BEHAVIOUR OF SHEARHEAD SYSTEM BETWEEN FLAT REINFORCED CONCRETE SLAB AND STEEL TUBULAR COLUMN

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CONTENTS

LIST OF TABLES .................................................................................. 8
LIST OF FIGURES .............................................................................. 10
NOTATIONS ....................................................................................... 17
ABSTRACT ......................................................................................... 19
ACKNOWLEDGEMENTS ..................................................................... 21
DECLARATION AND COPYRIGHT...................................................... 22

CHAPTER 1  INTRODUCTION ............................................................. 23
1.1 BACKGROUND ............................................................................... 23
1.2 OBJECTIVES AND ORIGINALITY ............................................. 27
1.3 THESIS OUTLINE ....................................................................... 28

CHAPTER 2  LITERATURAL REVIEW ................................................. 30
2.1 GENERAL INTRODUCTION .......................................................... 30
2.2 METHODS OF ENHANCING PUNCHING SHEAR RESISTANCE ...... 32
  2.2.1 Flexural reinforcement ................................................................. 32
  2.2.2. Shear reinforcement ................................................................. 35
2.3 PUNCHING SHEAR BEHAVIOUR OF CONCRETE FLAT SLABS ...... 46
2.4 NUMERICAL SIMULATIONS OF PUNCHING SHEAR BEHAVIOUR . 49
2.5 DESIGN METHODS FOR PUNCHING SHEAR RESISTANCE ............. 51
  2.5.1 Effective depth (d) ................................................................. 51
CHAPTER 4 NUMERICAL MODELLING METHODOLOGY ..........102

4.1 SIMULATION METHODOLOGY ........................................102
  4.1.1 ABAQUS/CAE .......................................................102
  4.1.2 Finite element type ..............................................103
    4.1.2.1 Solid elements ..............................................103
    4.1.2.2 Shell elements ..............................................104
  4.1.3 Interactions ......................................................105
    4.1.3.1 Tie constraints ..............................................105
    4.1.3.2 Embedded elements ........................................106
    4.1.3.3 Coupling .....................................................107
  4.1.4 Concrete material model ......................................108

4.2 ELASTIC SLAB BEHAVIOUR BY TIMOSHENKO ...............111
  4.2.1 Timoshenko analytical solution ..............................111
  4.2.2 Comparison with numerical results ...........................113

4.3 NUMERICAL SIMULATION OF MARZOUK’S TESTS ..........114
  4.3.1 Introduction to the experiments ............................114
  4.3.2 Numerical models .............................................115
  4.3.3 Numerical results .............................................117

4.5 CONCLUSIONS ......................................................120

CHAPTER 5 VALIDATION OF ABAQUS SIMULATION:

COMPARISON AGAINST THE AUTHOR’S OWN TESTS .... 121

5.1 MODELLING TEST 1 ..................................................121
  5.1.1 Geometric arrangement .......................................121
5.1.2 Boundary conditions and interactions ..............................................123
5.1.3 Material properties ........................................................................... 125
5.1.4 Comparison between numerical and test results ................................. 128
  5.1.4.1 Load-deformation and maximum load carrying capacity ............... 128
  5.1.4.2 Strain comparison .......................................................................... 129
  5.1.4.3 Crack pattern ................................................................................ 133

5.2 MODELING TEST 2 ..................................................................................135
  5.2.1 Full model for the second test ............................................................ 135
  5.2.2 Combined models for the second test ............................................... 137
  5.2.3 Comparison between numerical and test results ................................ 138
    5.2.3.1 Load-deformation relationship ...................................................... 138
    5.2.3.2 Comparison of strains ................................................................. 140
    5.2.3.3 Comparison of crack development ............................................. 141

5.3 SENSITIVITY STUDY ............................................................................. 143
  5.3.1 Material properties .......................................................................... 144
    5.3.1.1 Young’s modules ........................................................................ 144
    5.3.1.2 Fracture energy-displacement relationship .................................. 144
    5.3.1.3 Failure tensile stress ................................................................. 145
    5.3.1.4 Dilation angle .............................................................................. 146
    5.3.2 Mesh sensitivity .............................................................................. 147
      5.3.2.1 Element size ............................................................................. 147
      5.3.2.2 Element type ............................................................................ 148

5.4 CONCLUSIONS ....................................................................................... 149
CHAPTER 6  PARAMETRIC STUDY: EFFECTS OF DIFFERENT DESIGN PARAMETERS .................................................................................................................. 151

6.1 INTRODUCTION TO LOAD CARRYING MECHANISM ................. 153

6.2 EFFECTS OF CHANGING COLUMN DIMENSIONS ....................... 154

6.2.1 Effects of column shape: square or circular ..................................154

6.2.2 Effects of varying column cross-sectional dimensions ..................156

6.3 EFFECTS OF SHEARHEAD ARM DESIGN .................................... 158

6.3.1 Effects of shearhead arm length ..................................................159

6.3.2 Effects of shearhead arm cross-section .......................................163

6.3.3 Effects of shearhead arm end angle .............................................165

6.3.4 Effects of shearhead continuity ...................................................168

6.3.5 Section summary .........................................................................172

6.4 EFFECTS OF SLAB THICKNESS ................................................. 173

6.4.1 Effects of slab thickness .............................................................173

6.4.2 Position of reinforcement ..........................................................176

6.4.3 Effects of shearhead arm position ...............................................177

6.5 EFFECTS OF TENSILE REINFORCEMENT ................................. 180

6.6 EFFECTS OF SERVICE HOLE POSITION .................................... 181

6.7 CONCLUSIONS .............................................................................183

CHAPTER 7 DEVELOPMENT OF A DESIGN METHOD ...... 185

7.1 ENLARGED COLUMN ASSUMPTION ........................................ 185

7.2 DESIGN CHECKS FOR SHEARHEAD ARM ............................... 191

7.2.1 Loading condition under shearhead arm ....................................191

7.2.2 Comparison for different design conceptions ..............................194
LIST OF TABLES

Table 3-1 Measured material properties .......................................................... 73
Table 4-1 Timoshenko method compared with ABAQUS analysis ....................... 113
Table 4-2 Full details for the Marzouk’s test slabs ........................................... 115
Table 5-1 Summary of material property definitions ........................................... 128
Table 6-1 Effects of column size on slab punching shear resistance .................... 156
Table 6-2 Effects of shearhead arm length ....................................................... 159
Table 6-3 Effects of shearhead arm cross-section ............................................. 163
Table 6-4 Effects of shearhead arm end angle ................................................ 166
Table 6-5 Effects of shearhead continuity ....................................................... 169
Table 6-6 Effects of slab thickness ................................................................. 174
Table 6-7 Effects of reinforcement position ..................................................... 177
Table 6-8 Effects of shearhead arm position .................................................. 178
Table 6-9 Effects of tensile reinforcement ratio ............................................... 180
Table 7-1 Comparisons between test and design code calculations based on enlarged column for the author’s tests .......................................................... 189
Table 7-2 Comparisons between design and numerical result for effects of column size .......................................................... 196
Table 7-3 Comparisons between design and numerical result for effects of sheararm section study .......................................................... 197
Table 7-4 Comparisons between design and numerical result for reinforcement ratio study .......................................................... 198
Table 7-5 Comparisons between design and numerical result for effective depth study ................................................................. 199
Table 7-6 Comparisons between design and numerical result for discontinued arm study ................................................................. 200
Table 7-7 Comparisons between design and numerical result for hole position study ................................................................. 201
LIST OF FIGURES

Figure 1.1 Comparison between brittle punching failure and ductile flexural failure ................................................................. 24
Figure 1.2 Typical types of shear reinforcement ................................................... 26
Figure 2.1 Punching failure mode in flat slab ..................................................... 30
Figure 2.2 Collapse of 2000 Commonwealth Avenue, Boston, US, 1971 ............. 31
Figure 2.3 Geometry of tested specimen in Guandalini's test ............................ 33
Figure 2.4 Critical shear crack pattern for the test specimens ......................... 33
Figure 2.5 Normalized load-deflection curves for all specimens .................... 34
Figure 2.6 Load-rotation comparison between the tests of Kinnunen with ACI 318 ................................................................. 35
Figure 2.7 Three difference failure modes ...................................................... 36
Figure 2.8 Shearband details ........................................................................ 39
Figure 2.9 Components of the NUUL system ................................................. 40
Figure 2.10 Typical failure mode of flat slab with NUUL system ....................... 41
Figure 2.11 Shearhead reinforcement developed by Corley and Hawkins ......... 42
Figure 2.12 Typical failure modes of flat slab without or with shearhead reinforcement ................................................................. 43
Figure 2.13 Typical arrangement for flat slab with CFT column ................. 45
Figure 2.14 Load-displacement curves from Lee, Kim and Song's test .......... 46
Figure 2.15 Contributions to punching shear capacity .................................... 48
Figure 2.16 Crack model of Hillerborg .......................................................... 50
Figure 2.17 Stress-crack width model proposed by Hillerborg ............... 50
Figure 2.18 Effective depth in normal flat slabs ...........................................51
Figure 2.19 EC2 definition of critical perimeters for slab without shear
reinforcement .................................................................................................52
Figure 2.20 EC2 critical perimeter for slab with an opening .........................52
Figure 2.21 BS 8110 definition of critical perimeter ....................................53
Figure 2.22 ACI 318 definition of critical perimeter ....................................53
Figure 2.23 ACI 318 treatment of opening ..................................................54
Figure 2.24 EC2 definitions of critical perimeter for slab with shear
reinforcement .................................................................................................54
Figure 2.25 BS 8110 guide on shear reinforcement ....................................56
Figure 2.26 Simplified critical perimeter for slab with shear reinforcement according
to BS8110 ..................................................................................................56
Figure 2.27 ACI 318 definition of critical perimeter for flat slab with shearhead ......57
Figure 3.1 Test rig arrangement .....................................................................63
Figure 3.2 Bolted slab in the test .................................................................64
Figure 3.3 Test specimen and reinforcement arrangement ..........................66
Figure 3.4 Discontinued reinforcement adopted for the first test .................67
Figure 3.5 Shearhead details for test 1 .........................................................68
Figure 3.6 Specimen arrangements for the second test ...............................70
Figure 3.7 Shearhead for test 2 ....................................................................72
Figure 3.8 LVDT’s positions for test 1 ..........................................................75
Figure 3.9 Strain gauge positions on the shearhead arm and slab for test 1 ....76
Figure 3.10 Tension face of the slab before the first test ..............................77
Figure 3.11 LVDT positions for test 2 ...........................................................78
Figure 3.12 Strain gauge positions for the second test ..................................79
Figure 3.13 Tension face of the second slab before test …………………………….. 80
Figure 3.14 Tension side of the first test slab before collapse …………………….. 81
Figure 3.15 Appearance of the first test slab face after failure …………………….. 82
Figure 3.16 Observed crack pattern at the reinforcement level of the first test……….. 83
Figure 3.17 Measured raw load-deflection curve for the column centre (see Figure 3.8 for LVDT locations) …………………………………………………………………………………….. 84
Figure 3.18 Measured strain results for the slab in test 1 …………………………. 85
Figure 3.19 Measured strain – load relationships for the shearhead arm in test 1 ... 86
Figure 3.20 Tension side of the second test slab just before collapse ………………. 87
Figure 3.21 Failure pattern of the second test ………………………………………. 89
Figure 3.22 Crack around the hole ………………………………………………… 90
Figure 3.23 Measured raw Load- deformation curves for the second test (see Figure 3.11 for LVDT locations) …………………………………………………………………………………….. 91
Figure 3.24 Measured slab strain – load relationships for test 2 …………………... 92
Figure 3.25 Measured strain-load relationships for the shearhead arm in test 2 …… 93
Figure 3.26 Through thickness cut patterns for the second test specimen ………… 94
Figures 3.27 Cut face A ……………………………………………………………………… 95
Figure 3.28 Cut face E ……………………………………………………………………… 96
Figure 3.29 Cut faces F and G ……………………………………………………………. 97
Figure 3.30 Crack details around the hole …………………………………………… 98
Figure 3.31 Overall crack pattern at the reinforcement level of the second test ……… 99
Figure 3.32 Sketches of shearhead arrangement through slab thickness …………. 100
Figure 4.1 Different types of solid elements ……………………………………….. 104
Figure 4.2 Differences between conventional and continuum shell elements ……… 104
Figure 4.3 Tie condition in the model ……………………………………………….. 106
Figure 4.4 Application of coupling procedure ........................................ 108
Figure 4.5 Hillerborg failure model for fracture energy-displacement model ......110
Figure 4.6 Load case for a simply supported slab ........................................ 111
Figure 4.7 ABAQUS model for a simply supported slab ................................ 113
Figure 4.8 Typical test specimen dimensions .............................................. 114
Figure 4.9 Reinforcement stress-strain curve adopted in the numerical analysis....116
Figure 4.10 Typical ABAQUS model for Marzouk’s test .............................. 116
Figure 4.11 Comparison of load-deformation curves for test NS1 ................. 118
Figure 4.12 Load-deformation curve comparison for test HS3 ...................... 118
Figure 4.13 Load-deformation curve comparison for test HS4 ...................... 118
Figure 4.14 Comparison of strains for test HS3 ........................................ 119
Figure 5.1 ABAQUS model for Test 1 ..................................................... 122
Figure 5.2 Deformed model for the supporting beams ............................... 124
Figure 5.3 Stress-strain relationships for steel reinforcement .......................... 125
Figure 5.4 Shearhead arm stress-strain relationship .................................... 126
Figure 5.5 Concrete stress-strain curve .................................................... 127
Figure 5.6 Comparison between numerical and test load-deflection curves for Test 1 ........................................................................................................ 129
Figure 5.7 Comparison between numerical and measured strains: concrete in comparison for position 222mm away from the slab centre as C57 presented in Figure 3.9 ........................................................................................................ 130
Figure 5.8 Comparison between numerical and measured strain: tensile reinforcement for position 222mm away from the slab centre as C49 presented in Figure 3.9 ........................................................................................................ 131
Figure 5.9 Comparison of strain on the flange of the shearhead arm: 100mm away from the outer side of the tubular column as C23 presented in Figure 3.9

Figure 5.10 Crack developing pattern from numerical model for the test 1

Figure 5.11 Comparison of critical perimeters for the first test

Figure 5.12 Full slab model for Test 2

Figure 5.13 Boundary condition for the second test

Figure 5.14 Quarter slab model without hole

Figure 5.15 Quarter slab model with hole

Figure 5.16 Comparison of numerical and test load-deformation curves for Test 2

Figure 5.17 Comparison of concrete compressive strain where is 530mm away from the centre of the slab as C29 shown in Figure 3.12

Figure 5.18 Comparison of strains in reinforcement bar where is 530mm away from the centre of the slab as C31 shown in Figure 3.12

Figure 5.19 Concrete crack developments

Figure 5.20 Comparison of critical perimeters second test

Figure 5.21 Comparison of load-deflection curves for different Young’s modulus

Figure 5.22 Effects of different fracture energy on load-deflection curves

Figure 5.23 Effects of maximum concrete tensile stress on load-deflection curve

Figure 5.24 Effects of dilation angle on load-deformation behaviour

Figure 5.25 Effects of element size on load-deformation curves

Figure 5.26 Comparison of load-deformation behaviour using shell and solid elements

Figure 6.1 Shearhead system under consideration
Figure 6.2 Model details for the reference case ........................................152
Figure 6.3 Enlarged column area .........................................................154
Figure 6.4 Effects of different column shapes on slab load-deformation curve....155
Figure 6.5 Comparison of effects of column size on slab load-deformation relationships ................................................................. 157
Figure 6.6 Effects of shearhead arm length on slab load-deformation relationship ................................................................. 159
Figure 6.7 Effects of shearhead length on failure mode .........................161
Figure 6.8 Pressure distribution under shearhead ..................................162
Figure 6.9 End angle of sheararm........................................................165
Figure 6.10 Effects of shearhead arm end angle on slab load-deformation relationship ................................................................. 167
Figure 6.11 Comparison between bending moment capacity and applied bending moment for different shearhead arm end angles .........................167
Figure 6.12 Continuous and Discontinuous shearheads .........................169
Figure 6.13 Short discontinuous shearhead arm: showing column punching through the slab with little contribution from the shearhead .........................170
Figure 6.14 Different reinforcement arrangement for discontinued shearhead system ................................................................. 171
Figure 6.15 Effects of reinforcement arrangement on slab load-deformation relationships ................................................................. 172
Figure 6.16 Failure modes for different slab thicknesses and shearhead arm sizes ................................................................. 176
Figure 6.17 Effects of shearhead arm position on slab load-deformation relationships ................................................................. 178
Figure 6.18 Position of diagonal shear crack .............................................. 179
Figure 6.19 Single service hole position ..................................................... 181
Figure 6.20 Effects of service hole position on slab load-deformation
relationships .................................................................................................. 182
Figure 6.21 Critical perimeter for different hole positions in the slab .......... 182
Figure 7.1 Determination of critical perimeter .............................................. 186
Figure 7.2 Effective depth of the flat slab with shearhead system ............... 187
Figure 7.3 Comparison for critical perimeters between measurement and design code calculations ................................................................. 188
Figure 7.4 Comparison of critical perimeter for Test 1 and Test 2............... 190
Figure 7.5 Assumed load distribution under shearhead arm ................. 191
Figure 7.6 Pressure distribution under shearhead arm according to ACI 318 .... 193
Figure 7.7 Comparison for the FE results and design calculations ............... 195
**NOTATION**

\( A_s \): The reinforcement area in the cross section

\( b_0 \): The perimeter of critical section (in)

\( b_v \): The breadth of the section;

\( c_i \): The size of the column measured in the direction of the span for which moments are being determined

\( d \): Effective depth of the flat slabs

\( E \): Modulus of elasticity of the material

\( f_{ck} \): Cylinder strength of the concrete

\( f_{cm} \): Mean compressive strength of the concrete

\( f_{cu} \): Cube strength of the concrete

\( G_f \): Fracture energy of concrete

\( G_{f0} \): Base value of fracture energy

\( h \): The depth of the slab

\( h_v \): The total depth of shearhead cross the section

\( I \): Distance from the slab centre to the hole centre

\( l_v \): The length of the sheararm section

\( M_p \): The plastic moment resistant capacity for the sheararm section just beside the column edges

\( M_1 \): The bending moment carried by the sheararm section

\( V_u \): The factored shear force at section

\( V_c \): The nominal shear strength provided by concrete

\( V_s \): The nominal shear strength provided by shear reinforcement
\( v_{\text{Rd,c}} \): Design punching shear resistance defined by EC2

\( u \): Critical perimeter of the flat slabs (mm)

\( \alpha \): The end angle of the sheararm

\( \alpha_r \): The ratio between the flexural stiffness of each shearhead arm and that the surrounding composite cracked slab section of width \( (c_2 + d) \) which should not less than 0.15

\( \gamma_m \): The partial safety factor for strength of materials defined in BS8110

\( \eta \): The number of the arms

\( \nu \): The Poisson’s ratio

\( \rho \): Reinforcement ratio in the slab

\( \phi \): The strength reduction factor for the tension-controlled members, 0.85 has been taken in this study
ABSTRACT

This thesis presents the results of an experimental, numerical and analytical study to develop a design method to calculate punching shear resistance for a new shearhead system between tubular steel column and reinforced concrete flat slab. This shearhead system enables two of the most popular structural systems, i.e. reinforced concrete flat slab floor and steel tubular column, to be used to produce efficient structures of low cost and short construction time. This research investigates slabs without and with a service hole adjacent to the column. The new shearhead system should not only possess sufficient punching shear resistance, but should also be efficient for construction.

The main methodology for this project was based on numerical finite element simulations verified by two full scale tests. These two tests were carried out in the University of Manchester’s Structural Testing Laboratory. The two specimens had the same slab size, thickness and reinforcement ratio, but differed in the column shape (rectangular or circular), central reinforcement arrangement (continuous or discontinuous), shearhead position in the slab thickness and shearhead fabrication arrangement. Recorded load-deflection and load-strain relationships, crack development and critical perimeter were used for detailed validation of using the commercial finite element software ABAQUS. The validated ABAQUS model was used to conduct a comprehensive parametric study to investigate the effects of a number of design parameters, including the effect of varied column size, shearhead arm length, shearhead arm cross section, shearhead arm angle, amount of flexural reinforcement, slab thickness, shearhead positions and hole positions.

The main conclusion from the parametric study was that the shearhead system could be treated as an enlarged column in normal flat slab structure. The parametric study enabled pressure distribution below the shearhead arms to be approximated for checking whether the shearhead arms would be sufficient for the enlarged column assumption to be valid. The parametric study results were also used to determine the
effective depth of the flat slab and critical punching shear perimeter of the slab with and without a service hole.

Using the enlarged column assumption, the punching shear resistance of all structures used in the parametric study were re-calculated using Eurocode 2 (EC2), British stand 8110 (BS8110) and American Concrete Institute code 318 (ACI 318). Comparison of calculation results using these three design methods indicates that both EC2 and BS8110 predicted very close value which reached very good agreement with the ABAQUS simulation (normally within 10%). Among these three design methods, ACI 318 was the only code that explicitly considered shearhead system. ACI 318 was not able to predict the slab critical perimeter length with good accuracy, however, its prediction of slab punching shear resistance achieved reasonably good agreement with numerical analysis results and were on the safe side. Based on these studies, a design method for calculating punching resistance of the proposed shearhead system between reinforced concrete flat slab and steel tubular column has been developed in this thesis.
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CHAPTER 1: INTRODUCTION

1.1 BACKGROUND

Flat slab construction has many advantages, including aesthetically pleasing appearance, flexible accommodation of building services, and low construction cost. Consequently, this type of construction is widely adopted for both residential and office buildings.

Currently, flat slab construction uses reinforced concrete in both the slabs and columns. However, using steel column can improve structural performance and shorten the construction lead time. This research is about developing a method to enable steel tubes to be used to replace reinforced concrete columns in flat-slab construction.

For the proposed new construction, as in the traditional reinforced concrete flat slab construction, punching shear is a critical design case. Because punching shear failure of flat slab supported by column is brittle without any warning sign, the consequence can be severe. For example, collapse of the north wing of the Sampoong department store in 1995 in Seoul, Korea, resulted in nearly 500 people being killed (Gardner et al., 2002). The difference between brittle punching failure and the more ductile mode of flexural failure can be easily shown by the load-deflection curves in Figure 1.1. After punching shear failure, the load carrying capacity of the structure drops instantly to a small fraction of its peak capacity, allowing people no time to escape. In contrast,
under flexural failure mode, the structure would experience gradual decline in load carrying capacity, giving people sufficient time to escape.

Figure 1.1: Comparison between brittle punching failure and ductile flexural failure (Menetrey, 2002)

To enhance punching shear capacity of flat slab structure, various construction methods have been developed by previous researchers, including:

1. Increasing the effective depth of the slab (M.P. Nielsen, 1998)
2. increasing the thickness of the slab around the column head with a drop panel or an inverted cone; (The Concrete Society, 2007)
3. reducing the effective length of the slab;
4. increasing the column head dimension;
5. changing the flexural reinforcement ratio or concrete grade; (Marzouk et al., 1998)
6. decreasing the load;
7. adding shear reinforcement (Papanikolaou et al., 2005)

Adding shear reinforcement represents the best way to enhance punching shear capacity without affecting serviceability of the structure. Therefore, recent research studies have mainly focused on finding the most efficient type of shear reinforcement. In addition to normal reinforcement bars, such as bent-up bars and stirrups, other methods include steel plates (Subedi and Baglin, 2003), shearhead systems (Corley and Hawkins, 1968), carbon fiber reinforcement polymers (CFRPs) (Sharaf et al., 2006), Studs (Elgabry and Ghali, 1990) and steel fiber reinforcement polymers (SFRPs) (El-Ghandour et al., 2003). Figure 1.2 shows some examples of shear reinforcements.

(a) Steel reinforcement polymer (El-Ghandour et al., 2003)

(b) CFRPs (Sharaf et al., 2006)
All these researches presented above are predicated on the use of concrete columns but it is believed that the greatest advantages of all come from combining a shear system with a steel tubular column, filled with concrete or not. Tubular columns have many advantages compared with concrete columns and have become popular in certain kinds of market. They allow easy and quick assembly and recycling. Because the steel tubular column may be erected at the same time as the concrete slab formwork and the tubular column may be poured at the same time as the slab, this type of construction gives major improvements in cycle time.
1.2 OBJECTIVES AND ORIGINALITY

This project will investigate the structural behaviour of a newly developed shearhead system for tubular columns with reinforced concrete flat slab. This new shearhead system described herewith was developed to enable the tubular steel columns to be used with concrete flat slabs so that it is quick to erect and allows service holes to be made adjacent to columns. The shearhead is different from existing systems: it does not require any shear reinforcement bars, only the normal flexural reinforcement; it can reduce slab depth, if this is not determined by deflection in the centre of the slab. The main objectives of this study are:

- to design an effective shearhead system for using steel tubular columns (both rectangular hollow section and circular hollow section) with reinforced concrete flat slab;
- to carry out an experimental study to demonstrate the enhanced punching shear capacity of the new shearhead system;
- to validate the simulation model using the commercial finite element software ABAQUS by the physical experiment results;
- to validate the simulation model using the commercial finite element software ABAQUS by checking the simulation results against the experiment results;
- to use the validated ABAQUS models to carry out a parametric study to investigate the effects of different structural arrangements on the performance of the new shearhead system;

The knowledge gained from this study will provide a platform for further researches to develop better design method to counter punching failure in flat slab structures.
1.3 THESIS OUTLINE

This thesis is divided into 8 chapters and organized in the following manner:

Chapter 1 introduces the general background and objectives for this study and outlines the contents of this thesis.

Chapter 2 is a literature review of relevant studies on punching shear behaviour of slab-column connections and their current design methods. It will assess a number of theoretical models for punching shear behaviour. It will also provide a review of numerical studies by others to help develop an appropriate numerical simulation method for this study.

Chapter 3 describes the physical tests carried out by the author on the new shearhead system between tubular column and concrete flat slab, including test set-up, material properties, instrumentations and test results.

Chapter 4 and Chapter 5 are intended to validate the numerical procedure used in this research. Chapter 4 presents the general numerical simulation method and some validation results by comparing results from the Timoshenko plate theory and the tests by Marzouk and Hussein (Marzouk and Hussein, 1991).

Chapter 5 further establishes validity of the numerical model by comparing the numerical simulation results with results of the two shearhead tests carried out by the
author, described in Chapter 3. In addition, sensitivity studies were carried out to select appropriate material model, element type and mesh size for extensive parametric study reported in this chapter which mainly based on the test models.

Chapter 6 presents the results of a comprehensive parametric study to investigate the effects of different design parameters, including column size, shearhead arm length, shearhead arm cross-section, reinforcement ratio, sheararm end angle and position of holes. Wherever necessary, explanations will be provided to develop fundamental understanding of the investigated structural behaviour. The parametric study was based on the common properties of the two tests (slab dimensions, reinforcement, column overall size), but the other parameters were varied to cover a wide range of design values.

Based on the parametric study results, Chapter 7 develops a manual design method for calculating the punching shear capacity of the new proposed shearhead system.

Chapter 8 summarises the work carried out in this research and the main conclusions attained, and proposes a number of topics for further research and development.
CHAPTER 2: LITERATURAL REVIEW

The problem of punching shear failure in slab-column connections subjected to concentrated loads has received great attention over several decades because of its importance in flat plate floor structures. The punching shear resistance of such structures is generally lower than the flexural bending capacity and punching shear failure is also brittle.

In order to help develop understanding of punching shear behaviour for the proposed steel tube – reinforced concrete flat slab system, this chapter reviews relevant previous researches, including the physical experiments, theoretical studies and finite element analysis.

2.1 GENERAL INTRODUCTION

Figure 2.1 shows typical failure pattern of punching shear failure in flat slab structures.

Figure 2.1: Punching failure mode in flat slab
Punching failure is brittle failure its occurrence can easily lead to progressive failure of the structure because the failed slab can easily fall down on to the next floor, causing cascading effect. (P.E. Regan, 1981)

Figure 2.2: Collapse of 2000 Commonwealth Avenue, Boston, US, 1971 (King and Delatte, 2003)

For example, Figure 2.2 shows the collapsed building due to punching failure 2000 Commonwealth Avenue, Boston, US in 1971 (King and Delatte, 2003). A small punching failure around a column on the roof caused two thirds of this 16-story apartment building to collapse in a few minutes and four workers died. Recorded punching failure dates back as early as 1911 (J.Feld, 1978). Not surprising, numerous research studies have been carried out to understand this phenomenon and to develop methods to improve punching shear resistance.
2.2 METHODS OF ENHANCING PUNCHING SHEAR RESISTANCE

Many methods have been developed by various researchers to enhance punching shear resistance. This section will review these studies to understand the effects of different design parameters and help explain the selection of shearhead as the appropriate system for further development.

2.2.1 Flexural reinforcement

Although punching failure is caused by concrete tension crack opening, slab flexural reinforcement can contribute to punching shear resistance through dowel action (Muttoni, 2008, Guandalini et al., 2009, Marzouk and Hussein, 1991).

For example, Guandalini (Guandalini et al., 2009) recently carried out a series of tests on slabs with different amounts flexural reinforcement ratio. A total of eleven slabs for three slab thicknesses (125mm, 250mm and 500mm) were tested with flexural reinforcement ratio between 0.22% to 1.5%. Figure 2.3 shows the test arrangement and slab details. Reinforcement in compression face was kept around 0.2%. All the test specimens clearly failed in punching shear failure mode as presented in Figure 2.4.
Figure 2.3: Geometry of tested specimen in Guandalini's test (Guandalini et al., 2009)

Double-size specimen
(u) PG-3 ($\rho = 0.33\%$)

Full-size specimens
(b) PG-1 ($\rho = 1.5\%$) (c) PG-2b ($\rho = 0.25\%$)
(d) PG-4 ($\rho = 0.25\%$) (e) PG-5 ($\rho = 0.33\%$)
(f) PG-10 ($\rho = 0.33\%$) (g) PG-11 ($\rho = 0.75\%$)

Half-size specimens
(h) PG-6 ($\rho = 1.5\%$) (i) PG-7 ($\rho = 0.75\%$) (j) PG-8 ($\rho = 0.28\%$) (k) PG-9 ($\rho = 0.22\%$)

Figure 2.4: Critical shear crack pattern for the test specimens (Guandalini et al., 2009)
Figure 2.5 Normalized load-deflection curves for all specimens (Guandalini et al., 2009)

Figure 2.5 compares the normalized load-deflection curves. The slab shear strength calculated using the American Concrete Institute code 318 (ACI 318) is also shown in this Figure. Since ACI does not take into account contribution from flexural reinforcement, it gives constant strength irrespective of the reinforcement ratio. However, the test results indicate that flexural reinforcement ratio had significantly influence on slab punching resistance.

Figure 2.6 (Muttoni and Windisch, 2009) shows similar results by Kinnunen and Nylander. (S.Kinnunen and Nylander, 1960)
Clearly, increasing the amount of flexural reinforcement may be used to improve punching shear resistance, especially for slabs with low flexural reinforcement ratio. However, such a method alone is unlikely to be able to provide sufficient punching shear resistance.

2.2.2. Shear reinforcement

Adding shear reinforcements is a very popular and effective way of increasing the punching shear resistance of slab-column connections by including the vertical component of the tensile resistance of the shear reinforcement crossing the punching shear cracks. Studs, stirrups and bent-up bars are the most common types of shear reinforcement in current construction of flat slabs.
As illustrated in Figure 2.7 (Menetrey, 2002), three failure modes may be observed: punching shear crack located inside, outside or just cross the shear reinforcement.

In Figure 2.7 (1), the punching crack is located between the column face and the first row of shear reinforcement; computation of the corresponding punching failure load must consider the interaction between punching shear failure and flexural failure when determining the punching shear crack inclination.

In Figure 2.7 (2), the punching shear crack is initiated outside the last row of the shear reinforcement; the punching shear capacity is computed similarly to a normal reinforced concrete slab except that the radius of the column should be the radius of punching shear crack initiation.

In Fig2.7 (3), the punching shear crack crosses the shear reinforcement; the punching shear resistance should be composed of the capacity of the shear reinforcement cross section and concrete initial cracking area.
The punching shear resistance of the slab will be the minimum of the three failures. The contribution of the shear reinforcement to punching shear resistance is computed by summing up the contribution of each reinforcement crossing the punching shear crack.

The shear reinforcement may be divided into three groups:

(1) Shear reinforcement made with bars of high-bond strength such as stirrups and bent-up bars (Figure 1.2 (c));

(2) Shear reinforcement made with plain bars and anchorage such as studs (Elgabry and Ghali, 1990), commonly referred to as headed reinforcement (Figure 1.2 (d)) ;

(3) Structural steel sections across the column section, such as steel arms (Corley and Hawkins, 1968), Figure 2.11.

Adding bent-up bars or stirrups is a very traditional method of enhancing the punching shear resistance of slabs and attracted a lot of research interests in the last decades (Papanikolaou et al., 2005). However, they are either not very effective (i.e. the stirrups would not reach yield point when the slab has failed) or not suitable for thin structural elements (due to difficulty in anchorage at the end of the reinforcement). The stud system (Figure 1.2 (d)) is a more effective solution for improving punching shear resistance compared with bent-up bars or stirrups. The shear studs require a big head area which is normally greater than 10 times the cross-
section area of the stem so as to give superior anchorage according to a previous research by Elgabry and Ghali (1990).

Although the above shear reinforcement systems can make great contributions to the punching shear resistance of flat slabs in reinforced concrete construction and have been widely adopted, they have disadvantages such as not being easy to fabricate and high cost. More importantly, when the column is a steel tube, there is no positive connection between the steel tube and the concrete slab. Because of this, the load carrying mechanisms shown in Figure 2.7 would not develop. Instead, punching shear failure would happen at the interface between the slab and steel tubular column. Therefore, these methods will not be effective and will not be pursued any further when using steel tubes as columns.

Selecting an effective system to provide punching shear resistance in steel tube/flat slab construction has to search elsewhere among the various more elaborate systems that have been developed by others researchers.

Pilakoutas and Li (Pilakoutas and Li, 2003) developed the ‘shearband’ system which is a shear reinforcement system using steel strips with high ductility. This system (shown in Figure 2.8) is easy and quick assembly and the strips have a very small thickness and can be placed directly after the flexural reinforcement are in place. The holes on the steel strips maintain good anchorage for the strips over the embedded length. This system can also be used as an addition to other shear reinforcement systems.
However, this system would not be able to be connected to the steel tubular column and without an effective connection, it would not be possible to transfer the shear force from the steel tubular column to the shear reinforcement system. But this shearband system is a positive way to enhance the other shear reinforcement systems even within this new developed system with tubular column.

Figure 2.8 Shearband details (Pilakoutas and Li, 2003)

Adding a steel plate to the flat slab to increase the effective column head area is another way of increasing the punching resistance capacity of the slab-column connection. As example, a NUUL system was developed by Subedi and Baglin
(Subedi and Baglin, 2003) as shown in Figure 2.9. This NUUL system is composed of a steel plate and many U bars. The U bars are welded onto the steel plate to allow the flexural reinforcement to cross in order to provide enough connection between the system and flexural reinforcements. This system could provide very strong slab-column connections owing to the massive amount of steel around the column head area.

Figure 2.9 Components of the NUUL system (Subedi and Baglin, 2003)
Figure 2.10 shows typical punching shear failure mode when using this system: the whole NUUL system had to punch through the slab as a bigger column head, demonstrating the effectiveness of the system. Although this system can achieve very high punching shear resistances and it is possible to adopt this system to steel tubes, the massive amount of steel used and the complicated construction method would not be welcomed by contractors.

Figure 2.10 Typical failure mode of flat slab with NUUL system (Subedi and Baglin, 2003)

Figure 2.11 shows details of a shearhead system developed by Corley and Hawkins in 1968 (Corley and Hawkins, 1968). This system uses structural steel sections welded together to form a grid which can then be placed around or through a column. Their study formed the basis of the shearhead reinforcement design guidance in the American Code Institute design code ACI 318 (ACI, 2005).
a) typical details of the slab-column connection

b) typical shearhead type

Figure 2.11 Shearhead reinforcement developed by Corley and Hawkins (Corley and Hawkins, 1968)

A total of 21 specimens with the above shearhead system (or without any shearhead reinforcement) were tested by Corley and Hawkins (Corley and Hawkins, 1968) and three typical failure modes (no shearhead, over-reinforcing and under-reinforcing) were detected in their experimental study.

- The failure surface of the slab without a shearhead extended from the intersection of the column face and the compression face of the slab, towards the tension face of the slab with an inclined angle of about 20-30 degree to
the horizontal until it reached the tension reinforcement level. This failure mode is depicted in Figure 2.12 (a).

- The failure surface of the slabs containing very heavy shearheads developed from the outer perimeter of the shearhead system if the flexural capacity of the shearhead at the face of the column was not exceeded. The inclined angle of the failure surface varied from around 20 to 45 degree to the horizontal. This kind of failure was defined as over-reinforcing in the study Corley and Hawkins (1968), and Figure 2.12 (b) shows a typical such failure perimeter.

- For the specimens with light shearhead, the failure surface started from inside the shearhead system because the flexural capacity of the sheararm was exceeded. The inclination of the failure face was around 30 degree to the horizontal. This kind of failures was defined as under-reinforcing in their study, and Figure 2.12 (c) shows one of such failure perimeters.

(a) No shearhead                    (b) Over-reinforcing                  (c) Under-reinforcing

Figure 2.12 Typical failure modes of flat slab without or with shearhead reinforcement (Corley and Hawkins, 1968)
The test results in this study indicated that the slabs with under-reinforcing shearheads failed at a shear stress on a critical section at the end of the shearhead reinforcement less than 4, and the use of over-reinforcing shearheads brought the shear strength back to about 4. Based on this conclusion from Corley and Hawkins (1968), a conservative design method for this shearhead system in flat slabs was developed and adopted in ACI 318 (ACI, 2005).

The shearhead system developed by Corley and Hawkins (1968) was for reinforced concrete columns. However, it could be adapted for steel tubular columns by connecting the steel sheararms to the steel tubular columns. This will form the basis of the shearhead system used in this research.

Based on the above concept, Lee, Kim and Song (Lee et al., 2008) recently developed the shearhead system shown in Figure 2.13 for Concrete Filled Tubular (CFT) columns. In their system, the flexural reinforcement may be anchored in three different ways: bars crossing the whole column area (FP), bar crossing the column but stopping at the other inner side of the column (HP) or bars being hooked inside the column hole (HK). As illustrated in Figure 2.13, either a T (ST) or I (SH) section can be welded to the column to act as the shearhead key to provide a sufficient structural element for shear transfer. When using T-section as shearhead keys, a stud was welded to the outer face of the tubular column at the level of the tensile reinforcements in order to delay the separation between the concrete and steel column outer face.
Lee et al (2007) carried out 8 tests on different arrangements of the system. Although their experimental results confirmed that the proposed system achieved sufficient punching shear resistance compared to the benchmark (BM-RC) specimen which used normal reinforced concrete column (Figure 2.14), Very limited analysis has been done for the punching ability of this system in this study according to the complicated behaviour of the connection. Only an experiential coefficient (1.14) has been proposed with the ACI 318 design guidance for this shearhead system based on their limit test results.
This proposed system is really complicated for fabrication. This system will not be pursued any further in this research, but it is noted that it is possible to use shearheads to connect flat slabs with tubular steel columns.

**2.3 PUNCHING SHEAR BEHAVIOUR OF CONCRETE FLAT SLABS**

In order to obtain a clear understanding of the performance of slab-column connections under punching, many theoretical analyses models have been proposed by various investigators, mainly based on the theory of the plasticity. Kinnunen and Nylander's (1960) proposed a theory based on their circular slab tests with ring reinforcement. This very early model neither considered the dowel effects nor the
compressive strain of the concrete on the bottom surface of the two-way reinforced slab. Later Kinnunen (S. Kinnunen, 1963) examined such effects, but the failure criterion remained arguable as failure of the slab was supposed to occur when the compressive strain of the concrete in a tangential direction on the bottom surface of the slab under the root of a shear crack reached a value obtained from specific test results. Regan and Braestrup (Regan and Braestrup, 1985, Gomes and Regan, 1999) presented a comprehensive review of the field. Fracture mechanics was first attempted to analyse punching failure by de Borst and Nauta (de Borst and Nauta, 1985). More recent studies of punching shear behaviour include Shehata and Regan (Shehata and Regan, 1989) who included stress concentration near the column face and Menetrey (Menetrey et al., 1997) who took into consideration influence of tensile stress in the concrete slab along the inclined cracks. Menetrey (Menetrey, 2002) proposed to include the following four components when calculating punching shear resistance:

- Concrete tensile force contribution ($F_c$),
- Dowel-effect contribution from flexural reinforcement ($F_{dow}$),
- Shear reinforcement contribution ($F_{sw}$),
- Contribution of the pre-stressing tendon ($F_p$).

These four components are illustrated in Figure 2.15.
Yankelevsky and Leibowitz (Yankelevsky and Leibowitz, 1999) developed a model based on rigid post-fracture behaviour of slab. Theodorakopoulos and Swamy (Theodorakopoulos and Swamy, 2002) developed a general model for punching shear behaviour. They assumed that punching was a failure mode combining shearing and splitting, occurring without concrete crushing, but under complex three dimensional stresses. Failure occurred when the tensile splitting strength of the concrete was exceeded.

All these analytical models help to understand punching shear behaviour and also are useful in selecting methods to enhance punching shear resistance. However, punching shear behaviour is an extremely complicated and precise treatment is still not possible. Instead, the strategy of this research is to adapt existing calculation methods, based on experimental and numerical simulation results.
2.4 NUMERICAL SIMULATIONS OF PUNCHING SHEAR BEHAVIOUR

A number of researchers have applied finite element models to simulate punching shear behaviour, including Menetrey (Menetrey et al., 1997) who simulated the tests of Kinnunen and Nylander (S.Kinnunen and Nylander, 1960), Hueste and Wight (Hueste and Wight, 1999) who developed the program DRAIN-2D, Hallgren and Bjerke (Hallgren and Bjerke, 2002), Martina and Karsten (Schnellenbach-Held and Pfeffer, 2002) using the general finite element package DIANA (DIANA, 2002), and Enochsson (Enochsson et al., 2007) using the general finite element package ABAQUS (ABAQUS, 2007).

The author will use the general finite element package ABAQUS and details will be given in Chapter 4.

The most important issue is to determine how concrete should be modelled. This research considered both using the general stress-strain relationship of concrete under tension and the crack model developed by Hillerborg (Hillerborg et al., 1976) based on fracture energy.
Figure 2.16 Crack model of Hillerborg (Hillerborg et al., 1976)

The Hillerborg model was found to give better results in ensuring numerical stability and agreement with test results. In this model as shown in Figure 2.16, crack is assumed to develop if the stress at the crack tip reaches the tensile strength of the concrete ($f_t$). The tensile stress is then assumed to decrease with increasing crack width $w$ until the crack width reached a limited value ($w_1$) when the tensile stress is equal to zero. A linear decreasing relation between crack width and tensile stress may be proposed as shown in Figure 2.17. This model introduces the concept of fracture energy, which is defined as the energy required to open a unit area of crack, which is equal to the area under the stress-displacement curve.

Figure 2.17 Stress-crack width model proposed by Hillerborg
2.5 DESIGN METHODS FOR PUNCHING SHEAR RESISTANCE

An important objective of this research is to assess applicability of current design methods. It will consider three of the most popular design methods: Eurocode 2 Part 1.1 (to be referred to as EC2) (CEN, 2002), British Standard BS 8110 (to be referred to as BS8110) (BSI, 1997) and American Concrete Institute code ACI 318-05 (to be referred to as ACI 318) (ACI, 2005). All three methods employ the same basis of design calculation of punching shear resistance: the punching shear resistance is the critical shear area multiplied by shear stress per unit area at the critical shear perimeter. Three key factors for calculating punching shear resistance are introduced below. Both EC2 and BS8110 adopt international system of units (mm, kg), but ACI 318 uses imperial units (inch, lb).

2.5.1 Effective depth (d)

In all three codes, the effective depth is defined from the compression face of the slab to the centroid of the tensile reinforcement level as shown Figure 2.18.

![Figure 2.18 Effective depth in normal flat slabs](image-url)
2.5.2 Critical perimeter

The critical perimeter is defined separately for flat slab with and without shear reinforcement.

**Slab without shear reinforcement**

In EC2, the basic critical perimeter \((u_1)\) for flat slab without shear reinforcements is normally taken to be at a distance \(2d\) from the loaded area and should be constructed so as to minimise length as shown in Figure 2.19. If there is an open area within \(6d\) near the loaded area, the control perimeter contained between two tangents drawn to the outline of the opening from the centre of the loaded area should be discarded as ineffective as shown in Figure 2.20.

![Figure 2.19 EC2 definition of critical perimeters for slab without shear reinforcement (CEN, 2002)](image)

![Figure 2.20 EC2 critical perimeter for slab with an opening (CEN, 2002)](image)
BS8110 defines a rectangular critical perimeter 1.5d from the loaded column face, as shown in Figure 2.21. Opening is treated in the same way as in EC2.

![Figure 2.21 BS 8110 definition of critical perimeter (Albrecht, 2002)](image1)

Due to the different research bases, ACI 318 has much smaller critical perimeter compared with EC2 and BS8110. ACI defines the critical perimeter ($b_o$) extending from the loaded area by $d/2$, as shown in Figure 2.22. The treatment of opening in ACI 318 is quite similar to the EC2 and BS8110 for the flat slabs without shearheads, but the limited distance 6d from the loaded area to the opening has been changed to 10 times the slab thickness. For flat slabs with shearheads, the ineffective portion of the perimeter shall be one-half of that defined in flat slabs without shearheads. The dash line in Figure 2.23 shows the effective critical perimeter for the flat slabs with openings.

![Figure 2.22 ACI 318 definition of critical perimeter (ACI, 2005)](image2)
Slab with shear reinforcement

EC2

The EC2 has two definitions of critical perimeter for slabs with shear reinforcement. When the space between two rows of the shear reinforcement is less than the 2d, the critical perimeter follows the definition of $u_{out}$ in Figure 2.24 (A). If the space between two rows of shear reinforcement is greater than the 2d, the critical perimeter follows the definition of $u_{out,ef}$ in Figure 2.24 (B). The recommended value for $k$ is 1.5.

Figure 2.24 EC2 definitions of critical perimeter for slab with shear reinforcement (CEN, 2002)
BS8110

The use of shear reinforcement other than links is not covered specifically in BS8110.

The design procedure is as follows: the shear capacity of unreinforced slab is checked first (see Figure 2.26 for critical perimeter). If the calculated shear stress does not exceed the design concrete shear stress \( (v_c) \), then no further checks are needed.

If the shear stress exceeds \( v_c \), then shear reinforcement should be provided on at least two perimeters according to Figure 2.25.

- The first perimeter of reinforcement should be located at approximately 0.5d from the face of the loaded area and should contain not less than 40% of the calculated area of the shear reinforcement added.
- The spacing of perimeters of reinforcement should not exceed 0.75d and the spacing of the shear reinforcement around any perimeter should not exceed 1.5d.
- The shear reinforcement should be anchored round at least one layer of tension reinforcement.

The shear stress should be checked on perimeters at 0.75d intervals until the shear strength is not less than the design concrete shear stress.
According to Figure 2.26, for a simplified calculation process, the critical area may be considered as a rectangular area at a distance d from the outmost shear reinforcement. (P.Y. Yan et al., 2008)
ACI 318

ACI 318 is the only code that has design guidance for punching resistance using shearhead. The critical slab section crosses each shearhead arm at $3/4$ of the distance from the column face to the end of the shearhead arm, as shown in Figure 2.27.

![Figure 2.27 ACI 318 definition of critical perimeter for flat slab with shearhead (ACI, 2005)](image)

2.5.3 Concrete shear strength resistance

**EC2** (CEN, 2002)

The unit punching shear resistance (N/mm$^2$) of concrete, without the transferred moment effect, is calculated as:

$$v_{Rd,c} = C_{Rd,c} k (100 \rho_t f_{ck})^{1/3} \geq v_{min}$$

where:

- $f_{ck}$ is the cylinder stress of concrete in N/mm$^2$
- $k = 1 + \sqrt{\frac{200}{d}} \leq 2$
\[ \rho_1 = \sqrt{\rho_{ly} \cdot \rho_{tx}} \leq 0.02 \]

\( \rho_{ly}, \rho_{ty} \) are the mean values of the reinforcement ratio within the column width plus 3d each side

\[ C_{Rd,c} = \frac{0.18}{\gamma_c} \]

\( \gamma_c \) is the partial safety factor for concrete, taken as 1.5 for the persistent and transient load, 1.2 for the accident load. This coefficient will not apply to the code design predictions for the later comparison between the codes and numerical/test results.

\[ v_{min} = 0.035k^{3/2}f_{ck}^{1/2} \]

**BS8110 (BSI, 1997)**

BS8110 gives the following equation to calculate the unit shear resistance \( (v_c) \) of concrete:

\[ v_c = 0.79 \times \left( \frac{100A_v}{b_d d} \right)^{1/3} \times \left( \frac{400}{d} \right)^{1/4} \times \left( \frac{f_{cu}}{25} \right)^{1/2} / \gamma_m \]

Where:

\( \gamma_m \) is 1.25 from the BS8110, this coefficient will not apply for the comparison between code design predictions and numerical/test results later.

\( d \) is 168mm;

\[ \frac{100A_v}{b_d d} \leq 3; \]

\[ \left( \frac{400}{d} \right)^{1/4} \] should not be taken as less than 0.67 for members without shear reinforcement and should not be taken as less than 1 for members with shear reinforcement providing a design shear resistance of 0.4N/mm²;

The maximum value of \( f_{cu} = 40 \) N/mm² is allowed in BS 8110.
ACI 318 (ACI, 2005)

For slab without shear reinforcement, the punching shear resistance of the slab is the smallest of the three following values:

(a) \( V_c = (2 + \frac{4}{\beta})\sqrt{f'c} b_0 d \)  
(b) \( V_c = (\frac{\alpha s d}{b_0} + 2)\sqrt{f'c} b_0 d \)  
(c) \( V_c = 4\sqrt{f'c} b_0 d \)

Where:

- \( \beta \) is the ratio of long side to short side of the load area
- \( \alpha_s \) is 40 for interior columns, 30 for edge columns, 20 for corner columns
- \( f'c \) is the specified compressive strength of concrete (psi)
- \( d \) is the effective depth of the slab (in)

For slab with shearhead system, the punching resistance of the slab \( (V_n) \) is equal to the punching resistance value from concrete \( (V_c) \) and contribution from the shear reinforcement \( (V_s) \).

\[ V_n = V_c + V_s \]

where:

\[ V_s = \frac{A_v f_{yt} d}{s} \]

\( V_c \) should not be taken greater than \( 2\sqrt{f'c} b_0 d \)

- \( A_v \) is the area of shear reinforcement within a distance \( s \) (in²)
- \( s \) is the spacing of shear reinforcement measured in a direction parallel to longitudinal reinforcement (in)
- \( f_{yt} \) is the yield strength of reinforcement (psi)
- \( d \) is the effective depth of the slab (in)
\( V_n \) should not be greater than \( 4\sqrt{f_c' b_0 d} \) for the critical parameter defined in the Figure 2.27 (with shearhead reinforcement) or \( 7\sqrt{f_c' b_0 d} \) for the critical parameter defined in Figure 2.22 (without shear reinforcement).

The following conditions should be observed:

- Shearhead arm should not be interrupted within the column section;
- The shearhead should not be deeper than 70 times of the web thickness of the steel shape;
- All compression flanges of the steel section should be located within 0.3d of compression surface of the slab;
- The end angle of the shearhead arm should not less than 30 degree with the horizontal.
- The plastic moment of the shearhead arm should achieve the required plastic moment strength \( (M_p) \) as described in section 7.2.1 in Chapter 7.

**2.6 SUMMARY**

This chapter has presented a brief review of punching shear behaviour in flat slab construction. To enable steel tubular column to be used, the shearhead system has been selected for further development. This system is more practical than other systems and was demonstrated by others to be feasible in providing sufficient punching shear resistance in combined steel tube-reinforced concrete slab system.
A brief review of past analytical studies on punching shear behaviour revealed the complex nature of this load carrying mechanism. This led the author to determine the strategy of this research, being to adapt current punching shear design calculation methods, rather than to develop a new analytical method. But this development will have to be based on assumptions generated from extensive finite element simulation results using simulation models that are validated by comparison against experimental results. These studies are presented in the main chapters of this thesis.
CHAPTER 3: EXPERIMENTAL RESULTS

As introduced in Chapter 1, this project will develop a new shearhead system for connection between steel tubular column and flat slab to enhance the slab punching shear resistance. Because this kind of construction has not been studied before, it is essential that some experiments are carried out to reveal behaviour of this type of structure and to provide data for validation of numerical simulations later.

This chapter describes the experiments carried out by the author and reports the experimental results. Due to constraint on time and resource, only two full scale tests were undertaken. These tests were performed at the University of Manchester’s Structural Testing Laboratory, supported by Corus (now TATA) Tubes.

3.1 TEST SETUP

A test rig with self-resistant system was built in the structural lab for the test specimen and it is shown in Figure 3.1 (UB: Universal Beam, UC: Universal Column). Six beams each with 600mm height were placed on the floor and they were connected to four big columns linked by two big crossing beams. This system supported the test specimen at the beam level which is around 1200mm above the ground level. The hydraulic pumping jack was inserted between the columns and the top cross beams to form a self-restraint system so that the laboratory floor only needed to resist the self weight of the test rig and the specimen. The test specimen was supported by a set of
four I-beams with varied cross-sections. The edge to edge distance between these support I beams, i.e. clear span of the slab, was 1605mm for both directions.

(a) Test rig arrangement

(b) Photo of the test rig

Figure 3.1 Test rig arrangement
The slab was bolted to the support beams at 4 locations along each edge as shown in Figure 3.2, and the slab was considered to be simply supported as the bolt bending stiffness was negligibly lower compared to the slab stiffness. Due to their flexibility, these I-beams experienced considerable deflections during slab loading, which had to be taken into consideration when determining the net slab deflection. The hydraulic jack and load cell were set up at the central position of the test rig over the test specimen.

![Figure 3.2 Bolted slab in the test](image)

**3.2 TEST SPECIMENS**

The purpose of this research was to investigate punching shear behaviour. Therefore, the reinforced concrete slabs of these two tests were provided with sufficient flexural reinforcement so that failure was in shear. Both tests were intended to produce the shear and bending moment combination which would be typical at an interior column of flat slab structure on a square grid of 5 meters with total imposed load of 3.5
KN/m². The test specimen size in both tests was 1825mm×1825mm on plan with 200mm depth.

3.2.1 First Test specimen

Figure 3.3(a) shows dimensions of this test specimen and Figure 3.3(b) shows the shearhead and the slab reinforcement. The test specimen was made up of an 1825mm square, 200mm thick slab with a centrally located 200×200×10 square tubular column through. Four lifting eyes with 1.2 ton capacity each were cast into the slab for lifting purpose. Load was applied from the top so that the test slab was reversed compared to that in real use, with two layers of reinforcement on the bottom face but no reinforcement on the top face. The minimum 20mm cover for the reinforcement was chosen for greatest effective depth. T12 bars at 145 mm centres were used in the bottom layer and T12 bars at 130 mm centres in the perpendicular direction were used on top of the bottom layer over the central 1.25 meters to ensure the slab failed in shear but not bending. Over the rest of the area, T12 bars at 300 mm centres and T12 bars at 200 mm centres were used respectively. The central reinforcement bars were discontinued by the shearhead system as shown in Figure 3.4.
(a) Design details for the specimen

(b) Photo of the specimen before casting

Figure 3.3: Test specimen and reinforcement arrangement
Shearhead construction

Figure 3.5 presents the shearhead system for the first test. This 480mm long shearhead system was designed to give a punching resistance of around 400KN following the EC2 method on the assumption of an enlarged column as described in Chapter 7. Four 102×44×7 joists with a top length of 140mm and a bottom length of 100mm were used to allow the shearhead system to sit on top of one reinforcement bar. In order to make the shearhead continuous, two 6mm thick and 100mm long fixing plates, on each column face, were made of equal width to the column face at one end and of equal width to the flange of the joist at the other end. Welds were made to each joist at both top and bottom flanges, as the red dash lines shown in Figure 3.5. This shearhead system was directly sat on the top of the reinforcement and across the slab from the top to the bottom, but the central reinforcement bars had to be discontinued to allow the steel tube to pass through the slab. This type of shearhead system was designed for the continued columns.
Figure 3.5: Shearhead details for test 1
3.2.2 Second Test Specimen

The shearhead system in Test 1 was designed to make the column continuous. However, it was a rather clumsy system requiring a lot of fabrication for welding and it would only be suitable for steel tubes with flat external surfaces (square/rectangular tubes). Test 2 was designed to improve the shearhead construction, but the columns would have to be story-high. If the second system were to be adopted in real construction, the storey-high columns would be connected by splice connections after the shearhead system is in place. The test specimen was made up of an 1825mm square, 200mm thick slab with a centrally located Circular Hollow Section (CHS) 219.1×6.3mm column through. Four lifting eyes with 1.2 ton capacity each were cast into the slab for lifting purpose. There were two layers of reinforcement on the bottom face but no reinforcement on the top face. Again 20mm cover for the reinforcement was adopted for greatest effective depth. The details of the reinforcement are presented in the Figure 3.6 (a). The reinforcement arrangement was almost the same as in Test 1 but adopted continued centre reinforcement bar by cutting slots in the steel tube, as shown in Figure 3.6 (b). This test also incorporated a 150mm diameter hole near the column area. This hole simulated a service hole that could be present in realistic construction. To represent the worst situation of the shearhead system, the whole shearhead system was greased over before casting concrete in order to de-bond it from the flat slab.
(a) Slab details for the second test

(b) Photo of the Second specimen before casting

Figure 3.6 specimen arrangement for the second test
Shearhead construction

In the first test, the shearhead was placed directly on top of the reinforcement mesh. Due to its height, the shearhead ended up in the middle of the slab. In the second test, the shearhead was slightly deeper and became flush with the compression surface of the slab. These two different positions of the shearhead system enabled the effects of different shearhead positions to be studied, particularly the crack initiation position for the determination of the critical punching shear perimeter.

As shown in Figure 3.7, this shearhead system was composed of two 210mm long and one 480mm long 120×60×3.6 rectangular hollow section (RHS) with 8mm full perpendicular welding. The ends of the shearhead tubes were cut at 45 degree angle. The shearhead system was placed into slots in the column and welded to the circular steel tubular column. Slab reinforcement was continuous through the slots in the circular steel column (Figure 3.6 (b)).

To avoid local bearing failure in the circular column in contact with the shearhead system, a 10mm thick plate was welded inside the column as shown in Figure 3.7. Three 20mm holes were drilled through the top edge of the shearhead to observe whether the hollow section was filled by concrete during slab casting.
Figure 3.7: Shearhead for test 2
3.3 MATERIAL PROPERTIES

Normal weight ready-mix concrete of nominal strength C40 (40MPa) was used, with maximum aggregate size of 25mm. Normal weight ready-mix concrete of nominal strength C40 (40MPa) was used, with the maximum aggregate size of 25mm. Nine 150mm cubes were cast using the same concrete batch for quality control. Three cubes were tested at 14 days, three at the day 28 and three on the same day of the slab test. The average value has been taken as the Table 3-1 presented. The Young’s modulus (E) of concrete was taken as the initial tangent to the strain-stress curve (Appendix A). No tensile test was carried out and the tensile properties were obtained following the recommendations in EC2 (Appendix B). Tensile tests were also conducted on samples of the reinforcement and shearhead steel. Table 3-1 gives the measured material properties.

Table 3-1: Measured material properties

<table>
<thead>
<tr>
<th></th>
<th>Concrete</th>
<th>Reinforcement</th>
<th>Shearhead</th>
</tr>
</thead>
<tbody>
<tr>
<td>E (N/mm²)</td>
<td>≈ 2.5×10⁵</td>
<td>2.1×10⁶</td>
<td>2.0×10⁶</td>
</tr>
<tr>
<td>Yield stress (N/mm²)</td>
<td>545.8</td>
<td>324.0</td>
<td></td>
</tr>
<tr>
<td>Tensile strength (N/mm²)</td>
<td>647.7</td>
<td>450.0</td>
<td></td>
</tr>
<tr>
<td>Elongation (%)</td>
<td>37.0</td>
<td>19.8</td>
<td></td>
</tr>
<tr>
<td>Cube strength on the day 28 (N/mm²)</td>
<td>≈ 45.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cube strength on the day of test (N/mm²)</td>
<td>≈ 45.0</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
3.4 TEST SETUP AND INSTRUMENTATION

3.4.1 Instrumentation for Test 1

Deflections at selected locations on the compression (top) side of the slab were measured by linear variable displacement Transducers (LVDTs) as shown in Figure 3.8. In addition, one more LVDT (C95) was put on the load cell bottom to measure displacement of the column centre.

Electrical resistance strain gauges were used to measure strains at selected locations on reinforcement and on the concrete compression face. Strain gauges were also placed on the shearhead flanges just beside the column surface and 100mm away from column side. The positions of the strain gauges are shown in Figure 3.9 and the arrows indicate directions of the measurements.
Figure 3.8: LVDT’s positions for test 1
To observe cracking on the tension face, a camera was put on the laboratory floor focusing on the central area of the bottom side of the slab which was painted white with wash lime of very little tension ability. Gridlines, shown in Figure 3.10, were
drawn on the bottom face of the test slabs to give some indication of locations of any crack.

3.4.2 Instrumentation for Test 2

Deflections at selected locations on the compression side of the slab were measured by linear variable displacement Transducers (LVDTs) as shown in Figure 3.11. These positions were carefully chosen with consideration of enlarged column assumption (details see Chapter 7). In addition, one more LVDT (C69) was put on the load cell bottom to measure the column central displacement.
Based on the strain results from Test 1, it was decided not to measure as many strains as in the first test. Thus, only three pairs of strain gauges were used, as shown in Figure 3.12. These positions were considered critical to obtaining information to assess existing code design methods. A three-way rosette strain gauge was also placed.
on the web of one shearhead to detect the shearhead arm behaviour. The arrows in Figure 3.12 indicate directions of these strain measurements.

![Strain gauge positions](image)

(a) Strain gauge positions on reinforcement and concrete face

(b) Strain gauge positions on shearhead arm

Figure 3.12 Strain gauge positions for the second test

As in Test 1, a camera was used to observe cracking on the tension (bottom) face of the slab. The tension face of the slab was also painted with the same white wash lime
and gridlines to help locate cracks as the first test. Figure 3.13 shows the bottom surface of Test 2 before test.

![Tension face of the second slab before test](image)

**Figure 3.13: Tension face of the second slab before test**

### 3.5 OBSERVATIONS AND RESULTS

#### 3.5.1 Test 1

The applied load was increased by 10 KN until 300 KN and 5 KN afterwards until slab failure, and there is a reload stage for 150KN according to the oil leak of the pump during the test. The data was collected manually for every 10KN below 300KN and every 5 KN after that.
3.5.1.1 Observations

The first visible cracks appeared roughly 50mm away from the middle of the column outside edge in the weak direction when the applied load was about 130KN. The weak direction was the direction with less effective depth for the reinforcement. Then a couple of more cracks from the middle and corner of the column edges appeared towards to the edges of the slab. These cracks became wider and wider with increasing load. A number of new visible cracks appeared before the load reached 350KN. After that, more cracks developed radially from the column face towards the slab edges. When the applied load reached 400KN, the cracks increased rapidly in width and until the slab failed at 417 KN. The failure load was taken as the maximum applied load before the shearhead punched through the slab and the recorded load experienced a sudden drop. Figure 3.14 shows the last moment of the slab tension side before it failed, and Figure 3.15 shows the compression and tension faces of the slab after reaching failure.

Figure 3.14: Tension side of the first test slab before collapse
(a) Compression face

(b) Tension face

Figure 3.15: Appearance of the first test slab face after failure
Figure 3.16: observed crack pattern at the reinforcement level of the first test

The failure parameter enclosed an approximate square area of about 800mm in dimension as observed from the slab tension face. The width of the flexural cracks near the column was about 1mm at failure. In contrast, on the compression surface (Figure 3.15(a)), there was hardly any sign of concrete in distress. The failure pattern at the normal reinforcement level has been observed in the test showed in Figure 3.16.

3.5.1.2 Load-deflection

Figure 3.17 shows the load versus deflection curve measured from the test. Only a selection of curves is presented because the LVDTs recorded similar deformations at the same distance from the slab centre, please check Appendix A for the rest data. Rapid reduction in load after reaching the peak value clearly indicates brittle punching
shear failure. The supporting I beams to the structure also experienced noticeable deflections. Later in numerical simulation (Chapter 5), these deflections should be deducted from the total slab deformation at the centre to give the net deformation of the slab centre relative to the edges. After the slab load suddenly dropped, no further attempt was made to further load the slab. Because the slab dimensions were small and the slab behaviour governed by punching shear, the bending deflections in the slab were small so all LVDTs, except the one at the centre column (C95), recorded similar deflections. The centre column and the points different after the slab reached failure point slab centre deflection was dominated by punching shear deflection. This is also proved by the decreasing stage of the load-deformation record difference between the centre column and rest points on the slab.

Figure 3.17 Measured raw load-deflection curve for the first test (see 3.4.1 for LVDT locations)

3.5.1.3 Strains

Figure 3.18 presents the measured strain-load relationships for the slab of the first test at different distance from the slab centre and Figure 3.19 presents the results for the
shearhead arm at different positions of the first test. Please check Appendix A for the rest data. Strain-load curves indicate complex slab behaviour and this will be used to check results of the numerical model in Chapter 5.

(a) Strains on the concrete compression face (see Figure 3.9 for strain gauge locations)

(b) Strains for the tension reinforcement bars (see Figure 3.9 for strain gauge locations)

Figure 3.18 Measured strain results for the slab in test 1
The reinforcement bar beside the column recorded compression strain after the load exceeded 200KN. This was caused by the discontinued reinforcement in the slab centre.

(a) Strain for the sheararm top side (see Figure 3.9 for strain gauge locations)

(b) Strain for the sheararm bottom side (see Figure 3.9 for strain gauge locations)

Figure 3.19 Measured strain – load relationships for the shearhead arm in test 1
3.5.2 Test 2

The applied load was increased by 10KN until the load reached 400KN, after which
the load was increased by 5KN until failure. The data was read manually after each
load increment.

3.5.2.1 Observations

The first visible cracks appeared from the edge of the circular column in the weak
direction when the applied load was about 180KN. Again the weak direction was the
direction with less effective depth for the slab reinforcement. Then more and more
cracks developed from the column edge towards to the edges of the slab radially. For
the quarter of the slab with the hole, the cracks formed much close to the column area
compared to the other three quarters of the slab without hole. Also most of the cracks
in the slab quarter with the hole stopped at the edge of the hole and did not develop
toward to the slab edges.

![Developed cracks for part with hole](image)

![Developed cracks for part without hole](image)

Figure 3.20: Tension side of the second test slab just before collapse
When the load reached 330KN, more cracks appeared around the column area, but not radially. After this stage, quite a few new cracks appeared but the existing cracks increased in width. There was a very rapid increase in crack width after the load reached around 520KN until the slab failed at 569KN. More cracks were detected before failure compared with the first test.

Figure 3.20 shows the appearance of the slab on the tension side just before the slab failed by punching. The cracks that developed in the solid area were quite similar to those observed in the first test, which were from the centre column area towards the slab edges. In the slab quarter with the hole, cracks also developed from the centre but they stopped by the edge of the hole. This made the critical perimeter in this part of the slab much smaller compared with that in the solid part of the slab. Figure 3.21 shows patterns of damage on both the compression and tension sides of the slab immediately after failure.

In contrast with test one, which showed hardly any damage on the compression side (Figure 3.15(a)), a big crushed area was observed in the second test specimen. Separation of the steel tube from the concrete core could be clearly seen on the slab compression face as shown in Figure 3.21 (a). On the tension side, large areas of concrete split off as a result of the higher applied load.
Figure 3.21 Failure pattern of the second test

(a) Compression face

(b) Tension face

Shearhead system punched through
In the slab quarter with the hole, the largest crack started at a depth of about 30mm away from the slab compression face (top) to the reinforcement level as shown in Figure 3.22. It then joined with the cracks originating from the ends of the shearhead arms. This resulted in a smaller critical punching shear area compared with the three solid quarters of the slab.

![Figure 3.22 Crack around the hole](image)

**3.5.2.2 Load-deflection relationships**

Figure 3.23 shows the record load – deformation curves for the selected positions in the second test specimen, please check Appendix A if the rest data needed. The slab behaved in a symmetrical way and similar displacements were recorded by LVDTs placed at the same distance from the slab centre. The steel tube punched through the slab by about 6mm at failure, from a comparison between deflections at the column
centre and the end of the shearhead arm. The deformation difference between the centre column (C69) and rest points on the slab after the failure load point achieved is also a solid evidence for the shearhead system punched through the slab.

Figure 3.23: Measured raw Load-deformation curves for the second test (see 3.4.2 for LVDT locations)

3.5.2.3 Strain results

Figure 3.24 plots the measured slab strain – load relationships for the slab concrete compression face and tension reinforcement. The main purpose of recording the strains was for validation of the numerical model later. Nevertheless, these results help to understand the slab behaviour. Because the reinforcement in Test 2 was continuous, the recorded reinforcement strains show a trend of monotonic increase, in contrast to the results of Test 1 (Figure 3.18), which had one discontinuous reinforcing bar at the slab centre. Figure 3.25 also indicates that the sheararm had not
reached yield strain when the slab failed, suggesting that the shearhead was entirely effective.

(a) Strains for the compression concrete face

(b) Strain for the tension reinforcement level

Figure 3.24: Measured slab strain – load relationships for test 2
3.5.2.4 Critical perimeter determination

For design calculation of flat slab punching shear capacity, the critical perimeter, defined as the position where punching failure occurs, is one of the most important factors. Although the crack patterns recorded on the tension side of the slab (Figure 3.15 and Figure 3.21) may be used to give some indication of the critical perimeter, since the crack may be caused by breaking off of the cover concrete, it was difficult to use this information to determine the actual location of the critical perimeter. It was also difficult to determine the angle of the critical crack through the slab thickness. In addition, the amount of energy released will depend on the main crack. It is therefore important to obtain precise information on the main crack. To collect this information, the second slab was cut through its thickness by clipper to reveal the main cracks through the slab thickness. This was done because its behaviour was complex when there was a hole in the slab. The first slab was not cut because the slab behaviour was more straightforward and cutting the slab was a very time-consuming and expensive process.

Figure 3.25 Measured strain-load relationships for the shearhead arm in test 2
Figure 3.26 shows the cutting plan for the second test specimen. Cuts A and B give information about the main crack through the solid quarters; cut E gives information of the main crack near the shearhead; cuts G and F are for investigating the main crack around the hole.

Figure 3.26 Through thickness cut patterns for the second test specimen
Cuts A and B

Figure 3.27 shows clearly that the main through thickness crack of the solid part of the slab developed from the ends of the shearhead arms towards the reinforcement at an angle of around 45 degree. The crack angle was much bigger above the reinforcement level, clearly indicating this was caused by concrete cover breaking off.

Figure 3.27 Cut Face A
**Cut E**

The crack at Cut E (Figure 3.28) started at the bottom of the slab at this location rather than somewhere within the slab depth. This means that the cracks started from this location, which is strong evidence that the shearhead system behaved as a bigger column for punching shear resistance and that the perimeter of the equivalent column was by increasing the actual column dimensions by the size of the shearhead arms.

![Figure 3.28 Cut face E](image)

**Cuts G and F**

Not surprisingly, crack development in the quarter of the slab with hole was quite different from the other three solid quarters. Figure 3.29 shows the cracks at cuts F and G. It appears that the cracks followed two separate lines. One initiated from the outside edge of the circular column and other from the end of the shearhead arm. Both cracks were at approximate 45 degree angle as in the solid parts of the slab.
Figure 3.29 Cut faces F and G

**Around the hole**

Figure 3.30 shows crack details around the hole, viewed from the tension side of the slab. Some cracks developed from the edge of the column but they all stopped outside the hole. The cracks developed around the hole were also at 45 degree angle, similar to the solid parts.
Figure 3.30 Crack details around the hole

**Overall crack pattern**

According to the cut face information presented before, the critical perimeter at the tension reinforcement level is presented in Figure 3.31 as indicated by the thick line. And this pattern will be compared with the enlarged column assumption according to BS8110 (BSI, 1997) and EC2 (CEN, 2002) in Chapter 7.
3.6 DISCUSSIONS

Both tests adopted similar column size, slab dimensions and reinforcement ratio, but test 2 had a hole. The same shearhead arm length was used but with different shearhead arm cross-sections. However, test 2 achieved much higher punching shear capacity than Test 1 (569 KN compared with 417 KN). As shown by the major crack positions, the sheararm position relative to the slab thickness was a major factor in determining the punching shear critical perimeter length. The major cracks started from the end of the sheararms and extended to the level of tensile reinforcement. As shown in Figure 2.32, this gave the shearhead system of Test 2 a larger critical
perimeter length compared to Test 1, hence allowing Test 2 to reach a higher punching shear resistance. The different behaviours of the top concrete compression face in the two tests also proved this proposal.

The difference in punching shear resistance of the two tests would not have been caused by the difference in the sheararms. The sheararms in the two different tests had the same length, but differed in cross-section dimensions and method of connection to the column. However, the strain records for both sheararms (Figures 3.19 and Figure 3.25) indicate that the sheararms did not yield when the slabs reached failure. Neither would the slight difference in the flexural reinforcements (all bars continuous in Test 2 but the centre bar discontinuous in Test 1) would have caused the significant difference in the slab punching shear resistance. Figure 3.18 shows that the reinforcements did not yield in Test 1 when the slab reached its failure load, suggesting that the reinforcement was not the governing factor. Nevertheless, as will be shown in section 6.3.4 through numerical simulations, continuity of the reinforcement may have some influence on the slab punching shear resistance, but the influence is not big and would not be able to explain such large difference recorded in the slab punching shear resistance of these two tests.

Figure 3.32 Sketches of shearhead arrangement through slab thickness
3.7 CONCLUSIONS

This chapter has presented details of two tests on a new connection system between steel tubular column and flat slab structure. The following conclusions may be drawn.

- Attaching shearhead to steel tubular column is an effective method of enhancing punching shear resistance around the column head. There was no need for shear reinforcement, which could mean considerable saving in cost. Among the two types of shearhead tested, the second test will be more practically feasible owing to reduced fabrication effort required.

- From detailed crack pattern obtained from the second test, the shearhead system performed like a bigger column whose dimension was obtained by adding the total lengths of the shearhead arms to the original column dimension in the same direction. This comes from the observation that the main through-slab thickness cracks originated from the ends of the shearhead arms.

- The vertical position of the shearhead appears to be an important factor in affecting slab punching shear resistance. The shearhead system in Test 2 was placed higher than Test 1, giving larger effective depth and critical perimeter, which led to 35% increase in slab punching shear capacity compared to Test 1, even though Test 2 incorporated a 150mm diameter hole near the column area.
CHAPTER 4: NUMERICAL MODELLING

METHODOLOGY

The general finite element model ABAQUS was used to carry out extensive numerical simulations to develop a database of results to help understand punching shear behaviour of steel tube to flat slab using shearhead construction and to develop a simplified design method. Validation of the simulation results using ABQUS will be established in this chapter and Chapter 5. This chapter will explain the basic simulation methodology, presenting various sensitivity study results and comparing numerical simulation results against the analytical results of Timoshenko (Timoshenko and Krieger, 1970) for elastic slab and the test results of Marzouk and Hussein (Marzouk and Hussein, 1991). Chapter 5 will present detailed comparison with the author’s own two tests reported in Chapter 3.

4.1 SIMULATION METHODOLOGY

ABAQUS (ABAQUS, 2007) is a power general finite element package with many options. Therefore, it is important that appropriate decisions are made when selecting key variables.

4.1.1 ABAQUS/CAE

ABAQUS/CAE is useful pre-processor to build a finite element model. It defines the model’s geometric, material properties, boundary condition, loading and also meshes
the structure. The numerical models in this study were constructed by the ABAQUS/CAE process.

4.1.2 Finite Element Type

To simulate the structure covered in this research, the following elements from ABAQUS were used: solid elements, truss elements and shell elements. Solid elements were used to model the concrete slab, shearhead system and tubular column, truss elements used to model reinforcement bars and shell elements used to model shearhead arms.

4.1.2.1 Solid elements

ABAQUS has a large library of solid elements each with different capability, accuracy and efficiency, a few of typical elements shown in Figure 4.1. C3D8 is a brick element only has integration nodes at its corners, and linear interpolation would be applied in each direction. C3D8 is the most general element adopted in most of the 3D finite element models according to its quick solution and good accuracy. C3D20 generally used for very detailed model, it has more integration points in each element compared with the C3D8. This could bring some benefit when the transfers in each element are very big, but this will increase the computing time lots. The C3D10M suits for some irregular shapes, but its accuracy not as good as the cube elements. Considering both the running time and accuracy for the numerical analysis, ABAQUS solid element type C3D8 was chosen in this numerical study.
Shell elements are usually applied to structure that has significantly smaller thickness compared to other two dimensions. Conventional or continuum shell elements may be used. Both types of shell element have similar kinematic and constitutive behaviours but continuum shell element looks like three-dimensional solid. Figure 4.2 compares these two types of shell element. For conventional shell element, its thickness is defined through the section property definition. In contrast, the thickness of continuum shell element is determined by the element nodal geometry.

Figure 4.2: Differences between conventional and continuum shell elements (ABAQUS, 2007)
A comparison will be made in section 5.3.2.2 in Chapter 5 by using these two different shell element types. However, using the conventional shell element is preferred because of the much shorter simulation time.

4.1.3 Interactions

A number of interactions exist in the structure, between different materials. It is important that these interactions are correctly treated to ensure efficient and accurate modelling of structural behaviour.

4.1.3.1 Tie constraints

In this research, the steel and concrete components of the structure are modelled using different elements. Since they can experience separation, it is not appropriate to use one set of nodes to represent the two different materials initially in contact with each other. Instead, “Tie” constraints were applied. Denoting one face as master and the other as slave, this constraint ties the slave and master faces together for the duration of a simulation. This tie means that each node on the slave surface has the same translational and rotational motion as the closest point on the master surface. The slave surface can be either element-based or node based surface. The master surface can be any type of surface. Two types of formulation are available: surface-to-surface formulation or node-to-surface formulation. The surface-to-surface formulation in general avoids stress noise and is the default in ABAQUS/Standard.
In this research, the steel tube column and the shearhead arms were tied together as a whole shearhead system. The bottom face of the shearhead arms was tied with the concrete due to their small relative motion. Figure 4.3 shows these ties.

![Figure 4.3 Tie condition in the model](image)

**4.1.3.2 Embedded elements**

The ABAQUS “embedded” element is used to represent the reinforcement bars. It is a simply and accuracy solution for the reinforcement in the concrete slabs.

Embedded element can be used to specify an element or a group of elements that lie
embedded in a group of host elements whose response is used to constrain the translational degrees of freedom of the embedded nodes. Either the default elements which are searched and selected by the program from the vicinity of the embedded elements can be used as host elements, or the user can define a set of host elements to limit the search to this subset in the model. A geometric tolerance should be defined for the embedded elements. Geometric tolerance is the distance within which the embedded nodes must lie and it is calculated by multiplying the average size of all non-embedded elements in the model by a factor. The default value is 0.05 (ABAQUS, 2007). For a three-dimensional model, the embedded element can be provided to beam, shell, membrane, solid, surface or truss which lie in solid elements. Embedded elements are generally used for rebar elements in ABAQUS.

4.1.3.3 Coupling

To avoid local stress concentrate, a concentrated load can be transferred to the top face of the column through the ABAQUS coupling constraint.

The coupling constraint provides coupling between a reference point and a group of nodes referred to as the “coupling nodes”. The coupling constraint includes kinematic coupling and distributing coupling constraint. Kinematic coupling constrains the motion of the coupling nodes to the reference node. This constraint can be applied to user-specified degrees of freedom at the coupling nodes with respect to the global or a local coordinate system. Distributing coupling constrains the motion of the coupling nodes to the translation and rotation of the reference point. This constraint is enforced in an average sense in a way that enables control of the transmission of loads through weighted factors at the coupling nodes. The distributing weight factors are calculated
automatically in ABAQUS if the surface is an element-based surface. Various weighting methods, such as uniform, linear, quadratic and cubic, can be used, allowing the applied forces to be transferred to the coupling nodes to vary inversely with the radial distance from the reference node. In the test condition, the load was continuing applied to the top of the column through a 25mm thick steel plate. To simulate this uniformed applied load, distributing coupling procedure has been adopted in this study as Figure 4.4 shows.

![Figure 4.4 Application of coupling procedure](image)

**4.1.4 Concrete Material Model**

Concrete material is notoriously difficult to model numerically and considerable effort has been spent in this research to select an appropriate concrete material model.
The stage prior to concrete cracking is considered as isotropic linear-elastic in this study. The plasticity stage of concrete can be defined as smeared cracking concrete or use the concrete damaged plasticity model in ABAQUS. They both can be used for modelling concrete in all types of structures with and without rebar. Their main differences are the concepts and design under different load applications.

Smeared cracking concrete model uses orientated damaged elasticity concepts to describe the reversible part of the material’s response after cracking failure. Concrete damaged plasticity model uses concepts of isotropic damaged elasticity in combination with isotropic tensile and compressive plasticity to represent the inelastic behaviour of concrete. Smeared cracking concrete model is only designed for application in which the concrete is subjected to essentially monotonic straining at low confining pressures, and the concrete damaged plasticity model can be used under monotonic, cyclic or dynamic loading under low confining pressures. This research used the damaged plasticity model for concrete to allow modelling of sudden failure under puncting shear.

The tension behaviour of concrete can be specified using fracture energy model or strain-stress model in the damaged plasticity model. The Hillerborg failure model (Hillerborg et al., 1976), as shown in Figure 4.5, is included in the ABAQUS material library for the fracture energy-displacement model. This simple model may not be able to follow the exact softening line of concrete in tension, but it can save much computation time and it has been shown to produce good simulation results by previous researchers. When using this model, the concrete tensile failure stress is
specified as a function of fracture energy, which is indicated by the area below the stress-displacement diagram shown in Figure 4.5. Appendix C provides more information on fracture energy value. Since there is large uncertainty in fracture energy value for any one grade of concrete, a sensitivity study was carried out to select appropriate fracture energy values for the concrete used in this research. Details will be presented in section 5.3.1.2 in Chapter 5. Implementation of this stress-displacement concept in a finite element model requires the definition of a characteristic length associated with an integration point. The characteristic crack length is based on the element geometry. For the solid concrete elements in this study, the cubic root of the integration point volume is used. The reason for such definition of the characteristic crack length is because of the unknown crack direction in advance. Therefore, elements with large aspect ratios will have rather different behaviour depending on the direction in which they crack and some mesh sensitivity remains because of this effect. Ideally elements that have aspect ratios close to one should be used.

![Figure 4.5: Hillerborg failure model for fracture energy-displacement model (ABAQUS, 2007)](image)
4.2 ELASTIC SLAB BEHAVIOUR BY TIMOSHENKO

As will be seen later in section 4.3 of this chapter and in Chapter 5, there will be some discrepancies between numerical modelling results and test results for slab deformations. To ensure that these differences are not due to inaccurate modelling, the simple case of the elastic deformation of a simply supported slab (Figure 4.6) has been modelled to check the basic numerical model. Exact analytical solution is available from Timoshenko (Timoshenko and Krieger, 1970).

\[
\omega = \frac{1}{\pi^4 D} \sum_{m=1}^{\infty} \sum_{n=1}^{\infty} \frac{a_{mn}}{\left(\frac{m\pi}{a}\right)^2 + \left(\frac{n\pi}{b}\right)^2} \sin \left(\frac{m\pi x}{a}\right) \sin \left(\frac{n\pi y}{b}\right) \tag{4-1}
\]

Figure 4.6 Load case for a simply supported slab (Timoshenko and Krieger, 1970)

4.2.1 Timoshenko analytical solution

Figure 4.6 shows a simply supported slab under a load \( P \) which is uniformly distributed over the central shaded area. The Timoshenko analytical solution for the slab out of plane deformation (\( \omega \)) is given in equation 4-1.
Where

$q$ is density of the uniformly distributed load.

\[
a_{mn} = \frac{16P}{\pi^2 mnuv} \sin \frac{m\pi \xi}{a} \sin \frac{n\pi \eta}{b} \sin \frac{mnu}{2a} \sin \frac{n\pi v}{2b}
\]

\[
D = \frac{Eh^3}{12(1-\nu^2)}
\]

In which

$h$ is the slab depth

and $m = 1, 3, 5, 7 \ldots$

Consider the following case (similar to the data used in test 1):

$u=v=200\text{mm}, a=b=1620\text{mm}, \xi = \eta = \frac{a}{2} = \frac{b}{2} = 810\text{mm}$ in Figure 4.6,

$h=200\text{mm}, E = 30000 \text{ N/mm}^2, \nu=0.2, P=400\text{KN}$.

According to equation (4-2):

$m = 1, \omega \approx 0.52\text{mm}$

$m = 3, \omega \approx 0.0057\text{mm}$

$m = 5, \omega \approx 0.0006\text{mm}$

……

So, the analytical maximum deflection for the centre of slab is 0.526 mm at a total load of 400 KN.
4.2.2 Comparison with numerical results

The same slab as in the previous section has been modelled by ABAQUS using the simulation methodology described in section 4.1. The ABAQUS model is shown in Figure 4.7. Solid elements were used and the element size was 50mm or 20mm.

![ABAQUS model for a simply supported slab](image)

Table 4-1 compares the two simulation results (using two different element sizes) and the analytical solution. The agreement is excellent.

<table>
<thead>
<tr>
<th>Method</th>
<th>Element size (mm)</th>
<th>Load (KN)</th>
<th>Deformation (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Timoshenko plate theory</td>
<td></td>
<td>400</td>
<td>0.526</td>
</tr>
<tr>
<td>ABAQUS</td>
<td>20</td>
<td></td>
<td>0.542</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td></td>
<td>0.554</td>
</tr>
</tbody>
</table>
4.3 NUMERICAL SIMULATION OF MARZOUK’S TESTS

A series of tests on punching shear behaviour of high-strength concrete slabs were carried out by Marzouk and Hussein (Marzouk and Hussein, 1991). These tests have been simulated using ABAQUS by the author.

4.3.1 Introduction to the experiments

Figure 4.8 shows typical dimensions of the test specimens. The test specimens were simply supported along the edges with the corners free to lift. In total, seventeen reinforced concrete slabs were tested with varied slab depths and reinforcement ratios between 0.49% and 2.33%. Reinforcing bars consisted of Grade 400 steel with actual tested yield strength of 490 N/mm². Rubber packing pieces were added just underneath the slab surface to make sure of uniform contact along the supports. Table 4-2 lists details of three of the tests chosen for detailed comparison due to their different types of behaviour.

Figure 4.8 Typical test specimen dimensions (Marzouk and Hussein, 1991)
Both flexural bending failure and punching shear failure were observed. Three typical tests that experienced punching shear failure (NS1, HS3 and HS4) were simulated using ABAQUS.

Table 4-2 Full details for the Marzouk's test slabs

<table>
<thead>
<tr>
<th>Slab no.</th>
<th>Compressive strength (N/mm²)</th>
<th>Bar size</th>
<th>Bar spacing (mm)</th>
<th>Column diameter (mm)</th>
<th>Slab thickness (mm)</th>
<th>Average depth (mm)</th>
<th>Steel ratio (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NS1</td>
<td>42</td>
<td>M10</td>
<td>71.4</td>
<td>150</td>
<td>120</td>
<td>95</td>
<td>1.473</td>
</tr>
<tr>
<td>HS3</td>
<td>69</td>
<td>M10</td>
<td>71.4</td>
<td>150</td>
<td>120</td>
<td>95</td>
<td>1.473</td>
</tr>
<tr>
<td>HS4</td>
<td>66</td>
<td>M15</td>
<td>93.7</td>
<td>150</td>
<td>120</td>
<td>90</td>
<td>2.370</td>
</tr>
</tbody>
</table>

4.3.2 Numerical models

3D deformable solid elements (3D8R) were used for the concrete core. For sensitivity study, two solid element sizes of 20mm and 40mm were used for test NS1. The aspect ratio was close to one. Truss elements were used for both the flexural reinforcements and anchor bars. The truss elements were embedded in the concrete elements. The concrete fracture energy model was according to the CIP-FIP model 1990 (CEB-FIP, 1993) and details are given in Appendix C. The Hillerborg concrete tension softening model was adopted in this series of models. The tension stress of the concrete was according to EC2 Table 3-1 (CEN, 2002), giving a tension stress of concrete for test NS1 of 2.1 N/mm². For the reinforcement bars, The Young’s modules was $2 \times 10^5$ N/mm² and the yield stress was 490 N/mm² as given by the report. Considering
that all these test cases were failed by punching but not deflection, a rough stress-
strain curve for the reinforcement bars based on this Author’s test data is given in
Figure 4.9.

Figure 4.9 Reinforcement stress-strain curve adopted in the numerical analysis

Due to the symmetry of the slab, only a quarter of the slab was modelled.

Figure 4.10 Typical ABAQUS model for Marzouk’s test

Figure 4.10 is a typical ABAQUS model. The column could only move in the vertical
direction (Z). For the two vertical mid-faces of the slab, the surface parallel to the X
direction was fixed with no movement in the Y direction and no rotation about the X direction; the surface perpendicular to the X direction was restrained for movement in the X direction and rotation about the Y direction. The slab was simply supported along a line 100mm away from the most outside face of the slab as in the test arrangement.

4.3.3 Numerical results

Test NS1 used normal strength concrete and experienced typical punching shear failure mode. It is used as the reference in this study. The ABAQUS solid element size was 20mm. Figure 4.11 compares the numerical and simulation load-deformation curves.

The simulation result for the slab load carrying capacity, defined as the maximum load, is in close agreement with the test result. However, the simulated deformations are much lower than the test results. This is typical of all simulation results, see Figure 4.12 for test HS3 and Figure 4.13 for test HS4. This may be explained by the test deformation results including that of the rubber bearing underneath the slab, but this value was unknown and was not included in the slab model. Figure 4.14 compares simulated and measured strains. The agreement is better than that shown in Figure 4.12 for the load-displacement curves.
Figure 4.11 Comparison of load-deformation curves for test NS1

Figure 4.12 Load-deformation curve comparison for test HS3

Figure 4.13 Load-deformation curve comparison for test HS4
Though the numerical study for Marzouk’s tests not perfect according to the material property missing, the result proved that Riks method is a sufficient way to predict the failure point for the punching failure in flat slab structures. This study earned more confidence in using the numerical model ABAQUS to simulate punching shear behaviour in flat slabs.

Figure 4.14 Comparison of strains for test HS3

(a) 50mm away from the centre of slab

(b) 250mm away from the centre of slab
4.5 CONCLUSIONS

Based on the simulation results and their comparison with analytical results of Timoshenko and experimental results of Marzouk’s tests, the general finite element software ABAQUS has been proved to be correctly used to simulate punching shear behaviour in flat slab. The damaged plasticity model in compression combined with the Hillerborg fracture energy model for tension was found appropriate for concrete.

Chapter 5 will present detailed simulation results for the author’s two tests (described in Chapter 3) and further sensitivity study results. Together, they present evidence of ABAQUS capability and suitable application of ABAQUS by the author.
CHAPTER 5: VALIDATION OF ABAQUS
SIMULATION: COMPARISON AGAINST THE
AUTHOR’S OWN TESTS

This chapter presents further validation study of using the general finite element software ABAQUS, by comparing numerical simulation results with the author's own tests and by carrying out a series of sensitivity studies based on the first test. Comparisons will be made for all measured results, including slab load-deformation, load-strain relationships and cracking pattern.

5.1 MODELLING TEST 1

5.1.1 Geometric arrangement

Due to symmetrical loading and structural arrangement, only a quarter of the test specimen was modelled. Figure 5.1 shows the simulation model, including the following 5 main parts:

- Concrete core
  
  Solid C3D8 elements were used and the maximum element size was 20mm. The dimension was 900mm square and 200mm thick with a hole in the corner to let the steel column through. The concrete core has some excised parts for the shearhead system assembly.
• Flexural reinforcement

Truss elements were used. The centres of the truss elements were the same as in the test and the cross-section area of the truss elements was 113.1 mm$^2$, corresponding to 12mm diameter round bars.

![ABAQUS model for Test 1](image)

Figure 5.1: ABAQUS model for Test 1

• Steel tube

A quarter of steel tube RHS 200×200×10 was modelled. Because the steel tube was not critical in slab behaviour, the root radius of the tube was not modelled. The modelled steel tube length was 250mm and the bottom of the steel tube was flush with the concrete core. Deformable solid elements C3D8R were used and the maximum element size was 20mm.
• **Shearhead arms**

The shearhead arms (using joist size 102×44×7) were modelled using deformable solid elements. The maximum element size was 15mm. The top length of the shearhead arm was 140mm and the bottom length of the shearhead arm was 100mm, exactly according to the test. The root radius of the joist section was not modelled.

• **Stiffener plates**

This is the 6mm thick trapezoidal plate welded to the top and bottom flanges of the joist section. It was modelled using C3D8 and the maximum mesh size was 15mm.

### 5.1.2 Boundary conditions and interactions

The shearhead arms and stiffener plates were tied together to make them work as a whole part, which were then tied to the column face. Friction was used to model the steel-concrete interface with a friction coefficient 0.45 ([http://www.supercivilcd.com/FRICTION.htm](http://www.supercivilcd.com/FRICTION.htm)). The flexural reinforcement truss elements were embedded inside the concrete core as host group.

In order to overcome convergence problem and to reduce computation time, the contact faces between the shearhead system (which includes both the shearhead arms and the stiffener plates) and the concrete core were assumed to have no relative translational or rotational motion.
When calculating the net slab deformation in the centre, the slab deformations along the supporting steel beams were assumed to be uniform as recorded at the centre of the beams. To validate this assumption, a numerical model for the steel support beams was set up and analysed using ABAQUS, as shown in Figure 5.2. At the recorded slab failure load (417kN), the relative deflection between the beam centre position and the slab end position was only 0.7mm out of a total of 5mm. Therefore, the uniform slab edge deformation assumption may be accepted.

Figure 5.2 Deformed model for the supporting beams

The symmetrical faces (the centre faces of the test slab) were restrained in movement in the respective perpendicular direction to the surface and also prevented from rotation about the axis parallel to the surface. A reference point at 30mm above the centre position of the steel tube was coupled (distributing coupling) to the top face of
the column to ensure that the applied load was evenly distributed to the simulation model. The column was allowed to move in the vertical direction only.

5.1.3 Material properties

The following four different materials were defined for the model structure.

- **Flexural reinforcement**
  The stress-strain relationship is shown in Figure 5.3. It was defined as elastic-plastic-strain stiffening material with Young’s modulus $E=2 \times 10^5$ N/mm$^2$ and Poisson’s ratio of 0.3.

![Stress-strain relationship for steel reinforcement](image)

Figure 5.3: Stress-strain relationship for steel reinforcement

- **Shearhead steel**
  Figure 5.4 shows the stress-strain relationship. It was defined as elastic-plastic-strain stiffening material with Young’s modulus $E=2 \times 10^5$ N/mm$^2$ and Poisson’s ratio is 0.3.
Figure 5.4: Shearhead arm stress-strain relationship

Concrete

Figure 5.5 shows the uni-dimensional stress-strain relationship. Under compression, it was defined as elastic, concrete damaged plasticity material with Young’s modulus $E=25\text{ GPa}$ and Poisson ratio of 0.2. The default dilation angle was 35°, but varying from 20° to 50° in the sensitive study. The concrete damaged plasticity model assumes non-associated potential plastic flow and used the Drucker-Prager hyperbolic function to make sure the flow direction is always uniquely defined. The ABAQUS default value (0.1) for eccentricity was used in this study, which implies that the material has almost the same dilation angle over a wide range of confining pressure stress values. The ABAQUS default value (1.16) was used for the ratio of initial equibaxial compressive yield stress to initial uniaxial compressive yield stress and a value of 0.667 was used for the ratio of the second stress invariant on the tensile
meridian to that on the compressive meridian adapted to account for different evolutions of strength under tension and compression.

Fracture energy was used to define the tension behaviour of concrete according to CEB-FIP 1990 (CEB-FIP, 1993). The tension stress adopted the conservative definition in Table 3-1 (Appendix B) in EC2 (CEN, 2002) relative to concrete compression stress measured in the test. The fracture energy of concrete was taken as 120N/m as presented in Appendix C following guidance in CEB-FIP model 1990. The tensile stress-strain curve in Figure 5.5 was obtained using a fracture energy of 120N/m and element size of 20mm.

![Concrete stress-strain curve](image)

Figure 5.5: Concrete stress-strain curve

- Super elastic steel

The column was defined as pure elastic material with a very high Young’s modulus $E = 2000\text{GPa}$ and Poisson ratio of 0.3.
Table 5-1 summarises the different material property models for the different parts of the structure.

Table 5-1 Summary of material property definitions

<table>
<thead>
<tr>
<th>Parts</th>
<th>concrete core</th>
<th>flexural reinforcement</th>
<th>Steel tube</th>
<th>Shearhead arms</th>
<th>Stiffener plates</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material model</td>
<td>Concrete: damaged plasticity in compression, fracture energy in tension</td>
<td>flexural reinforcement</td>
<td>Super elastic steel</td>
<td>Shearhead arm steel</td>
<td>Shearhead arm steel</td>
</tr>
</tbody>
</table>

5.1.4 Comparison between numerical and test results

5.1.4.1 Load-deformation and maximum load carrying capacity

Since the supporting steel beams were not included in the numerical model, the measured deflection was the relative deflection recorded at the slab centre to the average value recorded at the centres of the supporting steel beams, i.e. displacement difference recorded at channel 81 and channel 105, see Figure 3.16 for LVDT locations. Figure 5.6 compares the recorded and simulated load-deformation curves.

The numerical L-D curve followed the test result very well initially, indicating accurate prediction of elastic slab behaviour. Afterwards, the simulation results indicate much stiffer slab behaviour in the load range of 150KN to 350KN. This relatively large difference may have been caused by the Hillerborg model (Hillerborg
et al., 1976) not being able to exactly follow the concrete post-crack behaviour. However, near slab failure, the predicted slab deformation increased rapidly and became close to the measured result.

![Figure 5.6 Comparison between numerical and test load-deflection curves for Test 1](image)

The numerical simulation gave a slab failure load of 365KN which is around 12% lower than the test result. This difference may be caused by uncertainty in concrete material property, particularly the fracture energy value of concrete. Nevertheless, considering the complexity of numerically simulating concrete structural behaviour, the numerical results are considered good and acceptable.

### 5.1.4.2 Strain comparison

There were some uncertainties in the supporting steel beam deflection, causing the simulated and measured slab deflections to be different. However, since the
supporting beam deflection may be considered rigid movement to the slab, it is expected that the slab strain prediction would be able to achieve better accuracy.

Figures 5.7 and 5.8 compare concrete compressive and reinforcement tensile strains. The agreement is highly satisfactory. The concrete compressive strain indicates considerable non-linear behaviour and the ABAQUS model was able to capture this behaviour. The reinforcement tensile strain was much below the yield strain, suggesting that the slab flexural bending capacity was much greater, which is in agreement with the observed punching shear failure. Both the numerical simulation and test results indicate more rapid increase in steel tensile strain at around 150 KN. This corresponds to the appearance of visible cracks appearing at this load.

Figure 5.7 Comparison between numerical and measured strains: concrete in comparison for position 222mm away from the slab centre as C57 presented in Figure 3.9
Figure 5.8 Comparison between numerical and measured strain: tensile reinforcement for position 222mm away from the slab centre as C49 presented in Figure 3.9

Figure 5.9 compares numerical and experimental strains for the shearhead arms on both the compression and tension flanges, at a distance 100mm away from the outer face of the column. Both the numerical and experimental results indicate that strains on the shearhead arm are much below the yield strain of steel.

Both Figures indicate that there are some differences between the numerical and experimental results, however, the results are quite close and the patterns of experimental measurements were closely followed by the numerical results. For example, the strain on the tension flange of the shearhead arm changed to compression at around 200KN and then returned to tension after the load increased to 360KN. This pattern of behaviour also appeared in the strain on the compression flange of the shearhead arm. Both changes were closely followed by the numerical model. These changes may have been caused by concrete crushing under the
shearhead arm at around 200kN, relieving the vertical load on the shearhead arm and transferring the load to the end of the shearhead arm, as shown in Figure 5.9 (b). This would have reduced the bending moment in the shearhead arm, producing the stress reversal shown in Figures 5.8 and 5.9. From 300kN, the reaction force at the end of the shearhead arm again induced bending moment in the shearhead arm, generating the initial strain pattern.

(a) Compression flange
(b) Tension flange

Figure 5.9 Comparison of strain on the flange of shearhead arm: 100mm away from the outer side of the tubular column as C23 presented in Figure 3.9

5.1.4.3 Crack pattern

Unlike the smeared crack concrete model, the concrete damaged plasticity model does not have the notion of crack developing at the material integration point. However, it is possible to introduce the concept of an effective crack direction with the purpose of obtaining a graphical visualization of the crack pattern in the concrete structure. Different criteria can be adopted within the framework of scalar-damage plasticity for the definition of the direction of cracking. Following Lubliner model (Lubliner et al., 1989), it is assumed that crack initiates at points where the tensile equivalent plastic strain (ABAQUS parameter PEEQT) is greater than zero and the maximum principal plastic strain is positive. The direction of the vector normal to the crack plane is assumed to be parallel to the direction of the maximum principal plastic tensile strain. Using this approach, the coloured area shown in Figure 5.10 (a) indicates places
where the concrete tensile equivalent plastic strain value was greater than zero. The red line in Figure 5.10 (b) shows how the cracks developed. The initial crack developed from the end of the shearhead arm to the bottom of the slab at an angle of 45-50°. The critical area at the bottom of the slab (tensile surface) would be around 400mm away from the centre face of the column, which is close the test observation. The critical area from the numerical analysis compared with the test record matched quite well as shown in Figure 5.11.

![Crack developing pattern from numerical model for the test 1](image)

(a)                                                     (b)

Figure 5.10 Crack developing pattern from numerical model for the test 1

From the above detailed comparisons and good agreement between numerical and test results for slab load-deformation curves, strains at different locations of the structure, and development of cracks, it may be concluded that the numerical model was able to accurately simulate the first test.
5.2 MODELING TEST 2

5.2.1 Full model for the second test

Test 2 had the same slab size, thickness and shearhead arm length as Test 1. The reinforcement arrangement was also the same, except that Test 2 used continuous reinforcement bars across the column area instead of using discontinuous reinforcement in Test 1. Also a circular steel column was used in Test 2 as opposed to a square steel column in Test 1. The material properties in both tests were the same.
In Test 2, a 150mm diameter hole was introduced to the slab adjacent to the column area. This hole made the test slab unsymmetrical, necessitating modelling the full slab. Nevertheless, in order to reduce computation effort for the parametric simulation, it was considered possible to divide the slab into three quarter slabs without hole and one quarter slab with hole. The response of the entire slab may be obtained by adding the loads of these four quarter sub-models at the same slab deformation. To confirm this assumption, this section will present simulation results for the full slab model and for combined three quarters of solid slab and one quarter of slab with hole. Figure 5.12 shows the full model. The same mesh method as for Test 1 was followed for Test 2.

Figure 5.12 Full slab model for Test 2 (boundary conditions shown in Figure 5.13)

Since the full slab was modelled, the same boundary condition as in the first test was adopted. The slab was simply supported along the support beam inner edges which were 110mm away from the outset face of the slab, the bolted locations were fixed as shown in Figure 5.13.
5.2.2 Combined models for the second test

Figure 5.14 shows one of the three quarter slabs without hole and Figure 5.15 shows one quarter slab with hole. The boundary conditions for the quarter models were the same as the quarter model for Test 1 in section 5.1.2.
Figure 5.15: Quarter slab model with hole

5.2.3 Comparison between numerical and test results

5.2.3.1 Load-deformation relationship

Figure 5.16 compares the numerical and test results of slab load-deformation curves. Two sets of numerical results are given: one obtained from the full slab model and one from combining three quarters of solid slab and one quarter of slab with hole (combined model). The slab centre deflection used to compare with the numerical model was converted from the raw measured record in the second test. The deflection of the crossing beams gave around 44KN/mm for the slab centre deformation.

The two sets of numerical results are very close, clearly suggesting that the combined model is suitable as an approximation to the full slab model. The failure load of the slab was predicted with good accuracy by the numerical model. The combined model gave a slab failure load of 590KN, being only 3.8% higher than the measured result of 568KN. The full model achieved 558KN which is only 1.8% lower than the test result.
For both tests, the numerical results give slightly stiffer slab behaviour than the test results. This is possibly due to the inaccurate representation of the supporting steel beams, being assumed to be vertically immovable but may have some flexibility. Also both Figures 5.6 (for Test 1) and 5.16 (for Test 2) indicate that the slabs experienced a rapid deterioration in stiffness. The numerical simulations for both tests also predicted this phenomenon, but the numerical loads (at around 250kN in both tests) were higher than the test loads (about 150 KN for the Test 1 and 100 KN for Test 2). This may be due to the actual tensile stress of the concrete being lower than that assumed in the numerical models. The sensitivity study results (Figure 5.23) indicate that changing the concrete tensile stress can change the load at which the slab stiffness decreases rapidly. Therefore, it is possible to make changes in the concrete tensile strength to bring the numerical simulation load-deflection curves much closer to the test results than presented in Figures 5.6 (for Test 1) and 5.16 (for Test 2). However, regardless of the concrete tensile strength used, the numerical models predicted the failure loads of the slabs with good accuracy. Taken together with the observation that the
numerical models gave satisfactory prediction of the measured strains and the simulated slab failure patterns agreed very well with the experimental results, it is considered that the numerical models are acceptable.

5.2.3.2 Comparison of strains

Figure 5.17 and 5.18 compare numerical and measured strains for the compression side of the slab and reinforcement bar which is 530mm away from the centre of the slab.

Figure 5.17: Comparison of concrete compressive strain where is 530mm away from the centre of the slab as C29 shown in Figure 3.12

Figure 5.18: Comparison of strains in reinforcement bar where is 530mm away from the centre of the slab as C31 shown in Figure 3.12
Figure 5.17 shows good agreement between the numerical and test results for concrete compressive strain. The slightly stiffer response from the numerical model than the test at the beginning may have been caused by approximate treatment of interaction between the shearhead arms and the concrete core. The numerical model assumed no relative movement between the concrete core and shearhead arm, resulting in a slightly stiffer slab response. The numerical model captured the drop in concrete compression strain when the applied load reached around 400KN. This may have been caused by the concrete failure along the initial cracks. Because when the concrete failure along the initial cracks happened, the compression face (top face in the test) of the slab would have bended up a little which would have reduced the compression strain on the top compression face.

Figure 5.18 compares strains in flexural reinforcement. The numerical result is stiffer than the test result (numerical strain < test strain), most possibly due to the Hillerborg concrete tensile model being stiffer than realistic concrete, thus reducing contribution from the tensile reinforcement.

5.2.3.3 Comparison of crack development

As described for Test 1, the principal equivalent plastic tensile strain (PEEQT) may be used to determine crack direction. The Figure 5.19 plots the crack patterns both in the slab and around the hole. The red straight lines are the directions of the maximum plastic strain which should be parallel to the crack developing directions. The black straight lines give some rough sketch of crack development.
As shown in the Figure 5.19 (a), for the solid part of the slab without hole, crack developed from the end of the shearhead arms and followed an angle of around 45-50 degree until the reinforcement level. The critical parameter through the centre (through thickness) face was around 530mm away from the centre of the slab. For the part of the slab around the hole, cracks developed from the edge of the hole towards the centre face of the slab following the angle shown in Figure 5.19 (b).

(a) Strain field in quarter solid slab

(b) Strain field in quarter slab with hole

Figure 5.19 Concrete crack developments

Using the output data from Figure 5.19, the critical perimeter on the reinforcement level from the numerical model could be established as the green lines shown in
Figure 5.20, which also shows the observed test results (red line shows). Agreement between the numerical analysis and test result is very close.

5.3 SENSITIVITY STUDY

To ensure that the numerical model is robust, sensitivity studies have been carried out to assess the effects of different input values, including material properties and element effects. The material properties study included the Young’s modules, Fracture energy, tensile stress and dilation angle for the concrete. The element effect study included element size and element type studies. Test 1 was used as the basis of this sensitivity study, the model details of the first test has published in the 5.1 in Chapter 5.
5.3.1 Material properties

5.3.1.1 Young’s modules

Figure 5.21 compares simulation results using the following three concrete Young’s modulus values $2 \times 10^4$ N/mm$^2$, $2.5 \times 10^4$ N/mm$^2$ and $3 \times 10^4$ N/mm$^2$.

This comparison shows clearly that the Young’s modulus does not have much effect on the maximum load. Since it is the maximum load that is the main interest, the nominal Young’s modulus of $2.5 \times 10^4$ N/mm$^2$ can be used.

![Figure 5.21 Comparison of load-deflection curves for different Young’s modulus](image)

5.3.1.2 Fracture energy-displacement relationship

The fracture energy of nominally the same concrete can vary depending on many factors such as curing condition, aggregate size or structural member size according to the CEB-FIP model code 1990 (CEB-FIP, 1993). It is important for to understand how the numerical results would be affected by this property so as to choose an
appropriate value. Three different values of fracture energy, 90N/m, 120N/m and 150N/m, were considered with the same tension stress 2.2 N/mm². A higher fracture energy means large displacement under tension. Figure 5.22 compares the simulation results.

A nominal value of 120N/m is the recommended value and values of 90N/m and 150N/m represent ±25% change. The corresponding failure loads are ±10% of the simulated failure load of 360KN at fracture energy of 120 N/m. This suggests that the input fracture energy value will have some effect on the slab punching shear resistance, but the effect is moderate and a nominal value of 120 N/m can be used.

Figure 5.22 Effects of different fracture energy on load-deflection curves

5.3.1.3 Failure tensile stress

The nominal maximum tensile stress was 2.2MPa, which is conservative estimation according to EC 2 (CEN, 2002). But this value can increase to 3.0MPa for the compressive strength according to Table 3-1 in EC 2. Figure 5.23 compares
simulation results using these two different tensile strength values, the fracture energy value remaining at 120 N/m. Though the deflections of these two models are not very close according to the displacement difference in property definition, the slabs both failed at around 360KN. Then the slab failure load was not sensitive to concrete tensile stress when using the fracture energy model.

![Figure 5.23 Effects of maximum concrete tensile stress on load-deflection curve](image)

Figure 5.23 Effects of maximum concrete tensile stress on load-deflection curve

### 5.3.1.4 Dilation angle

Dilation angle was introduced to reflect the ratio of the volumetric strain and deviatoric strain to show the strain relations. This figure used to define the concrete damaged plasticity in ABAQUS as an important factor. Unfortunately, there was no standard value for the dilation angle from either previous researchers or the ABAQUS manual. But this value normally should be between around 20° and 50° from previous researches. The average value 35° was adopted as the reference value. Figure 5.24 compare simulation results for dilation angles of 20, 35 and 50°. The effect of using
such a large range of dilation angles is moderate, with the failure load being within 5% of the failure load obtained using a dilation angle of 35°.

![Figure 5.24 Effects of dilation angle on load-deformation behaviour](image)

5.3.2 Mesh sensitivity

5.3.2.1 Element size

In general, the smaller mesh sizes would lead to more accurate results and the mesh size should be as small as allowed by computational effort. However, this does not strictly apply to concrete due to concrete softening induced stress concentration. Under this condition, smaller elements can cause a lot of convergence problems. Therefore, the concrete mesh size should be small, but not too small. It should normally be equal or slightly bigger than the maximum aggregate size. In the test, the maximum aggregate size was around 20mm. Mesh sizes of 10mm, 20mm and 40mm were considered in the sensitivity study based on the Marzouk's test model described.
in section 4.3.2 in Chapter 4. Figure 5.25 compares the results and indicates that a mesh size of 20mm is sufficient.

![Figure 5.25: Effects of element size on load-deformation curves](image)

5.3.2.2 Element type

Either solid or solid-like shell elements may be used for the shearhead arms. However, using solid elements would consume a lot of computation time. It would also cause many convergence problems due to the complicated geometric arrangement using circular column and very thin webs in the second test specimen. Using shell elements would be a more desirable option. Figure 5.26 compares simulation results for using both solid and shell elements. The results are very close, giving confidence in using shell elements.
5.4 CONCLUSIONS

This chapter has presented comparisons between numerical and test results for two slab punching shear tests carried out by the author. Detailed comparisons for the slab failure load, deformation, strain and cracks between the test results and numerical simulations using ABAQUS. The numerical results are in good agreement with the test results.

According to those sensitivity studies based on the first test model, the numerical model in the ABAQUS shows quite reliable result with those different affecters. And also following conclusions could be drew from this study:
• The concrete Young's modulus has very small effect on punching shear capacity.

• The concrete fracture energy has the biggest influence on punching shear capacity. A nominal value of 120 N/m can be used, with fracture energy values varying 25% resulting in about 10% change in punching shear capacity.

• If the concrete fracture energy is kept the same, the concrete tensile stress of the concrete has little effect on slab failure load.

• For the shearhead arms, both shell and solid finite elements may be used, but using shell elements is much more efficient in simulation.

• The solid finite element size for concrete may be taken as 20mm.

And after this sensitivity study, those data selected for the author’s test modelling described in this Chapter for modelling study would continue used for the further parameter study in the next Chapter.
CHAPTER 6: PARAMETRIC STUDY: EFFECTS OF DIFFERENT DESIGN PARAMETERS

The new shearhead system developed in this study shows high potential for use as an effective connector between flat slab and steel tubular column. For a moderate fabrication cost, the shearhead system enables two attractive structural systems, flat slab floor and steel tube column, to be used. Unfortunately, due to financial and time constraints, the author was only able to carry out two tests on this new shearhead system and these two tests have given sufficient direction on possible load carrying mechanisms in this system. Nevertheless, thorough understanding of the effects of different design parameters is essential to enable development of an efficient design method to realize the potential of this new shearhead system. This will be done through an extensive numerical study using ABAQUS whose application to this problem has been validated, as detailed in Chapters 4 and 5. The purpose of this chapter is to use the validated ABAQUS model to investigate the effects of different design parameters on structural behaviour of this new shearhead system.

Figure 6.1 Shearhead system under consideration
Figure 6.1 shows the shearhead system under consideration and this parameter study will include the following design parameters: column size, shearhead arm length, shearhead arm cross-section dimensions, shearhead arm end angle ($\alpha$), amount of flexural reinforcement, slab thickness and shearhead arm position in the slab thickness direction.

(a) Sketch of the reference slab

(b) Shearhead system details in reference model

Figure 6.2: Model details for the reference case
Figure 6.2 shows details of the reference case based on which the parametric studies were carried out by changing a reference value to other values. A concentrated load was applied through the reference point which was coupled with the top column face. Except where mentioned, the numerical model was based on one quarter slab as described in section 5.2.1 of Chapter 5. The results presented are for the quarter slab, not the whole slab.

6.1 INTRODUCTION TO LOAD CARRYING MECHANISM

The aim of this parametric study is to obtain extensive numerical modelling results to help develop a design calculation method. This requires a clear understanding of the load carrying mechanism in the shearhead system so that the numerical study results have clear objectives: to check this load carrying mechanism and to help determine appropriate values to be used in design calculations.

It is assumed that the following load carrying mechanism occurs in the new shearhead system:

(1) The new shearhead system acts as an enlarged column to resist punching shear. The enlarged column size equals to the original column size plus the shearhead arm length. The enlarged column size will be referred to as the shearhead size. This is sketched in Figure 6.3.

(2) To satisfy the assumption in (1), the shearhead arm should be effective. The shearhead arm becomes ineffective when it loses its load carrying capacity. This
happens when loading on the shearhead arm exceeds load carrying capacity of the shearhead arm cross-section.

(3) Isolating the shearhead system from the structure, it is assumed that the applied load through the column is entirely resisted by the shearhead system. An important value to be determined is how the applied load is distributed along the shearhead arms.

![Figure 6.3 Enlarged column area](image)

6.2 EFFECTS OF CHANGING COLUMN DIMENSIONS

This study covered changes in column shape (circular or square) and cross-sectional dimensions.

6.2.1 Effects of column shape: square or circular

The two column sizes were Square Hollow Section (SHS) 200×200×10 and Circular Hollow Section (CHS) 219.1×10. As presented in Figure 6.4, using SHS column
produced 2.4% difference in slab punching shear capacity from using CHS column due to a slightly larger critical perimeter provided by the CHS column. This effect is quite small overall.

Figure 6.4 compares the load-deformation relationships for using the two different columns. The solid quarter model of the second test specimen was adopted as the reference model here, but the column size was changed in this study.

In both cases, the load-deformation relationships were very close and the slab failure load was around 155KN. This result suggests that with introduction of the shearhead system, the original column shape has little effect. In the new arrangement, the applied load from the column is transmitted through the shearhead system, therefore, the slab failure load is mainly decided by the enlarged shearhead area instead of the column area.

![Figure 6.4 Effects of different column shapes on slab load-deformation curve](image)

Figure 6.4 Effects of different column shapes on slab load-deformation curve
6.2.2 Effects of varying column cross-sectional dimensions

Based on the assumption that the shearhead system acts as an enlarged column size, it is expected that the slab punching shear capacity should not change regardless of the original column size, provided that the shearhead arm is fully effective and the enlarged column size is the same. To confirm this, a series of models with different column sizes, but maintaining the overall enlarged column size by adjusting the shearhead arm length, were investigated. Table 6-1 lists the simulation cases and their failure loads. CHS 274x10 and CHS 219x10 were used. The total shearhead dimension (full shearhead arm length through the slab in one direction) remained the same at 480mm, 600mm and 960mm. Table 6-1 also summarises the slab failure loads.

Table 6-1 Effects of column size on slab punching shear resistance

<table>
<thead>
<tr>
<th>Column section (mm× mm)</th>
<th>CHS 274x10</th>
<th>CHS 219x10</th>
<th>CHS 274x10</th>
<th>CHS 219x10</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shearhead arm length (mm)</td>
<td>480</td>
<td>600</td>
<td>960</td>
<td></td>
</tr>
<tr>
<td>slab punching shear resistance (KN)</td>
<td>158</td>
<td>157</td>
<td>173</td>
<td>169</td>
</tr>
<tr>
<td>Difference between using different column sizes ($= \frac{F_{274} - F_{219.1}}{F_{219.1}}$)</td>
<td>0.6%</td>
<td>2.3%</td>
<td>11.2%</td>
<td></td>
</tr>
</tbody>
</table>

From the results shown in Table 6-1, the slab failure loads for total shearhead length of 480mm and 600mm were very close to each other, the difference being 0.6% and
2.3% respectively, for using the two different column sizes with over 20% difference in the original column size. The effect of using different column sizes was bigger at about 11% when the total shearhead length was 960mm. This happened because the shearhead arm length exceeded its effective length when combined with the smaller column.

Figure 6.5 compares the load-deformation curves for the six cases listed in Table 6-1. Except for the total shearhead length of 960mm, the results for the other two shearhead lengths (480mm and 600mm) with varied column sizes (CHS 274 x10 and CHS 219 x10) are very close. The slopes of the Load-Deformation curves are very similar for the 480mm and 600mm shearhead arms with different column sizes. The 960mm sheararm with column size CHS 219 x10 has very similar Load-Deformation curve with that of the slab with 600mm sheararm. This suggests that for the shearhead section used, an arm length of 600mm represents the limit of effectiveness.

![Figure 6.5 Effects of column size on slab load-deformation relationships](image-url)
These results confirm the conclusion from the column shape study: the shearhead system behaved like a bigger column. Provided the bigger column size is the same and the shearhead arms are effective, the shape and size of the original column has little effect on slab punching shear resistance. Increasing the column size would not really affect the slab punching resistance unless it is used to enable the shearhead arm length to stay within its effective length. Such was the case when the total shearhead length was 960mm. When using the smaller column size (CHS 219x10), the shearhead arm length exceeded its effective length (i.e. the shearhead arm exceeded its cross-sectional resistance). When using the larger column size (CHS 273x10), the shearhead arm length was reduced, which allowed it to stay within its effective length.

### 6.3 EFFECTS OF SHEARHEAD ARM DESIGN

From the study presented above, the effective shearhead arm length plays a significant role in deciding the punching shear capacity of this new shearhead system. It is important to determine a method to calculate the effective length of the shearhead arm. The effective shearhead arm length is the length above which the shearhead cross-section’s load carrying capacity is exceeded and the shearhead becomes ineffective in transferring the applied column load to the slab. The shearhead arm may be treated as a cantilever beam. To evaluate its load carrying capacity, it is necessary to examine the reaction force on the shearhead arm. Shearhead arm design parameters include shearhead arm length, cross-sectional dimensions and angle of cut.
6.3.1 Effects of shearhead arm length

Table 6-2 lists the cases investigated. The shearhead arm length varied from 320mm to 960mm. Detailed slab load-deflection curves are shown in Figure 6.6 and Table 6-2 summarises the slab failure loads.

Table 6-2 Effects of shearhead arm length

<table>
<thead>
<tr>
<th>Shearhead arm length (mm)</th>
<th>320</th>
<th>400</th>
<th>480</th>
<th>600</th>
<th>780</th>
<th>960</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column section</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shearhead arm cross-section and end condition</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Failure load (KN)</td>
<td>124</td>
<td>150</td>
<td>157</td>
<td>169</td>
<td>164</td>
<td>161</td>
</tr>
</tbody>
</table>

Figure 6.6 Effects of shearhead arm length on slab load-deformation relationship
The results in Table 6-2 show that the slab failure loads increased with increasing shearhead arm length until the shearhead arm length reached 600mm, after which further increase in shearhead arm length resulted in small decreasing slab load carrying capacity.

This may be explained by the two different failure modes with changing shearhead arm length. When the shearhead arm length is shorter than its effective length, the shearhead arm is entirely effective. Failure of the slab was due to the slab reaching its full punching shear resistance. This is demonstrated by the failure pattern as indicated in Figure 6.7(a). Under this condition, the main crack developed from the end of the shearhead arm to the slab tension side as indicated by the red dash line. This mode of behaviour happened when the shearhead arm was within the effective length (400mm, 480mm and 600mm in this study). When the shearhead arm length was longer than the effective length, failure occurred in the shearhead arm area as shown by the red dash line in Figure 6.7 (b). Although the shearhead arm length increased (780mm and 960mm in this study), the critical perimeter did not increase and the main crack did not start from the end of the sheararm.

In order to find out a way to determine the effective length of shearhead arm, the load (pressure) distribution underneath the shearhead arm was investigated. Figure 6.8 presents pressure distribution and its history at different slab loading (Unit is KN for the values) for two shearhead arm lengths. Initially, the pressure distribution was almost uniform, suggesting near rigid behaviour of the shearhead system. The pressure distribution is similar to that assumed in ACI318 (ACI, 2005). When
approaching slab failure, the pressure distribution became concentrated towards the end of the shearhead arm.

(a) Shearhead arm fully effective before slab punching shear failure
(b) Shearhead arm failure before slab reaching its punching shear resistance

Figure 6.7 Effects of shearhead length on failure mode

In Chapter 7, an analytical method will be presented, based on the conclusions of this study, to check load carrying capacity of the shearhead arm to ensure that its effective length is not exceeded so that the slab can develop its full punching shear resistance.
(a) Shearhead arm length = 480mm

(b) Shearhead arm length = 600mm

Figure 6.8 Pressure distribution under shearhead
6.3.2 Effects of shearhead arm cross-section

According to the assumption of shearhead arm failure mode, the slab punching shear resistance is limited by the shearhead arm cross-section properties. To explore this, the shearhead arm cross-section was varied to give different plastic bending moment and shear resistances.

Table 6-3 Effects of shearhead arm cross-section

<table>
<thead>
<tr>
<th>Shearhead arm cross-section</th>
<th>A: RHS 120×60×3.6</th>
<th>B: RHS 120×60×2.4</th>
<th>C: RHS 120×60 with 3.6mm web and 1.8mm flange</th>
<th>D: I-section 120×60×3.6</th>
<th>E: I-section 120×60×4.8</th>
<th>F: RHS 120×60 with 4.8mm web and 0.1mm flange</th>
<th>G: RHS 120×60×3.6</th>
<th>H: RHS 120×60×6.3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plastic moment of section (cm³)</td>
<td>48.0</td>
<td>32.9</td>
<td>36.6</td>
<td>36.6</td>
<td>47.8</td>
<td>33.8</td>
<td>48.0</td>
<td>79.3</td>
</tr>
<tr>
<td>Shear capacity (KN)</td>
<td>179</td>
<td>119</td>
<td>90</td>
<td>90</td>
<td>119</td>
<td>5</td>
<td>179</td>
<td>313</td>
</tr>
<tr>
<td>Shearhead arm length (mm)</td>
<td></td>
<td>600</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>960</td>
<td></td>
</tr>
<tr>
<td>Slab failure load (KN)</td>
<td>169</td>
<td>140</td>
<td>151</td>
<td>152</td>
<td>165</td>
<td>86</td>
<td>161</td>
<td>187</td>
</tr>
</tbody>
</table>

Table 6-3 lists the detailed parametric study cases and summarises the slab failure loads. Shearhead arms using both rectangular tube sections and I-sections were considered. Some of the dimensions of these shearhead arm cross-sections are not
realistic and they were artificially changed to enable different ratios of shearhead arm cross-sectional bending and shear resistance to be investigated.

It is worth pointing out the rationale behind the above numerical simulations.

**Cases A and E:**

Case A used a rectangular hollow section for the shearhead arm and Case E used an I section. Both sections have similar plastic moment resistance (plastic modulus = 48 cm$^3$), but Case E had only two thirds of the shear capacity of Case A. Their slab failure loads and modes were close to each other because the shearhead arm behaviour was governed by its bending moment capacity, not its shear resistance.

**Cases B and E:**

Case B had the same shearhead arm shear resistance with Case E, but only 70% of its bending moment resistance capacity. Again, shearhead arm behaviour was governed by bending, so Case B reached a slab failure about 18% lower than in Case E.

**Cases B and F:**

Case F is an extreme theoretical case with a similar bending resistance as Case B but almost zero shear resistance. Unsurprisingly, model F failed very early because the shearhead arm was ineffective.

**Cases C and D:**

Cases C and D had the same shearhead arm cross-section bending and shear resistance but with different cross sections. Their failure loads were almost identical. This result proved again that the shape of shearhead arm cross-section was not a design factor.
Cases G and H

The shearhead arm bending moment and shear capacities for Case H were 65% and 75% higher than Case G. Consequently, the effective shearhead arm in Case H was longer than in Case G. This resulted in the Case H failure load being 16% higher than in Case G.

From these study results investigating the effects of changing shearhead arm shape and cross-sectional dimension, it is clear that the shape of the shearhead arm cross-section has little effect. The bending moment and shear resistance of the shearhead arm cross-section are the influential effecters which affect the slab resistance.

6.3.3 Effects of shearhead arm end angle

The results of pressure distribution underneath shearhead arm (shown in Figure 6.8) indicate that towards slab failure, pressure was concentrated at the end of the shearhead arm. Therefore, the end angle (the angle between the cut line and top compression face of the sheararm as shown in Figure 6.9) is expected to have important influence on the slab punching shear resistance.

Figure 6.9 End angle of sheararm
In all the studies reported above, the end angle was \(45^\circ\) which is the same as the author’s Test 2. The ACI code (ACI, 2005) requires that the end angle should not be less than \(30^\circ\) to enable the shearhead arm to be efficient. Table 6-4 lists the shearhead arm end angle investigated in the numerical parametric study and summarises the slab failure loads.

Table 6-4 Effects of shearhead arm end angle

<table>
<thead>
<tr>
<th>End Angle (degree)</th>
<th>25</th>
<th>30</th>
<th>37</th>
<th>45</th>
<th>52</th>
<th>60</th>
<th>75</th>
<th>90</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shearhead arm details</td>
<td>600mm RHS 120×60×3.6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Column</td>
<td>CHS 219.1×10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Slab failure load (KN)</td>
<td>75</td>
<td>133</td>
<td>139</td>
<td>169</td>
<td>141</td>
<td>139</td>
<td>138</td>
<td>115</td>
</tr>
</tbody>
</table>

The results in Table 6-4 indicate clear importance of the shearhead arm end angle, with an angle of \(45^\circ\) producing the highest slab failure load at 157KN which doubles the lowest slab failure load of 75 KN when the end angle was \(25^\circ\).

Figure 6.10 is the load-deflection relationships at the centre point of the slab for the sheararms with different end angles. The effect of changing shearhead arm end angle on slab capacity may be explained by the different failure modes.

Two failure modes may be identified. When the shearhead arm end angle is high (45 to \(90^\circ\)), failure of the shearhead arm occurs at the connection to the column. Under this circumstance, since the pressure is mainly underneath the inclined surface at the end of the shearhead arm, a smaller end angle gives a longer length for the distributed pressure, causing lower bending moment to the shearhead arm connection. Therefore
the slab load carrying capacity decreased when the end angle changed from the $45^\circ$ to $90^\circ$.

Figure 6.10 Effects of shearhead arm end angle on slab load-deformation relationship

*D=Degree

Figure 6.11 Comparison between bending moment capacity and applied bending moment for different shearhead arm end angles
When the end angle is small (less than 45°), failure may occur near the sharp tip of the shearhead arm due to its much reduced depth and bending resistance.

To confirm this, Figure 6.11 compares the shearhead arm bending moment resistance ($M_p$) along its length with the applied bending moment ($M_1$) on the shearhead arm calculated using the proposed shearhead arm pressure distribution. In the Figure, the full shearhead arm cross-section capacity is indicated by a flat value of $M_p$. In all cases, the shearhead arm had the same length. When the end angle was 30°, the shearhead arm reached its bending moment capacity in the inclined section. Interestingly, when the end angle was 45° and 60°, the shearhead arm failed at similar locations on the flat area even though the applied loads were quite different as shown in Table 6-4.

To summarise, when designing a shearhead arm, the end angle should be carefully considered. The optimum angle is 45°.

6.3.4 Effects of shearhead continuity

The ACI code (ACI, 2005), which is the only code that includes shearhead connection, requires that continuous flexural reinforcement and shearhead be used across the column area. However, the ACI code was for using with reinforced concrete columns. The proposed new shearhead system of this research is for use with steel tubular columns. Insistence on using continuous reinforcement and shearhead may lose some of the advantages of this system because meeting this ACI code requirement may
reduce flexibility of the system. The parametric study in this section was conducted to investigate whether continuous reinforcement and shearhead was necessary. Table 6-5 lists details of this parametric investigation. Figure 6.12 shows arrangements for continuous and discontinuous shearhead systems near the column area. Other conditions were the same as the reference case introduced in section 5.2.2 in Chapter 5.

Table 6-5 Effects of shearhead continuity

<table>
<thead>
<tr>
<th>Shearhead arm length (mm)</th>
<th>480</th>
<th>480</th>
<th>600</th>
<th>600</th>
<th>780</th>
<th>780</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shearhead condition</td>
<td>C*</td>
<td>DisC**</td>
<td>C</td>
<td>DisC</td>
<td>C</td>
<td>DisC</td>
</tr>
<tr>
<td>Slab capacity (KN)</td>
<td>157</td>
<td>103</td>
<td>169</td>
<td>143</td>
<td>164</td>
<td>149</td>
</tr>
<tr>
<td>Difference between C* and DisC**</td>
<td>1.52</td>
<td>1.18</td>
<td>1.10</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*: Continuous shearhead; **: Discontinuous shearhead

![Continuous shearhead](image1)

![Discontinuous shearhead](image2)

Figure 6.12: Continuous and Discontinuous shearheads
Table 6-5 suggests differences in slab punching shear resistance depending on the continuity of the shearhead system, with the continuous arrangement giving higher values. However, as the shearhead arm length increased, the difference in slab punching shear resistance reduced. For example, when the shearhead arm length was 780mm, the difference was only about 10%. For the shortest shearhead arm length (480mm), the difference was quite high, at around 50%. The reason was that for the short discontinuous shearhead arm, there was only 10mm distance on each side of the column, thus allowing the column to punch through the slab without much engagement with the wider slab, which is shown in Figure 6.13. When the shearhead arm was continuous, there was much better contact between the shearhead and the slab. When the shearhead arm was longer, there was sufficient contact between the shearhead arm and the slab, thus alleviating the adverse effect of the discontinuous shearhead arrangement.

![Figure 6.13: Short discontinuous shearhead arm: showing column punching through the slab with little contribution from the shearhead](image)

If the steel tubular column is continuous, then it is not possible for the longitudinal reinforcement to be continuous through the column, as was the case in the author’s Test 1. The effects of discontinuous reinforcement were investigated. Figure 6.14
shows four reinforcement arrangements. In all cases, the shearhead was discontinuous and the shearhead length was 300mm. All the four reinforcement cases had similar reinforcement ratio ($\rho$) but with different reinforcement positions near the shearhead. Figure 6.15 compares the slab load-deformation curves. The slab behaviour in all cases was similar. The result indicates that the punching shear performance of a flat slab is only slightly affected by detailed longitudinal reinforcement arrangement around the shearhead system.

![Figure 6.14](image)

(a) case 0, $\rho = 0.41\%$

(b) case 1, $\rho = 0.44\%$

(c) case 2, $\rho = 0.44\%$

(d) case 3, $\rho = 0.44\%$

Figure 6.14 Different reinforcement arrangement for discontinued shearhead system
6.3.5 Section summary

For design method development (Chapter 7), the following conclusions from the study in this section on shearhead arm parameters should be considered:

(1) The shearhead arm cross-section shape has little effect on slab behaviour;
(2) The mode of shearhead arm failure should be checked by its bending and shear resistances;
(3) The shearhead arm end angle has some effects on slab load carrying capacity. An angle of 45° appears to be the most optimum, producing the highest slab load carrying capacity;
(4) If the shearhead arms are not continuous through the column, the slab will suffer some reduction in load carrying capacity. However, if there is sufficiently long
shearhead arm length, the reduction in slab load carrying capacity due to discontinuous shearhead is moderate;

(5) To check whether the shearhead arm is effective, it is conservative to assume that the pressure underneath the shearhead arms is concentrated at the end of the shearhead arm, along the inclined surface.

6.4 EFFECTS OF SLAB THICKNESS

In reinforced flat slab construction using reinforced columns, the effective depth of the slab is defined as the height from the concrete compression face to the tension reinforcement level. For the proposed new shearhead system, cracks in concrete initiate from the tips of the shearhead arms and the effective depth of the slab should be measured from this position. To confirm this, numerical models with variable slab thickness and changing positions of shearhead arms were carried out.

6.4.1 Effects of slab thickness

Table 6-6 lists the cases investigated and their associated dimensions. The slab thickness changed from 168mm to 250mm and the shearhead arm length was 240mm and 300mm. The longitudinal reinforcement ratio was kept the same at 0.41%. The shearhead arm cross-section of SHS 120×60×4.8 was replaced with SHS 120×60×3.6 when the slab thickness was 250mm.
From Table 6-6, it is clear that increasing the slab thickness produced higher slab load carrying capacity. With the same slab thickness, using a smaller shearhead arm (but keeping the length) cross-section (RHS 120x60x3.6) resulted in lower slab load carrying capacity than using a bigger shearhead arm cross-section (RHS 120x60x4.8) when the slab thickness was 250mm. This was caused by the smaller shearhead arm cross-section not being effective to allow the full punching shear resistance of the slab to be developed. This can be shown in Figure 6.16, which compares positions of the diagonal cracks for slab thickness of 250mm and shearhead arm length of both 480mm and 600mm. When the smaller shearhead arm cross-section (RHS 120x60x3.6) was used (Figure 6.16(b)), the diagonal crack initiated at a level below the top of the shearhead arm. When the bigger shearhead arm cross-section (RHS 120x60x4.8) was used (Figure 6.16(c)), the diagonal crack initiated from the tip of the shearhead arm, indicating full effectiveness of this shearhead. When a thinner slab (168mm) was used (Figure 6.16(a)), because the punching shear resistance of the slab was low, the smaller shearhead arm cross-section (RHS 120x60x3.6) was sufficient to allow full development of the slab punching shear resistance and the diagonal shear.
crack initiated from the tip of the shearhead arm. The diagonal crack initiated at a level below the top of the shearhead arm also happened with longer sheararm length (600mm) with enhanced sheararm section (RHS 120x60x4.8) as presented in Figure 6.16(d).

(a) slab thickness =168mm, shearhead arm length =480mm, cross-section size=RHS 120x60x3.6

(b) slab thickness =250mm, shearhead arm length =480mm, cross-section size=RHS 120x60x3.6
(c) slab thickness = 168mm, Shearhead arm length = 480mm, cross-section size = RHS 120×60×4.8

(d) slab thickness = 250mm, Shearhead arm length = 600mm, cross-section size = RHS 120×60×4.8

Figure 6.16: Failure modes for different slab thicknesses and shearhead arm sizes

6.4.2 Position of reinforcement

The position of the longitudinal reinforcement determines the effective depth of the slab for punching shear calculation. Clearly this is an important factor. Table 6-7 presents the results of a study to investigate the effects of reinforcement position on slab punching shear resistance. The reinforcement was either directly underneath the shearhead system or had a cover of 20mm.
Table 6-7 indicates significant effect of reinforcement position on slab punching shear resistance. It is recommended that the distance from the concrete tension surface to the longitudinal reinforcement level be kept the minimum in order for the slab to achieve its maximum punching shear resistance.

Table 6-7: Effects of reinforcement position

<table>
<thead>
<tr>
<th>Slab thickness (mm)</th>
<th>200</th>
<th>250</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shearhead arm Details</td>
<td>Length 480mm, cross-section RHS 120×60×3.6, flush with top concrete compression face</td>
<td></td>
</tr>
<tr>
<td>Position of tension flexural reinforcement</td>
<td>Just Underneath shearhead</td>
<td>With 20mm Cover from slab tension face</td>
</tr>
<tr>
<td>Slab effective depth D* (mm)</td>
<td>132</td>
<td>168</td>
</tr>
<tr>
<td>Slab resistance (KN)</td>
<td>106</td>
<td>157</td>
</tr>
</tbody>
</table>

*: Defined as the height from top concrete compression face to reinforcement level

6.4.3 Effects of shearhead arm position

Since the diagonal shear crack initiates from the tip of the shearhead arm, the effective slab thickness is calculated from this position to the level of tensile reinforcement. Therefore, it is expected that by keeping all design parameters the same, changing the position of the shearhead arm in the slab thickness direction will affect the slab punching shear resistance. Table 6-8 lists the numerical cases used to investigate this effect.
The shearhead arm either sat on top of the tensile reinforcement or had its top flush with the concrete compression face. Figure 6.17 presents the slab load-deformation relationships and slab load carrying capacities are summarized in Table 6-8.

Table 6-8 Effects of shearhead arm position

<table>
<thead>
<tr>
<th>Case ID</th>
<th>480-L Arm*</th>
<th>480-H Arm**</th>
<th>600-L Arm</th>
<th>600-H Arm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slab thickness (mm)</td>
<td></td>
<td>200</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shearhead arm Details</td>
<td>Length = 240mm, cross-section= RHS 120×60×3.6</td>
<td>Length = 300mm, cross-section= RHS 120×60×3.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Column section</td>
<td>CHS 219.1×10</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Slab effective depth D* (mm)</td>
<td>132</td>
<td>168</td>
<td>132</td>
<td>168</td>
</tr>
<tr>
<td>Slab capacity (KN)</td>
<td>107</td>
<td>157</td>
<td>127</td>
<td>169</td>
</tr>
</tbody>
</table>

*: shearhead arm on top of tensile reinforcement

**: shearhead top flush with concrete compression face

D*: distance from tip of shearhead arm to position of tensile reinforcement

Figure 6.17 Effects of shearhead arm position on slab load-deformation relationships
By moving the shearhead arm from sitting on top of tensile reinforcement to its top surface being flush with concrete compression surface, the slab punching shear resistance was increased by 30-40%. This clearly suggests that it is not appropriate to define the slab effective depth as the distance from the concrete compression surface to the tensile reinforcement, which was the same in both cases of shearhead arm positions. Instead, the slab effective depth should be measured from the tip of the
shearhead arm. This is confirmed in Figure 6.18 which indicates that the diagonal shear cracks started from the tip of the shearhead arm.

**6.5 EFFECTS OF TENSILE REINFORCEMENT**

Dowel-force contribution from flexural reinforcement was considered important to slab punching shear resistance (Menetrey, 2002) However, the effect of tensile reinforcement ratio is included in EC2 (CEN, 2002) and BS8110 (BSI, 1997) but not ACI 318 (ACI, 2005). An independent study was carried out in this research to investigate contributions of dowel force. Table 6-9 lists the numerical study cases, based on the author’s Test 2.

Table 6-9 Effects of tensile reinforcement ratio

<table>
<thead>
<tr>
<th>Slab thickness (mm)</th>
<th>200</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shearhead arm Details</td>
<td>Length = 320mm, cross-section= RHS 120×60×3.6</td>
</tr>
<tr>
<td>Column section</td>
<td>CHS 219.1×10</td>
</tr>
<tr>
<td>Reinforcement ratio (%)</td>
<td>0.41</td>
</tr>
<tr>
<td>Slab capacity (KN)</td>
<td>124</td>
</tr>
</tbody>
</table>

Table 6-9 clearly shows that tensile reinforcement ratio had considerable effect on slab punching shear resistance.
6.6 EFFECTS OF SERVICE HOLE POSITION

Figure 6.19 shows the case studies to investigate the effects of changing the hole position on punching shear resistance of slab. In all cases, the hole diameter was 150mm. And other details of the model were same to the quarter model described in the section 5.2.2 in Chapter 5.

![Figure 6.19: Single service hole position](image)

Figure 6.20 compares the slab load-deformation results. The slab stiffness was slightly affected by the hole position if the hole was within the critical perimeter. The reduction in slab stiffness was significant when the hole was outside the critical parameter (distance of hole from centre of slab I=500mm).
Figure 6.20 Effects of service hole position on slab load-deformation relationships

The hole position affects the slab critical perimeter and hence the slab punching shear capacity. Figure 6.21 compares slab critical perimeters for the four cases, the critical
perimeter lengths were 265mm (I=190mm), 310mm (I=280mm), 306mm (I=340mm) and 255mm (I=500mm) for the four hole positions. The initial cracks were developed from around 400mm away of the centre slab face to the edge of the holes in most of the cases except for the case when the hole was totally outside the enlarged column area as shown in Figure 6.3 (I=500mm).

The punching resistance capacity of the slab is not been affected a lot by the hole positions if the hole within the enlarged column area in this study. The slab with hole exist shows good punching resistance here compared with the slab part without hole (157KN). The flat slab with shearhead system could approach the prediction of the punching resistance of the flat slab with hole in current codes easily. And the flexural capacity of the slab should be check if the hole exist outside of the enlarged column area also mentioned in this study.

For a conservative consideration, the punching resistance of the slab part with hole would follow the existing code design recommendations (Appendix D).

6. 7 CONCLUSIONS

In the proposed new shearhead connector between flat slab and steel tubular column, slab punching shear capacity is affected by a number of design parameters. For development of a design method, the following assumptions may be made:

- Neither the column size nor shape has any effect;
- The slab effective depth should be measured from the tip of the shearhead arm
to tensile reinforcement;

- The dowel effect of tensile reinforcement should be included when calculating slab punching shear resistance;
- Bending and shear failure modes of the shearhead arm should be checked;
- The end angle of shearhead arm is an important factor and the optimum angle is $45^\circ$;
- Conservatively, pressure underneath shearhead arm may be assumed to be concentrated under the inclined surface;
- The critical perimeter that within the hole extend area should not accounted as the EC2 (CEN, 2002) and BS8110 (BSI, 1997) described excepted the hole totally located outside of the enlarged column area as described in Figure 2.20 in the Chapter 2.

Chapter 7 will present a design method based on the above assumptions and compare slab punching shear resistance predictions of this design method with the numerical results.
CHAPTER 7: DEVELOPMENT OF A DESIGN METHOD

The parametric study reported in Chapter 6 led to the establishment of a number of assumptions on load carrying mechanism in the proposed new shearhead system. This chapter will develop a design method based on these assumptions. The new design method will be checked by comparing predictions of slab load carrying capacity against results of the parametric study reported in Chapter 6.

7.1 ENLARGED COLUMN ASSUMPTION

One main assumption is that the shearhead system behaves as an enlarged column in the normal flat slab structures. In the normal flat slab structure with reinforced concrete column, the critical perimeter for calculating slab punching shear resistance is obtained by extending a shear crack from the edge of the column on the compression side of the slab to tensile reinforcement, as shown in Figure 7.1(a) (Menetrey, 2002). Both from the physical test result in Chapter 3 and parametric study results in Chapter 6, in the new shearhead system, the shear crack extends from the tip of the shearhead arm to tensile reinforcement as shown in Figure 7.1(b). Therefore, if the shearhead arms are fully effective, the enlarged column size equals to the original column side plus the total length of the shearhead arms in that direction. The enlarged column is rectangular as shown in Figure 6.3 in Chapter 6, regardless of the shape of the original column.
Using this enlarged column, the punching shear capacity of the slab can be calculated using any of the conventional code design method such as EC2 (CEN, 2002) and BS8110 (BSI, 1997) even though neither considers shearhead.

Using the above assumption, a comparison was made between the recorded slab punching shear capacity and the modified design code values for the author’s two tests reported in Chapter 3. Table 7-1 summarises the calculation results, comparing both the critical perimeter length and the slab punching shear capacity. When using design EC2 and BS8110 design codes, the enlarged column was used and the enlarged column size was a square of 480mm. ACI 318 (ACI, 2005) was the only design code that explicitly treats shearhead. According to the results in Chapter 6, the slab effective depth was taken from the top of the compression face of the shearhead to the tensile reinforcement. This is shown in Figure 7.2. This gave effective depth for Test 1 of 126mm, and for Test 2 of 168mm. Figure 7.3 compares the measured and calculated critical perimeters for the two tests. Appendix D gives details of the design calculations.
From the comparisons in Figure 7.3 and Table 7-1, the critical perimeter lengths of both tests were predicted using EC2 (CEN, 2002) and BS8110 (BSI, 1997). Even though ACI318 was the only code that explicitly considered shearhead system, its prediction of the critical perimeter length for the two tests was only about $1/3^{\text{rd}}$ of the measured value. Figure 7.4 further provides details of comparison between BS 8110 and EC2 calculations of critical perimeter for Test 2, the test critical perimeter being determined based on information obtained after cutting the slab thickness as explained in section 3.4.2.4 in Chapter 3.

Table 7-1 indicates that using the three design codes gave calculated slab punching shear capacity between around 72% and 93% of the recorded slab punching shear resistance, with BS8110 giving the most accurate values. Even though ACI 318 gave grossly inaccurate prediction of critical perimeter length, its calculation of slab punching shear resistance achieved less accuracy compared with the other two design codes. Overall, BS8110 seems the most reliable, for both critical perimeter length and punching shear resistance.
(a) Test 1

(b) Test 2

Figure 7.3: Comparison for critical perimeters between measurement and design code calculations
Table 7-1 Comparisons between test and design code calculations based on enlarged column for the author’s tests

<table>
<thead>
<tr>
<th>Method</th>
<th>Critical perimeter length (mm)</th>
<th>Design shear stress of concrete (N/mm²)</th>
<th>Slab punching shear capacity (KN)</th>
<th>$\frac{V_{cal}}{V_{test}}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Test 1</strong></td>
<td>Measured</td>
<td>3500</td>
<td>$V_{test}=416.8$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>EC 2</td>
<td>3503</td>
<td>0.88</td>
<td>$V_{cal}=386.7$</td>
</tr>
<tr>
<td></td>
<td>BS 8110</td>
<td>3432</td>
<td>0.90</td>
<td>$V_{cal}=389.3$</td>
</tr>
<tr>
<td></td>
<td>ACI 318</td>
<td>1347</td>
<td></td>
<td>$V_{cal}=331.4$</td>
</tr>
<tr>
<td><strong>Test 2</strong></td>
<td>Measured</td>
<td>3305</td>
<td>$V_{test}=568.2$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>EC 2</td>
<td>3662</td>
<td>0.88</td>
<td>$V_{cal}=541.4$</td>
</tr>
<tr>
<td></td>
<td>BS 8110</td>
<td>3503</td>
<td>0.90</td>
<td>$V_{cal}=529.6$</td>
</tr>
<tr>
<td></td>
<td>ACI 318</td>
<td>1254</td>
<td></td>
<td>$V_{cal}=411.3$</td>
</tr>
</tbody>
</table>
Figure 7.4 Comparison of critical perimeters for Test 1 and Test 2
7.2 DESIGN CHECKS FOR SHEARHEAD ARM

The design method assumes that the shearhead system acts as an enlarged column. This is based on the condition that the shearhead arm does not fail before the slab reaches its punching shear resistance. This section will present a method to check shearhead arm.

7.2.1 Loading condition under shearhead arm

To check the shearhead arm, it is assumed that the shearhead arm acts as a laterally and torsionally restrained cantilever. Shearhead arm fails when its bending moment resistance, shear resistance or combined bending moment and shear resistance is exceeded anywhere within its length. It is necessary to determine the shearhead arm loading condition.

Figure 7.5 Assumed load distribution under shearhead arm
Results from the parametric study in Chapter 6 (section 6.3.1) suggest that most of the load from the column was transmitted to the concrete slab through bearing under the inclined surface area of the shearhead arm when approaching slab failure. As a conservative assumption, the applied column load is uniformly distributed under the inclined surface area of the shearhead arm. This is shown in Figure 7.5.

If the end angle, $\alpha$ in Figure 7.5, is around the optimum value of 45º, failure of the shearhead arm is at the shearhead arm to column junction.

Based on the idealized pressure distribution shown in Figure 7.5, the maximum shearhead arm length can be obtained from the following equations.

**Bending capacity**

For the case of the shorter (lower) side of the shearhead arm extending beyond the column:

$$M_p \geq \frac{V_u}{\eta} (l_v - c_1 \frac{h_v}{2} - \frac{h_v}{2 \tan \alpha})$$

Where: $M_p$ is the bending moment resistance of the shearhead arm.

For the case of the shorter (lower) side of the shearhead arm not extending beyond the column:

$$M_p \geq \frac{V_u}{\eta} \left( \frac{2l_v - c_1}{4} \right)$$

**Shear capacity**

$$V_{pl} \geq V_u$$
Where $V_{pl}$ is the shear resistance of the shearhead arm cross-section.

In contrast, ACI 318 assumes the following pressure distribution underneath the shearhead arm as in Figure 7.6:

Accordingly, the maximum shearhead arm length is determined by:

$$M_p = \frac{V_u}{2\phi \eta} \left[ h_v + \alpha_v \left( l_v - \frac{c_l}{2} \right) \right]$$

However, as explained in section 6.2.1 in Chapter 6, this pressure distribution describes that at the beginning of loading and the pressure distribution approaching slab failure should be used when calculating slab resistance.
7.2.2 Comparison for different design conceptions

To assess the difference in slab punching shear resistance as a result of using the two different pressure distributions under the shearhead system, a comparison was made for the shearhead arm length parametric study reported in section 6.2.1 in the previous chapter. Figures 7.7 (a) and (b) present the calculation results.

Using the ACI 318 pressure distribution, the maximum shearhead arm length is 820mm with a punching shear resistance 155.4KN; the corresponding values are 645mm/173.0KN using BS8110 and 645mm/171.7KN using EC2.

If using the proposed pressure distribution of this Chapter (Figure 7.6), the corresponding values are: 610mm/122.8KN using ACI 318, 550mm/159.9KN according to BS8110 and 550mm/158.3KN if following EC2.

As expected, because the ACI 318 pressure distribution assumes higher value close to the column, it gives higher value of limit length for the shearhead arm than using the proposed method of pressure distribution. Although the predicted slab punching shear resistance using the ACI 318 pressure distribution are actually closer to the ABAQUS numerical values, it is recommended not to use the ACI 318 pressure distribution. Instead, the proposed pressure distribution is preferred because the proposed pressure distribution better represents that near slab failure; it is on the conservative (safe) side, and it maintains the same margin of safety of the different code calculation methods for flat slab design using normal reinforced concrete column.
(a) Shearhead arm length limit according to ACI 318 pressure distribution (Figure 8.6)

(b) Shearhead arm length limit according to the proposed pressure distribution (Figure 8.5)

Figure 7.7: Comparison between the FE results and design calculations
7.3 COMPARISON WITH PARAMETRIC STUDY

RESULTS

Using the design procedure presented above, all the parametric study cases were reanalyzed to obtain failure mode and failure load. Results are presented in Tables 7-2 – 7-7.

Table 7-2 Comparisons between design and numerical results for effects of column size

<table>
<thead>
<tr>
<th>Column section</th>
<th>CHS 274×10</th>
<th>CHS 219×10</th>
<th>CHS 274×10</th>
<th>CHS 219×10</th>
<th>CHS 274×10</th>
<th>CHS 219×10</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shearhead arm length $l_v$ (mm)</td>
<td>480</td>
<td>600</td>
<td>960</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ABAQUS slab resistance $V_{nm}$ (KN)</td>
<td>158</td>
<td>157</td>
<td>173</td>
<td>169</td>
<td>179</td>
<td>161</td>
</tr>
<tr>
<td>Predicted value from EC2 $V_{ec}$ (KN)</td>
<td>148.3</td>
<td>164.5</td>
<td>158.3</td>
<td>165.1</td>
<td>158.3</td>
<td></td>
</tr>
<tr>
<td>Difference $= V_{ec} / V_{nm}$</td>
<td>93.9%</td>
<td>94.5%</td>
<td>95.1%</td>
<td>93.4%</td>
<td>92.2%</td>
<td>98.3%</td>
</tr>
<tr>
<td>Predicted value from BS8110 $V_{bs}$ (KN)</td>
<td>149.6</td>
<td>166.6</td>
<td>159.9</td>
<td>166.6</td>
<td>159.9</td>
<td></td>
</tr>
<tr>
<td>Difference $= V_{bs} / V_{nm}$</td>
<td>94.7%</td>
<td>95.3%</td>
<td>96.3%</td>
<td>94.6%</td>
<td>93.1%</td>
<td>99.3%</td>
</tr>
<tr>
<td>Predicted value from ACI $V_{aci}$ (KN)</td>
<td>110.1</td>
<td>121.6</td>
<td>133.7</td>
<td>122.8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Difference $= V_{aci} / V_{nm}$</td>
<td>69.7%</td>
<td>69.7%</td>
<td>70.3%</td>
<td>72.0%</td>
<td>74.7%</td>
<td>76.3%</td>
</tr>
</tbody>
</table>
Table 7-3: Comparison between design and numerical results for shear arm section study

<table>
<thead>
<tr>
<th>Shear arm section</th>
<th>A: RHS 120×60×3.6</th>
<th>B: RHS 120×60×2.4</th>
<th>C: RHS 120×60 with 3.6m web and 1.8m flange</th>
<th>D: I section 120×60×3.6</th>
<th>E: I section 120×60×4.8</th>
<th>F: RHS 120×60 with 4.8m web and 0.1m flange</th>
<th>G: RHS 120×60×3.6</th>
<th>H: RHS 120×60×6.3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear arm length (mm)</td>
<td>600</td>
<td>960</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ABAQUS slab resistance $V_{nm}$ (KN)</td>
<td>169</td>
<td>140</td>
<td>151</td>
<td>152</td>
<td>156</td>
<td>86</td>
<td>161</td>
<td>187</td>
</tr>
<tr>
<td>Predicted value from EC2 $V_{ec}$ (KN)</td>
<td>158.3</td>
<td>149.5</td>
<td>151.9</td>
<td>151.9</td>
<td>158.3</td>
<td>64.0</td>
<td>158.3</td>
<td>174.0</td>
</tr>
<tr>
<td>Difference $= V_{ec}/V_{nm}$</td>
<td>93.4%</td>
<td>106.8%</td>
<td>100.6%</td>
<td>99.9%</td>
<td>101.5%</td>
<td>74.4%</td>
<td>98.3%</td>
<td>93.1%</td>
</tr>
<tr>
<td>Predicted value from BS8110 $V_{bs}$ (KN)</td>
<td>159.9</td>
<td>150.8</td>
<td>153.3</td>
<td>153.3</td>
<td>159.7</td>
<td>64.0</td>
<td>159.9</td>
<td>175.5</td>
</tr>
<tr>
<td>Difference $= V_{bs}/V_{nm}$</td>
<td>94.6%</td>
<td>107.7%</td>
<td>101.5%</td>
<td>100.9%</td>
<td>102.4%</td>
<td>74.4%</td>
<td>99.3%</td>
<td>93.9%</td>
</tr>
<tr>
<td>Predicted value from ACI $V_{aci}$ (KN)</td>
<td>121.6</td>
<td>114.7</td>
<td>116.5</td>
<td>116.5</td>
<td>121.6</td>
<td>64.0</td>
<td>122.8</td>
<td>140.0</td>
</tr>
<tr>
<td>Difference $= V_{aci}/V_{nm}$</td>
<td>72.0%</td>
<td>82.0%</td>
<td>77.1%</td>
<td>76.6%</td>
<td>77.9%</td>
<td>74.4%</td>
<td>76.3%</td>
<td>74.9%</td>
</tr>
</tbody>
</table>
Table 7-4: Comparison between design and numerical results for reinforcement ratio study

<table>
<thead>
<tr>
<th>Reinforcement ratio (%)</th>
<th>0.41</th>
<th>0.73</th>
<th>0.18</th>
<th>0.41</th>
<th>0.73</th>
<th>0.18</th>
<th>0.41</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear arm length (mm)</td>
<td>320</td>
<td>480</td>
<td>600</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ABAQUS slab resistance</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$V_{nm}$ (KN)</td>
<td>124</td>
<td>140</td>
<td>112</td>
<td>157</td>
<td>200</td>
<td>136</td>
<td>169</td>
</tr>
<tr>
<td>Predicted value from EC2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$V_{ec}$ (KN)</td>
<td>121.3</td>
<td>144.6</td>
<td>112.6</td>
<td>148.3</td>
<td>179.5</td>
<td>126.0</td>
<td>158.3</td>
</tr>
<tr>
<td>Difference $= V_{ec}/V_{nm}$</td>
<td>97.8%</td>
<td>103.3%</td>
<td>100.5%</td>
<td>94.5%</td>
<td>89.8%</td>
<td>92.7%</td>
<td>93.7%</td>
</tr>
<tr>
<td>Predicted value from BS8110</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$V_{bs}$ (KN)</td>
<td>121.6</td>
<td>145.1</td>
<td>114.1</td>
<td>149.6</td>
<td>181.1</td>
<td>127.8</td>
<td>159.9</td>
</tr>
<tr>
<td>Difference $= V_{bs}/V_{nm}$</td>
<td>98.1%</td>
<td>103.6%</td>
<td>101.9%</td>
<td>95.3%</td>
<td>90.6%</td>
<td>94.0%</td>
<td>94.6%</td>
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<tr>
<td>Predicted value from ACI</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>$V_{aci}$ (KN)</td>
<td>106.3</td>
<td>110.1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Difference $= V_{aci}/V_{nm}$</td>
<td>85.7%</td>
<td>75.9%</td>
<td>98.3%</td>
<td>69.7%</td>
<td>55.1%</td>
<td>89.4%</td>
<td>72.0%</td>
</tr>
</tbody>
</table>
Table 7-5: Comparison between design and numerical results for effective depth study

<table>
<thead>
<tr>
<th>Slab thickness (mm)</th>
<th>168</th>
<th>200</th>
<th>250</th>
<th>250</th>
<th>168</th>
<th>200</th>
<th>250</th>
<th>250</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear arm Length (mm)</td>
<td>600mm</td>
<td></td>
<td></td>
<td>480mm</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear arm section</td>
<td>RHS 120×60×3.6</td>
<td>RHS 120×60×4.8</td>
<td>RHS 120×60×3.6</td>
<td>RHS 120×60×4.8</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Column details</td>
<td>CHS 219.1×10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ABAQUS slab resistance ( V_{nm} ) (KN)</td>
<td>108</td>
<td>169</td>
<td>206</td>
<td>225</td>
<td>101</td>
<td>157</td>
<td>200</td>
<td>222</td>
</tr>
<tr>
<td>Predicted value from EC2 ( V_{ec} ) (KN)</td>
<td>122.4</td>
<td>158.3</td>
<td>223.3</td>
<td>230.9</td>
<td>108.1</td>
<td>148.3</td>
<td>222.5</td>
<td>222.5</td>
</tr>
<tr>
<td>Difference ( = \frac{V_{ec}}{V_{nm}} )</td>
<td>113.3%</td>
<td>93.7%</td>
<td>108.4%</td>
<td>102.6%</td>
<td>107.0%</td>
<td>94.5%</td>
<td>111.2%</td>
<td>100.2%</td>
</tr>
<tr>
<td>Predicted value from BS8110 ( V_{bs} ) (KN)</td>
<td>124.1</td>
<td>159.9</td>
<td>224.5</td>
<td>232.4</td>
<td>109.3</td>
<td>149.6</td>
<td>223.7</td>
<td>223.7</td>
</tr>
<tr>
<td>Difference ( = \frac{V_{bs}}{V_{nm}} )</td>
<td>114.9%</td>
<td>94.6%</td>
<td>109.0%</td>
<td>103.3%</td>
<td>108.2%</td>
<td>95.3%</td>
<td>111.9%</td>
<td>100.8%</td>
</tr>
<tr>
<td>Predicted value from ACI ( V_{aci} ) (KN)</td>
<td>95.9</td>
<td>121.6</td>
<td>163.0</td>
<td>167.8</td>
<td>84.0</td>
<td>110.1</td>
<td>159.7</td>
<td>159.7</td>
</tr>
<tr>
<td>Difference ( = \frac{V_{aci}}{V_{nm}} )</td>
<td>88.8%</td>
<td>72.0%</td>
<td>79.1%</td>
<td>74.6%</td>
<td>83.2%</td>
<td>69.7%</td>
<td>79.9%</td>
<td>71.9%</td>
</tr>
</tbody>
</table>
Table 7-6: Comparison between design method and numerical results for discontinued arm study

<table>
<thead>
<tr>
<th>Case study name</th>
<th>600 H C</th>
<th>600 H DisC</th>
<th>600 L C</th>
<th>780 H C</th>
<th>780 H DisC</th>
</tr>
</thead>
<tbody>
<tr>
<td>ABAQUS slab resistance $V_{nm}$ (KN)</td>
<td>169</td>
<td>145</td>
<td>131</td>
<td>164</td>
<td>149</td>
</tr>
<tr>
<td>Predicted value from EC2 $V_{ec}$ (KN)</td>
<td>158.3</td>
<td>117.4</td>
<td>158.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Difference $= \frac{V_{ec}}{V_{nm}}$ Predicted value</td>
<td>93.7%</td>
<td>109.2%</td>
<td>89.6%</td>
<td>96.5%</td>
<td>106.2%</td>
</tr>
<tr>
<td>Predicted value from BS8110 $V_{bs}$ (KN)</td>
<td>159.9</td>
<td>119.0</td>
<td>159.9</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Difference $= \frac{V_{bs}}{V_{nm}}$ Predicted value from BS8110</td>
<td>94.6%</td>
<td>110.3%</td>
<td>90.8%</td>
<td>97.5%</td>
<td>107.3%</td>
</tr>
<tr>
<td>Predicted value from ACI $V_{aci}$ (KN)</td>
<td>121.6</td>
<td>95.9</td>
<td>122.8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Difference $= \frac{V_{aci}}{V_{nm}}$ value from ACI</td>
<td>72.0%</td>
<td>83.9%</td>
<td>73.2%</td>
<td>74.9%</td>
<td>82.4%</td>
</tr>
</tbody>
</table>

H: Higher sheararm position, the top compression face of the sheararm at the level of the top compression face of the slab

L: Lower sheararm position, sheararm directly sit on the tension reinforcement level which only has 20mm cover from the tension side of the slab

C: continual sheararm adopted

DisC: discontinued sheararm adopted
Table 7-7: Comparison between design and numerical results for hole position study

<table>
<thead>
<tr>
<th>I value (mm)</th>
<th>190</th>
<th>290</th>
<th>340</th>
</tr>
</thead>
<tbody>
<tr>
<td>ABAQUS slab resistance $V_{nm}$ (KN)</td>
<td>110</td>
<td>124</td>
<td>123</td>
</tr>
<tr>
<td>Predicted value (failure mode) from EC2 $V_{ec}$</td>
<td>68.0</td>
<td>94.4</td>
<td>103.1</td>
</tr>
<tr>
<td>Difference $= \frac{V_{ec}}{V_{nm}}$</td>
<td>61.8%</td>
<td>76.1%</td>
<td>83.8%</td>
</tr>
<tr>
<td>Predicted value from BS8110 $V_{bs}$ (KN)</td>
<td>64.3</td>
<td>85.9</td>
<td>93.7</td>
</tr>
<tr>
<td>Difference $= \frac{V_{bs}}{V_{nm}}$</td>
<td>58.5%</td>
<td>69.3%</td>
<td>76.2%</td>
</tr>
<tr>
<td>Predicted value from ACI $V_{aci}$ (KN)</td>
<td>49.4</td>
<td>68.6</td>
<td>74.0</td>
</tr>
<tr>
<td>Difference $= \frac{V_{aci}}{V_{nm}}$</td>
<td>44.9%</td>
<td>55.3%</td>
<td>60.2%</td>
</tr>
</tbody>
</table>

From the above comparisons between the numerical simulation results and design calculation values, the enlarged column assumption works well for all different cases. Comparing the three design methods, EC2 and BS8110 have very similar accuracy and their calculation results are close to the ABAQUS simulation results (almost within 10% for all the study cases). Even though ACI 318 could not predict the critical perimeter length (see Figure 7.4 in Chapter 7), the results in Table 7.2-7.7 suggest that the ACI 318 predictions of slab punching shear capacity also achieved good accuracy and they are on the safety side.
7.4 CONCLUSIONS

This Chapter has proposed and validated a design method for calculating slab punching shear resistance. The shearhead system is treated as an enlarged square column. The applied column load is assumed to be uniformly distributed under the inclined surface area at the end of shearhead arm. This load distribution is used to check the bending and shear resistance of the shearhead arm to ensure that the shearhead arm does not fail before the slab reaches its punching shear resistance.

Comparisons of slab failure mode and punching shear resistance between the proposed method and ABAQUS simulations for the various parametric studies reported in Chapter 6 indicate that the proposed method correctly identified the slab failure mode and predicted slab punching shear resistance with the same accuracy as in normal flat slab design using reinforced concrete column. Both BS 8110 and EC2 appeared to give results in close agreement with ABAQUS numerical results. The ACI 318 method gave quite conservative results when using the proposed stress distribution of this study.

As a summary, the following steps may be used to determine an appropriate shearhead arm:

1) Calculate the required punching shear resistance (loading calculation);
2) Calculate the required critical perimeter length;
3) Choose the required shearhead arm length;
4) Select shearhead arm height based on slab thickness and reinforcement diameter and cover. Check the bending and shear resistance of the shearhead arm cross-section according to the pressure distribution in Figure 7.5.

Either BS 8110 or EC2 may be used.

Appendix E presents an example according to the above design process using EC2.
CHAPTER 8 CONCLUSIONS AND
RECOMMENDATIONS FOR FUTURE STUDIES

This chapter presents a summary of the main conclusions from this study and recommends a number of further studies.

8.1 CONCLUSIONS

This research has developed a new shearhead system between conventional reinforced concrete flat slab and steel tubular column. Based on the results of two full scale tests and extensive numerical analysis using the finite element software ABAQUS, this research has developed a practical design method for the proposed shearhead system. This study has proven that the proposed shearhead system is effective in providing sufficient punching shear resistance between reinforced concrete flat slab and steel tubular column. From the results of the detailed numerical parametric study using ABAQUS, it has been concluded that the shearhead system can be considered as an enlarged column. As a result of this assumption, conventional reinforced concrete structural design codes such as EC2 (CEN, 2002), BS8110 (BSI, 1997) and ACI 318 (ACI, 2005) can be used to calculate the punching resistance of reinforced concrete flat slab when using the proposed shearhead system in conjunction with steel tubular column, which may be empty or concrete filled (Because the column does not affect the slab punching shear resistance, this study applies to both unfilled and filled tubular columns).
This research work reported in this thesis has the following main components:

- Reports of two full scale tests carried out by the author (Chapter 3);
- Validation of ABAQUS models for the proposed structural system, based on the author’s two tests and previous analytical and experimental tests by others. A sensitivity study of mesh size and material property variables was also carried out (Chapter 4-5);
- Extensive parameter study to investigate the effects of different design variables, including column shape and size, shearhead arm details (cross-section type and dimensions, length, end details, position), flexural reinforcement details (amount, arrangement, position), and service hole, using the validated numerical model (Chapter 6); This study enabled design assumptions to be made for the proposed shearhead system;
- Design calculations of punching shear resistance for all the numerical parametric study cases using conventional reinforced concrete codes EC2, BS8110 and ACI 318, based on the design assumptions generated from the parametric study (Chapter 7).

The following main conclusions may be drawn:

- Attaching a shearhead system to steel tubular column is an effective method of enhancing punching shear resistance when used in combination with reinforced concrete flat slab;
- The general finite element software ABAQUS can be used to accurately simulate punching shear behaviour for the proposed reinforced concrete flat slab – steel tubular column system with a shearhead;
- The fracture energy based Hillerborg damaged plasticity model for concrete should be used. The simulation results were moderately sensitive to the
fracture energy value used, but the CEB-FIP recommendation for calculating fracture energy may be used.

The main conclusion of the parametric study was the enlarged square column assumption. The enlarged column size was the original column size plus the total lengths of the shearhead arms in the same direction. This conclusion was supported by the following results from the parametric study:

- Neither the column size nor the shape had any effect on the slab punching shear resistance provided the shearhead system did not fail prematurely;
- The critical punching shear perimeter of the flat slab calculated by using the conventional design methods but the enlarged column size was very close to numerical simulation results.

To calculate punching shear resistance of the proposed shearhead system based on the enlarged column assumption, the following conditions should be observed:

- The shearhead arms may be treated as cantilever beams. They should not fail prematurely in either bending or shear;
- To check for the shearhead arm load carrying capacity, the pressure distribution underneath the shearhead arms may conservatively be assumed to be uniformly distributed under the inclined surface;
- The end inclination angle (to the slab surface) of the shearhead arm is an important factor and the optimum value is 45°;
- The slab effective depth should be measured from the tip of the shearhead arm
to the level of tensile reinforcement. This is different from conventional code
design methods in which the distance is measured from the compression face
of the slab to the tensile reinforcement level;

- Dowel effect of the tensile flexural reinforcement should be included when
calculating slab punching shear resistance;

Based on the enlarged column assumption, conventional reinforced concrete design
methods can be used to calculate the slab punching shear resistance.

- EC2 and BS8110 predict very similar values. Their predictions are in very
good agreement with the ABAQUS numerical results (normally within 10%).
- ACI 318 was the only code that explicitly considered shearhead system.
  However, its prediction of critical punching shear perimeter did not agree with
  either the experimental or numerical results as well as using the other two
codes. Nevertheless, its predictions of the slab punching shear resistance
  achieved reasonably good agreement with the numerical analysis results and
  were on the safe side.

8.2 RECOMMENDATIONS FOR FUTURE RESEARCH

This research has explored a solution for a new shearhead system between normal
reinforced concrete slab and steel tubular column based on an experimental and
numerical study. Due to financial and time constraint, only two tests were performed.
These two tests would not be sufficient to validate all the assumptions as a result of
numerical simulations. Also the numerical simulation was carried out using ABAQUS,
which was available to the author, but alternative numerical simulation software such as DIANA, may be more suitable because of its superiority in handling concrete material properties. To thoroughly prove the proposed design method, the following further research studies are considered appropriate:

- In both tests performed by the author, failure was in the concrete slab. Additional tests would be useful to examine slab behaviour if failure is in the shearhead arms;
- Detailed experimental observation of pressure distribution underneath the shearhead arms should be performed;
- This study is very limited on the effects of service hole. More extensive experimental and numerical studies would be required;
- Use of alternative numerical simulation method, such as DIANA, should be explored;
- The lateral force effect may be concluded in the further research;
- The study in this thesis is for ambient temperature only. Further research should be conducted to investigate whether the proposed method is applicable to elevated temperatures under fire condition.
Appendix A: Recorded data from the physical tests

Figure A.1 The average Stress-Strain curve for 28 day concrete

Figure A.2 Measured raw Load-Deformation curves for the first test
Figure A.3 Strain record for the tension reinforcement bars in the first test

Figure A.4 Strain record on the concrete compression face in the first test
Figure A.5 Measured raw Load-Deformation curves for the second test
### Strength classes for concrete

<table>
<thead>
<tr>
<th>$f_{ck}$ (MPa)</th>
<th>12</th>
<th>16</th>
<th>20</th>
<th>25</th>
<th>30</th>
<th>35</th>
<th>40</th>
<th>45</th>
<th>50</th>
<th>55</th>
<th>60</th>
<th>67</th>
<th>75</th>
<th>85</th>
<th>95</th>
<th>105</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_{cm}$ (MPa)</td>
<td>15</td>
<td>20</td>
<td>25</td>
<td>30</td>
<td>37</td>
<td>45</td>
<td>50</td>
<td>55</td>
<td>60</td>
<td>67</td>
<td>75</td>
<td>85</td>
<td>95</td>
<td>105</td>
<td>110</td>
<td>115</td>
</tr>
<tr>
<td>$f_{cm}$ (MPa)</td>
<td>20</td>
<td>24</td>
<td>28</td>
<td>33</td>
<td>38</td>
<td>43</td>
<td>48</td>
<td>53</td>
<td>58</td>
<td>63</td>
<td>68</td>
<td>78</td>
<td>88</td>
<td>98</td>
<td>105</td>
<td>110</td>
</tr>
</tbody>
</table>

### Analytical relation / Explanation

- $f_{cm} = f_{ck}^2 / (50.60)$  
- $f_{cm} = 0.30 (f_{ck})^{0.35}$ for $f_{ck} > 50.60$
- $f_{cm} = 0.7 f_{ck}$ for 5% failure
- $f_{cm} = 1.3 f_{ck}$ for 95% failure
- $E_{cm} = 22 (f_{cm})^{0.5}$ in MPa
- $f_{ck} = (2.8+271 f_{ck})^{1/4}$ for $f_{ck} = 50$ Mpa
- $f_{cm} = 0.68 (f_{cm})^{0.5}$

### Deformation characteristics for concrete (CEN, 2002)

- $\varepsilon_1 (\%)$ = 1.8, 1.9, 2.0, 2.1, 2.2, 2.25, 2.3, 2.4, 2.45, 2.5, 2.6, 2.7, 2.8, 2.8  
  - see Figure 3.2 for $f_{ck} = 50$ Mpa
  - $\varepsilon_1 (\%) = 0.7 f_{cm}^{0.8} < 2.8$

- $\varepsilon_2 (\%)$ = 3.5, 3.2, 3.0, 2.8, 2.8, 2.8  
  - see Figure 3.2

- $\varepsilon_3 (\%)$ = 2.0  
  - see Figure 3.3

- $\varepsilon_{0.2} (\%)$ = 3.1, 2.9, 2.7, 2.6, 2.6  
  - see Figure 3.3 for $f_{ck} = 50$ Mpa
  - $\varepsilon_{0.2} (\%) = 2.8+3(0.6-f_{ck})/100$ for $f_{ck} = 50$ Mpa

- $\varepsilon_{0.3} (\%)$ = 1.75, 1.8, 1.9, 2.0, 2.2, 2.3  
  - see Figure 3.4 for $f_{ck} = 50$ Mpa
  - $\varepsilon_{0.3} (\%) = 2.6+3(0.6-f_{ck})/100$ for $f_{ck} = 50$ Mpa

- $\varepsilon_{0.5} (\%)$ = 3.5, 3.1, 2.9, 2.7, 2.6, 2.6  
  - see Figure 3.4 for $f_{ck} = 50$ Mpa

- $\varepsilon_{0.7} (\%)$ = 2.8+3(0.6-f_{ck})/100 for $f_{ck} = 50$ Mpa
Appendix C: Fracture Energy calculation ($G_f$):

As described in the CEB-FIP model (CEB-FIP, 1993), in absence of experimental data $G_f$ may be estimated as:

$$G_f = G_{f\theta} \left( \frac{f_{cm}}{f_{c,m0}} \right)^{0.7}$$  \hspace{1cm} (Appendix C-1)

Where

$f_{c,m0} = 10\text{N/mm}^2$

$f_{cm} = 43\text{N/mm}^2$.

$G_{f\theta}$ is the base value of fracture energy depending on the maximum aggregate size ($d_{max}$) from the following table:

<table>
<thead>
<tr>
<th>$d_{max}$ (mm)</th>
<th>$G_{f\theta}$ (N/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>0.025</td>
</tr>
<tr>
<td>16</td>
<td>0.030</td>
</tr>
<tr>
<td>32</td>
<td>0.058</td>
</tr>
</tbody>
</table>

For a maximum aggregate size of 25mm, $G_{f\theta}=0.044$,

$G_f \approx 0.12\text{N/mm} = 120 \text{N/m}$
Appendix D: Design calculations for the author’s tests

1: Based on EC2

Test 1

\[ d_1 = 126 \text{mm} \]

\[ u_{1,ec_2} = 480 \times 4 + 2\pi \times (2 \times 126) \approx 3503 \text{mm} \]

![Figure D.1 Critical perimeter for EC2 without hole](image)

\[ v_{Rd,c} = C_{Rd,c} k' \left( 100 \rho_1 f_{ck} \right)^{\frac{1}{3}} \geq v_{\text{min}} \]

Where:

- \( C_{Rd,c} \) is 0.18 here,
- \( v_{\text{min}} \) is calculated using equation 6.3N in EC2

\[ k' = 1 + \sqrt{\frac{200}{d}} \leq 2.0 \text{ (d is in mm);} \]

\[ f_{ck} = 35 \text{MPa} \]

Giving \( v_{Rd,c} \approx 0.88N/mm^2 \)

\[ V_{Rd} = v_{Rd,c} u_{1,ec_2} d_1 = 0.88 \times 3503 \times 126 \approx 386.7KN \]
Test 2

d_2 = 168 \text{mm}

u_{2, ec_2} = 2\pi \times 2d \times \frac{(360 - 63)}{360} + 4 \times 480 \approx 3662 \text{mm}

Figure: D.2 Critical perimeter for EC2 with hole

v_{Rd,c} \approx 0.88 \text{N/mm}^2

V_{Rd} = v_{Rd,c} u_{2, ec_2} d_1 = 0.58 \times 3662 \times 168 \approx 541.3 \text{KN}

2: Based on BS 8110

Test 1

C1=C2=480\text{mm}

d_1 = 126\text{mm}

u_{1,Bs} = 480 \times 4 + 1.5 \times 8 \times 126 \approx 3432 \text{mm}
\[ v_c = 0.79 \times \left( \frac{100A_s}{b_v d_1} \right)^{\frac{1}{3}} \times \left( \frac{400}{d_1} \right)^{\frac{1}{4}} \times \left( \frac{f_{cu}}{25} \right)^{\frac{1}{3}} \]

Where:

\[ \frac{100A_s}{b_v d_1} \leq 3, \left( \frac{400}{d_1} \right)^{\frac{1}{4}} \geq 0.67 \]

Using the maximum value of \( f_{cu} = 40N/mm^2 \) allowed in BS 8110

giving \( v_c \approx 0.90N/mm^2 \)

\[ V_{c1} = v_c u_{1, Bs} d_1 = 0.90 \times 3432 \times 126 \approx 389.2KN \]

**Test 2**

C1=C2=480mm

\( d_2 = 168mm \)

\[ u_{2, Bs} = \left( 480 \times 4 + 1.5 \times 8 \times 168 \right) \times \frac{3}{4} + 2 \times (240 + 1.5 \times 168) \times \tan(29.25)^o \]

\[ \approx 3503mm \]
Figure D.4 Determination of critical perimeter for BS8110 with hole

$v_c \approx 0.90\text{N/mm}^2$

$V_{c_2} = v_c u_{2,bs} d_2 = 0.90 \times 3503 \times 168 \approx 529.6\text{KN}$

3: Based on ACI318

Test 1

$d_1 = 126\text{mm}$

$u_{1,ACI} = 8 \times \sqrt{(0.5 \times 126 + 100)^2 + [0.75 \times (240 - 100) - 0.5 \times 126]^2}$

$\approx 1347\text{mm}$

Figure D.5 Determination of critical perimeter
\[ V_{n1} = 0.33u_{1,ACI} d_1 \sqrt{f_{ck}} = 0.33 \times 1347 \times 126 \times \sqrt{35} \approx 331.4 KN \]

**Test 2**

\[ d_2 = 168 \text{ mm} \]

\[ u_{2,ACI} = 8 \times \]

\[ \sqrt{(0.5 \times 168 + 109.55 \times \sin 45^\circ)^2 + [0.75 \times (240 - 109.55) + 109.55(1 - \sin 45^\circ) - 0.5 \times 168]^2} - 71 \approx 1254 \text{ mm} \]

![Figure D.6 Determination of critical perimeter](image)

\[ V_{n2} = 0.33u_{2,ACI} d_2 \sqrt{f_{ck}} = 0.33 \times 1254 \times 168 \times \sqrt{35} \approx 411.3 KN \]
Appendix E: Design example using EC2

Design data:

![Design structural arrangement](image)

The layout of the structure shows in Figure E.1 and

- Slab thickness: 200mm
- Loading: Imposed load = 3.5N/mm²
- Material properties: Concrete density = 2500kg/m³  
  C40 concrete  
  Plastic stress of steel = 345 N/mm²

The top compression faces of sheararms flush with the top concrete compression face.
Normal reinforcements placed 20mm away from the slab face to get the minimum cover.

1) **Load calculation:**

Weight of slab=0.2×25×6×6=180KN

Variable load=3.5×6×6=126KN

Ultimate load: F=1.35×180+1.5×126=432KN
2) Minimum critical perimeter length required

\[ v_{rd,c} = C_{rd,c} k' (100 \rho_f f_{uk}^{0.1})^{\frac{1}{3}} \geq v_{\text{min}} \]

Where: \( k' = 1 + \sqrt{\frac{200}{d}} \leq 2.0 \)

\[ C_{rd,c} = 0.18 / \gamma_c \]

Here \( v_{rd,c} = 0.581 \text{N/mm}^2 \), giving:

\[ u_b = \frac{V}{v_{rd,c} \times d} = \frac{423000}{0.581 \times 168} = 4334 \text{mm} \]

3) Minimum shearhead length \( l_v \):

\[ 8l_v + 4\pi d \geq u_b \Rightarrow l_v \geq \frac{u_b - 4\pi d}{8} \]

Giving: \( l_v \geq 278 \text{mm} \)

Use \( l_v = 280 \text{mm} \)

4) Select RHS 120x60x3.6 with end angle 45° as shearhead arm. Check shearhead arm capacity.

(a) Shear capacity check:

\[ V_{\text{arm,s}} = 3.6 \times 2 \times 120 \times 345 \times 0.6 = 178.8 \text{KN} \geq \frac{V}{4} = 105.75 \text{KN} \]

(b) Plastic bending moment capacity check:

\[ M_p = 48000 \times 345 = 16.56 \text{KN.m} \geq \frac{V_u}{\eta} \left( l_v - \frac{c_t}{2} - \frac{h_t}{2 \tan \alpha} \right) = 12.69 \text{KN.m} \]
PUBLICATIONS

P.Y. Yan, Y.C. Wang and A. Orton, “Development of an effective shearhead system to eliminate punching shear failure between flat slabs and tubular columns”, 12th international symposium of tubular structures (ISTS12), Shanghai, China, 2008, pp.441-448


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tubular columns. The 12th international symposium on tubular structures. Shanghai, China, Taylor & Francis Group.


