_y = \theta = \delta/l$). Internal energy stored in the positive yield line is written as:

$$W_{yl} = (m_{sx} + m_{sy}) \cdot l \cdot \theta \quad (7.32)$$

where m_{sx} and m_{sy} are the positive bending moment resistance of composite slabs per unit width in the respective x - and y -directions, which are given as (Park and Gamble 2000):

$$m_s = 0.9 A_s f_{yR} (d_s - 0.59 A_s f_{yR} / f_c') \quad (7.33)$$

where A_s is the tension steel area per unit width of the slab, f_{yR} is the yield strength of reinforcing steel, f_c' is the compressive cylinder strength of concrete and d_s is the effective depth of slab.

Then total internal strain energy is the sum of the respective energy stored in the reinforcement, the steel decking, joint components, plastic hinges and 45° -direction yield lines (Fig. 7.5 (a)), as given in Eq. (7.6).

7.2.3.5 External work

According to assumption (2), rigid plate $OEFG$ rotates about x -axis, so that vertical deflections of plate $OEFG$ ($V(y)$) remain constant along x -axis and linearly vary along y -axis, and vice versa for plate OGH . Therefore,

Plate $OEFG$:

$$V(y) = \delta y / l \quad (7.34)$$

Plate OGH :

$$V(x) = \delta x / l \quad (7.35)$$

External work by applied load w is:

$$\begin{aligned} W_{ex} &= \int w dA \cdot V(x, y) = 4w \left(\int_0^l dx \int_0^x \frac{\delta y}{l} dy + \int_l^r dx \int_0^l \frac{\delta y}{l} dy + \int_0^l dy \int_0^y \frac{\delta x}{l} dx \right) \\ &= 4w \left(\frac{\delta r l^2}{2} - \frac{\delta l^2}{3} + \frac{\delta l^2}{6} \right) = 4w \left(\frac{\delta r l^2}{2} - \frac{\delta l^2}{6} \right) \end{aligned} \quad (7.36)$$

Equating internal work (Eq. (7.6)) with external work (Eq. (7.36)), one can obtain the relationship between UDL (w) and slab central deflection δ .

7.2.4 Procedural description of the proposed model

The proposed model can be implemented in Matlab or Excel. As shown in Fig. 7.9, the geometric and material properties of the floor systems are given as initial input data. The bounds of the three stages are then determined, viz. $0 < \delta \leq h_e$, $h_e < \delta \leq h_g$ and $h_g < \delta \leq 2h_g$. At small deformation, the linear load-deformation relationship (w - δ) of composite slabs is obtained using thin plate theory (Eq. (7.1)). The load sustained by the steel members can be calculated using virtual work principle (Eq. (7.5)). Small deformation stage is linearly linked to large deformation stage by transitional stage. The slope of the straight line is determined by the increase in load between $\delta=h_e$ and $\delta=h_g$. With an initial value of δ ($\delta_0=h_g$) for large deformation stage, one can calculate extensions of reinforcement, steel decking (Eqs. (7.9) ~ (7.21)) and joint components (Eqs. (7.22) ~ (7.25)). It should be mentioned that reinforcement and joint components will fracture once their extensions reach the respective limits, e.g. the limiting extension of reinforcement is Δ_{lim} . After obtaining the deformations, internal energy can be calculated by Eqs. (7.26) through (7.33). With Eq. (7.36), one can calculate external work based on δ , assumed shape function and w . Equating internal energy with external work, one can express w as a function of δ . Subsequently, the failure criterion ($\delta > 2h_g$) is applied. If the failure criterion is not met, an increment $\Delta\delta$ will be added to δ and iterative calculations continue to run until the failure criterion is met. The pair (w, δ) constitutes the response for large deformation stage.

7.2.5 Model Limitations

The proposed model does not consider fracture of steel decking so that its continuing contribution is only terminated by the global failure criterion ($\delta > 2h_g$). It is very difficult to define the limiting extension of steel decking in composite slabs, which needs to be further studied based on extensive experimental tests.

If both girder-to-column and beam-to-column joints are unreasonably weak, the 3D

floor system may prematurely collapse. In such cases, the failure criterion ($\delta > 2h_g$) may need to be reconsidered. Nevertheless, this will not be a concern for normally-designed building structures, as demonstrated in Section 7.3.2, which shows that even the structure with only fin-plate joints can survive until $\delta > 2h_g$.

Fig. 7.9: The procedure to implement the proposed model

7.3 Verifications and Discussions

7.3.1 Comparisons with the tests

The authors have conducted experimental tests on four one-third scale 3D composite floors under an internal column removal scenario (Chapter 3). Four specimens were labelled as 2×3-S-IC, 2×2-S-IC, 2×3-W-IC and 2×3-S-PI (Section 3.2). Since Specimen 2×3-S-PI did not exactly match the boundary condition, only Specimens 2×3-S-IC, 2×2-S-IC and 2×3-W-IC were used to verify the proposed model, even though 2×3-W-IC has weak composite action, which is not considered in the assumptions in Section 7.2.3.2. The discrepancy between the prediction and the test result for 2×3-W-IC due to weak composite action is discussed subsequently.

According to the procedure in Fig. 7.9, the static load-displacement responses of the three specimens are calculated as shown in Fig. 7.10, where h_e and h_g are the respective equivalent thickness of composite slab and depth of composite girder including the slab. Fig. 7.10 shows that the model can predict the static response conservatively well except for 2×3-W-IC, since the proposed model is based on full composite action. However, the calculated result tends to the test result at large deformation stage and the predictions on the ultimate load and displacement are fairly well (Fig. 7.10 (b)). The reason is that, at large deformation stage, the load resistance of 2×3-W-IC was enhanced by more dominant CA in the double-span girder and double-span beam, and TMA in the slabs compared with 2×3-S-IC. Therefore, the model can be applied to cases with partial composite action. It can be seen that the prediction for 2×2-S-IC is not as good as those for 2×3-S-IC and 2×3-W-IC (Fig. 7.10). The reason is that, for specimen 2×2-S-IC, the applied multi-point loads could not simulate UDL, since the loading system was designed for 2 m × 3 m slab panels so that loading points were too close to the perimeter girders in the test of 2×2-S-IC.

(a) Specimen 2×3-S-IC

(b) Specimen 2×3-W-IC

(c) Specimen 2×2-S-IC

Fig. 7.10: Comparisons of the load-displacement responses between the tests and proposed model

7.3.2 Comparisons with FE simulations

Alashker and El-Tawil (2011) proposed a mechanical model for predicting the collapse resistance of composite floors subjected to an internal-column loss scenario (Fig. 7.11 (a)). Six simulation cases in Alashker et al. (2010) were used to verify their model. The basic model is named B3-D1-R1, where the numerical part of the first field refers to the number of bolts. The numerical parts of the second and third fields are the respective ratios of the decking thickness and the amount of reinforcement compared to the basic model. For example, B4-D2-R2 indicates a specimen with 4 rows of bolts in each joint, double the decking thickness and the amount of reinforcing mesh. The floor dimensions and steel member sections are shown in Fig. 7.11 (a) and the joint configuration is shown in Fig. 7.11 (b). Likewise, the proposed model in this paper is also verified by the six numerical studies and compared with the mechanical model proposed in Alashker and El-Tawil (2011).

Fig. 7.11: Basic information of the simulation cases (Alashker and El-Tawil 2011)

As shown in Fig. 7.12 (a)~(f), Alashker and El-Tawil (2011) could predict the numerical peak load and corresponding displacement well. However, their model could not capture the load-displacement response at the early stage, so it could not be used to calculate the energy absorption capacity. In comparison to their model, the proposed method can predict the ultimate displacement more accurately and ultimate load more conservatively. As shown in Table 7.1, the mean ratio of the proposed

model to simulation results for ultimate displacement is 0.98, while that of Alashker's model is 1.07. The respective mean ratios of the proposed model and Alashker's model to the simulation result for ultimate load are 0.80 and 0.88. The underestimation in load-carrying capacity is mainly due to different joint component models adopted in the numerical models and the proposed model. The numerical models in Alashker et al. (2010) used the force-displacement relationship, as shown in Fig. 7.13 (a), for bolt-row components, whereas the proposed model uses Taib (2012)'s method to calculate the component property for fin-plate joints (Fig. 7.13 (b)). The crucial difference is the component behaviour in compression. Alashker et al. (2010) opined that the component would not fracture in compression and the maximum resistance could be maintained. In comparison, the author adopts more accurate component model which predicts that component will experience strength softening due to bolt shearing failure. As a result, the proposed model predicts a lower load compared to the numerical result at the final stage. This difference is more evident for the models using more bolts in each connection, as shown in Fig. 7.12 (e) and Fig. 7.12 (f). Besides, in the proposed model, the load-carrying capacity at small deformation stage is not sensitive to the thickness of steel decking. Therefore, halving the decking thickness would not decrease the resistance significantly, leading to deviations between the proposed model and FE simulation results in the linear elastic stage (Fig. 7.12 (c)). However, the proposed model incorporates decking thickness at large deformation stage. Thus, the model tends to FE simulation results at large deformation stage. As a result, the model can predict the ultimate state conservatively.

After comparing the model with actual test and simulation results (Alashker and El-Tawil 2011), the authors gain confidence in the model's capability to assess the progressive collapse resistance for 3D composite floors with varying slab aspect ratios, joint type, number of bolts, slab thickness, reinforcement ratio and steel decking thickness.

* Alashker model denotes the mechanical model in Alashker and El-Tawil (2011).

(a) B3-D1-R1

(b) B3-D2-R1

(c) B3-D0.5-R1

(d) B3-D1-R0

(e) B4-D1-R1

(f) B5-D1-R1

Fig. 7.12: Comparisons of the load-displacement curves between the simulation results, proposed model and model in Alashker and El-Tawil (2011)

(a) Component property used in
(Alashker et al. 2010)

(b) Component property used in the
proposed model

Fig. 7.13: Different component properties used in numerical models and proposed model

Table 7.1: Comparisons of ultimate limit state between the simulation results and proposed model

Case ID	Ultimate load (kN/m ²)					Ultimate displacement (mm)				
	Model	A*	FE*	Model/FE	A/FE	Model	A	FE	Model/FE	A/FE
B3-D1-R1	3.92	4.40	4.93	0.80	0.89	1114	1220	1118	1.00	1.09
B3-D2-R1	5.6	6.85	6.74	0.83	1.02	1114	1220	1307	0.85	0.93
B3-D0.5-R1	3.07	3.17	4.00	0.77	0.79	1114	1220	1131	0.98	1.08
B3-D1-R0	3.87	3.39	4.29	0.90	0.79	1114	1220	1069	1.04	1.14
B4-D1-R1	4.06	4.68	5.36	0.76	0.87	1114	1220	1070	1.04	1.14
B5-D1-R1	4.20	5.20	5.76	0.73	0.90	1114	1220	1175	0.95	1.04
Mean				0.80	0.88				0.98	1.07
SD*				0.06	0.08				0.06	0.07

* A — Alashker's model (Alashker and El-Tawil 2011); FE-Finite element simulation results; SD — Standard deviation

7.4 Worked Example

To aid engineers in applying the model (Fig. 7.9) and to identify which terms in Eq. (7.6) are the most important, it is meaningful to go through a worked example. Control specimen 2×3-S-IC is chosen to illustrate the procedure.

The input data is as follows:

Slab dimensions: $rl=3000$ mm, $l=2000$ mm (Fig. 7.5 (a)). The coefficient η related to the slab aspect ratio (Eq. (7.1)) is equal to 0.0022.

Steel decking: depth $h_p=40$ mm, thickness $t=0.9$ mm, yield strength $f_{y,D}=333.5$ MPa, percentage of yielded area $\Psi=0.45$.

Reinforced concrete: $\varnothing 6$ reinforcing mesh at 100 mm spacing in both directions, yield strength $f_{y,R}=340.3$ MPa and ultimate strain $\varepsilon_u=0.23$. Reinforcing mesh was directly placed on the steel decking, with a depth of concrete topping $h_c=25$ mm, and compressive cube strength of concrete ($f_{ck,cube}$) is 54 MPa so compressive cylinder strength f'_c is approximately 44 MPa.

Frame members: H-section (H 200×100×5.5×8) is used for girders with yield strength of 378.9 MPa, and secondary beams use H-section 150×100×5×7 with yield strength of 283.7 MPa.

Determination of three stages:

$$h_e = h_c + h_p/2 = 45 \text{ mm}, h_g = 200 + 40 + 25 = 265 \text{ mm}$$

Small deformation stage: $0 \leq \delta \leq 45$ (Linear elastic)

Transitional stage: $45 \text{ mm} \leq \delta \leq 265 \text{ mm}$

Large deformation stage: $265 \text{ mm} \leq \delta \leq 530 \text{ mm}$ (Plastic)

7.4.1 Small deformation stage

As the structure is assumed to be linear elastic at small deformation stage, the load at the end of this stage ($\delta=45$ mm) can be determined through Eqs. (7.1) and (7.5).

7.4.1.1 Load contribution of composite slabs

To calculate the UDL carried by the slab (w_{slab}), flexural rigidity of the slab (D) has to be determined based on an equivalent cross-section of the slab (Fig. 7.2 (a)).

$$I_c = \frac{1}{12} \times 1 \times (45\text{mm})^3 = 7594\text{mm}^3$$

$$I_s = 1 \times 0.9\text{mm} \times (22.5\text{mm})^2 + 283\text{mm}^2 / 1000\text{mm} \times (22.5\text{mm} - 16\text{mm})^2 = 468\text{mm}^3$$

$$n = \frac{E_s}{E_c} = \frac{2.06 \times 10^5}{3.45 \times 10^4} = 6$$

$$D = 3.45 \times 10^4 \times (7594 + (6-1) \times 468) = 3.42 \times 10^8 \text{ N} \cdot \text{mm} \quad (\text{Eq. (7.4)})$$

$$w_{\text{slab}} = \frac{\delta D}{0.0022a^4} = \frac{45\text{mm} \times 3.42 \times 10^8 \text{ N} \cdot \text{mm}}{0.0022 \times (4000\text{mm})^4} = 0.0273 \text{ N/mm}^2 = 27.3 \text{ kN/m}^2 \quad (\text{Eq. (7.1)})$$

7.4.1.2 Load contribution of steel sub-frame

(1) Strain energy stored in sub-frame

The properties of joint components in this worked example were obtained from the joint component tests performed by Jiang et al. (2016) (Appendix). For bolted flush-end-plate joints (Fig. 7.4), the distances between the respective bolt rows to the rotation centre are known ($h_1=146$ mm, $h_2=96$ mm) and bolt-row forces F_1 and F_2 are obtained as follows:

$$F_1 = -0.0078\Delta S_1^4 + 0.3824\Delta S_1^3 - 6.4745\Delta S_1^2 + 46.426\Delta S_1 \text{ kN} \quad (0 \leq \Delta S_1 \leq 25 \text{ mm})$$

$$F_2 = \begin{cases} 248.52\Delta S_2 \text{ kN} & (0 \leq \Delta S_2 \leq 0.12 \text{ mm}) \\ 9.59\Delta S_2 + 29.29 \text{ kN} & (0.12 < \Delta S_2 \leq 1.64 \text{ mm}) \\ 1.08\Delta S_2 + 43.25 \text{ kN} & (1.64 < \Delta S_2 \leq 12.15 \text{ mm}) \\ -2.67\Delta S_2 + 88.79 \text{ kN} & (12.15 < \Delta S_2 \leq 33.25 \text{ mm}) \end{cases}$$

where ΔS_1 and ΔS_2 are the respective extensions of two bolt-row components in the tension zone (Fig. 7.4).

At the end of small deformation stage ($\delta=45$ mm), the rotation of girder-to-column joints is 0.0225. Hence, the extensions of end- and middle-row components (Fig. 7.4) are equal to 3.28 mm ($\Delta S_1=0.0225 \times 146$ mm) and 2.16 mm ($\Delta S_2=0.0225 \times 96$ mm), respectively. The energy stored in each component can be calculated by integrating the F_r - ΔS_r curve upon the corresponding component extension (ΔS_r). Consequently, the respective energy stored in the end- and middle-row components are equal to 184.47×10^3 N·mm and 83.9×10^3 N·mm. The internal energy stored in the sub-frame is calculated as follows:

$$W_{in,frame} = 4 \times (83.9 + 184.47) \times 10^3 = 1.07 \times 10^6 \text{ N} \cdot \text{mm}$$

(2) External work done by the applied load

With the known δ (45 mm), deflections at all the loading points 1~12 (Fig. 7.3) can be determined. Besides, the magnitude of each point load can be expressed in terms of UDL on the slab sustained by the sub-frame (w_{frame}) based on tributary area, as shown in Fig. 7.3 and Table 7.2.

External work:

$$\begin{aligned} W_{ex,frame} = \sum_i P_i \cdot v_i &= (4 \times 2.25 w_{frame} \cdot 5.625 + 4 \times 2.25 w_{frame} \cdot 16.875 + 2 \times 1.5 w_{frame} \cdot 11.25 \\ &+ 2 \times 1.5 w_{frame} \cdot 33.75) \times 10^6 = 337.5 \times 10^6 w_{frame} \text{ (N} \cdot \text{mm)} \end{aligned}$$

$$W_{ex,frame} = 337.5 \times 10^6 w_{frame} = W_{in,frame} = 1.07 \times 10^6$$

$$w_{frame} = 0.0032 \text{ N/mm}^2 = 3.2 \text{ kN/mm}^2$$

UDL value w at the end of small deformation stage is given by:

$$w = w_{frame} + w_{slab} = 3.2 + 27.3 = 30.5 \text{ kN/m}^2$$

From the result, it can be seen that flexural capacity of the slab is much greater than that of the sub-frame, since the slab flexural rigidity is very high before slab severely cracks compared to that of finite moment-resisting joints. That is to say, the slab is the major flexural contributor at small deformation stage, so that it is acceptable to

neglect the influence of steel beams on slab behaviour at this stage.

Table 7.2: Deflections and loads at the loading points

No.	1	2	3	4	5	6
Deflection (mm)/Load (10^6N)	5.625/2.25 w_{frame}	11.25/1.5 w_{frame}	5.625/2.25 w_{frame}	16.875/2.25 w_{frame}	33.75/1.5 w_{frame}	16.875/2.25 w_{frame}
No.	7	8	9	10	11	12
Deflection (mm) /Load (10^6N)	16.875/2.25 w_{frame}	33.75/1.5 w_{frame}	16.875/2.25 w_{frame}	5.625/2.25 w_{frame}	11.25/1.5 w_{frame}	5.625/2.25 w_{frame}

Table 7.3: Extensions of reinforcement and steel decking at deformed regions

OH	OE		EF	FH		
Reinforcement $\Delta_{OH,R}^x$ (Eq. (7.9))	Reinforcement $\Delta_{OE,R}^y$ (Eq. (7.10))	Decking $\Delta_{OE,D}^y$ (Eq. (7.10))	Reinforcement $\Delta_{EF,R}^x$ (Eqs. (7.11)~(7.13))	Reinforcement $\Delta_{FH,R}^y$ (Eqs. (7.14) and (7.16)~(7.19))	Decking $\Delta_{FH,D}^y$ (Eqs. (7.15), (7.20) and (7.21))	
$0 \leq y < 2000$	$0 \leq x \leq 3000$	$0 \leq x \leq 3000$	$0 \leq y < 1786$ $1786 \leq y < 2000$	$0 \leq x \leq 3000$	$0 \leq x \leq 1588$ $1588 \leq x \leq 2000$ $2000 \leq x \leq 3000$	
15.6 mm	19.7 mm	13.2 mm	0 $y/\cos(0.132)-y-15.6$ (mm)	0	0 $x/\cos(0.132)-x-13.2$ (mm)	4.4 mm

7.4.2 Large deformation stage

Assigning an initial deflection ($\delta_0=h_g=265$ mm) in the large deformation stage (Fig. 7.9), the model will run in loops until the failure criterion ($\delta>2h_g$) is met. Due to page limitation, only the first loop will be presented here.

7.4.2.1 Extensions of reinforcement, steel decking and joint components

(1) Extensions of reinforcing mesh and steel decking (Eqs. (7.9) to (7.21))

According to the given geometric information, the terms $\theta=265/2000=0.132$, $h_{R,g}=149$ mm, $h_{D,g}=100$ mm, $h_{R,b}=118$ mm, $\Delta_{lim}=100$ mm $\times$ 0.227=22.7 mm. Extensions of reinforcing mesh and steel decking concentrated at deformed regions OH , OE , EF and FH are included in Table 7.3. Specifically,

Region OH ($0\leq y<2000$):

$$\Delta_{OH,R}^x = h_{R,b} \cdot \theta = 118 \times 0.132 = 15.6 \text{ mm} < 22.7 \text{ mm} \quad (\text{Eq. (7.9)})$$

Region OE ($0\leq x\leq 3000$):

Reinforcement:

$$\Delta_{OE,R}^y = h_{R,g} \cdot \theta = 149 \times 0.132 = 19.7 \text{ mm} < 22.7 \text{ mm} \quad (\text{Eq. (7.10)})$$

Steel decking:

$$\Delta_{OE,D}^y = h_{D,g} \cdot \theta = 100 \times 0.132 = 13.2 \text{ mm} \quad (\text{Eq. (7.10)})$$

Region EF :

$$y_{0,R} = \frac{118 \times 0.132}{1/\cos(0.132)-1} = 1768 \text{ mm} \quad (\text{Eq. (7.11)})$$

$$y_{lim} = \frac{22.7 + 15.6}{1/\cos(0.132)-1} = 4364 > 2000, \text{ then } y_{lim}=2000 \text{ mm} \quad (\text{Eq. (7.12)})$$

$$\Delta_{EF,R}^x(y) = \begin{cases} 0 & 0 \leq y \leq 1768 \text{ mm} \\ y/\cos 0.132 - y - 15.6 & 1768 \leq y \leq 2000 \text{ mm} \end{cases} \quad (\text{Eq. (7.13)})$$

Region *FH*:

Reinforcement:

$$x_{0,R} = 149 \times 0.132 \times (1/0.132 - 1) = 2241 > 2000 \quad (\text{Eq. (7.14)})$$

$$\Delta_{FH,R}^y = 0 \quad (0 \leq x \leq 3000 \text{ mm}) \quad (\text{Eq. (7.17)})$$

Steel decking:

$$x_{0,D} = \frac{100 \times 0.132}{1/\cos(0.132) - 1} = 1588 < 2000 \text{ mm} \quad (\text{Eq. (7.15)})$$

$$\Delta_{FH,D}^y(x) = \begin{cases} 0 & (0 \leq x \leq 1588 \text{ mm}) \\ x/\cos(0.132) - x - 13.2 & (1588 \leq x \leq 2000 \text{ mm}) \\ 2000(1/\cos(0.132) - 1) - 13.2 = 4.4 & (2000 \leq x \leq 3000 \text{ mm}) \end{cases} \quad (\text{Eq. 7.21})$$

(2) Extensions of joint components

All the components in girder B1, beam B2 and inner beam B3 are shown in Fig. 7.14 (a), (b) and (c), respectively. It should be mentioned that at the hogging end of girder B1 (Fig. 7.14 (a)), the component corresponding to the bottom bolt row is neglected since it is in compression zone where compression is mostly sustained by the girder flange. The spring properties (F_r - ΔS_r) of all the joint components (*C1*~*C11*) have been obtained by the authors through component tests (Jiang et al. 2016). Extensions of components *C1*~*C11* are included in Table 7.4. The detailed calculations are shown below:

(a) Components at the hogging end of girder B1— *C1* and *C2* (Fig. 7.14 (a))

Top row— *C1*:

The terms $\theta=0.132$, $d_1=50$ mm, the limit extension is 25.0 mm.

$$\Delta_{\text{hog},1} = d_1 \cdot \theta = 50 \times 0.132 = 6.6 \text{ mm} < 25.0 \text{ mm} \quad (\text{Eq. (7.22)})$$

Bottom-flange component— *C2*:

The term $d_2=-100$ mm, the limit contraction is 151.5 mm.

$$\Delta_{\text{hog},2} = -100 \times 0.132 = -13.2 \text{ mm} \quad (13.2 < 151.5 \text{ mm}) \quad (\text{Eq. (7.22)})$$

(b) Components at the sagging end of girder B1— C3, C4 and C5 (Fig. 7.14 (a))

Bottom row— C3:

The term $d_1=50$ mm, the limit extension is 25.0 mm.

$$\begin{aligned} \Delta_{\text{sag},1} &= l_y / \cos \theta - l_y + d_1 \theta \\ &= 2000 / \cos(0.132) - 2000 + 50 \times 0.132 = 24.3 \text{ mm} < 25.0 \text{ mm} \end{aligned} \quad (\text{Eq. (7.23)})$$

Middle row— C4:

The term $d_1=0$, the limit extension is 33.3mm.

$$\Delta_{\text{sag},2} = 2000 / \cos(0.132) - 2000 = 17.5 \text{ mm} < 33.3 \text{ mm} \quad (\text{Eq. (7.23)})$$

Top row— C5:

The term $d_3=50$ mm, the limit extension is 25.0 mm.

$$\begin{aligned} \Delta_{\text{sag},3} &= l_y / \cos \theta - l_y - d_3 \theta \\ &= 2000 / \cos(0.132) - 2000 - 50 \times 0.132 = 11.1 \text{ mm} < 25.0 \text{ mm} \end{aligned} \quad (\text{Eq. (7.23)})$$

(c) Components at the hogging end of beam B2— C6 and C7 (Fig. 7.14 (b))

Top row— C6:

The term $d_1=25$ mm, the limit extension is 56.6 mm.

$$\Delta_{\text{hog},1} = d_1 \cdot \theta = 25 \times 0.132 = 3.3 \text{ mm} < 56.6 \text{ mm}$$

Bottom row— C7:

The term $d_2=-25$ mm, the limit contraction is 15.3 mm.

$$\Delta_{\text{hog},2} = d_2 \cdot \theta = -25 \times 0.132 = -3.3 \quad (3.3 < 15.3 \text{ mm}) \quad (\text{Eq. (7.22)})$$

(d) Components at the sagging end of B2— C8 (Fig. 7.14 (b))

$$\Delta_{\text{sag},B2} = \delta \cdot \theta / 2 = 265 \times 0.132 / 2 = 17.4 \text{ mm} < 56.6 \text{ mm} \quad (\text{Eq. (7.24)})$$

(e) Components at the hogging end of inner beam B3— C9 and C10 (Fig. 7.14 (c))

Top row— C9:

The term $d_1=25$ mm, the limit extension is 56.6 mm.

$$\Delta_{\text{hog},1} = d_1 \cdot \theta = 25 \times 0.132 = 3.3 \text{ mm} < 56.6 \text{ mm} \quad (\text{Eq. (7.22)})$$

Bottom row—*C10*:

The term $d_2=-25$ mm, the limit contraction is 15.3 mm.

$$\Delta_{\text{hog},2} = d_2 \cdot \theta = -25 \times 0.132 = -3.3 \quad (3.3 < 15.3 \text{ mm}) \quad (\text{Eq. (7.22)})$$

(f) Components at the sagging end of B3— *C11* (Fig. 7.14 (c))

$$\Delta_{\text{sag},B3} = \delta \cdot \theta / 4 = 265 \times 0.132 / 4 = 8.7 \text{ mm} < 56.6 \text{ mm}$$

Table 7.4: Extensions of joint components

$C1$ (mm)	$C2$ (mm)	$C3$ (mm)	$C4$ (mm)	$C5$ (mm)	$C6$ (mm)	$C7$ (mm)	$C8$ (mm)	$C9$ (mm)	$C10$ (mm)	$C11$ (mm)
Eq. (22)		Eq. (23)			Eq. (22)		Eq. (24)	Eq. (22)		Eq. (25)
6.9	-13.2	24.3	17.5	11.1	3.3	-3.3	17.4	3.3	-3.3	8.7

Table 7.5: Energy stored in deformed regions of composite slabs

Deformed region	OH (N·m)	OE (N·m)	EF (N·m)	FH (N·m)
Equation used	Eq. (26)	Eq. (27)	Eq. (28)	Eq. (29)
Energy stored	2707.6	10688.6	22.9	1299.7
Quantity	4	4	4	4
Percentage of total energy*	9.06%	35.78%	0.08%	4.35%

* Total energy indicates energy stored in the entire floor, so the sum of percentages of OH , OE , EF and FH is not equal to 100%.

Table 7.6: Energy stored in joint components

Component No.	$C1$ (N·m)	$C2$ (N·m)	$C3$ (N·m)	$C4$ (N·m)	$C5$ (N·m)	$C6$ (N·m)	$C7$ (N·m)	$C8$ (N·m)	$C9$ (N·m)	$C10$ (N·m)	$C11$ (N·m)
Energy stored	569.2	5386.9	2801.7	851.8	1129.2	120.2	126.2	1066.6	120.2	126.2	420.2
Quantity of component	2	2	2	2	2	2	2	4	4	4	8
Percentage of total energy*	0.95%	9.02%	4.69%	1.43%	1.89%	0.20%	0.21%	3.57%	0.40%	0.42%	2.81%

Fig. 7.14: Numbering and geometric information of joint components

7.4.2.2 Internal energy

(1) Internal energy stored in the reinforcing mesh and steel decking at deformed regions OH, OE, EF and FH (Table 7.5)

Region OH (Eq. (7.26)):

$$F_R = \frac{\pi \cdot d^2 \cdot f_y}{4 \cdot d_{sp}} = \frac{\pi \cdot 6^2 \times 340.3}{4 \times 100} = 96.2 \text{ N} \cdot \text{mm}$$

$$W_{OH} = 96.2 \times 15.6 \times 1800 = 2707638 \text{ N} \cdot \text{mm}$$

Region OE (Eq. (7.27)):

$$F_D = \Psi \cdot f_{y,d} \cdot A_D = 0.45 \times 333.5 \times 0.9 = 135.1 \text{ N} \cdot \text{mm}$$

$$W_{OE} = W_{OE,R} + W_{OE,D} = 96.2 \times 19.7 \times 2800 + 135.1 \times 13.2 \times 2800 = 10688618 \text{ N} \cdot \text{mm}$$

Region along EF (Eq. (7.28)):

$$\begin{aligned} W_{EF} &= 96.2 \times \left(\frac{1}{\cos(0.132)} - 1 \right) \cdot \frac{2000^2 - 1768^2}{2} - \frac{96.2 \times 118 \times 265 \times (2000 - 1768)}{2000} \\ &= 22875 \text{ N} \cdot \text{mm} \end{aligned}$$

Region along FH (Eq. (7.29)):

$$W_{FH,R} = 0$$

$$\begin{aligned} W_{FH,D} &= 135.1 \times \left(\frac{1}{\cos(0.132)} - 1 \right) \cdot \frac{2000^2 - 1588^2}{2} - \frac{135.1 \times 100 \times 265}{2000} \cdot (2000 - 1588) \\ &\quad + 135.1 \cdot \left(\frac{2000}{\cos(0.132)} - 2000 - \frac{100 \times 265}{2000} \right) \times (3000 - 2000) = 1299746 \text{ N} \cdot \text{mm} \end{aligned}$$

(2) Internal energy stored in joint components

The spring properties ($F_r - \Delta S_r$) of all the joint components ($C1 \sim C11$) have been obtained in Appendix. Internal energy stored in all the connection components is shown in Table 7.6, obtained by integrating $F_r - \Delta S_r$ curves of joint components upon on the respective extensions (ΔS_r). An example of calculating internal energy stored in $C6$ is shown as follows:

The force-displacement relationship of $C6$ refer to Eq. (A.1).

$$\begin{aligned} W_{C6} &= 0.5 \times 1.23 \times 1.23 \times 34.18 + 0.5 \times (1.23 \times 34.18 + 2.54 \times 3.3 + 38.84) \times (3.3 - 1.23) \\ &= 118.2 \text{ N} \cdot \text{m} \end{aligned}$$

The calculations for the other components are not repeated.

Internal energy in all deformed joints:

$$W_{jc} = 2 \sum_{i=1}^5 W_{Ci} + 2(W_{C6} + W_{C7} + 2W_{C8}) + 4(W_{C9} + W_{C10} + 2W_{C11})$$

$$= 30580 \text{ N} \cdot \text{mm} = 30.6 \text{ kN} \cdot \text{m}$$

As shown in Fig. 7.15, among all the joints, the ones at the hogging ends of the double-span girder store the most energy (10.0% of the total energy), followed by the sagging ends of the double-span girder (8.0%), the sagging ends of the double-span beam (3.6%), and the sagging ends of the double-span inner beams (2.8%). The energy quantities stored in the simple joints at the hogging ends of the double-span beam and inner beams are marginal.

(3) *Internal energy stored in plastic hinges (Eq. (7.31))*

Plastic hinges formed in beams and inner beams. $W_y = 88800 \text{ mm}^3$, $f_{yb} = 283.7 \text{ kN/mm}^2$,

$$W_{ph} = \sum_i W_y f_{y,b} \theta = 6 \times 88800 \times 283.7 \times 0.132 = 19912098 \text{ N} \cdot \text{mm} = 19.9 \text{ kN} \cdot \text{m}$$

(4) *Internal energy in yield lines (Eqs. (7.32) and (7.33)):*

$$m_{sx} = 0.9 \times 135.1 (65 - 0.59 \times 135.1 / 44) = 7683 \text{ N} \cdot \text{mm/mm} = 7.7 \text{ kN} \cdot \text{m/m}$$

$$m_{sy} = 0.9 \times 96.2 (22 - 0.59 \times 96.2 / 44) = 1814 \text{ N} \cdot \text{mm/mm} = 1.8 \text{ kN} \cdot \text{m/m}$$

$$W_{yl} = (7724 + 1814) \times 2000 \times 0.132 = 2527519 \text{ N} \cdot \text{mm} = 2.5 \text{ kN} \cdot \text{m/m}$$

Total internal work (Eq. (7.6)):

$$W_{in} = 4(10688.6 + 22.9 + 2707.6 + 1299.7 + 2527.5) + 30584 + 19912.1$$

$$= 119481.3 \text{ N} \cdot \text{m} = 119.5 \text{ kN} \cdot \text{m/m}$$

After each term of energy is quantified, the most significant (region *OE*) and trivial (region *EF*) contributors are identified at this deflection ($\delta = 265 \text{ mm}$) for this sub-structure. As shown in Fig. 7.15, all the terms in Eq. (7.6) are sorted in descending importance as region *OE*, joints at ends of double-span girder, beam and inner beams, plastic hinges in beams, region *OH*, 45°-direction yield lines, region *FH* and region *EF*, accounting for 35.8%, 25.6%, 16.4%, 9.1%, 8.5%, 4.4% and 0.1% of total

contribution, respectively. This is reasonable since region OE is the support of the composite slab in short-span direction, which is consistent with the rib direction of the steel decking. Besides, regions OE and OH are always subjected to hogging moments so they are mobilised from an early stage, whereas regions EF and FH are initially subjected to sagging moments then changed into tension. The contribution of region FH will catch up with those of regions OE and OH with increasing deflection. In contrast, region EF is the support of the composite slab in long-span direction and does not incorporate contribution of steel decking. Consequently, the contribution of region EF is always marginal, which can be omitted for simplification.

* 35.8% denotes the percentage of energy stored region OE .

Fig. 7.15 : Energy in individual structural components

7.4.2.3 External work

$$W_{ex} = 4w \left(\frac{265 \times 2000 \times 3000}{2} - \frac{265 \times 2000^2}{6} \right) = 2.47 \times 10^9 \text{ mm}^3 \cdot w \quad (\text{Eq. (7.36)})$$

From the result, it can be seen that flexural capacity of the slab is much greater than

that of the sub-frame, since the slab flexural rigidity is very high before slab severely cracks compared to that of finite moment-resisting joints. That is to say, the slab is the major flexural contributor at small deformation stage, so that it is acceptable to neglect the influence of steel beams on slab behaviour at this stage.

7.4.2.4 Uniform load w

$$W_{\text{ex}} = 2.47 \times 10^9 \text{ mm}^3 \cdot w = W_{\text{in}} = 119481300 \text{ N} \cdot \text{mm}$$

$$w = 0.0484 \text{ N/mm}^2 = 48.4 \text{ kN/m}^2$$

If $\delta < 2h_g$, an increment $\Delta\delta$ will be added to δ and the calculations return to the starting point of the loop. This procedure is repeated until $\delta > 2h_g$. All the pairs of (w, δ) constitute the response at large deformation stage, as shown in Fig. 7.10 (a).

7.4.3 The entire response

The load-versus-deflection pairs (w_i, δ_i) at the end of small deformation stage and the commencement of large deformation stage are calculated as $(30.7 \text{ kN/m}^2, 45 \text{ mm})$ and $(48.4 \text{ kN/m}^2, 265 \text{ mm})$, respectively. These two points define the transitional stage as shown in Fig. 7.10 (a).

7.5 Case Studies

In view of the proposed model's capability to capture effects of the governing parameters such as joint type and slab aspect ratio, in this section, further studies are conducted to find out the type of joints which can be used for girders and beams to improve robustness of structures. Afterwards, several combinations of joints are recommended for practice. Based on a recommended combination of joints, the effect of slab aspect ratio is studied. Besides, the mechanical model can be used to quantify individual contributions of structural components. Against this backdrop, the effects of aspect ratio and joint type are discussed.

7.5.1 Robustness of steel structures with various joint combinations

7.5.1.1 Joint design

In addition to flush-end-plate (FEP) and double web-cleat (WC) connections chosen

for the experimental programme, extended-end-plate (EEP) and fin-plate (FP) connections are also commonly used for moment-resisting and simple joints, respectively. Therefore, the following combinations of joints are studied, as included in Table 7.7. For ease of reference, these combination labels (the first column in Table 7.7) are used to represent different sub-structures. For instance, FEP-WC indicates the 3D sub-structure with flush-end-plate joint for girders and web-cleat joint for secondary beams. It should be mentioned that FEP-WC (control) is exactly the same as control specimen 2×3-S-IC including material and geometric information for all the structural members (Section 3.2). For the other seven sub-structures, the only changes are the respective combinations of different joints. Specifically, extended-end-plate and fin-plate connections are developed from the respective flush-end-plate and double web-cleat connections based on the same connecting members and similar dimensions, as shown in Fig. 7.16 and Fig. 7.17.

Fig. 7.16: Moment-resisting joints for girder-to-column connections

Fig. 7.17: Simple joints for secondary beams

Table 7.7: Joint combinations for sub-structures

Combination reference	Joint location	
	Girder-to-column	Beam-to-column & Inner beam-to-girder
FEP-WC (control)	flush-end-plate	double web-cleat
EEP-WC	extended-end-plate	double web-cleat
EEP-FP	extended-end-plate	fin-plate
FEP-PF	flush-end-plate	fin-plate
WC-WC	double web-cleat	double web-cleat
WC-FP	double web-cleat	fin-plate
FP-FP	fin-plate	fin-plate
WC-FEP	double web-cleat	flush-end-plate

7.5.1.2 Determination of joint component properties

To apply the proposed model for these sub-structures, necessary information is known except for force-displacement relationships of joint components in extended-end-plate and fin-plate connections. As recommended in EC3-1-8 (2005), the extended part and the portion between the beam flanges can be separately modelled as equivalent T-stubs for an extended-end-plate connection (Fig. 7.18). The properties

of components between the beam flanges have been quantified in Appendix (Section A.4.2), while that of the outer component (T-stub 4 or T-stub 5 in Fig. 7.19) is calculated using Piluso et al. (2001a)'s model. With this method, it is found that the failure mode of T-stub 4 or T-stub 5 is governed by the failure of T-stub flanges combined with bolt yielding, which is the same as that of T-stub 1, 2 and 3 as observed in the test (Fig. 4.6 (c)). Finally, the force-deformation (F - ΔS) relationship of T-stub 4 and 5 is shown in Fig. 7.20 and explicitly expressed in the function below:

$$F = \begin{cases} 144.8 + 1.73\Delta S \text{ (kN)} & (0.812 < \Delta S \leq 18.452 \text{ mm}) \\ 121.46 + 54.12\Delta S \text{ (kN)} & (0.381 < \Delta S \leq 0.812 \text{ mm}) \\ 81.32 + 270.95\Delta S \text{ (kN)} & (0.233 < \Delta S \leq 0.381 \text{ mm}) \\ 349 \Delta S \text{ (kN)} & (0 \leq \Delta S \leq 0.233 \text{ mm}) \end{cases}$$

Fig. 7.18: Modelling an extended-end-plate as separate T-stubs (EC3-1-8 2005)

Fig. 7.19: Individual components in the extended-end-plate connection

Fig. 7.20: Force-deformation relationship of outer components (T-stub 4 and 5) in extended-end-plate connection

Fig. 7.21: Individual components in fin-plate connection

For the fin-plate connection (Fig. 7.17 (b)), the two bolt-row components share the same properties which can be determined by Taib (2012) as in Fig. 7.22. It is the bolt shear failure that governs the failure of the components.

Fig. 7.22: Force-deformation relationship of components in fin-plate connection

For ease of application, the curve in Fig. 7.22 can be simplified as linear piece-wise functions as follows:

$$F = \begin{cases} 69.5 - 6.1\Delta S \text{ (kN)} & (4.53 < \Delta S \leq 16 \text{ mm}) \\ 48.9 + 8\Delta S \text{ (kN)} & (1.96 < \Delta S \leq 4.53 \text{ mm}) \\ 50.9\Delta S \text{ (kN)} & (1 < \Delta S \leq 1.96 \text{ mm}) \\ 0 & (-1 \leq \Delta S \leq 1 \text{ mm}) \\ 48.6\Delta S \text{ (kN)} & (-2.03 \leq \Delta S < 1 \text{ mm}) \\ -50.1 + 7.8\Delta S \text{ (kN)} & (-4.53 \leq \Delta S < 2.03 \text{ mm}) \\ -69.5 - 6.1\Delta S \text{ (kN)} & (-16 \leq \Delta S < -4.53 \text{ mm}) \end{cases}$$

With all the above information, it is ready to start the procedure (Fig. 7.9). The results are discussed in subsequent sections.

7.5.1.3 Load-displacement curves of sub-structures with different joint combinations

As expected, EEP-WC has the greatest load-carrying capacity while FP-FP the lowest one (Fig. 7.23 and Table 7.8). With the load-displacement curves (Fig. 7.23), the energy absorption capacity (EAC) of each substructure can be obtained through

integration as included in Table 7.8. The figures in Table 7.8 show that EAC of EEP-WC is 36.1% greater than that of FP-FP. Although FEP-WC and EEP-FP have similar EAC, 514.7 kN·m versus 518.6 kN·m, FEP-WC seems more robust than EEP-FP. As shown in Fig. 7.24, FEP-WC can remain stable and sustain loads at large deformation stage although failures of some structural components cause fluctuations of the load-displacement curve. In comparison, the load-displacement curve of EEP-FP continues to descend at large deformation stage, since the fin-plate connections cannot stabilise the sub-structure after failure of most components in the extended-end-plate connections. The reasons are further discussed as follows:

The double-span girder with extended-end-plate connections is mobilised right from the beginning and fail at an early stage, so that the extended-end-plate joints mainly contribute at small and transitional stage rather than large deformation stage. As demonstrated in Fig. 7.25, the three sub-structures EEP-WC, FEP-WC and WC-WC share the same connection (WC) for secondary beams but differ in moment-resisting capacity for girder-to-column connections. The differences in load-carrying capacity before large deformation stage are significant but they gradually diminish thereafter. However, if the connection for secondary beams has limited strength and ductility (such as FP), the residual components in the girder-to-column connections can still contribute moderately at large deformation, as shown in Fig. 7.26. In this case, a difference in load-carrying capacity between FP-FP and the other three sub-structures can be observed up to the end.

Interestingly, if simple WC is used for girder-to-column connections, sub-structure WC-FP (Fig. 7.26) can even maintain the same load as that of EEP-FP and FEP-FP at large deformation stage. For the comparison among WC-WC, EEP-WC and FEP-WC (Fig. 7.25), it is also observed that the three sub-structures with the same joint type used in secondary beams possess the same load-carrying capacity at large deformation stage. Therefore, double web-cleat joint is an alternative choice for girder-to-column connections.

From the above discussions on joint selection, it can be concluded that moment-resisting joints in girder-to-column connections govern structural resistance at small and transitional stage, while simple joints for girders or beams govern the behaviour

at large deformation stage. In conventional design (moment-resisting joint for girders and simple joint for secondary beams), strength and ductility of simple joints must be ensured to guard against progressive collapse, since they are the major players at large deformation stage. Otherwise, slabs cannot withstand the vertical loads after failure of all the joints. Alternatively, structures can be designed with simple WC joints for all connections, sacrificing some flexural rigidity and load-carrying capacities at small deformation stage. In view of the energy absorption capacities (Table 7.8) and the performance at large deformation stage (Fig. 7.23 through Fig. 7.26), combinations EEP-WC and FEP-WC are recommended. Combination WC-WC can also be used when no seismic resistance is required and criterion of serviceability deflection is met. Among them, FEP-WC and WC-WC are efficient choices considering fabrication and construction.

Out of curiosity, robustness of sub-structure WC-FEP is also assessed. Specifically, WC is used for girders (y-direction beam in Fig. 7.5 (a)) and inner beams, while FEP for beam-to-column connection. The configurations for FEP and WC connections are shown in Fig. 7.16 (a) and Fig. 7.17 (a), respectively. Fig. 7.27 shows that the load-carrying capacity of WC-FEP is greater than that of FEP-WC and even slightly greater than that of EEP-WC at the beginning of large deformation stage. At the final stage, the load-carrying capacities of these three sub-structures tend to be a similar value. As a result, EAC of WC-FEP (539.6 kN·m) is between those of FEP-WC (514.7 kN·m) and EEP-WC (558 kN·m), as shown in Table 7.8. Hence, it is feasible to select moment-resisting joints for longer-span “beam-to-column” connections and simple WC joints for shorter-span “girder-to-column” connections.

Fig. 7.23: Load-displacement curves of all the sub-structures

Fig. 7.24: Comparison between EEP-FP and FEP-WC

Fig. 7.25: Comparison among the structurers with the same connection (WC) in secondary beams

Fig. 7.26: Comparison among the structurers with the same connection (FP) in secondary beams

Fig. 7.27: Comparison between EEP-FP, FEP-WC and WC-FEP

Table 7.8: Energy absorption capacities of sub-structures with different joint combinations

Energy absorption capacity (kN·m)							
EEP-WC	FEP-WC	EEP-FP	FEP-FP	WC-WC	WC-FP	FP-FP	WC-FEP
558	514.7	518.6	484.2	476	447.3	409.9	539.6

7.5.1.4 Individual contributions in sub-structures with different joint combinations

With the proposed model's capability, contributions from major load-resisting contributors can be separately ascertained. Fig. 7.28 shows the individual contributions of composite slabs, the double-span girder and the double-span

secondary beams (including beams and inner beams) at large deformation stage when the central deflection ranges from one depth of composite girder (h_g) to twice h_g .

(a) EEP-WC

(b) FEP-WC

(c) EEP-FP

(d) FEP-FP

(e) WC-WC

(f) WC-FP

Fig. 7.28: Individual contributions from composite slabs, girders and secondary beams

From Fig. 7.28, there are two findings. Firstly, the slab contribution (TMA) is more dominant in the sub-structure with weak joint combination (say, FP-FP), since such simple joints are more vulnerable and give full play to slabs. However, the overall resistance of sub-structures cannot compete with the ones with robust joints EEP-WC and FEP-WC (Fig. 7.23). Secondly, it can be concluded that secondary beams make a greater contribution than girders at large deformation stage. The difference in contribution between girders and secondary beams for EEP-FP (Fig. 7.28 (c)) is minimal since the extended-end-plate joint for girder is extremely robust while the fin-plate joint for secondary beams is too weak. The maximum difference is observed in sub-structure WC-FEP (Fig. 7.28 (h)). In this case, the contribution of secondary beams is even comparable to that of the slab, due to the use of FEP joint in beam-to-column connections.

7.5.2 Structural behaviour of sub-structures with different slab aspect ratios

7.5.2.1 Design of sub-structures

As recommended in the last section, the joint combination FEP-WC in the test programme is a good choice for robust structures. Therefore, further studies on slab aspect ratio can be based on this joint combination. Specifically, the commonly-used slab aspect ratio in practice, such as 1:1, 1.25:1, 1.5:1 and 1.75:1, are studied. Among them, 1:1 and 1.5:1 correspond to Specimens 2×2-S-IC and 2×3-S-IC, respectively, which are abbreviated as “2×2” and “2×3” in this section. Accordingly, 1.25:1 and

1.75:1 come from sub-structures “2×2.5” and “2×3.5” with the respective slab panel dimensions of 2m×2.5m and 2m×3.5m (refer to Fig. 3.2). All the other structural information remains the same as those for Specimens 2×2-S-IC and 2×3-S-IC (Section 3.2).

7.5.2.2 *Effect of slab aspect ratio on robustness*

Fig. 7.29 shows the UDL versus central displacement relationships for sub-structures 2×2, 2×2.5, 2×3 and 2×3.5. As expected, load-carrying capacity proportionally decreases with increasing slab aspect ratio. However, joint components in all the sub-structures fail at respective same displacements, as shown in Fig. 7.29, since the same dimension in the shorter span results in the same extensions for the respective joint components and the same rotation for plastic hinges (Eqs. (7.22) ~ (7.25)). That is to say the energy stored in each sub-frame is equal, which is quantified in the subsequent section. However, the load contributions of the sub-frames are inversely proportional to slab aspect ratio (Fig. 7.30) due to different deformed profiles of the slabs (refer to Eq. (7.36)). The same concept can be applied to composite slabs, where the respective extensions of steel reinforcement and steel decking in each sub-structure are the same according to Eqs. (7.9) ~ (7.21). Besides, there is less reinforcement and steel decking in the composite slab with a smaller slab aspect ratio, which achieves a greater load-carrying capacity (Fig. 7.31). That is to say, the slab with a smaller aspect ratio deflects in a more beneficial profile, promoting the development of TMA in slabs.

Therefore, it can be concluded that the sub-structure with a smaller slab aspect ratio experiences more prominent geometry nonlinearity which contributes to more increases in load. However, as shown in Fig. 7.31, fracture of reinforcement along the perimeters (regions *OE* and *OH* in Fig. 7.5) causes more significant decreases in load-carrying capacity for 2×2 compared with the other sub-structures, due to less reinforcement in the composite slabs of 2×2. As a result, the differences of load-carrying capacity among the composite slabs of 2×2, 2×2.5, 2×3 and 2×3.5 are very small at the final stage.

*GS-1st and GS-all indicate the respective failure of the first-row and all components at the sagging (S) end of the double-span girder (G).

BS-all indicates failure of all the components at the sagging end of the double-span beam (B).

Fig. 7.29: Load-displacement curves of sub-structures with different slab aspect ratios

Fig. 7.30: Load contributions of sub-frames

Fig. 7.31: Load contributions of composite slabs

Compared with limited data acquisition in actual tests, numerical and analytical models have the same advantage of quantifying energy stored in structural components. From this particular angle, numerical and analytical results are compared in the following section.

7.5.3 Comparisons Between Numerical and Analytical Analyses

As presented in Section 7.5.1.4, based on the proposed model, individual contributions of the three alternate load paths, viz. the slab, the double-span girder and the double-span secondary beams, are separated and compared through calculating energy stored in the respective structural members. The comparisons generally show that the slab contributes most to vertical resistance, followed by the double-span secondary beams and the double-span girder makes the least contribution. This finding can be confirmed by numerical simulations, since one can request history-output for energy of any members using ABAQUS, as shown in Fig. 7.32 and Fig. 7.33.

Fig. 7.32: Comparison between analytical and FE models for 2×3-S-IC (or FEP-WC)

As shown in Fig. 7.32, good agreement between the proposed analytical model and the FE model in terms of energy quantities in different structural members, further demonstrating the reliability of the proposed model. For 2×2-S-IC, the analytical model slightly overestimates the contribution of the double-span secondary beams (Fig. 7.33 (b)) and underestimates that of the double-span girder (Fig. 7.33 (c)) compared to the simulation results. Nevertheless, the summation of these two parts are well predicted (Fig. 7.33 (d)).

Fig. 7.33: Comparison between analytical and FE models for 2×2-S-IC (or 2×2)

Generally, it can be seen that energy quantity stored in the slab, the double-span girder and the double-span beams in a substructure predicted by the analytical model is slightly greater than that computed using FE simulations (Fig. 7.32 and Fig. 7.33). However, the analytical model still predicts the load-carrying capacity conservatively compared to simulations, as shown in Fig. 7.34. The reason is that, in the analytical model, internal energy is assumed to be just stored in the slabs of immediate affected bays, the double-span girder and the double-span beams, whereas slabs outside of the

immediate affected bays, perimeter beams and the corresponding joints in FE models also deform and store energy.

(a) 2×3-S-IC

(b) 2×2-S-IC

Fig. 7.34: Comparison of load-deflection curve between proposed model and FE simulations

As discussed in Section 7.5.2.2, the sub-structure with a smaller slab aspect ratio can sustain a greater load. However, energy stored in the sub-frame remains the same and the energy quantity in the slab is even smaller compared with the sub-structure with a larger aspect ratio according to the proposed model, which is numerically shown in Fig. 7.35. This inference based on the proposed model can be further verified by comparing simulation results of 2×3-S-IC and 2×2-S-IC, as shown in Fig. 7.35. Therefore, it can be concluded that the sub-structure with a smaller slab aspect ratio experiences more prominent geometry nonlinearity which contributes to more increases in load.

(a) Energy stored in sub-frames

(b) Energy stored in slabs

Fig. 7.35: Comparison between 2×3-S-IC and 2×2-S-IC in terms of energy stored in slab and sub-frame

7.6 Summary and Conclusions

In this chapter, a mechanical model is proposed to assess the progressive collapse resistance of 3D composite floor systems subjected to internal column loss. The model incorporates three branches to predict the load-displacement response at small deformation (linear elastic), transitional and large deformation (plastic) stages. The load contribution at small deformation stage arises from flexural behaviour in the composite slab based on small-deformation thin plate theory and in the steel beams using virtual work principle. Transitional stage links small and large deformation stages. Additional assumptions are made in large deformation stage to quantify internal energy stored in respective structural components and external work done by applied load. Key assumptions include deformed shape of the floor system, component-based model for joints and rigid-plastic material model for steel. The load corresponding to any deflection can be computed based on conservation of energy. In this way, the static load-displacement response at large deformation can be obtained by implementing step-by-step procedure. After comparing with three actual test results and six numerical simulations, the model shows good capability to predict load-deflection responses for 3D composite floors. Besides, the model can capture the effects of variables such as slab dimensions, slab aspect ratio, joint type, number of bolts, slab thickness, reinforcement ratio and steel decking thickness.

Based on the proposed model, robustness of eight sub-structures with different types of joints are compared. Besides, four sub-structures with different aspect ratios are compared in terms of resistance. Strictly, the analytical results are further verified by numerical simulations from the perspective of energy stored in individual structural members. From these case studies, the following conclusions can be drawn:

- (1) Moment-resisting joints in girder-to-column connections govern structural resistance at small and transitional stages, while simple joints govern the behaviour at large deformation stage.
- (2) Secondary beams make a greater contribution than girders at large deformation stage.
- (3) The slab contribution (TMA) is more dominant in the sub-structure with weak joint combination (say, FP-FP). However, the overall resistance of such sub-

structures cannot compete with the ones with robust joint combinations EFP-WC or FEP-WC.

- (4) A smaller slab aspect ratio can increase load-carrying capacity at large deformation stage due to greater development of TMA in slabs.

To design a robust steel building against progressive collapse, some recommendations are given:

- (1) In conventional design (moment-resisting joint for girders and simple joint for secondary beams), strength and ductility of simple joints must be ensured to guard against progressive collapse, since they are the major players at large deformation stage. Otherwise, slabs cannot withstand the vertical loads after failure of all the joints.
- (2) Alternatively, structures can be designed with robust simple connections (such as WC) for all joints, sacrificing some flexural rigidity and load-carrying capacities at small deformation stage.
- (3) It is feasible to select moment-resisting joints for longer-span beam-to-column connections and simple WC joints for shorter-span girder-to-column connections.

CHAPTER 8 CONCLUSIONS AND FUTURE WORKS

8.1 Conclusions

This research investigates the structural behaviour of three-dimensional steel-frame-composite-floor systems (3D composite floor systems) subjected to an internal column removal scenario through experimental, numerical and analytical studies.

Experimental Studies

Four large-scale 3D floor systems were quasi-statically tested to failure under an internal column removal scenario. Among them, one was the control specimen, while the other three varied in terms of slab aspect ratio, degree of composite action between steel beams and slabs, and boundary condition, respectively. The test set-up of the specimens simulated restraints from surrounding structures, and reaction forces in the restraints could be measured. To achieve the experimental objectives, the test results are elucidated in two parts.

The first part clarifies load-resisting mechanisms (Alternate Load Path) of 3D composite floor systems following the loss of an internal column. It could be clearly observed that the applied load was transferred through three paths including the double-span girder, the double-span beam and the composite slabs. The load contributions of both the double-span girder and double-span beam were further attributed to flexure and catenary action (CA). The mobilisation and development of the load-resisting mechanisms were important to the robustness of the structure, which are summarised plus some other valuable findings, as below:

- (1) At small deformation stage, the vertical loads were mainly transferred by flexure and compressive arch action (CAA) in the double-span girder and flexure in composite slabs to the surrounding intact frame. Most of the loads shift from the removed column were redistributed to adjacent columns with moment connections through the double-span girder. At large deformation stage, the double-span beam and the composite slabs with inner beams were the major contributors to the increase in vertical load. Hence, the redistributed

vertical loads were transferred from the columns connected with the double-span girder to remaining surrounding columns.

- (2) The contributions of composite slabs with inner beams at ultimate state accounted for at least 1/3 of the total vertical load. In the case of 2×3-W-IC, the contributions of flexural action in the double-span girder and double-span beam, CA in the double-span girder and double-span beam, and slab with inner beams at the ultimate state accounted for 36.4%, 26.7%, and 36.9%, respectively.
- (3) CAA in the double-span beam was not as significant as that in the double-span girder. CA in the double-span composite girder and double-span composite beam were mobilised at about one girder or one beam depth.
- (4) Tensile membrane action was present in the composite floor systems and the tensile zone propagated outwards towards the peripheral at a very early stage (1.1 times slab thickness).
- (5) Failure modes of 3D composite floor system developed in a ductile way if beam-to-column connections had enough rotational capacity.
- (6) In the rectangular slab system, the girders deformed linearly, while secondary beams deformed in a multi-linear manner similar to a polynomial deflection curve caused by uniform loads.

The second part reveals the effects of slab aspect ratio, composite action and boundary condition on load-carrying capacity, ductility, failure mode, deformation characteristics, load redistribution among the remaining columns and reaction forces from surrounding structures. The main conclusions are summarised as follows:

- (1) The load-carrying capacity of specimen 2×2-S-IC was much greater than those of the other three specimens due to a smaller span length in beam-direction and a smaller aspect ratio which increased flexural capacity at small deformation stage and promoted development of TMA at large deformation stage, while the deformation capacity of 2×2-S-IC was the smallest among the

four specimens. The equal span of slabs in both directions delayed the failure of girder-to-removed-column connection. The columns at the ends of the double-span girder and the double-span beam sustained similar percentages of total load in the specimen with square slab before failure of joint components. Loads sustained by the double-span-girder end columns were slightly transferred to the other columns due to failure of joint components in the double-span girder.

- (2) Vertical load redistribution is significantly affected by slab aspect ratio and connection ductility. For composite floor systems with rectangular slabs, the load supposed to be sustained by the removed column was mostly redistributed to the adjacent columns in short-span direction before complete failure of the girder-to-removed column connection. Afterwards, most of the load was sustained by the columns at the ends of double-span beam, followed by the columns at the corners of the affected bays.
- (3) Composite action between slabs and girders as well as between slabs and beams had marginal influence on ultimate load-carrying capacity. Although weaker composite action initially decreased flexural resistance at small deformation stage, it improved ductility significantly which is beneficial for the development of CA and TMA in the subsequent stages. Load redistribution pattern in 2×3-W-IC was similar to that in 2×3-S-IC until the final stage, when the columns at the ends of the double-span beam in 2×3-W-IC sustained a greater percentage of the total load compared with those in 2×3-S-IC. This was due to more significant CA developed in the double-span beam in 2×3-W-IC.
- (4) More flexible boundary conditions could not increase the applied load after the flexural stage. Along the free edge, lateral deformation was concentrated at the end of the double-span girder. The catenary action generated in the double-span girder caused yielding of the peripheral column base. This kind of column could easily buckle if they are also subjected to axial compression forces from upper storeys. Therefore, special attention should be paid to the stability of peripheral columns when designing for catenary action in double-

span beams. For this specimen, the boundary condition (with Penultimate Internal column) only slightly influenced the vertical load redistribution among the remaining columns.

- (5) For composite floor systems, the flexural rigidity of steel columns was sufficient to support the development of CAA so that additional horizontal restraints from surrounding structural members were not so beneficial at CAA stage. However, the stiffness of horizontal restraints dominated the floor system at CA stage.
- (6) Strength and ductility of girder- and beam-to-column joints are crucial to robustness of 3D composite floor systems. Enhancing strength and ductility of each bolt-row component can increase ultimate load-carrying capacity and postpone collapse of the floor system, respectively.
- (7) Typical yield pattern of composite slabs consisted of negative yield lines along the perimeters and positive yield lines along the double-span girders and beams, and paths connecting corner columns to the nearest loading points.

Numerical Studies

Using finite element (FE) package ABAQUS, macro-models are built and validated at slab- and system-levels, showing the capability to predict the load-displacement relationship and failure mechanisms for 3D composite floor subjected to a column removal scenario. The verified modelling method is used for further static and dynamic analyses. The static simulation results reveal the significance of composite slabs and inter-relationship between the load-resisting mechanisms as follows:

- (1) Slabs can enhance the robustness of a 3D composite floor system as long as the beam-to-column connections have sufficient ductility. Specifically, the slab can significantly increase the load-carrying capacity of composite floor systems when comparing with the corresponding skeletal frames. However, slabs would decrease the deformation capacity of the structure. Between the skeletal frame and full-composite floor system, the floor system with partial composite action has good ductility without compromising load-carrying

capacity.

- (2) A change in the slab thickness or degree of composite action would not affect the ultimate load-carrying capacity significantly due to the robustness of 3D composite floor systems in which the load-resisting mechanisms are complementary depending on structural features of the system. For instance, prominent flexure would suppress the CA, since they are the two complimentary components in the double-span girder and double-span beam. Shorter span, stronger composite action and a thicker slab can promote the flexure and suppress CA, and vice versa. The slab contribution at final stage arose from TMA is independent of flexure and CA in the double-span girder and double-span beam, which remains constant if steel reinforcement and decking, and slab dimensions the same.

A full-scale 3D composite floor system model subjected to an internal column loss scenario is statically analysed to obtain the static response. Subsequently, the model is analysed under sudden column loss and the results are summarised into the pseudo-static response, which are compared with the corresponding static response to obtain the dynamic increase factors (DIFs). The obtained DIFs are then compared with those specified in DoD (2013) and those of 2D steel frames with the same connection types. All the conclusions are summarised as follows:

- (1) The dynamic displacement is greater than the corresponding quasi-static displacement under the same load, and the difference will rise with increasing loads. The displacement-based DIF_u at the ultimate dynamic load-carrying capacity is about 2.8.
- (2) The resistance of the 3D composite floor system under sudden column loss decreases by 31.6%. Specifically, the static and dynamic load-carrying capacities are around 19.3 kN/m^2 and 13.0 kN/m^2 , respectively. The dynamic ultimate state does not correspond to the static ultimate state in terms of load and deformation.
- (3) The pseudo-static response obtained by dynamic numerical analyse agree well with the pseudo-static response by energy method. However, the energy method does not give any indication of dynamic ultimate limit state.

(4) The trend of DIF_p – deformation curve for a structure depends on its characteristic of static load-displacement curve.

(5) In 3D composite floor systems, the force-based DIF_p decreases with increasing central displacement, and will not increase significantly during CA or TMA stage. Even so, the DIF_p recommended in DoD (2013) tends to be unsafe at large displacement stage.

(6) The 3D composite floor system requires more ductility than 2D steel frames with the same connection types, and the DIF_p for the 3D composite floor system is smaller than those for 2D steel frames.

Analytical Studies

A mechanical model is proposed to assess the progressive collapse resistance of 3D composite floor systems subjected to an internal column loss. The model incorporates three branches to predict the load-displacement response at small deformation (linearly), transitional (linearly) and large deformation (nonlinearly) stages. The load contribution of each structural component can be counted large deformation stage, so the development of load transfer path and the failure mechanism can be captured by this model. After comparing with the actual test and simulation results, the model shows good a capability to predict load-deflection curves for 3D composite floors. Besides, the model can capture the effects of variables such as slab dimensions, slab aspect ratio, joint type, number of bolts, slab thickness, reinforcement ratio and steel decking thickness. Most importantly, the procedure of the model can be implemented in a spreadsheet, which provides a simple and numerical robust tool for engineers to calculate progressive collapse resistance of structures for missing column scenarios.

Based on the proposed model, robustness of eight sub-structures with different types of joints are compared, besides, four sub-structures with various of aspect ratios are also compared in terms of resistance, leading to four conclusions:

- (1) Moment-resisting joints in girder-to-column connections govern the structural resistance at small and transitional stage, while simple joints no matter used for girders or beams govern the behaviour at large deformation

stage.

- (2) Secondary beams make a greater contribution than girders at large deformation stage.
- (3) The slab contribution is more dominant in the sub-structure with a weak joint combination (say, fin-plate + fin-plate). However, the overall resistance of such sub-structures cannot compete with the ones with robust joint combinations “extended-end-plate + web-cleat” or “flush-end-plate + web-cleat”.
- (4) A smaller slab aspect ratio can increase load-carrying capacity due to a more beneficial deformed profile (or geometry nonlinearity) for development of TMA in slabs.

To design a robust steel building against progressive collapse, some recommendations are given:

- (1) In conventional design (moment-resisting joint for girders and simple joint for secondary beams), strength and ductility of simple joints must be ensured to against progressive collapse, since they are the major players at large deformation stage. Otherwise, slabs cannot withstand the vertical loads after failure of all the joints.
- (2) Alternatively, structures can be designed with robust simple joints (such as web-cleat) for all connections, only sacrificing some load-carrying capacities at small deformation stage.

8.2 Future Works

According to the limitations of this thesis, the suggestions for future research are as follows:

- (1) This research is limited to internal column removal scenarios. Hence, future works can be dedicated to other column removal scenarios such as penultimate external column, corner column by means of experimental tests, numerical simulations and analytical derivations.
- (2) The dynamic effect is the inherent behaviour of a building subjected to sudden

column loss. Besides the numerical studies, free-fall tests should be carried out on 3D composite floor systems to detail the actual failure mechanisms and obtain more realistic DIF. The test results can be used to validate the DIF in DoD (2013) which is derived based on numerical results.

- (3) The specimens in this research are limited to one standard storey based on the assumptions that only the bays above the removed column are affected and that each affected storey behave similarly. Some studies showed the simplifications are appropriate, while some others did not. Hence, whether the global vierendeel action is determinant or not, should be further evaluated by reliable numerical models or by experimental tests if possible.
- (4) Although the assumptions in (3) are reasonable, the surrounding column should be applied with loads from upper storeys. This could be numerically studied in the future, since the considerable vertical compression forces combined with catenary action in the double-span girders or double-span beams are detrimental to the edge columns.
- (5) For the proposed mechanical model, the failure criterion of reinforcement across cracks (Eq. (7.8)) is rather empirical, which could be further improved through theoretical derivations and numbers of well-designed bond-slip tests. The proposed model does not consider fracture of steel decking so that its continuing contribution is only terminated by the global failure criterion ($\delta > 2h_g$). It is very difficult to define the limiting extension of steel decking in composite slabs, which needs to be further studied based on extensive experimental tests.

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APPENDIX A: EXPERIMENTAL DETERMINATION OF SHEAR CONNECTOR RESISTANCE AND BOLTED JOINT COMPONENT PROPERTIES

A.1 Overview

This appendix aims to experimentally quantify the shear stud resistance and joint component properties in the 3D composite floor systems for the applications in numerical (Chapter 6) and analytical (Chapter 7) studies. Specifically, four push-out tests were carried out to study the shear resistance of studs in the composite slabs. To obtain the force-displacement relationships of components in bolted web-cleat joints, two specimens were tested under axial compression and six under axial tension. Besides, two pairs of T-stub flanges and three bolted flush-end-plate connections were tested under axial tension to quantify the tensile properties of T-stub components corresponding to bolt-rows. For the property of the component in compression, three girder stubs were tested under compression forces aligned with one of the flanges. These tests were conducted by the members in the author's research group (Jiang et al. 2016), so the tests are briefly introduced while the determination of component properties are presented in this thesis.

A.2 Push-out Tests

A.2.1 Description of the tests

In accordance with 3D floor system tests, the composite slab consisted of 40 mm deep by 0.9 mm thick profiled steel decking, 65 mm thick concrete, and $\varnothing 6$ reinforcing bars placed with 100 mm spacing in both directions (Fig. 3.3).

The push-out tests were conducted on Specimens B-93, B-370, G-75 and G220 (Fig. A. 1). Specimens B-93 and B-370 were extracted from the composite beams, which were used to investigate the shear behaviour when the loading direction is transverse to ribs of profile sheeting (Fig. A. 1 (a) and (b)). The terms “93” and “370” denoted the spacings of shear studs in mm for strong and weak composite action (Fig. 3.3), respectively. Similarly, G-75 and G-220 were extracted from the composite girders, which were used to investigate the shear behaviour when the loading direction is

parallel to ribs of profile sheeting. The spacings between shear studs in the respective G-75 and G-220 were 75 mm and 220 mm (Fig. A. 1 (c) and (d)), which were accordance with the 3D composite floor systems with strong and weak composite action (Fig. 3.3), respectively.

Fig. A. 1: Push-out test specimens (Jiang et al. 2016)

Fig. A. 2 shows the test set-up and instrumentation for push-out tests. One linear variable displacement transducer (LVDT) and three dial indicators were installed to measure the displacements at the top of composite slabs, while two LVDTs were installed to measure the displacements at the top end of steel beam, as shown in Fig. A. 2. Thus, the relative displacement between the composite slabs and the steel beam can be obtained. Meanwhile, the applied load was recorded by the load cell under the hydraulic jack.

Fig. A. 2: Set-up and instrumentation for push-out tests (Jiang et al. 2016)

A.2.3 Experimental results — Load-slip curves

Fig. A. 3 shows the load-slip curves for specimens G-75, B-93, G-220 and B-370. Among them, B-93 had the greatest shear resistance, while Specimen G-75 had the lowest one. To achieve comparability and provide verifying data for the shear stud model in the numerical models (Chapter 6), load-slip curves of all specimens are normalised as shown in Fig. A. 4. Each curve represents the response of a unit composite slab including 1m-length arrangement of shear studs. For instance, LPR-75 represents a unit composite slab which is Loaded in Parallel to Ribs, with 75 mm shear stud spacing. The curve of LPR-75 is obtained through multiplying the half load of G-75 by $1000/75$ and remaining the same displacement as that of G-75. LTR-185 represents a unit composite slab which is Loaded Transversely to Ribs, with 185 mm shear stud spacing. It should be noted that in B-93, two studs in one trough are bound as an equivalent one. Hence, the curve of LTR-185 is obtained through multiplying the half load of B-93 by $1000/185$ and remaining the same displacement as that of B-93.

Fig. A. 3: Load-slip curves of push-out tests (Jiang et al. 2016)

Fig. A. 4: Normalised load-slip curves

A.3 Tests on Web-cleat Joints

A.3.1 Description of tests

The objective of this section is to quantify the component parameters of web-cleat joints in the 3D composite floor systems. Specifically, there were two components aligned with the respective bolt rows (components 1 and 2 in Fig. A. 5), which were able to sustain both tension and compression. Each component combined the corresponding basic components including beam web in tension, angle in tension, angle in bending, bolt in tension, bolt in shear and column (or girder) web in bending. However, only the overall combined component parameters were needed without necessity to quantify those of the basic components. Thus, the parameters of combined components could be quantified through separate axial tensile and axial compressive loading assuming that components 1 and 2 shared the same property. With this notion, four specimens named A1, A2, A3 and A4 were tested under axial tension, while A5 and A6 subjected to axial compression and tension successively (Table A. 1). Besides, two more factors including bolt-hole size as well as friction between the beam web and angles were studied. The designed bolt-hole size was 18 mm in diameter but the actual delivered ones ranged from 18 mm to 20 mm, which motivated Jiang et al. (2016) to study the effect of bolt-hole size. In Table 6.2, the painted angles (A1 and A5) were directly recycled from the tested 3D floor systems

which were still in elastic stage, while the unpainted angles were newly fabricated with the same material.

Fig. A. 5: Components in web-cleat connection

Table A. 1: Parameters of all specimens (Jiang et al. 2016)

Specimen	Hole size(mm)	Painted or not	Load condition
A-1	20	Yes	Tension
A-2	20	No	Tension
A-3	20	No	Tension
A-4	18	No	Tension
A-5	20	Yes	Compression & Tension
A-6	20	No	Compression & Tension

Fig. A. 6 shows the test set-up and configuration of a specimen. For a web-cleat jointed to a column or a girder, the deformation of the column or girder was small enough to be neglected, so only the deformation of the beam web, angles, and bolts were considered. Hence, the beam stubs recycled from the tested 3D floor systems, were connected to a “rigid plate” by double angles (A1 through A6). The rigid plate was fixed on a supporting column, of which axial force could be measured. The other ends of beam stubs were vertically connected to the hydraulic jack through a pin aligned with the centroid of beam, to ensure the axial loading. The column axial force can be compared with the reading from the load cell underneath the jack (Fig. A. 6) for double check. Two LVDTs were used to measure displacements at the top end of

beam stubs, while four dial indicators recorded that of the rigid plate. The difference between these two sets of readings was the elongation of a specimen.

Fig. A. 6: Test set-up of beam-to-column connections (Jiang et al. 2016)

A.3.2 Experimental results

The load-displacement curves of Specimens A-5 and A-6 under axial compression are presented in Fig. A. 7. The common failure of A-5 and A-6 was bolt fracture caused by shear (Fig. A. 8). Besides, the significant elongation of bolt holes was observed in A5. The axial tension tests on A-5 and A-6 were carried out after the accomplishment of the compression tests by replacing the fractured bolts with new ones.

Fig. A. 7: Load-displacement curves of the web-cleat joints under compression
(Jiang et al. 2016)

(a) Specimen A-5

(b) Specimen A-6

Fig. A. 8: Failure mode of web-cleat joints subjected to axial compression (Jiang et al. 2016)

For the tension tests, it should be noted that the test result of Specimen A-2 was questionable due to an equipment fault, so the data of A-2 was abandoned. Fig. A. 9 shows the load versus displacement curves of the other specimens. Angles of all specimens fractured at bolt holes in tensile test (Fig. A. 10), same in the 3D system tests (Fig. 4.6 (f)).

Fig. A. 9: Load-displacement curves of the web-cleat joints under tension (Jiang et al. 2016)

Fig. A. 10: Failure mode of the web-cleat joints subjected to axial tension (Jiang et al. 2016)

A.3.3 Property of component

As aforementioned, the axial load was equally sustained by components 1 and 2 in parallel, whose identical property could be separated straightforwardly based on load-displacement curves in Fig. A. 9. The curves (Fig. A. 9) are consistant except for A-4 with 18 mm bolt-hole size. Reasonably, A-4 is not considered for quantifying component parameters for the 3D floor system tests with 20 mm bolt-hole size in angles. Besides, A-5 is also kicked out due to the initial elongation of the bolt holes. For ease of application, the test results of A1, A3 and A6 can be simplified and unified into one load-displacement curve by piece-wise linnear fitting at distinct stages (Fig. A. 11). Similarly, the unified load-displacement curve for A-5 and A-6 under axial compression is obatined (Fig. A. 7). Finally,the author can quantify the property of one component by halving the load on the same displacement basis, as shown in Fig. A. 12 and Eq. (A.1).

$$F = \begin{cases} 22.4\Delta S \text{ kN} & -7.0 \leq \Delta S < 0 \text{ mm} \\ -18.8\Delta S - 288.4 \text{ kN} & -15.3 < \Delta S \leq -7.0 \text{ mm} \\ 34.18\Delta S \text{ kN} & 0 \leq \Delta S < 1.23 \text{ mm} \\ 2.54\Delta S + 38.8 \text{ kN} & 1.23 < \Delta S \leq 28.9 \text{ mm} \\ -4.05\Delta S + 229.1 \text{ kN} & 28.9 < \Delta S \leq 56.6 \text{ mm} \end{cases} \quad (\text{A.1})$$

Fig. A. 11: Linear fitting for test results of web-cleat connection

Fig. A. 12: Property of component in web-cleat connection

A.4 Tests on Flush-end-plate Joints

A.4.1 Description of tests

The tests in this section aimed to identify the properties of components in flush-end-plate joints, including three T-stub components sustaining tension and two components sustaining compression (Fig. A. 13). The former and the latter components correspond to the respective bolt-rows and girder flanges. The property of T-stub flange corresponding to inner bolt-row (T-stub 2 in Fig. A. 13) is different from those of bolt-rows (T-stub 2 and T-stub-3) adjacent to beam flanges (EC 3-1-8

2005). However, T-stubs 1 and 3 (Fig. A. 13) share the same property due to symmetry. Hence, to separate the properties of all the components in tension, the property of T-stub 2 (Fig. A. 13) was obtained first by testing on two pairs of identically designed T-stub flanges TF-1 and TF-2 (Fig. A. 14). Besides, three bolted end-plate joints EP-1, EP-2 and EP-3 were tested under axial tension to obtain the overall load-displacement curves (stiffness). Subsequently, the property of T-stubs 1 and 3 could be determined by subtracting the stiffness of T-stub 2 from the overall stiffness of the joint. The test set-up for EP-1, EP-2 and EP-3 is similar to web-cleat joint tests (Fig. A. 6).

Fig. A. 13: Components in flush-end-plate joints

(a) Dimensions of T-stub flanges

(b) Test set-up

Fig. A. 14: Tests on T-stub flanges TF-1 and TF-2 (Jiang et al. 2016)

In the 3D composite floor system tests, it was observed that the girder flanges in compression buckled at hogging-moment joints (Fig. 6.9 (a)). EC3-1-8 (2005) also

assumes that the compression centre of the bolted end-plate joint in bending is located at the centre of the girder flange in compression. Therefore, girder flange in compression was a critical component, whose property was quantified by testing on three girder stubs GFC-1, GFC-2 and GFC-3 recycled from the 3D floor system tests. In each test, the hydraulic jack applied a compressive load aligned with one flange (Fig. A. 15), while LVDTs and Dial indicators at the respective top and bottom ends of the girder stub were used to measure the deformation of the flange.

Fig. A. 15: Test set-up for girder flange in compression (Jiang et al. 2016)

A.4.2 Experimental results and properties of components

A.4.2.1 Middle-row T-stub flanges in tension

Fig. A. 16 shows the load-displacement curves for TF-1 and TF-2, which agreed very well with each other before the final failure. The average load of TF-1 and TF-2 versus each displacement can be depicted as a scatter plot, which can be fitted into a piecewise linear function for easy application at distinct loading stages, as shown in Fig. A. 17 and Eq. (A.2).

$$F = \begin{cases} 248.52\Delta S_2 \text{ kN} & (0 \leq \Delta S_2 \leq 0.12 \text{ mm}) \\ 9.59\Delta S_2 + 29.29 \text{ kN} & (0.12 < \Delta S_2 \leq 1.64 \text{ mm}) \\ 1.08\Delta S_2 + 43.25 \text{ kN} & (1.64 < \Delta S_2 \leq 12.15 \text{ mm}) \\ -2.67\Delta S_2 + 88.79 \text{ kN} & (12.15 < \Delta S_2 \leq 33.25 \text{ mm}) \end{cases} \quad (\text{A.2})$$

Fig. A. 16: Load-displacement curves of TF-1 and TF-2 (Jiang et al. 2016)

Fig. A. 17: Property of middle-row component T-stub 2

A.4.2.2 Bolted end-plate joints in tension

As shown in Fig. A. 18, the failure mode of EP-1, EP-2 and EP-3 was the bolt fracture accompanied with end-plate fracture. Fig. A. 19 shows that the applied loads on all the three specimens increased rapidly with the initial stiffness before the displacement of 2.5 mm, followed by slow growth until the respective ultimate states corresponding to the failure of end-row bolts (Fig. A. 18). To achieve a better fitting result, the polynomial instead of linear fitting is applied for the test data of overall end-plate joints (Fig. A. 20). Finally, the property of end-row components in tension (T-stubs 1 and 3) can be obtained by halving the difference between the overall response (Fig. A. 19) and the response of middle-row component (Fig. A. 17). As a result, the property of end-row component is shown in Fig. A. 20 and Eq. (A.3).

$$F = -0.0078\Delta S^4 + 0.3824\Delta S^3 - 6.4745\Delta S^2 + 46.426\Delta S \text{ (kN)} \quad (\text{A.3})$$

$(0 < \Delta S \leq 25 \text{ mm})$

(a) EP-1

(b) EP-2

(c) EP-3

Fig. A. 18: Failure mode of bolted end-plate joints in tension (Jiang et al. 2016)

Fig. A. 19: Load-displacement curves of end-plate joints subjected to tension (Jiang et al. 2016)

Fig. A. 20: Curve fitting for overall end-plate joints subjected to tension

Fig. A. 21: Property of end-row components T-stub1 and T-stub 3

A.4.2.3 Girder flange in compression

Each girder stub was tested under compression to the buckling of girder flange together with the adjacent web (Fig. A. 22 (a)), when the load decrease to around half of the peak. Fig. A. 22 (b) shows the load-displacement curves of Specimens GFC-1, GFC-2 and GFC-3. The curves of GFC-2 and GFC-3 agreed well with each other during the entire loading process, while the curve of GFC-1 was consistent with them at elastic stage but discrepant from them subsequently. The discrepancy may be caused by the low accuracy of alignment before testing GFC-1. However, due to uncertain reasons, all the three test results are fitted into a piecewise linear function (Fig. A. 23).

(a) Failure mode

(b) Load-displacement curve

Fig. A. 22: Test result of girder flange in compression (Jiang et al. 2016)

Fig. A. 23: Property of component in compression in the bolted end-plate joints