PROGRESSIVE COLLAPSE RESISTANCE OF RC FLAT PLATE SUBSTRUCTURES UNDER SINGLE- AND DOUBLE-COLUMN REMOVAL SCENARIOS

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Progressive collapse is defined as the spread of an initial local failure of a critical load bearing structural element to the remaining structure, ultimately leading to the collapse of the entire building or of a large portion of it. After the partial collapse of the Ronan Point apartment tower in 1968, when a localised gas explosion ultimately led to the collapse of the whole corner of the building, progressive collapse firstly drew public attention due to its catastrophic consequences on human casualties and economic losses. Other recent collapse events, such as those of the Murrah Federal Building in 1995, Sampoong department store in 1995 and World Trade Centre towers in 2001, have further highlighted the importance of this topic and made engineering communities aware of the necessity to prevent the occurrence of progressive collapse in building structures.

In the past decades, considerable research has been devoted to investigate progressive collapse of building structures. These research typically followed an alternate load path approach. This approach allows an initial failure to occur, while the remaining structure needs to seek alternate load paths to dissipate damage and prevent progressive collapse. In practice, the progressive collapse resistance of building structures is generally evaluated by notionally removing one or more load bearing structural elements, representing the initial failure. Despite all research efforts made, they have predominately focused on frame systems. Relevant investigations on reinforced concrete (RC) flat plate structures, a commonly used structural system, are still scarce.

The flat plate system contains slabs, of uniform thickness, directly supported by columns without using beams, drop panels and column capitals. This provides structures with increased storey height, architectural convenience and significant savings in construction costs. Nevertheless, the slab-column connection is prone to punching shear failure, which may propagate to the vicinity of connections, potentially leading to progressive collapse. The present study aims at filling some of the current knowledge gap with comprehensive experimental, analytical and numerical investigations on RC flat plate structures.
The experimental program consists of four quasi-static large-deformation tests performed on three 1/3-scale, 2×2-bay RC flat plate substructures subjected to critical column removal scenarios. The test specimens included one specimen tested under two consecutive corner column removal scenarios, one specimen under an edge column removal and one specimen under the scenario of concurrently removing both an edge and an interior column. For the loading scheme, a multi-point loading system was specially designed to simulate an increasing uniformly distributed load (UDL). The loading points of this system were equally positioned on the bays adjacent to the removed column(s) to apply the load until failure of the slab. Throughout each test, the structural behaviour of the slab was monitored by extensive apparatuses mounted to the slab both internally and externally. Failure and post-failure behaviours, failure modes, and collapse resisting mechanisms were observed and analysed. Additionally, a complement of analytical studies to the experimental tests, based on the observed yield lines, was carried out to estimate the flexural capacity of the test specimens.

To numerically investigate the progressive collapse behaviour of RC flat plate structures, a set of 3D nonlinear finite element modelling approaches, using LS-DYNA software, was established. A total of five numerical models were explicitly created based on the four experimental tests in present research and one from the literature. The proposed modelling approach was basically composed of appropriate definitions of element types, bond-slip relationships, loadings and boundary conditions, and more importantly development of the material models for concrete and reinforcements and calibration of failure criteria for element erosion. A quasi-static loading scheme in displacement control in the numerical models was adopted, the same as that in the conducted tests, through explicitly modelling the multi-point loading system in order to replicate the true test conditions. The numerical models were respectively validated by comparing the load-displacement responses, failure modes, crack patterns, and displacement and strain results against the experimental ones, which confirmed the appropriateness and reliability of the proposed models. The validated modelling techniques could be applied to further perform parametric studies of the progressive collapse resistance of RC flat plate substructures and prototype structures.
In summary, the significant and original contribution of this research is to (1) obtain a solid understanding of the collapse behaviour of RC flat plate structures under critical column removal scenarios, (2) examine the analytical solution using the yield line theory to predict the flexural capacity of RC flat plate structures, (3) establish a set of numerical modelling techniques which can reasonably replicate the experimental process and provide valuable references of numerically analysing progressive collapse of RC flat plate structures, and (4) give design insights into Australian and international Standards in mitigating progressive collapse of RC flat plate structures.
STATEMENT OF ORIGINALITY

This work has not previously been submitted for a degree or diploma in any university. To the best of my knowledge and belief, the thesis contains no material previously published or written by another person except where due reference is made in the thesis itself.

(Signed) _________________________________ (Date)   24/10/2019   

Fuhao Ma
ACKNOWLEDGEMENT OF PAPERS INCLUDED IN THIS THESIS

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Included in this thesis are publications presented in Chapters 3 and Chapter 4 which are co-authored with other researchers. My contribution to each co-authored paper is outlined at the front of the relevant chapter. The bibliographic details for these papers including all authors are:

Chapter 3:

Chapter 4:


Appropriate acknowledgements of those who contributed to the research but did not qualify as authors are included in each paper.

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In the afternoon of one day in October, 2019, I finally complete my PhD thesis. Gold Coast is still super sunny, just like it always is. Looking at the thick and more than 200 page hardcopy of my thesis and smelling it just after being printed out, I cannot use any words to tell what I feel right now. Possibly because I am struggling to guess that how many readers I may have and if all the efforts I made on this research really mean something, or just because on one announces that it is time to say goodbye to my 20 year studies.

Writing thesis is plain and boring, but during managing those data and materials, I am reminded of many moments of hardwork and sweetness which I would never forget.

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LIST OF PUBLICATIONS

The following publications are all peer-reviewed and produced from this research to disseminate the innovations and findings:

**Journal publications**


**Conference Publications**


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<th>Description</th>
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<tbody>
<tr>
<td>$A_s$</td>
<td>Area of tension reinforcement per unit width</td>
</tr>
<tr>
<td>$B$</td>
<td>Ductile shape softening parameter</td>
</tr>
<tr>
<td>$C_H$</td>
<td>Hardening rate</td>
</tr>
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<td>$D$</td>
<td>Brittle shape softening parameter</td>
</tr>
<tr>
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<td>Linear shape parameter</td>
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<tr>
<td>$D_2$</td>
<td>Quadratic shape parameter</td>
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<td>$E_c$</td>
<td>Elastic modulus of concrete</td>
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<td>Elastic modulus of reinforcement</td>
</tr>
<tr>
<td>$F_u$</td>
<td>Ultimate applied load from the hydraulic jack</td>
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<tr>
<td>$G_c$</td>
<td>Shear modulus of concrete</td>
</tr>
<tr>
<td>$G_{F0}$</td>
<td>Fracture energy at $f'_c = 10$ MPa as a function of the maximum aggregate size</td>
</tr>
<tr>
<td>$GFC$</td>
<td>Fracture energy in uniaxial stress</td>
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<td>$GFS$</td>
<td>Fracture energy in pure shear stress</td>
</tr>
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<td>$GFT$</td>
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<tr>
<td>$K_c$</td>
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<td>$L$</td>
<td>Column centre-to-centre distance</td>
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<tr>
<td>$L_c$</td>
<td>Length of the negative yield line at the edge columns</td>
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<tr>
<td>$L_{ef}$</td>
<td>Effective span between columns</td>
</tr>
<tr>
<td>$L_n$</td>
<td>Length of clear span between columns</td>
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<tr>
<td>$L_P$</td>
<td>Length of the positive yield line at the removed column</td>
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<tr>
<td>$N_H$</td>
<td>Hardening initiation</td>
</tr>
<tr>
<td>$P[\text{Collapse}]$</td>
<td>Probability of progressive collapse of a structure</td>
</tr>
<tr>
<td>$P[\text{Collapse}/E]$</td>
<td>Probability of occurrence of local failure</td>
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<tr>
<td>$P[E]$</td>
<td>Probability of occurrence of an abnormal event</td>
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## NOTATIONS

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<tr>
<td>( PMOD )</td>
<td>Modified moderate pressure softening parameter</td>
</tr>
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<td>( PWRC )</td>
<td>Shear-to-compression transition parameter</td>
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<tr>
<td>( PWRT )</td>
<td>Shear-to-tension transition parameter</td>
</tr>
<tr>
<td>( R )</td>
<td>Cap aspect ratio; measured radius of the quarter-circular yield line; resistance</td>
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<tr>
<td>( U_A )</td>
<td>Output voltage</td>
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<tr>
<td>( U_E )</td>
<td>Excitation voltage</td>
</tr>
<tr>
<td>( V )</td>
<td>Shear force</td>
</tr>
<tr>
<td>( V_{\text{flex}} )</td>
<td>Shear force at flexural failure</td>
</tr>
<tr>
<td>( V_u )</td>
<td>Punching shear strength of the slab</td>
</tr>
<tr>
<td>( W )</td>
<td>Maximum plastic volume compaction</td>
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<td>( W_E )</td>
<td>External work for T1 and T2 done by the applied UDL</td>
</tr>
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<td>External work for S-E done by the applied UDL</td>
</tr>
<tr>
<td>( W_{E,S-EI} )</td>
<td>External work for S-EI done by the applied UDL</td>
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<td>Internal work done along all the yield lines for S-EI</td>
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<td>Internal work done along all the yield lines for T1</td>
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<td>( W_{I,T2} )</td>
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<td>( X_D )</td>
<td>Cap initial location</td>
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### Roman Lowercase Letters

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<td>( b_0 )</td>
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</tr>
<tr>
<td>( d )</td>
<td>Slab effective depth</td>
</tr>
<tr>
<td>( d_g )</td>
<td>Maximum diameter of aggregate</td>
</tr>
<tr>
<td>( d_{g0} )</td>
<td>Diameter of reference aggregate</td>
</tr>
<tr>
<td>( c_{\text{clear}} )</td>
<td>Clear distance between ribs</td>
</tr>
<tr>
<td>( d )</td>
<td>Slab effective depth</td>
</tr>
<tr>
<td>( f'_c )</td>
<td>Concrete compressive strength</td>
</tr>
<tr>
<td>( f_{cm} )</td>
<td>Mean concrete compressive strength</td>
</tr>
<tr>
<td>( f_y )</td>
<td>Yield strength of reinforcement</td>
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### NOTATIONS

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<tr>
<td>$g_1$</td>
<td>Superimposed dead load</td>
</tr>
<tr>
<td>$g_2$</td>
<td>Dead load</td>
</tr>
<tr>
<td>$m_u$</td>
<td>Ultimate bending moment resistance per unit width</td>
</tr>
<tr>
<td>$m_{unc}$</td>
<td>Negative bending moment resistance per unit width in the column strip</td>
</tr>
<tr>
<td>$m_{unm}$</td>
<td>Negative bending moment resistance per unit width in the middle strip</td>
</tr>
<tr>
<td>$m_{upb}$</td>
<td>Positive bending moment resistance per unit width</td>
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<td>$q$</td>
<td>Live load</td>
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<tr>
<td>$q_u$</td>
<td>Ultimate applied UDL to the slab through the loading tree</td>
</tr>
<tr>
<td>$r_s$</td>
<td>Radius of circular isolated slab</td>
</tr>
<tr>
<td>$s$</td>
<td>Relative displacement of reinforcing bars to concrete</td>
</tr>
<tr>
<td>$s_1$</td>
<td>Relative displacement of reinforcing bars to concrete at the first bond stress condition</td>
</tr>
<tr>
<td>$s_2$</td>
<td>Relative displacement of reinforcing bars to concrete at the second bond stress condition</td>
</tr>
<tr>
<td>$s_3$</td>
<td>Relative displacement of reinforcing bars to concrete at the third bond stress condition</td>
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<tr>
<td>$\nu_c$</td>
<td>Poisson’s ratio of concrete</td>
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<tr>
<td>$w$</td>
<td>Wind load</td>
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### Greek Letters

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<td>$\alpha$</td>
<td>Tri-axial compression surface constant term</td>
</tr>
<tr>
<td>$\alpha_1$</td>
<td>Torsion surface constant term</td>
</tr>
<tr>
<td>$\alpha_2$</td>
<td>Tri-axial extension surface constant term</td>
</tr>
<tr>
<td>$\beta$</td>
<td>Tri-axial compression surface exponent</td>
</tr>
<tr>
<td>$\beta_1$</td>
<td>Torsion surface exponent</td>
</tr>
<tr>
<td>$\beta_2$</td>
<td>Tri-axial extension surface exponent</td>
</tr>
<tr>
<td>$\gamma_{c,max}$</td>
<td>Maximum shear strain of concrete</td>
</tr>
<tr>
<td>$\delta$</td>
<td>Virtual displacement at the removed column</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>-----------------</td>
<td>-----------------------------------------------------------------------------</td>
</tr>
<tr>
<td>$\varepsilon_{c,\text{max}}$</td>
<td>Maximum effective strain of concrete</td>
</tr>
<tr>
<td>$\varepsilon_s$</td>
<td>Strain of reinforcement</td>
</tr>
<tr>
<td>$\varepsilon_y$</td>
<td>Yield strain of reinforcement</td>
</tr>
<tr>
<td>$\theta$</td>
<td>Tri-axial compression surface linear term</td>
</tr>
<tr>
<td>$\theta_1$</td>
<td>Torsion surface linear term</td>
</tr>
<tr>
<td>$\theta_2$</td>
<td>Tri-axial extension surface linear term</td>
</tr>
<tr>
<td>$\lambda$</td>
<td>Tri-axial compression surface nonlinear term</td>
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<tr>
<td>$\lambda_1$</td>
<td>Torsion surface nonlinear term</td>
</tr>
<tr>
<td>$\lambda_2$</td>
<td>Tri-axial extension surface nonlinear term</td>
</tr>
<tr>
<td>$\rho$</td>
<td>Reinforcement ratio</td>
</tr>
<tr>
<td>$\tau_b$</td>
<td>Bond stresses between concrete and reinforcement along the beam axial direction</td>
</tr>
<tr>
<td>$\tau_{bf}$</td>
<td>Remaining bond stress at a large relative displacement of reinforcing bars to concrete along the beam axial direction</td>
</tr>
<tr>
<td>$\tau_{bmax}$</td>
<td>Maximum bond stress between concrete and reinforcement along the beam axial direction</td>
</tr>
<tr>
<td>$\tau_{bu,\text{split},1}$</td>
<td>Peak local bond resistance in the absence of confining stirrups</td>
</tr>
<tr>
<td>$\tau_{bu,\text{split},2}$</td>
<td>Peak local bond resistance in the presence of confining stirrups</td>
</tr>
<tr>
<td>$\psi_u$</td>
<td>Rotation of the slab outside the column region at punching shear failure</td>
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**Symbol**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
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<tbody>
<tr>
<td>$\Delta$</td>
<td>Deflection</td>
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INTRODUCTION

1.1 Background

An initial and localised damage of a critical structural element, usually caused by accidental loads such as vehicular impacts, gas explosions, terrorist attacks or unexpected events during construction, may trigger the collapse of the entire structure or a large portion of it. This type of failure, disproportionate to the original localised damage, is referred to as “progressive collapse” (ASCE 2010).

In recent years, progressive collapse has drawn wide-spread attention due to a number of catastrophic building collapses. This type of failure mode was firstly brought to the attention of engineering communities due to the partial collapse of the Ronan Point apartment tower in 1968 resulted from a relatively “small” gas explosion (Pearson and Delatte 2005). Shortly after this event, design guidelines and provisions against progressive collapse were issued in British and Canadian Standards. Since these events, growing concerns about terrorism threats, especially after the collapse of the Murrah Federal Building in 1995 and World Trade Centre towers in 2001 incurred by terrorist attacks, have further heightened this topic to an unparalleled level and increased the public awareness of the importance of preventing progressive collapse.

On the other hand, regardless of deliberate attacks, progressive collapse can also be triggered by natural hazards, accidents and construction errors. However, these extreme situations are not always considered in the original design of buildings, because additional design for accidental loading conditions, with a low probability of occurrence, is uneconomical in commercial and residential buildings. In contrast to this, progressive collapses can lead to catastrophic and devastating consequences resulting in human
casualties and economic losses, which were witnessed by a number of past collapse events. Given the characteristics of low probability yet high consequence of progressive collapse, challenges to develop a set of reliable approaches to evaluating the performance of the collapse resistance of existing structures, as well as economically refining current design guidelines against progressive collapse are imperative and timely to be addressed. Under such circumstances, progressive collapse has gained increasing interests from engineers and academics, along with substantial investigations aiming to advance its mechanism understanding and mitigating design strategies.

Concerning the existing studies on progressive collapse of concrete structures, the majority of efforts have been devoted to reinforced concrete (RC) frame structures. However, RC flat plate structures, which may be more vulnerable to progressive collapse owing to the absence of beams, have received less attention. Nowadays, RC flat plate structures are very popular in Australia and overseas, especially in the construction of residential high-rise buildings. Even though this structural system brings easy formwork installation, low construction cost, flexible location of services and a reduced floor-to-floor height, its vulnerability to punching shear failure presents a major deficiency. Punching shear failure is regarded as the most probable cause of an initial local damage that may propagate throughout the entire structure, eventually resulting in a total or a large portion of collapse of the structure (Hawkins and Mitchell 1979). Such a failure mode, i.e. progressive collapse of RC flat plate structures initiated by punching shear failure, has been demonstrated by the collapse of the 2000 Commonwealth Avenue condominium in 1971 (King and Delatte 2004), the collapse of the Pipers Row car park in 1997 (Wood 2003) and the collapse of the Sampoong department store in 1995 (Gardner et al. 2002).

To date, RC flat plate structures have been mainly investigated with respect to punching shear failure at isolated single slab-column connections. While extensive research on single slab-column connections pertinent to lateral restraints and post-punching performance has been carried out, they cannot, to some extent, account for the actual load redistribution in the RC flat plate system during a progressive collapse event. On the other hand, existing collapse investigations on multi-column RC flat plate structures have been nearly all limited to single column removal scenarios, but they have no representativeness
of more severe initial damages (i.e. more than one failed column). Moreover, while limited design guidelines (BS 2006; DoD 2009; ASCE 2010; GSA 2013; ACI318 2014) mainly provided by American and European codes deal with progressive collapse, no relevant regulations have been introduced in Australian Standards. The above shortcomings in exploring the progressive collapse behaviour and resistance of RC flat plate structures are the primary motivation of this study.

1.2 Research Methodology and Objectives

Among a number of progressive collapse studies, the alternate load path approach is likely the most popular research method due to its threat-independent characteristic (Keyvani et al. 2014). This approach allows an initial failure to occur while the remaining structure needs to seek alternate load redistribution paths to dissipate damage and prevent progressive collapse. According to the collapse-resistant design guidelines of DoD (2009) and GSA (2013), the collapse resistance of a structure can be examined following the alternate load path method by notionally removing one or several load bearing structural elements.

In an attempt to provide much-needed evidence for addressing the aforementioned issues and limitations (as described in Section 1.1), in this study, the progressive collapse behaviour of RC flat plate specimens under different column removal scenarios was investigated experimentally:

- a total of three 1/3-scale, 2×2-bay RC flat plate substructure specimens were constructed and tested by statically removing one or two columns at critical regions. Four experimental tests were performed: two consecutive corner column removal scenarios, an edge column removal scenario and an edge-interior-column removal scenario. These scenarios represent different locations and extents of initial damages. A quasi-static loading scheme was used to apply an increasing uniformly distributed load (UDL) to the slab until the collapse was reached. By doing so, the collapse resistance of the substructure was evaluated and the
structural behaviour in the failure and post-failure stages was recorded and discussed.

and numerically:

- A 3D nonlinear finite element modelling approach using LS-DYNA software was established, and five different numerical models were built to mimic the four experimental tests conducted in this study and one from the literature so as to predict the collapse response and resistance of the designed specimens. Firstly, the material models for concrete and reinforcements were calibrated, and appropriate bond-slip effects, failure criteria and contacts between reinforcing bars were explored based on the experimental observations and results. Then, the numerical models were validated through comparisons of the load-displacement responses, failure modes and crack development against experimental tests.

As a complement to the experimental study, an analytical study based on the observed yield lines in the experiment were also conducted to estimate the collapse resistance of the test specimens.

1.3 Research Significance

The research work presented in this study is likely to contribute to the current knowledge pool of progressive collapse of RC flat plate structures, because:

- For the first time, a total of four experimental tests were undertaken at large deformations under the loss of all critical columns, including two consecutive corner column removals, an edge column removal and an edge-interior-column removal, to gain a solid understanding of the progressive collapse behaviour of RC flat plate structures through the analyses of structural responses, collapse resisting mechanisms, load redistribution characteristics and failure modes.
- A set of innovative numerical modelling techniques was established and reasonably validated against experimental results. Detailed numerical results of internal forces, displacements and strains which cannot be easily obtained in the experiment therefore could be broadly supplemented. Future parametric
studies for examining the key parameters of improving the collapse resistance of RC flat plate structures become feasible. Moreover, valuable hints of building up the complex finite element model for collapse analyses of RC flat plate structures can be provided.

- Analytical predictions using a yield line theory were performed based on the experimental observations to verify the applicability of this method to the estimation of the flexural capacity of RC flat plate structures.

- To date, design guidelines against progressive collapse has been introduced in Europe and North America, none of relevant design regulations however can be found in Australian Standards. The research findings in this study, derived from the specimens which were designed in accordance with Australian Standards, can put forward significant insights into future Australian design regulations in mitigating progressive collapse of RC flat plate structures.

### 1.4 Thesis Organisation

This thesis consists of six chapters which are organised as follows:

- Chapter 1 states the research problem and its significance, for which the research methodologies and objectives are introduced.

- Chapter 2 explains the definitions of RC flat plate structures and progressive collapse in a detailed manner by highlighting their main features and setting out existing noteworthy investigations on them. Then, the relationship between them is clarified, and experimental, theoretical and numerical studies on progressive collapse of RC flat plate structures is reviewed.

- Chapter 3 experimentally investigates progressive collapse of RC flat plate substructures subjected to two consecutive corner column removal scenarios with different slab corner configurations, based on the published journal paper Ma et al. (2019b).

- Chapter 4 experimentally investigates progressive collapse of RC flat plate substructures subjected to an edge-column and an edge-interior-column removal scenarios, based on the submitted journal paper Ma et al. (2019a).
Chapter 5 firstly develops five 3D nonlinear finite element models based on the conducted experimental tests. Then, the numerical models are validated through comparisons of the load-displacement responses, failure modes, crack patterns, and displacement and strain results against the experimental ones, which confirms the appropriateness and reliability of the proposed modelling techniques. Finally, progressive collapse of RC flat plate structures is numerically investigated.

Chapter 6 summarises the main findings of this research and provides recommendations for future work.
LITERATURE REVIEW

2.1 Introductory Remarks

Firstly, this chapter introduces RC flat plate structures and progressive collapse separately with respect to the main characteristics and existing noteworthy investigations on them. After obtaining a solid understanding of these two aspects, the link between them and the necessity to carry out a more in-depth study are explained. Thereafter, state-of-the-art work on progressive collapse of RC flat plate structures is critically reviewed. The research gaps are therefore identified.

2.2 RC Flat Plate Structures

2.2.1 Overview

RC flat plate structures are a type of structural system that only consists of slabs and columns. A typical RC flat plate structure is shown in Figure 2-1. The slab has a uniform thickness and is normally reinforced in two orthogonal directions and directly supported by columns. Traditional structural elements such as beams, girders and drop panels do not exist in this system. This leads to noticeable economic and architectural advantages in terms of easy formwork, low construction cost and an increased floor-to-floor space for a given building height. Therefore, RC flat plate structures have become one of the most popular structural systems being extensively used in multistorey residential, office and industrial buildings, and car parks in the majority of Australian urban areas as well as many other regions worldwide.
2.2.2 Advantages and Major Issues

There are a few reasons that explain why RC flat plate structures have been frequently constructed for different building purposes. The principal feature of this system is that it provides the architecture with beamless floors, as a result, flat ceilings and flexibilities in partition and maintenance can be achieved. Through comparisons with traditional beam-column slab systems, non-functional ceilings are required in flat plates to accommodate the inherent beam protrusion, which in turn reduces construction cost. Moreover, formwork that occupies a major part of a construction budget, especially for RC structures, can save not only a great deal of construction cost but also much construction time by using flat plates. More importantly, the elimination of beams and girders in a flat plate system can generate more storeys being constructed when a restricted building height is given. In addition, it reveals a prominent convenience that suitable and sized openings are easier to be reserved for the purpose of service lines with the adoption of flat plates. Consequently, such advantages result in the flat plate structural system often providing the most economical solution among other construction types.
CHAPTER 2

However, there are also some problems that have posed inevitable limitations of using flat plates. The major issues of flat plates include poor deflection control, inadequate resistance to lateral loading and brittle punching shear failure at slab-column connections. The relatively low rigidity due to the absence of beams and girders and thin slab make the structures susceptible to large deformations and floor vibrations. Moehle et al. (1988) stated that the inherently low transverse stiffness of a flat plate has dramatically limited bending moment transfer, which can easily cause great structural damages under an earthquake loading. Therefore, the application of flat plates in seismic regions is restricted to mid-rise buildings which are normally provided with shear walls (Farhey et al. 1993). Moreover, slabs and columns are the mere structural elements of a flat plate structure resulting in the weakest area around the junction between the slab and the column, where a brittle type of punching shear failure is most likely to happen. This is because that all the gravity loads are eventually transferred through columns, in consequence, junction regions can exhibit severe stress concentrations generated by locally high shear forces and bending moments. A schematic view of a punching shear failure is indicated in Figure 2-2, where the column punches through the slab causing a truncated cone and surrounding diagonal cracks. On the other hand, although the flat plate structural system is exposed to several critical drawbacks in practice, these do not limit its widespread application. To date, there has been a considerable body of research investigating valid approaches to strengthening the slab-column connections and predicting the ultimate load capacity of flat plates. Such efforts, to some extent, can overcome the aforementioned drawbacks and limitations via appropriate design practice.


Figure 2-2 Punching shear failure
2.2.3 Improved Performance of Flat Plates

2.2.3.1 Punching Shear Strengthening Methods

Given that relative to flexural failure, a much lower load is normally required to cause punching shear failure, designing sufficient punching shear strength around slab-column connections is crucial. In an attempt to enhance the punching shear strength, there are several methods summarised by Pilakoutas and Li (2003):

- Increasing the overall thickness of the slab;
- Increasing the cross-section of the column;
- Increasing the safety factor corresponding to a given design load;
- Decreasing the span of the slab;
- Adding extra structural elements such as drop panels;
- Providing suitable shear reinforcement.

The first five methods provide solutions being expensive, impractical or architecturally non-applicable. Therefore, the final method, designing appropriate shear reinforcement, has been broadly adopted to prevent punching shear failure.

The most common form of shear reinforcement is to apply stirrups which were first introduced and investigated by Graf (1938) and later by Elstner and Hognestad (1956). The shear reinforcement system, in the shape of cages, has been commercially employed and prefabricated in many countries. However, it was found that the cage-shaped shear reinforcement is inconvenient and more time is required for it to be incorporated into the main flexural reinforcement owing to limited cover clearance and difficult anchoring operations. Moreover, Pilakoutas and Li (2003) pointed out that it is ineffective to use shear reinforcement made by steel bars, as many experimental investigations have proven that the failure in slabs is normally prior to the yielding of steel bars. Therefore, endeavours to develop many other types of shear reinforcement for flat plates have been undertaken.

An efficient solution to design shear reinforcement is to weld shear studs on a metal strip, which has been adopted in North America and Germany. Relevant experimental tests of the shear stud reinforcement were undertaken by Mokhtar et al. (1985). Another
noticeable shear reinforcement system is commonly called shear-band system which has been used in America, Europe and Japan. This system was put forward and experimentally studied by Pilakoutas and Li (2003), in which major benefits were identified, such as easy and efficient installation, minimal loss of concrete cover, applicability to thin slabs and high efficiency in shear resistance. In addition, it is known that fibre reinforcements can boost the material and mechanical properties, as well as control crack propagation for structures. Experimental tests regarding the effects of fibres on the punching shear resistance have shown that the steel fibres can dramatically improve the ultimate shear resisting capacity at slab-column connections by increasing their ductility, and the relationship between the resistance and fibre content is almost linear (Harajli et al. 1995). In the circumstances that numerous existing flat plates need adequate punching shear reinforcement for safety reasons, the approach to using post-installed shear reinforcements by installing steel bars into predrilled holes then bonding them by high-strength epoxy adhesive, can provide a successful solution (Ruiz et al. 2010). Moreover, a similar post-installed shear reinforcement in the form of steel bolts which were anchored by a washer-nut system can significantly enhance punching shear strength and retrofit existing slab-column connections (Adetifa and Polak 2005; Polak and Bu 2013).

2.2.3.2 Ultimate Load Capacity Enhanced by Membrane Actions

Over the last decades, extensive research has been conducted to predict the ultimate load capacity of flat plates by means of evaluating their punching shear strengths (Kinnunen and Nylander 1960; Moe 1961; Hewitt and Batchelor 1975; Muttoni 2008). It is noteworthy that the majority of these studies mainly focused on isolated and simply supported slabs. However, methods that use isolated and simply supported slabs without lateral restraints underestimate the ultimate load bearing capacity of flat plates mainly due to the neglect of compressive membrane action (Criswell 1974). Hon et al. (2005) stated that the existence of compressive membrane action can improve the overall stiffness of the slab and further increase the capacities in shear and flexure. Nowadays, it is well accepted that the punching shear strength and load bearing capacity in laterally
restrained RC slabs can be enhanced by introducing the compressive membrane action (Ockleston 1958; Vecchio and Tang 1990; Salim and Sebastian 2003; Ramos et al. 2011).

Figure 2-3 demonstrates the formation of the compressive membrane action in a typical flat slab. When the slab is subjected to a gravity load displayed in Figure 2-3 (a), the slab deflects accompanying with cracks on the concrete tension surface and reinforced bars being stretched. As shown in Figure 2-3 (b), the strains on the tension surface at the end and middle of the slab will be significantly bigger than those on the compression surface. Therefore, the net tensile strain at mid-depth motivates the slab to expand outward, causing lateral movement at the slab ends. To some extent, this potential movement is restricted by lateral restraint with respect to surrounding columns as well as boundary slabs. In consequence, the compressive membrane forces (marked as N in Figure 2-3 (c)) under the gravity load and laterally restrained conditions are generated, resulting in an increased flexural capacity in the slab exhibited in Figure 2-3 (d). Such a mechanism in response to the generated in-plane compressive forces is termed as the compressive membrane action known for considerably increasing the load carrying capacity of the slab. Furthermore, as the slab undergoes large deformations with concrete crushing and spalling, the tensile membrane action will be developed in the slab with the lateral restraint, analogous to the compressive membrane action.

Figure 2-3 Compressive membrane action demonstration: (a) slab subjected to gravity load; (b) slab elongates upon cracking; (c) restrained elongation induces axial compression; (d) axial compression increases flexural capacity (Vecchio and Tang 1990)
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The enhancement of the compressive membrane action on the load carrying capacity is mostly effective prior to the ultimate flexural capacity. Thereafter, the compressive membrane forces start reducing as the displacement continues increasing. At this stage, the cracks will gradually penetrate the full-depth of the slab accompanying with reinforcement yielding. Eventually, the applied load is mainly resisted by steel bars in the form of reinforcement net, which is termed as tensile membrane action. In contrast to the compressive membrane action which is induced by the lateral restraints of the slab against its outward movement, the tensile membrane action is developed due to the lateral restraints preventing the slab from moving inward. It is noted that continuous reinforcement and solid anchorage to the slab supports are also important to ensure the development of the tensile membrane action. Well-developed tensile membrane action can largely increase the deformation capacity of the slab as well as the post-failure load carrying capacity of the slab (Park 1964).

2.3 Progressive Collapse

2.3.1 Overview

In recent years, research aimed to investigate progressive collapse of building structures has attracted extensive attention due to many collapse events and the increasing concerns about terrorism threats. Worldwide representative building collapses, such as the collapses of the Ronan Point apartment tower in 1968, Murrah Federal Building in 1995, Sampoong department store in 1995 and Pipers Row car park in 1997, have unfortunately demonstrated the substantial consequences of progressive collapses on human casualties and economic losses. A typical formation of a progressive collapse can be distinguished into four stages which are demonstrated in Figure 2-4 (Wang et al. 2014).
Past progressive collapse events show that the response of structural elements after an initial damage to accidental loads is dynamic and nonlinear in both geometry and material aspects (Smilowitz 2002). In a typical collapse event, the sudden loss of one or more critical bearing elements motivates the structure to seek a new equilibrium position via redistributing the load that is supposed to be sustained by the lost element(s) to other elements. The generated geometric change will produce a superposition of internal static and dynamic forces, and inertia force representing the dynamic and nonlinear behaviours of the structure.

In this section, several well-known collapse events are reviewed by introducing the collapsed buildings, triggers and consequences. To mitigate progressive collapse of building structures, relevant regulations proposed by various codes and guidelines are briefly introduced. Apart from the design strategies against progressive collapse in these official documents, state-of-the-art findings from researchers can also be found hereafter.
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2.3.2 Historical Progressive Collapse Events

2.3.2.1 Ronan Point Apartment Tower, 1968

The progressive collapse of building structures first drew public attention after the partial collapse of Ronan Point apartment tower in Newham, East London (Pearson and Delatte 2005). The building was a large panel system consisting of precast concrete walls and directly supported floor slabs. On May 16, 1968 in the morning, a gas leak induced an explosion which blew out a supporting precast concrete wall near the exterior corner of the building on the 18th floor. The explosion caused significant loss of the vertical load bearing capacity of the upper floors, which subsequently led to a collapse of the floor corner area of the whole building (Figure 2-5). In this collapse failure incident, four people were killed and 17 injured.

It was reported by Griffiths et al. (1968) that the resistance of prefabricated vertical supporting walls was poor to lateral loading, causing the initial trigger as the reason of this collapse. Moreover, the collapse of the upper floors progressively happened mainly due to the lack of structural continuity and robustness. Although the collapse of Ronan Point apartment tower was not as significant as many other collapse events in recent years, this type of failure was firstly impressed by its disproportion to the initial damage. In this context, the first wave of research interests were inspired to propose the design provisions for resisting progressive collapse. Specifically, stipulations in UK building design codes related to the structural integrity were introduced following the Ronan Point apartment tower collapse.
Figure 2-5 Ronan Point apartment tower collapse (Pearson and Delatte 2005)

2.3.2.2 Murrah Federal Office Building, 1995

The second most tragic collapse was the Murrah Federal Office Building in downtown Oklahoma City, US on April 19, 1995 due to a terrorist attack (Osteraas 2006). The Murrah Federal Office Building was a conventional 9-storey RC frame structure designed in 1970. In the façade of the building, it had a continuous transfer girder on the third floor supported by principal columns at the first two levels with a spacing of 12.2 m. However, a truck loaded with bombs was exploded near one of the ground floor columns, resulting in the shear failure of three columns. The transfer girder then encountered flexural failure owing to the increased bending moment subjected to an increased span of 48.8 m of the transfer girder, which caused the collapse of approximately the half of the building. 168 people were killed in this event. Figure 2-6 demonstrates the Murrah Federal Office Building after collapse.
Prior to the collapse of Murrah Federal Office Building, it was commonly agreed that initial damage caused by accidental events has a low probability to trigger a collapse of the whole structure, and designing the resistance of progressive collapse for structures was not promising. Nevertheless since this collapse event, engineers and researchers have realised that with the development of industrialisation and the increasing demand on anti-terrorist, it is necessary to ensure the safety and stability of structures in terms of progressive collapse.

Source: https://www.firerescue1.com/fire-ems/articles/2156994-Oklahoma-City-bombing-What-happened-20-years-ago/

Figure 2-6 Murrah Federal Office Building collapse

2.3.2.3 Sampoong Department Store, 1995

The Sampoong department store located in Seoul, South Korea was a typical RC flat plate structure with floors directly supported by columns (Park 2012). This 5-storey building was opened in December, 1989 as a department store. It was witnessed that the collapse began with a punching shear failure around one column on the fifth floor, following which the collapse of the entire building (Figure 2-7) occurred on June 29, 1995. 502 people died and 937 people were injured in this collapse.

The investigation on this event revealed many potential causes of the collapse including design errors, poor management and supervision on quality control during construction,
function changes to the fifth floor and reduction of the columns’ cross-sectional areas on the fifth floor (Park 2012). More importantly, it was found that the most critical causes were the design and function changes during building maintenance.

![Figure 2-7 Sampoong department store collapse (Park 2012)](image)

2.3.2.4 *Pipers Row Car Park, 1997*

On the night of March 20, 1997, another progressive collapse attributed to punching shear failure happened. It was the Piper Row car park in Wolverhampton, UK, representing a RC flat plate construction where a punching shear failure occurred around one column on the top floor, eventually triggering a progressive failure of neighbouring columns (Wood 2003). Figure 2-8 shows a 120-tonne section of the top floor collapsed. As the building was still in construction and the collapse occurred at 3 am, no one was injured.

Although the design of Pipers Row car park basically complied with the regulations in CP 114, it was found that the standard yielded an over-prediction of the punching shear strength of RC flat plate structures (Wood 2003). Investigations also uncovered several important construction issues. Concrete strength was not consistent in structural elements with localised less strength than the target one, as some concrete was not batched with
enough cement, and not properly mixed and compacted. High degradation of concrete took place due to water and crystallisation of de-icing salts. Both of the slab top and bottom reinforcements were arranged with less concrete cover leading to severe corrosion of reinforced bars. Regardless of the above mentioned factors that caused a reduction in shear strength, imbalanced dead load subjected to the poor concrete compaction also increased shear stresses around some columns.

Figure 2-8 Pipers Row car park collapse (Adlparvar et al. 2006)

2.3.2.5 Other Collapse Events of RC Flat Plate Structures

While a surge of research interests were initiated by two of the most significant progressive collapses (i.e. Ronan Point apartment tower and Murrah Federal Office Building), more attention should be given to RC flat plate structures in resisting progressive collapse due to its popularity and relatively low robustness comparing to RC frame structures. Besides, punching shear failure, referred to as the most critical failure of RC flat plate structures, can make such structural systems more susceptible to progressive collapse. In addition to the aforementioned collapse events, progressive collapse of RC flat plate structures has been further evidenced by collapses of the 2000
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Commonwealth Avenue in 1971, Skyline Plaza Apartment in 1973 and Harbour Clay Condominium in 1981. It is noteworthy that the common feature of these tragedies was an initial local punching shear failure that spread from element to element, ultimately resulting in extensive damages. In this context, investigations on the collapse mechanisms and exploring effective collapse-resistant approaches for RC flat plate structures are much needed (King and Delatte 2004; Schellhammer et al. 2012).

2.3.3 Progressive Collapse Design Codes and Guidelines

In the history of progressive collapse events, they generally led to two waves of research interests, which were triggered by the collapses of the Ronan Point apartment tower in 1968 and World Trade Centre towers in 2001. These two events directly facilitated the generation and refinement of design codes and guidelines against progressive collapse. Crowder et al. (2004) summarised the timeline of the release of these design regulations following a few major progressive collapse events, as shown in Figure 2-9.

![Figure 2-9 Timeline of issuing several design regulations (Crowder et al. 2004)](image-url)

Figure 2-9 Timeline of issuing several design regulations (Crowder et al. 2004)
2.3.3.1 *Fundamental Design Methods for Resisting Progressive Collapse*

To date, the fundamental design methods to mitigate progressive collapse of building structures are shown in Figure 2-10. The indirect method denotes an implicit approach to improving the progressive collapse resistance of structures through the provisions of minimum levels of strength, continuity and ductility. This method mainly focuses on enhancing the strength of joint connections by adopting special detailing, further enhancing redundancy and ductility of a structure. This method has been incorporated into many design codes, considering that reinforcing robustness of structures can function under any abnormal conditions, thereby improving the overall structural performance. However, this method is not cost-effective without the specifications of abnormal loads and initial damages of structural elements.

![Diagram of Basic Design Strategies for Progressive Collapse Mitigation](image)

Figure 2-10 Fundamental design methods for resisting progressive collapse

The direct method consists of alternate load path method and specific local resistance method. The alternate load path method allows an initial failure to occur, while the remaining structure needs to seek alternate load paths to dissipate damage and prevent progressive collapse. Following this method, the progressive collapse resistance of a structure can be evaluated, by removing one or more load bearing structural elements then examining whether the remaining structure can resist subsequent failures. Since that this method is independent of abnormal loads, it is applicable to any types of hazards which
can cause structural member loss. The specific local resistance method is used to strengthen critical structural elements, as a result, these structural elements can survive from an abnormal loading. The common form of this method is to increase the design load factors.

Apart from preventing progressive collapse in aspect of a structure itself, another method is to reduce the exposure of structures to hazards with effective engineering hazard controls, from which initial element failure can be eliminated at the source. However, to some extent, this method cannot be implemented in reality due to the unpredictable nature of hazards.

The above design methods against progressive collapse display a condensed summary and a simply classification. Specifically, among existing design codes and guidelines, design methods with different emphases to reducing the susceptibility of structures to progressive collapse are briefly explained in the following section. Advantages and disadvantages of each method are also individually elaborated in Section 2.3.3.2 when introducing relevant design codes.

2.3.3.2 Existing Design Codes in Practice

ACI (American Concrete Institute) 318-14

ACI318 (2014) recommends to enhance the overall integrity of structures by adding minor changes to the reinforcement layout. The primary objective of this approach is to confine the initial element damage, so that the failure only propagates in a controlled area and the whole structure maintains stable. However, the problem regarding progressive collapse resistance of structures is not explicitly resolved, in particular as follows:

- Responses of a structure after losing one or more critical structural elements are not included.
- Stipulations with respect to loading and strength details which represent key factors for determining the risk of progressive collapse of building structures are not covered.
Simply increasing structures’ integrity with minimum reinforcement lacks considerations in the load redistribution, structural behaviours under large deformations and post-failure capacities, all which are critical to systematically understand and address progressive collapse of building structures.

**ASCE (American Society of Civil Engineers) 7-10**

ASCE (2010) maintains that it is not impractical to design structures that are capable of eliminating any collapses caused by abnormal loadings. Yet, avoiding or reducing the progressive collapse potential of building structures can be achieved through rational designs arresting the collapse in a small region. To this end, two design methods were proposed: direct design method and indirect design method which are the most prevalent design methods to date against progressive collapse. As aforementioned, the direct design method comprises the alternate load path method seeking new load equilibrium to absorb initial damage, and the specific local resistance method requiring sufficient strength to avoid initial damage at the level of local structural elements. In addition, the indirect design method emphasises on ensuring minimum strength, continuity and ductility in the design process to prevent the occurrence of progressive collapse.

Among the design methods above, the alternate load method included in the direct design method has been prominently adopted in the considerable research with respect to progressive collapse. Moreover, the precondition of this method is to identify the most critical structural elements that may lead to a large portion of collapse of the structure due to their losses. In this regard, the progressive collapse resistance of structures can be assessed by evaluating the capacity of the remaining structure after notionally removing one or more critical structural elements.

**Eurocode EN 1991-1-7**
BS (2006) requires horizontal and vertical ties to be provided in the building in order to mobilise all structural members during an accidental event by means of the indirect design method. Horizontally, ties should be arranged around the perimeter of each floor and roof of the building as well as column and wall regions. Vertically, ties should be continuously arranged along the columns from the foundation to the roof level.

Furthermore, the identified critical structural elements should be able to withstand an abnormal load of 34 KN/m² in both horizontal and vertical directions by means of the specific local resistance method. This value is subjected to the accidental load of the gas explosion from the collapse of the Ronan Point apartment tower. Principles for systematic risk assessment regarding local failure due to accidental actions, such as vehicular impacts and explosions, are also covered in this code.

**Department of Defense (DoD) United Facilities Criteria (UFC) 4-023-03**

DoD (2009) has been amended several times since the first issued version in 2005. It provides design guidelines to reduce the risk of progressive collapse of new and existing building structures. Regulations in this design guideline are applicable to all buildings with three or more storeys, as these buildings are more susceptible to progressive collapse. Pertinent design approaches for reducing the possibility of progressive collapse defined in ASCE (2010) are also incorporated in DoD (2009). The indirect design method increases the progressive collapse resistance of structures through specifying minimum tensile forces in order to tie the structure together (shown in Figure 2-11), namely increasing structures’ continuity, ductility and redundancy. This approach via employing horizontal and vertical ties is similar to that in BS (2006). The direct design method includes alternate load path method and specific local resistance method which explicitly reflect progressive collapse during the design process. The alternate load path method requires that the structure can sustain after a loss of one or more structural members and locally confine the damage. The specific local resistance method requires that the structure or structural elements can resist a defined load. Additionally, analysis procedures such as linear static, nonlinear static and nonlinear dynamic are adopted in this design guideline.
While DoD (2009) has developed detailed and complete design provisions, key issues are still exposed and need to be further investigated including:

- Adoption of connection modelling parameters based on seismic loading cases is inappropriate to simulate the connection behaviour under the column removal scenarios.
- The post-failure capacity or the residual load carrying capacity of a structural system under large deformations is not identified.
- Contribution of slabs in resisting progressive collapse is not addressed, which is important to provide a more realistic evaluation in the design process.

![Figure 2-11 Tie forces in a frame structure (DoD 2009)](image)

**General Services Administration (GSA)**

Analysis procedures and design guidelines to resist progressive collapse for new and existing federal buildings have been recommended in GSA (2013). To assess the applicability of using collapse-resistant guidelines, a flow chart indicated in Figure 2-12 is proposed to identify the necessity of evaluating the potential of progressive collapse of building structures. The design requirements in GSA (2013) is mainly based on the
alternate load path method, which is also widely employed in DoD (2009). The scenarios to implement this method include removing column on the perimeter at the middle of the long or short side of the building, corner and interior locations at the ground level, using linear static, nonlinear static, linear dynamic and nonlinear dynamic analyses. It is noted that a load combination of $2DL + 0.5 LL$ is suggested to apply for the above column removal scenarios on each whole floor area, where DL is dead load and LL is live load. In addition, an acceptance criteria of Demand-Capacity Ratios (DCR) is specified in the design guidelines in GSA (2013) to identify the potential of progressive collapse of building structures. The demand and capacity that conform to structural members are determined by the elastic static analyses in moments and axial forces. With distinguishing structures by regular and irregular buildings, the allowable DCR values are $DCR < 2.0$ for the former and $DCR < 1.5$ for the latter.
2.3.4 Brief Review of Existing Progressive Collapse Studies of Frame Structures

As introduced previously, the column loss scenario is a common tool to assess the collapse resistance of building structures by means of the alternate load path method. This method is also suggested by several design guidelines such as DoD (2009) and GSA (2013). Thus, extensive research activities including in-situ tests, laboratory tests and analytical studies on investigating progressive collapse of RC frame and steel structures under removing
one or more load bearing structural elements took place. Among these studies, most recent literature is summarised in this section.

2.3.4.1 RC Frame Structures

Of the three main resisting mechanisms developed in RC beams above the removed column, catenary action at large deformations is regarded as the most prominent mechanism to mitigate progressive collapse, comparing to flexural action and compressive arch action (Sasani and Kropelnicki 2008; Alogla et al. 2016). Regan (1975) pointed out that the tensile strength and ductility of structural members provided by the longitudinal reinforcement of beams are the keys to motivate catenary action. According to Orton (2007), further findings revealed that catenary action in a beam starts to be involved when a deformation equivalent to about the depth of the beam is achieved. Apart from numerous experimental studies on RC sub-assemblages subjected to different column removal scenarios (Yi et al. 2008; Su et al. 2009; Choi and Kim 2011; Yap and Li 2011; Yu and Tan 2012, 2013a; Lim et al. 2017), theoretical studies have been conducted (Izzuddin et al. 2008; Li et al. 2011), full-scale tests (Sadek et al. 2011; Sheffield et al. 2011) and in-situ tests (Sasani et al. 2007; Sasani et al. 2011) representing more realistic scenarios were also carried out.

Along with substantial studies on progressive collapse of RC frame structures, constructive methods to increase the collapse resistance have arisen. Orton (2007) proposed a scheme to resist progressive collapse by strengthening the beam with carbon fibre reinforced polymer (CFRP), which could in turn increase structures’ continuity and ductility. In addition, CFRP has major advantages of easy fabrication, simple installation and long service duration. Yu and Tan (2013b) suggested a new detailing technique that involves an additional reinforcement layer placed in the middle depth of beams, bottom reinforcement partial de-bonding and partial hinges, enhancing the collapse resistance of RC frame structures through catenary action mechanism.
2.3.4.2 Steel Frame Structures

In the past decades, investigations on progressive collapse have been not only limited to concrete structures, but also passed to steel structures. To understand the resisting mechanisms, determine the collapse resistance and eventually amend current design approaches regarding progressive collapse of steel frame structures, the alternate load path method has been substantially adopted (Khandelwal and El-Tawil 2007; Sadek et al. 2010; Sadek et al. 2011; Yang and Tan 2013a, 2013b). These studies all demonstrated that under column loss scenarios, flexural action is only prominent at an early stage, while tensile failure dominated by catenary action controls the final failure modes of steel frame structures at collapse deformations.

Catenary action is generally incorporated in beams and connections, and largely influenced by rotation capacities and ductility. Therefore, to ensure the full development of catenary action in resisting progressive collapse, the performance of welded connections under column loss scenarios has been examined by Karns et al. (2009), Sadek et al. (2010) and Kim and Yu (2012). Moreover, due to the simplicity of bolted steel connections, they have been popularly used in Europe, however, also exposed a higher vulnerability than the welded connection to progressive collapse. Yang and Tan (2012) and Yang and Tan (2013c) experimentally explored the performance of bolted-angle connections and found that this type of connections can increase the ductility and load carrying capacity.

So far, considerable research has been committed to progressive collapse of frame structures, whereas relevant efforts on RC flat plate structures are still scarce. Next section will be devoted to reviewing progressive collapse of RC flat plate structures, i.e. the main focus of this study.

2.4 Existing Studies on Progressive Collapse of RC Flat Plate Structures

Comparing to frame structures, RC flat plate structures are inherently vulnerable to punching shear failure resulting from the absence of beams which can aid in distributing
the load. In a circumstance of one or more columns damaged due to exceptional loads, high shear forces and bending moments will be locally generated, leading to severe stress concentrations in the surrounding slab-column connections. This could trigger the failure propagation horizontally and further vertically, eventually turning out an entire collapse of the structure or a large portion of it. Collapse events of the Ronan Point apartment tower in 1968, Sampoong department store in 1995 and Pipers Row car park in 1997 have demonstrated such failure modes. Therefore, RC flat plate structures are likely the most susceptible structural system to progressive collapse, which comes to an agreement with Hawkins and Mitchell (1979) and Keyvani et al. (2014).

Previous literature reviewed in this thesis introduces RC flat plate structures and progressive collapse separately, which are essential to understand these two aspects and the link between them. In an attempt to see a more specific picture of progressive collapse of RC flat plate structures, the most relevant and state-of-the-art work on this research topic is presented in this section. Given that extensive research activities on progressive collapse have been given to frame structures and collapse studies on RC flat plate structures are still limited, investigations on frame structures are briefly reviewed whist these on RC flat plate structures are reviewed in a more detailed manner.

2.4.1 Experimental Studies

Qian and Li (2012b) clarified the slab effects on the vertical force (gravity load) resistance under the loss of a corner column. Two series of specimens, referred to three complete scaled-down RC frame substructures and three incomplete scaled-down RC beam-column sub-assemblages as illustrated in Figure 2-13, were constructed and tested to quantify the contribution of the slab to the total load carrying capacity of RC frame structures against progressive collapse. In this study, all specimens were loaded monotonically at the removed corner column in a static scheme. While the structural behaviour was captured and analysed in aspects of load-displacement curves, crack patterns and failure mechanisms, the performance of the slab resisting the applied load was assessed. The result showed that with the existence of the slab, the load carrying capacity was increased by up to 63% relative to the load resistance of RC beam-column
sub-assemblages. This indicates that the potential of progressive collapse can be remarkably reduced with the interference of the slab in a typical RC frame structure.

Even though the conducted experimental work was aiming at investigating RC frame structures against progressive collapse, the emphasis on the slab impact spontaneously attracted curiosities towards the performance of RC flat plate structures against progressive collapse. What is more, Qian and Li (2013d) put forward a strengthening scheme for RC flat slabs resisting collapse by applying fibre-reinforced polymers (FRPs). The main innovation of the proposed strengthening method is the availability to existing building structures, easy fabrication and mounting, and long service duration.
Two identical scaled-down, 2 × 2-bay RC flat plate substructures were experimentally studied by Yi et al. (2008) under the loss of an interior column (Z5 in Figure 2-14), and the loss of exterior column then corner column (Z4 and Z9 in Figure 2-14), respectively. It was advocated that using a quasi-static loading scheme is appropriate to simulate the sudden loss of a column in spite of the dynamic and strain-rate effects, as it is easier to capture the slab behaviours and load redistributions comparing to a dynamic loading scheme. According to the dynamic increased factor of two recommended by GSA, a uniformly distributed load (UDL) was increased until reaching twice the design load, then statically releasing the load from the mechanical jack until zero. To evaluate the residual capacity of the specimen after the UDL, a downward concentrated load was applied at the removed interior column. With the same loading scheme used in the first specimen, the second specimen lost its bearing capacity at an applied load of 1.78 and 1.17 times the design load subjected to the loss of exterior column and corner column, respectively, without applying the downward concentrated load. The test results provided valuable insights that RC flat plate structures are more vulnerable to progressive collapse following the loss of an exterior or corner column relative to an interior column. The analytical
study indicated that the actual load carrying capacity of the substructure was higher than the estimation based on the yield line theory.

Figure 2-14 Experimental loading scheme and measurement: (a) specimen plan view; (b) loading scheme under an interior column loss; (c) loading scheme under an exterior/corner column loss (Yi et al. 2008)

Russell et al. (2015) tested seven 1/3-scale RC flat plate substructures statically and dynamically subjected to different column removal scenarios including corner column, penultimate edge column and internal edge column. It is noted that for simplicity, there were no rotational and translational restraints provided in all tests instead of simply supports at slab edges, which was different from actual boundary conditions. A plan view of the experimental setup can be seen in Figure 2-15. The findings obtained from the test results are summarised as follows:

- It is applicable to monitor the dynamic response of RC flat plate structures under the sudden loss of column removal scenario with assistance of a high speed camera.
- The alternate load path can be developed when RC flat plate structures have spans in two directions. It was observed that punching shear failure was more prominent than flexural failure.
The considerable residual capacity was found after the elastic stage mainly due to material and geometric nonlinearities.

The dynamic increased factor of two proposed by GSA was found to be conservative. It was also found that investigations on progressive collapse of RC flat plate structures using a static loading scheme are adoptable, as the dynamic effects was observed less significantly.

In order to understand the collapse mechanisms of RC flat plate structures, Qian and Li (2015) proposed a series of experimental tests under an interior column removal, with design variables of drop panels and post-installed shear bolts. It is noteworthy that a load distribution tree (item 9 in Figure 2-16) were adopted to simulate an increasing UDL which represents a more realistic loading method investigating progressive collapse (Alashker et al. 2010). Moreover, all specimens had 350 mm extended overhangs which were loaded by steel weights (item 7 in Figure 2-16) to simulate the continuity of the surrounding slabs. An overview of the experimental setup is demonstrated in Figure 2-16. The test results revealed that the propagation of punching shear failure could be a major
issue resulting in a progressive collapse of RC flat plate structures. The post-failure load carrying capacity was found after the first peak load mainly due to the development of dowel and tensile membrane actions. By contrasting the specimens strengthened by drop panels and post-installed shear bolts, it was found that adding drop panels could largely increase both flexural and punching shear capacities of RC flat plate structures. However, using post-installed shear bolts led to an increased capacity in punching strength but less effectiveness in post-failure capacity.

Figure 2-16 Schematic view of the specimen (Qian and Li 2015)
With the demonstration of the collapse of the Murrah Federal Office Building in 1995 where the truck loaded bomb resulted in the failure of three columns, building structures may suffer more severe initial damages caused by accidental events beyond the failure of only one column. In this context, Qian et al. (2018) constructed and tested two identical 1/4-scale, 2 × 2-bay RC flat plate substructures reinforced with drop panels, to simulate different extents of initial damages, by simultaneously removing one column (interior) and two columns (interior and edge) in each test. Figure 2-17 graphically presents the experimental configurations of the two tests. It was witnessed that with a total weights of 12.5 kPa applied to the slab, no punching shear failure and collapse occurred in both tests. Such experimental work filled the blank of the progressive collapse study of RC flat plate structures mostly limited to single column removal.
2.4.2 Theoretical Studies

One of the earliest investigations on progressive collapse of flat plates was undertaken by Hawkins and Mitchell (1979). Multiple factors which could initiate and propagate a collapse in flat plates were identified. It was found that punching shear failure at an interior slab-column connection is possibly the primary cause attributed to a collapse. The
factors influencing punching shear strength include loading period, edge restraints, rotational demands and bending moments. Whether the initial failure will spread to other structural members or not depends on the capability of the moment transfer of flat plates. Continuous bottom reinforcement through columns was recommended as the effective approach to increasing continuity and ductility of flat plates, namely moment transferring abilities. In addition, the other possible defence-lines preventing progressive collapse are the mobilization of tensile membrane action, torsional strips and higher design loads.

In order to prevent the occurrence of punching shear failure in flat plates, many researchers have proposed various approaches to predicting the punching strength of slab-column connections. One of the earliest predicted models were suggested by Kinnunen and Nylander (1960) which was derived from a series of experimental tests of circular slabs loaded along the edge and centrally supported by circular columns. This model was further improved by Broms (1990) where the size effect was incorporated. Furthermore, Muttoni (2008) proposed a critical shear crack theory to determine the punching strength of slab-column connections as a function of several parameters of the slab. The derived equations are presented in the following (Muttoni 2008):

\[ \frac{V_u}{b_0 d \sqrt{f_c'}} = \frac{9}{1 + 15 \frac{\psi_u d}{d_{g0} + d_g}} \]

where \( V_u \) is the punching shear strength of the slab; \( b_0 \) is the control perimeter of the column; \( d \) is the effective depth of the slab; \( f_c' \) is the concrete compressive strength; \( \psi_u \) is the rotation of the slab outside the column region at punching shear failure; \( d_{g0} \) is the reference aggregate size; \( d_g \) is the maximum diameter of the aggregate. The slab rotation \( \psi_u \) is calculated as:

\[ \psi_u = 1.5 \frac{r_s f_y}{2d E_s} \left( \frac{V}{V_{flex}} \right)^{1.5} \]

where \( r_s \) is the radius of circular isolated slab; \( f_y \) is the yield strength of reinforcement; \( E_s \) is the modulus of elasticity of reinforcement; \( V \) is shear force; \( V_{flex} \) is the shear force at flexural failure. Comparing the predicted results using the critical shear crack
theory to a series of 87 experimental test results showed that the the coefficient of variation between these two types of results was only 8%.

Ellingwood (2006) developed a set of procedures to classify and determine different abnormal loading scenarios, to examine the survivability of generally designed building structures under a localised damage, and to assess and reduce the potential of progressive collapse. A framework aiming at strategically dealing with a balance between risk reduction and challenges in reality was established using principles of probabilistic analyses and structural reliability. By doing so, assessment on existing collapse-resistant design regulations in national codes (American, Canadian and European Standards) was performed. The probability of progressive collapse of a structure due to an abnormal event in Ellingwood (2006) is defined as:

\[
P[\text{Collapse}] = P[\text{Collapse/LF}]P[\text{Collapse/E}]P[E]
\]

where \( P[\text{Collapse}] \) is the probability of progressive collapse of a structure; \( P[\text{Collapse/E}] \) is the probability of occurrence of local failure; \( P[E] \) is the probability of occurrence of an abnormal event.

In recent years, the yield line theory, which is proposed by Park and Gamble (2000) and competent to predict the load carrying capacity of the slab at bending failure, has been employed to analytically estimate the collapse resistance of the slab. According to Park and Gamble (2000), the ultimate bending moment per unit width of the slab reinforcement \( m_u \) is expressed as:

\[
m_u = A_s f_y \left( d - 0.59 A_s \frac{f_y}{f_{c'}} \right)
\]

where \( A_s \) is the area of tension reinforcement per unit width, \( d \) is the slab effective depth, \( f_y \) is the yield strength of the slab reinforcement, and \( f_{c'} \) is the concrete compressive strength of the slab.

Yi et al. (2014) employed the yield line theory to predict the load carrying capacity of the RC flat plate specimens. Figure 2-18 shows the depicted yield line patterns based on the
experimental observations subjected to interior, edge and corner column removal scenarios. Given the equilibrium of external and internal work and observed yield lines, the predicted load carrying capacity of the slab under interior column removal scenario \( (q_u) \) was derived from the following equation (Yi et al. 2014):

\[
8m_u \left[ \frac{(R_b/R_a) \sin \alpha}{2} + \frac{\sin \beta}{2} \right] + \frac{4m_b' L_b}{R_b} + \frac{8m_{a1}' L_{a1}}{R_a} + \frac{4m_{a2}' L_{a2}}{R_a} = \frac{2}{3} q_u [(2L_{a1} + L_{a2})R_a + L_b R_b]
\]

where \( m_{a1}' \) and \( m_{b}' \) denote the negative bending moment resistances per unit width in the middle strip along \( L_{a1} \) and \( L_b \); \( m_u \) denotes the negative bending moment resistance per unit width in the column strip along \( L_{a2} \); \( R_a \) and \( R_b \) denote the distance from the interior column to the observed negative yield lines, as shown in Figure 2-18(a); \( L_{a1} \), \( L_{a2} \) and \( L_b \) denote the length of the observed negative yield lines, as shown in Figure 2-18(a); \( \alpha \) and \( \beta \) denote the angles between the observed positive yield lines, as shown in Figure 2-18(a). Similar derivations to predict the load carrying capacity of the slab under edge and corner column removal scenarios were also implemented based on the yield line patterns illustrated in Figure 2-18(b) and (c). The predicted results showed that the load carrying capacities were underestimated by 26.8% and 9.6% under interior and edge column removal scenarios, respectively, and overestimated by 19.7% under corner column removal scenario.
Figure 2-18 Yield line patterns under: (a) interior column removal; (b) edge column removal; (c) corner column removal (Yi et al. 2014)

Qian and Li (2015) examined the applicability of the yield line approach to the estimation of the slab capacity under interior column removal scenario. The observed yield line pattern is displayed in Figure 2-19. The external and internal work is expressed as:
\[ \sum W_E = q_u \frac{2\delta}{3} (r_1 h_1 + r_2 h_2) \quad 2.6 \]

\[ \sum W_I = 4\delta \left[ m_N^s \left( \frac{r_1}{h_1} + \frac{r_2}{h_2} \right) + 2m_P^s \left( \frac{h_1 \sin \alpha}{2} + \sin \frac{\beta}{2} \right) \right] \quad 2.7 \]

where \( \sum W_E \) and \( \sum W_I \) denote the total external and internal work done by the applied load and yield lines, respectively; \( q_u \) denotes the applied load; \( \delta \) denote the virtual displacement at the removed interior column; \( r_1, r_2, h_1, h_2, \alpha \) and \( \beta \) are marked in Figure 2-19; \( m_N^s \) and \( m_P^s \) denote the negative and positive bending moment resistances per unit width. The analytical result showed an underestimation of 22.6\% against the experimental result, due to the existence of compressive membrane action.

Figure 2-19 Yield line pattern under interior column removal (Qian and Li 2015)
2.4.3 Numerical Studies

Due to the limitations of the experimental test including space, execution and costs, the inherent advantages of numerical simulations on probing progressive collapse of RC flat plate structures have motivated engineers and researchers to devote a great deal of efforts.

Kang et al. (2009) established a numerical model that could simulate the nonlinear behaviours of slab-column connections resulting from two shake table tests. The comparisons between experimental and numerical results indicated that the model is able to estimate different stages of the flexural yielding of the slab due to unbalanced moment transfer. The successful development of this model proved that numerically simulating the slab-column connections under dynamic loading is feasible and also provides future numerical simulations on the flat plate system with a useful reference.

On the basis of the isolated slab-column tests without lateral restraints performed by Mirzaei and Muttoni (2008), Keyvani et al. (2014) proposed a finite element model which is capable of simulating both punching and post-punching behaviours of the slab-column connections. To investigate the effect of compressive membrane action, lateral restraints were added to the numerical model. The friction shear force induced by compressive membrane action was found to be able to largely increase the punching shear strength. However, current design codes and guidelines neglect the contribution of compressive membrane action when estimating the punching shear strength of flat plates. By applying the same numerical modelling technique to a flat plate system, it was found that the progressive collapse resistance of flat plates with the aid of compressive membrane action had a lower vulnerability comparing to those without it. It was also claimed that continuous bottom reinforcement in the slab passing through columns plays a dominant role in enhancing the post-punching capacity.

Previous to 1989 in American Concrete Institute (ACI) design codes, there was no pertinent requirements on the slab shear reinforcement and continuous bottom reinforcement passing through columns. For these flat plate buildings, the tensile membrane action regarded as one of the most important mechanisms resisting progressive collapse can only be little developed. In this context, a macro model was proposed by Liu et al. (2015a) and used to assess the vulnerability of older flat plate structures to
progressive collapse. Figure 2-20 displays each component of the macro model. To simulate the flexural behaviour of the slab and punching shear failure, shell and connector elements were jointly used, respectively. The proposed macro model was validated through 24 experimental tests performed on isolated slab-column connections under multiple loading schemes. By applying this macro model to a system level of the older flat plate structure using both static and dynamic loading schemes in Liu et al. (2015b), it was revealed that this type of structures has a higher possibility to encounter progressive collapse after suddenly losing one column. Moreover, losing an exterior column in older flat plate structures exposed a greater risk of progressive collapse relative to losing an interior column.

![Macro model](image)

Qian et al. (2018) using a 3D nonlinear finite element modelling approach performed by LS-DYNA software simulated the dynamic process of multi-column RC flat slab structures reinforced with drop panels. With the validation of the proposed numerical models by comparing the vertical and horizontal displacements, strains and crack patterns against the experimental ones, further numerical studies were carried out by increasing the applied UDL beyond the design service load in order to predict the ultimate dynamic...
state of the test specimens. It was found that the dynamic ultimate load carrying capacities of RC flat slab substructures under the interior column removal and edge-interior-column removal were 1.92 and 1.18 times their design service load, respectively. Various dynamic column loss scenarios in RC flat slab substructures were further numerically analysed, which included a single column removal of corner, exterior edge and interior, and a double column removal of exterior edge and interior, and exterior edge and corner. It was indicated that the scenario of concurrently removing exterior edge and corner columns may cause more severe failure (shown in Figure 2-21) than any mentioned column removal cases above.

![Figure 2-21 Simulated failure mode of RC flat slab substructures under the column missing of corner and exterior edge columns (Qian et al. 2018)](image)

2.5 Concluding Remarks

Existing work on RC flat plate structures has been substantially conducted over the last decades. Yet, the major issues of RC flat plate structures against progressive collapse are still not well addressed or at an initial stage. This is mainly because:

- A considerable research body of flat plates has been devoted to isolated slab-column connections or slab-column sub-assemblages without considering slab effects.
• Studies on the ultimate load capacity of flat plates only involve compressive membrane action subjected to a relatively small deformation, which is controversial to apply to estimate collapse capacities. The most recent investigations have revealed that the post-failure stage under large deformations can provide non-ignorable residual capacities to assist in resisting progressive collapse.

• Little studies on failure mechanisms at the post-failure stage including load redistributions and tensile membrane action have been done.

• For simplicity, some of existing experimental tests adopted inappropriate boundary conditions (e.g. simply support and non-rotational restraints), which is not equivalent to a realistic scenario.

This chapter identifies the research gaps of the current progress in investigating progressive collapse of RC flat plate structures, which motivates this study to provide the knowledge pool with additional experimental, numerical and analytical evidences.

The next two chapters introduce a comprehensive experimental program which includes large-deformation collapse tests of RC flat plate substructures under critical column removal scenarios, such as single corner column, single edge column, and double interior and edge columns. This experimental program aims to obtain a solid understanding of the progressive collapse behaviour of RC flat plate structures, as well as collect invaluable experimental data for analytical studies and developing numerical models.
CHAPTER 3

EXPERIMENTAL STUDY ON THE PROGRESSIVE COLLAPSE BEHAVIOUR OF RC FLAT PLATE SUBSTRUCTURES SUBJECTED TO CORNER COLUMN REMOVAL SCENARIOS

Statement of contribution to co-authored published paper

This chapter includes a co-authored and peer reviewed paper. The bibliographic details (published status) of the co-authored paper, including all authors, are:

Note that the format of this paper has been changed according to the guidelines of the thesis, and supplementary experimental details which were not included in this paper for publication now have been provided in “APPENDIX A, B and C” for references.

My contribution to this paper involved: literature review, experimental design, experimental setup, experimental data collection, data analyses, discussion of test results, development of analytical study, paper writing and responding to reviewers.

The contribution of this published paper to the advancement of the research data: for the first time, two experimental tests undertaken at large deformations under two consecutive corner column removals to gain a solid understanding of the progressive collapse
CHAPTER 3

behaviour of RC flat plate structures through the analyses of structural responses, collapse resisting mechanisms, load redistribution characteristics and failure modes.

(Signed) _________________________________ (Date)  24/10/2019
Fuhao Ma

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Experimental study on the progressive collapse behaviour of RC flat plate substructures subjected to corner column removal scenarios

ABSTRACT

Reinforced concrete (RC) flat plate structures are broadly used in car parks, residential and office buildings due to their economic and architectural advantages. However, this structural system is inherently prone to punching shear failure, which may propagate horizontally and vertically, ultimately leading to the progressive collapse of the entire structure or of a large portion of it. This paper presents the experimental results from two quasi-static large-displacement tests performed on a 1/3 scale, 2×2-bay, RC flat plate substructure subjected to corner column removal scenarios. The specimen was tested twice with different corner reinforcement configurations: (i) firstly, one corner column, with torsional strips, was removed and the Uniformly Distributed Load (UDL) on the bay adjacent to the removed column was increased to failure (Test T1), and (ii) secondly, as the damage was concentrated in the vicinity of removed corner column in (i), the corner column diagonally opposite to the first removed one, without torsional strips, was removed. The UDL on the bay adjacent to the second removed column was also increased to failure (Test T2). Different failure and post-failure behaviours, failure modes, and collapse resisting mechanisms between the two tests were witnessed, presented and analysed. Results show that 80% to 110% of the applied load is transferred to the two edge columns adjacent to the removed corner column throughout the entire two tests. The ultimate load carrying capacity for T1 is found to be 1.7 times smaller than the one for T2.

Keywords

Flat plate structure; Punching shear; Progressive collapse; Corner column removal.
3.1 Introductory Remarks

Initial local damage of a critical load bearing structural element, usually caused by design and construction errors or abnormal loading events (e.g., blasts, terrorist attacks, vehicular impacts or earthquakes), may propagate through the structure and ultimately lead to the collapse of the entire building or of a large portion of it. This type of structural failure, disproportionate to the initial damage, is referred to as “progressive collapse” (ASCE 2010). Although the progressive collapse of buildings has a low frequency of occurrence, the resulting human casualties and economic losses could be catastrophic. Since the partial collapse of the Ronan Point tower block in 1968, when a gas leak induced an explosion near the exterior corner of the 18th floor, ultimately leading to the collapse of the whole corner of the building, improving design guidelines to effectively prevent progressive collapse in buildings has gained considerable interests (Pearson and Delatte 2005).

The fundamental design approaches recommended by collapse-resistant design guidelines (DoD 2009; GSA 2013) to mitigate the progressive collapse of buildings can be distinguished into either indirect or direct design methods. To date, the direct Alternate Load Path (ALP) design method is likely to be the most popular one due to its threat-independent characteristic (Keyvani et al. 2014). Specifically, the ALP method allows the load carrying capacity of a structure to be examined by notionally removing one or several load bearing structural elements. In line with the ALP design method, a number of in-depth studies on the progressive collapse resistance of reinforced concrete (RC) structures under a column removal scenario have been conducted analytically (Li et al. 2011; Kim and Yu 2012; Qian and Li 2013a), experimentally (Yi et al. 2008; Dat and Tan 2013, 2014; Qian et al. 2015; Ren et al. 2016; Lu et al. 2017) and numerically (Li et al. 2016; Pham et al. 2017a; Pham et al. 2017b). However, these studies have predominately focused on frame systems. Relevant investigations on RC flat plate structures, a commonly used structural system, are still scarce (Qian and Li 2012b, 2012c, 2013b, 2014; Yi et al. 2014; Qian and Li 2015; Russell et al. 2015; Peng et al. 2017; Xue et al. 2018).

In RC flat plate structures, the slab is directly supported by columns without beams, drop panels and column capitals. This leads to easy formwork, low construction cost and an
increased floor-to-floor space for a given building height. These noticeable economic and architectural advantages have turned flat plates into one of the most popular structural systems worldwide. It is broadly used in car parks, residential and office buildings. However, when compared to frame structures, flat plates are inherently prone to punching shear failure resulting from the absence of beams which assist in distributing the load (Loo and Guan 1997; Park and Gamble 2000; Ruiz et al. 2010). In the event of a column loss, due to either punching shear failure or abnormal loads, high shear forces and bending moments will be locally created at the adjacent slab-column connections, leading to high stress concentrations at these locations. In this context, the failure could propagate horizontally and vertically (Hawkins and Mitchell 1979; Mirzaei and Sasani 2011). Three major progressive collapse events of RC flat plate structures have unfortunately demonstrated the tragic reality of such failure modes, namely the 2000 Commonwealth Avenue condominium collapse in 1976 (King and Delatte 2004), the Pipers Row car park collapse in 1997 (Wood 2003) and the Sampoong department store collapse in 1995 (Gardner et al. 2002).

The majority of existing investigations on RC flat plate structures have focused on isolated single slab-column connections (Criswell 1974; Melo and Regan 1998; Keyvani et al. 2014). While studies on single slab-column connections pertinent to either lateral restraints (Rankin and Long 1987; Salim and Sebastian 2003) or post-punching performance (Mirzaei and Muttoni 2008; Miguel et al. 2013; Habibi et al. 2014) have been carried out, they cannot to some extent, account for the actual load redistribution paths of the RC flat plate system in a progressive collapse event. With respect to the progressive collapse research on multi-column flat plate substructures, experimental studies on scaled RC flat plate substructures under an interior column removal scenario were carried out by Yi et al. (2014), Qian and Li (2015) and Xue et al. (2018). A sudden edge column loss scenario was experimentally investigated by Russell et al. (2015) and Peng et al. (2017) to simulate the dynamic responses of RC flat plate structures. Qian and Li (2012c) and Qian and Li (2014) conducted static and dynamic loading tests on scaled RC flat plate substructures subjected to a corner column loss. It is noteworthy that losing a corner column is likely the most vulnerable scenario to cause progressive collapse of RC flat plate structures. This is because the secondary resisting mechanisms (e.g., compressive and tensile membrane actions) largely provided by the lateral restraints
cannot be effectively developed under a corner column removal scenario (Qian and Li 2012b). Additionally, periphery columns of a building have the highest likelihood of being exposed to accidental loadings, such as vehicle collision. To enhance the progressive collapse resistance of RC flat plate structures subjected to a corner column loss, Qian and Li (2012c) proposed a series of strengthening schemes by bonding carbon-fibre-reinforced polymer (CFRP) laminates to the slab. Besides, Qian and Li (2013b) experimentally quantified the effect of drop-panels on the progressive collapse resistance of RC flat plate structures and indicated that reinforcing this type of structure with drop panels can considerably reduce the occurrence of progressive collapse.

In actual RC flat plate structures, the periphery of the slab is typically strengthened by torsional strips (or spandrel beams) to enhance punching shear strength of the edge and corner columns (AS3600 2009), and sometimes a flat plate may be constructed with overhangs serving as balconies. However, studies on these aspects, which may bring uncertainties into estimating the collapse capacity of RC flat plate structures, are still lacking. Thus investigating the failure behaviour of RC flat plate structures with torsional strips or overhangs is necessary. Experimental investigations aiming at capturing the structural behaviour of the flat plate substructure under a corner column removal scenario and different corner reinforcement configurations are presented in this paper. The experimental results were obtained from two quasi-static large-displacement tests performed on a 1/3 scale, 2×2-bay, RC flat plate substructure. The scaled substructure was tested twice: (i) firstly, one corner column was removed and the Uniformly Distributed Load (UDL) on the bay having the removed column was increased (Test T1), and (ii) secondly, as minimum damage spreading outside the loaded bay was observed after the first test, the corner column diagonally opposite to the first removed column on the same specimen was also removed (Test T2). The load in T2 was also applied by increasing the UDL on the bay adjacent to the second removed column. For T1, the edges of the slab next to the removed column were flush with the columns and were reinforced with torsional strips. For T2, the edges of the slab next to the removed column had a 500 mm overhang from the column centre and had no torsional strips. This paper firstly describes the experimental design and setup. Secondly, the failure and post-failure behaviours, failure modes and collapse resisting mechanisms of the tested specimen are presented and discussed. The load redistributions, crack patterns, displacements and
strain results are also analysed. Finally, yield line theory is employed to estimate the slab flexural capacity for both T1 and T2 cases.

### 3.2 Experimental Program

3.2.1 Specimen Design

3.2.1.1 General

The prototype structure on which the test specimen is based is a 4-storey and 4×4 bay RC flat plate car park, designed in accordance with the Australian Standard on Concrete Structures AS3600 (2009). In the prototype, the RC slab is directly supported by 450 mm square columns that are spaced at a centre-to-centre distance of 6 m in both directions. The height between storeys is 3 m. The design dead load acting on the prototype structure is 6.7 kPa, due to a slab thickness of 270 mm; the design live load is 5 kPa, attributable to a car park under medium vehicle traffic conditions.

Given a limited space in the laboratory, a 1/3 scaled, 2×2-bay, substructure extracted from the ground floor of the prototype was constructed and tested. The authors acknowledge that the test results obtained from the scaled specimen will exhibit a size effect when compared to the full-scale prototype, especially for brittle punching shear failure (Bazant and Cao 1987; Birkle and Dilger 2008). Nevertheless, this will not influence the load redistribution pattern, which is deemed as one of the most significant mechanisms of progressive collapse, and the overall behaviour of the slab. Published experimental studies on progressive collapse of RC flat plate structures reviewed in the previous Section 3.1 “Introduction” adopted similar scale factors ranging from 1/4 to 1/2.34. Also note that to minimise the size effect, the aggregate size was reduced to 6 mm in the 1/3 scaled specimen instead of a typical aggregate size of 20 mm in full-scale structures.

Figure 3-1(a) illustrates the extracted region of the substructure encircled by the shaded rectangle. A perspective view of the test specimen with column numbering is shown in Figure 3-1(b), where the slab had an overall dimension of 4,575 mm × 4,575 mm × 90
mm. Columns had a scaled height of 1,000 mm and were spaced at a centre-to-centre distance of 2,000 mm. In the test specimen, the edges along C3-C9 and C7-C9 had two 500 mm overhangs to simulate the continuity effects of the adjacent slabs. Yet the periphery edges along C1-C3 and C1-C7 were flush with columns, which resembles the flush appearance of the prototype. During casting of the specimen, column C1, which was to be removed in T1, was replaced by a temporary steel column with a rectangular hollow section (RHS). Column C9, which was to be removed in T2, was similar to all other columns and was only cut off prior to applying the UDL in T2 (see Section 3.2.3 “Loading Scheme”).

![Figure 3-1 Specimen details: (a) substructure extraction from the ground floor of the prototype structure; (b) extracted specimen (Unit: mm)](image)

3.2.1.2 Steel Reinforcement and Concrete Slab

The slab reinforcing bars of the test specimen should be proportionately scaled down relative to the prototype structure. However, as the required reinforcing bar size was not commercially available, the same reinforcement ratios as those of the prototype structure were adopted for the test specimen. N8 (8 mm diameter and N ductility class (D500N8)) reinforcing bars, with measured (i) yield strength of 564 MPa, (ii) yield strain of 3200 με, (iii) ultimate strength of 786 MPa, and (iv) fracture elongation of 14.9%, were used for
the top and bottom slab reinforcements. The stress-strain curve of N8 slab reinforcement can be seen in Figure 3-2. Detailed slab reinforcement size, spacing, ratio ($\rho$) and layout are provided in Table 3-1 and Figure 3-3(a) and (c). Note that torsional strips (detailed in Figure 3-3(b)) along the slab edges (C1-C3 and C1-C7) were designed to comply with the specifications in AS3600 (2009). The slab had a mean concrete compressive strength of 43.5 MPa at 28 days, measured from six cylinder tests performed in accordance with AS1012.1 (2014). The nominal clear concrete cover of the slab was 10 mm for both top and bottom layers of reinforcements.

![Stress-strain curve of slab reinforcement](image)

Figure 3-2 Stress-strain curve of slab reinforcement

### 3.2.1.3 Columns

As typically occurring in construction, the columns were cast first (at least 2 weeks before the slab) and positioned on the strong floor before pouring the slab. The columns had reinforcing bars sticking out to penetrate the slab which reached 80 mm above the top surface of the slab to form a column stub with a height of 90 mm including a 10 mm concrete cover. The top face of the columns was at the same elevation as the bottom surface of the slab formwork. In order to avoid undesirable failure occurring in the
columns, a higher concrete compressive strength than the slab (measured as 58.6 MPa at 28 days from six cylinder tests) and an overdesigned reinforcement ratio of 6.8 % were used for the columns. Figure 3-3(d) illustrates an elevation view of a column and its cross-section with reinforcement details. As described above, a 90 mm high column stub was fabricated for each column on the top of the slab to form vertical continuity of the column. A 90 mm high bottom column stub underneath the slab was also fabricated at the removed column C1 in T1. In addition, the bottom of each column was reinforced with a 220 mm high 100UC14.8 steel universal column, itself welded to a 20 mm thick end steel plate, which was used to bolt the specimen to the test rig. This created an enlarged cross-section of 200 mm x 200 mm at the bottom of each column to fit the 100UC14.8, which was ribbed using shear connectors to bond to the concrete. Supplementary information of experimental design and preparation can be found in “APPENDIX A and B”.

Table 3-1 Reinforcement details

<table>
<thead>
<tr>
<th>Strips</th>
<th>Divided strips</th>
<th>Top reinforcement</th>
<th>Bottom reinforcement</th>
<th>Stirrup</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column strip (CS)</td>
<td>①</td>
<td>N8@110</td>
<td>ρ = 0.49%</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>②</td>
<td>N8@160</td>
<td></td>
<td>N/A</td>
</tr>
<tr>
<td>Middle strip (MS)</td>
<td>③</td>
<td>N8@250</td>
<td>ρ = 0.28%</td>
<td>N/A</td>
</tr>
<tr>
<td>Edge strip (ES)</td>
<td>④</td>
<td>N8@180</td>
<td>ρ = 0.61%</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>⑤</td>
<td>N8@85</td>
<td></td>
<td>R4.75@90</td>
</tr>
</tbody>
</table>

(Note: strips ①–⑤ are graphically shown in Figure 3-3 (a). The width of divided strip ① is equal to twice the slab overall depth plus the column width over which at least 25% of the top reinforcement of the column strip (① + ②) has to be distributed (AS3600 2009)).
Figure 3-3 Reinforcement layout and dimensions of the slab and column: (a) slab top reinforcement; (b) torsional strip details; (c) slab bottom reinforcement; (d) column dimensions and reinforcement details (Unit: mm) (Note: ST and SB are strain gauges glued to the top and bottom slab reinforcements, respectively.)

3.2.2 Test Setup

3.2.2.1 UDL Applied to the Slab

Imposing an increasing UDL either on the entire slab or on those bays adjacent to the removed column is being typically used to evaluate the collapse resistance of flat plates (Yi et al. 2014; Qian and Li 2015; Peng et al. 2017). In this study, the UDL was applied to the slab in two different ways. Firstly, a six-point loading tree (described later in this section) was positioned on the 2,000 mm × 2,000 mm bay having the corner column removed and connected to a 250 kN capacity and 600 mm stroke servo-controlled MOOG hydraulic jack. The loading tree with a connection to the hydraulic jack enables the UDL on the bay to be increased until failure of the slab. Secondly, a constant design live load of 5 kPa was applied to the remaining three bays of the slab with overhang(s) through uniformly laying out steel blocks, each with a nominal mass of 125 kg. This loading arrangement follows the design procedure recommended by DoD (2009), in which the UDL acting on the bay(s) adjacent to the removed column is increased relative to the other bays. Figure 3-4 provides an overview of the applied UDL for T1. It is noted that for T2, no constant UDL of 5 kPa was applied to the damaged bay C1-C2-C5-C4, as further discussed in Section 3.2.3 “Loading Scheme”.

Regarding the loading tree, the two ends of the hydraulic jack were mounted on swivels to prevent shear forces being transferred to the hydraulic jack. To evenly transfer the concentrated load from the hydraulic jack to the six loading points, the loading tree was comprised of one horizontal steel RHS beam and two 50 mm thick triangular steel plates. The steel RHS beam was simply supported between the centres of gravity of the steel plates, i.e., one end was allowed to slide through slotted holes manufactured in the support. The load was applied by the hydraulic jack to the mid-span of the steel RHS beam. To
ensure a stable loading system, the supports of the steel RHS beam were set below the point of application of this jack load. Two swivels were also used to connect the steel RHS beam to its supports, therefore allowing each steel plate to move independently from each other and follow the deformation of the slab. The three corners of each triangular steel plate were fitted with ball-socket joints (made of a steel ball connected to the steel plate and a 50 mm diameter polyvinyl chloride (PVC) round socket pad with a low friction coefficient) to transfer the load vertically to the slab. To spread the load under each loading point and avoid local failure of the slab, a 300 mm × 200 mm, 20 mm thick, steel plate was positioned below each ball-socket joint. Furthermore, to prevent the development of horizontal forces in the slab induced by the loading tree, the horizontal displacements of one loading point of each steel plate were restrained while the remaining two points were allowed to slide in either X- or Y- direction only. Figure 3-5 illustrates the features of the loading tree.

Figure 3-4 Overview of T1 setup
Figure 3-5 Loading tree: (a) positioned loading tree on the slab; (b) layout of loading points (Unit: mm)

3.2.2.2 Instrumentation

The behaviour of the slab was monitored by custom-made load cells bolted underneath each column, strain gauges (SG), linear variable displacement transducers (LVDT) and rotation transducers (RT). Each custom-made load cell consisted of four independent
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strain gauged steel circular hollow section (CHS) elements which were rigidly bolted to two steel plates (shown in Figure 3-3(d)). The tension and compression axial forces and bending moments resisted at the column bases were measured by the load cells throughout T1 and T2. Further details of the design of the custom-made load cells can be found in Xue et al. (2018). A more detailed introduction of measurements of strains, forces, displacements and cracks is provided in “APPENDIX C”.

Strains for both T1 and T2 were measured by means of 25 strain gauges (ST1-ST6 and SB1-SB19 for T1, and ST7-ST12 and SB20-SB35 for T2) with 3 mm gauge length and 10 strain gauges (CT1-CT6 and CB1-CB4 for T1, and CT7-CT12 and CB5-CB8 for T2) with 90 mm gauge length glued to the steel reinforcing bars and surfaces of the concrete slab, respectively. Strain gauge numbering and locations for each test are given in Figure 3-3(a) and (c) for the reinforcing bars and in Figure 3-6 for the slab surfaces.

The displacement transducers for both T1 and T2 consisted of 11 (V1-V11 for T1 and V12-V22 for T2) wire transducers connected below the slab measuring the vertical displacements of the slab, and two laser transducers (H1-H2) measuring the horizontal displacements of the slab. In addition, four rotation transducers (R1-R2 for T1 and R3-R4 for T2) were mounted on the slab next to columns and on the top of columns to record the relative rotation between the slab and either column C2 or C6. Locations and numbering of all transducers for each test are shown in Figure 3-6.

Due to safety concerns when monitoring the crack developments during Tests T1 and T2, different procedures were implemented to monitor the top and bottom surface cracks of the slab at key stages. For the top crack monitoring, cracks were drawn by an operator walking on the slab while being attached to a safety harness. Photos were also manually taken by the operator. For the bottom crack monitoring, a camera was mounted on a group of tracks parallel to the slab edge enabling the photos of the entire bottom surface to be remotely taken without people going underneath the slab. Using appropriate lighting, even hairline cracks were able to be captured.
3.2.3 Loading Scheme

In this study, a quasi-static loading scheme was applied throughout T1 and T2. The loading scheme is divided into four distinct phases detailed below:

*Phase 1 – Use of temporary support:* The portal frame (Item 1 in Figure 3-4) was positioned to be aligned with C1-C7 for T1. The hydraulic jack was positioned and pin-
connected to the top of the removed column stub. The connecting device is displayed as Item 4 in Figure 3-4. This column stub was lifted by less than 1 mm to remove the underneath temporary steel RHS column. The hydraulic jack was then moved back to its original position and kept at this position in displacement control to simulate an undamaged structure. For simplicity in T2, instead of using the hydraulic jack, a slightly tensioned steel chain was connected from a bridge crane to C9. The column C9 was then cut off and the bridge crane was used as the temporary support.

**Phase 2 – Live load and loading tree positioning:** The slab was loaded with steel blocks on the other three bays with overhang(s) to simulate the constant live load, and the loading tree, weighing 8.7 kN, was placed in the bay with removed column to simulate increased UDL. Note that the portal frame was laterally moved to the middle span of this bay to reposition the hydraulic jack for the loading tree. Note also that for T2, the steel blocks were not positioned in bay C1-C2-C5-C4 which was already damaged during T1.

**Phase 3 – Release of jack-bearing force:** In this phase, the column reaction force transferred to the hydraulic jack (T1) or to the bridge crane (T2) in Phase 2 was released to 0 kN quasi-statically.

**Phase 4 – Increase of UDL to failure:** The portal frame and the hydraulic jack were positioned above the loading tree. The load cells of the hydraulic jack and of each column, all displacement transducers and strain gauges (see Section 3.2.2.2 “Instrumentation”) were initialised to zero. The hydraulic jack was then pin-connected to the loading tree and the UDL on the slab was increased by driving the hydraulic jack in displacement control. Displacement loading rates for T1 were varied in different stages as below:

- At 1 mm/min up to the displacement of the removed column C1 reaching 143 mm, corresponding to well-developed cracks.
- At 3 mm/min up to the displacement of C1 of 307 mm, corresponding to the load reaching a plateau and failure well developed in the slab.
- At 5 mm/min till the end of the test, i.e., at the displacement of C1 being 409.5 mm.

Displacement loading rates for T2 were also varied in different stages as below:
• At 1 mm/min up to the displacement of the removed column C9 reaching 85 mm, corresponding to the occurrence of punching shear failure at C8.
• At 3 mm/min up to the displacement of C9 of 145 mm, corresponding to the occurrence of punching shear failure at C6.
• At 5 mm/min till the end of the test, i.e., at the displacement of C9 being 475.2 mm.

Note that T2 was conducted 12 days after T1 due to installation and connection of required experimental instruments, and cutting of column C9. The tests were not repeated as the results were shown to be repeatable in Xue et al. (2018).

3.3 Test Results and Discussion

3.3.1 General

Only experimental results arising from Phase 4 (Increase of UDL to failure) are presented and discussed herein. Note that for T1, 2.5 kN was transferred to the hydraulic jack during Phase 2. For simplicity, only displacements and strains at selected key locations are analysed and discussed. To capture the crack developments (see Section 3.2.2.2 “Instrumentation”), T1 and T2 were paused at key stages at which the drops of the applied load, inherent to pausing a test driven in displacement control, are not showed in the load-displacement curves for clarity.

3.3.2 Overall Behaviour

3.3.2.1 Test T1

Figure 3-7 plots the applied load versus column C1 displacement (V11) for T1. Two prominent load resisting stages can be identified, before and after the first peak load (FPL) (see Figure 3-7):
Stage 1 – Flexural action: Up to the FPL, at the onset of the applied load of 85.1 kN and the C1 displacement of 92.8 mm (i.e., approximately one slab depth), the flexural action is regarded as the sole resisting mechanism with opening of small cracks at the top surface of the slab and yielding of the top reinforcements near C2 and C4. Specifically, at the beginning of Stage 1, a linear relationship between the load and displacement can be witnessed in Figure 3-7, up to about 40 kN and the corresponding C1 displacement of 7.1 mm when initial cracks (IC) were observed to develop on the top surface of the slab (Figure 3-8(a)). At the IC, the cracks originated from C2 and C4, and propagated towards C5. A non-linear relationship is then observed, corresponding to cracks further propagating from C2 and C4 to ultimately form complete quarter crack rings at the FPL, as shown in Figure 3-8(b). From the experimental observations and loading history, it is speculated that when quarter rings merged, several load drops of 2.1 kN, 4 kN and 5.1 kN at displacements of 13.6 mm, 21.8 mm and 48.5 mm, respectively were resulted. It is noted that a quarter crack ring propagated beyond the column removal bay C1-C2-C5-C4 to bays C2-C3-C6-C5 and C4-C5-C8-C7. Additionally, positive crack lines (Figure 3-8(d)) at the slab soffit developed from the C1 displacement of 21.8 mm. These positive crack lines are mainly running diagonally, parallel to line C1-C5. After reaching the FPL, almost no new positive crack lines were observed, instead existing cracks became wider as the displacement continued to increase.

Stage 2 – Bending failure: The second stage was initiated by a 5.1 kN drop right after reaching the FPL, corresponding to the formation and opening of a new crack ring on the top surface of the slab, located closer to C1 than previously developed rings, as drawn in Figure 3-8(c) in comparison to Figure 3-8(b). Subsequently, cracks along this ring further developed to ultimately penetrate through the entire thickness of the slab corresponding to (i) a sudden 9.1 kN drop of the applied load at the C1 displacement of 106.2 mm, and (ii) yielding of the bottom reinforcement (SB4 and SB17, as described later in Section 3.3.5 “Strain Development”). Subsequently, the applied load gradually decreased with an increased displacement of C1, matching with cracks opening up further on the top surface (Figure 3-9(a)) and concrete crushing and spalling on the bottom surface (Figure 3-9(b)) along the cracks. The distances measured from column C1 to the full-depth flexural cracks were 1,345 mm and 1,350 mm, respectively, along the two edges of the slab (Figure 3-9(a)), which are roughly equivalent to the top reinforcement cut-off positions.
for C2 and C4 (1,370 mm). At the C1 displacement of about 305 mm when the crack width was sufficiently large, the corner portion of the slab with the removed column C1 was only connected to the remaining substructure by the bottom reinforcement. Afterwards, the detached corner rotated in a rigid body motion around this large crack ring at a relative constant applied load of 39 kN, on average, till the end of the test. According to above experimental observations and results, it is apparent that the failure was mainly localised by the formation of a negative yield line, along which the full-depth flexural cracks developed ultimately.

3.3.2.2 Test T2

Due to the 500 mm overhang along C3-C9 and C7-C9, a higher load was applied during T2, when compared to T1, to reach failure. Also due to different slab configurations, a different structural behaviour than T1 was observed for T2. It can be summarised in three distinct failure stages:

Stage 1 – Flexural action: Prior to the FPL, the same load resisting mechanism was witnessed in T2, relative to T1. At about 40 kN, the linearity between the load and displacement ended with the initial cracks (IC). The initial stiffness of 8.7 kN/mm for T2 was found to be 1.5 times higher than the one for T1. A similar crack pattern to T1 was observed with the cracks starting from C6 and C8, and propagating towards C5 (Figure 3-8(a)). At an applied load of about 56 kN, the load started to increase relatively steadily by about 1 kN/mm up to the FPL at 119 kN. The crack pattern on the top surface of the slab at the FPL for T2 is depicted in Figure 3-8(b) and shows that contrary to T1, the ring cracks in T2 mainly developed in the two adjacent bays. This is likely because the reinforced overhangs, providing continuity of the slab, contributed to resisting the applied load by involving the adjacent bays. The cracks at the slab bottom surface in T2 at the FPL present a similar pattern to T1 but are denser and longer, primarily due to the larger positive bending moment. The load drop of 28.5 kN at the FPL with the corresponding C9 displacement of 74.3 mm was the result of punching shear failure occurring at C8, as shown in Figure 3-10(a).
Stage 2 – Post-FPL: After punching shear occurred at C8, the applied load increased again until punching shear occurred at C6 with the corresponding C9 displacement of 140.6 mm. Specifically, at the beginning of Stage 2, the applied load increased at 1.39 kN/mm to the C9 displacement of 99.9 mm, following that, punching-like cracks around C6 and C5 developed, corresponding to a relatively constant applied load. When punching shear failure ultimately occurred at C6, the applied load dropped by 24.7 kN. From this stage onwards, the load was resisted by flexural and tensile membrane actions, while dowel action allowed C8 and C6 to continue carrying the load. After the C9 displacement of 200 mm, the load increased steadily up to the post-failure peak load (PPL) at a rate of 0.16 kN/mm. The formation of the full-depth flexural cracks was firstly witnessed at the C9 displacement of about 240 mm, indicating a decreased contribution of the flexural action, relative to the tensile membrane action, in resisting the load. The distances measured from column C9 to the full-depth flexural cracks were about 1,310 mm and 1,350 mm, respectively, along the two edges of the slab (Figure 3-10(a)), being roughly equivalent to the top reinforcement cut-off positions for C6 and C8 (1,370 mm). Unlike T1, the load carrying capacity continued to increase to the PPL of 143.3 kN after the formation of the full-depth flexural cracks.

Stage 3 – Post-PPL: As the relative displacement between the damaged columns (C6 and C8) and the slab increased, the bottom reinforcing bars passing through these columns were bent considerably. At the PPL, the bottom reinforcing bar running between C6 and C9 fractured (Insert for C6 in Figure 3-10(a)) with a load drop of 20.7 kN. Thereafter, the full-depth cracks were further developed from the slab edge to the interior, which was accompanied by the concrete splitting and the bottom reinforcing bars ripping out of the concrete cover (Figure 3-10(b)). After the PPL, the applied load gradually decreased with an increased displacement of C9 until the test was terminated at the displacement of 475.2 mm. The final crack patterns of T2 can be seen in Figure 3-8(c) and (d).
Figure 3-7 Load-displacement curves for T1 and T2 (Note: IC, FPL and PPL denote initial cracks, first peak load and post-failure peak load, respectively.)
Figure 3-8 Crack patterns at top and bottom surfaces of the slab: (a) top surface cracks at IC; (b) top surface cracks at FPL; (c) top surface cracks at final stage; (d) bottom surface cracks at final stage (Note: T1 and T2 cracks are shown with dotted red and plain black lines, respectively.)
Figure 3-9 Failure modes of T1: (a) top view; (b) bottom view
3.3.3 Load Redistribution

The reaction force at each support column was acquired during the two tests through the custom-made load cells. The results for each column are plotted versus the displacement of the removed column in Figure 3-11(a) for T1 and in Figure 3-12(a) for T2. On the other hand, Figure 3-11(b) and Figure 3-12(b) plot the percentages of the applied load distributed to different column groups for T1 and T2, respectively, namely “adjacent” columns (C2 and C4 for T1 and C6 and C8 for T2), “interior” column (C5 for both T1 and T2) and “boundary” or “other” columns (C3, C6, C7, C8 and C9 for T1, and C2, C3, C4 and C7 for T2).

Figure 3-11 and Figure 3-12 show that 80% to 110% of the applied load is transferred to the two edge columns adjacent to the removed corner column throughout the entire two tests. A similar finding was found in Xue et al. (2018). In Stage 1 of both tests, 10% to 20% of the applied load was transferred to the interior column C5, while about the same amount was resisted by the boundary columns, but in tension. For T1 and after reaching the FPL, the load resisted by the interior and boundary columns gradually decreased to zero, resulting in the adjacent edge columns carrying the entire applied load. In T2, different behaviours were observed, with the load resisted by the interior and boundary columns being relatively constant after reaching the FPL. Still for T2, after punching
shear occurred at C8, about 12% of the applied load was directly transferred to the interior column C5 and another 5% to boundary columns C2, C3, C4 and C7.

During the whole process of load redistributions of the two tests, up to 12.4% in T1 and 18.6% in T2 of the loads were taken in tension by the boundary columns as shown in Figure 3-11 and Figure 3-12, relative to the total reaction forces. Therefore, the loads transferred to the boundary columns were not negligible, showing the importance of considering the adjacent bays in the experimental tests to fully replicate the actual collapse behaviour of the slab.
Figure 3-11 Load distribution to columns for T1: (a) individual column distribution; (b) percentage distribution
3.3.4 Slab Deflection

The readings of the vertical displacement transducers located along the diagonal lines C1-C5 for T1 and C9-C5 for T2 are selected in this section to show the slab deflection at critical stages. Figure 3-13 shows that up to FPL for T1 and PPL for T2, the entire bay deforms in a parabolic profile due to the bending deformation imposed to the slab. After these stages and due to the development of the yield lines (between transducer V5 and V7 for T1, and V16 and V18 for T2), any further deformation of the bay is mainly located between the yield line and the removed column.

Figure 3-12 Load distribution to columns for T2: (a) individual column distribution; (b) percentage distribution
Figure 3-13 Slab deflection at key locations along the diagonal line of: (a) T1; (b) T2
3.3.5 Strain Development

Figure 3-14 presents the strain results developed in the slab top reinforcing bars for both T1 and T2. The locations of strain gauges on the top reinforcement are depicted in Figure 3-3(a), where the strain gauges for T1 and T2 are symmetrically positioned with respect to the interior column C5, and located at the edge (ST3-ST6 for T1 and ST9-ST12 for T2) and interior (ST1-ST2 for T1 and ST7-ST8 for T2) column regions adjacent to the removed column. When comparing the strains from Gauges ST1, ST2, ST3 and ST5, which stayed below the yield strain $\varepsilon_y$, the strain readings from Gauges ST4 and ST6 (along and parallel to slab edges) increased rapidly up to the FPL and the bars yielded at the C1 displacement of 37.8 mm. This indicates that the top reinforcing bars contributed to carrying the applied load are mainly the ones perpendicular to the yield lines and located along the slab edges C4-C1 and C2-C1. A similar behaviour was noticed in T2, for which the strains of Gauges ST10 and ST12 also increased rapidly prior to the FPL to reach the yield strain at C9 displacements of 38.2 mm and 51.4 mm, respectively. Contrary to T1, and likely due to different load resisting mechanisms after the FPL, both the strain readings of ST7 and ST8 (located near the interior column C5) exceeded the yield strain after punching shear occurred at C6 and increased significantly to about 16,100 $\mu \varepsilon$ at the PPL. The strain values of Gauges ST9 and ST11 (located near the edge columns C6 and C8, respectively) increased gradually and became approximately constant after the C9 displacement reaching 325 mm.
Figure 3-14 Top reinforcement strain in: (a) T1; (b) T2 (Note: $\Delta_{FPL}$, $\Delta_{PPL}$, $\Delta_{P6}$ and $\Delta_{P8}$ represent displacements at the FPL, PPL, punching shear failure at C6 and punching shear failure at C8, respectively.)

Figure 3-15 illustrates the strain readings of the slab bottom reinforcement along the diagonal lines C1-C5 for T1 and C9-C5 for T2 versus the vertical displacement of the removed column. The locations of the strain gauges on the bottom reinforcement are
depicted in Figure 3-3(c). The strain gauge readings from SB1-SB4 and SB20-SB23 (all located between the yield lines and the interior column C5) follow the same trend, with the strains being larger when the gauges are located closer to the yield line (SB4 and SB23). For T1, the reading of Gauge SB5, located between the yield line and the removed column C1, showed a low strain of 1,288 με due to the rigid body motion of the failed cantilevered portion of the slab. However for T2, due to the tensile membrane action (see “Stage 2” in Section 3.3.2.2 Test T2), the reading of Gauge SB24, also located between the yield line and the removed column C9, showed yielding of the reinforcing bar with a maximum strain value of 4,466 με. Moreover, the strain in the bottom reinforcement along the interior column C5 and edge columns (C4 for T1 and C6 for T2) is found to be considerably higher for SB25 (T2) relative to SB6 (T1), emphasising that the adjacent bays were involved more in resisting the applied load in T2.

(a)
The strain readings measured by the gauges on the bottom reinforcement located along lines C1-C4 in T1 and C9-C6 in T2, and glued to the bars perpendicular to the slab edge, are plotted in Figure 3-16. The strain readings for T1 show that none of these bottom reinforcing bars yielded throughout the test. On the contrary, the strain readings for T2 show that after punching shear occurred at C6, all gauges recorded a significant increase, mainly attributable to the effect of tensile membrane action.
Figure 3-16 Bottom reinforcement strain perpendicular to slab edges and orientated along: (a) C1-C4 in T1; (b) C9-C6 in T2

The remaining strain gauges (SB12-SB19 for T1 and SB31-SB38 for T2) on the bottom reinforcement are used to measure the strain distributions along the integrity bars passing through the edge and corner columns with the gauge locations shown in Figure 3-3(c) and the strain results plotted in Figure 3-17. For T1, it is witnessed that the perimeter integrity bars are in tension, and elongated within the yielding range, except for the strain measured from Gauge SB17 which was mounted at the edge of top reinforcing bar. The strain reading from Gauge SB17 increase rapidly at about the FPL, well coinciding with the experimental observations that large slab bending occurred at the full-depth flexural cracks (near the edge of top reinforcing bars shown in Figure 3-9(a)). For T2, the large strain readings from Gauges SB32 and SB36, symmetrically located relative to column C6, are also due to slab bending. At the interface of the slab-column C6 connection, gauge SB34 was broken just prior to the punching shear failure of column C6, possibly due to the relative displacement between column C6 and the slab.

The strain readings from Gauges SB24, SB26, SB27, SB37 and SB38 in T2 were all found to exceed the yield strain after the C9 displacement reaching to about 234 mm, proving that the tensile membrane action in the cantilevered portion of the slab was induced by the two-way tensile reinforcement net.
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Figure 3-17 Bottom reinforcement strain passing through edge and corner columns and parallel to: (a) C1-C4 in T1; (b) C9-C6 in T2

Figure 3-18 and Figure 3-19 show the concrete strain variations of all gauges (shown in Figure 3-6) on the slab top and bottom surfaces, respectively. To accurately reflect the strain measurements, the gauge readings are only presented up to the displacement of the removed column being 90 mm, as it was observed that the strain gauges were either
located across crack openings or isolated by cracks during tests. For T1, the slab top surface was in tension at the column regions (CT1, CT4 and CT5) and the interior slab region (CT2). The strain reading of Gauge CT5 (near column C2) increased rapidly at the initial stage of the loading, agreeing well with the crack pattern at the IC (shown in Figure 3-8(a)). For T2, the strain development shows a very similar pattern to T1, but at a higher intensity evidenced by the measured strains from Gauges CT7, CT8 and CT10 relative to CT1, CT2 and CT4, respectively.

On the slab bottom surface, Gauges CB1 and CB3 for T1 and Gauges CB5 and CB7 for T2 located near the columns (corresponding to C5 and C2 for T1 and C5 and C8 for T2) were continuously in compression due to slab bending. On the path of C2-C4 in T1 and C6-C8 in T2, Gauges CB2 and CB6 measured in tension. The sharper increase of the strain from Gauge CB6 was possibly attributable to cracks developing across the gauge. The strains of Gauges CB4 and CB8 along the edges of the slab were measured in tension with low magnitudes.

![Graph showing strain variation](image)
Figure 3-18 Concrete strain on top surface of slab: (a) T1; (b) T2
3.4 Yield Line Analysis

The ultimate flexural capacity of the tested specimen is estimated in this section by using the yield line theory (Park and Gamble 2000). The applied loads reached at the FPLs for T1 and T2, prior to which the flexural action was the sole load resisting mechanism, are chosen to make a comparison with the predictions due to the yield line theory. The yield line patterns of T1 and T2 used in the analysis are presented in Figure 3-20, in reference to the observed cracks (Figure 3-8). The ultimate bending moment per unit width $m_u$ resisted by one layer of slab reinforcement is given as (Park and Gamble 2000):

$$m_u = A_s f_y \left( d - 0.59 A_s \frac{f_y}{f_c'} \right)$$  \hspace{1cm} (3.1)

where $A_s$ is the area of tension reinforcement per unit width, $d$ is the slab effective depth, $f_y$ is the yield strength of the slab reinforcement, and $f_c'$ is the concrete compressive strength of the slab.
According to the reinforcement layout shown in Figure 3-3(a) and (c), the positive yield moment per unit width resisted by the orthogonal reinforcement mesh on the slab bottom layer ($m_{upb}$) is calculated as 7.01 kNm/m using Equation 3.1. For the reinforcement in the top layer of the slab, the negative yield moment per unit width resisted by (i) the orthogonal reinforcement mesh in the column strip ($m_{unc}$) is calculated as 13.75 kNm/m, and (ii) the unidirectional reinforcement in the middle strip ($m_{umn}$) is 7.99 kNm/m.

Based on the concept given in Park and Gamble (2000), the internal work done along all the yield lines for T1 ($W_{I,T1}$) can be derived as:

$$W_{I,T1} = \delta m_{upb}(2 \tan \alpha_1 + \beta_1) + 2m_{unc}\alpha_2$$

$$+ 2m_{umn}\left(\frac{\beta_2}{2} + \frac{\sin(2\alpha_2 + 2\beta_2)}{4} - \frac{\sin 2\alpha_2}{4}\right)$$

Equation 3.2

The internal work done along all the yield lines for T2 ($W_{I,T2}$) can be derived is:

$$W_{I,T2} = \delta m_{upb}\left(2 \tan \alpha_1 + \beta_1 + \frac{2L_p}{R}\right) + 2m_{unc}\left(\alpha_2 + \frac{L_c}{R}\right)$$

$$+ 2m_{umn}\left(\frac{\beta_2}{2} + \frac{\sin(2\alpha_2 + 2\beta_2)}{4} - \frac{\sin 2\alpha_2}{4}\right)$$

Equation 3.3

In Equations 3.2-3.3, the virtual displacement at the removed column $\delta$, the length of the positive yield line at the removed column $L_p = 0.5$ m, the length of the negative yield line at the edge columns $L_c = 0.5$ m, the measured radius of the quarter-circular yield line $R = 1.925$ m, and the measured angles $\alpha_1 = 18^\circ$, $\alpha_2 = 15^\circ$, $\beta_1 = 53^\circ$ and $\beta_2 = 24^\circ$ are all marked in Figure 3-20.

The external work ($W_E$) for T1 and T2 done by the applied UDL is:

$$W_E = \delta \frac{\pi R^2 q_u}{12}$$

Equation 3.4

where $q_u$ is the ultimate applied UDL to the slab through the loading tree. The ultimate applied load from the hydraulic jack ($F_u$) is calculated as:
where $L$ is the column centre-to-centre distance.

By rearranging Equations 3.1-3.5, the ultimate applied loads from the hydraulic jack due to flexural failures are predicted as 96.9 kN for T1 and 141.3 kN for T2. Adding the dead weight of the bay having the removed column, i.e., 8.6 kN for T1 and 13.5 kN for T2, to the measured load by the hydraulic jack, the experimental FPL becomes 93.7 kN for T1 and 132.5 kN for T2. The predicted FPL values are therefore 3% and 7% higher than the experimental values for T1 and T2, respectively. The overestimation of the capacity by the yield line theory for T2 is probably because the FPL was initiated by punching shear failure of the edge column C8. Likely due to the flexural action being dominant up to the FPL, it is found that the yield line theory is able to estimate the flexural capacity of the tested specimen in this study. This finding is however different from those obtained by Yi et al. (2014), Qian and Li (2015) and Xue et al. (2018), in which the slab flexural capacities were underestimated by the yield line theory for the edge and interior column removal scenarios, respectively.
3.5 Concluding Remarks

The experimental investigation in this study, through two corner column removal tests, performed on a 1/3 scale, 2×2-bay, RC flat plate specimen draws conclusions as follows:

- The flexural action was evidenced to be the sole load resisting mechanism up to the FPL in both T1 and T2. The load carrying capacity was exhausted afterwards in T1 due to the development of full-depth flexural cracks, while a hybrid load resisting mechanism of flexural, tensile membrane and dowel actions was witnessed in T2 till the PPL.

- The failure mode of T1 was fully dominated by flexural failure. Punching shear failure occurred in T2 at the two edge columns adjacent to the removed corner column. Full-depth flexural cracks were observed in both tests and identically located at the cut-off position of top reinforcing bars of edge columns adjacent to the removed corner column.

- 80% to 110% of the applied UDL was transferred to the two edge columns adjacent to the removed corner column throughout the entire tests. The boundary columns (C3, C6, C7, C8 and C9 for T1, and C2, C3, C4 and C7 for T2) resisted the applied load in tension, with the sum of forces up to 12.4% for T1 and 18.6% for T2 of the total reaction forces.

- Given two different slab corner configurations in T1 and T2, the ultimate load carrying capacity for T1 is 85.1 kN, equivalent to 71.5% of the FPL and 59.4% of the PPL of T2. Additionally, for T2, the load carrying capacity in the post-failure stage (PPL) is 1.2 times higher than that of its flexural stage (FPL).

- The periphery of the slab being strengthened by torsional strips is proven to help prevent the occurrence of punching shear failure at edge columns. However, this does not necessarily enhance the overall load carrying capacity of the slab. With the existence of overhangs surrounding a removed corner column, the contribution of the adjacent bays in resisting the applied load is found to be increased and the progressive collapse risk of flat plates can therefore be reduced.

- Yield line theory was found to be applicable to estimate the slab flexural capacities under both corner column removal scenarios in the present study.
EXPERIMENTAL STUDY ON THE PROGRESSIVE COLLAPSE BEHAVIOUR OF RC FLAT PLATE SUBSTRUCTURES SUBJECTED TO EDGE-COLUMN AND EDGE-INTERIOR-COLUMN REMOVAL SCENARIOS

Statement of contribution to co-authored published paper

This chapter includes a co-authored and peer reviewed paper. The bibliographic details of the co-authored paper, including all authors, are:
Ma, F., Gilbert, B. P., Guan, H., Lu, X., and Li, Y. (2019). "Experimental study on the progressive collapse behaviour of RC flat plate substructures subjected to an edge-column or edge- and interior-column removal scenario." (Accepted on 27/01/2020)

Note that the format of this paper has been changed according to the guidelines of the thesis, and supplementary experimental details which were not included in this paper for publication now have been provided in “APPENDIX A, B and C” for references.

My contribution to this paper involved: literature review, experimental design, experimental setup, experimental data collection, data analyses, discussion of test results, development of analytical study, paper writing and responding to reviewers.

The contribution of this published paper to the advancement of the research data: for the first time, two experimental tests undertaken at large deformations under an edge column removal and an edge-interior-column removal to gain a solid understanding of the progressive collapse behaviour of RC flat plate structures through the analyses of
structural responses, collapse resisting mechanisms, load redistribution characteristics and failure modes.

(Signed) _________________________________ (Date)  24/10/2019
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Co-author: Professor Xinzheng Lu

(Countersigned) ___________________________ (Date)  21/10/2019
Co-author: Professor Yi Li
CHAPTER 4

Experimental study on the progressive collapse behaviour of RC flat plate substructures subjected to edge-column and edge-interior-column removal scenarios

ABSTRACT

Existing studies on progressive collapse of reinforced concrete (RC) flat plate structures have mainly focused on single column loss scenarios. However, accidental events, such as earthquakes, blasts or vehicle collisions, may cause a more severe initial damage beyond the loss of only one column. To fill this gap in knowledge, this study reports two quasi-static large-displacement experimental tests on two nominally identical 1/3-scale, 2×2-bay RC flat plate substructures under an edge-column (S-E) and an edge-interior-column (S-EI) removal scenarios. Two types of uniformly distributed loads (UDLs) were applied to the slab in the two tests: (1) an increased UDL during the tests on the bays adjacent to the removed column(s) and (2) a constant 5 kPa UDL elsewhere to simulate the design live load. In both tests, punching shear failures were observed and led to the first peak loads (FPLs) which were all followed by a sharp load drop. Subsequently, the load increased again to reach the post-failure peak loads (PPLs). The experimental results showed that the PPL was 9.6% higher than the FPL in S-E and 81.9% higher in S-EI. Relative to S-E at the PPL, S-EI presented a more ductile structural behaviour, in which the applied load at the PPL was 26.9% lower, but with a 112 % larger displacement, than S-E. In both tests, tensile membrane actions were observed at large deformations and found to be essential in developing post-failure capacities. Such resisting mechanism cannot be neglected when investigating progressive collapse of RC flat plate structures. The yield line theory was found to overestimate the flexural capacity of the two tests.

Keywords

RC flat plate structures; Progressive collapse; Edge-column removal; Edge-interior-column removal; Punching shear failure.
4.1 Introductory Remarks

Reinforced concrete (RC) flat plate structures are a very common and simple structural system, and nowadays have been prevalently used in Australia and worldwide such as in car park, residential and office building constructions. This structural system has uniformly thick slabs directly supported by columns without beams, drop panels and column capitals, therefore providing an overall structure with an increased number of storeys, architectural convenience and significant savings in construction costs. However, the slab-column connection is prone to punching shear failure (Diao et al. 2019), which may propagate to the surrounding connections, potentially leading to disastrous progressive collapse (Hawkins and Mitchell 1979).

Progressive collapse was initially spotlighted by the partial collapse of the Ronan Point apartment tower in London in 1968. With the increasing concerns about terrorist attacks in recent decades, especially since the collapse of the Murrah Federal Building in Oklahoma City in 1995 and the World Trade Centre towers in New York City in 2001, research on progressive collapse has attracted significant attention in the engineering community. Accordingly, design and strengthening methods, aiming at preventing the occurrence or mitigating the consequences of progressive collapse of building structures, have been proposed in building codes (BS 2006; ASCE 2010) and special guidelines (DoD 2009; GSA 2013). Among these methods, the direct design method using the Alternate Load Path (ALP) approach has been broadly adopted, as it is threat-independent (i.e. regardless of the accidental event), and able to directly evaluate the progressive collapse resistance of building structures. Following the ALP approach, a number of recent studies have focused on the progressive collapse of RC frame structures at various levels, i.e. beam-column sub-assemblages (Qian and Li 2012a; Yu and Tan 2012, 2013a; Lin et al. 2016; Pham and Tan 2017), single-storey substructures (Dat and Tan 2013; Qian and Li 2013c; Dat and Tan 2014; Lu et al. 2017), multi-storey substructures (Yi et al. 2008; Xiao et al. 2015; Qian and Li 2017) and actual structures (Sasani et al. 2007; Sasani and Sagiroglu 2008, 2010; Sasani et al. 2011). However, RC flat plate structures, which may be more vulnerable to progressive collapse owing to the absence of beams, attracted less attention. Given historical progressive collapse events of RC flat plate structures triggered by brittle punching shear failure, such as the 2000 Commonwealth Avenue
condominium collapse in Boston in 1971, the Sampoong department store collapse in Seoul in 1995 and the Pipers Row car park collapse in Wolverhampton in 1997, relevant research activities on RC flat plate structures are highly desired.

Regarding the investigations on the progressive collapse of RC flat plate structures, various column removal scenarios following the ALP approach have been investigated to better understand the collapse resisting mechanisms of this type of system. For an interior column removal scenario, scaled RC flat plate substructures were experimentally tested by Yi et al. (2014), Qian and Li (2015), Peng et al. (2018) and Xue et al. (2018) by applying different loading arrangements including uniformly distributed weight loading, multi-point loading, dynamic loading and concentrated loading. Tests of edge column removal scenarios performed on scaled RC flat plate substructures were also carried out statically (Yi et al. 2014) and dynamically (Peng et al. 2017). In these studies, it was found that punching shear failure occurs initially at one of the columns nearest to the removed one, then potentially propagates to other columns. Under these column removal scenarios, tensile membrane actions were observed to greatly contribute to the post-failure load carrying capacity in resisting collapse. Moreover, due to a laterally unrestrained boundary conditions when losing a corner column, i.e. the slab edges are unrestrained and free to move, the RC flat plate substructure in Yi et al. (2014) was found to be more vulnerable to progressive collapse than when losing an interior or an edge column. In this case, the failure mode was dominated by flexural failure without occurrence of punching shear failure, which was also confirmed by Ma et al. (2019b). Further, Ma et al. (2019b) pointed out that the load carrying capacity of the slab can be enhanced by adding overhangs, i.e. having the slab extended past the exterior columns, therefore reducing the progressive collapse risks of RC flat plate structures. In this configuration, punching shear failure was observed.

The above investigations were conducted on single column removal scenarios at various locations. However, in RC flat plate structures, they are only representative in the case of low intensity of abnormal loadings and relatively large column spacing. Otherwise, building structures may suffer more severe initial damages caused by accidental events such as earthquakes, blasts or terrorist attacks. This can be demonstrated by the collapse of the Murrah Federal Building where the truck loaded with explosives resulted in
concurrent failure of three columns (Osteraas 2006). Apart from the study on the dynamic response of an RC flat plate substructure with drop panels under a two-column removal scenario in Qian et al. (2018), studies on RC flat plate structures without drop panels, which are mostly employed in practice, under a multiple-column removal scenario are nearly non-existent. Therefore, this study reports the experimental work on two 2×2-bay RC flat plate substructures under an edge-column (S-E) and an edge-interior-column (S-EI) removal scenarios, representing different extents of an initial damage. The tested specimens for S-E and S-EI were 1/3 scaled and extracted from the same region of a prototype flat plate structure (Xue et al. 2018; Ma et al. 2019b) and had the same dimensions and reinforcement detailing. Both tests were performed quasi-statically by increasing the Uniformly Distributed Load (UDL) on the bays adjacent to the removed column(s) until large deformations and severe failure of the structure occurred. Firstly, this paper introduces the experimental design and setup. Then the overall structural responses of the two tests are documented and discussed based on the load-displacement relationships, failure modes, crack patterns, load distributions, and displacement and strain results. Finally, the application of the yield line theory to estimate the flexural capacity of the two tested substructure specimens was performed.

4.2 Experimental Program

4.2.1 Specimen Design

The tested specimens were two nominally identical 2×2-bay substructures extracted from the ground floor of a four-storey flat plate car park prototype structure, as illustrated in Figure 4-1(a) and (b). The prototype building was designed in accordance with the Australian Standard on Concrete Structures (AS3600 2009). In the car park prototype, the RC slab has a uniform thickness of 270 mm which creates a dead load of 6.7 kPa. A live load of 5 kPa was considered under medium vehicle traffic conditions. More information on the prototype structure can be found in Ma et al. (2019b). The tested substructures were scaled to 1/3 on account of space restriction in the laboratory. Similar scale factors were successfully adopted in published experimental studies of progressive collapse of RC flat plate structures (Yi et al. 2014; Qian and Li 2015; Peng et al. 2017, 2018). The
tested specimens had overall dimensions of 5,000 mm × 4,575 mm, a 90-mm thick slab and nine columns with cross-sectional dimensions of 150 mm × 150 mm. The centre-to-centre distance between the columns was 2,000 mm in both directions, and the column height was 1,000 mm. As shown in Figure 4-1(b), there were 500 mm overhangs for the slab edges along columns C1-C3, C3-C9 and C7-C9 to simulate the effects of continuity provided by the surrounding slabs of the prototype building. The slab edge along columns C1-C7 was flush with the columns to represent the actual appearance of the prototype building. In this study, both specimens, cast with a nominal concrete compressive strength of 32 MPa and with an aggregate size of 6 mm, were fabricated. It is noted that during the specimen casting, the columns to be removed during the experiments, i.e. column C4 in S-E and columns C4 and C5 in S-EI, were replaced by temporary steel columns of a rectangular hollow section (RHS).

![Figure 4-1 Overview of the tested specimens: (a) substructure extraction from the ground floor of the prototype building; (b) extracted specimen for S-E. (Unit: mm)](image)

(Note: the temporary steel column at C4 was for S-E, while the two temporary steel columns at C4 and C5 were for S-EI.)

The concrete compressive strengths of the slabs at 28 days were measured from 6 concrete cylinder sample tests, following the recommendation in the Australian Standard AS1012 (AS1012.1 2014), and were averagely equal to 34.8 MPa for S-E and 34.1 MPa for S-EI. The reinforcement ratios of the slab were taken the same as those of the prototype building. Considering a nominal clear concrete cover of 10 mm for the top and bottom slab
reinforcements, and the availability of the reinforcing bar size on the market, N8 (8 mm diameter and N ductility class (D500N8)(AS/NZS4671 2001)) bars were then used. The material properties of the slab reinforcement were measured out of three samples, tested according to AS1391 (AS1391 2007), as (i) yield strength of 564 MPa, (ii) 3200 µε and (iii) ultimate strength of 786 MPa. Figure 4-2 and Table 4-1 provide the slab dimensions, the reinforcement arrangement including column strips and middle strips, the reinforcing bar size and the reinforcement ratios. To comply with the requirements of AS3600 (AS3600 2009), torsional strips were designed along the slab edge C1-C7. The spacing, size and configuration of stirrups are given in Figure 4-2(b). Figure 4-2(d) shows an elevation view of a column and its cross-section with reinforcement details. It is noted that the columns were cast before the slab with the reinforcement sticking out, they had a higher concrete strength than the slab (measured as an average compressive strength at 28 days of 50.9 MPa from 6 concrete cylinder test samples) and were overdesigned with a reinforcement ratio of 6.77%. A more specific description of the columns can be found in Ma et al. (2019b). Complements to more detailed experimental design and preparation are given in “APPENDIX A and B”.

Table 4-1 Reinforcement details

<table>
<thead>
<tr>
<th>Strips</th>
<th>Strip numbering</th>
<th>Top reinforcement</th>
<th>Bottom reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column strip</td>
<td>① N8@110</td>
<td>ρ = 0.49%</td>
<td>N8@285 ρ = 0.24%</td>
</tr>
<tr>
<td></td>
<td>② N8@160</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Middle strip</td>
<td>③ N8@250</td>
<td>ρ = 0.28%</td>
<td></td>
</tr>
</tbody>
</table>
Figure 4-2 Reinforcement layout and dimensions of the slab and column: (a) slab top reinforcement; (b) torsional strip detailing; (c) slab bottom reinforcement; (d) column dimensions and reinforcement detailing. (Unit: mm)

4.2.2 Test Setup

4.2.2.1 Loading Scheme

The loading scheme used in this study was in accordance with the design recommendations in the DoD (2009) in which the collapse resistance of structures can be examined by imposing an increased UDL on the bays adjacent to the removed column(s). Such an approach is commonly used in progressive collapse studies (Dat and Tan 2013, 2014; Yi et al. 2014; Qian and Li 2015; Qian et al. 2016; Pham et al. 2017a; Ma et al. 2019b). This approach is supported by the tests performed on existing buildings in Sasani et al. (2007), Sasani and Sagiroglu (2008) and Sasani and Sagiroglu (2010) which tended to show that the axial forces in the columns above the removed one vanished rapidly and that the distributed gravity loads on floors contributed mostly to structural downward movement.

Therefore, in this study, two types of UDLs were applied to the slabs, namely a constant UDL of 5 kPa to simulate the design live load and an increasing UDL till failure. The constant UDL was applied by uniformly laying out weights (i.e. steel blocks and sand bags (see Figure 4-3(a) and (b)) on the bays not adjacent to the removed column(s) and on the overhangs. A multi-point loading method was used to simulate the increasing UDL through a loading tree which was connected to a 250 kN capacity MOOG servo-controlled hydraulic jack. Specifically: (1) for S-E, the increasing UDL was applied, through a six-point loading tree, on the area formed by columns C1, C2, C8 and C7, while the remaining areas were loaded with 5 kPa, and (2) for S-EI, the increasing UDL was applied, through a twelve-point loading tree, on the area formed by columns C1, C3, C9 and C7, while only the overhangs were loaded with 5 kPa. Double loading points in S-EI relative to S-E were because two columns (C4 and C5) were concurrently removed during test and the loading points were designed to equally distribute the applied load on the bays having the
removed column(s). Note that, for S-E only, an extra 10 kPa was applied directly below the loading tree by uniformly hanging steel blocks underneath the slab with the help of threaded rods and oversize washers (see “Steel blocks underneath the slab” in Figure 4-3(a)). This UDL was applied to ensure that the 250 kN capacity hydraulic jack would have enough capacity to reach failure.

An overview of the loading configuration is shown in Figure 4-3(a) and (b) for S-E and S-EI, respectively. The loading trees were designed to transfer the applied load from the hydraulic jack equally to each loading point. A slotted hole was machined at one support of each spreader beam, therefore limited axial load was developed in the beam. It is acknowledged that the sliding offered by the slotted holes at the support of each spreader beam may be limited at large deformations, which therefore may produce some axial forces in the spreader beams and potential differences in the load distribution among the loading points. However, such limited axial forces only developed at large deformations. On the other hand, the loading trees are still one of the most practical solutions to apply an increasing UDL. They were often employed in the literature and when multi-point loading trees were used, slotted holes were rarely present. The present research has further improved the loading tree design by introducing slotted holes. A more detailed description of the type of loading trees employed can be found in Ma et al. (2019b).
A quasi-static loading scheme was adopted for the two tests, which consisted of three distinct loading phases identified in each test as follows:

**Loading Phase 1 (LP1) – Weights loading and loading tree positioning:** At the beginning of LP1, the edge column (C4) to be removed was connected to the hydraulic jack to ensure that the slab remained at its original undeformed position after the temporary steel RHS column was removed. This process allowed recording of the load transferred to this column when loading the constant UDL. For S-EI, the temporary support under column C5 was kept. In the second stage: (1) for S-E, the slab was loaded with the steel blocks and sand bags to reach the 10 kPa on the two bays adjacent to the removed edge column and 5 kPa elsewhere, as introduced before. (2) For S-EI, only the overhangs were loaded with steel blocks to reach the design live load of 5 kPa and no steel blocks were applied below the slab. In the third stage, the loading tree was positioned on the slab.

**Loading Phase 2 (LP2) – Release of temporary support(s):** In the second loading phase, the edge column (C4) reaction force transferred to the hydraulic jack during LP1 was quasi-statically released to 0 kN, in displacement control at a stroke rate of 0.5 mm/min, i.e. representing a complete loss of the edge column. For S-EI only, the portal frame and hydraulic jack were re-positioned and connected to the interior column stub (C5), the load
carried by this column was transferred to the hydraulic jack and the underneath temporary steel RHS column was removed. Finally, the interior column reaction force transferred to the hydraulic jack was released following the same procedure mentioned above for the edge column.

*Loading Phase 3 (LP3) – Increase UDL to failure:* In the third loading phase, the portal frame and hydraulic jack were re-positioned and connected to the loading tree. The hydraulic jack was then quasi-statically driven in displacement control, evenly spreading the applied load to each loading point on the slab. To achieve a time-efficient loading process, different displacement loading rates were implemented at key stages. The loading rates were:

- 1 mm/min up to a column C4 displacement of 74 mm and 157 mm for S-E and S-EI, respectively, corresponding to the first peak load (FPL) and well-developed cracks.
- 3 mm/min up to a column C4 displacement of 339 mm and 719 mm for S-E and S-EI, respectively, corresponding to the post-failure peak load (PPL) and well-observed failures.
- 5 mm/min till the end of the test, i.e. at a column C4 displacement of 463 mm and 1021 mm for S-E and S-EI, respectively.

### 4.2.2.2 Instrumentation

In S-E and S-EI, the slab behaviour during loading was monitored by the following apparatuses mounted at key locations:

- The applied load was measured by the built-in load cell of the hydraulic jack and the axial forces and moment reactions transferred to the columns were measured by custom-made load cells bolted underneath each column (see “Load cell” in Figure 4-3(a) and Xue et al. (2018) for more details).
- The slab overall displacement was monitored by (1) 12 linear variable displacement transducers (LVDTs) mounted below the slab measuring its
vertical displacements, and (2) 4 laser displacement transducers mounted on the
sides of the slab measuring its horizontal displacements.

- The strains developed in the reinforcement bars were measured by 45 3-mm
  strain gauges glued to both the top and bottom reinforcements at critical locations.
  The concrete strains were measured by 8 90-mm strain gauges glued at key
  positions on the slab top surfaces.

Only selected readings are presented and discussed in this paper. Locations of discussed
displacements and strains are given where applicable in Sections 4.3.3 and 4.3.4 “Vertical
and Horizontal Displacements” and “Strain Results”. A more detailed introduction of
measurements of strains, forces, displacements and cracks is provided in “APPENDIX
C”.

4.3 Test Results and Discussion

4.3.1 Overall Behaviours

4.3.1.1 Substructure S-E

LP1 and LP2: At the end of LP1, a reaction force of 22.6 kN (see Table 4-2) was
transferred to the hydraulic jack replacing the column C4 from loading the 10 kPa steel
blocks underneath the slab and 5 kPa elsewhere (see Section 4.2.2.1 “Loading Scheme”).
During LP1, no cracks were observed indicating that the specimen was able to sustain
twice its design live load (i.e. 10 kPa) under normal service conditions. Before starting
LP2, all load cell readings were reset to zero and Table 4-3 gives the redistribution of the
jack reaction force to the remaining eight columns at the end of LP2. The difference of
0.5 kN between the jack reaction force of 22.6 kN in Table 4-2 and the sum of the load
cell readings of 23.1 kN in Table 4-3 is within the 1-3% accuracy of the custom-built
column load cells (Xue et al. 2018). The jack reaction force was mainly transferred to the
three columns (C1, C5 and C7), being the closest to the removed edge column C4. Tensile
reaction forces were measured in the remaining five columns, totalling 9.5 kN (see Table
4-3). The percentage of the individual reaction forces with respect to the total
redistributed force is also presented in Table 4-2 and Table 4-3. After the jack reaction force was completely released, a displacement of 13.6 mm of the removed edge column C4 was measured, along with initial minor cracks (IC) on the top surface of the slab originating from the slab-column connections at columns C1, C2, C5, C7 and C8, and propagating parallel to the slab edges.

Table 4-2 Column reaction forces at the end of LP1

<table>
<thead>
<tr>
<th></th>
<th>C1</th>
<th>C2</th>
<th>C3</th>
<th>C4 (Jack)</th>
<th>C5</th>
<th>C6</th>
<th>C7</th>
<th>C8</th>
<th>C9</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Force</td>
<td>13.6</td>
<td>21.5</td>
<td>8.5</td>
<td>22.6</td>
<td>42.3</td>
<td>13.1</td>
<td>11.8</td>
<td>22.1</td>
<td>7.9</td>
<td>163.3</td>
</tr>
<tr>
<td>Percentage</td>
<td>8.3</td>
<td>13.2</td>
<td>5.2</td>
<td>13.8</td>
<td>25.9</td>
<td>8.0</td>
<td>7.2</td>
<td>13.6</td>
<td>4.8</td>
<td>100.0</td>
</tr>
</tbody>
</table>

(Note: negative column reaction forces denote columns in tension.)

Table 4-3 Column reaction forces at the end of LP2

<table>
<thead>
<tr>
<th></th>
<th>C1</th>
<th>C2</th>
<th>C3</th>
<th>C4 (Jack)</th>
<th>C5</th>
<th>C6</th>
<th>C7</th>
<th>C8</th>
<th>C9</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Force</td>
<td>11.7</td>
<td>-3.4</td>
<td>-1.3</td>
<td>0.0</td>
<td>12.0</td>
<td>-1.9</td>
<td>8.9</td>
<td>-2.0</td>
<td>-1.0</td>
<td>23.1</td>
</tr>
<tr>
<td>Percentage</td>
<td>50.9</td>
<td>-14.8</td>
<td>-5.4</td>
<td>0.0</td>
<td>51.9</td>
<td>-8.4</td>
<td>38.5</td>
<td>-8.5</td>
<td>-4.2</td>
<td>100.0</td>
</tr>
</tbody>
</table>

LP3: Figure 4-4(a) plots the applied UDL (in kPa over an area of 8 m²) versus the edge column C4 displacement for S-E. Note that the self-weight of the loading tree (equivalent to 1.2 kPa) is not considered in Figure 4-4(a). The dotted line represents the extrapolated relationship between the applied 10 kPa loading from the steel blocks and the C4 displacement measured at the end of LP2 (Point IC). From 10 kPa to 11.4 kPa, corresponding to the displacement of C4 increasing to 14.3 mm, an almost linear initial stiffness of 0.8 kPa/mm was observed. Then the applied load continued increasing also almost linearly with the stiffness dropping to about 0.14 kPa/mm till a C4 displacement of 74 mm and the FPL of 19.8 kPa. The FPL was triggered by punching shear failure occurring at the interior column C5. As a result, the load suddenly dropped to 14.3 kPa, yet the C4 displacement only increased by 1.3 mm. Up to the FPL, flexural cracks radiating from the column peripheries further developed and connected with each other along the lines C1-C2, C2-C5-C8 and C7-C8, as well as formed half-ring cracks passing by columns C1, C5 and C7. At the slab soffit, positive cracks ran diagonally, parallel to lines C2-C4 and C4-C8 (see Figure 4-5(b)). Prior to the FPL, the load was resisted by not
only the flexural action but also the compressive membrane action (as described later in Section 4.3.3 “Vertical and Horizontal Displacements”).

Upon reaching the FPL, severe concrete cracking and detaching were observed at column C5 on the side of the bays loaded by the loading tree, while no reinforcing bars were fractured. Despite of the top reinforcing bars being gradually bent at column C5, dowel action allowed the column to continue resisting the applied load. Therefore, the applied load increased again up to 21 kPa at a C4 displacement of 193.5 mm where punching-like cracks occurred at columns C2 and C8, resulting in a slight load drop to 20.2 kPa. Subsequently, the applied load further increased until the PPL was reached at 21.7 kPa, corresponding to a C4 displacement of 339.5 mm. Then the applied load could not increase further beyond the PPL, mainly due to additional punching-like cracks developing at columns C2 and C8 and the complete formation of the yield lines diagonal to columns C1 and C7 (see Figure 4-6). The test was ultimately ended at a C4 displacement of 463.4 mm when the applied load significantly dropped to 17 kPa without a reascending trend. Throughout the post-failure stage, the load was 9.6% higher at the PPL relative to the FPL, attributable partly to the flexural and dowel actions, and most importantly to the tensile membrane action. The recorded crack patterns at the end of the test on both the slab top and bottom surfaces are presented in Figure 4-5. The pattern of plastic hinges is also shown in Figure 4-5 with the yield lines drawn in bold. Significant concrete spalling and crushing were observed along these lines.
Figure 4-4 Load-displacement curves of: (a) S-E; (b) S-EI
4.3.1.2 Substructure S-EI

LP1 and LP2: After loading the 5 kPa steel blocks on the surrounding overhangs and positioning the loading tree in LP1, the recorded vertical displacements at removed
columns C4 and C5 were less than 3 mm at the end of LP2. Throughout LP1 and LP2, no visible cracks were observed.

LP3: Figure 4-4(b) plots the applied UDL (in kPa over an area of 16 m\(^2\)) versus the edge column C4 displacement for S-EI. Note that the edge column C4 displacement is used to allow comparisons to S-E. Similarly, the self-weight of the loading tree (equivalent to 1.3 kPa) is not considered in Figure 4-4(b). Due to the double-span effect in S-EI, initial flexural cracks (IC) on the top surface of the slab were first observed at 2.5 kPa. The initial stiffness of the load-displacement curve prior to the IC was coincidently the same as that of S-E (i.e. 0.8 kPa/mm). Then the structural behaviour entered the plastic stage and the stiffness significantly dropped to 0.08 kPa/mm up to an applied load of 8.5 kPa and a C4 displacement of 74.6 mm. With further loading, flexural cracks on the top surface of the slab between columns connected with each other, typically along the path of C1-C2-C6-C8-C7, while punching-like cracks densely developed around columns C2, C6 and C8 together with popping and cracking sounds. At a C4 displacement of 157.4 mm, the FPL was reached due to punching shear failure at the edge column C8 resulting in the applied load dropping from 9.4 kPa to 8.6 kPa. It is noted that the C4 displacement at the FPL in S-EI was about twice that of S-E, however, the corresponding applied loads in S-E (19.8 kPa) and S-EI (9.4 kPa) follow an inverse comparison to the displacements. Prior to the FPL, the same load resisting mechanisms (i.e. flexural and compressive membrane actions) as those in S-E were witnessed.

From the FPL onwards, the applied load was mainly resisted by the tensile membrane and flexural actions, with the aid of the dowel action from the reinforcing bars locally bent at column C8 which continued carrying the load after punching shear failure. Such an alternate load path enabled the applied load to increase almost linearly at approximately 0.015 kPa/mm till the PPL was reached at 17.1 kPa. The load dropped by 2.2 kPa at the PPL due to secondary punching shear failure at the edge column C8. Relative to S-E at the PPL, the structural behaviour of S-EI showed a more ductile characteristic as the C4 displacement at the PPL in S-EI was 112 % higher than the one in S-E. After the load drop at PPL, no reinforcing bars fractured at column C8, and the applied load reascended again but was eventually exhausted at a C4 displacement of 943 mm. The recorded crack patterns at the end of the test on both the slab top and bottom surfaces are presented in
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Figure 4-7 where the pattern of plastic hinges is also shown with the yield lines drawn in bold. Figure 4-8 reveals the final failure modes of S-EI.

![Figure 4-7 Crack patterns at the top surface (a) and bottom surface (b) of the slab for S-EI](image)

![Figure 4-8 Failure modes of S-EI](image)
4.3.2 Load Transfer

Throughout the two tests, the custom-made load cells underneath each column captured the load distributions and redistributions. For simplicity and due to symmetry, the columns in S-E are grouped as interior column (C5), corner columns (C1 and C7), edge columns (C2 and C8) and boundary columns (C3, C6 and C9). The columns in S-EI are grouped as corner columns (C3 and C9), edge column (C6) and other columns (C1, C2, C7 and C8). Note that due to the large bending moments caused by the double-span effect, the load cells under columns C1, C2, C7 and C8 stopped measuring the loads after the FPL in S-EI. Figure 4-9(a) and (b) plot the distributions of the increasing UDL during LP3 to different column groups in percentage for S-E and S-EI, respectively. In addition, curves in Figure 4-9(a) are plotted from a C4 displacement of 13.6 mm due to LP1 and LP2 in S-E.

For S-E in Figure 4-9(a), the interior column C5 was found to carry the highest proportion of the applied load for both FPL (34.9%) and PPL (35%), the latter despite C5 having punched through at the FPL. The force transferred to each corner column (C1 and C7) adjacent to the removed column was 16.2% and 14.7% of the applied load for FPL and PPL, respectively. Such reactions at each corner column were 220% and 240% lower than the force resisted by C5 for FPL and PPL, respectively. The ratio of the force transferred to the interior column C5 to the one transferred to each edge column (C2 and C8) was found to be 2.8 at FPL and 2.0 at PPL. At the FPL, the reaction at column C5 significantly dropped at punching to be mainly redistributed to columns C2 and C8. The sum of boundary columns (C3, C6 and C9) carried an initial 15.4% of the applied load contributed by the 5 kPa and 10 kPa weights loading in LP1 and LP2, respectively. Subsequently, the sum of the support forces at boundary columns gradually decreased, regardless of minor load redistribution at the FPL. This can be explained by the overturning moment created by the increasing UDL in LP3.

For S-EI in Figure 4-9(b), the edge column C6 constantly carried about 20% of the applied load throughout the whole test. The sum of the applied load transferred to columns C1, C2, C7 and C8 varied from 65% to 80% which was about four times the force carried by the edge column C6. The load carried by the corner columns C3 and C9 increased first to
about 10% of the total applied load then decreased thereafter to less than 5% at the end of the test.

Figure 4-9 Load distribution to column groups for: (a) S-E; (b) S-EI
4.3.3 Vertical and Horizontal Displacements

The deflection profiles of the double-span loaded bays at the critical stages (IC, FPL, PPL and the end of the test), along C1-C4 based on four equally spaced LVDTs for S-E, and along C1-C4 and C2-C5 based on three equally spaced LVDTs for S-EI, are depicted in Figure 4-10. With respect to S-E, the curves at the IC and FPL in Figure 4-10(a) indicate a bending deformation and reflect that the failure was only limited to the slab-column regions without much concrete cracking in the main loaded bays. At the PPL and at the end of the test, the significant slope changes 1,000 mm away from column C1 were due to the yield lines (see Figure 4-6) at which the slab, to some extent, rotated. In Figure 4-10(b) for S-EI, it is illustrated that the slab at regions between C2 and C5 almost deformed as a rigid body from the beginning till the end of the test. However, along the slab edge C1-C4, positive and negative rotations were observed at 667 mm and 1,333 mm, respectively, from column C1 at both the PPL and the end of the test. This agrees well with the experimental observations showing two prominent yield lines passed across the slab edge C1-C4 (see Figure 4-8). It is noted that Figure 4-10 demonstrates the slab vertical displacements measured at key locations, which are connected by straight lines, for different loading stages.

(a)
The horizontal movement of the slab for S-E and S-EI is plotted against the vertical displacement of the edge column C4 in Figure 4-11(a) and (b), respectively. The measuring locations are also given in the figures. Due to concrete cracking on the side surface of the slab occurring at large deformations and affecting the readings of the horizontal laser displacement transducers, the values are disregarded after a vertical displacement of 180 mm at column C4 for S-E and 400 mm for S-EI. The horizontal movement in the two tests initially expanded up to a maximum of 1.2 mm for S-E and 1.7 mm for S-EI, reflecting the existence of compressive membrane actions. The transition from the compressive membrane action to the tensile membrane action in S-E was found at a C4 displacement of about 74 mm, corresponding to the FPL. The same transition point in S-EI was found at a C4 displacement of 96 mm which was however 61 mm prior to the FPL. This phenomenon, that was also found in Yi et al. (2014) where the transition point recorded at columns occurred ahead of that at the middle span of centre-to-centre columns, could possibly account for the location of horizontal displacement.
transducers. The much larger inward horizontal movement in S-EI relative to S-E can be explained by the far more extensive development of the tensile membrane action in S-EI.

Figure 4-11 Horizontal displacements of: (a) S-E; (b) S-EI
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4.3.4 Strain Results

This section presents and analyses the strain readings from the gauges mounted on the slab reinforcing bars and the slab top surfaces for S-E and S-EI. Note that, due to the sensitivity and brittleness of strain gauges, especially during concrete cracking, only valid strain gauge readings are presented herein and outliers were disregarded for clarity.

4.3.4.1 Reinforcing Bars

Figure 4-12 plots the strain gauge readings in the top reinforcing bars positioned in the loaded bays for S-E. Strain gauge numbering is also given in Figure 4-12. Except for gauges ST5 and ST6 next to the removed column C4, the remaining gauge readings reveal that the top reinforcing bars at columns C1, C2 and C5 all yielded. It should be noted that a maximum compressive strain of 200 $\mu$ε in gauge ST6 was captured at a C4 displacement of 17 mm, indicating the involvement of the compressive membrane action, but limited to a small extent. On the other hand, no compressive strains were recorded in gauge ST5, implying that the compressive membrane action was likely mobilised in the C1-C7 direction and less likely in the C4-C5 direction due to the laterally unrestrained edge at removed column C4. Relative to strain readings from gauges ST2 and ST7 where the strains dramatically increased and reached the yield strength $\varepsilon_y$ at an early stage (at about a C4 displacement of 36 mm), the strain in gauge ST4 increased rapidly after the FPL where load redistributions were initiated owing to punching shear failure at column C5. This agrees well with a higher portion of the applied load being taken up by columns C2 and C8 after the FPL (see Figure 4-9(a)). Moreover, the measured reinforcement yielding in gauges ST1 and ST3 located at the interior boundary of the loaded bays was caused by punching shear failure at column C5 and punching-like cracks at column C2, respectively.

Figure 4-13 plots the strain gauge readings in the top reinforcing bars positioned in the loaded bays for S-EI. Strain gauge numbering is also given in Figure 4-13. A similar pattern of strain development in the slab top reinforcement bars to that in S-E can be observed in Figure 4-13. Gauges ST2, ST7 and ST9 bonded to the bars perpendicular to the removed columns C4 and C5 were significantly elongated, while gauge ST3 located
near column C3 and diagonally adjacent to the removed column C5 also saw a large strain value, beyond the yield strain $\varepsilon_y$. For gauges (ST1, ST4, ST5, ST6, ST8 and ST10) bonded to the reinforcing bars at or parallel to the removed columns, relatively small strain values were measured. Unlike S-E, no compressive strains were found in S-EI in gauges ST5 and ST6, i.e. near the removed column C5.

Figure 4-12 Top reinforcement strains in S-E

Figure 4-13 Top reinforcement strains in S-EI
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Figure 4-14 illustrates the strain distribution along the bottom reinforcing bars passing through the removed column C4 in S-E. Strain gauge locations are given in the figure. Strain readings within the main loaded bays (SB1, SB2, SB4, SB8, SB9 and SB10) showed strain values below or slightly exceeding the yield strain $\varepsilon_y$, except at gauges SB3 and SB7. The latter values being two to three times higher than $\varepsilon_y$ were attributed to SB3 located on a yield line (see Figure 4-6) and SB7 located within the punching shear failure zone of column C5. At corner column C1, comparing to the considerably deformed top reinforcing bar evidenced by ST7 in Figure 4-12, gauge SB5 indicated that the bottom reinforcing bar passing through the column did not undergo very large deformations. Additionally, a maximum strain of 3800 $\mu\varepsilon$ was found in gauge SB6 in the constant 5 kPa region, suggesting that the bays adjacent to the loaded ones were well involved in resisting the applied load.

Figure 4-15 illustrates the strain distribution along the bottom reinforcing bars passing through the removed columns C4 and C5 in S-EI, with the gauge numbering also given in the figure. Besides gauges SB8 and SB13 at the two ends of the bottom reinforcing bar passing through columns C4-C6, the remaining gauges (SB9-SB12) along this reinforcing bar show an approximately even strain, just above the yield strain $\varepsilon_y$, demonstrated by interweaving strain curves in Figure 4-15. Along the bottom reinforcing bar passing through columns C2 and C5 (SB1, SB2 and SB3), larger strain values, greater than five times $\varepsilon_y$, were recorded due to the tensile membrane action. Strain values greater than seven times $\varepsilon_y$, with abrupt strain increase after a C4 displacement of 200-500 mm, were recorded for the bar passing through columns C1 and C4 (gauges SB5, SB6 and SB7) resulting from the yield lines, punching-like cracks and tensile membrane actions, which can be referenced to Figure 4-8.
Additionally, the strain results recorded by the gauges glued to the remaining bottom reinforcement bars are shown in Figure 4-16 for S-E and in Figure 4-17 for S-EI. Gauge positions and numbering are shown in the figures. The strain readings for both tests indicated that the bottom reinforcement resisted the applied load as a tensile reinforcement net in two directions, mainly resulting from the tensile membrane action. Moreover, the strain magnitude was found to be in general proportional to the slab
deformation, namely, the strain readings were larger when being close to the removed edge columns (i.e. SB16, SB17 and SB18 in S-E and SB19, SB20 and SB21 in S-EI).

Figure 4-16 Bottom reinforcement strains perpendicular to the reinforcing bars passing through the removed column in S-E

Figure 4-17 Bottom reinforcement strains perpendicular to the reinforcing bars passing through the removed columns in S-EI
4.3.4.2 Slab Concrete

Figure 4-18 and Figure 4-19 display the concrete strain variations on the slab top surface at key locations for S-E and S-EI, respectively. Strain gauge positions and numbering are given in the figures. To ensure the validity of the measured strain for concrete, the gauge readings are only plotted up to a C4 displacement of 180 mm, according to the observations that there were gauges located on cracks or isolated by cracks. The stiff increases of strains in gauges CT1, CT2, and CT5 in S-E, and CT1 and CT5 in S-EI match well with the location of the ICs on the slab top surface, which originated from the slab-column connections next to these gauges (as mentioned in Section 4.3.1 “Overall Behaviours”). It is noteworthy that strains measured by gauges CT4 in S-E, and CT4 and CT6 in S-EI showed increasing compression values before a C4 displacement of 80 mm, then decreased. This phenomenon again proves the existence of the compressive membrane action in S-E and S-EI but limited to a small extent within a displacement of one slab depth. Furthermore, the strain value in CT3 in S-E remained almost 0 \( \mu \varepsilon \), further suggesting that the compressive membrane action was not developed along the C4-C5 direction due to the laterally unrestrained edge at the removed column C4 (the same finding in reference to ST5 in Figure 4-12).
4.4 Yield Line Predictions

Yield line theory is a well-founded method used to predict the flexural capacity of RC flat plate structures and has been successfully employed in Yi et al. (2014), Qian and Li (2015), Xue et al. (2018) and Ma et al. (2019b) for instance. While the yield line theory does not account for membrane actions, prior to the FPL in S-E and S-EI, the flexural action was dominant with very limited compressive membrane action being involved (see Section 4.3.4 “Strain Results”). The theory can then apply and the yield line configurations of the tested specimens for subsequent calculations are based on the observed crack patterns at the FPL, as shown in Figure 4-20. Using the yield line predictions, the reserved flexural capacity of the slab can be obtained when a higher punching shear strength is designed for the columns (i.e. FPL is initiated by flexural failure). According to Park and Gamble (2000), the ultimate bending moment per unit width of the slab reinforcement $m_u$ is expressed as:
where $A_s$ is the area of tension reinforcement per unit width, $d$ is the slab effective depth, $f_y$ is the yield strength of the slab reinforcement, and $f'_c$ is the concrete compressive strength of the slab.

Given the reinforcement ratios and arrangement detailed in Table 4-2 and Figure 4-2, together with the slab effective depth of 72 mm, the measured yield strength of the slab reinforcement of 564 MPa and the measured concrete compressive strength of the slab of 34.8 MPa for S-E and 34.1 MPa for S-EI, the ultimate bending moment of the slab reinforcement can be determined. For the slab top reinforcement, the negative bending moment resistance per unit width in the column strip ($m_{unc}$) is calculated as 13.62 kN-m/m (S-E) and 13.61 kN-m/m (S-EI), and the negative bending moment resistance per unit width in the middle strip ($m_{unm}$) is calculated as 7.95 kN-m/m (S-E) and 7.94 kN-m/m (S-EI). For the slab bottom reinforcement, the positive bending moment resistance per unit width ($m_{upb}$) is calculated as 6.78 kN-m/m for both S-E and S-EI.

For S-E, the internal work ($W_{I,S-E}$) done by the positive (solid lines in Figure 4-20) and negative (dash lines in Figure 4-20) yield lines in the slab is:

$$W_{I,S-E} = (4m_{upb}L + 2m_{unc}L + 2m_{unm}L)\delta/L$$

where $\delta$ is the virtual displacement at the removed edge column, and $L$ is the column centre-to-centre distance. For S-EI, the internal work ($W_{I,S-EI}$) done by the positive and negative yield lines in the slab is:

$$W_{I,S-EI} = (4m_{upb}L + 2m_{unc}L + 2m_{unm}L)\delta/0.7L$$

$$\quad + (2m_{upb}L + m_{unc}L + m_{unm}L)\delta/L$$

The external work ($W_{E,S-E}$) for S-E done by the applied UDL is:
CHAPTER 4

\[ W_{E,S-E} = \delta \frac{2L^2 q_u}{3} \] 4.4

The external work \((W_{E,S-EI})\) for S-EI done by the applied UDL is:

\[ W_{E,S-EI} = \delta \frac{31L^2 q_u}{15} \] 4.5

where \(q_u\) is the applied UDL.
By equating Equations (2) and (4), and Equations (3) and (5), the flexural capacities of the slab based on the yield line theory are calculated to be 26.3 kPa for S-E and 16.4 kPa for S-EI. Considering the self-weights of the slab and the loading tree, the experimental FPLs become 23.3 kPa for S-E and 12.9 kPa for S-EI. The predicted flexural capacities of the slab overestimate the experimental FPLs by 13% and 27% for S-E and S-EI, respectively. This result is different from similar studies conducted by Yi et al. (2014) and Qian and Li (2015) in which the predictions, also using the yield line theory, underestimated the ultimate load carrying capacity. The overestimation in this study is likely attributable to the punching shear failure initiating the FPLs in S-E and S-EI due to the relative small reinforcement ratio of 0.49% in the column strip (see Table 4-1), which prevented the slab flexural capacity to be fully developed. Note that using similar experimental configurations and reinforcement ratios in Ma et al. (2019b), the yield line calculation also overestimated the capacity by up to 9%. The 23% overestimation of the FPL by the yield line theory in S-EI, relative to the 13% overestimation of the FPL in S-E may be because of the double-span effect which reduced the punching strength at the slab-column connections (Muttoni 2008).
4.5 Concluding Remarks

This paper reports two quasi-static large-displacement experimental tests performed on two nominally identical 1/3-scale, 2×2-bay RC flat plate substructures under an edge-column (S-E) and an edge-interior-column (S-EI) removal scenarios. The obtained experimental results have been presented and discussed in detail. Main highlights of this study are summarised as follows:

- Prior to the first peak load (FPL) in S-E and S-EI, the applied loads were resisted by a dominant flexural and a limited compressive membrane actions. Thereafter, the resisting mechanism of the two tests was a combination of flexural, tensile membrane and dowel actions until the post-failure peak load (PPL) was reached. The causes of the FPL and the PPL in both tests were punching shear failure and the complete formation of the yield lines, respectively.

- For S-E, the interior column C5 carried 34.9% at the FPL and 35% at the PPL of the applied load, which are more than twice the load carried by each edge column or corner column. For S-EI, the majority of the applied load was distributed to the columns (C1, C2, C6, C7 and C8) adjacent to the removed columns C4 and C5 with only 10% at the FPL and almost none of the load at the PPL carried by corner columns C3 and C9.

- After the FPLs initiated by punching shear failures in both tests, the post-failure stages enabled the slab to resist collapse and the PPL at large deformations was found to be 9.6% higher than the FPL in S-E and 81.9% higher in S-EI. These indicate that at large deformations, the residual load carrying capacity of a flat plate structural system can be generated through alternate load paths and must not be ignored when investigating the resistance of progressive collapse.

- Comparing to the edge-column removal scenario, concurrently removing the edge and interior columns can reduce the ultimate load carrying capacity by 26.9%, which would therefore increase the progressive collapse risk of RC flat plate structures. However, at the PPL, the deformation of S-EI was found to be 112% larger than that of S-E.

- Strain results showed that the top reinforcing bars at large deformations (after the FPL) resisted the applied load to a greater and about the same extent than the bottom reinforcing bars in S-E and S-EI, respectively.
According to the yield line predictions, it is extrapolated that enhancing the punching shear strength of slab-column connections for the tested specimens, which were designed in compliance with the Australian Standards, could increase the FPLs of S-E and S-EI by 13% and 23%, respectively.
CHAPTER 5

NUMERICAL MODELLING OF RC FLAT PLATE SUBSTRUCTURES UNDER MULTIPLE COLUMN REMOVAL SCENARIOS

5.1 Introductory Remarks

Following the experimental and analytical studies of progressive collapse behaviour of RC flat plate structures in the last two chapters, this chapter is devoted to numerically replicate the experimental process and obtain more in-depth analyses and understanding of the failure mechanisms of the conducted experimental tests. To this end, the finite element method (FEM), which provides a powerful technique for solving complex problems in structural analysis, was used in this study. Among numerous FEM programs, the commercial explicit software LS-DYNA was selected to carry out the numerical analyses due to its reliable material models for high nonlinearities and the nature of avoiding convergence issues.

A 3D nonlinear finite element modelling approach, using LS-DYNA software, was established to investigate the progressive collapse behaviour of RC flat plate substructures. The numerical models were established based on the four quasi-static large-displacement tests performed on 1/3-scale, 2×2-bay, RC flat plate substructures ("CHAPTER 3 and CHAPTER 4") and one similar test from the literature. In the experimental tests, a specially designed multi-point loading system was used to simulate an increasing UDL imposed on the removed-column bay(s) till failure. In the numerical model, this loading system was explicitly simulated in order to replicate the true test conditions. The validation of the proposed numerical models was completed through comparisons of the load-displacement responses, failure modes, crack development and displacement and strain results against the experimental ones.
Section 5.2 describes some general considerations when creating numerical models, including the selection of element types, the definition of the bond-slip relationship between concrete and reinforcement, the application of UDLs and boundary conditions, and the simulation of the loading tree. Section 5.3 introduces the material models used for concrete and reinforcement and the failure criteria for simulating concrete damages by element deletion. Sections 5.4-5.8 cover the validation of numerical models under two corner column removal scenarios (T1 and T2), one edge column removal scenario (S-E), one edge-interior-column removal scenario (S-EI) and one interior column removal scenario (ND).

5.2 General Considerations of Numerical Models

In addition to define and calibrate appropriate material models and failure criteria when simulating the highly nonlinear flat slab behaviour under column removal scenarios, selecting reasonable keywords in LS-DYNA to reflect load application, boundary conditions, contact between reinforcing bars at large deformations, etc. are also critically important. This section presents some general considerations of applying these keywords to the numerical models.

5.2.1 Element Types

In the numerical model, three types of elements were used, which were solid elements for concrete and loading pads, beam elements for reinforcing bars and loading beams and shell elements for triangular loading plates. Depending on the functions of these elements, different sections and material properties were given accordingly. Note that for all elements, the value of damping ratio was taken as zero in the numerical models.

The default eight-node 3D constant stress solid elements were used for concrete. This type of solid elements is only integrated by one point at the element centre (reduced integration), thereby providing computational efficiency yet having a potential of
unrealistic element distortion under large stresses. In the proposed numerical model, this issue was addressed by utilising a relative fine mesh, and introducing *CONTROL_HOURGLASS keyword. The parameters to inhibit the hourglass modes in the keyword were set with standard type 1 viscosity and a default coefficient of 0.1.

The default two-node Hughes-Liu beam elements with $2 \times 2$ Gauss quadrature integration at cross sections were used for reinforcing bars. The reinforcing bars were explicitly modelled with a circular cross section, at which the diameters were specified identically to the actual sizes of the slab and column reinforcements. The length of the reinforcement beam elements was dimensioned similarly to the characteristic length of the concrete solid elements.

The default 2D Belytschko-Tsay shell elements with two integration points through the shell thickness were used for triangular steel plates for loading purpose. The main function of the triangular steel plates was to transfer loads to each loading point, so that an increased UDL can be applied to the slab. Therefore, the steel plates were only meshed by four elements, but defining a high stiffness to avoid undesired deformations (see the later Section 5.2.4 “Modelling of Loading Tree”).

5.2.2 Bond-slip Relationship

It was observed in “CHAPTER 3 and CHAPTER 4” that after punching shear failure or concrete crushing and spalling, reinforcing bars were pulled out. A simple assumption of perfect bond relationship between concrete and reinforcement by using shared nodes may lead to an overestimation of the slab load carrying capacity and the premature fracture of reinforcing bars (Bao et al. 2012).

In an attempt to model the bond-slip behaviour in this numerical analysis, *CONSTRAINED_BEAM_IN_SOLID keyword was employed via defining the concrete solid elements as the master component and the reinforcement beam elements as the slave component. By doing so, both the acceleration and velocity of the reinforcement beam elements were constrained to move with the concrete solid elements. Such constraints can
be selected along either all directions or only excluding the beam axial direction. The constraint along the beam axial direction represents the bond-slip relationship between concrete and reinforcing bars, and was user-defined. The bond-slip model proposed by Taerwe and Matthys (2013) (fib Model Code for Concrete Structures 2010), which has been successfully applied to similar numerical analyses in Pham et al. (2017b) and Yu et al. (2018), was adopted. The bond stresses $\tau_b$ between concrete and reinforcement along the beam axial direction can be calculated as (Taerwe and Matthys 2013):

$$\tau_b = \tau_{bmax} \left( \frac{s}{s_1} \right)^a \quad \text{for } 0 \leq s \leq s_1$$

$$\tau_b = \tau_{bmax} \quad \text{for } s_1 \leq s \leq s_2$$

$$\tau_b = \tau_{bmax} - \left( \tau_{bmax} - \tau_{bf} \right) \frac{(s - s_2)(s_3 - s_2)}{(s_3 - s_2)} \quad \text{for } s_2 \leq s \leq s_3$$

$$\tau_b = \tau_{bf} \quad \text{for } s_3 \leq s$$

where $\tau_b$ is a function of the relative displacement $s$ of reinforcing bars to concrete, as shown in Figure 5-1, determination of the parameters in the above equations is based on Table 5-1. $\tau_{bmax}$ and $\tau_{bf}$ are the maximum bond stress and remaining bond stress at a large relative displacement (s), respectively, between concrete and reinforcement along the beam axial direction; $s_1$, $s_2$, and $s_3$ are the relative displacements of reinforcing bars to concrete at three bond stress conditions.

It can be seen that the parameters $\tau_{bmax}$, $\tau_{bf}$, $s_1$, $s_2$, and $s_3$ are calculated based on the concrete compressive strength and the bond conditions. The concrete strength of the tested slabs can be found in Sections 3.2.1 and 4.2.1, and “Splitting failure” and “All other bond cond.” as shown in Figure 5-1 and Table 5-1 were chosen for the bond conditions in this study. Note that the derived user-defined functions according to Equations 5.1-5.4 and Figure 5-1 were input into the keyword *DEFINE_FUNCTION to regulate the bond-slip behaviour of reinforcing bars in concrete. Relevant codes for T1, T2, S-E and S-EI models are attached in “APPENDIX D”.

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Figure 5-1 Bond-slip relationship between concrete and reinforcement (Taerwe and Matthys 2013) (Note: $\tau_{bu,split,1}$ and $\tau_{bu,split,2}$ are peak local bond resistance in the absence and presence, respectively, of confining stirrups.)

Table 5-1 Parameters used for the bond-slip relationship (Taerwe and Matthys 2013)

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<tr>
<td>Good bond cond.</td>
<td>All other bond cond.</td>
<td>Good bond cond.</td>
<td>All other bond cond.</td>
<td>Unconfined</td>
<td>Stirrups</td>
<td>Unconfined</td>
</tr>
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<td>$1.25\sqrt{f_{cm}}$</td>
<td>$2.5\sqrt{f_{cm}}$</td>
<td>$2.5\sqrt{f_{cm}}$</td>
<td>$1.25\sqrt{f_{cm}}$</td>
<td>$1.25\sqrt{f_{cm}}$</td>
</tr>
<tr>
<td>$\tau_{bu,\text{split}}$</td>
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<td>$8.0\left(\frac{f_{cm}}{25}\right)^{0.25}$</td>
<td>$5.0\left(\frac{f_{cm}}{25}\right)^{0.25}$</td>
<td>$5.0\left(\frac{f_{cm}}{25}\right)^{0.25}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$s_1$</td>
<td>1.0 mm</td>
<td>1.8 mm</td>
<td>3.6 mm</td>
<td>5.1</td>
<td>5.1</td>
<td>5.1</td>
</tr>
<tr>
<td>$s_2$</td>
<td>2.0 mm</td>
<td>4.0 mm</td>
<td>1.25</td>
<td>1.25</td>
<td>1.25</td>
<td>1.25</td>
</tr>
<tr>
<td>$a$</td>
<td>0.4</td>
<td>0.4</td>
<td>0.4</td>
<td>0.4</td>
<td>0.4</td>
<td>0.4</td>
</tr>
<tr>
<td>$\tau_{bf}$</td>
<td>$0.40\tau_{\text{max}}$</td>
<td>$0.40\tau_{\text{max}}$</td>
<td>0</td>
<td>$0.4\tau_{bu,\text{split}}$</td>
<td>0</td>
<td>$0.4\tau_{bu,\text{split}}$</td>
</tr>
</tbody>
</table>

1) $c_{\text{clear}}$ is the clear distance between ribs

(Note: $f_{cm}$ is the mean concrete compressive strength, $\varepsilon_s$ and $\varepsilon_{s,y}$ denote the steel strain and that at yielding, respectively.)
As mentioned above, the reinforcing bars were explicitly modelled with a circular cross section, which followed the actual arrangement of the slab reinforcement i.e. two contact layers in two directions at the slab top and bottom. To avoid penetrations among beam elements at large-deformation stages in the numerical model, the keyword *CONTACT_AUTOMATIC_GENERAL was applied to simulate the interaction between reinforcing bars. Such contact restraints between reinforcing bars within the slab cannot be neglected, because the experimental results showed that the top and bottom slab reinforcements resisted the applied load in the form of united nets at post-failure stages.

5.2.3 Loadings and Boundary Conditions

The quasi-static loading scheme controlled by displacements, the same as the actual experimental loading scheme, was applied to the numerical model by introducing the keyword *BOUNDARY_PRESCRIBED_MOTION_SET. The loading application was implemented by selecting the prescribed loading node and releasing its downward translational freedom only, then defining its motion mode as being controlled by displacements. A defined curve of the displacement versus the time was also required. To reduce dynamic effects, every 100-mm displacement travel took one second and every second had 500 time intervals. Moreover, the UDL in the form of the weights of steel blocks and sand bags in the experimental tests was equivalently simulated on the slab top surface through the keyword *LOAD_SEGMENT_SET.

For the boundary conditions of the modelled substructures, the bottom nodes of columns belonging to both the concrete solid elements and reinforcement beam elements were translationally constrained in three directions (i.e. x-, y- and z-directions). At removed columns, the same treatment as the experimental configuration was followed by having only column stubs above the slab and no supports underneath the slab. To obtain the reaction force of each column in the numerical model, the keyword *DATA_CROSS_SECTION_SET was applied to read the sum of the reaction force contributed by concrete and reinforcement elements. Accordingly, the keyword
*DATABASE_SECFORC was applied for post-processing when exporting the reaction force data from output files.

5.2.4 Modelling of Loading Tree

As described previously in the experimental tests (“CHAPTER 3 and CHAPTER 4”), all the slabs were quasi-statically loaded by an increasing UDL through a specially designed multi-point loading system imposed on the bay(s) having removed column(s) till failure. In order to replicate the true test conditions, the loading tree was explicitly modelled in the numerical model. Note that the main purpose of employing the loading tree was to transfer the applied load from the hydraulic jack equally to each loading point. As a result, the loading beam was simply supported between the gravity centres of the triangular steel plates. The same design approach was used to model the loading tree in the numerical model.

Figure 5-2(a) displays the modelling details of the loading tree. At the loading pin connecting the triangular loading plate to the loading pad, the rotational degree of freedoms (DOFs) in x-, y- and z-directions were released to simulate a ball-socket joint connection. To achieve a simply supported condition at two ends of the loading beam, one end had the rotational DOFs in x-, y- and z-directions released and the other had the rotational DOFs in y- and z-directions released and translational DOF in x-direction released. The loading point was located at the middle span of the loading beam, and two ends of the loading beam were located at the projection point of the gravity centre of the triangular shell plate. To simulate irregular deformations among the loading points (i.e. the displacements at each loading point on the bay(s) having the removed column(s) could be different), rubber blocks were created under the loading pads with different Young’s moduli.

With an applied loading speed of 100 mm and 500 time intervals per second, the rubber blocks were deformed variably, as shown in Figure 5-2(b). In this circumstance, the force readings at six loading points are presented in Figure 5-3, indicating an acceptable results of almost the same transferred load up to 100 kN among loading points. Throughout all
the experimental tests, the applied load at each loading point was found to be less than 25 kN. Therefore, the simulated loading tree was valid in applying the displacement-controlled load to the slab in the numerical model.

Figure 5-2 Loading tree model: (a) detailing; (b) deformed loading tree for examining
A complete geometrical element configuration of the numerical model of T1 is given in Figure 5-4. The dimension and position of reinforcing bars and the geometries of the slab and column were established according to the actual test specimen in T1. The distribution of loading points on the slab and the dimension of each part of the loading tree were also identical to the experimental ones.
Figure 5-4 Numerical model of T1: (a) concrete and loading tree; (b) reinforcement
5.3 Material Models

Extensive failures have been witnessed in a number of real-world collapse events of RC structures and the experimental tests conducted in this study. Appropriately simulating the highly nonlinear material behaviour is vital to produce reasonable numerical results. The constitutive models for concrete and reinforcement selected from LS-DYNA material library to be employed to the numerical analyses in this study are introduced below.

### 5.3.1 Concrete

To date, the continuous surface cap model (CSCM) used for concrete has exhibited characteristics of being accurate, stable and sophisticated in simulating RC frame structures under column loss scenarios (Pham et al. 2017a; Pham et al. 2017b; Yu et al. 2018). Hence, the CSCM performed by the keyword *MAT_159 was adopted to define the material properties of concrete solid elements. Figure 5-5 shows the parameters that need to be filled in the keyword with a total of seven subsections. These subsections will be explained and the parameters will be derived below.

![Figure 5-5 Keyword *MAT_159 in LS-DYNA](image)

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135
In subsections 1 and 2, the density of concrete (RO) was set to 2,400 kg/m³; the rate effect option (IRATE) was turned off because quasi-static loading scheme was always applied in the experiments in this study; the failure criterion using the maximum principal strain (ERODE) for element deletion was turned off, as other failure criteria were used (introduced in Section 5.3.3); the remaining parameters were set to be default values.

The main parameters defined for the CSCM are in subsections 3, 4 and 5. The elastic stress-strain relationship of concrete follows Hooke’s law, depending on the shear modulus \( G_c \) and the bulk modulus \( K_c \) which are formulated in Equations 5.6-5.7 where \( E_c \) is the elastic modulus formulated in Equation 5.5 according to Taerwe and Matthys (2013) and the Poisson’s ratio \( v_c \) was taken as 0.2. After the initial elastic stage, concrete yielding will occur. The yield surface of the CSCM is shown in Figure 5-6 with a smooth and continuous intersection between the shear surface and hardening compaction surface (cap), therefore referred to as the continuous surface cap model. A detailed description of the parameter derivations for the yield surface of the CSCM can be found in Murray (2007). For the shear surface, three sets of parameters, subjected to failure modes of tri-axial compression, torsion and extension, were used and determined based on Equations 5.8-5.19. For the cap surface, one set of parameters were used and determined based on Equations 5.20-5.21. Moreover, it was indicated by Murray et al. (2007) that pre-peak strain hardening was negligible. Therefore, it was not considered in this study and the default parameters are given in Equations 5.22-5.26.

\[
E_c = 21.5 \times 10^3 \times \left( \frac{f_c'}{10} \right)^{\frac{1}{2}} \tag{5.5}
\]

\[
G_c = \frac{E_c}{2(1 + v_c)} \tag{5.6}
\]

\[
K_c = \frac{E_c}{3(1 - 2v_c)} \tag{5.7}
\]

where \( f_c' \) denotes the concrete compressive strength.
Figure 5-6 Yield surface of CSCM (LSTC 2017)

\[ \alpha = -3.0000 \times 10^{-3}(f'_c)^2 + 3.1697 \times 10^{-1}f'_c + 7.7047 \]

\[ \theta = 1.3216 \times 10^{-5}(f'_c)^2 + 2.3548 \times 10^{-3}f'_c + 2.1401 \times 10^{-1} \]

\[ \lambda = 1.0500 \times 10^{-1} \]

\[ \beta = 1.9290 \times 10^{-2} \]

\[ \alpha_1 = 7.4735 \times 10^{-1} \]

\[ \theta_1 = -3.8895 \times 10^{-7}(f'_c)^2 - 3.9317 \times 10^{-6}f'_c + 1.5820 \times 10^{-3} \]

\[ \lambda_1 = 1.7000 \times 10^{-1} \]

\[ \beta_1 = 1.9972 \times 10^{-5}(f'_c)^2 + 2.2655 \times 10^{-4}f'_c + 8.1748 \times 10^{-2} \]

\[ \alpha_2 = 6.6000 \times 10^{-1} \]

\[ \theta_2 = -4.8697 \times 10^{-7}(f'_c)^2 - 1.8883 \times 10^{-6}f'_c + 1.8822 \times 10^{-3} \]

\[ \lambda_2 = 1.6000 \times 10^{-1} \]

\[ \beta_2 = 1.9972 \times 10^{-5}(f'_c)^2 + 2.2655 \times 10^{-4}f'_c + 8.1748 \times 10^{-2} \]
where $\alpha$, $\theta$, $\lambda$, and $\beta$ are tri-axial compression surface constant term, linear term, nonlinear term and exponent, respectively; $\alpha_1$, $\theta_1$, $\lambda_1$, and $\beta_1$ are torsion surface constant term, linear term, nonlinear term and exponent, respectively; $\alpha_2$, $\theta_2$, $\lambda_2$, and $\beta_2$ are tri-axial extension surface constant term, linear term, nonlinear term and exponent, respectively; $N_H$ and $C_H$ are hardening initiation and hardening rate, respectively; $R$ is cap aspect ratio; $X_D$ is cap initial location; $W$ is maximum plastic volume compaction; $D_1$ and $D_2$ are linear and quadratic shape parameters, respectively.

Subsection 6 in the keyword *MAT_159 includes damage parameters for strain softening of the concrete solid elements. One set of default parameters were used based on Equations 5.27-5.35.
\[ GFC = 100GFS \]  \hspace{1cm} 5.31

\[ PWRC = 5.0 \]  \hspace{1cm} 5.32

\[ PWRT = 1.0 \]  \hspace{1cm} 5.33

\[ PMOD = 0.0 \]  \hspace{1cm} 5.34

where \( B \) and \( D \) are ductile and brittle shape softening parameters, respectively; \( G_{FB} \) is fracture energy at \( f'_c = 10 \) MPa as a function of the maximum aggregate size; \( GFS \), \( GFT \) and \( GFC \) are fracture energies in pure shear stress, uniaxial tension and uniaxial stress, respectively; \( PWRC \) and \( PWRT \) are shear-to-compression and shear-to-tension transition parameters, respectively; \( PMOD \) is modified moderate pressure softening parameter.

The mesh size dependency of the CSCM in relation to the simulation convergence issues was studied by Murray et al. (2007), where it was found that using a reasonable mesh size can only have very limited impact on strain softening. Similar numerical studies of Pham et al. (2017a), Pham et al. (2017b), Yu et al. (2018) and Qian et al. (2018) successfully employed the CSCM with the characteristic concrete solid element size ranging from 13.4 mm to 25 mm. In this numerical study, considering the geometry and computing efficiency, the characteristic concrete solid element size of 22.4 mm was adopted.

5.3.2 Reinforcement

The keyword \*MAT_PIECEWISE_LINEAR_PLASTICITY was employed to model the reinforcement. A multilinear elasto-plastic material model is used in this keyword by defining the elastic stage through Young’s modulus and the plastic stage through a user-defined stress-strain curve obtained from the uniaxial tensile test. It is noted that the function of strain rate effect was turned off due to the quasi-static loading scheme of the experimental tests. A mesh size of 25-mm length was applied to all the slab reinforcement beam elements.
5.3.3 Failure Criteria

Reviewing the slab failure mode in the experimental tests described in “CHAPTER 3 and CHAPTER 4”, the flexural and punching shear failures were the main reasons causing the loss of the slab load carrying capacity. Figure 5-7 shows the performance of the load-displacement curve of the proposed numerical model simulating the flexural failure of specimen T1 without defining any failure criterion for concrete solid elements. It can be seen that the flexural behaviour at the early stage prior to the first peak load (FPL) was able to be reasonably captured in the simulation thanks to the defined strain softening in the CSCM. But purely relying on the CSCM was incompetent when it came to more severe flexural failure at large deformations, especially after reaching the FPL accompanied with yield lines and full-depth flexural cracks observed in the experimental test of T1. Moreover, simulations, on punching shear failure which was brittle and could cause a sudden load drop in the load-displacement curve, required a new failure criterion for the concrete solid elements.

Figure 5-7 Simulation of the load-displacement curve for T1 without defining failure criterion
To address the above mentioned modelling challenges, the keyword *MAT_ADD_EROSION was introduced to the numerical model. A strain-based criterion for erosion is executed by this keyword to better capture severe concrete cracking and spalling. With this criterion, the concrete solid elements will be deleted when their strains exceed the defined erosion strain. Specifically, two types of strain-based criteria were applied in terms of different failure modes, which are the maximum effective strain $\varepsilon_{c,max}$ applied to the main slab regions for the flexural failure and the maximum shear strain $\gamma_{c,max}$ applied to the slab-column regions for the punching shear failure. So far, the existing literature has not provided any clear rules to determine the strain values of these two criteria. In this numerical study, appropriate values of these two criteria were calibrated based on the experimental results and observations. Furthermore, it was found that applying a combination of these two failure criteria to the numerical modes is satisfactory to replicate the experimental process.

5.4 Modelling of Corner Column Removal Scenario (T1)

5.4.1 Failure Criteria Calibration

It was observed that the failure mode of T1 was fully dominated by flexural failure. The failure criterion using the maximum effective strain $\varepsilon_{c,max}$ was applied to simulate such a failure. To determine the value of $\varepsilon_{c,max}$, a limit study was carried out by the trial and error method. With different values assumed for $\varepsilon_{c,max}$, the relationships of the applied load versus the vertical displacement of removed corner column are shown in Figure 5-8. It can be seen that prior to the FPL, the numerical load-displacement curves with all the values of $\varepsilon_{c,max}$ have almost identical stiffness and are in good agreement with the experimental one. The difference between the simulated and the experimental FPL is found to be less than 5%. Nevertheless, only few concrete solid elements were deleted, implying that the failure criterion was not activated up to the FPL. The well captured flexural behaviour of the slab before the FPL was contributed by strain softening defined in the CSCM. After the FPL, It was found that using $\varepsilon_{c,max} = 0.01$ and 0.02 could cause overmuch element deletions, which was reflected by underestimations of the residual load.
carrying capacity of the slab. These can be further demonstrated by the failure modes (Figure 5-9(b) and (c)) in the simulation showing significant detachment of the corner slab from the remaining slab, comparing to the observed failure mode (Figure 5-9(a)) in the experiment. On the other hand, use of $\varepsilon_{c,max} = 0.05$ resulted in an overestimation of the residual load carrying capacity and less deletions of solid elements shown in Figure 5-9(f). Using $\varepsilon_{c,max} = 0.03$ and 0.04 was found to reasonably match the load-displacement curves and the failure modes shown in Figure 5-9(d) and (e). With the performance of $\varepsilon_{c,max} = 0.03$ and 0.04 examined by simulating the rest of the experimental tests (i.e. T2, S-E and S-EI), using $\varepsilon_{c,max} = 0.03$ was found to be able to produce the better fits overall.

Figure 5-8 Load-displacement curves for T1 using different values of the maximum effective strain (Note: FPL denotes first peak load.)
Figure 5-9 Failure modes of T1: (a) experimental observation; (b) simulation using $\varepsilon_{c,max} = 0.01$; (c) simulation using $\varepsilon_{c,max} = 0.02$; (d) simulation using $\varepsilon_{c,max} = 0.03$; (e) simulation using $\varepsilon_{c,max} = 0.04$; (f) simulation using $\varepsilon_{c,max} = 0.05$
5.4.2 Numerical Model Validation

Following the calibration of the failure criterion, the proposed numerical model is validated by comparing crack patterns, vertical displacement and strain results herein. Crack patterns on the top and bottom surfaces of the slab of specimen T1 observed in the experiment are compared to the numerical ones, as exhibited in Figure 5-10. In the simulation, cracks were not able to be tracked using the CSCM, instead, the damage index termed as the effective plastic strain in LS-DYNA is equivalently produced in contours. In addition to the full-depth flexural cracks developed in the experiment which were satisfactorily reproduced by deleting concrete solid elements in the simulation (Figure 5-9(d)), other relatively thin cracks on the top and bottom surfaces of the slab could also be simulated by the distribution of the effective plastic strain of the concrete solid elements.
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Figure 5-10 Crack patterns at top and bottom surfaces of the slab for T1: (a) plastic strain contours at top surface in the simulation; (b) crack pattern at top surface in the experiment; (c) plastic strain contours at bottom surface in the simulation; (d) crack pattern at bottom surface in the experiment.

The measured vertical displacement of T1 along the diagonal line C1-C5 is compared to the numerical results at key stages in Figure 5-11. It is indicated that a very similar deformed shape of the slab was predicted numerically. Strain comparisons are based on three selected strain gauge readings (SB15, SB17 and SB19) against the numerical strain results at the same locations on the bottom reinforcement passing through columns C1 and C4. Figure 5-12 illustrates that reasonable matching was obtained. It is noted that the much larger strain value of ST17 than those of SB15 and SB19 was due to the formation of a yield line, which can be referred to Section 3.3.5 “Strain Development”. With the validated results from the displacements and strains, the proposed numerical model is confirmed to be reliable.

Figure 5-11 Comparison of the slab vertical displacement profiles for T1 (Note: IC and FPL denote initial cracks and first peak load, respectively.)
Figure 5-12 Strain comparison for T1 (Note: SB are strain gauges glued to the bottom slab reinforcement. The numbering of SB15, SB17 and SB19 can be referred to Figure 3-3(c).)

5.5 Modelling of Corner Column Removal Scenario (T2)

5.5.1 Failure Criteria Calibration

It is evidenced in Section 5.4 that the failure criterion using the maximum effective strain is satisfactory in simulating the flexural failure occurred in T1. However, this failure criterion was found to be unsuitable for simulating punching shear failure at slab-column connections, as this brittle failure requires large amount of the concrete solid elements to
be deleted instantly in the numerical model. In order to simultaneously capture both flexural failure and punching shear failure in the T2 model, the failure criterion using the shear strain $\gamma_{c,max}$ within the same keyword *MAT_ADD_EROSION was selected and applied to the slab-column connection regions covering an area of 450 mm $\times$ 450 mm, and $\varepsilon_{c,max} = 0.03$ remained for the rest of the slab for simulating flexural failure. Note that the dimensions of the slab-column connection area using $\gamma_{c,max}$ were based on the control perimeter of the punching shear resistance stipulated in ACI318 (2014), where the side length of the control perimeter is calculated as the overall width of the column plus four times the slab effective depth (i.e. $150 + 4 \times 72 = 438$ mm, 450 mm was taken due to the solid element size of 25 mm). A schematic view of the definition of these two regional failure criteria in the T2 model is provided in Figure 5-13.

![Figure 5-13 Definition of regional failure criteria for T2](image)

Multiple values of $\gamma_{c,max}$ were studied in the T2 model. The performance of these values on the load-displacement relationships relative to the experimental one is presented in Figure 5-14. As this failure criterion was only applied to the slab-column regions governing the punching shear strength, the comparison of the $\gamma_{c,max}$ values was focused at the FPL initiated by punching shear failure in T2. It was found that using $\gamma_{c,max} = 0.005$
could cause an underestimation of the FPL, while the FPL was overestimated by using \( \gamma_{c,\text{max}} = 0.02 \) and 0.03. A reasonable fit with the test results was achieved by using \( \gamma_{c,\text{max}} = 0.01 \) with an acceptable difference of 4% from the experiment at the FPL. The disparities between the numerical and experimental load-displacement curves can be seen at load drops after the FPL in Figure 5-14 where the load falls dramatically in the simulation more than that in the experiment. Moreover, unlike the experiment where punching shear failures at columns C6 and C8 occurred separately with about 67-mm displacement difference, the modelled punching shear failures at these two columns occurred successively almost at the same time. These are because: (1) the failed concrete solid elements were deleted directly with no considerations of concrete interlocking and friction on the punching shear surface in a real situation, therefore leading to a larger load drop at punching shear failure in the simulation, and (2) the numerical model had the idealistic nature of symmetry on the loading, geometry and material, which facilitated the occurrence of two very adjacent punching shear failures at two symmetric columns. On the other hand, despite the larger load drops, the applied load in the simulation caught up subsequently until the post-failure peak load (PPL) was achieved.

Figure 5-14 Load-displacement curves for T2 using different values of the shear strain
5.5.2 Numerical Model Validation

Corresponding to $\gamma_{c,max} = 0.01$, the simulated crack patterns and failure modes of T2 are further compared to the experimental observations. Figure 5-15 evidences that the final failure modes of T2 are effectively modelled. Particularly, punching shear failures at columns C6 and C8, full-depth flexural cracks and their locations can be alternatively visualised in the numerical model. In Figure 5-16, the distribution of the effective plastic strain contours can be equivalent to the simulated cracks. For both top and bottom surfaces of the slab, the crack patterns in the simulation are similar to the experimental ones in terms of the location, direction and density of cracks. Figure 5-17 depicts the numerical and experimental vertical displacements for T2 along the diagonal line C5-C9. A similar deformed shape of the slab was observed from the numerical model and experimental test with good agreements at the IC and FPL and a reasonable correlation at the PPL.

(a)
Figure 5-15 Failure modes of T2: (a) experimental observation; (b) simulation
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Figure 5-16 Crack patterns at top and bottom surfaces of the slab for T2: (a) plastic strain contours at top surface in the simulation; (b) crack pattern at top surface in the experiment; (c) plastic strain contours at bottom surface in the simulation; (d) crack pattern at bottom surface in the experiment

Figure 5-17 Comparison of the slab vertical displacement profiles for T2 (Note: IC, FPL and PPL denote initial cracks, first peak load and post-failure peak load, respectively.)

5.6 Modelling of Edge Column Removal Scenario (S-E)

5.6.1 Failure Criteria Calibration

Said and Elrakib (2013) stated that reinforcing with stirrups can considerably improve the shear strength by enhancing the effect of dowel action and restricting the opening of shear cracks. In reference to Figure 3-3(a) and (c), and Figure 4-2(a) and (c), stirrups were arranged at edge columns C1, C4 and C7 in test S-E. With the successful application of the failure criterion of the shear strain $\gamma_{c,max}$ to simulate punching shear failure in the T2 model, the same failure criterion in the S-E model was employed to these column regions with an area of 450 mm $\times$ 300 mm but a higher value of 0.08. The $\gamma_{c,max} = 0.08$ at the column regions reinforced with stirrups was calibrated by a limit study which compared
the load-displacement relationship and the eroding state of concrete. Figure 5-18 indicates the load-displacement fits of the S-E model, under several shear strain values at the columns reinforced with stirrups, with the test result. More severe failures than the experimental observation (Figure 5-19(a)) were detected when $\gamma_{c,max} = 0.03$, 0.05 and 0.07 (Figure 5-19(b), (c) and (d)), consequently causing earlier load drops at the PPL or shorter post-failure stages. With $\gamma_{c,max} = 0.1$, the load-displacement curve became stiffer and less element deletions at the columns reinforced with stirrups (Figure 5-19(f)) in contrast to the test results. While using $\gamma_{c,max} = 0.08$ was found to be suitable in respect of 6 kN and 3 kN load differences between the experiment and simulation at the FPL and PPL, respectively, with reasonable element deletions (Figure 5-19(e)). Note that $\gamma_{c,max} = 0.01$ was employed to the rest of the column regions without stirrups and $\varepsilon_{c,max} = 0.03$ was employed to the remaining slab. The definition of the regional failure criteria is shown in Figure 5-20.

Figure 5-18 Load-displacement curves for S-E using different values of the shear strain at columns reinforced with stirrups
Figure 5-19 Failure at column C7 reinforced with stirrups in S-E: (a) experimental observation; (b) simulation using $\gamma_{c,max} = 0.03$; (c) simulation using $\gamma_{c,max} = 0.05$; (d) simulation using $\gamma_{c,max} = 0.07$; (e) simulation using $\gamma_{c,max} = 0.08$; (f) simulation using $\gamma_{c,max} = 0.1$
5.6.2 Numerical Model Validation

With the defined regional failure criteria, the numerical model has demonstrated a competency in reproducing the load-displacement relationship for the test specimen S-E. Other validations are further carried out in this section. Comparisons of typical failure modes between the experiment and simulation are demonstrated in Figure 5-21 where punching-like cracks at columns C1, C2, C7 and C8, severe punching shear failure at column C5 and yield lines were all vividly modelled. The simulated crack patterns at top and bottom surfaces of the slab for S-E versus the experimental observed ones are presented in Figure 5-22. The yield lines and visible cracks are in the form of large amount of element deletions and concentrations of large plastic strains in the simulation (see Figure 5-22(a) and (c)), respectively, agreeing well with the crack patterns drawn in the experiment (see Figure 5-22(b) and (d)). Furthermore, the slab vertical displacement profiles at critical stages (IC, FPL and PPL) along the slab edge C1-C5 are compared in Figure 5-23 between the simulation and experiment. Fine matching can be seen within...
the IC and FPL stages, whilst a reasonable similarity yet a relatively larger difference at the PPL is disclosed.

*Figure 5-21 Failure modes of S-E: (a) experimental observation; (b) simulation*
Figure 5-22 Crack patterns at top and bottom surfaces of the slab for S-E: (a) plastic strain contours at top surface in the simulation; (b) crack pattern at top surface in the experiment; (c) plastic strain contours at bottom surface in the simulation; (d) crack pattern at bottom surface in the experiment
5.7 Modelling of Edge-Interior-Column Removal Scenario (S-EI)

As mentioned previously in “CHAPTER 4”, S-E and S-EI were identical test specimens with different column(s) being removed. Therefore, the same definition of regional failure criteria in the S-E model (Figure 5-20) was used in the S-EI model, as demonstrated in Figure 5-24. The values of the failure criteria for the S-EI model were also kept consistent with those used in the S-E model. Unlike the T1, T2 and S-E models, only a half model was created for the S-EI model to save the computing time on account of the symmetry of both the slab and the loading tree. In consequence, extra boundary conditions were added on the symmetric plane where all the nodes of the concrete solid elements and reinforcement beam elements were translationally restrained in the direction perpendicular to this plane.
Figure 5-24 Definition of regional failure criteria for S-EI

Figure 5-25 plots the comparison of the load-displacement relationships between the simulation and experiment of S-EI. It can be seen that the simulated FPL occurred prior to the experimental one yet with only 4% difference in magnitude. This might be attributable to different loading conditions. To be specific, the applied point load from the hydraulic jack in the experiment could not be equally transferred to each loading point through the loading tree, especially at large deformations where the displacement of each loading point was quite different. This is unlike an idealistic load distribution over the loading points in the simulation. After the FPL, the applied load in the simulation increased again with an approximate stiffness to that in the experiment. At the PPL, a vertical displacement of 453.8 mm at the removed interior column and an applied load of 253.3 kN were obtained from the S-EI model, in comparison to a vertical displacement of 511.7 mm and an applied load of 274.1 kN in the experiment.
The experimental and numerical failure modes of S-EI are compared in Figure 5-26. It demonstrates that key failures were able to be mimicked, specifically including punching-like cracks at columns C6, C7 and C9, punching shear failure at C8 with the slab reinforcement being pulled out, and the location of yield lines. The experimental and numerical crack patterns of S-EI are depicted in Figure 5-27. On the top surface of the slab, the relatively dense cracks at slab-column regions as well as the development of crack connections from column to column was well captured. On the bottom surface of the slab, the main path of cracks in the simulation followed lines of C4-C5 and C5-C9, almost identical to the observed cracks in the experiment.
Figure 5-26 Failure modes of S-EI: (a) experimental observation; (b) simulation
Figure 5-27 Crack patterns at top and bottom surfaces of the slab for S-EI: (a) plastic strain contours at top surface in the simulation; (b) crack pattern at top surface in the experiment; (c) plastic strain contours at bottom surface in the simulation; (d) crack pattern at bottom surface in the experiment

5.8 Modelling of Interior Column Removal Scenario (ND)

In Sections 5.4-5.7, the proposed numerical models have been validated with reasonable matching between experimental and numerical results. The validated models are subjected to two corner column removal scenarios, an edge column removal scenario and an edge-interior-column removal scenario. In this section, the same modelling technique is applied to simulate the experimental test of an interior column removal scenario performed on a similar flat plate specimen (ND). The test results are obtained from the
literature and utilised here to further validate the proposed numerical model beyond the conducted experimental tests in present research.

5.8.1 Brief of the Experimental Test

The selected experimental study (Qian and Li 2015), which aimed to investigate progressive collapse of flat plates under an interior column removal, was performed on a 2×2-bay RC slab directly supported by columns. The dimensions and reinforcement detailing of the test specimen are shown in Figure 5-28, where the slab had an overall dimension of 3,750 mm × 3,750 mm × 55 mm and the column had a cross-section of 200 mm × 200 mm. Columns had a height of 300 mm underneath the slab and a height of 55-mm stub above the slab. A total of eight columns excluding the interior one were constructed and spaced at a centre-to-centre distance of 1,500 mm in two directions. In the test specimen, four edges of the slab had 375-mm overhangs to simulate the continuity effects of the adjacent slabs. The concrete cover for the slab both top and bottom reinforcements was 7 mm and for the columns were 20 mm. The concrete compressive strength was 25.2 MPa for both the slab and column. The slab reinforcement was given as a yield strength of 500 MPa and an ultimate yield strength of 617 MPa. The column reinforcement was given as a yield strength of 529 MPa and an ultimate yield strength of 608 MPa.

Figure 5-29 provides an overview of the experimental setup. A similar loading scheme to the ones introduced in “CHAPTER 3 and CHAPTER 4” was adopted in this test. A twelve-point loading tree was positioned on the slab with all the loading points being equally spaced and connected to a hydraulic jack. The loading tree with a connection to the hydraulic jack enables the UDL on the bay to be increased until failure of the slab. On the overhangs, a constant design live load was applied by laying out steel weights to simulate the boundary restraints.
Figure 5-28 Specimen details of ND (Qian and Li 2015) (Unit: mm)

Figure 5-29 Experimental setup of ND (Qian and Li 2015)
5.8.2 Numerical Model Validation

Figure 5-30 compares the load-displacement relationships between the simulation and experiment of ND. Generally, the simulated trend of the applied load against the vertical displacement at the removed interior column was found to be able to well capture the experimental one. At an early stage, the flexural behaviour was dominant in both experiment and simulation, which ended up with punching shear failure at the FPL where the ND model underestimated the applied load by 5% and overestimated the displacement by 12%. Thereafter, the applied load in the ND model, after a short suspension stage, continuously increased up to the PPL due to tensile membrane action, which matched reasonably with the experiment. At the PPL, a vertical displacement of 156.9 mm at the removed interior column and an applied load of 205 kN were obtained from the ND model, in comparison to a vertical displacement of 160.7 mm and an applied load of 206 kN in the experiment. Furthermore, crack patterns at top and bottom surfaces of the slab for ND were also mimicked as shown in Figure 5-31. On the top surface of the slab, it can be seen that cracks originated from columns and cracks between columns and connected with each other were experimentally replicated. On the bottom surface of the slab, the simulated dense cracks around the removed interior column and cracks along the paths from corner columns to the removed interior column show a high similarity to the experimental ones.
Figure 5-30 Load-displacement curve for ND
Figure 5-31 Crack patterns at top and bottom surfaces of the slab for ND: (a) plastic strain contours at top surface in the simulation; (b) crack pattern at top surface in the experiment; (c) plastic strain contours at bottom surface in the simulation; (d) crack pattern at bottom surface in the experiment

5.9 Concluding Remarks

To numerically investigate the progressive collapse behaviour of RC flat plate substructures under multiple column loss scenarios, a set of 3D nonlinear finite element modelling techniques, using LS-DYNA software, are proposed in this chapter. Firstly, general considerations of the numerical model are introduced, including appropriate definitions of element types, bond-slip relationships, loadings and boundary conditions. Secondly, to achieve the same loading conditions as those in the experiment, the loading tree was explicitly modelled and proven to be validly used in the numerical model. Thirdly, the constitutive models for concrete and reinforcement are introduced with step-by-step explanations on each parameter. Finally, the numerical models are validated, which in turn provides a numerical solution to predict the collapse behaviour and capacity of RC flat plate substructures. Throughout the numerical study, the following key findings are highlighted herein:

- With careful definition of the bond-slip relationship proposed by fib Model Code for Concrete Structures 2010 and the CSCM model for concrete in the numerical...
model, an accurate prediction of the flexural behaviour of the slab prior to the first peak load (FPL) can be obtained.

- After the FPL and under large deformations, applying the calibrated failure criteria using the maximum effective strain to the numerical model can overcome convergence issues and achieve good fits with test results.

- The calibrated failure criteria using the shear strain shows a competency to capture brittle punching shear failure and a sudden load drop in the load-displacement curve. In addition, a limit study of the shear strain proves that a higher value should be given at the column regions reinforced with stirrups.

- The validated numerical models are capable of predicting the overall structural behaviour of RC flat plate substructures under different column removal scenarios, in terms of the load-displacement relationships, failure modes, crack patterns, the slab vertical displacement profiles and strain results.
CONCLUSIONS AND FUTURE WORK

6.1 Summary of Research Findings

The primary goal of research on progressive collapse of reinforced concrete (RC) flat plate structures is to establish a set of rational design guidelines, which can improve the collapse resistance or safety level of flat plate structural systems when unexpected abnormal events occur. This is crucial to contribute to the future development of relevant Australian Standards and also the further amendment of relevant existing specifications in other international Standards. However, the current research progress has been mainly limited by the following identified research gaps:

- Progressive collapse investigations in the literature have predominantly focused on frame systems with relatively less efforts devoted to flat plate structural systems. This leads to a lack of fundamental understanding with respect to the structural behaviour and resisting mechanisms of progressive collapse of flat plates.

- Existing studies on flat plates, mostly subjected to isolated slab-column connections or small deformations, cannot account for the actual system responses and post-failure behaviour in a collapse event.

With the above incentives, the research conducted in this thesis was motivated to enrich the current knowledge pool of progressive collapse of flat plates by (1) carrying out a series of experimental tests under critical column removal scenarios, (2) analytically predicting the flexural capacity of the test specimens and (3) establishing a set of 3D nonlinear finite element modelling techniques for numerical analyses.

In summary, the main findings of this research work are concluded as below:
• Chapter 3 includes two experimental tests of RC flat plate substructures subjected to two consecutive corner column removal scenarios. The first scenario (T1) was removing one corner column strengthened by torsional strips. The second scenario (T2) was removing the corner column surrounded by overhangs, diagonally opposite to the first removed one. Different failure modes were observed in the two tests with only flexural failure observed in T1 while combined flexural and punching shear failures observed in T2. In comparison to T1, a noticeable post-failure stage was obtained in T2 resulting in the ultimate collapse resisting capacity of 1.2 times higher than that of T1. The progressive collapse risk of flat plates was therefore found to be reduced by having surrounding overhangs along the slab edges.

• Chapter 4 includes two experimental tests of RC flat plate substructures subjected to edge-column (S-E) and edge-interior-column (S-EI) removal scenarios. Both failure and post-failure stages were observed in the two tests. The peak loads in the later stage were found to be 9.6% and 81.9% higher than these in the former stage in S-E and S-EI, respectively. Although a higher risk of progressive collapse under the edge-interior-column removal scenario was found by the ultimate collapse resisting capacity in S-EI of 26.9% lower than that in S-E, a more ductile failure behaviour under the same column removal scenario was revealed by the deformation capacity in S-EI of 112% larger than that in S-E.

• In Chapters 3 and 4, the analytical predictions using the yield line theory on the flexural capacity of the test specimens were found to be accurate under corner column removal scenarios when having laterally unrestrained boundary conditions. However, beyond this scenario, the yield line theory might be inapplicable to estimate the flexural capacity, which was evidenced by the resulting overestimations under edge-column and edge-interior-column removal scenarios.

• In Chapter 5, a set of 3D nonlinear finite element modelling approaches, using LS-DYNA software, was established. The proposed numerical models were validated through multiple comparisons against the experimental results in respect of loads, displacements, strains, failure modes and crack patterns, from which satisfactory and reliable simulation results were obtained.
6.2 Recommendations for Future Work

For the experimental study, a series of quasi-static large-displacement tests were performed in this research on 1/3-scale, 2×2-bay, RC flat plate substructures under critical column removal scenarios. Similar experimental tests under a dynamic loading scheme need to be carried out. Additionally, similar experimental tests using a strengthening method by externally bonding composite materials to the slab are worthy of being conducted.

For the numerical study, following the proposed numerical models which have been validated, parametric studies, including reinforcement detailing of the slab, column size, concrete strength and slab thickness, should be carried out. The numerical results from the parametric studies can demonstrate valuable design strategies to resist progressive collapse in RC flat plate structures. Further, numerical investigations of the structural behaviour and collapse resistance of full-scale RC flat plate structures under different column removal scenarios are expected.
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REFERENCES


REFERENCES


EXPERIMENTAL DESIGN

“APPENDIX A” mainly focuses on the experimental design which describes the assumptions and design procedures of a general prototype car park, and the extractions and scaling of the substructures used for collapse tests. In contrast to the relevant contents in “CHAPTER 3 and CHAPTER 4”, supplementary information is provided herein in a more detailed manner.

A.1 Prototype Building

A.1.1 Predetermined Design Assumptions

The designed prototype building is a 4-storey and 4×4-bay RC flat plate structure, representing a car park to be built on Gold Coast, QLD, Australia, subjected to a medium vehicle traffic condition. The prototype building has a uniform slab thickness of 270 mm and a centre-to-centre distance of 6,000 mm between columns in two directions. The distance between floors is 3,000 mm and the columns have a cross-section of 450 mm × 450 mm. A schematic view of the prototype building is shown in Figure A-1.
According to Building Code of Australia (BCA 2011), a Building Importance Level 2 was taken for the design, considering that the prototype car park does not contain a large number of people. The annual probability of exceedance for wind corresponding to Building Importance Level 2 is 1:500. Note that snow and earthquake were not considered in the design in the Gold Coast region.

According to AS3600 (2009), durability was designed to comply with requirements for concrete quality and curing. The exposure classification of the prototype building was determined as B1, i.e. when the building is located at near-coast (1 km to 50 km from coastline) and any climatic zones. With respect to the class B1, it is required that a minimum concrete characteristic compressive strength of 32 MPa should be used, and concrete needs to be continuously cured at least seven days and reach a minimum average strength of 20 MPa before stripping off formwork. For erosion protection, concrete cover is required to be no less than 40 mm when standard formwork and compaction are used.

A.1.2 Design Procedures

In the prototype building, a superimposed dead load of 3 kPa ($g_1$) taking account of braking, ceilings, floor finishes and ductwork, and a live load of 5 kPa ($g$) under a medium vehicle traffic condition in accordance with AS/NZS1070.1 (2002) were assumed when
designing the slab. With a uniform thickness of 270 mm and an assumption of 1.5% reinforcement by volume in the slab, the self-weight of the slab was calculated as 6.68 kPa \((g_2)\). By following design instructions stipulated in AS/NZS1070.2 (2002), the maximum wind load \((w)\) were calculated as 1.59 kPa for windward walls, 0.72 kPa for leeward walls and 1.30 kPa for the roof. For a complete loading consideration, the load combinations recommended by AS/NZS1070.0 (2002) are shown in Table A-1. It is noted that Table A-1 only lists the combination of different types of loads, yet the live load and wind load have various situations.

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Combination</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.35g</td>
</tr>
<tr>
<td>2</td>
<td>1.2g + 1.5q</td>
</tr>
<tr>
<td>3</td>
<td>1.2g + w + 0.4q</td>
</tr>
<tr>
<td>4</td>
<td>0.9g + w</td>
</tr>
</tbody>
</table>

By analysing these load combinations, the design moments were determined as -361 kN.m at the slab ends and 195 kN.m at middle span. Accordingly, the reinforcement ratios for the bending design of the slab were derived as 0.49% for the top reinforcement at column strips, and 0.25% for both the top reinforcement at middle strips and the bottom reinforcement. Then, the serviceability was checked and found to satisfy the total deflection divided by the effective span \(\Delta/L_{ef}\) of less than 1/250.

**A.2 Test Specimens**

Considering the limited space in the laboratory, substructures were extracted from the ground floor of the prototype building, and the dimensions of the substructures were 1/3-scaled for construction. This scaled factor reduced the dimensions of the substructures to a slab thickness of 90 mm, a column height of 1,000 mm and a column cross-section of
150 mm × 150 mm. Concrete cover in the scaled slab was 10 mm for the top and bottom layers of reinforcements. Given a typical aggregate size of 20 mm in full-scale buildings, the aggregate size was reduced to 6 mm in the scaled slab. The same nominal compressive concrete strength of 32 MPa was used for all specimens. The same reinforcement ratios as these of the prototype building were also used.

Figure A-2 shows a typical slab reinforcement arrangement specified in AS3600 (2009) to which a two-way slab is required comply. Slight changes were made for simplicity. For the slab top reinforcement in test specimens, the length of reinforcing bars was changed from combined $0.2L_n$ and $0.35L_n$ to all $0.35L_n$ in column strips and $0.2L_n$ in middle strips, where $L_n$ is the length of clear span between columns. For the bottom reinforcement, the reinforcing bars were changed from combined overlaps and discontinuities to all continuities at columns. In order to provide sufficient anchorage compensating the loss of continuity effect from the adjacent slabs, hooks were designed for the slab top and bottom reinforcements at slab edges. The slab reinforcement of the test specimens is given in Figure 3-3(a) and (c), and Figure 4-2(a) and (c), and further reiterated in “CHAPTER 3 and CHAPTER 4”.
As mentioned in previous sections, an alternate load path approach has been broadly employed to investigate progressive collapse of structures by notionally removing one or more columns. Following this approach, critical column removal scenarios, such as single corner column, single edge column, and double interior and edge columns, were considered in this study. As a result, the substructures were extracted from different locations of the ground floor of the prototype building, as shown in Figure A-3. Test 1 and Test 2 represent two single column removal scenarios with different slab corner reinforcement configurations (see Section 3.2.1 “Specimen Design”). Test 3 and Test 4 represent the scenarios of removing either a single edge column, or a single interior and a single edge columns.
Figure A-3 Specimen extraction of each test from the ground floor of the prototype building

For all substructures as seen in Figure A-3, an extension of 1,500-mm overhangs (scaled down to 500 mm with an 1/3 scale factor in test specimens) was considered for the slab edges, along which the slab was supposed to be continuous with the adjacent slabs in the prototype building. During every experimental testing, a constant 5-kPa UDL was applied to the overhangs simulating the design live load. By doing so, the continuity effect provided by the adjacent slabs was simulated. Such an approach has been often applied in experimental tests (Qian and Li 2012b; Yi et al. 2014; Qian and Li 2015; Xue et al. 2018).

To comply with the relevant specifications of AS3600 (AS3600 2009), torsional strips or spandrel beams are required to be designed at exterior columns (i.e. edge and corner columns). Considering that RC flat plate structures with a clean floor appearance (i.e. no (spandrel) beams, column capitals, drop panels) are popularly employed in practice, torsional strips in the form of internally reinforcing corner and edge slab-column connections with stirrups were adopted in this experimental program (detailing of torsional strips can be referred to Figure 3-3 and Figure 4-2).
APPENDIX B

EXPERIMENTAL PREPARATION

“APPENDIX B” is a complement to experimental technique details lacked in “CHAPTER 3 and CHAPTER 4”, especially for reinforcement arrangement and concrete casting. The experimental preparation exhibits how the test specimens were constructed step by step.

B.1 RC Columns

Prior to the slab casting, RC columns were required to be positioned together with the slab formwork and cured at least two weeks before pouring the slab. Unlike the slab which required a large amount of concrete, concrete for columns was mixed and self-cast in the laboratory. The mix ratio for the column concrete was calculated in accordance with the Mix Design and Test Methods in Kett (2009) guidelines. In order to avoid undesirable failure occurring in the columns, a target concrete strength of 40.3 MPa was required for the mix design. The final concrete mix ratio of each content per cubic meter corresponding to the strength of 40.3 MPa is listed in Table B-1. The flow chart in Figure B-1 is the concrete mixing procedures in reference to AS1012.1 (2014) which was followed when casting columns.

Table B-1 Column concrete mix ratio per cubic meter

<table>
<thead>
<tr>
<th>Weight of coarse aggregates (kg)</th>
<th>Weight of fine aggregates (kg)</th>
<th>Weight of cement (kg)</th>
<th>Weight of water(kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>692</td>
<td>782</td>
<td>512</td>
<td>264</td>
</tr>
</tbody>
</table>
Figure B-1 Concrete mixing procedures (AS1012.1 2014)

Figure B-2 provides an overview of the preparation of column casting. After that the concrete was properly mixed in a concrete mixer and satisfied the slump test, it was gradually poured into the formwork by shovels and vibrated to remove air voids. When the formwork was full with concrete, the top surfaces of columns were finished smoothly and uniformly by trowels. During each column casting, cylinders with a standard size of 100-mm diameter and 200-mm height complying with AS1012.1 (2014) were filled with concrete for material testing. The formwork was stripped off after at least two weeks, prior to which wet rags were placed on the column top surface to keep the concrete moist. Figure B-3 displays three stages of constructing the RC columns including reinforcement welding, concrete casting and formwork stripping off. Note that reinforcing bars at the bottom end of the column were welded to a 20-mm thick steel plate which was bolted to
the strong floor in laboratory, and the extra length of reinforcing bars sticking out at the top end of the column was to penetrate the slab during the slab concrete casting and form column stubs above the top surface of the slab.

Figure B-2 Column casting preparation

Figure B-3 Column reinforcement welding, column after casting and column after stripping off formwork
B.2 RC Slabs

B.2.1 Reinforcement

After all RC columns being constructed and achieved at least 70% of the target concrete strength, they were positioned on the strong floor and set up with formwork. The slab formwork, as shown in Figure B-4, was made of dimensioned plywood. The slab top and bottom reinforcements were positioned in the formwork based on the designed spacing. To prevent the reinforcement mesh moving during concrete casting, reinforcing bars were tied together at intersections to generate a solid reinforcement net. In order to room a 10-mm concrete cover, two sizes of chairs (see “Chairs for top and bottom reinforcements” in Figure B-4) were used for the top and bottom reinforcements. Additionally, 10-mm thick battens were placed at four sides of the formwork to ensure the concrete cover of the slab sides. Strain gauges were glued to reinforcing bars at selected locations, and wrapped by gauze and epoxy (see “Strain gauge” in Figure B-4) to be waterproof. Cables of strain gauges were wired (see “Strain gauge wiring” in Figure B-4) on battens bridging over the formwork. Torsional strips (see “Torsional strips” in Figure B-4), as described in Section “EXPERIMENTAL DESIGN”, were arranged in the form of rectangular cages using stirrups.

Figure B-4 Slab reinforcement arrangement
B.2.2 Concrete

The slab concrete was delivered and cast by a commercial concrete company. A nominal compressive concrete strength of 32 MPa was requested. On the day of casting, concrete was properly mixed and shipped by a truck mixer (Figure B-5(a)). A slump test (Figure B-5(b)) was firstly carried out to check the workability of concrete. After achieving a satisfactory slump result, concrete was pumped through conduits, and poured into the slab formwork. A vibrator spear was used to compact concrete ensuring a uniform consistency (Figure B-5(c)). Finally, the slab surface was finished smoothly and uniformly by trowels. About one hour after the casting, a chemical water-based concrete curing membrane was sprayed onto the slab and wetted hessian bags were placed on the top surface of the slab to keep it moist. Additionally, during the slab casting, standard size (i.e. 100-mm diameter and 200-mm height) cylinders were cast for material testing. These cylinder samples were put on the slab afterwards ensuring the same curing condition with the slab. The strength of the cylinder samples was tested at 28 days and on the day of formal slab testing to reflect the slab concrete strength. Concrete sample cylinder tests were performed in accordance with AS1012.1 (2014).

![Figure B-5 Concrete casting: (a) truck mixer loaded with concrete; (b) slump test; (c) concrete pouring](image)

In reference to Figure B-6, a temporary steel column was used to replace the column(s) to be removed in order to prevent undesired damages before testing. The slab formwork was taken off two weeks after concrete casting when 70% of the target concrete strength was roughly achieved. Then, the slab were painted in white and drawn with a grid of 500
mm × 500 mm on the top and bottom surfaces to clearly observe cracks during testing. The cables of strain gauges were extended to a connection terminal and neatly managed on the slab.

Figure B-6 Concrete surface finishing
INSTRUMENTATION AND RECORDING SYSTEM

Given that no such large-scale RC structural tests were previously performed in the laboratory, a recording system used to monitor the overall structural behaviour of the slab was developed. A large number of apparatuses were mounted to the test specimen internally and externally. In this recording system, strain and displacement results at key locations and reaction force and moment results at each column were measured, and cracks on the top and bottom surfaces of the slab were photographically recorded. Across the whole course of testing, the experimental data were collected by the National Instruments (NI) data logger, and written automatically and synchronously per second, eventually pooling into a single file. To this end, a graphical program was developed in LabVIEW software, which processed all the recorded raw data via internal computing into practicable experimental results (i.e. strains, displacements and forces).

The working flow of the recording system is shown in Figure C-1. The printed circuit board (PCB) box and linear variable displacement transducer (LVDT) box were self-manufactured, for simplicity, to be able to directly connect strain gauges and LVDTs, respectively, with single plug-in. The power supply was connected to all apparatus. Any measured variations from apparatus would flow into data logger in the form of voltages and then were written in LabVIEW software.
C.1 Strain Measurements

The principle of strain measurements is based on the Wheatstone bridge circuit which is typically illustrated in Figure C-2. The circuit is composed of four resistances, one input voltage and one output voltage. One of the four resistances will be replaced by a strain...
gauge with the same resistance value as the other three. In this way, any recorded strain variations of this strain gauge can be reflected by its resistance changes, which can be further indicated by the measured output voltage readings. The PCB box as introduced previously was designed based on such a theory.

![Wheatstone bridge circuit](image)

Figure C-2 Wheatstone bridge circuit (Note: $R_1$-$R_4$ are four resistances; $U_A$ and $U_E$ are output voltage and excitation voltage.)

In each experimental test, a total of 44 strain gauges with a length of 3 mm (see Figure C-3(a)) and 15 strain gauges with a length of 90 mm (see Figure C-3(b)) were glued to reinforcing bars and the top and bottom surfaces of the slab, respectively. Note that the initial readings of strain gauges were set to zero before formal testing, the measured strains denote the generated deformations at mounted locations thoroughly due to loading.

![Strain gauges](image)

Figure C-3 Strain gauges: (a) 3 mm for reinforcing bars; (b) 90 mm for concrete
C.2 Force and Moment Measurements

Similar to the application of Wheatstone bridge circuit to the design of the PCB box, the four resistances in the circuit (Figure C-2) were all replaced by strain gauges when designing the load cells. This bridge configuration is known as a full bridge. With more strain gauges being applied to the circuit, high accuracy and sensitivities and compensation of temperature effect can be achieved, which is important to load cells. Each load cell consisted of four independent steel circular hollow sections (CHSs) bolted at each corner of a 20-mm thick steel plate (see Figure C-4(a)). Each CHS were glued with four strain gauges forming a full bridge (see Figure C-4(b)). Before mounting the custom-built load cell to columns, calibrations on the axial force and moment were implemented, as demonstrated in Figure C-5 where the load cell was loaded by a servo-controlled test system (MTS) actuator with a lever arm. The accuracy through comparing the readings of the load cell and MTS actuator revealed to be within a range of 97-100%. Then, each column was bolted to one load cell, itself was bolted to the strong floor. A supplementary description regarding the load cell design can be found in Xue et al. (2018).

Figure C-4 Load cell: (a) mounted load cell; (b) CSHs
C.3 Displacement Measurements

Different types of linear variable transducers were used to measure the slab displacements, and slab rotations relative to the columns. The slab vertical displacement was measured by wire LVDTs (Figure C-6(a)) with a long travel distance of 1,500 mm but a relatively low accuracy. The slab horizontal displacement was expected to be in a smaller range than the slab vertical displacement, therefore measured by laser LVDTs (Figure C-6(b)) and rod (Figure C-6(c)) LVDTs with shorter travel distances of 70 mm and 150 mm, respectively, but higher accuracies. Figure C-6(d) shows a rotation linear variable transducer ranging from negative 90 to positive 90 degrees, which was used to record the slab rotation relative to the column.
C.4 Crack Monitoring

During the experimental testing, crack development was monitored at key stages by taking photos. Due to safety concerns, as there was a potential of a collapse of the loaded slab, necessary secure procedures were followed. On the top surface of the slab, crack photos were taken by an operator walking on the slab while being attached to a safety harness. To observe cracks on the bottom surface of the slab, camera tracks were designed as shown in Figure C-7, where steel tendons were fixed at two ends and stretched in tension, constituting camera tracks parallel to the slab edge. A GoPro camera was mounted on the track, and moved by attaching a string on it and pulling the string at two ends. By this means, crack photos on the bottom surface were able to be taken remotely.
without people going underneath. As seen in Figure C-7, using appropriate lighting next to the camera, even hairline cracks were able to be captured. It is noted that each crack photo on surfaces of the slab normally covered an effective area of 500 mm × 500 mm the same as the size of the grid (see Figure B-6). These crack photos were trimmed and put together afterwards to form complete crack patterns of the whole slab.

Figure C-7 Camera track
The bond-slip relationship between concrete and reinforcement, used for numerical analyses in this study, is explained in Section 5.2.2. Relevant coding for numerical models of T1/T2 and S-E/S-EI are provided hereafter.

### D.1 Coding for T1/T2 Models

```
*CONSTRAINED_BEAM_IN_SOLID_ID
  coupid
  title
    1
  slave   master  sstyp  mstyp
  ncoup   cdir
    2   1   0   0   0   0   0
  2   0
  start   end   axfor
  0.0   5.0   0   0
*CONSTRAINED_BEAM_IN_SOLID_ID
  coupid
  title
    2
  slave   master  sstyp  mstyp
  ncoup   cdir
    201  101   1   1   0   0   0
  2   1
  start   end   axfor
```

$$\begin{align*}
0.0 & \quad 5.0 & \quad 0 & \quad -10 \\
*\text{CONSTRAINED\_BEAM\_IN\_SOLID\_ID} \\
& \# \text{ coupid} \\
& \text{title} \\
& \quad 3 \\
& \# \text{ slave} \quad \text{master} \quad \text{sstyp} \quad \text{mstyp} \\
& \quad \text{ncoup} \quad \text{cdir} \\
& \quad 301 \quad 101 \quad 1 \quad 1 \quad 0 \quad 0 \\
& \quad 2 \quad 1 \\
& \# \text{ start} \quad \text{end} \quad \text{axfor} \\
& \quad 0.0 \quad 5.0 \quad 0 \quad -11 \\
\end{align*}$$

*DEFINE\_FUNCTION

10

float force(float slip, float leng)
{
    float force, pi, d, area, shear, pf;
    pi = 3.1415926;
    d = 8;
    area = pi*d*leng;
    pf = 1.0;
    if (slip < 0.1) {
        shear = 15.188*slip+1.1373;
    } elseif (slip < 0.3) {
        shear = 6.8019*slip+2.0294;
    } elseif (slip < 0.5) {
        shear = 4.4759*slip+2.7159;
    } elseif (slip < 1.0) {
        shear = 3.1063*slip+3.4526;
    } elseif (slip < 1.8) {
        shear = 2.1379*slip+4.4379;
    } elseif (slip < 3.6) {
        shear = 8.2443*pf;
    } elseif (slip < 4.0) {
        shear = -12.3665*slip+52.7635;
    } else {
        shear = 3.298*pf;
    }
    force = shear*area;
    return force;
}

*DEFINE\_FUNCTION
float force1(float slip1, float leng1)
{
    float force1, pi1, d1, area1, shear1, pf1;
    pi1 = 3.1415926;
    d1 = 4.75;
    area1 = pi1*d1*leng1;
    pf1 = 1.0;
    if (slip1 < 0.1) {
        shear1 = 15.188*slip1 + 1.1373;
    } elseif (slip1 < 0.3) {
        shear1 = 6.8019*slip1 + 2.0294;
    } elseif (slip1 < 0.5) {
        shear1 = 4.4759*slip1 + 2.7159;
    } elseif (slip1 < 1.0) {
        shear1 = 4.4759*slip1 + 3.4526;
    } elseif (slip1 < 1.8) {
        shear1 = 4.4759*slip1 + 3.4526;
    } elseif (slip1 < 3.6) {
        shear1 = 8.2443*pf1;
    } elseif (slip1 < 4.0) {
        shear1 = -12.3665*slip1 + 52.7635;
    } else {
        shear1 = 3.298*pf1;
    }
    force1 = shear1*area1;
    return force1;
}

D.2 Coding for S-E/S-EI Models

*CONSTRAINED_BEAM_IN_SOLID_ID
#$  coupid
title
    1
#$  slave master sstyp mstyp
ncoup cdir
APPENDIX D

2 1 0 0 0 0
2 0
$# start end axfor
 0.0 5.0 0 0
*CONSTRAINED_BEAM_IN_SOLID_ID
$# coupid
title
 2
$# slave master sstyp mstyp ncoup cdir
 201 1 1 0 0 0 0
2 1
$# start end axfor
 0.0 5.0 0 -10
*CONSTRAINED_BEAM_IN_SOLID_ID
$# coupid
title
 3
$# slave master sstyp mstyp ncoup cdir
 301 1 1 0 0 0 0
2 1
$# start end axfor
 0.0 5.0 0 -11
*DEFINE_FUNCTION
 10
float force(float slip,float leng)
{
  float force,pi,d,area,shear,pf;
  pi = 3.1415926;
  d = 8;
  area = pi*d*leng;
  pf = 1.0;
  if (slip < 0.1) {
    shear = 13.585*slip*pf+1.0173;
  } elseif (slip < 0.3) {
    shear = 6.0841*slip*pf+1.8152;
  } elseif (slip < 0.5) {
    shear = 4.0036*slip*pf+2.4293;
  } elseif (slip < 1.0) {
    shear = 2.7785*slip*pf+3.0883;
  }
} elseif (slip < 1.8) {
  sheaf = 1.9123*slip*pf1+3.9696;
} elseif (slip < 3.6) {
  shear = 7.374*pf1;
} elseif (slip < 4.0) {
  shear = -11.061*slip*pf1+47.194;
} else {
  shear = 2.9496*pf1;
}
force1 = shear1*area1;
return force1;

*DEFINE_FUNCTION

11
float force1(float slip1, float leng1){
  float force1, pi1, d1, area1, shear1, pf1;
p1 = 3.1415926;
d1 = 4.75;
area1 = pi1*d1*leng1;
pf1 = 1.0;
if (slip1 < 0.1) {
  shear1 = 13.585*slip1*pf1+1.0173;
} elseif (slip1 < 0.3) {
  shear1 = 6.0841*slip1*pf1+1.8152;
} elseif (slip1 < 0.5) {
  shear1 = 4.0036*slip1*pf1+2.4293;
} elseif (slip1 < 1.0) {
  shear1 = 2.7785*slip1*pf1+3.0883;
} elseif (slip1 < 1.8) {
  shear1 = 1.9123*slip1*pf1+3.9696;
} elseif (slip1 < 3.6) {
  shear1 = 7.374*pf1;
} elseif (slip1 < 4.0) {
  shear1 = -11.061*slip1*pf1+47.194;
} else {
  shear1 = 2.9496*pf1;
}
force1 = shear1*area1;
return force1;