BEHAVIOUR AND DESIGN OF HEADED STUD SHEAR CONNECTORS IN COMPOSITE STEEL-CONCRETE BEAMS

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A thesis submitted in partial fulfillment of
the requirements for the degree of
Doctor of Philosophy

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December 2008
STATEMENT OF AUTHENTICATION

I hereby declare that this submission is my own work and to the best of my knowledge it contains no material previously published or written by another person, nor material which to a substantial extent has been accepted for the award of any other degree or diploma at University of Western Sydney or any other educational institution, except where due acknowledgement is made in the thesis. Any contribution made to the research by others, with whom I have worked at University of Western Sydney or elsewhere, is explicitly acknowledged in the thesis.

I also declare that the intellectual content of this thesis is the product of my own work, except to the extent that assistance from others in the project’s design and conception or in style, presentation and linguistic expression is acknowledged.

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ABSTRACT

Composite steel concrete beams are formed by connecting concrete slabs to a supporting structural steel beam. In the early 1900’s, composite beams were considered favourable for bridge design but in recent decades, composite steel and concrete structures are employed extensively in modern high rise buildings. The flexural strength of composite beams is greatly influenced by the strength and ductility of the shear connectors between the structural steel beam and the concrete slab. The behaviour of the shear connectors is important in understanding the shear force transmitted and the degree of slip which occurs at the interface of the steel and concrete.

Composite steel-concrete beams are becoming increasingly popular in multistorey buildings due to their higher span/depth ratio, reduced deflections and increased stiffness value. However, their performance is highly dependent on the load-slip characteristics of the shear connectors. More recently, trapezoidal profiled slabs are becoming increasingly more popular for high-rise buildings when compared with solid slabs because they can achieve large spans with little or no propping and they require less concrete and plywood formwork. However, the profiles used to achieve these savings can have a detrimental effect on the shear connector behaviour.

The issues emphasised in this thesis are:

- the effects of steel fibres as a strengthening system in composite steel-concrete beams.
• the effects of elevated temperatures on the behaviour of headed stud shear connectors for composite steel-concrete beams
• the long term effects on the behaviour of the composite steel-concrete beams
• the effects of strain regimes on the behaviour of the composite steel-concrete beams
• the effects of the combination of axial tension and shear loading on the behaviour of composite steel-concrete beams

This thesis consists of many of numerical studies. They include the mechanical behaviour and various loading conditions of the behaviour and strength of headed stud shear connectors for composite steel-concrete beams. The finite element package known as ABAQUS was used to compare the existing experimental studies by other researchers. The outcome of the numerical analysis is very satisfying and also contains design recommendations. Furthermore, a methodical parametric study is undertaken using the model that has been properly calibrated. These parametric studies broaden the range of application for design guidance. It is fervently hoped that the findings herein are useful in contributing further insight to the continuing evolution of composite steel-concrete structures.
ACKNOWLEDGEMENTS

I would like to express my deep and sincere appreciation to my supervisor Professor Brian Uy, Head of School of Engineering, University of Western Sydney for his boundless help, guidance, professionalism and encouragement throughout my Ph.D. I would like to also acknowledge Prof. Brian Uy for his suggestions, support, constant review throughout every stage and for his excellent advice and suggestions of this research including analysis and manuscript preparation.

I also would like to acknowledge Mr. Alexander Filonov for his funding and financial assistance. This research was supported by the Australian Research Council Linkage Grant Scheme and Bluescope Lysaght Technology at Chester Hill, Sydney.

I am indebted to my husband, Pedram, for his constant love, endless support and understanding. I have always been able to count on you. A special thanks to my parents, Francis Xavier Chee and Mary Chew. Without your understanding and support, I might not have come this far.

Finally, I would like to thank the School of Engineering at University of Western Sydney for providing a conducive environment for me to do my research. I am very fortunate to be able to work with outstanding lecturers, administrative and computing staff, fellow colleagues and students. The daily communication, interaction and laughter with them contribute towards a fruitful progress in research.
PREFACE

This thesis is submitted in partial fulfillment of the requirements for the degree of Doctor of Philosophy at University of Western Sydney (UWS), Sydney, Australia. The work described herein was performed by the candidate in the School of Engineering, UWS. The candidate was supervised by Professor Brian Uy during a period from January 2006 to December 2008.

The thesis has been supported by papers that have been submitted for consideration, accepted or published in internationally renowned journals and conferences. These papers in addition to research reports are listed in the following:

Journal papers


**Conference papers**


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<td>$A_{1p}$</td>
<td>the cross sectional area at the base of the concrete slab</td>
</tr>
<tr>
<td>$A_{1s}$</td>
<td>the cross sectional area at the top of the concrete slab</td>
</tr>
<tr>
<td>$A_s$</td>
<td>the cross sectional area of shear connectors</td>
</tr>
<tr>
<td>$a$, $b$, $c$</td>
<td>parameters for elevated temperature behaviour for composite steel-concrete beams</td>
</tr>
<tr>
<td>$B$</td>
<td>width of the slab</td>
</tr>
<tr>
<td>$b_f$</td>
<td>width of the steel flange</td>
</tr>
<tr>
<td>$b_o$</td>
<td>width of the steel profiled ribs</td>
</tr>
<tr>
<td>$b_w$</td>
<td>width of the steel web</td>
</tr>
<tr>
<td>$C_{p}, C_a$</td>
<td>specific heat for concrete or steel structures</td>
</tr>
<tr>
<td>$c_1, c_2$</td>
<td>parameters of force-slip relationship for composite steel-concrete beams</td>
</tr>
<tr>
<td>$D$</td>
<td>depth where neutral axis lies</td>
</tr>
<tr>
<td>$d$</td>
<td>slip capacity of the shear stud connector or diameter of shear connectors</td>
</tr>
<tr>
<td>$d_f$</td>
<td>depth of the steel flange</td>
</tr>
<tr>
<td>$d_w$</td>
<td>depth of the steel web</td>
</tr>
<tr>
<td>$E$</td>
<td>elastic modulus of concrete</td>
</tr>
<tr>
<td>$E_c$</td>
<td>mean modulus of elasticity of concrete</td>
</tr>
<tr>
<td>$E_{c,EM}$</td>
<td>modified elastic modulus for the effective method</td>
</tr>
<tr>
<td>$E_{c,MS}$</td>
<td>modified elastic modulus for the mean stress method</td>
</tr>
<tr>
<td>$E_f$</td>
<td>design action effect for fire condition</td>
</tr>
<tr>
<td>$E_{n}$</td>
<td>design action effect for ambient condition</td>
</tr>
<tr>
<td>$E_s$</td>
<td>elastic modulus of steel fibres</td>
</tr>
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</table>
\( E_{sc} \) modulus elasticity of shear connector

\( E_{a,\theta} \) slope of linear elastic range

\( f_c \) mean compressive strength of concrete

\( f_{ck}, f'_c \) characteristic strength of compressive cylinder strength of concrete

\( f_{c,\theta} \) ultimate stress of concrete

\( f_{p,\theta} \) proportional limit of structural steel

\( f_t \) tensile strength of concrete

\( f_{tu} \) residual strength of concrete

\( f_{us} \) ultimate stress of shear connectors

\( f_y \) yield strength of structural steel

\( f_{y,\theta} \) effective yield strength of structural steel

\( F \) fibre content

\( F_u \) ultimate load

\( G \) dead load

\( H \) enthalpy for temperatures

\( H \) height of stud shear connector

\( h, h_d \) height of shear connectors

\( h, h_p \) height of the decking of the profiled slab

\( k \) reduction factor for profiled slab

\( k_{AS} \) reduction factor for profiled slab in Australian Standards

\( k_{EC} \) reduction factor for profiled slab in Eurocode

\( k_{AISC} \) reduction factor for profiled slab in American Institute Steel Construction

\( k_{c,t} \) reduction coefficient for tensile strength of concrete

\( L/d \) fibre aspect ratio
\( N_{\text{concrete}} \) forces acting on concrete element
\( N \) the number of studs in a rib
\( N_d \) the axial tensile capacity of the headed shear stud
\( N_{sd} \) the applied axial tensile load
\( N_{\text{steel}} \) forces acting on steel element
\( n \) number of shear connectors in a shear span
\( P \) shear capacity of shear connectors or pressure
\( P_{AS} \) ultimate load for Australian Standards
\( P_{AISC} \) ultimate load for American Institute of Steel Construction
\( P_d \) the shear capacity of headed shear stud
\( P_{EC} \) ultimate load for Eurocode
\( P_{sd} \) the applied shear load
\( P_t \) ultimate load at time \( t \)
\( P_{(28)} \) ultimate load at 28 days
\( P_u \) ultimate load
\( Q \) live load or applied load
\( Q_{\text{rib}} \) the strength of a stud in formed steel deck
\( Q_{\text{SOL}} \) the strength of the stud shear connector in a flat soffit slab
\( Q_u \) ultimate load
\( s \) longitudinal shear slip
\( T \) temperature
\( t \) time or thickness of the concrete slab
\( t_r \) time application of stress
\( t_s \) age of concrete in days when shrinkage start
\( t_0 \) the final time or time of application of stress
$V_f$ fibre volume fraction

$w$ the average rib width

$\alpha, \beta$ parameters in load-slip characteristics of shear connectors

$\alpha$ parameter used to define strain regimes for concrete

$\beta_s$ the coefficient to represent the development of shrinkage according to time

$\varepsilon^{el,d}$ the effective strain which is the sum of instantaneous strains

$\varepsilon_c$ concrete compressive strain

$\varepsilon'_c$ strain corresponding to $f'_c$

$\varepsilon_{c,\theta}$ strain at ultimate stress of the concrete according to temperature

$\varepsilon_{co}$ strain value at yield stress

$\varepsilon_{cu}$ ultimate compressive strain

$\varepsilon_{cu1\theta}$ ultimate strain of the concrete according to temperature

$\varepsilon^{el,d}$ effective strain (sum of instantaneous strain)

$\varepsilon''_c$ fraction of viscous strain which obtained from an age of loading time

$\varepsilon_{ps}$ strain value when strain hardening commences

$\varepsilon_{p,\theta}$ strain at the proportional limit

$\varepsilon_{p,\theta}$ strain at the proportional limit

$\varepsilon_{d,\theta}$ stain that occurs after concrete hardening up to time infinity

$\varepsilon_{u,\theta}$ ultimate strain according to temperature

$\varepsilon_{y,\theta}$ yield strain according to temperature

$\varepsilon_{us}$ ultimate yield strain of the steel structure

$\varepsilon_{ys}$ yield strain of the steel structure

$\gamma$ parameter used for stress-strain curve for concrete
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<tr>
<td>$\eta$</td>
<td>fibre orientation in three dimensional random distribution</td>
</tr>
<tr>
<td>$\sigma_c$</td>
<td>concrete compressive stress</td>
</tr>
<tr>
<td>$\tau_d$</td>
<td>bond stress</td>
</tr>
<tr>
<td>$\sigma_t$</td>
<td>tensile strength of concrete</td>
</tr>
<tr>
<td>$\sigma_{us}$</td>
<td>ultimate stress of the steel material</td>
</tr>
<tr>
<td>$\sigma_{ys}$</td>
<td>yield stress of the steel material</td>
</tr>
<tr>
<td>$\psi$</td>
<td>ageing coefficient for concrete</td>
</tr>
<tr>
<td>$\chi$</td>
<td>ageing coefficient</td>
</tr>
<tr>
<td>$\phi$</td>
<td>creep coefficient</td>
</tr>
<tr>
<td>$\gamma$</td>
<td>parameter used to define stress-strain curve for concrete</td>
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<td>$\lambda$</td>
<td>thermal conductivity</td>
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<td>$\sigma_c$</td>
<td>concrete compressive stress</td>
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<td>$\sigma_{us}$</td>
<td>ultimate stress of the steel material</td>
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<tr>
<td>$\sigma_{ys}$</td>
<td>yield stress of the steel material</td>
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<tr>
<td>$\kappa_s, \kappa_p$</td>
<td>the ratio of the proportional cross sectional area of the concrete in high temperature</td>
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<td>$\mu_1, \mu_2$</td>
<td>parameters for interaction diagram</td>
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<tr>
<td>$\theta$</td>
<td>temperature</td>
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<tr>
<td>$\xi$</td>
<td>load ratio between design effects in fire condition with respect to design effect in ambient condition</td>
</tr>
<tr>
<td>$\Delta l/l$</td>
<td>thermal expansion</td>
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LIST OF SUBSCRIPTS

\begin{tabular}{ll}
\textit{AS} & Australian Standard \\
\textit{AISC} & American Standard \\
c & concrete slab \\
\textit{EC} & British Standard (Eurocode) \\
s & structural steel beam \\
sc & shear connection \\
t & time \\
y & yield \\
u & ultimate \\
\theta & temperature \\
\tau & time \\
\end{tabular}

LIST OF ABBREVIATION

\begin{tabular}{ll}
\textit{AS} & Australian Standard \\
AISC & American Standard \\
EC & British Standard (Eurocode) \\
EM & Effective Method \\
\textit{FEA} & Finite Element Analysis \\
FEM & Finite Element Method \\
MS & Mean Stress Method \\
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1.1 INTRODUCTION

Nowadays, the use of composite steel-concrete floor slab systems in high rise buildings and bridges is a widespread practice. The flexural strength of composite beams is greatly influenced by the strength and ductility of the shear connectors between the structural steel beam and the concrete slab. Amongst the numerous advantages over the traditional reinforced concrete slabs, are their lightweight and the simplicity with which the steel deck is handled and erected. Even though the concept of composite steel-concrete slabs began in the 1920’s the application did not develop until the 1950’s, Ollgaard et al. (1971). More recently, trapezoidal profiled slabs are becoming increasingly more popular for high rise buildings when compared with solid slabs because they can achieve large spans with little or no propping and they require less concrete and plywood formwork. However, the profiles used to achieve these savings can have a detrimental effect on the shear connector behaviour.

This chapter presents a synopsis of the research work involved in the present study and also outlines its significance and contributions. Background knowledge which highlights the grey areas still requiring further research and consideration is also described. These issues are further discussed in Chapter 2 as part of the literature
CHAPTER 1 – Introduction

review. This thesis endeavours to investigate the identified deficits concerning the effects of various issues on the behaviour of stud shear connectors for composite steel-concrete beams. They include the effects of steel fibres, elevated temperatures, long term analysis, strain regimes on the concrete slab elements and the combination of axial tension and shear loading on the behaviour of the headed stud shear connectors on composite steel-concrete beams.

1.2 BACKGROUND AND MOTIVATION FOR RESEARCH

Although composite steel and concrete members have been studied extensively since the late 1960’s, the limit states and design procedures with respect to strength and serviceability are still being researched. With regard to flexural members, the main research has focused on the strength and behaviour of composite beams with or without profiled steel decking. As a result, the American Institute of Steel Construction (AISC) provisions, in addition to European and Canadian steel codes, have covered the strength and behaviour of composite beam construction extensively. However, the behaviour of headed stud shear connectors has not been studied extensively.

Push tests are used in order to study the behaviour of headed stud shear connectors. The standardised push test specimens for both solid and profiled slabs are illustrated in Figure 1.1 and Figure 1.2 respectively. When headed stud shear connectors are used in design, one must be able to predict their ability to resist the longitudinal forces that arise between the steel and concrete. Hawkins and Mitchell (1984) stated that it is difficult to predict the strength of headed stud shear connectors. The reason is that when the embedded headed stud shear connector
approaches failure, the behaviour is very complex because of the inelastic deformations in the stud under the combined effects of shear, bending and tension. The concrete surrounding the headed stud shear connector are subjected to cracks due to high splitting forces caused by headed stud shear connectors and there is the likelihood that a non-ductile failure will occur. If the concrete strength is very high, the headed shear stud fracture will occur prior to concrete failing.

In the 1960’s and 1970’s the headed stud shear connector strength prediction equations for solid slabs were developed based on a series of the results of push-out tests. In the late 1970’s, Grant et al. (1977) undertook full scale beam tests and the equations were modified for the use of profiled steel sheeting. Then the problem of where headed stud shear connector should be placed arises. Grant et al. (1977) showed that the position of the headed stud shear connector affects the strength of the stud. This is because the amount of concrete between the profiled steel sheeting and the stud, on the stud’s load-bearing side, is different, as shown in Figure 1.3. The strong and weak position is illustrated in Figure 1.3. The position of the stud in the rib is not considered in the present strength equations used in the AISC (1999), British Standards Institution (2004) and AS 2327.1 (2003) because when the stud strength equations were developed from Grant et al. (1977), the experimental studies mostly used the studs welded in the centre of the deck rib.

Many experimental studies undertaken by researchers around the world has shown that the existing shear stud strength prediction equations in the AISC (1999) specification are not conservative. Whilst in the British Standards Institution (2004), the shear stud strength prediction is overly conservative. With respect to AS 2327.1
(2003), the issues related to the strength of welded headed stud shear connectors have been studied but they do not consider the height and gauge of the profiled steel sheeting.

In the 1990’s Easterling and Young (1992) showed that results from push-out tests can be used to accurately predict beam test results. The failure modes were accurately predicted. The authors stated that the headed stud shear connectors in the weak position failed by punching through the web of the profiled steel sheeting and the headed studs in the strong position failed by concrete failure and none failed by stud fracture. The authors also conclude that the weak position of headed studs behaved in a more ductile manner when compared with the strong position of headed studs. It should be borne in mind that the experimental work has been carried out on push tests and beams tests only considered failure modes. The study did not include the comparison of ultimate strength of headed shear stud connectors for both push tests and beams.

In order to determine the effect of stud tensile strength on shear connection, Lyons et al. (1994) performed 48 solid slab and 87 profiled slab push-out tests. The variables included position, height, and arrangement of the headed shear stud as well as height and gauge of the profiled steel sheeting. Similar to Easterling and Young (1992), the test results showed that the weak position of studs are very ductile and are influenced by the profiled steel sheeting strength. The author also concluded that the current AISC (1999) equations are less conservative for shear connections with profiled steel sheeting. Moreover the author stated that the equations described by Mottram and Johnson (1990) were found to be more conservative and accurate when
compared with their experimental results but still insufficient for headed stud shear connectors in the weak position. Instead Lyons et al. (1994) suggested an upper limit equation on headed stud shear connector strength of $0.8A_F u$, where $F_u$ is the tensile strength of the stud. The effect of tensile strength on the push test tends to present an impression that the headed stud shear connectors will exhibit a lower strength and experience higher slip deformation. Various tests have been conducted to investigate this but the findings were inconclusive. This thesis is to investigate the tensile force that affected the behaviour of headed stud shear connectors by adding steel fibres to reduce the cracking and strengthening of the structures.

Composite steel-concrete beam performance in fire has long been attributed to a loss of strength and stiffness due to thermal degradation. Bailey et al. (1999) state that most of the researchers and designers neglect the fundamental observations of the interactions between structural components and those members who restrained them when exposed to elevated temperatures. Clifton (2001) has undertaken full-scale fire testing of a typical composite steel-concrete office building. The author has proven that the interaction of all the structural components of composite steel-concrete can not be overlooked. Steel beams in composite action with a concrete slab when heated from below by fire have been shown to support loads well beyond the expected failure temperature of the steel alone. As predicted, in this situation the concrete slab is offering additional strength after the steel beam has failed and these results show that even after the steel beam had undergone excessive deflections the structural capacity was maintained. The results mentioned have been supported by Buchanan (2001). The cause of the steel’s reduced performance at elevated temperatures can mainly be attributed to a reduction of strength and stiffness. The
experimental work that been carried out only consider the behaviour of composite steel-concrete beams and slabs. The study has not included the consideration of local issues such as the behaviour of headed shear connectors. Thus, this thesis will also emphasise the behaviour of headed stud shear connectors at elevated temperatures.

When concrete creep and shrinkage is considered, the deformations will increase with time. A lot of research has been undertaken in determining the accuracy of prediction for creep and shrinkage for concrete by various researchers such as Bazant (2001), Bazant and Kwang (1985), Johnson (1974), Gardner and Zhao (1993) and Bohner and Muller (2006). Gilbert and Bradford (1995) stated that the time dependent deformations of concrete due to creep and shrinkage is very complicated and only limited research has been published on the time varying behaviour for composite steel-concrete beams. Composite steel-concrete beam designs are mainly affected by the behaviour of the shear connection. This thesis considers the creep and shrinkage behaviour of the concrete which indirectly affects the shear connection of composite steel-concrete beams.

As previously mentioned, various researchers have considered the failure mode for beam tests and push tests. Hicks (2007) tested a full-scale composite steel-concrete beam and conducted companion push tests. The beam demonstrated excellent ductility with the slip capacity exceeded the value suggested by existing international standards. When comparing the headed stud shear capacity between beam tests and push tests, there is a significant difference. The behaviour of headed studs in push test specimens provide conservative estimates of strength and they do not reflect the ductility that the beam tests exhibit. This thesis proposes to modify the
strain regimes in the concrete element of a push test to provide a more accurate depiction of the strain regimes in beams.

For flooring systems, the headed studs are normally subjected to a combination of shear and bending loads. This has been investigated by Mirza and Uy (2008). In wall systems, the headed studs are subjected to shear and tensile axial loading. The tensile axial loading may cause both the concrete and headed stud to be exposed to tensile failure. There is significant research looking at the behaviour of headed studs when subjected to shear loading, Becher (2005), El-Lobody and Young (2006), Gattesco and Giuriani (1996), Hanswille et al. (2008), Johnson (2000), Lam and El-Lobody (2005), Wu (2006) and Baskar et al. (2002). The nature of this thesis is to consider the combined effect of the combination of shear and axial loading through extensive finite element analysis.

In view of the limited research on the effect of the headed stud shear connectors on the composite beam; this thesis intends to contribute to this deficiency by carrying out finite element analysis to simulate the effects of steel fibres, elevated temperatures, long term effects, strain regimes and the combination of tensile axial and shear loading. It intends to show that all the abovementioned effects may be feasible and beneficial for the industry to take into consideration to allow headed stud shear connectors or concrete anchors to be used for modelling applications and to appraise the possibility of the current design rules.
1.3 OBJECTIVES AND SCOPE OF THESIS

The research work carried out in this thesis is to investigate the issues which have been discussed in Section 1.2 so as to provide further insight and understanding on those issues. The work mainly considers numerical studies compared with existing experimental results. In particular, the emphasis is on determining the design and behaviour of headed stud shear connectors for composite steel-concrete members, in terms of strength, stiffness and ductility. The main objectives and contributions to be derived from this thesis are outlined as follows:

(1) To look at the effects of steel fibres as a strengthening system in the shear connector of composite steel-concrete beams.
(2) To study the effects of elevated temperatures on the behaviour of headed stud shear connectors for composite steel-concrete beams
(3) To evaluate the performance of headed stud shear connectors when long term loading is considered
(4) To accurately determine the shear strength capacity of the headed stud connectors especially for profiled slabs.
(5) To observe failure effects on headed stud shear connectors when combining tensile axial and shear loading
(6) To develop an accurate design curve based on the finite element analysis to simulate the design and behaviour of composite steel-concrete beams.
(7) To undertake a systematic parametric study based on a calibrated model to assess the effects of various parameters influencing the behaviour of the beams using finite element analysis.
To review the validity of appropriate design methods on their suitability and applicability for use in composite steel-concrete structures which utilise the headed stud shear connector fundamental characteristics of strength, stiffness and ductility. Recommendations based on numerical results are also proposed.

1.4 LAYOUT OF THESIS

The composition of the thesis is organised into eight main chapters. Each chapter commences with an introduction to present an overview of the contents within that particular chapter. At the end of the chapter, the key findings and results which are attained are summarised.

Chapter 2 presents a comprehensive review on the existing research work which has been published in the open literature and which relates to the areas of interest in this thesis. This not only includes the important findings and results based on various experimental and analytical work, but also the relevant design rules and guidelines in international codes and standards. The literature review is basically divided into five main categories which are concerned with the issues of steel fibres, elevated temperatures, long term effects, strain regimes in concrete elements for composite steel-concrete beams and various load combinations of composite steel-concrete structures.

Chapter 3 explains the mechanical behaviour of materials in composite steel-concrete beams which include concrete, structural steel beam, profiled steel sheeting, steel reinforcement and stud shear connectors. Generally constitutive laws are used
to define stress and strain characteristics of a material. The accuracy of the analysis heavily depends on the constitutive laws involved to define the mechanical behaviour. The comprehensive details of the material properties for finite element analysis are described in the chapter. Author also explained which constitutive model is the most accurate for the finite element model.

Chapter 4 described the finite element analysis developed to simulate the response of a push test for composite steel-concrete beams. The finite element method is comprised of three major phases. They include pre-processing, solution and post-processing. In this chapter, the pre-processing and solution will be explained in detail. The pre-processing, in which the author develop a finite element mesh to divide the subject geometry into sub domains for mathematical analysis, and apply material properties and boundary conditions. The solution procedure of the program involving the governing matrix equations from the model and solution for the primary quantities. It is also important to recognise the limitations of FEA. The finite element method can reduce product testing, but cannot totally replace it. Therefore in this thesis, the author compared the finite element model against independent experimental studies undertaken by various researchers.

Chapter 5 explains the existing push test experimental studies conducted by various researchers, along with the details of the test specimens, material properties, instrumentation arrangement, test set-up and loading procedures are described herein. The finite element model is then compared with existing experimental results of push tests from various researchers. Verifications against existing experimental results are essential to demonstrate the accuracy and reliability of the finite element model.
Using a properly calibrated model, a set of systematic parametric studies is also undertaken to further investigate the influence of varying different parameters on the behaviour of the headed stud shear connectors.

In this chapter, the author also explains the importance of accuracy and conservatism. The validity of several existing methods and design models was evaluated in this chapter with regard to their applicability in the construction industry. The three fundamental properties which were investigated are the shear strength capacity, the initial stiffness and the slip capacity.

Chapters 6, 7, 8 and 9 are similar to Chapter 5 but the author look at the problems concerning the effects of various issues on the behaviour of stud shear connectors for composite steel-concrete beams. Chapter 5, 6, 7, 8 and 9 study the effect of steel fibres, long term effects, elevated temperatures, strain regimes and the combination of tensile axial and shear loading on the behaviour and design of headed stud shear connectors respectively.

Chapter 10 provides a summary of the conclusions drawn from the comparison of the finite element models and existing experimental data in the present study. Recommendations and suggestions for further research are also provided.
1.5 SUMMARY OF CHAPTER

This chapter has presented a synopsis of the research work to be observed in this thesis. The general layout and contents of each chapter of the thesis have also been depicted. The lack of understanding of several pertinent issues is acknowledged and this allows the scope and objectives of the thesis to be defined.
4 x 4 reinforcing mesh are provided both ways.

Figure 1.1 Details of Push Test Specimen for Solid Slab
4 x 4 reinforcing mesh are provided both ways.

Figure 1.2 Details of Push Test Specimen for Profiled Slab
Figure 1.3 Position of Shear Connectors in Push Test Specimens
CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

This chapter presents a summary of the existing research work and studies published in the literature which are relevant to the area under discussion in this thesis. It attempts to provide a summary of the important findings and conclusions from those studies and to highlight the existing gaps in knowledge which are to be investigated in this thesis. The development of composite structures is not extensively covered herein because it has been considered to be comprehensively documented in publications by Johnson (1994), Oehlers and Bradford (1995), Viest et al. (1997), Uy and Liew (2003) and Nethercot (2003).

An essential component of a composite beam is the shear connection between the steel section and the concrete slab. This connection is provided by mechanical shear connectors, which allows the transfer of forces in the concrete to the steel and vice versa and also resists vertical uplift forces at the steel-concrete interface. For solid slab, the shear connectors are welded to the top flange of the steel beam before the slab is cast. These connectors ensure that the two different materials that constitute the composite section act as a single unit. A variety of shapes and devices have been in use as shear connectors and economic considerations continue to motivate the
development of new products. Presently, the headed stud is the most widely used shear connector in composite construction. It’s popularity stems from proven performance and the ease of installation.

Ollgaard et al. (1971) undertook an experimental investigation involving the testing of 48 push out specimens with 16 mm and 19 mm studs embedded in the concrete slab. The experiments revealed that the ultimate strength of the shear connector was influenced by the compressive strength and modulus of elasticity of concrete. From the experimental investigation, the authors derived the following empirical equation shown in Equation 2.1:

\[ Q_u = 1.106 A_s f'_c^{0.3} E_c^{0.44} \]  

(2.1)

where:

- \( Q_u \) = Ultimate shear capacity of the stud connector (kN)
- \( A_s \) = the cross-sectional area of the stud connector (mm\(^2\))
- \( f'_c \) = Specified concrete compressive strength (N/mm\(^2\))
- \( E_c \) = Elastic modulus of concrete (N/mm\(^2\))

In order to simplify the equation for design purposes, Ollgaard et al. (1971) proposed the use of Equation 2.2 below. The equation provides the headed stud shear capacity based on the failure of concrete crushing. It has been adopted by the American Institute of Steel Construction AISC (1999) in their Load and Resistance Factor Design (LRFD) standard for evaluating the nominal strength of a stud connector embedded in a solid concrete slab.

\[ Q_u = 0.5 A_s \sqrt{f'_c E_c} \]  

(2.2)
As mentioned in Chapter 1, Grant et al. (1977) investigated the behaviour of composite beams with profiled slabs and the shear connector strength. A modification to the equation developed by Fisher (1970) was made to include the height of the stud shear connector. The strength of the stud in the ribs of composite beams with profiled steel slab can be determined by Equation 2.3.

\[
Q_{rib} = \frac{0.85}{N} \left( \frac{H - h}{h} \right) \left( \frac{w}{h} \right) Q_{sol} \leq Q_{sol}
\] (2.3)

where,

- \( Q_{rib} \) = the strength of a stud in formed steel deck
- \( N \) = the number of studs in a rib
- \( H \) = the height of stud shear connector
- \( h \) = the height of rib
- \( w \) = the average rib width
- \( Q_{sol} \) = the strength of the stud shear connector in a flat soffit slab

In this chapter, the main issue of the design and behaviour of headed stud shear connection for composite steel-concrete beams is discussed, particularly on the subject of the effect of steel fibres added into the concrete slab, the effect of elevated temperatures, the long-term effects, the effect of strain regimes on the concrete element, and the effect on the combination of tensile axial and shear loading. The major part of this thesis focuses on the numerical simulation of the behaviour of headed stud shear connection on composite steel-concrete beams.
2.2 EFFECTS OF STEEL FIBRES

2.2.1 Introduction

The use of steel fibres in reinforced concrete has a long history, starting from 1874, when builders began using steel waste to add to concrete mixes, but in the 1920’s, due to some concerns with regard to the inconsistency of steel fibre distribution, high material cost, corrosion, and the rules of proportioning being unclear, fibres did not gain much popularity. It was not until the 1960’s that the building industry began to gradually overcome these concerns. By the 1990’s, rapid growth in the use of steel fibres in European countries was evident, for example, 80 % of flooring in the Netherlands and 25 % in Germany used steel fibres.

2.2.2 Effects of Steel Fibres on Concrete Slab

Swamy and Al-Ta'an (1981) conducted a series of experimental studies which consisted of 15 reinforced concrete beams using steel fibres. The experiments were undertaken to study the influence of steel fibres on the deformation characteristics and ultimate flexural strength of concrete beams. The beam sizes were consistent, whilst the variables in the experiment were the type and amount of tensile steel bars and the volume and location of the steel fibres in the concrete. The steel fibres were provided either over the whole depth of concrete or in the tension zone of the concrete.

Swamy and Al-Ta'an (1981) showed that steel fibres are effective in resisting deformation in all stages of loading starting from the first crack to failure. It was also shown that the beams with steel fibre volume over the full depth displayed greater
resistance in deformation than those with steel fibres in the tension zone. It was confirmed that the steel fibres increased the flexural rigidity which inhibited crack growth and widening. Another conclusion was that steel fibres only have minimal impact on the ultimate flexural strength. The maximum increase was 10.5%. The authors stated that this was due to the very high yield strength bars used in the beams. Swamy and Al-Ta'an (1981) stated that since the ultimate strength increments were only modest, in a practical sense, it was not economical to use fibres to achieve a high ultimate strength.

Narayanan and Darwish (1987) investigated the use of steel fibres as shear reinforcement. They tested 49 beams and loaded under four symmetrical concentrated loads. Six of the beams were constructed without web reinforcement, 10 beams were reinforced conventionally with stirrups along the shear span and 33 beams contained crimped steel fibres as web reinforcement with different volumes of fibres. The tests concluded that the addition of steel fibres improved the resistance to the growth of cracks and at the same time increased the tensile strength. They also observed that concrete without reinforcement and steel fibres would fail immediately after the formation of the first crack. On the other hand, beams with stirrups and steel fibres continued to resist higher shear stresses beyond the first cracks.

Several diagonal cracks in fibre reinforced concrete were observed which indicated that the redistribution of stresses occurred beyond cracking. The steel fibres resisted tensile stress until they were completely pulled out from the concrete. Shear cracks between conventional reinforced concrete and fibre reinforced concrete were not significantly different, but the spacing of cracks seems to be closer for fibre
reinforced concrete. It was also observed that spalling occurred in conventional reinforced concrete, but was totally eliminated for fibre reinforced concrete. The results also proved that fibre volume caused different failures to concrete beams. If the volume of fibres is less than 1%, shear failure can occur, but flexural failure will occur when more than 1% volume of fibres was provided. If more than 1% volume of fibres is provided, longitudinal reinforcement has to be increased to prevent flexural failure. The presence of fibres also improves the effectiveness of dowel resistance by enhancing the tensile strength of concrete. The splitting plane along the reinforcement is decreased. A decrease of curvature when fibres are added to the concrete indicates that it can improve the ductility of the concrete beam.

Haselwander et al. (1995) used steel fibres in reinforced concrete members to compare the experimental test and theoretical value. The authors proved that by adding steel fibres into the concrete structures, the formation of cracks was not prevented, but the evenly distributed steel fibres inhibited increases in crack length and width when exposed to impact and vibration. Haselwander et al. (1995) performed the tests with two identical beams, one with fibres and another without fibres. Both of the beams were impacted with a falling mass from 20 m fall height. The results of the test are shown in Figure 2.1 which compares the beam deflection versus time relationship for the two beams. From Figure 2.1, it is noticeable that the beam with steel fibres has a smaller deflection. It also showed a 50% increase in the deflection of the beam. The authors concluded that steel fibres can improve the stiffness of the beam.
Casanova and Rossi (1997) proposed a design method for fibre reinforced concrete in structural elements. Their main focus was on the analysis of cracked section element, accurately characterising the material used and defining the design method. The authors design method was based on mechanical analysis concerning cracked sections. They are material limit states which include limiting crack openings in tension, post-cracking stress level in tension and strain limit in compression. From their analysis, Casanova and Rossi (1997) stated that bending and shear loading showed that the proposed design method and experimental data obtained were agreeable, but not for concentrated load. They suggested that more experimental data was required for evaluation of its relevancy.

Further studies were conducted by Lim and Oh (1997) concerning the shear strength of concrete. Their experiment involved three series of reinforced concrete beams having an identical cross section with a total of nine beams with variables being steel fibre volume fraction and content of the stirrups. The steel fibres varied from 0 % to 2 % and the ratios of the stirrups were from 0 % to 100 %. The beams were tested under a four point loading condition, and the load applied to the test beam was two equal concentrated loads by means of a steel spreader beam. The authors found that the compression strength, splitting tensile strength and flexural strength were increased by 25 %, 2 % and 55 %, respectively, when 2 % of steel fibres were added. The additional observation from Lim and Oh (1997) was spalling of the concrete. The concrete with steel fibres held the matrix together during the post-cracking stage to prevent spalling. The authors concluded that the addition of steel fibres reduced the amount of reinforcement required.
2.2.3 Effects of Steel Fibres on Composite Steel-Concrete Beams

Martin (2003) stated that currently, composite structures have been commonly used in construction in order to save time in construction and subsequently reduce structural costs. Composite beam designs are greatly affected by the behaviour of the shear connection. The main factors affecting shear connection are the strength of the shear connectors and the strength of the concrete that surrounds the shear connectors. In the 1960’s, welded wire fabric (WWF) was used in composite beams as secondary reinforcement for temperature and shrinkage in the concrete. The apparent problems that arose were the correct positioning of the WWF and the cost of labour and time. One improvement to the existing concept is with the application of steel fibres. Steel fibres are preferred to steel reinforcement or mesh because they reduce dead loads; subsequently they reduce the load transferred to the foundations, and at the same time improve the moment capacity, fire resistance and control cracking.

Craig et al. (1985) compared the behaviour of composite steel-concrete beams using steel fibres and conventional reinforcement. The authors established that composite steel-concrete with steel fibres could sustain higher loads than plain concrete. When steel fibre reinforcement was added to the concrete, the concrete exhibited improved confinement and improved bond. Moreover, steel fibres also enhanced the rotational and moment capacity.

Lam and Nip (2002) performed horizontal push off tests on six composite beams. The parameters changed included transverse reinforcement and steel fibres. The push off specimens comprised four 600 mm wide hollow core units connected to a 254 x 254 x 73 universal column with six 19 mm diameter x 100 mm height pre-welded
headed studs at 150 mm centres. Four 1100 mm long transverse bars were placed in the open cores and in-situ concrete was placed and compacted in the central gap. Details of the composite beam specimens and results are given in Table 2.1. The experimental results showed that by adding 2% of steel fibres, both compressive and flexural strength increased by 25% and 20%, respectively. Shear stud failure occurred where transverse bars of T16 were used and with others, there was concrete failure around studs. All concrete with steel fibres had a higher observed test load than their counterparts. Meanwhile, there was a greater slip ductility where the increment was up to 75% and the shear capacity was at 16% increment. This thesis will emphasise the effect of steel fibres on the behaviour of headed stud shear connection on composite steel-concrete beams for both solid and profiled slabs.

Robery (2002) stated that the combination of steel fibres with composite slabs not only increased structural stiffness and ductility, they also provided slabs with fire rating ranging from 60 to 90 minutes. Two aspects are to be considered when steel fibre reinforced concrete is used in composite beams; shear stud resistance, ductility, and the ability of steel fibre reinforced concrete to resist transverse shear in the slabs adjacent to the shear studs. The experiments undertaken by Robery (2002) were to determine the optimum economic quantity of steel fibre reinforced concrete. By using only 30 kg/m$^3$ of steel fibre, the shear stud resistance specified in Eurocode 4 British Standards Institution (2004) needs to be slightly downgraded. The reduction factor for Eurocode 4 is 0.96. Even though the reduction factor was used, the shear studs were still considered to be ductile. Detailed investigation was not made for using a higher density of steel fibres to reduce shear stud resistance. This thesis examines this effect.
According to Roberts-Wollmann et al. (2004), using steel fibres to replace welded wire fabric (WWF) as secondary reinforcement reduces both deflection and cracks in composite beams. The authors also demonstrated that it is preferable to use steel fibres rather than synthetic fibres. The slab reinforced with 29.6 kg/m³ of steel fibres had a higher ultimate strength (18 %) than a slab reinforced with WWF. Using more steel fibres also resulted in higher ultimate strength with approximately 37 % higher [29.6 kg/m³ to 14.8 kg/m³]. The authors also showed that the composite beam reinforced with 0.9 kg/m³ of synthetic fibres failed at a load equivalent to WWF. Both studies did not consider the profiled slab. This thesis examines the influence of steel fibres for profiled slab as a potential continuation of Roberts-Wollmann et al. (2004) research.

2.3 EFFECTS OF LONG TERM BEHAVIOUR

2.3.1 Introduction

In composite steel-concrete structures, the concrete element is generally unique among other structural materials in that it interacts with its environment undergoing inevitable complex physical and chemical volume changes. The process of hydration and the molecular structure of the concrete element give certain characteristics to this material such as aging, creep and shrinkage. These characteristics as a group are known as long-term deformations. Even though much research such as Troxell et al. (1958), Hanson (1968), Takahashi and Kawaguchi (1980), Neville et al. (1983), Bazant and Prasannan (1989), Parott (1991) and Granger and Bazant (1995) have been devoted to creep and shrinkage analysis and despite the major successes, this complex problem is still far from being fully understood, especially when concerned with composite steel-concrete construction.
Under sustained stress, the strain in the concrete element of a composite steel-concrete beam will progressively increase with time due to creep and shrinkage. Even though creep and shrinkage of concrete is considered to cause deformations with increasing time, there is no evidence of significant damage to the concrete material. Concrete structures whose long-term serviceability has been compromised by creep deformation can result in a drastic reduction of their design life span (Bazant and Tsubaki (1980) and Mazzotti and Savoia (2003)).

2.3.2 Effects of Creep and Shrinkage

Long term behaviour consists of creep and shrinkage. They significantly affect the structural behaviour at service loads. Creep causes the strain in concrete to gradually increase with time under sustained stress. Compared with instantaneous deformation, the deformation of a member increasing with time may develop to many times higher than the deformation due to load. Gradual development of creep increases the curvature and this can result in an increase in the beam deflection. The age of concrete at the instant of loading influences the development of creep. Furthermore, creep also depends highly on humidity, temperature and the surface area which is exposed to the environment.

Shrinkage is a reduction in volume caused mainly by the loss of moisture during the drying process. Besides loss of moisture, chemical reactions also develop shrinkage. Shrinkage is known as the time dependent strain measured at constant temperature in an unloaded and unrestrained specimen. The progress of shrinkage starts at time \( t_s \), when curing of concrete stops and strain is developed at time \( t > t_s \), it can be defined as
\[ \varepsilon_s(t, t_c) = \beta \varepsilon_{t_0} \]  \hspace{1cm} (2.4)

where,

\[ \varepsilon_{t_0} = \text{the strain that occurs after concrete hardening up to time infinity} \]

\[ \beta = \text{constant parameter that depends on the type and duration of curing} \]

A typical stress-strain curve for concrete is shown in Figure 2.2. Under service load conditions, a linear relationship is assumed. The instantaneous strain \( \varepsilon_c(t_0) \) that occurs immediately after the stress application \( \sigma_c(t_0) \) at time \( t_0 \) can be described in Equation 2.5.

\[ \varepsilon_c(t_0) = \frac{\sigma_c(t_0)}{E_c(t_0)} \]  \hspace{1cm} (2.5)

where,

\[ t_0 = \text{the time of application of the stress} \]

\[ \sigma_c(t_0) = \text{the applied concrete compressive stress} \]

\[ E_c(t_0) = \text{the modulus of elasticity of concrete at age } t_0 \]

When a sustained stress is applied at time \( t \), the strain increase with time due to creep is defined in Equation 2.6 below and shown in Figure 2.3.

\[ \varepsilon_c(t) = \frac{\sigma_c(t_0)}{E_c(t_0)} \left[1 + \varphi(t, t_0) \right] \]  \hspace{1cm} (2.6)

From Figure 2.3, the creep coefficient can be defined as the ratio of creep strain to instantaneous strain where this value depends heavily on the age of concrete when the stress is first introduced. The value increases according to the age of concrete but reduces at a descending rate and diminishes after 2 to 5 years. Depending on the
concrete strength, the final strain value is about 1.2 to 3 times the magnitude of the instantaneous strain.

For increasing stress histories, the principle of superposition concurs with experiments undertaken by Gilbert (1988) where the creep curve produced by the increasing stress history is assumed to be equal to the sum of the creep curve produced by each stress increment acting independently as in Figure 2.4. Figure 2.4 demonstrates that when applied stresses changes with time, the total strain of concrete is given by:

$$\varepsilon_c(t) = \varepsilon_c(t_0) + \varepsilon_c(t,t_0) + \delta\varepsilon_c(\tau_i) + \delta\varepsilon_c(t,\tau_i) + \varepsilon_{cs}(t)$$

(2.7)

where,

$$\varepsilon_c(t) = \text{the total strain in concrete at time } t$$

$$\varepsilon_c(t_0) = \text{the instantaneous strain in concrete at time } t_0 \text{ due to initial stress applied at time } t_0$$

$$\varepsilon_c(t,t_0) = \text{the creep strain in concrete at time } t \text{ due to initial stress applied at time } t_0$$

$$\delta\varepsilon_c(\tau_i) = \text{the instantaneous strain in concrete at time } \tau_i \text{ due to increment stress applied at that particular time.}$$

$$\delta\varepsilon_c(t,\tau_i) = \text{the creep strain in concrete at time } t \text{ due to increment stress applied at time } \tau_i$$

$$\varepsilon_{cs}(t) = \text{the shrinkage strain in concrete at time } t$$

Due to the limited experimental data available regarding the strength of concrete due to creep, a simple model from CEB-FIP (1990) was used for the creep and shrinkage damage strength interaction of concrete. The stress and strain law is used
to describe the concrete behaviour as a function of its strength. The different ageing
time of stress and strain law of concrete is in Figure 2.5. From Figure 2.5, the
concrete with stiffness and strength can be evaluated at:

- **time** $t_0$, where

$$
\varepsilon_{\text{el,d}}^{\text{el}}(t_0) = \text{the effective strain which is the sum of instantaneous strains}
$$

$$
\varepsilon_{\text{eff}}^{\text{v}}(t_0, \Delta t) = \beta \text{ is a fraction of viscous strain which is obtained from an age of}
$$

loading at time $t_0$

$\Delta t$ is the time under loading which is $t_r - t_0$

- **time** $t_r$, where

$$
\varepsilon_{\text{el,d}}^{\text{el}}(t_r) = \text{the effective strain which is the sum of instantaneous strain}
$$

$$
\varepsilon_{\text{eff}}^{\text{v}}(t_r, \Delta t) = \beta \text{ is a fraction of viscous strain which is obtained from an age of}
$$

loading at time $t_r$

$\Delta t$ is a time under loading where $\varepsilon_{\text{el,d}}^{\text{el}}(t_0) > \varepsilon_{\text{el,d}}^{\text{el}}(t_r)$ and $\varepsilon_{\text{eff}}^{\text{v}}(t_0, \Delta t) > \varepsilon_{\text{eff}}^{\text{v}}(t_r, \Delta t)$

to produce $\varepsilon_{\text{eff}}^{\text{v}} > \varepsilon_{\text{eff}}^{\text{v}}$

When time dependent analysis is considered, the Poisson’s ratio also changes
according to time. Various proposals have been presented by Krajcinovic and
Fonseka (1981) and di Prisco and Mazars (1996) to define the Poisson’s ratio
evolution as a function of longitudinal strain. The models that best fit many
experimental results were undertaken by Kupfer et al. (1969) and Mazzotti et al.
(2000) known as regression models called a power law as is shown in Figure 2.6.
2.3.3 **Effects of Creep and Shrinkage on Composite Steel-Concrete Beams**

Time-dependent analysis for composite steel and concrete beams with shear connectors including slip between steel and concrete interface from Tarantino and Dezi (1992) proved that the stresses in the concrete depend on the stiffness of the connection system. In contrast, Bradford (1991) affirmed that due to creep and shrinkage, the shear force per unit length on connectors tends to decrease with time and most importantly, the stiffness connectors have less influence on the deflection. Bradford and Gilbert (1989) and Bradford and Gilbert (1992) used a simplified approach to evaluate creep and shrinkage effects in steel concrete composite beams based on the age-adjusted effective modulus to model the stress and strain relationship for concrete where aging coefficient, $\chi$ was introduced, and verified that residual stress resulted in a decrease in concrete stiffness. Moreover, it also demonstrated that the effect of creep and shrinkage dominated the reduction in concrete stiffness at service load. The authors were supported by an analytical model with layer approach done by Kwak et al. (2000)

Gilbert and Bradford (1995) studied the behaviour of continuous composite beams under sustained service loads for both short and long-term cracking. The analysis took into account the cracking of the concrete slab in the negative moment regions. The authors conducted a series of full scale continuous composite beams under uniformly distributed load. To verify the numerical model, Gilbert and Bradford (1995) tested several beams over a period of 340 days. Figure 2.7 illustrates that the measurements for both short and long-term deflections compared with the theoretical values. These measurements were used to validate the theoretical model. The authors commented that in negative bending, the slab with tension cracks under
service load had less contribution to strength, and its stiffness had largely decreased. From Figure 2.7, it can be observed that both experiment and theoretical deflections are in good agreement. Gilbert and Bradford (1995) concluded that the time-dependent deflections were caused by creep and shrinkage in the concrete. Further studies are implemented in the thesis herein by examining the behaviour of headed shear stud connector in composite steel-concrete beams when long-term analysis is taken into consideration.

2.4 EFFECTS OF ELEVATED TEMPERATURES

2.4.1 Introduction

Fire is a very complex phenomenon which can cause structural damage. Fires can occur at any time in buildings, and the safety of occupants and maintaining the integrity of the structure are of major importance. The response of a structural member exposed to fire is governed by the rate that it is heated, and this is because the mechanical properties of the material decrease as the temperature rises and likewise, the structural resistance of a member reduces with temperature rise. Fire safety design is an important aspect of building design because a properly designed building system greatly reduces the hazards to life and limits property loss. Inberg (1928) stated that the research on fire safety design started almost 80 years ago.

When a composite steel-concrete beam is subjected to fire, both the structural steel beam and concrete slab are exposed directly to fire. On the other hand, shear connectors are indirectly heated by heat transfer from the structural steel which is illustrated in Figure 2.8. Fire will cause these elements to lose their mechanical
strength with respect to the temperatures reached. However, the mechanical behaviour of composite beams exposed to fire is much more complicated because of the different materials present.

2.4.2 Effects of Elevated Temperatures on Composite Steel-Concrete Beams

For most existing international standards, the rules for building structures design under fire exposure are based on the response of isolated members tested under fire. The design is similar to the traditional design method for ambient temperatures. In reality, when composite steel-concrete beams exposed to fire, their behaviour is greatly dependant on the heat transfer through the structural members, the surrounding structures and also the headed shear connectors. Heat transfer dominates the behaviour of composite steel-concrete building when subjected to fire. This has been highlighted by numerous publications by researchers Cooke et al. (1988), Usmani et al. (2001), Huang et al. (2004), Yu et al. (2006), Lamont et al. (2004) and Lawson (2001). Most of these studies were based on numerical modeling and comparison with experimental studies using Cardington Test. The aim of this thesis is to highlight on the design and behaviour of headed stud shear connectors for both composite steel-concrete beams under elevated temperatures.

Sanad et al. (1999) performed a set of four full-scale multi-storey composite steel-concrete structures fire tests. The experimental tests were conducted on a building with profiled slab. The experimental studies concentrated on the heat transfer for both concrete slab and structural steel beams. The authors also focused
on the deflection at the mid-span of the composite steel-concrete beam when exposed to fire.

Lamont et al. (2001) highlighted the importance of the temperature evolution in determining the structural response. They also carried out four British Steel fire tests on the 8 storey composite steel-concrete building at Cardington. The authors detected that there is no adequate temperature attained in the concrete slab. Therefore, Lamont et al. (2001) used a finite element heat transfer program known as HADAPT to model the heat transfer for composite steel-concrete slabs. The deficiency of the paper mentioned above was that both Sanad et al. (1999) and Lamont et al. (2001) did not draw attention to the behaviour of headed stud shear connectors when composite steel-concrete beams were exposed to fire. This thesis emphasised on the behaviour of headed stud shears connector when exposed to fire.

2.4.3 Effects of Elevated Temperatures on Shear Stud Connectors

Zhao and Aribert (1992) conducted a series of experimental push tests subjected to elevated temperatures. The varying parameters included the concrete compressive strength and structural steel beam as shown in Table 2.2. Table 2.2 shows that the ultimate shear capacity is greatly dependant on the concrete compressive strength, $f'_c$, diameter of shear connectors, $d$ and size of structural steel beams. The authors also proposed an analytical study to determine the accuracy of heat transfer through the push test. Figure 2.9 proves that the calculated value and experimental data are in good agreement. It should be borne in mind that the authors only used solid slabs for the experimental studies and did not look at the effects of headed stud shear
connectors. The thesis described herein expands Zhao and Aribert (1992) research by considering both solid and profiled slabs and also studying the behaviour of headed stud shear connectors when exposed to elevated temperatures.

Zhao and Kruppa (1996) investigated the behaviour of shear connectors and composite beams under elevated temperatures. Three main series of push out tests were performed. They included 12 push tests performed at room temperature in order to obtain the ultimate shear capacity, 4 push tests were carried out with the increase in temperature without load to determine the influence of elongation due to temperature and finally, 31 push tests were conducted at elevated temperatures with different loading conditions. The experiments have established the evolution of shear capacity of different types of connectors as a function of temperature as well as their force-slip relationships at elevated temperatures. The third series of experiments provided the evidence in regards to failure of the headed stud shear connectors and phenomenon of local instability of continuous composite beams at elevated temperatures. This thesis will further investigate the effects of elevated temperatures on the behaviour of headed stud shear connector. Furthermore, an investigation of headed stud shear capacity with respect to fire time exposure will be explored herein.
2.5 EFFECTS OF STRAIN REGIMES

2.5.1 Introduction

When a push test is carried out, the shear connectors are subjected to pure shear, however in comparison, when a composite steel-concrete beam is subjected to bending, the shear connectors will be subjected to both bending and shear. For a simplified explanation, consider the push test with a horizontal axis of symmetry, shown in Figure 2.10. The profiled slabs in Figure 2.10 has been modified to illustrate the realistic stresses and thus the effects of torque on the ribs. A horizontal line of the cross-section at the interface of the concrete slab and structural steel beam will be referred to as the neutral axis of the composite steel-concrete beam. Next consider a typical element of the composite beam between two planes perpendicular to the beam’s neutral axis. From an elevation, the element is denoted as aabb. When the element is subjected to loading in the web of the structural steel beam, shown in Figure 2.10a, it will be pushed as illustrated in Figure 2.10b. However section aabb still remains undeformed. Therefore, the structure is in pure shear where critical loading conditions are applied to the shear connectors in composite steel-concrete beams. In this case, the strain will remain constant along the section, as shown in Figure 2.10c.

When a composite steel-concrete beam is subjected to bending moments, consider the beam with a vertical axis of symmetry given in Figure 2.11. The horizontal line of the cross-section is similar to the push test explained above. When the element is subjected to loading on the concrete slab shown in Figure 2.11a, the structure will bend as illustrated in Figure 2.11b, and then the section cedd deforms
to become \( c'e'd' \). Thus, the cross-section will be subjected to bending. For this case, the strain gradually changes throughout the depth of the section as illustrated in Figure 2.11c.

One should borne in mind, when steel-concrete composite beam is loaded bending moment jointed by headed stud shear connectors, the shearing of junction between steel and concrete is in existence, the section does not satisfies the hypothesis of plane section any more, so it is necessary to make clear the distribution law of section shear stress. Figure 2.11c illustrated that the strain distribution shown is in worst case scenario. In this research, the strain difference of steel and concrete composite beam under pure shear and bending moments is analysed on the basis of elastic theory and is compared with results finite element analyses. Therefore it can be used to instruct the design and construction of practical engineering.

### 2.5.2 Effects of Strain Regimes on Shear Connectors

The behaviour of shear connectors in terms of static and fatigue strength and load-slip relationships is generally obtained by means of push-out tests and composite beam tests. Carlsson and Hajjar (2000) mentioned that results from beam tests are more difficult to analyse as the connectors are loaded by various forces and affected by residual stresses and non-linearity of the concrete slab and steel beam. Results from push out tests were generally more conservative than beam tests. This can be mainly attributed to the force distribution acting on the connectors. In push out tests, the connectors are primarily subjected to direct shear, but in beams the connectors
are acted upon by a complex interaction of axial force, shear force and bending moment.

Furthermore, some additional strength may also be derived from the reduction of longitudinal shear force due to the presence of frictional force at the slab-beam interface. This was highlighted by Seracino et al. (2004). The implication was that this would increase the endurance of the shear connectors and extend the design life of the connectors.

However, even as push out tests have gained wide acceptance for experimental purposes, Oehlers and Bradford (1995) pointed out that the results from these tests varied widely. This was due to the use of different sizes, shapes, material properties, arrangement of specimens, number and position of connectors, and support restraints. The different modes of failure such as failures in the stud shank, weld, steel flange and the profiled sheeting further increased the scatter of results. In addition, shear, embedment and splitting failures of the slab also had an effect. These secondary non-ductile failure modes can be avoided by providing proper detailing, adequate cover and reinforcement.

Hicks (2007) has undertaken both beam tests and push tests with trapezoidal decking to study the resistance and ductility of shear connector. The main objective in the design of the beam specimens was to provide the most unfavourable combination of factors that may occur in current practice to quantify the level of enhancement to the shear stud connectors resistance when compared with the push tests. Moreover, the author focused on the level of safety that exists within the
current composite construction. The experiments proved that the behaviour of the headed stud shear connector in the push test is more conservative in strength when compared with the beam tests and is shown in Figure 2.12. Hicks (2007) stated that any brittleness exhibited in a push test resulting to the deficiency in the push test specimen itself rather than the shear connection. Therefore, this thesis investigated the effects of strain regimes in the push test to acquire a more representative result as would be expected in beams.

2.6 EFFECTS OF THE COMBINATION OF AXIAL AND SHEAR LOADINGS

2.6.1 Introduction

Constant requirements for cost-efficient structural forms in engineering industry have led to the increasing used of composite steel-concrete structures. Due to the economic point of view, nowadays a lot of construction is implemented in this form of construction. In order to increase the strength and stiffness of composite steel-concrete structures, the slip between the structural steel beam and concrete slab has to be minimised. Therefore, one way of achieving this is by welding the headed stud shear connectors in the upper flange and embedding them onto the concrete slab.

The increasing use of headed stud shear connectors under a combination of axial and shear loading has resulted in a need for more research on their behaviour and strength for the composite steel-concrete beams. For flooring systems, the headed studs are normally subjected to combined shear and bending loading. This will be investigated by the author herein. As for wall system, the headed studs are subjected to shear and tensile axial loading. The tensile axial loading will cause both the
concrete and headed stud to be exposed to tensile failure. There is a lot of researchs looking at the behaviour of headed studs when subjected to shear loading (Becher (2005), El-Lobody and Young (2006), Gattesco and Giuriani (1996), Hanswille et al. (2008), Johnson (2000), Lam and El-Lobody (2005), Wu (2006), Baskar et al. (2002).

The early years of developing composite steel-concrete beams, the partial shear interaction is briefly presented by Leon and Viest (1996) where three models based on analogous hypotheses were mentioned. The shear force and interface slip relationship were assumed behaving linearly based on study by Newmark et al. (1951). On the other hand, the force-slip relationship which characterises the headed stud shear connectors is behaving non-linearly according to Johnson and Molenstra (1991). Then Faella et al. (2003) conducted a nonlinear numerical procedure to take account for both shear connection behaviour and tension stiffening effect in the cracked zone of concrete slab. Some experimental comparisons showed the accuracy of the proposed numerical procedure. The authors did not consider the behaviour of headed stud shear connectors, therefore the thesis herein will show thses effects.

2.6.2 Effects of Combination of Axial Tensile and Shear Loading on Headed Stud Shear Connectors

According to Cook et al. (2007) and Wisser et al. (2000) the demand for greater flexibility in the planning, design and strengthening of concrete element in the composite steel-concrete system has resulted in an increase use of fastening systems. The fastening system includes headed studs or headed bolts. When headed studs are exposed to the tensile load, it can be observed that there are two major failure modes.
They include concrete failure and steel failure. Concrete failure happens when the embedment depth of the headed stud is too small. When the headed studs are embedded deeper in concrete, the combination failure modes of concrete and steel will be observed.

There are different types of failure modes when composite steel-concrete structures subjected to combination of shear and axial loading. According to Eligehausen et al. (2006), the different possibility of failure modes are divided into five types. They include concrete failure under axial force, concrete failure under shear force, local cone failure, steel failure if the headed stud and pull out of the headed stud. In this thesis, the headed studs and concrete subjected to both shear and axial force are the initial basis for this investigation.

Saari et al. (2004) had conducted experimental programmes to quantify the strength and deformation capacities of headed stud for the use of infill wall systems. A modification of traditional push out test was made to accommodate the application of shear and axial tensile loading. Due to cost limitation, limited experimental studies were performed. The experiments looked at two different steel reinforcement configurations. One provided little confining reinforcement around headed studs and the other used reinforcement cage to provide sufficient confinement to the headed studs. The limitation of this paper was that the experimental studies were not compared with finite element analysis studies. Therefore, the author is going to use the experimental results to compare with the finite element models.
Pryout failure can also occur when the headed stud is pulled out of the concrete for push test. According to Anderson and Meinheit (2005), when the headed stud are too short and stocky, the pryout failure will occur. Moreover, the authors also proved that the arrangement and spacing of headed studs and shear load applied played an important role for the effect of pryout failure. The authors concluded that the headed stud with $h_d/d$ less than 4.5 may invoke the pryout failure which causes the headed stud ultimate capacity to be less than that predicted in the existing standards. From this research, the authors found out that the well suited $h_d/d$ is 5.4 to 7.4.

Anderson and Meinheit (2000) proved that in order to get the maximum capacity of headed stud, the studs has to be well embedded in the concrete. Beside pryout failure, another failure which will affect the ultimate capacity of headed is the shear stud shank failure. Shear stud shank failure is governed by the ultimate tensile strength of the stud. They also proved that the headed stud cannot develop its full capacity based on $1.0A_sF_u$. The authors recommended that additional work should be undertaken to study the influence of other loading besides shear loading. Therefore, this thesis will utilise the headed studs when subjected to a combination of shear and axial tensile forces.

Due to the exposed to axial tension loading, Odenbreit and Fromknecht (2007) suggested that the concrete cone failure could be prevented by implementing the hanger reinforcement. The hanger reinforcement can be placed in the concrete to cover the tension force produced in the concrete by the headed studs. Figure 2.13 shows the hanger reinforcement to prevent concrete pull out due to axial loading.
Figure 2.14 illustrates the hanger reinforcement to prevent shearing of headed studs from shear loading.

2.7 SUMMARY OF CHAPTER

This chapter has presented an in-depth review of the research literature which is relevant to the objectives of the thesis. The limited experimental work and lack of research on the issues of steel fibres, elevated temperatures, long-term behaviour, strain regimes and combined axial tension and shear loading on the headed stud shear connection in the composite steel-concrete beam are evident.

The existing analytical work on composite beams was found to be extensive although most relied on finite-element methods which were either formulated using a force-based or displacement-based method or both. The numerical studies on headed stud shear connection are less established. This thesis describes a nonlinear finite element model developed using ABAQUS, Karlsson and Sonrensen (2006a), Karlsson and Sonrensen (2006b) and Karlsson and Sonrensen (2006c) to study the behaviour of shear connectors in both solid and profiled steel sheeting slabs. In addition to analysing the influence of the shear connectors on the structural performance, steel fibres are introduced to further augment the ductility and strength of the shear connection region in the slab.

This thesis also presents the development of an accurate finite element model using ABAQUS to study the behaviour of shear connectors in push tests incorporating the time-dependent behaviour of concrete. The behaviour of composite steel-concrete beams at elevated temperatures is an important problem. The
motivation for this thesis is to increase the awareness of the structural engineering community to the concepts behind composite steel-concrete structural design for fire exposure. The behaviour of reinforced concrete slabs under fire conditions strongly depends on the interaction of the slabs with the surrounding elements which includes the structural steel beam, steel reinforcing and shear connectors. This thesis is to consider the effects of elevated temperatures on the behaviour of the composite steel-concrete beams for both solid and profiled steel sheeting slabs.

The use of conventional push-out test results to represent the characteristics of the shear connectors in the solid slab was also found to be appropriate. This is not the case when the conventional push-out test was used for profiled slabs. To achieve a similar effect as solid slabs, the push out tests reported in this thesis considered a novel alternative of using different strain regimes. Furthermore, some of the findings on the behaviour of the shear connectors embedded in concrete which are cracked in tension are inconsistent, although recent tests and analyses of composite beams generally indicated that there was no reduction in connector strength.
Table 2.1 Beam Specimens and Results of the Push Test Series, Lam and Nip (2002)

<table>
<thead>
<tr>
<th>Ref</th>
<th>Headed Shear Studs (mm)</th>
<th>In situ Concrete Strength (N/mm²)</th>
<th>Transverse reinforcement</th>
<th>Steel Fibres</th>
<th>First cracking (kN)</th>
<th>Slip at Max. load</th>
<th>Mode of failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>PO1</td>
<td>19x100</td>
<td>C30</td>
<td>4T10</td>
<td>-</td>
<td>33</td>
<td>2.4</td>
<td>CF</td>
</tr>
<tr>
<td>PO2</td>
<td>19x100</td>
<td>C30</td>
<td>4T10</td>
<td>2 % by weight</td>
<td>40</td>
<td>4.2</td>
<td>CF</td>
</tr>
<tr>
<td>PO3</td>
<td>19x100</td>
<td>C30</td>
<td>4T12</td>
<td>-</td>
<td>46</td>
<td>2.3</td>
<td>CF</td>
</tr>
<tr>
<td>PO4</td>
<td>19x100</td>
<td>C30</td>
<td>4T12</td>
<td>2 % by weight</td>
<td>47</td>
<td>3.6</td>
<td>CF</td>
</tr>
<tr>
<td>PO5</td>
<td>19x100</td>
<td>C30</td>
<td>4T16</td>
<td>-</td>
<td>47</td>
<td>6.7</td>
<td>SF</td>
</tr>
<tr>
<td>PO6</td>
<td>19x100</td>
<td>C30</td>
<td>4T16</td>
<td>2 % by weight</td>
<td>53</td>
<td>8.1</td>
<td>SF</td>
</tr>
</tbody>
</table>
Table 2.2 Zhao and Aribert (1992) Push Test Specimens

<table>
<thead>
<tr>
<th>Test</th>
<th>$d$ (mm)</th>
<th>$f_c$ (MPa)</th>
<th>Steel Beam</th>
<th>$Q_u$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>19</td>
<td>34.5</td>
<td>HEAA300</td>
<td>124.6</td>
</tr>
<tr>
<td>2</td>
<td>19</td>
<td>34.5</td>
<td>HEAA300</td>
<td>125.3</td>
</tr>
<tr>
<td>3</td>
<td>19</td>
<td>30.3</td>
<td>HEAA300</td>
<td>135.0</td>
</tr>
<tr>
<td>4</td>
<td>19</td>
<td>42.2</td>
<td>HEAA300</td>
<td>144.8</td>
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<td>5</td>
<td>16</td>
<td>43.5</td>
<td>HEAA300</td>
<td>103.0</td>
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<td>22</td>
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<td>HEAA300</td>
<td>155.0</td>
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<td>7</td>
<td>19</td>
<td>35.6</td>
<td>HEAA260</td>
<td>126.0</td>
</tr>
</tbody>
</table>
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Figure 2.3 Creep Under Sustained Stress, Ghali et al. (2002)
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CHAPTER 3
MECHANICAL BEHAVIOUR OF THE CONSTITUENT MATERIALS

3.1 INTRODUCTION

Load-slip response for headed shear stud connectors has long been recognised as one of the significant factors affecting the strength and service capacity for composite steel-concrete structures. Even though considerable research for both analytical and experimental has been devoted to headed stud shear connectors behaviour, the identification and extent of the controlling parameters that govern the load-slip response have still not been clearly identified. A major requirement of any research programme designed to address these issues is a convenient, accurate, and reliable analysis methodology that will permit any design engineer to easily construct a computer model of a composite structure that will predict the strength capacity.

In general, constitutive laws are used to define the stress-strain characteristics of materials. The accuracy of the analysis is dependent on the constitutive laws used to define the mechanical behaviour. The primary objective in this thesis is to develop a reliable understanding of the mechanical behaviour for developing finite element push-out test models that can accurately predict the shear stud capacity. The finite element programme ABAQUS is used to quantify the behaviour of the shear
connection in composite beams. The main components affecting the behaviour of the shear connection in composite beams are the concrete slab, steel beam, profiled steel sheeting, reinforcing bars and shear connectors. These components must be carefully modelled to obtain an accurate result from the finite element analysis. A three-dimensional finite element model has been developed to simulate the geometric and material nonlinear behaviour of composite beams.

3.2 CONCRETE

3.2.1 Introduction

The core material of composite steel-concrete structures consists of plain concrete and shear studs welded to structural steel beams. Shanmugam et al. (2002) stated that due to the presence of a large number of shear studs in concrete, it behaves differently from plain or reinforced concrete. The authors also described that concrete can be modelled in using an anisotropic material model because this is generally used for materials that exhibit different yield or creep behaviour in different directions. As the analysis progressed, cracking of the concrete in tensile regions introduced instability in the numerical computations, which forced analysis to stop prematurely. The authors observation is supported by Marzouk and Chen (1993). In this thesis, the mechanical behaviour at ambient and elevated temperatures is considered. When elevated temperature is involved, the main properties required to carry out an accurate calculation of the temperature distribution in a composite cross-section are the specific heat, thermal expansion and thermal conductivity.
3.2.2 Concrete at Ambient Temperature

Plain concrete was recommended by Carreira and Chu (1985), where the stress in compression is assumed to be linear up to a stress of $0.4f'_c$. Beyond this point, stress is represented as a function of strain according to Equation 3.1.

$$\sigma_c = \frac{f'_c \gamma (\epsilon_c / \epsilon'_c)}{\gamma - 1 + (\epsilon_c / \epsilon'_c)^\gamma}$$  \hspace{1cm} (3.1)

where

$$\gamma = \left| \frac{f'_c}{32.4} \right|^3 + 1.55 \text{ and } \epsilon'_c = 0.002$$

For concrete in tension, the tensile stress is assumed to increase linearly relative to the strain until the concrete cracks. After the concrete cracks, the tensile stresses decrease linearly to zero. The value of strain at zero stress is usually taken to be 10 times the strain at failure, which is shown in Figure 3.1.

3.2.3 Thermal Properties of Concrete

3.2.3.1 General

An important design consideration for concrete includes the effects of fire. The behaviour of concrete slabs subjected to fire conditions is complex. In a fully developed fire, to prevent fire spread to the upper floors, the slab has to carry and withstand the applied loads and prevent collapse during and after the fire.

The effect of fire, which is not generally considered in typical structural design practice, involves the thermal conductivity, specific heat and high thermal expansion
of the concrete. This will cause the surrounding structure to respond against these effects and generate compressive forces in the heated concrete slab.

### 3.2.3.2 Thermal conductivity

Thermal conductivity is the capability of a material to conduct heat, and is defined as the ratio of heat flux to the temperature gradient. It represents the uniform flow of heat through concrete of unit thickness over a unit area subjected to a unit temperature difference between the two opposite faces Bazant and Kaplan (1996). The thermal conductivity of siliceous aggregate concrete as represented in Eurocode 2 British Standards Institution (2004), shown in Figure 3.2, and is defined as:

\[
\lambda = 2 - 0.2451(\theta/100) + 0.0107 \left(\frac{\theta}{120}\right)^2 \text{ (W/mK)} \quad \text{for } 20^\circ C < T < 1200^\circ C
\]  

### 3.2.3.3 Specific heat

The specific heat of a material, as defined by Harmathy (1970), is the amount of heat per unit mass which is required to change the temperature of the material by a degree. It is represented by Equation 3.3.

\[
C_p = \left(\frac{\partial H}{\partial T}\right)_p
\]

where

- \(H\) = Enthalpy,
- \(T\) = Temperature
- \(P\) = Pressure.
The specific heat of concrete with siliceous aggregates as a function of temperature according to Eurocode 2 British Standards Institution (2004) is represented using the equations below, and is shown in Figure 3.3.

\[ C_p(\theta) = 900 \text{ (J/kg K)} \quad \text{for} \quad 20^\circ\text{C} \leq \theta \leq 100^\circ\text{C} \]  
\[ (3.4) \]

\[ C_p(\theta) = 900 + (\theta - 100) \text{ (J/kg K)} \quad \text{for} \quad 100^\circ\text{C} \leq \theta \leq 200^\circ\text{C} \]  
\[ (3.5) \]

\[ C_p(\theta) = 1000 + (\theta - 200)/2 \text{ (J/kg K)} \quad \text{for} \quad 100^\circ\text{C} \leq \theta \leq 400^\circ\text{C} \]  
\[ (3.6) \]

\[ C_p(\theta) = 1100 \text{ (J/kg K)} \quad \text{for} \quad 400^\circ\text{C} \leq \theta \leq 1200^\circ\text{C} \]  
\[ (3.7) \]

### 3.2.3.4 Thermal expansion

Due to its isotropic nature, concrete exhibits thermal expansion when it is subjected to a temperature change. According to Bazant and Kaplan (1996), cracking occurs when stresses develop in concrete structures due to non-uniform thermal expansion. The thermal expansion of concrete with siliceous aggregates expressed as a function of temperature according to Eurocode 2, British Standards Institution (2004) is represented by Equations 3.8 and 3.9, and is shown in Figure 3.4

\[ \varepsilon_c(\theta) = -1.8 \times 10^{-4} + 9 \times 10^{-6} \theta + 2.3 \times 10^{-11} \theta^3 \quad \text{for} \quad 20^\circ\text{C} \leq \theta \leq 700^\circ\text{C} \]  
\[ (3.8) \]

\[ \varepsilon_c(\theta) = 14 \times 10^{-3} \quad \text{for} \quad 700^\circ\text{C} \leq \theta \leq 1200^\circ\text{C} \]  
\[ (3.9) \]

### 3.2.3.5 Stress-strain relationships of concrete at elevated temperatures

The most substantial consequence of fire on a concrete slab is the stiffness and strength degradation which may lead to eventual collapse. It is important to study the concrete property changes according to temperature. The stress-strain relationship of
concrete with siliceous aggregates expressed as a function of the temperature according to Eurocode 2 British Standards Institution (2004), follows the Equations 3.10 to 3.12, and the given distributions in Figures 3.5 and 3.6 represent the compressive and tensile stress-strain behaviour of the concrete, respectively.

**Compressive stress-strain relationship:**

\[
\sigma_c(\theta) = \frac{3f'_{c,\theta}}{\varepsilon_{c1,\theta} + \left(\frac{\varepsilon}{\varepsilon_{c1,\theta}}\right)^3} \quad \text{for} \; \varepsilon \leq \varepsilon_{c1,\theta}
\]  

(3.10)

\[
\sigma_c = 0, \; \varepsilon = \varepsilon_{cu1,\theta} \quad \text{(linear behaviour is adopted)} \quad \text{for} \; \varepsilon_{c1(\theta)} < \varepsilon \leq \varepsilon_{cu1,\theta}
\]

(3.11)

where,

\(f'_{c,\theta}\) = ultimate stress of concrete

\(\varepsilon_{c1,\theta}\) = strain at ultimate stress of the concrete

\(\varepsilon_{cu1,\theta}\) = ultimate strain of the concrete

**Tensile stress-strain relationship:**

\[
\sigma_t(\theta) = k_{c,t}(\theta)\sigma_t
\]

(3.12)

where

\(\sigma_t\) = tensile strength of concrete

\(k_{c,t}\) = reduction coefficient for tensile strength of concrete

\(k_{c,t}(\theta) = 1.0 \quad \text{for} \; 20^\circ C \leq \theta \leq 100^\circ C\)

\(k_{c,t}(\theta) = 1.0 - 1.0(\theta-100)/500 \quad \text{for} \; 100^\circ C \leq \theta \leq 600^\circ C\)
Figure 3.5 illustrates that the compressive strength of the concrete decreases when temperature increases, but the strain of the concrete increases with temperature. The tensile strength of the concrete also decreases with an increase in temperature, as depicted in Figure 3.6. A tensile stress also can be achieved for temperatures up to $500^\circ C$. The modulus of elasticity of the concrete in Figure 3.7 decreases with an increment in temperature. The reduction of the modulus of elasticity is due to the rupture of bonds in the microstructure of the cement paste when the temperature increases and due to the onset of rapid short-term creep.

### 3.2.4 Concrete Properties with Steel Fibres

According to Lok and Xiao (1999), even though concrete is weak in tension and lacks the necessary toughness and ductility, once it is reinforced, its mechanical properties will be altered. For concrete with steel fibres, Lok and Xiao (1999) described the stress-strain relationship as:

\[
\sigma = \begin{cases} 
2 \left( \frac{\varepsilon}{\varepsilon_{co}} \right) - \left( \frac{\varepsilon}{\varepsilon_{co}} \right)^2 & , \varepsilon \leq \varepsilon \\
\sigma = f_c & , \varepsilon \leq \varepsilon_{cu} \end{cases}
\]

(3.13)

(3.14)

where,

- $f_c$ is the compressive strength taken from the compressive cylinder strength test and usually is defined as $0.85f'_c$;
- $\varepsilon_{co}$ is defined as the strain value at the yield stress;
- $\varepsilon$ is the concrete strain value;
- $\varepsilon_{cu}$ is the ultimate compressive strain and a conservative value for fibre reinforced concrete of 0.003 was used. However, improved values suggested by different
researchers which include Swamy and Al-Ta'an (1981) recommended the value to be 0.0035 for a 1 % steel fibre concentration. Hassoun and Sahebjam (1985) recommended a value of 0.004 for 1 to 3 % of fibre concentration and Lok and Xiao (1999) suggested that the recommended value be 0.0038 for 0.5 to 2 % of steel fibres. For the purposes of the finite element analysis, the author have used the values suggested by Lok and Xiao (1999).

For concrete in tension, Lok and Xiao (1999) expressed the stress-strain relationship as:

\[
\sigma = f_t \left[ 2 \left( \frac{\varepsilon}{\varepsilon_{co}} \right) - \left( \frac{\varepsilon}{\varepsilon_{co}} \right)^3 \right] \Rightarrow 0 \leq \varepsilon \leq \varepsilon_{co} 
\]

(3.15)

\[
\sigma = f_t \left[ 1 - \left( 1 - \frac{f_{tu}}{f_t} \right) \left( \frac{\varepsilon - \varepsilon_{t0}}{\varepsilon_{t1} - \varepsilon_{t0}} \right) \right] \Rightarrow \varepsilon_{t0} \leq \varepsilon \leq \varepsilon_{t1} 
\]

(3.16)

\[
\sigma = f_{tu} \Rightarrow \varepsilon_{t1} \leq \varepsilon \leq \varepsilon_{tu} 
\]

(3.17)

where,

- \( f_t \) is defined as the ultimate tensile strength which can be determined through a direct tensile test;
- \( \varepsilon_{t0} \) is defined as the ultimate strain;
- \( f_{tu} \) is known as the residual strength and is defined as \( \eta V_f \tau_d \frac{L}{d} \) by Lok and Xiao (1998);
- \( \varepsilon_{t1} = \varepsilon_d \frac{L}{d} \), \( V_f \) = the fibre volume fraction;
- \( \tau_d \) = the bond stress;
- \( \frac{L}{d} \) = the fibre aspect ratio;
\[ E_s = \text{the elastic modulus of steel fibres;} \]
\[ \eta = \text{the fibre orientation in three dimensional random distribution. Hannant (1978) suggested that } \eta = 0.5, \text{ and Lok and Xiao (1999) suggested the value of } \eta = 0.405, \text{ which is used in this thesis herein.} \]

The stress-strain relationship of concrete with steel fibre reinforcement is shown in Figure 3.8. This model is adopted in the analysis due to details that are provided to distinguish the softening behaviour as a result of strain hardening of the steel fibre reinforced concrete. The predicted strengths using the derived expressions were compared with the experimental data, and good agreement was evident. In order to avoid significant error in the prediction, limitations in the material properties and the amount of steel fibres are provided.

### 3.2.5 Constitutive Modelling Approach

ABAQUS has several options for plasticity models. Karlsson and Sonrensen (2006b) stated these plasticity models are applicable to the inelastic behaviour of concrete. Most of these models are incremental, in that the total strain is separated into an elastic part, on which all changes in stress depend, and a plastic part. Incremental plasticity models generally are defined by the following:

(i) a yield function,

(ii) a flow rule,

(iii) a hardening law.
ABAQUS has two yield criteria suitable for predicting behaviour of concrete. They are:

(i) Von Mises criterion,

(ii) Hill criterion,

One of a range of nonlinear material behaviours that ABAQUS can simulate is plasticity. The common solution in ABAQUS to the nonlinear problems is to apply the loading in steps where the load in each step is being divided into increments. The Newton-Raphson method can be used to represent the response of the structure to a load increment which is solved by iteration, then the sensible monitoring of progress towards convergence to ensure a computationally efficient solution. After a material point has yielded, its stress components are updated for the increment using an implicit integration scheme.

This thesis is looking at the various effects on the behaviour of headed stud shear connectors. From the effects of steel fibres, strain regimes and combination of axial tension and shear loading, the author adopted the concrete smeared cracking model for the concrete element. Whilst for the effects of long-term analysis and elevated temperatures, the author used the concrete damaged plasticity.

3.2.5.1 Concrete smeared cracking

The concrete smeared cracking model does not track individual “macro” cracks. Constitutive calculations are performed independently at each integration point of the finite element analysis model to consider the presence of cracks in which the cracks

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affect the stress and material stiffness associated with the integration point. When isotropically hardened yield surface is active, the stress is dominated by compressive stresses that cause an independent “crack detection surface”. This is used to determine if a point fails by cracking. This failure surface is a linear relationship between the equivalent pressure stress and the Von Mises equivalent stress. Once a crack is formed, crack orientation is stored for subsequent calculations, and subsequent cracking at the same point is orthogonal to this direction. Bear in mind that there will be no more than three cracks at any point.

The failure ratios option in ABAQUS can be used to define the shape of the biaxial failure surface by specifying four ratios for ultimate stress and strain values of biaxial and uniaxial stress states. In this thesis, the ABAQUS default values have been used. The concrete behaviour is considered dependent on the reinforcement. Therefore, the effects associated with the reinforcing bars and concrete interface, such as bond slip and dowel action, are calculated by introducing tension stiffening to simulate the load transfer across cracks through the reinforcement. Tension stiffening models the post failure stress–strain behaviour for direct strain across cracks and allows the user to define the strain-softening behaviour for cracked concrete.

3.2.5.2 Concrete damaged plasticity

Concrete damaged plasticity model uses an isotropic damaged elasticity in combination with isotropic tensile and compressive plasticity, to better represent the inelastic behaviour of concrete. The concrete damaged plasticity in ABAQUS is used
to define yield function, flow potential, and viscosity parameters. Therefore, the model is suitable for the long-term analysis where viscosity parameters were used, whilst the flow potential was used to determine the heat transfer in the elevated temperature. Lubliner et al. (1989) proposed that the concrete model uses the yield function with the modifications suggested by Lee and Fenves (1998) to consider different progression of strength characteristics under tension and compression. The progression of the yield surface is defined by hardening variables, known as equivalent tensile and compressive plastic strains. The equivalent tensile and compressive plastic strains can be automatically calculated by ABAQUS after the definitions of elastic material behaviour. The tensile and compressive stress–strain behaviour outside the elastic range uses concrete tension stiffening and concrete compression hardening options. Lastly, the tensile and compressive damage uses the concrete tension damage and concrete compression damage options in ABAQUS, respectively.

A viscoplastic regularisation of the constitutive equations with a small value for the viscosity parameter, where the small value compared to the characteristic time increment can be defined to help improve the convergence rate in the concrete softening and stiffness degradation regimes. This causes the consistent tangent stiffness of the concrete, with softening and stiffness degradation, to become positive for sufficiently small time increments.
3.3  STRUCTURAL STEEL, REINFORCING STEEL, SHEAR CONNECTORS AND PROFILED STEEL SHEETING PROPERTIES

3.3.1  Structural Steel, Reinforcing Steel and Profiled Steel Sheeting at Ambient Temperatures

The stress-strain characteristics of reinforcing steel, shear connectors and profiled steel sheeting are essentially similar to structural steel. Their behaviour is initially elastic after which yielding and strain hardening develop. A piecewise linear approach was found to be sufficiently accurate to represent the stress-strain relationship. Moreover, these curves are utilised in the model when the stress-strain data is not available.

According to Loh et al. (2003), the stress-strain relationship for structural steel is represented as a simple elastic-plastic model with strain hardening. The mechanical behaviour for both compression and tension is assumed to be similar. Figure 3.9 represents the stress-strain relationship for steel and Table 3.1 indicates the different values of stress and strain for each material.

3.3.2  Shear Connectors at Ambient Temperatures

The most common type of shear connector used in composite construction is the 19 mm diameter headed shear stud. These connectors provide the composite action between the concrete slab and the steel beam, and are not only responsible for transferring shear forces at the slab-steel beam interface, but also prevent vertical separation at the interface. Many researchers have generated various non-linear curves using the approach of Aribert and Labib (1982).
\[ F_j = F_{\text{max}} (1 - e^{-\beta s})^\alpha \] \hspace{1cm} (3.18)

Aribert and Labib (1982) provided a combination of \( \alpha = 0.8, \beta = 0.7 \text{ mm}^{-1} \), while Johnson and Molenstra (1991) presented values of \( \alpha = 0.558, \beta = 1.0 \text{ mm}^{-1} \), and also \( \alpha = 0.989, \beta = 1.535 \text{ mm}^{-1} \). An improvement was given by Gattesco and Giuriani (1996) to simulate the actual behaviour of the connectors from Equation 3.19 in order to represent the connector load-slip characteristics, with \( \alpha = 0.97, \beta = 1.3 \text{ mm}^{-1} \) and \( \gamma = 0.0045 \text{ mm}^{-1} \). Moreover, Loh et al. (2003) proved that Equations 3.18 and 3.19 compared well with the experimental data obtained, as shown in Figure 3.10. From Figure 3.10, the mechanical properties of shear studs are modelled as a bilinear stress-strain model similar to Figure 3.9 excluding the strain hardening range.

\[ F_j = P_{\text{max}} \left[ \alpha (1 - e^{-\frac{-\beta s}{\alpha}})^{0.5} + \gamma s \right] \] \hspace{1cm} (3.19)

### 3.3.3 Thermal Properties of Structural Steel, Reinforcing Steel, Shear Connectors and Profiled Steel Sheeting

#### 3.3.3.1 General

For thin-walled steel sections, the thickness is such that their temperature across the section is considered uniform. Similarly for concrete, the effects of thermal conductivity, specific heat and high thermal expansion of the structural steel, steel reinforcing, profiled steel sheeting and shear connectors are considered when the temperature changes.
3.3.3.2 Thermal conductivity

The thermal conductivity of steel depends mainly on the amount of alloying elements and on the heat treatment. The thermal conductivity of steel, $\lambda$, according to Eurocode 3 British Standards Institution (2005) is determined from the following equations, and is presented in Figure 3.11.

$$\lambda = 54 - 3.33 \times 10^{-2} \theta_s \text{ W/mK} \quad \text{for } 20^\circ C \leq \theta_s \leq 800^\circ C$$  \hspace{1cm} (3.20)

$$\lambda = 27.3 \text{ W/mK} \quad \text{for } 800^\circ C \leq \theta_s \leq 1200^\circ C$$  \hspace{1cm} (3.21)

where,

$\theta_s$ is the steel temperature in [$^\circ C$]

3.3.3.3 Specific heat

The specific heat of the steel $C_a$ is expressed in Eurocode 3, British Standards Institution (2005) using the following equations, and is shown in Figure 3.12.

$$C_a = 425 + 7.73 \times 10^{-1} \theta_s - 1.69 \times 10^{-3} \theta_s^2 + 2.22 \times 10^{-6} \theta_s^3 \text{ (J/kgK)}$$  \hspace{1cm} (3.22)

For $20^\circ C \leq \theta_s \leq 600^\circ C$

$$C_a = 666 + 13002/738 - \theta_s \text{ (J/kgK)}$$  \hspace{1cm} (3.23)

For $600^\circ C \leq \theta_s \leq 735^\circ C$

$$C_a = 545 + 17820/\theta_s - 731 \text{(J/kgK)}$$  \hspace{1cm} (3.24)

for $735^\circ C \leq \theta_s \leq 900^\circ C$

$$C_a = 650 \text{ (J/kgK)}$$  \hspace{1cm} (3.25)

For $900^\circ C \leq \theta_s \leq 1200^\circ C$

where,
\( \theta_s \) is the steel temperature [°C]

### 3.3.3.4 Thermal expansion

Harmathy (1993) stated that the thermal expansion of steels depends mainly on the heat treatment. The coefficient of thermal expansion of steel at room temperatures is expected to be \( 11.4 \times 10^{-6} \text{ m} \cdot \text{C}^{-1} \). Furthermore, the thermal elongation of structural and reinforcing steel according to Eurocode 3 (British Standards Institution (2005) are evaluated using the Equations 3.26 to 3.28 and is illustrated in Figure 3.13.

\[
\Delta l/l = 1.5 \times 10^{-5} \theta_s + 0.4 \times 10^{-8} \theta_s^2 - 2.416 \times 10^{-4} \quad \text{for } 20^\circ C \leq \theta_s \leq 750^\circ C
\]
\[
\Delta l/l = 1.1 \times 10^{-2} \quad \text{for } 750^\circ C \leq \theta_s \leq 860^\circ C
\]
\[
\Delta l/l = 2 \times 10^{-5} \theta_s - 6.2 \times 10^{-3} \quad \text{for } 860^\circ C \leq \theta_s \leq 1200^\circ C
\]

### 3.3.3.5 Stress-strain relationships of structural steel, reinforcing steel, shear connectors and profiled steel sheeting at elevated temperatures

Most normal constructional steels have well-defined yield strengths at normal temperatures. Harmathy (1970) and Buchanan (2001) stated that the yield plateau becomes less noticeable with temperature rises. Upon further temperature increase, the ultimate strength of the steel declines steadily. The stress-strain relationships may be applied to steel in both tension and compression. The effects of high temperature on creep have also been taken into account.
The stress-strain relationships of structural steel as a function of temperature according to Eurocode 3, British Standards Institution (2005) follows the Equations 3.29 to 3.33, and they are illustrated in Figure 3.14

\[
\sigma(\theta) = \varepsilon E_{a,\theta} \quad \text{for } \varepsilon \leq \varepsilon_{p,\theta} \quad (3.29)
\]

\[
\sigma(\theta) = f_{p,\theta} - c + (b/a)\left[ a^2 - (\varepsilon_{y,\theta} - \varepsilon)^2 \right]^{0.5} \quad \text{for } \varepsilon_{p,\theta} \leq \varepsilon \leq \varepsilon_{y,\theta} \quad (3.30)
\]

\[
\sigma(\theta) = f_{y,\theta} \quad \text{for } \varepsilon_{y,\theta} \leq \varepsilon \leq \varepsilon_{t,\theta} \quad (3.31)
\]

\[
\sigma(\theta) = f_{y,\theta}[1 - (\varepsilon - \varepsilon_{t,\theta})/(\varepsilon_{u,\theta} - \varepsilon_{t,\theta})] \quad \text{for } \varepsilon_{t,\theta} < \varepsilon < \varepsilon_{u,\theta} \quad (3.32)
\]

\[
\sigma(\theta) = 0 \quad \text{for } \varepsilon = \varepsilon_{u,\theta} \quad (3.33)
\]

where,

\[
\varepsilon_{p,\theta} = \frac{f_{p,\theta}}{E_{a,\theta}}
\]

\[
\varepsilon_{y,\theta} = 0.02
\]

\[
\varepsilon_{t,\theta} = 0.15
\]

\[
\varepsilon_{u,\theta} = 0.20
\]

\[
a^2 = (\varepsilon_{y,\theta} - \varepsilon_{p,\theta})(\varepsilon_{y,\theta} - \varepsilon_{p,\theta} + c / E_{a,\theta})
\]

\[
b^2 = c(\varepsilon_{y,\theta} - \varepsilon_{p,\theta})E_{a,\theta} + c^2
\]

\[
c = \frac{(f_{y,\theta} - f_{p,\theta})^2}{(\varepsilon_{y,\theta} - \varepsilon_{p,\theta})E_{a,\theta} - 2(f_{y,\theta} - f_{p,\theta})}
\]

\[
f_{y,\theta} = \text{effective yield strength}
\]

\[
f_{p,\theta} = \text{proportional limit}
\]

\[
E_{a,\theta} = \text{slope of the linear elastic range}
\]

\[
\varepsilon_{p,\theta} = \text{strain at the proportional limit}
\]

\[
\varepsilon_{y,\theta} = \text{yield strain}
\]
\[ \varepsilon_{t,\theta} = \text{limiting strain for yield strength} \]

\[ \varepsilon_{u,\theta} = \text{ultimate strain} \]

The ultimate strength of the structural steel decreases when the temperature increases, as illustrated in Figure 3.14. Furthermore, the modulus of elasticity decreases with an increase in temperature. The relationship of the modulus of elasticity of the structural steel according to temperature is illustrated in Figure 3.15.

3.4 SUMMARY OF CHAPTER

As in any finite element model, the issue of assigning reliable material properties to the model that adequately represent the real situation or prototype, pretences a major problem. In the finite element analysis of composite steel-concrete push testing, the ability to model the failure mode of the headed shear stud connectors would be of primary concern. In addition, the capability to incorporate the effects of tension stiffening and shear retention in the reinforced concrete material definition would also be viewed as desirable. The ABAQUS program has the ability to model for these properties. However, in order to be able to compel the program effectively using these features, a suitable choice needs to be made of parameters that govern the model behaviour. This has led to an investigation of the original sources of the material property values required for implementation of these models in the ABAQUS code.
### Table 3.1 Stress-Strain Value for Structural Steel Beam, Shear Connectors, Profiled Steel Sheeting and Steel Reinforcing

<table>
<thead>
<tr>
<th>Element</th>
<th>$\sigma_{us}$</th>
<th>$\varepsilon_{ps}$</th>
<th>$\varepsilon_{us}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel beam</td>
<td>$1.28\sigma_{ys}$</td>
<td>$10\varepsilon_{ys}$</td>
<td>$30\varepsilon_{ys}$</td>
</tr>
<tr>
<td>Steel Reinforcing</td>
<td>$1.28\sigma_{ys}$</td>
<td>$9\varepsilon_{ys}$</td>
<td>$40\varepsilon_{ys}$</td>
</tr>
<tr>
<td>Profiled Sheeting</td>
<td>-</td>
<td>$20\varepsilon_{ys}$</td>
<td>-</td>
</tr>
<tr>
<td>Shear Connectors</td>
<td>-</td>
<td>$25\varepsilon_{ys}$</td>
<td>-</td>
</tr>
</tbody>
</table>
Figure 3.1 Stress-Strain Relationship for Concrete at Ambient Temperature, Carreira and Chu (1985)
Figure 3.2 Thermal Conductivity of Concrete, EC2 British Standards Institution (2004)
Figure 3.3 Specific Heat for Concrete, EC2 British Standards Institution (2004)
Figure 3.4 Concrete Thermal Expansion, EC2 British Standards Institution (2004)
Figure 3.5 Compressive Stress-Strain Relationships at Elevated Temperature for Concrete, EC2 British Standards Institution (2004)
Figure 3.6 Tensile Stress-Strain Relationship at Elevated Temperature for Concrete, EC2
British Standards Institution (2004)
Figure 3.7 Elastic Modulus of Concrete with Temperature, EC2 British Standards Institution (2004)
Figure 3.8 Stress-Strain Relationship for Concrete with Steel Fibres, Lok and Xiao (1999)
Figure 3.9 Stress-Strain Relationship for Structural Steel in Ambient Temperature, Loh et al. (2003)
Figure 3.10 Load-Slip Relationship for Shear Studs Loh et al. (2003)
Figure 3.11 Thermal Conductivity for Structural Steel, EC3 British Standards Institution (2005)
Figure 3.12 Specific Heat of Structural Steel, EC3 British Standards Institution (2005)
Figure 3.13 Thermal Expansion of Structural Steel, EC3 British Standards Institution (2005)
Figure 3.14 Stress-Strain Relationship at Elevated Temperature for Structural Steel, EC3
British Standards Institution (2005)
Figure 3.15 Modulus of Elasticity of Structural Steel at Elevated Temperatures EC3

British Standards Institution (2005)
4.1 INTRODUCTION

The finite element analysis (FEA) also sometimes referred to as the finite element method (FEM) can be defined as a numerical technique for finding approximate solutions of partial differential equations as well as of integral equations. The solution approach is based either on eliminating the differential equation completely or rendering the partial differential equations into an approximate system of ordinary differential equations, which are then solved using standard techniques such as Euler's method, Runge-Kutta methods.

According to Clough (2001), the Finite Element Method (FEM) was first developed in 1943 by R. Courant, who utilised the Ritz method of numerical analysis and minimisation of variational calculus to obtain approximate solutions to vibration systems. Turner et al. (1956) established a broader definition of numerical analysis where the authors focused on the stiffness and deflection of a complex structure. Clough (2001) also stated that in the early 1970's, finite element analysis was limited to expensive mainframe computers generally owned by the aeronautics, automotive, defence, and nuclear industries. Since the rapid decline in the cost of computers, finite element analysis has been developed to an incredible precision. Finite element
analyses are now able to be used to solve previously intractable problems and produce accurate results for a wide range of parameters.

Significant effort in research has concentrated on the modelling and analysis of composite steel-concrete slabs. However, according to Veljkovic (1993) the use of the method of applying the non-variable shear interaction property obtained from push tests will not produce satisfactory results if they are not modified in accordance with the response and behaviour in the slabs. Daniels and Crisinel (1993a) and Daniels and Crisinel (1993b) developed a finite element procedure using plane beam elements for analysing single and continuous spans of composite slabs. The procedure incorporated the nonlinear behaviour of material properties, but the authors did not look at the behaviour of the load-slip capacity of the headed stud shear connectors. The thesis herein deals with modelling the headed stud shear connectors using the finite element method with a focus on the effects of steel fibres, long-term effects, elevated temperatures, strain regimes and a combination of axial tensile and shear loading.

The primary objective in this thesis is to develop a convenient and reliable analysis methodology for developing finite element push-out test models that can accurately predict the shear stud capacity. This thesis describes the development of a finite element analysis model, provides validation through comparison with existing experimental test data, and illustrates its application to typical composite steel-concrete beams.
In order to avoid the high cost and considerable time requirements when conducting full-scale experimental testing, there is a need to develop more refined analysis tools to investigate the ultimate shear capacity for push tests for composite steel-concrete structures. The availability of high-speed computers and commercial finite element packages facilitate the development of these tools through sophisticated three-dimensional finite element analyses. Currently, information is lacking regarding the use of commercial software to study the headed stud shear capacity for composite steel-concrete structures. The primary focus of this thesis has been to propose comprehensive finite element modelling procedures using commercially available software that is capable of capturing the complete load-slip response for steel-concrete composite structures. This work incorporates ABAQUS software and describes the modelling techniques used in conjunction with this programme. Figure 4.1 shows the outline of the process in ABAQUS, starting from the geometric modelling until the execution of the analysis.

4.2 GENERAL DESCRIPTION OF ABAQUS

ABAQUS was used to execute the numerical analysis for the design and behaviour of headed stud shear connectors. ABAQUS is a suite of general-purpose, advanced nonlinear finite element analysis (FEA) programs. ABAQUS is used throughout the world for stress, heat transfer, and other types of analysis in structural, civil and engineering-related applications. The ABAQUS suite consists of three core products: ABAQUS/Standard, ABAQUS/Explicit and ABAQUS/CAE.
Some of the more significant features in the ABAQUS code include a wide range of element types, such as continuum elements which consist of one-dimensional, two- dimensional and three-dimensional beams, membranes and shells. Element formulations in ABAQUS are suitable for large displacements, rotations and strains. The material models can be used for metals, sand, clay, concrete, jointed rock, plastics and rubber. The user-defined subroutines permit the inclusion of additional material models and element types. Finally, other ABAQUS features includes fracture mechanics capability.

In this thesis, the author used ABAQUS/Standard because it enables a wide range of linear and nonlinear engineering simulations to be carried out efficiently, accurately, and reliably. There is no limit on the size of the time increment that can be used for most analyses in ABAQUS/Standard, and moreover the accuracy governs the time increment in ABAQUS/Standard. The flexibility provided by this integration allows ABAQUS/Standard to be applied to those portions of the analysis that are well-suited to an implicit solution technique, such as static, low-speed dynamic, or steady-state transport analyses.

4.3 MODELLING PROCEDURE IN ABAQUS

Advances in computational features and software have brought the finite element method within reach of both academic research and engineers in practice by means of general-purpose nonlinear finite element analysis packages, with one of the most used nowadays being ABAQUS. The program offers a wide range of options regarding element types, material behaviour, numerical solution controls, graphic user interfaces, auto-mesh, sophisticated post-processors and graphics to speed up
the analyses. In this thesis, the structural system modelling is based on the use of this commercial software.

In ABAQUS, the modelling procedure can be divided into four major steps. They are:

I. Spatial modelling and execution
II. Analysis procedure and control
III. Output analysis
IV. Analysis techniques.

Part I normally consists of part, property, assembly and mesh. Its function is to create individual parts of the structures. For push tests as in Figure 4.2, the parts are concrete slab, headed shear stud connectors, structural steel beam, reinforcing steel and profiled steel sheeting. All the parts created herein are three-dimensional solid elements, except for reinforcing bars.

Part II explains the analysis types that are used herein. The analysis type that is used is static stress analysis. In this part, it also describes constraints, contacts and interaction model used. Moreover, it also discusses the loading and boundary conditions. Part III describes how to attain the output from ABAQUS. For the analyses, the stress distribution of the push test, the ultimate load capacity and slip capacity for the headed stud shear connectors are the outputs obtained. Finally, Part IV provides the results of the analysing process.
4.4 FINITE ELEMENT TYPE AND MESH

Karlsson and Sonrensen (2006c), discussed the basic modelling concepts such as defining nodes and surfaces, the conventions and input formats that should be followed when using ABAQUS. Material properties of each material will be determined and the detailed explanation is in Chapter 3. After determining the appropriate material properties, it is followed by the assembly of the parts where the process is created and placed under all parts instances. Finally, the selection of accurate mesh is carried out.

A good quality mesh is a major issue in finite element analysis. The mesh should be fine enough for good detail where information is needed, but not too fine, or the analysis will require considerable time. A mesh should have well-shaped elements with only mild distortion and moderate aspect ratios. The thesis herein put considerable effort into the generation of well-shaped meshes. This will include setting element densities, gradients in element size, concatenation of lines and re-meshing individual areas to get accurate results.

Three-dimensional solid elements are used to model the push off test specimens in order to achieve an accurate result from the finite element analysis provided they are not distorted, particular in quadrilaterals and hexahedrals. Karlsson and Sonrensen (2006d) stated that the solid elements can be used for both linear and complex nonlinear simulations involving contact, plasticity and large deformations. They are also available for stress, heat transfer which is applicable for analyses in this thesis.
CHAPTER 4 – Finite Element Analysis

For both the concrete slab and the structural steel beam, a three-dimensional eight-node element (C3D8R) as in Figure 4.3 with linear approximation of displacements, reduced integration with hourglass control, eight nodes and three translational degrees of freedom was used to improve the rate of convergence. ABAQUS can provide response information at the nodes and element stresses at designated integration points within the element. Stresses at various points throughout the thickness of the element can also be provided where this is important for the study. Furthermore, C3D8R provides a constant volumetric strain throughout the element which prevents mesh locking when the material response is incompressible.

A three-dimensional thirty-node quadratic brick element (C3D20R) as in Figure 4.4 with quadratic approximation of displacements, reduced integration of twenty nodes and three translational degrees of freedom for the shear connectors was chosen due to higher accuracy that can be achieved. Furthermore, their ability to capture stress concentrations more effectively thus, better for modelling geometric features. Another reason is that there is only one quadratic element C3D20R in the transverse direction of the shear stud connectors.

A four-node doubly curved thin shell element (S4R) shown in Figure 4.5 was used for the profiled steel sheeting because it is the most appropriate type of element to model thin walled steel structures. The S4R element has six degrees of freedom per node and proved to provide accurate solutions, and also permits quadratic deformation over four nodal coordinates, membrane action and plain strain behaviour. Moreover, the element’s membrane is not sensitive to element distortion.
The S4R elements are more accurate in contact modelling than conventional shells, since they employ two-sided contact taking into account changes in the thickness. Moreover, S4R is also very useful to accurately compute the thermal stresses where an average temperature is used at the integration location in linear elements so that the thermal stress is constant throughout the shell surface.

Reduced integration is used for C3D8R, C3D20R and S4R because it uses a lower-order of integration to form the element stiffness. According to Karlsson and Sonrensen (2006d), the reduced integration reduces running time especially in three dimensions. Bear in mind that the second order reduced integration elements generally yield more accurate results than the corresponding fully integrated elements.

A two-node linear three-dimensional truss element (T3D2) with linear approximation of displacement, two nodes and three translational degrees of freedom for the steel reinforcing was used. The main purpose was that the axial direction can be released using the two-node linear displacement equation provided in ABAQUS. It is typically defined as embedded in the orientated surface of the concrete. The main reason was that the author were not overly concerned with the slip between the reinforcing bar and concrete.

The generated mesh was designed to give an optimal accuracy where the fine mesh surrounds the shear connectors and a coarse mesh was used elsewhere. A sensitivity analysis was conducted by the author to ensure the accuracy of the developed model.


4.5 *BOUNDARY CONDITIONS*

Numerical simulations have to consider the physical processes in the boundary region. In most cases, the boundary conditions are very important for the simulation of the region’s physical processes. Different boundary conditions may cause quite different simulation results. Improper sets of boundary conditions may introduce non-physical influences on the simulation system, while a proper set of boundary conditions can avoid that. In this thesis, the determination of the boundary conditions for a quarter push test model becomes very important in order to get accurate results.

The boundary conditions in a finite element model must limit translation or rotation in a manner appropriate to the case at hand. Boundary conditions can be used to imply symmetric behaviour in a structure that has symmetry, so that the model size can be halved, quartered, or similarly reduced, if the loading of the structure is also symmetrical. The goal here is to approximate reality in an acceptable way by avoiding the time-consuming while applying the use of contact and non-linear elements.

Boundary conditions that represent structural supports specify values of displacement and rotation variable at appropriate nodes. To facilitate a more economical solution, finite element meshes may also use symmetry where this can be implemented with symmetrical boundary conditions. Figures 4.6 and 4.7 represent the quarter push test model for Lam and El-Lobody (2005), El-Lobody and Young (2006), Zhao and Aribert (1992), Zhao and Kruppa (1996) and Lam’s modified models for both solid and profiled slabs. The nodes that lie on the symmetrical surface known as Surface 1 is for concrete, shear connectors, structural steel beams, steel reinforcing and profiled steel sheeting are restricted from moving in the x-
direction. All the nodes in the middle of the structural steel beam web, which are designated as Surface 2, are restricted to move in the $y$-direction. All the nodes of the concrete and the profiled steel sheeting, which are designated Surface 3, are restricted to move in the $z$-direction.

Figures 4.8 and 4.9 represent the half push test model for Becher (2005) and Wu (2006) and Hicks (2007) for both solid and profiled slabs. Because they represent a half model, the nodes that lie on the other symmetrical surface are known as Surface 1. Surface 1 consists of concrete, shear connectors, structural steel beam, steel reinforcing, and profiled steel sheeting, and they are restricted from moving in the $z$-direction. The model of Saari et al. (2004) shown in Figure 4.10 was modelled as a whole model due to the addition of tensile axial load and thus symmetry could not be apply.

4.6 LOAD APPLICATIONS

In this thesis, there are three different types of load applications for different finite element models. When modelled Lam and El-Lobody (2005) and El-Lobody and Young (2006) experiments, the application of a static concentrated load was applied to the centre of the web. For Zhao and Aribert (1992) and Zhao and Kruppa (1996), the author applied a uniformly distributed load throughout the centre of the web according to the experimental studies. When modelling the effect of steel fibres, a uniformly distributed load was applied to the centre of the web for Becher (2005) and Wu (2006) models. When the combination of axial tension and shear loading was taken into consideration, an additional uniformly distributed shear load was
applied to the upper surface of the structural steel illustrated in Figure 4.11 and 4.12. Finally, a uniformly distributed tensile axial load was applied at the middle of the concrete slab for Saari et al. (2004) model as illustrated in Figure 4.10.

The modified RIKS method was employed to the load in order for the load to be obtained through a series of iterations for each increment for a non-linear structure. The RIKS method was used for the nonlinear analysis to ensure that any unloading was captured. Furthermore, this method was used to predict unstable and nonlinear collapse conditions of a structure. The load magnitude was used as an additional unknown and solved simultaneously for the loads and displacements. To achieve accurate results in ABAQUS, the RIKS method has the ability to use the arc length along the static equilibrium in load-displacement space. The initial increments will be adjusted if the finite element model fails to converge. Finally, the value of load after each increment is computed automatically. The final result will be either the maximum value of the load or the maximum displacement value.

4.7 CONTACT AND INTERFACE ANALYSIS

Composite beams are usually made of a structural steel beam linked to a concrete slab by some sort of shear connection, which may allow for relative tangential displacements between the elements. This effect, also called partial interaction, plays an important role in the analysis and design of such structures. A very similar behaviour may show up in several other structural systems, such as precast concrete slabs, wood–concrete floor systems, sandwich panels and multi layered wood beams, either glued to each other or connected by some mechanical
device. In the research herein, there are four contact elements considered. The contacts included headed stud shear connector and concrete element, concrete and structural steel beam, profiled steel sheeting and concrete and profiled steel sheeting and structural steel beam. Surface to surface contact, with a small sliding option, was used for all the contacting surfaces to fully transfer the load from the structural steel beam web to the headed stud shear connectors and, eventually, to the concrete slab member. The contact areas in the headed stud shear connector and concrete, concrete structural steel beam, profiled steel sheeting and concrete and profiled steel sheeting and structural steel beam are illustrated in Figure 4.13 as Contact A, B, D and E respectively.

The contact surfaces of the headed stud shear connector shank and head were always chosen to be master surfaces (as the headed stud shear connector is of stiffer material) with all the other contact surfaces considered as slaves. During the analytical study and a friction coefficient of 0.25 was adopted for all the contact surfaces. Simulating the contact interaction between the parts of a shear connection using ABAQUS/Standard is a very sensitive and difficult issue to achieve, but it is of satisfactory accuracy when established.

The difficulties arise because of special arrangements needed to bring the connection parts into initial contact. Firstly, the mesh should be fine enough for each element’s node of the master surface to face a corresponding node of the slave surface elements. Secondly, the load should be applied extremely slowly until contact is established. Lastly, the boundary conditions need to be assigned in a proper way, to achieve sensible behaviour at the connection and move away from
any singularity problems that may arise. Therefore, each headed stud shear connectors was restrained, as described above, for the first analysis step, and then freed of any restraint for the later steps.

The headed stud shear connector was fixed at the bottom shank because the headed stud shear connectors were welded to the flange of the structural steel beam illustrated in Figure 4.13 as Contact C. The actual characteristics of the studs were modeled using the tie constraint option of the ABAQUS package. Tie constraint is a surface interaction model for defining load–slip and breakable bonds between the contact boundaries. It constrains the slave node, on the concrete surface, to the corresponding master node, on the steel beam, according to the prescribed load–slip relationship of the connectors up to failure of the interaction element.

4.8 SENSITIVITY ANALYSIS

The behaviour of steel-concrete composite beams is strongly influenced by the type of headed stud shear connection between the steel beam and the concrete slab. For accurate analytical predictions, the structural model must account for the interlayer slip between these two components. In numerous engineering applications especially in the fields of structural optimisation and structural reliability analysis, an accurate response sensitivity analyses are needed as much as the corresponding response simulation results.

A sensitivity analysis of steel-concrete composite structures was studied for the finite element modelling in this thesis. In particular, the effect of the concrete constitutive behaviour and different modelling considerations were evaluated. The
sensitivity of the numerical solution was investigated with respect to concrete properties, such as concrete tensile strength and concrete compression model. The material properties available in the literature were used as realistic testbeds for the response sensitivity analysis.

Another sensitivity analysis is with respect to modelling consideration. Several simplifications and assumptions were used in the models of the composite steel-concrete beams simulated in this study. In this section, the numerical solution is tested in order to check the validity of the main assumptions used during the simulation. In particular, the sensitivity to mesh size and structural steel beam and headed stud shear connectors interface modelling is tested.

4.8.1 Sensitivity to Concrete Tensile Strength

For negative moment regions, in reality concrete has significant strength before cracking, and after cracking, it has a stiffening effect on the reinforcing steel. After cracking, concrete cannot carry any force across the cracks which have to be carried by reinforcing steel. However, in between cracks, concrete can carry the load which will minimise the force transferred to the reinforcing steel. This effect is known as tension stiffening. According to Collins and Mitchell (1991), the relationship of concrete tensile strength is shown in Equations 4.1 and 4.2 and illustrated in Figure 4.14. Figure 4.14 shows that, when compared with experimental data in small strain situations, the model seems to be conservative. Moreover, at a point when it is 10% of the strain value, the graph did not give zero value. For these reason a multilinear representation was needed.
\[ f_c = E_c \varepsilon_{cf} \quad \text{for } \varepsilon_{cf} \leq \varepsilon_{cr} \]  
\[ f_c = \frac{\alpha_1 \alpha_2 f_{cr}}{1 + \sqrt{500 \varepsilon_{cf}}} \quad \text{for } \varepsilon_{cf} > \varepsilon_{cr} \] 

where,

- \( \varepsilon_{cf} \) = strain in concrete caused by stress
- \( f_{cr} \) = strain at which concrete cracks
- \( \alpha_1 \) = bond characteristic of reinforcement
- \( \alpha_2 \) = loading time period
  - = 1.0 short term monotonic
  - = 0.7 sustained or repeated load

In the sensitivity analysis, the author used the model suggested by Carreira and Chu (1985). The detail explanation is presented in Chapter 3 Section 3.2.2. Kaklauskas and Ghaboussi (2001) proposed the tensile stress-strain relationship for concrete based on flexural beam test. The changing parameters are depth of beams and diameter of bar reinforcement. Besides considering the influence due to cracking, bond and tension stiffening, the author also considered shrinkage. The stress-strain model adopted is shown in Figure 4.15. Figure 4.15 illustrates the parameters used to represent tension stiffening effect which are \( \alpha_1 \) and \( \alpha_2 \). They are related to cracking stress and cracking strain. The authors performed curve fitting by using MATLAB 5 and resulted the following relationship shown in Equation 4.3.

\[ \alpha_1 = 7.12p^2 - 27.6p + 32.8 \]  
\[ \alpha_2 = 6 \text{ if } p > 2 \% \]
It is interesting to note that several authors have proposed to use the different parameters for the concrete tensile strength. The failure mode remained the same during the simulations, but the peak load was different for different model. Figure 4.16 calculated the error percentage of 2.7%, 4.7% and 9% for Carreira and Chu (1985), Kaklauskas and Ghaboussi (2001) and Collins and Mitchell (1991), respectively. The numerical results also indicate a slight increase in stiffness with the concrete tensile strength in the Kaklauskas and Ghaboussi (2001) model. However, from a practical point of view, such an increase is negligible. It is important to point out the significance of these results. Therefore, the model suggested by Carreira and Chu (1985) was used for finite element modelling herein. The concrete tensile strength cannot be used as the unique failure criterion for predicting the force-slip relationship for headed stud shear connectors. As will be shown later, concrete compressive strength plays a more important role in predicting this type of failure.

4.8.2 Sensitivity to Concrete Compressive Strength

Three different models were used in this study to predict the behaviour of concrete in compression. The stress-strain relationship for concrete in compression is broadly available in most papers. Carreira and Chu (1985) suggested that the stress and strain behaviour for concrete in compression with consideration of strain softening. The detailed explanation for the model is in Chapter 3, Section 3.2.2. The model fitted significantly well when compared with the experimental data. Moreover, the curve also shows the ascending and descending branches of the stress-strain relationship.
In the CEB-FIP Model Code, CEB-FIP (1990), the stress-strain model analysed is for short-term loading under uniaxial compression. In CEB-FIP (1990), two equations are used to model the stress-strain relationships, as shown in Equations 4.4 and 4.5, and the stress-strain curves were illustrated in Figure 4.17. On the other hand it is also stated that the descending branch of the curve in Figure 4.17 is influenced by the length of the member subjected to compression. CEB-FIP recommends that the accurate length of the member subjected to compression be approximately 200mm.

\[
\sigma_c = \frac{E_{c1}}{E_{c1}} \left( \frac{\varepsilon_c}{\varepsilon_{c,1}} - \left( \frac{\varepsilon_c}{\varepsilon_{c,1}} \right)^2 \right) f_{cm} \quad \text{for} \quad 0 \leq \varepsilon_c \leq \varepsilon_{c,\text{lim}} \tag{4.4}
\]

\[
\sigma_c = \frac{f_{cm}}{E_{c1}} \left( \frac{\xi}{\varepsilon_{c,\text{lim}} / \varepsilon_{c,1}} \right) - \frac{2}{\left( \varepsilon_{c,\text{lim}} / \varepsilon_{c,1} \right)^2} \left( \frac{\varepsilon_c}{\varepsilon_{c,1}} \right)^2 + \frac{4}{\left( \varepsilon_{c,\text{lim}} / \varepsilon_{c,1} \right)^2} - \varepsilon \left( \frac{\varepsilon_c}{\varepsilon_{c,1}} \right) \quad \text{for} \quad \varepsilon_c > \varepsilon_{c,\text{lim}} \tag{4.5}
\]

where,

\[E_{c1}\] = the tangent modulus of concrete

\[\sigma_c\] = the compression stress of concrete in N/mm²

\[\varepsilon_c\] = the compression strain of concrete

\[\varepsilon_{c,1}\] = -0.0022

\[E_{c1}\] = \(f_{cm} / 0.0022\)

\[f_{cm}\] = the secant modulus from the origin to the peak compressive stress \(f_{cm}\)

Lam and El-Lobody (2005) treated the compression in concrete as an elastic plastic material. The authors suggested this model because they comprehended that no finite element model could handle unloading cycles with concrete due to cracking.
problem. Furthermore, the authors also assured that the concentration was on the failure mode of headed stud shear connectors, therefore, it is acceptable to use the bilinear stress and strain curve for concrete compressive strength, shown in Figure 4.18.

Different concrete strength is showed in Figure 4.19. The numerical results are shown in Figure 4.19 are in good agreement between experimental and numerical results is obtained for any of the three models. The error percentage calculated were 2.5 %, 3.6 % and 10.2 % for Carreira and Chu (1985), CEB-FIP (1990) and Lam and El-Lobedy (2005) respectively. The author found that from a practical point of view, the use of the Carreira and Chu (1985) model provided adequate accuracy.

4.8.3 Sensitivity to Mesh Size

Numerical simulations must be objective. The results of the calculations made with them should not depend on subjective aspects such as the choice of mesh or element size. When looking at the behaviour of the headed stud shear connectors for composite steel and concrete beams, two elements play the major part in the finite element analysis. They include concrete slab element and headed stud shear connectors element. Therefore, in this thesis herein, the author looked at the mesh configuration for concrete and headed stud shear connectors element. Three mesh configurations were used, as shown in Figures 4.20 and 4.21.

The obtained force-slip curves are shown in Figures 4.22 and 4.23 for three types of mesh sizes on concrete element for solid and profiled slabs, respectively. The error percentage calculated were 3.5 %, 0.8 % and 4.2 % for Mesh A, Mesh B and Mesh
C, respectively, for solid slab. Whilst, for profiled slab, the error percentage calculated were 4 %, 1.6 % and 9.6 % for Mesh A, Mesh B and mesh C, respectively. Figures 4.24 and 4.25 illustrate that the three different types of mesh sizes for headed stud shear connectors. For solid slab, the error percentage calculated were 6.6 %, 1.2 % and 3.6 % for Mesh D, Mesh E and Mesh F, respectively. For profiled slab, the error percentages shown were 8.3 %, 1.6 % and 2.5 % for Mesh D Mesh E and Mesh F, respectively.

All three simulations predict almost identical results. Slight differences are observed in the ultimate loads and slips. However, the differences are within the margin of error expected for a numerical simulation. The results presented in this section confirm the objectivity of the numerical solution. Additionally, it is interesting to note that excellent results are obtained when the medium coarse Mesh B and Mesh E is used. This can be explained by the fact that the damage processes, resulting in concrete cracking and headed stud shear connectors deformation. Therefore, adequate results can be obtained from simulations using element sizes Mesh B and Mesh E for concrete and headed stud shear connectors element, respectively.
4.8.4 Sensitivity to Structural Steel Beam and Headed Stud Shear Connectors Interface Modelling

The effect of modelling the structural steel beam and headed stud shear connectors interface is investigated in this section. The headed stud shear connector elements were bonded to the structural steel beam using tied contact interfaces. In this approach, each of the nodes on the headed stud shear connectors has the same displacement as the point on the surface of the structural steel beam to which it is the closest. This allows for the modelling of normal and shear stresses along the entire structural steel beam and headed stud shear connectors interface.

Figure 4.26 illustrates the two types of interface between structural steel beam and headed stud shear connectors. For comparison purposes, a model with welding fillet is included. Interface A is the simple interface between the two surfaces, whilst Interface B illustrates that the welding fillet is taken into account. The numerical results are presented in Figure 4.27. The error percentages shown were 1 % and 2.5 % for Interface A and Interface B, respectively. The margin of error is small, as expected for a numerical simulation.

The models analysed show only a slight difference in ultimate shear capacity. This general behaviour was observed for both interfaces. The similarity in the numerical results implies that even for shear stud fracture, the modelling of the welding fillet has a minor effect in the overall load-slip response. It is the author’s opinion that the fracture energy of the structural steel beam and headed stud shear connectors interface are the key parameters for modelling the behaviour of composite steel-concrete beams failing by studs fracture. This energy controls the damage
processes taking place along the interface. Experimental procedures are needed to
determine this energy. For simplification of the modelling, the author decided that
the application of Interface A is appropriate to determine accurate and reliable
results.

4.9 SUMMARY OF THE CHAPTER

Finite element analysis is a computer-based numerical technique for representing
the behaviour of engineering structures. In this thesis, it is used to determine the
stress distributions of the whole push test specimen, the ultimate load capacity and
slip behaviour in the headed stud shear connectors. It can be used to analyse either
small or large-scale slip under loading or applied displacement. It can also analyse
elastic deformation, or "permanently bent out of shape" plastic deformation. Several
alternative finite element models having different modelling assumptions, boundary
conditions, and different loading conditions were developed to represent the push
tests of the composite steel-concrete beams. This chapter emphasises the importance
of choosing a suitable finite element type and an adequate mesh size. The main
reason is to obtain an accurate result when compared with the experimental studies.
At the same time, the boundary condition and load application are also considered to
be important factors to obtain similar experimental environments. The numerical
results were sensitive to the constitutive model and mesh geometry, which
demonstrate the objectivity of the modelling approach used during this thesis. The
differences are within the margin of error expected for a numerical simulation.
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Figure 4.2 Push Test Specimens
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Mesh A: 5 x 21 Mesh

Mesh B: 6 x 26 Mesh

Mesh C: 8 x 35 Mesh

Figure 4.20 Mesh Configuration for Concrete Element
Mesh D: 4 x 8 Mesh

Mesh E: 5 x 16 Mesh

Mesh F: 12 x 31 Mesh

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CHAPTER 5

EFFECTS OF STEEL FIBRES ON HEADED STUD SHEAR CONNECTORS FOR COMPOSITE STEEL-CONCRETE BEAMS

5.1 INTRODUCTION

Steel fibres are known to improve the crack resistance and energy absorption, due to the presence of steel fibres throughout the concrete. Strength is also enhanced, as the steel fibres bridge any cracks, enabling the concrete to support load after failure. This chapter focuses on improving the shear capacity of headed stud shear connectors in composite steel-concrete slabs with the addition of steel-fibre reinforcement. The objective of this research work is to consider the effect of steel fibres on composite steel-concrete beams for both solid and profiled slabs.

As stated recently, trapezoidal profiled slabs are becoming increasingly more popular for high-rise buildings when compared with solid slabs because they can achieve large spans with little or no propping, and they require less concrete and plywood formwork. However, the profiles used to achieve these savings can have a detrimental effect on the shear connector behaviour. Therefore, steel fibres are introduced to further augment the ductility and strength of the shear connection region in the slab, especially profiled slabs. The results obtained from the finite element analyses were verified against existing experimental results. Moreover, the
finite element results will be compared against existing international standards, which include AS 2327.1 (2003), British Standards Institution (2004) and AISC (1999). The optimum value of steel fibres required to determine the ultimate shear capacity and slip capacity for headed stud shear connectors is also being determined herein.

5.2 EXPERIMENTAL SPECIMEN DETAILS

Two types of experimental investigations were considered for comparison with the finite element models developed in this research. They include Becher (2005) for solid slabs and Wu (2006) for profiled slabs. All experimental tests were based on Eurocode 4 British Standards Institution (2004), where a push test specimen consists of a steel beam connected to two concrete slabs, either a solid slab or profiled slab through shear connectors. The concrete slabs were embedded in mortar or gypsum onto the test bed and a uniformly distributed load was applied onto the upper end of the web of the steel beam. However, some compromise has been made due to time limits, equipment and budget restrictions.

The modified push test, as schematically shown in Figures 5.1 and 5.2 involves a standard push test as described with a roller support at one end to eliminate any horizontal resistance being imposed on the slab. The slab dimensions were 600 x 600 x 120 mm; 50 mm shorter and 30 mm thinner than that suggested by Eurocode 4 British Standards Institution (2004). In order to provide identical lateral and longitudinal reinforcement strength, 4 x 4 reinforcing mesh bar was used as a substitute for 5 x 4 reinforcing mesh which was outlined in Eurocode 4 British
Standards Institution (2004). The steel beam used was a 200UB25. Each specimen was initially loaded to 40% of the expected failure load, and then cycled 25 times between 5% and 40% of the expected failure load. Finally, each specimen was loaded until failure occurred.

Twelve push tests were carried out to determine the load-slip behaviour of the shear connectors for both the solid slab and profiled slab configurations. Three specimens of different steel fibre reinforcement concentrations with two push tests each were tested, as illustrated in Table 5.1.

5.3 NUMERICAL ANALYSIS: COMPARISON WITH EXPERIMENTAL PUSH TESTS

5.3.1 General

In order to determine the effect of steel fibres on the composite steel-concrete beams, push tests were carried out to verify the load-slip and shear capacity behaviour. Six specimens of different steel fibre quantities and slab profiles were tested to validate the present finite element model shown in Table 5.1. Table 5.2 compares the ultimate load of the six tests with the finite element analysis. Figure 5.3 verifies that the optimum dosage of steel fibres is $F=0.3$ because an increment of $F=0.6$ shows that no visible advantage is gained in concrete strength or ductility for this range of concrete strength. The ultimate load discrepancy is in good agreement with the experimental results. Generally, it can be observed that the ultimate loads for solid slabs are higher than profiled slabs.
5.3.2 Solid Slabs

The solid push tests showed that no cracks were found on the concrete slab until the ultimate load was reached. Cracks were only observed surrounding the stud after the solid slab failed. All the three tests for the solid slab illustrated that the dominant failure mode was stud fracture where the shear connectors sheared off near the weld collar. The failure mode of the shear connectors is similar to that mentioned by Yam (1981) and Lam and El-Lobody (2005). Figures 5.4, 5.5 and 5.6 illustrate that the shear connectors experienced significant deformation around their base. In the experiments, yielding of the stud elements was discerned near the shear connector collar followed by the maximum compressive stress being reached by the concrete elements around the shear connector. It was observed that cracking is lower when the fibre content of \( F = 0.3 \) was added to the concrete. In the finite element model, the failure mode was determined where the shear connector reached the maximum stress value before the other element reached their maximum stress. The maximum stress value was reached when the shear connectors reached the ultimate load and sheared off the concrete element.

For solid slabs, the failure modes were governed by fracture of the shear connectors. The stress contours showed a higher stress value before failure occurred with the increased fibre content which is shown in Figure 5.7. In Figure 5.8, the additions of steel fibres provide higher stiffness and ultimate shear capacity. One important observation was that the deformation decreased with the increase in fibre content. There is an optimum value for steel fibre content in concrete. From Figure 5.3, it is illustrated that when the value \( F = 0.6 \), the concrete strength is lower than \( F = 0.3 \). The reason for that is due to the fact that when the fibre content is too high in
the concrete, the fibres themselves become too congested in the concrete mix. Therefore, in this study, the optimum value for fibre content is assumed to be $F=0.3$. Furthermore, in experimental studies, for fibre content of $F=0.6$, not only was stud fracture observed, but concrete crushing failure also occurred. One more important observation was that by adding steel fibres into the concrete, even though the ultimate shear capacity increased but the ductility decreased. Therefore, it is important to add the optimum value into the concrete to avoid brittle failure of the concrete.

5.3.3 Profiled Slabs

The profiled slab exhibited initial cracking in the middle of the slab along the trough of the profiled slab which was initiated by concrete failure shown in Figure 5.9. When steel fibres were added to the concrete, instead of concrete failure, stud failure was introduced. Therefore, it is shown that steel fibres improve the concrete strength and cracking in the surrounding concrete shown in Figure 5.10. According to Hannant (1978), when the concrete started to crack, the steel fibres will align across the crack to prevent cracking. Whilst Figures 5.9 and 5.10 show that a back breaking type failure has occurred, this is not the primary cause and it has been exacerbated by concrete tensile and compression failure in the specimen which has then given rise to large deformations and a bending failure of the slab.

Figure 5.11 illustrates that two types of failures occurred, these being stud and concrete failure due to clumping of the steel fibres. In the finite element model, when there is no steel fibre in the concrete, the concrete reached its ultimate stress before
the shear connectors reached their maximum stress. Therefore the failure mode was concrete failure as observed in the experimental study. When $F=0.3$ steel fibres were added into the concrete, the shear connector reached the maximum stress value before the concrete reached its cracking stress. For $F=0.6$, the failure modes were governed by both fracture of shear connectors and concrete failure. In the finite element model, it was observed that the concrete reached the maximum stress before the shear connectors.

Figure 5.12 shows the concrete element in the trough of the profiled slab reaches a maximum stress before the shear connector element attains its maximum value. Profiled slabs indicated that the failure mode was dominated by concrete failure where the concrete cracked before the shear connectors sheared off near the weld collar with the exception of when $F=0.3$ steel fibres was added into the concrete. The failure mode observed is similar to that mentioned by El-Lobody and Young (2006). It was observed that the profiled slab experienced lower deformation and lower stress in the concrete and shear connector because of the contribution of the profiled steel Lysaght W-Dek sheeting. It was observed that cracking through the trough was lower when the fibre content was $F=0.3$, which proved that the steel fibres increased the stiffness and ductility of the concrete. Figure 5.13 shows that higher fibre content produced a higher stiffness and ductility. The optimal amount of fibres was found to be $F=0.3$. A similar observation was found when steel fibres were added into the concrete where the ductility decreased. Therefore, it is important to add the optimum value into the concrete to avoid brittle failure.
5.3.4  Positioning of Shear Connectors in the Profiled Slabs

Finite element studies have been undertaken in order to look at the effect of shear connectors placed in different positions in the profiled slab. The different positions of the shear connectors are illustrated in Figure 5.14.

In the initial finite element analysis, the shear connectors were placed in the perceived strong position. They are similar to the experimental tests undertaken by Wu (2006). When there were no steel fibres added to the concrete, the load-slip behaviour of the push test shows that the failure load was 52 kN. In the weak position of the finite element models, the failure load decreased to 41 kN, which is equivalent to a reduction of about 22 % in the strength of the shear connectors. The finite element model demonstrates that the concrete element surrounding the shear connectors starts to fail. When load is applied to the middle position of the profiled slab, the specimen also did not reach the strength attained by the shear connectors that were placed in the strong region. The strength of shear connectors reduced to a value of 46 kN, which is a 11 % reduction. The results are shown in Figure 5.15 and it can also be observed that the shear connectors in the strong position are more ductile than the other two positions. For the strong and middle positions of the shear connectors, their stiffness is similar. Conversely, the stiffness in the weak position is lower than the other two positions.

When steel fibres with a quantity $F=0.3$ were added to the concrete, the shear connectors in the strong position attained an ultimate load of 55 kN. In the weak position of the finite element models, the load applied decreases to 46 kN, which is a reduction of 15 % in strength. The middle position of the shear connectors shows a
maximum load of 50 kN, which is a 8% reduction when compared with the strong side. This is illustrated in Figure 5.16. When compared with concrete without steel fibres, the reduction in strength is lower. This observation suggests that the steel fibres assist to prevent concrete from cracking.

When steel fibres $F=0.6$ were added into the concrete, the strong position had an ultimate load of 50 kN. Both weak and middle positions of the finite element models were analysed, and it was shown in Figure 5.17 that the load was reduced to 41 kN and 46 kN, respectively. The reductions of 18% and 8% in the strengths when compared with the addition of steel fibres $F=0.3$ show that there is an optimum value at which the increase in strength in relation to the amount of steel fibres in concrete becomes minimum.

5.4 COMPARISON OF EXPERIMENTAL STUDIES AND FINITE ELEMENT SOLUTIONS WITH EXISTING INTERNATIONAL STANDARDS

A comparison was made to evaluate the experimental studies and finite element solutions with three existing international standards. They included the Australian Standard AS 2327.1 (2003), British Standards Institution (2004) and American Standard AISC (1999).

The strength of the shear connector is dependent on four principal factors; $E_c$ is the mean modulus of elasticity of the concrete, $E_{sc}$ is the modulus of elasticity of the shear connector, $f_c$ is the compressive strength of the concrete and $f_{us}$ is the ultimate
strength of the shear connector. The failure of a stud when steel fracture dominates is
given by:

\[ P_{AS} = 0.63d^2 f_{us} \]  \hspace{1cm} (5.1a)

\[ P_{EC} = 0.628d^2 f_{us} \]  \hspace{1cm} (5.1b)

\[ P_{AISC} = 0.785d^2 f_{us} \]  \hspace{1cm} (5.1c)

In each equation above, the stud failure will only take place when the concrete
strength is relatively high. When the concrete is weak, Equations 5.2(a, b and c) will
tend to govern. The equations below depend greatly on \( f_{ck} \) which is the characteristic
compressive cylinder strength of the concrete, and \( E_c \), the mean modulus of elasticity
of the concrete.

\[ P_{AS} = 0.31d^2 \sqrt{f_{ck}E_c} \]  \hspace{1cm} (5.2a)

\[ P_{EC} = 0.29d^2 \sqrt{f_{ck}E_c} \]  \hspace{1cm} (5.2b)

\[ P_{AISC} = 0.39d^2 \sqrt{f_{ck}E_c} \]  \hspace{1cm} (5.2c)

When the steel profiled slab is taken into consideration, Equations 5.1 and 5.2
above are replaced by a reduction factor \( k \). The value of \( k \) is defined in the equations
below:

\[ k_{AS} = 1.148 - (0.18 / \sqrt{n}) \]  \hspace{1cm} (5.3a)
\[ k_{EC} = \left( \frac{0.7}{\sqrt{n}} \right) \left( \frac{b_o}{h_p} \right) \left( \frac{h}{h_p} \right)^{-1} \leq k_{i,\text{lim}} \]  

(5.3b)

\[ k_{AISC} = \left( \frac{0.85}{\sqrt{n}} \right) \left( \frac{b_o}{h_p} \right) \left( \frac{h}{h_p} \right)^{-1} \leq 1 \]  

(5.3c)

where,

\[ n \] is the number of studs,

\[ b_o \] is the width of the steel profiled ribs,

\[ h_p \] is the height of the decking profile,

\[ h \] is the height of the stud.

A summary of the ultimate loads for the shear connector, \( P \) which are calculated using the Australian, British, and American Standards, is given in Table 5.3. Table 5.3 shows that both the experimental results and finite element solution fall between the theoretical values calculated from the three standards.

For the solid slab, the calculated theoretical values for both the Australian and British Standards corresponded with the experimental observations. On the other hand, the American Standard seems to be less conservative because the theoretical calculations prove that the shear connectors have higher capacities when compared with the experiments. For the profiled slab, the current design rules are adjusted purely based on a reduction factor \( k \) associated with the empirical strengths of the solid slab beams. Shear connection behaviour in reality is a complex mechanism and the logic of this approach is questionable, as the theoretical calculations contradict the experimental observations shown in Table 5.3.
5.5 PARAMETRIC STUDIES

The finite element models are developed to predict the capacity of shear connectors. Several finite element models are tested with different levels of modelling and different mesh size. The final model is verified through comparison with push-out test results. The verified finite element model was then used to conduct parametric studies aimed at investigating the effect of several parameters, such as concrete compressive strength and concrete slab thicknesses on resistance capacity. The results of the parametric study are treated statistically to produce a mathematical model to estimate the resistance capacity of the headed stud shear connectors.

Various analyses are conducted, classified as four main groups G1, G2, G3 and G4 using variables of compressive strengths and thicknesses of concrete. The details of parametric study are shown in Table 5.4. In group G1 and G2, the thickness is 120 mm for both solid and profiled slabs, respectively, but the concrete compressive strengths vary. Group G3 and G4 have a thickness of 150 mm with varying concrete compressive strengths. A total of 24 models are considered in analysis to record the ultimate load for each model. The ultimate shear capacity variations are plotted in Figure 5.18 for groups G1, G2, G3 and G4. The relationship of ultimate load capacity, $Q$ as a function of concrete strength property, $f_c'$ can be determined from Figure 5.18 and shown in the equations below.

$$Q = f_c'^4 \times 10^{-4} - 1.32 f_c'^3 \times 10^{-2} + 0.247 f_c'^2 + 8 f_c' - 86.42 \text{ for G1} \tag{5.4}$$

$$Q = 2 f_c'^4 \times 10^{-4} - 2.29 f_c'^3 \times 10^{-2} + 0.87 f_c'^2 - 9.52 f_c' + 106.61 \text{ for G2} \tag{5.5}$$
\[ Q = 2f_c^4 \times 10^{-4} - 1.97f_c^3 \times 10^{-2} + 0.80f_c^2 - 10.87f_c' + 67.84 \text{ for G3} \] (5.6)

\[ Q = 4f_c^4 \times 10^{-4} - 5.92f_c^3 \times 10^{-2} + 3.02f_c^2 - 63.94f_c' + 551.4 \text{ for G4} \] (5.7)

Figure 5.18 illustrates that thicker the slab has higher ultimate load capacity. When the fibre volume of \( F = 0.3 \) is added into the concrete slab, the ultimate shear capacity initially increases according to the concrete strength properties. The ultimate load increases until it reaches the concrete strength of 35 N/mm\(^2\). When the concrete strength is higher than 35 N/mm\(^2\), the ultimate load is decreased. The finite element model shows that the decrease in the ultimate load is due to the brittleness of the concrete. When the concrete strength is high, the concrete itself already has a very high performance, but when steel fibres are added into the high concrete strength, instead of having an increase in the strength, it makes the high strength concrete more brittle. When the push test is loaded, the deformation of the headed shear studs will cause the brittle concrete to fail instead of the stud. It was noticed that 150 mm thick slab increased the headed stud shear connectors performance when compared to 120 mm slab because thicker slab increase the cracking capacity of concrete. When steel fibres were added into the slab, the performance of headed stud shear connectors increased.

In order to prove that the finite element analyses were modelled accurately, the initial finite element model for Becher (2005) and Wu (2006) were measured from Figure 5.18. The finite element analysis result for Becher (2005) was 125.06 kN with 29.46 N/mm\(^2\) concrete strength and 124 kN was measured from Figure 5.18. The finite element analysis result for Wu (2006) was 54.54 kN with 37 N/mm\(^2\) concrete
strength and 55.50 kN was measured from Figure 5.18. The discrepancy is less than 2%. Therefore, it is proven that Figure 5.18 is reliable and accurate.

5.6 DESIGN MODELS AND RECOMMENDATIONS

The parametric studies shown in Section 5.4 was able to simulate the overall behaviour of headed stud shear connectors in push testing when steel fibres are included in the concrete slab. Furthermore, simpler graphs based on the numerical analyses were produced and are suitable for practising engineers. Figures 5.19 and 5.20 illustrate the ultimate shear capacity versus concrete strength properties for solid and profiled slabs, respectively.

Figures 5.19 and 5.20 illustrate that when the concrete strength property is less than 35 N/mm$^2$, the addition of steel fibres into concrete slab increase the ultimate capacity of the headed stud shear connectors. However, when the concrete strength property is more than 35 N/mm$^2$, the addition of steel fibres will reduce the ultimate shear capacity. When the concrete strength is less than 35 N/mm$^2$ it is suitable to add steel fibres to enhance the strength of the concrete and increase the cracking capacity. As soon as the concrete property is higher than 35 N/mm$^2$, the addition of steel fibres will cause the concrete to become brittle instead of strengthening it. The parametric study proved that the steel fibres are only useful for concrete strength property of less than 30 N/mm$^2$. Figures 5.19 and 5.20 are useful for design engineers in practice because the ultimate load capacity for headed shear stud connectors can be determined with or without steel fibres.
5.7 SUMMARY OF THE CHAPTER

An accurate finite element model has been developed to investigate the behaviour of the shear connection in composite steel-concrete beams for both solid and profiled slabs. Based on the comparisons between the results obtained from finite element models and available experimental results, it is observed that they are in good agreement. All the failure modes have been accurately predicted by the finite element model.

One primary issue that has been solved when steel fibres were included in a composite steel-concrete beam was the improved stiffness and cracking capability for both the solid and profiled slabs. Even though steel fibres did not show any major gain in concrete strength, the effectiveness of steel fibres was only shown when the concrete began to crack, particularly in profiled slabs. From both the experimental tests and finite element analysis, it is illustrated that the inclusion of steel fibres in concrete improves the cracking load of the concrete. This is a major advantage especially for continuous beams in hogging moment regions. Moreover, the ultimate load of the push tests also increases with the inclusion of steel fibres for both the solid and profiled slabs.

From the parametric study, it can be concluded that using the finite element method to simulate push-out test is acceptable. The numerical model used to estimate the shear resistance of headed stud shear connectors is suggested to be used within the limit of the investigated parameters.
Table 5.1 Push Test Specimens Parameters

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Slab Profile</th>
<th>Percentage of steel fibres</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Solid slab</td>
<td>0.0</td>
</tr>
<tr>
<td>2</td>
<td>Solid slab</td>
<td>0.3</td>
</tr>
<tr>
<td>3</td>
<td>Solid slab</td>
<td>0.6</td>
</tr>
<tr>
<td>4</td>
<td>Lysaght W-Dek Profiled slab</td>
<td>0.0</td>
</tr>
<tr>
<td>5</td>
<td>Lysaght W-Dek Profiled slab</td>
<td>0.3</td>
</tr>
<tr>
<td>6</td>
<td>Lysaght W-Dek Profiled slab</td>
<td>0.6</td>
</tr>
</tbody>
</table>
Table 5.2 Comparison of Experimental Results and Finite Element Analysis

<table>
<thead>
<tr>
<th>Test No</th>
<th>Concrete strength (N/mm²)</th>
<th>Exp. result (kN)</th>
<th>FEM result (kN)</th>
<th>Result Discrepancy (Exp./FEM)</th>
<th>Failure Mode (*)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>31.7</td>
<td>104.8</td>
<td>110.0</td>
<td>0.95</td>
<td>S.F</td>
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<tr>
<td>2</td>
<td>34.9</td>
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<td>125.1</td>
<td>0.95</td>
<td>S.F</td>
</tr>
<tr>
<td>3</td>
<td>30.6</td>
<td>106.4</td>
<td>112.8</td>
<td>0.94</td>
<td>S.F and C.F</td>
</tr>
<tr>
<td>4</td>
<td>33.9</td>
<td>49.9</td>
<td>52.4</td>
<td>0.95</td>
<td>C.F</td>
</tr>
<tr>
<td>5</td>
<td>36.8</td>
<td>51.2</td>
<td>54.6</td>
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<td>S.F</td>
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<tr>
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<td>47.5</td>
<td>53.7</td>
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<td>S.F and C.F</td>
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<tr>
<td>Mean value</td>
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<td>0.94</td>
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</table>

*S.F denotes stud failure and C.F concrete failure
### Table 5.3 Comparison of Ultimate Load of Shear Connector for Different Design Standards

<table>
<thead>
<tr>
<th>Slab Factor (F)</th>
<th>Fibre Factor (F)</th>
<th>Experimental Results (kN)</th>
<th>FEM Results (kN)</th>
<th>Australian Standard</th>
<th>British Standard</th>
<th>American Standard</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Stud</td>
<td>Con.</td>
<td>Stud</td>
</tr>
<tr>
<td>S.S</td>
<td>0</td>
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<td>110.0</td>
<td>113.7</td>
<td>75.3</td>
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<td>125.1</td>
<td>130.8</td>
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<td>130.4</td>
</tr>
<tr>
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<td>94.1</td>
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<tr>
<td>P.S</td>
<td>0</td>
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<td>52.4</td>
<td>56.9</td>
<td>37.6</td>
<td>35.1</td>
</tr>
<tr>
<td></td>
<td>0.3</td>
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<td>54.6</td>
<td>65.4</td>
<td>43.3</td>
<td>40.4</td>
</tr>
<tr>
<td></td>
<td>0.6</td>
<td>47.5</td>
<td>53.7</td>
<td>71.1</td>
<td>47.1</td>
<td>43.9</td>
</tr>
</tbody>
</table>

S.S denotes solid slab, P.S denotes profiled slab, Stud denotes stud failure, Con. denotes concrete failure.
Table 5.4 Dimensions and Details of Parametric Study

<table>
<thead>
<tr>
<th>Group</th>
<th>Specimen</th>
<th>Slab Dimension (mm)</th>
<th>Concrete Strength, $f_c$ (N/mm²)</th>
</tr>
</thead>
<tbody>
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4 x 4 reinforcing mesh are provided in both ways with 10 mm diameter reinforcing bars with 180 mm spacing.

Figure 5.1 Details of Solid Slab Push Test Specimen for Becher (2005)
4 x 4 reinforcing mesh are provided in both ways with 10 mm diameter reinforcing bars with 180 mm spacing.

Figure 5.2 Details of Profiled Slab Push Test Specimen of Wu (2006)
Figure 5.3 Ultimate Load versus Fibre Volume
Figure 5.4 Concrete Damage at the Base of the Shear Connectors for Concrete with $F = 0$ Fibres
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Figure 5.17 Load-Slip Relationship for Different Shear Connectors Positions with Steel Fibre Content $F = 0.6$
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Figure 5.18 Ultimate Load versus Concrete Strength
Figure 5.19 Ultimate Load versus Concrete Strength for Solid Slab
Figure 5.20 Ultimate Load versus Concrete Strength for Profiled Slab
CHAPTER 6
EFFECTS OF LONG TERM BEHAVIOUR ON HEADED STUD SHEAR CONNECTORS FOR COMPOSITE STEEL-CONCRETE BEAMS

6.1 INTRODUCTION

For composite steel-concrete structures, concrete not only provides the compressive strength, but also the fire resistance to the floor surface, whilst the steel predominantly provides the tensile strength. Johnson (1974) stated that when acting compositely through a shear stud, the composite steel-concrete member is stiffer and stronger. When concrete creep and shrinkage are considered, the deformation will increase with time. When the creep and shrinkage behaviour of the concrete is considered, the factors affecting the shear connection are the stiffness and strength of shear connectors and the stiffness and strength of the surrounding concrete. The strength of concrete is reduced according to time due to creep and shrinkage.

Although time-dependent deformations for concrete are not a major concern with respect to the collapse of structures, they are important issues that need consideration for serviceability and durability of structures. There are physically two different types of time-dependent deformations for concrete. One is purely dependent on the stress which is known as the creep strain, \( \varepsilon_{cr} \) and the other is independent of stress and is known as the shrinkage strain, \( \varepsilon_{sh} \).
When the composite steel-concrete structure is exposed to a dry atmosphere, the concrete element will gradually shrink. Shrinkage occurs when the concrete volume reduces, and this appears in time in spite of the stress state. The rate of shrinkage strain is gradually increased with time at a reduced rate shown in Figure 6.1.

According to CEB-FIP (1990), the development of shrinkage starts at time \( t_s \), when moist curing stops and strain starts to develop at a time \( t > t_s \) and is defined in Equation 6.1 below:

\[
\varepsilon_{\text{cs}}(t, t_s) = \varepsilon_{\text{cs0}} \beta_s (t - t_s)
\]  

(6.1)

where,

\( \varepsilon_{\text{cs0}} \) is the notation given to the shrinkage coefficient. The value depends upon the quality of concrete and the environment's humidity.

\( \beta_s \) is the coefficient to represent the development of shrinkage according to time

\( t \) is the age of concrete in days

\( t_s \) is the age of concrete in days when shrinkage starts

Tarantino and Dezi (1992), Dezi et al. (1995), and Dezi and Tarantino (1993) studied the creep analysis of composite beams where the flexibility of shear connection was taken into account. These studies were to investigate the effect of slip on the steel-concrete interface and determine the distribution of the shear force of the composite steel-concrete beams.

Dezi et al. (1998) simplified the complex numerical methods with a method known as the algebraic method. The algebraic method is a method used for non-
complex composite structure. However, for shear connectors, the author used the pseudo-elastic analysis because the headed stud shear connectors are known to behave in a complex manner. Furthermore, this study is also made possible by defining a modified Young’s modulus for concrete. These methods are the effective modulus method, EM and the mean stress method, MS. The creep effect is evaluated by means of a pseudo-elastic analysis using the EM method and is defined in Equation 6.2, whilst the shrinkage effect is modified by creep and is evaluated by means of a pseudo-elastic analysis using the MS method as defined in Equation 6.3.

\[
E_{c,EM}(t,t_0) = \frac{E_c}{1 + \phi(t,t_0)} \tag{6.2}
\]

\[
E_{c,MS}(t,t_s) = \frac{E_c}{1 + 0.5\phi(t,t_s)} \tag{6.3}
\]

where,

- \(E_c\) is the elastic modulus of the concrete
- \(\phi(t,t_0)\) is the creep coefficient
- \(t\) is the loading time
- \(t_0\) is the final time
- \(t_s\) is the age of concrete at the beginning of shrinkage

According to Gilbert (1988), the creep coefficient changes with time and the values of the creep coefficient calculated are shown in Figure 6.2. By using Equation 6.4 below, the value of creep coefficient reduces the elastic modulus of concrete. The relationship of the elastic modulus of concrete with respect to time is illustrated in Figure 6.3, which highlights that the elastic modulus of concrete is
reduced with respect to time. It reduces significantly up to day 400, and after that the reduction value is less insignificant.

\[ E'_c = \frac{E_c}{1 + \psi \phi} \quad (6.4) \]

where,

\( \phi \) is the creep coefficient and

\( \psi \) is an aging coefficient where the most appropriate value of 0.5 is used in the German design code, DIN 1045-1 (2001) to allow for time-dependent induced stress.

Bearing in mind that both equations suggested by Dezi et al. (1998) and Gilbert (1988) are similar. Therefore, the author proposed to apply the simplified algebraic method for the ABAQUS analysis herein.

### 6.2 EXPERIMENTAL SPECIMEN DETAILS

Experimental investigations undertaken by Lam and El-Lobody (2005) for solid slabs were used and compared with the finite element analysis when creep is taken into consideration. In order to compare the results of solid slabs used for calibration with W-DEK profiled steel sheeting slabs, the dimensions of the concrete slab, structural steel beams, steel reinforcement and shear connectors were held constant.

Initially, experimental push tests undertook by Lam and El-Lobody (2005) were modelled without the time dependent behaviour. The main reason is to prove that the finite element model was accurately modelled. Then, the finite element models were incorporated with the time dependent behaviour.
In this chapter, the author only considered when shear connectors were positioned in the middle of the profiled sheeting. Mirza and Uy (2007) found that the strength of the shear connectors increased by 11% when they are placed on the strong side. Conversely, when the shear connectors were positioned on the weak side, the strength of the shear connectors showed a reduction of 13%.

All experiments were designed based on Eurocode 4, British Standards Institution (2004) which specifies a push test specimen that consists of a steel beam connected to two concrete slabs either solid slab or profiled slab through shear connectors. However, some compromise has been made due to time limits, equipment and budget restrictions. The modified push tests shown in Figures 6.4 and 6.5 involve slab dimensions of 619 x 469 x 150 mm. In order to provide identical lateral and longitudinal reinforcement strength, 4 bars of 10 mm diameter reinforcement in each direction were used. The steel beam used was a 254 x 254 UC 73. Both the concrete slab and structural steel beam were attached to each other by 19 mm diameter shear connectors of 100 mm in height.

6.3 NUMERICAL ANALYSIS: COMPARISON WITH EXPERIMENTAL PUSH TESTS

6.3.1 General

In order to determine the effects of creep and shrinkage on composite steel-concrete beams, the push tests were analysed using ABAQUS to verify the shear capacity of the headed stud shear connectors. Figure 6.6 showed that the finite element model at 28 days is similar to the experimental study carried out by Lam and
El-Lobody (2005). At 28 days, there is no consideration of time dependent behaviour to the concrete, the ultimate shear of the push test showed that the failure load was 105.4 kN. Whilst, the finite element model showed 102.6 kN. The discrepancy between experiment and finite element model is 2.6%. Therefore, the finite element model was accurately model.

Ten finite element models were analysed for each solid and profiled slab shown in Table 6.1. Based on the finite element analyses, the variable parameters are time where concrete behaviour changes when the creep and shrinkage is taking into consideration. It should be borne in mind that, for the purpose of comparison of the ultimate shear capacity for headed stud shear connectors in this study, the author utilise the slip control function in ABAQUS. The author impedes the analyses at 16mm for solid slabs in order to compare with the existing experimental study and 6mm for profiled slabs.

Figures 6.6 and 6.7 generally show that the shear capacity for solid slabs is higher than profiled slabs. It is observed that both slabs behaved in a similar manner. The strength of the composite structure was reduced according to time when creep and shrinkage are taken into account. At the same time, creep and shrinkage also reduced the stiffness of the structure. Even though creep and shrinkage reduced the capacity of both the slabs, the solid slab had a greater slip and strength capacity when compared with the profiled slab.
6.3.2 Solid Slab

The solid slab illustrated that the dominant failure mode was stud fracture where the shear connectors sheared off near the weld collar as shown in Figure 6.8. In the finite element models, it was observed that the headed stud shear connectors reached the maximum stress before concrete failure. After the headed studs reached the maximum stress and started to shear off, the stud connectors will crush the concrete on one side and cause the concrete to fail by crushing. The failure mode of the shear connector fracture is similar to that observed by Yam (1981) and Lam and El-Lobody (2005). The shear connectors experienced significant deformation around their base with time. Figure 6.8 also illustrates that at day 28 the headed stud shear connectors can withstand an ultimate load of 110 kN, but after day 2000 the shear connectors loses its ultimate capacity to 97 kN. The shear stud lost 12% of its capacity when creep and shrinkage are taken into account.

6.3.3 Profiled Slab

The profiled slab showed signs of first cracking occurring in the middle of the slab along the trough of the profiled sheeting which was caused by concrete failure, as shown in Figure 6.9. From the finite element model, the concrete element was observed to reach its cracking stress before the headed stud shear connector reached its maximum stress. Even though Kim et al. (1999) stated that the inclusion of profiled steel sheeting resulted in less concrete cracking due to its contribution to the tensile strength, but the author still noted that the failure mode was caused by concrete failure where the concrete cracked before the shear connectors fractured near the weld collar. The observed failure mode is similar to that mentioned by El-Lobody and Young (2006). Figure 6.9 also illustrates that at day 28 the headed stud
shear can withstand ultimate load of 58 kN, but after day 2000 the headed stud shear connectors loses its ultimate capacity to 49 kN. The headed stud lost 16 % of its capacity when creep and shrinkage are taken into account.

6.3.4 Shear Resistance of Headed Stud Shear Connectors with Respect to Time

Based on the results of push test finite element analyses, a series of numerical analyses has been carried out which established the progression of shear force resistance when creep and shrinkage are incorporated. The shear forces were measured according to an ultimate slip rate of 4 mm. The shear force ratios, $P(t)/P(28)$ related to different time periods of the push tests are plotted in Figure 6.10. The solid slab showed a reduction of 12 % in the shear force resistance, while the profiled slab showed a 16 % reduction. Both slabs illustrated that the shear force reduced significantly up to 400 days, after that the reduction value was not significant. The concrete creeps significantly in the first 400 days and the Young’s modulus of concrete is reduced accordingly, leading to significant reduction in shear resistance of the headed stud shear connection.

Creep is dependent on the stress and affected area. Figure 6.11 shows the stress distribution for both solid and profiled slabs. In Figure 6.11a, Region A when compared to Region B, the affected area is smaller. When the affected area is smaller, the stress will tend to accumulate in the Region A more than Region B. In time, Region A will have a higher stress than Region B, and with the combination of less concrete area, the profiled slab shows a higher reduction in shear force resistance than the solid slab.
Figure 6.12 shows the slip for both slabs behaved correspondingly and it is observed that the slip increases significantly in the first 400 days, and then the increase becomes insignificant. Therefore, it can be assumed that time-dependent slip can be ignored after 400 days. The slip capacity for solid slabs was higher than profiled slabs.

6.4 PARAMETRIC STUDIES

An extensive parametric study consisting twenty six series of tests was conducted using the finite element model to look at the effect of creep and shrinkage analysis on the behaviour of headed stud shear connectors for both solid and profiled slabs. The verified finite element model was used to conduct a parametric study aiming to investigate the effect of several parameters, such as height and thickness of connector and concrete compressive strength on the headed stud shear resistance capacity. Constants were the number of stud connectors per slab, steel reinforcement, concrete slab dimensions, structural steel beam and profiled steel sheeting. Each series consisted of ten tests where the push tests were subjected to variable time which ranged from 28 to 2000 days. Tables 6.2a-p show the headed stud shear connectors used were 13, 16, 19 and 22 mm with various concrete strengths of 25, 30, 35 and 40 N/mm².

Table 6.3 shows the obtained ultimate shear capacity of headed stud shear connectors with varying diameters and concrete strength from the finite element model. The results show that the ultimate shear capacity for all the headed shear stud
connectors increased with the increase in concrete strength. Table 6.3 also demonstrates that for all the headed stud shear connectors, the failure mode is governed by concrete failure with the lower concrete strength, whilst with the higher concrete strength, the failure mode is dominated by stud yielding failure. The purpose of these tests is described below.

### 6.4.1 Effects of Headed Stud Shear Connectors

The first parameter evaluated was the effect of height and thickness of the headed stud shear connectors on the composite steel-concrete beams. Conventionally, push-out tests have used 19 x 100 headed stud shear connectors. Thirty tests were performed to verify that the height and thickness of headed stud shear connectors would cause changes in the ultimate capacity of composite steel-concrete beams. The desired mode of failure for composite steel-concrete structures was stud shearing failure. From the parametric studies, the headed stud shear connectors with 13 x 65 mm and 16 x 75 mm show a lower stud capacity than 19 x 100 mm and 22 x 100 mm studs. When 19 and 22 mm diameter are used, the resistance against concrete splitting is provided by the capacity of the headed stud shear connectors. The push test specimens with smaller stud diameter reduced the shear capacity than those with bigger stud diameter. Push test specimens with 19 and 22 mm diameter studs only failed when the ultimate strength of the headed stud was reached. This effect is particularly important when a partial shear connection design is adopted when the ductility of the shear connectors becomes an important issue.
6.4.2 Effects of Concrete Compressive Strength

The second parameter investigated was the effect on stud strength of the concrete compressive strength. These tests were used to determine if the stud strength was reduced by changing the concrete compressive strength. The stud strength increased when higher concrete compressive strength was used. Thus one could conclude that using a higher concrete compressive strength in composite steel-concrete specimens automatically increases the stud strength. Higher concrete compressive strength will have lower concrete splitting. Therefore, for a higher concrete compressive strength, when the concrete splitting occurs, the tensile strength developed by the headed stud shear connectors is much higher, and the shear resistance of the stud will rise. From the parametric studies, the failure modes changes from concrete failure at the lower concrete compressive strength to stud shearing failure at the higher concrete compressive strength. Lower concrete compressive strength showed that the failure of push-out specimens tested occurred in concrete slab and initiated by longitudinal splitting of the slab which is similar as mentioned in Liu (2006). When higher concrete compressive strength was used, the stud will achieve the maximum stress before the concrete reaches its tensile stress. This is the reason why the headed stud shear connectors reached their maximum capacity before concrete failure.

6.5 DESIGN MODELS AND RECOMMENDATIONS

The parametric studies shown in Section 6.4 is able to simulate the overall behaviour of headed stud shear connectors in push testing when creep and shrinkage are included in the concrete slab. Most of the push tests analysed by the author contained either solid slab or profiled slab with WDEK profile of 10 mm thick.
Strictly speaking, the result revealed herein is only relevant for such cases mentioned above only. Graphs based on the numerical analyses were produced and are suitable for practising engineers to estimate the shear capacity of the headed stud shear connectors for design purposes. Figures 6.13 to 6.16 illustrate the ultimate shear capacity according to time for solid and profiled slabs with concrete strengths of 25, 30, 35, and 40 N/mm$^2$, respectively.

Figures 6.13 to 6.16 illustrate that when creep and shrinkage are accounted for in the analysis of the composite steel and concrete beams, the ultimate shear capacity loses its strength significantly for the first 400 days, and after 400 days the reduction in shear capacity can be ignored.

### 6.6 SUMMARY OF THE CHAPTER

In order to prove that an accurate finite element model has been developed to investigate the behaviour of the shear connection in composite steel-concrete beams for both solid and profiled slabs when creep is taken into account, the finite element models were initially compared with existing push test experimental studies. From the finite element analyses, when creep and shrinkage are considered, the reduction in stiffness, ultimate shear capacity and slip capacity for both the solid and profiled slabs was observed. From the finite element analyses, the solid slab demonstrated that the failure mode is dominated by shear yielding failure, whilst failure in the profiled slabs can be attributed to concrete failure.
Extensive parametric studies suggested that the lower concrete strength will cause concrete failure, whilst higher concrete strength will cause the headed stud shear connectors to reach its ultimate capacity. The graphs provided by these parametric studies are very useful for a design engineer to estimate the shear capacity when creep and shrinkage are taken in consideration. It was noticed that increasing the concrete strength for composite steel-concrete specimens influenced the lower and upper bounds increase in terms of both the ultimate load and the associated slip capacity. In some cases, the failure mode of the push tests can change from slab crushing to stud failure. Moreover, the corresponding load–slip curve becomes stiffer. In conclusion, it may be possible for the developed finite element model to replace the need for expensive experimental study in the future which could save time and money.
Table 6.1 Push Test Specimens Parameters

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Table 6.2a Parametric Study for Push Test Specimens

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Figure 6.1 Shrinkage Curve with Respect to Time, Bazant and Jirásek (2002)
Figure 6.2 Creep Coefficient of Concrete with Respect to Time, Gilbert (1988)
Figure 6.3 Elastic Modulus of Concrete with Respect to Time, Gilbert (1988)
Figure 6.4 Push Test Specimen Details for Solid Slab
Figure 6.5 Push Test Specimen Details for Profiled Slab
Figure 6.6 Load versus Slip Relationship with Respect to Time for Solid Slab
Figure 6.7 Load versus Slip Relationship with Respect to Time for Profiled Slab
Figure 6.8 Stress Contours for Push Tests for Solid Slab
Figure 6.9 Stress Contours for Push Tests for Profiled Slab
Figure 6.10 Shear Resistance of Shear Connectors with Respect to Time for Ultimate Slip Rate = 4 mm
CHAPTER 6 – Effects of Long Term Behaviour

Figure 6.11 Push Test Showing Stress Distribution

(a) Profiled Slab

(b) Solid Slab
CHAPTER 6 – Effects of Long Term Behaviour

Figure 6.12 Slip Behaviour of Shear Connectors with Respect to Time
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CHAPTER 7
EFFECTS OF ELEVATED TEMPERATURES ON HEADED STUD SHEAR CONNECTORS FOR COMPOSITE STEEL-CONCRETE BEAMS

7.1 INTRODUCTION

Design of multi-storey buildings for fire resistance has conventionally been undertaken on the assumption that the building will suffer partial or total collapse, unless all the structures are protected from temperature rise under fully developed fire conditions. Since 1990, a growing body of evidence from fires in buildings and from the results of advanced analyses has shown this not to be the case. This may be because the design structural fire severity which means the impact that the design fire would have on the structural system is low. This chapter presents an overview of headed stud shear connectors response to fire. It then outlines appropriate design procedures to be used in composite steel-concrete buildings under different levels of structural fire severity.

It is important to understand the reaction of a multi-storey composite steel-concrete building to fire in order to understand the effects of elevated temperatures on the building and hence to determine the appropriate design procedure to check for the adequacy of the building if subjected to a fully developed fire attack. When a composite steel-concrete beam reaches its limiting temperature, it still has sufficient
strength to support the long-term design loads indefinitely and without excessive deflection.

Fire engineering is a widely used term in modern building design, but it has many meanings. To the structural engineer, it means taking account of the load-carrying behaviour of structural components at the temperatures that would be experienced in a real fire. To the regulating authority or fire expert, it might mean studying the temperature rise in fires where the intensity of the fire is a function of the fire load, the compartment size and the ventilation conditions. To the architect, it means the incorporation of fire protection or detection devices to reduce the potential spread of a fire.

These are essentially all aspects of the same problem, that is, a better understanding of how buildings actually behave in fire. It could be claimed that most buildings today would behave better in fire than their predecessors because of general reductions in fire load, more robust construction, better fire-protection measures, and faster fire-brigade response times.

The aim of this chapter is to investigate the fire resistance of the headed stud shear connectors on composite steel-concrete beams for both solid and profiled slabs as a part of the frame using numerical analysis methods. In the finite element analysis, the chapter focuses on the realistic behaviour of the push test in fire. The effects of structural stability and its quantification, mechanical interaction between the structural members in fire have been investigated. Another aim of the study is to
CHAPTER 7 – Effects of Elevated Temperatures

develop a new ultimate load ratio-time curve considering numerous parameters for structural design

7.2 EXPERIMENTAL SPECIMEN DETAILS

7.2.1 General

According to Cooke et al. (1988), the fire resistance of composite steel-concrete beams is obtainable in terms of their load carrying capacity, reliability and thermal insulation. Therefore, the main components affecting these are the concrete slab, structural steel, steel reinforcing, profiled steel sheeting and shear connectors. In this chapter, the finite element model was used to investigate the behaviour of shear connectors in composite steel-concrete beams with both solid and profiled steel sheeting slabs under elevated temperatures.

7.2.2 Experimental Investigation for Push Tests

There are three different experimental series results which were compared with the finite element analysis herein. The push tests performed by Lam and El-Lobody (2005) and El-Lobody and Young (2006) considered shear connection of a solid and profiled slab, respectively, at ambient temperatures, and these were compared with the finite element analysis method results. In this chapter, the author only took into account the position of the shear connectors in the middle of the profiled steel sheeting. Shear connectors are usually laid either on the strong side or weak side of the profiled slab. In order to avoid the adverse effect of rib punch-through failure and other failure modes such as longitudinal splitting and concrete pull-out, shear connectors are preferred to be placed on the strong side. The strong side position for
shear connectors can lead to significant improvements in strength and ductility of the connectors. Mirza and Uy (2007) showed that the strength of the shear connectors increased by about 11% when they are placed on the strong side. Conversely, when the shear connectors were positioned on the weak side, the strength of the shear connectors showed a reduction of about 13%. Other authors used similar push tests but at elevated temperatures, and these were analysed against ambient temperatures. The experimental investigation undertaken by Zhao and Aribert (1992) at elevated temperatures with different loading conditions were also compared with the finite element analysis. These were based on solid slab behaviour.

7.3 ANALYTICAL PROCEDURE

7.3.1 General

The response of a composite steel-concrete structural member exposed to fire is governed by the rate of heating. This is because the mechanical properties of materials decrease as the temperature rises and, likewise, the structural resistance of a member reduces with a temperature rise.

In this study, two components were evaluated. They comprise structural and temperature analyses. The structural analysis is where the structure is simulated at ambient temperature, and the result was compared with experimental studies. The temperature analysis simulated the behaviour of the structure as a function of time using measured temperature distributions of the structural elements from independent tests.
7.3.2 Structural Analysis

The structural analysis is performed for structures in the ambient condition. This push test specimen is similar to the standard push test according to Eurocode 4, British Standards Institution (2004), but only one shear connector is connected to each flange. This is due to the assumption that the load from the structural steel beam is transferred equally to the shear connectors. In order to reduce the simulation time and utilise the structural symmetry, only a quarter of the push test specimen was modelled. The material properties are defined in Section 3.3.1 for concrete, Sections 3.3.1 and 3.3.2 for the structural steel, reinforcing steel, headed shear stud connectors and profiled steel sheeting.

7.3.3 Temperature Analysis

The temperature analysis is performed independently of the structural analysis. To perform the temperature analysis, the geometry of the cross-section is similar to the structural analysis specimen. Conversely, its material properties are defined as in Section 3.2.3, 3.3.3 and 3.3.4 for concrete, structural steel and shear connectors, respectively. The materials in the section can vary from element to element, and their properties are temperature dependent. Zhao and Kruppa (1996) stated that the mechanical behaviour is much more complicated when the temperature changes because there are two materials involved, which are mainly concrete and steel. At elevated temperatures, the different heating conditions directly influence their own mechanical behaviour which will consequently modify their initial interaction effect. The push test specimen model is similar to the structural analysis model in order to compare the results.
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Fire is usually represented by a temperature-time curve which gives the average temperature reached during a fire in a small sized compartment or in the furnaces used for fire resistance tests. International standards are based on the standard fire defined by the heat exposure given by ISO834 (1999), and is represented by the curve shown in Figure 7.1

\[ T = 345 \log_{10}(8t + 1) + 20 \]  
(7.1)

where,

- \( T \) is the average furnace temperature, in °C
- \( t \) is the time, in minutes

In more complex analyses, different heating models are considered to represent the temperature development in different zones of the fire compartment. This is the case, for instance, when the fire is transferred from the structural steel beam to the concrete slab, shear connector and profiled steel sheeting. Steel elements have an adverse behaviour in this respect due to the very high thermal conductivity of the steel. Rapid heating of the whole profile takes place as a result. In comparison, composite elements exhibit a favourable behaviour due to the great thermal inertia of the elements and the low thermal conductivity of the concrete.

An accurate computer simulation of heat flow through a composite deck slab has been completed by Cooke et al. (1988) using a program called FIRES-T to determine the distribution of temperatures throughout the slab according to time. Lamont et al. (2004) also studied composite steel-concrete structure temperature distributions using HADAPT, and compared these with the Cooke et al. (1988) approach. Their
results were in good agreement, therefore, the resulting temperature distributions have been adopted here by the author for the application of the finite element ABAQUS analysis. The idealised temperature distribution diagrams are presented in Figures 7.2 and 7.3 for solid and profiled slabs, respectively. Figures 7.2 and 7.3 show the concrete, structural steel beam, profiled steel sheeting, and shear connectors divided into layers which is necessary to differentiate the temperature distributions according to time. Table 7.1 provides the temperature distribution of the layers with respect to time. The layers refer to Figures 7.2 and 7.3, respectively.

7.4 NUMERICAL ANALYSIS COMPARISON WITH EXPERIMENTAL PUSH TESTS

7.4.1 General

For the purposes of this part of research, finite element push tests were modelled to determine the load-slip behaviour of the shear connectors. The first analyses considered were compared with the experimental investigations undertaken by Zhao and Aribert (1992) for solid slabs. The second series of finite element models considered ambient temperatures in order to compare the experimental tests, and then the finite element models were modified to study the temperature changes according to time. The temperature changes according to time are illustrated in Table 7.1. The third series of analyses involved finite element models comprising of temperatures at different load levels. The load levels considered were 20, 40 and 60 % of the ultimate load. Two different kinds of slabs were studied, namely, a push test for a solid slab and a push test for a trapezoidal profiled slab. The dimensions and concrete strength of the finite element models is shown in Table 7.2.
7.4.2 Solid Slab for Zhao and Aribert (1992)

In order to verify that the finite element analysis was accurately undertaken, the experimental investigation of Zhao and Aribert (1992) was used for calibration purposes. The author completed push test experiments at ambient and at elevated temperatures. The specimens had a mean concrete compressive strength of 34.5 N/mm$^2$. The push test results for ambient temperatures of the finite element analysis are provided in Figure 7.4. Experimental results showed that the shear connectors had an ultimate shear carrying capacity of 125 kN, and the finite element model produced an ultimate load of 131 kN which resulted in a 5% discrepancy.

The push tests at elevated temperatures with different loading conditions and comparison with the finite element models are shown in Figure 7.5. The results are in good agreement with the experimental investigation. The temperature and slip are measured at the bottom shear stud where it is welded to the steel beam. According to Figure 7.6, the stress distribution contours proves that the higher the load applied the lower the temperature the specimens could withstand.

7.4.3 Solid Slab

For solid slabs, the finite element model is compared with the experimental study undertaken by Lam and El-Lobody (2005). The specimens had a concrete compressive strength of 35 N/mm$^2$. The second series of finite element analyses are highlighted in Figure 7.7. It is demonstrated that the experimental result and the finite element model results are in good agreement. The maximum load is 102 kN
with 6 mm slip and 113 kN with 9 mm slip for the experimental result and finite element model results, respectively.

The failure mode of the shear connectors herein is similar to that described by Yam (1981) and Lam and El-Lobody (2005). Figure 7.8 illustrates that the shear connectors experienced significant deformation around their base. In the experiments, yielding of the stud element was discerned near the shear connector collar followed by maximum compressive stress being reached by the concrete elements around the shear connector.

### 7.4.4 Profiled Slab

For the profiled steel sheeting slab, the finite element model is compared with the experimental study undertaken by El-Lobody and Young (2006). The specimen had a concrete strength of 35 N/mm$^2$. The second series of finite element model analyses are shown in Figure 7.9. The results depicted in Figure 7.9 confirm that the experimental and finite element model results are in close agreement. The maximum load is 83 kN with 1.9 mm slip and 84 kN with 1.3 mm slip for the experimental results and finite element model result, respectively.

The profiled slab revealed initial cracking in the middle of the slab along the trough of the profiled slab, which is caused by concrete failure. The concrete element in the trough of the profiled slab reached a maximum stress before the shear connector element. Both the experimental tests and finite element model demonstrated that the failure mode was dominated by the concrete deforming where the concrete crushed and cracked before the shear connectors fractured near the weld.
collar. The failure mode observed is similar to that mentioned by El-Lobody and Young (2006). Figure 7.10 shows that the profiled slab experienced less deformation and lower stresses in the concrete and shear connector which is attributed to the strength contribution of the profiled steel sheeting.

### 7.5 PARAMETRIC STUDIES

#### 7.5.1 General

The verified finite element model was used to conduct a parametric study aiming to investigate the effect of elevated temperatures on the behaviour of headed stud shear connectors. Constants were the number of stud connectors per slab, steel reinforcement, concrete slab dimensions, structural steel beam, and profiled steel sheeting. For the parametric studies, both solid and profiled slabs were considered to study the effect of temperature on the behaviour of the shear connectors.

#### 7.5.2 Solid Slab

Another finite element model analysis carried out was to study the effect of temperature changes on the shear connector. This series of analyses were able to obtain the load versus slip curve, as shown in Figure 7.11, where the strength of the shear connector is decreased with the increment of temperature.

Figure 7.11 illustrates that the strength of the shear connector declines when the temperature increases. Moreover, the ultimate load is reduced when the temperature increases. Initially, for the first 10 minutes of the fire, the ultimate load reduces by 35
% compared with the ultimate load at ambient temperatures. After 180 minutes of fire, the ultimate load reduced to a maximum of 57%. The failure is governed by the ultimate stress of the concrete. The ultimate stresses of concrete reduced according with temperature. When the structures reach the ultimate stress, the concrete started to crack which allowed failure to occur. The results of the push test finite element analyses revealed a strength reduction with respect to time.

The ultimate load ratio \( \frac{P_u(\theta)}{P_u(20^\circ C)} \) which relates to different time periods of the push tests can be determined from Equations 7.2 to 7.5, and they are illustrated in Figure 7.12.

\[
\frac{P_u(\theta)}{P_u(20^\circ C)} = -3.45 \times 10^{-2} t + 1 \quad \text{for } 0 \text{ min} \leq t \leq 10 \text{ min} \tag{7.2}
\]

\[
\frac{P_u(\theta)}{P_u(20^\circ C)} = -9.4 \times 10^{-3} t + 0.7483 \quad \text{for } 10 \text{ min} \leq t \leq 20 \text{ min} \tag{7.3}
\]

\[
\frac{P_u(\theta)}{P_u(20^\circ C)} = -1.10 \times 10^{-3} t + 0.5837 \quad \text{for } 20 \text{ min} \leq t \leq 120 \text{ min} \tag{7.4}
\]

\[
\frac{P_u(\theta)}{P_u(20^\circ C)} = -6 \times 10^{-5} t + 0.4527 \quad \text{for } 120 \text{ min} \leq t \leq 180 \text{ min} \tag{7.5}
\]

where

\( P_u(\theta) \) is the ultimate load at a degree Celsius

\( P_u(20^\circ C) \) is the ultimate load at 20 °C

\( t \) is the time in minutes

Figure 7.12 illustrates that the ultimate load reduces significantly from 0 up to 20 minutes, and after that the reduction value is not as large. Therefore, in reality, the first 20 minutes of fire is very crucial because the structure loses about 35% of its
strength. Different stress contours and deformed shapes of the concrete slab, structural steel beam and shear connectors of different fire rating can be observed in Figure 7.13.

In order to study the behaviour and strength of the shear connectors when exposed to temperature changes, several load levels were applied to the push test until it failed. Figure 7.14 shows that during fire exposure, the maximum load that the push test can withstand is up to 40 % of the ultimate load for solid slabs up to 180 minutes. If the load is 60 % of the ultimate load, the structure can resist only 10 minutes of fire exposure before the structure fails.

7.5.3 Profiled Slab

Further analysis was undertaken to study the effect of temperature changes on the shear connector. These series of analyses are undertaken to attain the load versus slip curves as shown in Figure 7.15, and the result is similar to that mentioned in Section 7.5.2 in which the strength of the shear connector degrades with an increment in the temperature.

Even though the ultimate load is reduced when the temperature is increased, when compared with the solid slab, the reduction of ultimate load is much lower. Initially, for the first 10 minutes of fire, the ultimate load reduces by 25 % compared with the ultimate load at ambient temperature. After 180 minutes of fire, the ultimate load reduces by a maximum of 31 %.
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The ultimate load ratio \([P_u(\theta)/P_u(20^\circ C)]\) related to different time periods of the push tests are determined from Equations 7.6 to 7.8 and plotted in Figure 7.16.

\[
\frac{P_u(\theta)}{P_u(20^\circ C)} = -2.51 \times 10^{-2} t + 1 \quad \text{for } 0 \text{ min} \leq t \leq 10 \text{ min} \quad (7.6)
\]

\[
\frac{P_u(\theta)}{P_u(20^\circ C)} = -6 \times 10^{-4} t + 0.7549 \quad \text{for } 10 \text{ min} \leq t \leq 60 \text{ min} \quad (7.7)
\]

\[
\frac{P_u(\theta)}{P_u(20^\circ C)} = -2 \times 10^{-4} t + 0.7279 \quad \text{for } 60 \text{ min} \leq t \leq 180 \text{ min} \quad (7.8)
\]

where,

- \(P_u(\theta)\) is the ultimate load at temperature \(\theta\) degree Celsius
- \(P_u(20^\circ C)\) is the ultimate load at 20 \(^\circ\)C (ambient temperature)
- \(t\) is the time in minutes

The ultimate load reduces significantly up to 20 minutes, and after that, the incremental reduction value is not as significant as in the early stages. Different stress contours and deformed shapes of the concrete slab, structural steel beam, profiled steel sheathing, and shear connector of varying fire ratings can be observed in Figure 7.17.

The third series of finite element analyses is similar to the one described in Section 7.5.2. Figure 7.18 shows that during fire exposure, the maximum load that the push test can withstand is up to 60 % of the ultimate load for profiled slabs up to 180 minutes. This result illustrates the potential role of the steel sheathing as a protective layer to the concrete slab. If the load is 80 % of the ultimate load, the structure fails when exposed to fire.
Figures 7.12 and 7.16 reveal that the ratio of the shear resistance at 180 minutes of fire exposure compared with ambient temperature was 0.45 and 0.7 for the solid and profiled slabs, respectively. Figure 7.19 shows the cross-sectional areas for both the solid and profiled slabs. From Figure 7.19, the ratio of the proportional area in a high temperature region, $\kappa$, is determined using Equation 7.9:

$$\kappa_s = \frac{A_{1s}}{A_{1s} + A_{2s}} \quad \text{or} \quad \kappa_p = \frac{A_{1p}}{A_{1p} + A_{2p}}$$

(7.9)

where,

$\kappa_s$ and $\kappa_p$ = the ratio of proportional area of concrete at high temperature

$A_{1s}$ and $A_{1p}$ = the cross-sectional area at the bottom of concrete

$A_{2s}$ and $A_{2p}$ = the cross-sectional area at the top of concrete

The proportional areas for solid and profiled slabs in the high temperature region are 0.45 and 0.27, respectively Figure 7.19. It appears that the failure mode of shear connectors was greatly influenced by the proportional area of concrete in a high temperature region, which was higher for the solid slab. Subsequently on this basis, profiled slabs perform better at high temperatures in comparison to ambient temperature.

7.6 DESIGN MODELS AND RECOMMENDATIONS

7.6.1 Different Load Distribution

For design purposes, the Australian Standard AS/NZ 1170.0 (2002) is used in order to determine the basic combinations for the strength limit state of the structures. The loads when it is in the ambient condition and exposed to fire, are
illustrated in Equations 7.10 and 7.11, respectively. The load ratio factor $\xi$ is the ratio between distributed loads when exposed to fire over distributed load at normal condition, and it is given in Equation 7.12.

\begin{align*}
E_n &= 1.2G + 1.5Q \\
E_f &= 1.1G + 0.4Q \\
\xi &= E_f / E_n
\end{align*} 

(7.10)  
(7.11)  
(7.12)

where $G$ is the dead load of the structures, $Q$ is the live load of the structures.

When $G = Q$, the distributed load for the ambient condition is $E_n = 2.7G$, the distributed load when exposed to fire is $E_f = 1.5G$ and the load ratio factor is $\xi = 0.55$. From Figure 7.12, the load ratio of 0.55 shows that the solid slab can resist the fire for 30 minutes before failure occurs. For the profiled slab, the structures can withstand the fire for more than 180 minutes as shown in Figure 7.16. This is because the profiled steel sheeting acted as a protective layer for composite steel-concrete structures during a fire. Moreover, according to Yu et al. (2006), due to the inherent efficiency of the profiled slab, the displacement of the concrete is lower when compared with a conventional solid slab. Therefore, Figures 7.12 and 7.16 are useful for designers to estimate the appropriate fire exposure for both solid and profiled slabs with different combination factors.
7.7 SUMMARY OF CHAPTER

Based on the comparisons between the results obtained from the finite element models and available experimental results, it was observed that they are in good agreement. All the failure modes were accurately predicted by the finite element model, and a maximum discrepancy of 10% was observed when comparing the finite element model with the experimental studies.

Composite steel-concrete beams subjected to fire are also discussed in this chapter. Numerical analyses of fire resistance of shear connectors were also investigated using the finite element analysis package, ABAQUS. These studies have allowed the investigation of the shear resistance of the connectors as a function of time at elevated temperatures. The finite element analysis allowed the elucidation of the failure of shear connectors at elevated temperatures.

Furthermore, this study was based on results where the ISO 834 fire was used as a basis for temperature analysis. This may not be the case for real buildings. However, the trends in this chapter may be of use for further studies. Further experimental research is considered necessary in order to validate the results above to enhance an understanding of fire resistance of composite steel-concrete structures.
Table 7.1 Temperature Changes According to Time

<table>
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<th>Time (t) min</th>
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<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
<th>F</th>
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Table 7.2 Dimensions and Concrete Strength for Finite Element Models

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<th>Series</th>
<th>Specimen</th>
<th>Sheeting Type</th>
<th>B (mm)</th>
<th>h (mm)</th>
<th>D (mm)</th>
<th>Exposure to Fire (min)</th>
<th>Concrete Strength (N/mm$^2$)</th>
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<td>600</td>
<td>10-180</td>
<td>34.5</td>
<td>Load at 17.5 kN per stud</td>
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<td>10-180</td>
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<td>10-180</td>
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<td>W-Dek</td>
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<td>W-Dek</td>
<td>900</td>
<td>115</td>
<td>600</td>
<td>10-180</td>
<td>35.5</td>
<td>Load until structural failure</td>
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<td>10-180</td>
<td>35.0</td>
<td>20 % of Ultimate Load</td>
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<td>W-Dek</td>
<td>900</td>
<td>115</td>
<td>600</td>
<td>10-180</td>
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<td>20 % of Ultimate Load</td>
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<td>150</td>
<td>619</td>
<td>10-180</td>
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<td>40 % of Ultimate Load</td>
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<td>619</td>
<td>150</td>
<td>619</td>
<td>10-180</td>
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<td>60 % of Ultimate Load</td>
</tr>
<tr>
<td></td>
<td>LPTA5a-LPTA5g</td>
<td>W-Dek</td>
<td>900</td>
<td>115</td>
<td>600</td>
<td>10-180</td>
<td>35.5</td>
<td>60 % of Ultimate Load</td>
</tr>
</tbody>
</table>

*a denotes 10 mins fire exposure, b denotes 20 mins fire exposure, c denotes 30 mins fire exposure, d denotes 60 mins fire exposure, e denotes 90 mins fire exposure, f denotes 120 mins fire exposure and g denotes 180 min fire exposure
Figure 7.1 ISO834 Fire Standard Curve
CHAPTER 7 – Effects of Elevated Temperatures

Figure 7.2 Temperature Distribution Diagram for Solid Slab, After Cooke et al. (1988)
Profiled Steel Sheeting
Structural Steel Beam
Shear Connector
Concrete Slab

Figure 7.3 Temperature Distribution Diagram for Profiled Slab, After Cooke et al. (1988)
Figure 7.4 Shear Capacity of Shear Connectors

- Finite Element Model
- Zhao and Aribert (1992) Data
Figure 7.5 Slip According to Temperature
### Figure 7.6 Stress Distribution at Different Loading Levels According to Time

<table>
<thead>
<tr>
<th>Load (kN)</th>
<th>Stress Distribution at time t = 10 min</th>
<th>Stress Distribution at time t = 90 min</th>
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250
Figure 7.7 Comparison Between Experimental Push Test and Finite Element Model for Solid Slab
Figure 7.8 Stress Contours and Deformed Shape for Solid Slab
Figure 7.9 Comparison between Experimental Push Test and Finite Element Model for Profiled Sheeting Slab
Figure 7.10 Stress Contours and Deformation Shape for Profiled Steel Sheeting Slab
Figure 7.11 Comparison of Solid Slab Push Tests with Temperature Changes According to Time
Figure 7.12 Shear Resistance of Shear Connectors at Elevated Temperatures According to Time

\[ \xi = 0.55 \]
Figure 7.13 Stress Contours and Deformed Shapes at Various Time Exposures

Time of fire exposure $t = 10$ min

Time of fire exposure $t = 30$ min

Time of fire exposure $t = 60$ min

Time of fire exposure $t = 90$ min

Time of fire exposure $t = 180$ min
Figure 7.14 Slip at Different Load Levels According to Time
Figure 7.15 Comparison of Profiled Slab Push Test with Temperature Changes According to Time
Figure 7.16 Shear Resistance of Shear Connectors at Elevated Temperature According to Time
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Figure 7.17 Stress Contours and Deformed Shapes at Various Time

Time of fire exposure \( t = 10 \) min

Time of fire exposure \( t = 30 \) min

Time of fire exposure \( t = 60 \) min

Time of fire exposure \( t = 90 \) min

Time of fire exposure \( t = 180 \) min
Figure 7.18 Slip at Different Load Levels According to Time
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Figure 7.19 Cross-Sectional Areas for Solid Slab and Profiled Slab

(a) Solid Slab
(b) Profiled Slab
CHAPTER 8

EFFECTS OF STRAIN REGIMES ON HEADED STUD SHEAR CONNECTORS FOR COMPOSITE STEEL-CONCRETE BEAMS

8.1 INTRODUCTION

The push test is meant to determine the basic strength of headed stud shear connectors in composite steel-concrete beams. It is always considered reliable as a lower bound method for the prediction of strength and stiffness in real composite steel-concrete beams. The stress state in and around the headed shear studs is highly complex. Metal decking used to form composite slabs have also been used standard push tests to assess the strength and ductility of headed stud shear connectors. Once again this has proven to be a reliable method for sheeting with dove-tailed and re-entrant profiles. The introduction of trapezoidal profiles of the latter has caused some concern about the methodology in which to assess the headed stud strength and ductility. The purpose of this chapter is to attempt to address some of the issues associated with the shear stud behaviour consisting of trapezoidal profiles.

The strength and ductility of headed stud shear connectors of steel and concrete composite beams is normally determined from a standard push test as outlined by Eurocode 4. Figure 8.1 illustrates push tests for solid and profiled slabs. The objective of the study was to carry out numerical push testing of similar sized specimens
to the conventional Eurocode 4, British Standards Institution (2004) push test. Backbreaking failures have been shown to be suppressed in numerical analyses, and thus, it is not considered necessary to model four ribs explicitly. When a push test is carried out, the shear connectors are subjected to pure shear, however, in comparison, when a composite steel-concrete beam is subjected to bending, the shear connectors will be subjected to both bending and shear.

Even though headed stud shear connectors can resist a complex combination of shear, bending and axial loads, magnitudes of these forces can change during a push test. Therefore, the behaviour of headed stud shear connectors also depends greatly on the geometry of the push test specimens and the strength of all the materials. In this study, the author has used pure shear as a way of identifying the applied load to the specimens in order to address the real stress state of headed stud shear connectors that suppress secondary failure modes, and thereby simulate the real situation in beams illustrated by Hicks (2007) and Ranzi et al. (2004).

8.2 EXPERIMENTAL SPECIMEN DETAILS

8.2.1 General

These experimental studies considered the behaviour of strain regimes for the concrete element. As discussed above, when the structure is subjected to pure shear, the longitudinal strain is constant throughout the section, however when it is subjected to pure bending, the strain gradually changes throughout the depth of the section. A linear variation in strain is schematically shown in Figure 8.2, and the value of strain is represented using a parameter defined as $\alpha$. When the structure is
subjected to pure shear, the strain distribution is represented by Figure 8.2a where \( \alpha = 1 \). For the case when the structure is subjected to pure bending, the strain distribution is represented by Figure 8.2e where \( \alpha = -1 \). Figures 8.2b, 8.2c and 8.2e illustrate the different variations of strain which represent the different locations of the neutral axis.

In order to modify the strain regimes in a concrete element from pure shear to pure bending, the theory of elasticity of Hooke’s law according to Popov and Balan (1998) was used. The author considered the cross-section of the composite steel-concrete beam deform by a parallel system of small forces, \( F_i \) in the axial direction, as shown in Figure 8.3. A combination of the two concepts above will enable the axial force acting in each cross-section to be determined.

### 8.2.2 Experimental Investigation for Push Tests

Initially, there are two different experimental investigations which are compared with the finite element analysis developed herein. The push tests performed by Lam and El-Lobody (2005) considered shear connection of both solid and profiled slabs. The author modified Lam’s solid slab to consider a profiled slab in order to evaluate the differences between the solid and profiled slabs. With the modified profiled slab, the author only took into account the presence of the shear stud in the middle position of the shear connectors in the profiled sheeting.

Mirza and Uy (2007) stated that the strength of the shear connectors can be increased by 11% when they are placed on the strong side when compared with the weak position. Conversely, when the shear connectors were positioned on the weak
side, the strength of the shear connectors displayed a reduction of 13%. The reader should bear in mind that the results shown are only applicable to the range of parameters studied herein.

Qualification needs to be made against the range of parameters such as the thickness of steel sheeting, shape of trough sheeting, arrangement and number of headed stud shear connectors in the trough as stated by Johnson and Yuan (1998a) and Johnson and Yuan (1998b). Further numerical analysis of these parameters could prove useful in the future.

The author also considered the experiments conducted by Hicks (2007) for both the push and beam tests as a comparison with the finite element analysis for the profiled slabs. For these analyses, the author used the shear studs that were placed in the favourable position as it was adopted in the experimental programme.

**8.2.3 Parametric Studies for the Push Tests**

The author also have undertaken extensive parametric studies by considering different types of profiled steel sheeting and different concrete strength property and their effects on the shear connector behaviour when different strain regimes were considered. Three different types of profiled steel sheeting were considered, and they include G1, G2 and G3 profiled steel decks. The dimensions, profiled steel sheeting concrete strength property details are illustrated in Tables 8.1 and 8.2.
8.3 NUMERICAL ANALYSIS: COMPARISON WITH EXPERIMENTAL PUSH TESTS

8.3.1 General

The studies show that the finite element push tests were modelled to determine the load-slip behaviour of the shear connectors when the strain regimes of the concrete element were being modified. The first series of analyses considered was compared with the experimental investigations undertaken by Lam and El-Lobody (2005) for solid slabs. The author then modified the solid slab to simulate a profiled slab to consider the differences in the results.

8.3.2 Solid Slab for Lam and El-Lobody (2005)

It can be observed in Figure 8.4 that the stiffness is similar for both the experiments and the finite element analyses within the elastic region. The experimental study showed that the maximum shear connector capacity was 118 kN whilst the finite element result obtained was 119 kN. This shows that the finite element model accurately analysed the experiments with a discrepancy of 0.7 %. When the model is subjected to bending where $\alpha = -1$, after the elastic region, the finite element analysis had a higher shear connector capacity when compared with the model subjected to pure shear where $\alpha = 1$.

From the finite element model, the author has also shown that when the composite steel-concrete beam is subjected to pure shear, the structure is subjected to the most detrimental loading case. This can be verified from Figure 8.4 where the model is subjected to bending, the shear connector capacity increased by 2.2 % to
121 kN. Even though the loading behaviour of the push test changed, 2.2 % of increment in the shear connector capacity is not significant.

In order to look at the different behaviour of the shear connectors when exposed to different loading conditions with varying neutral axes, Figure 8.4 establishes that the different strain regimes in solid slabs is not critical where increments in 0 %, 0.7 % and 1.3 % for $\alpha = 0.5$, $\alpha = 0$ and $\alpha = -0.5$, respectively. Therefore, it is illustrated that the push test to determine the shear connector capacity for composite steel-concrete solid slabs is reliable and accurate.

From Figure 8.4, it is observed that the failure mode changes when the value of $\alpha$ changes. When $\alpha = 1$, the failure mode is governed by shear stud failure. When $\alpha = 0.5$, 0, -0.5 and -1, the failure mode is governed by concrete failure where the concrete starts to crack surrounding the shear connector due to the addition of load when the value of $\alpha$ changes. The failure modes can also be observed graphically in Figure 8.5.

### 8.3.3 Profiled Slab for Modified Lam and El-Lobody (2005)

Figure 8.6 illustrates that the stiffness is similar for the finite element analyses within the elastic region. The finite element analysis showed that the maximum shear connector capacity was 68 kN when $\alpha = 1$ for the push test under pure shear, however when the model is subjected to the bending condition where $\alpha = -1$, the maximum shear connector capacity was 92 kN. This shows a significant increment of 26.6 %. The result is expected when the push test changes from pure shear to pure
bending due to the loading condition for the worst case scenario when compared with pure bending.

Figure 8.6 illustrates that the different strain regimes in the profiled slab are crucial where an increment in 6.8, 12.5 and 18.5 % for $\alpha = 0.5$, 0 and -0.5, respectively. Therefore, it is shown that the push test to determine the shear connector capacity for composite steel-concrete profiled slabs is unreliable, as it does not allow for the inclusion of the beneficial effects of bending.

From Figure 8.6, it can be seen that the failure mode changes when the value of $\alpha$ changes. When $\alpha = 1$ and 0.5, the failure mode is governed by concrete failure where the concrete cracks through the middle of the trough. This failure mode observed is similar to experimental studies undertaken by El-Lobody and Young (2006). When $\alpha = 0$, -0.5 and -1, the failure mode is dominated by stud failure where the shear connectors started to shear off the concrete, causing the concrete to crack surrounding the shear connector. The failure mode also can be observed in Figure 8.7

8.3.4 Hicks (2007) Experimental Studies

The second experimental series considered was that associated with the experiments conducted by Hicks (2007) for both push tests and beam tests. Figure 8.8 shows the comparison between the experimental tests undertaken by Hicks’ and the finite element model performed by the author. The results are as stated by Hicks (2007), the initial similarity is the stiffness for both the push test and beam test. However, the maximum shear connector capacity and slip measured in the push test
is well below the level attained in the beam test. Figure 8.8 verifies the accuracy of
the finite element analysis. For the push tests, the experimental study showed a
maximum shear capacity of 84 kN and the finite element analysis showed a value of
89 kN with a discrepancy of 5.7 %. As for the beam tests, the experimental study and
finite element analysis illustrated that the maximum shear capacities were 124 kN
and 128 kN, respectively, with a discrepancy of 3.3 %.

Furthermore, Figure 8.8 shows that the maximum shear connector capacity for
push tests was 89 kN, whilst for the beam tests, the maximum shear connector
capacity was 128 kN. This illustrates an increment of 30.6 %. From the experimental
studies and finite element analyses, failure in the push tests was governed by
concrete failure at the trough rather than stud failure. When the simulated beam test
is considered, the failure is dominated by stud failure. The behaviour of the shear
connectors in the push test provides a conservative result and they do not replicate
the strength and ductility that can be achieved in a beam test. Therefore, the standard
push test to determine the shear connector capacity for a profiled slab is
questionable.

8.4 PARAMETRIC STUDIES

8.4.1 General

The finite element analysis has allowed parametric studies to be carried out, and
the case that the author solved has been modelled, particularly in calibration with
independent experimental test results undertaken by Hicks (2007). This may not be
the case for real buildings. However, the trends in this chapter may be of use for
further studies. In this case, it has been shown that the finite element model correctly predicts the behaviour of shear connectors in composite steel-concrete structures for both solid and profiled slabs. For solid slabs, when the strain regimes were varied from pure shear to pure bending, the maximum capacity did not show any critical change, whilst for profiled slabs, any minor change was crucial. Hence, a parametric study was conducted to study the effects of the capacity and behaviour of shear connectors by changing the profiled steel sheeting geometries.

A total of fifteen push tests were investigated in the parametric study. The push tests were divided into three major groups. These were G1, G2 and G3. Each group included five different push tests having different $\alpha$ values with the same type of profiled steel sheeting, as described in Figure 8.2. The dimensions of the profiled steel sheeting and details of the parametric study are explained in Table 8.1 and shown in Figure 8.9.

Figure 8.10 illustrates the force-slip relationship for profile steel sheeting Type G1, G2 and G3. It can be seen that between the three profiled steel sheeting types, G1 can withstand larger forces when compared with the other two sheeting types. Therefore, the G1 type has a higher strength capacity and is considerably more ductile. Figure 8.10 reveals a maximum shear connector capacity of 135, 125, and 128 kN for G1, G2 and G3 profiled steel sheeting, respectively. G1 illustrates that it has a shear connector capacity 4.8 % higher than that of the G3 trapezoidal profiled sheeting and 7.3 % higher than that of the G2 profiled steel sheeting. Figure 8.10 also shows that the shear strength capacity of composite steel-concrete profiled slabs is greatly dependent on the width and depth of the ribs of the profiled steel sheeting.
8.4.2 G1 Profiled Slab

The first group of parametric studies, the author varied the strain regimes from $\alpha = 1, 0.5, 0, -0.5$ and -1 for the G1 profiled steel sheeting. Figure 8.11 depicts the force-slip relationship of the G1 profiled steel sheeting with the variation of $\alpha$ value. It can be observed that the composite steel concrete beam increases the shear strength and ductility when the structure is varied from pure shear to pure bending. When the structure is subjected to pure shear, the maximum shear connector capacity of 98 kN is achieved. However, when it is subjected to pure bending, the maximum shear capacity increases to 135 kN, which is a 27.2 % shear capacity increment.

In order to add the $\alpha$ value into the force-slip relationship, the Aribert and Labib (1982) force-slip relationship was used which is described in Equation 8.1.

$$Q = Q_u \left(1 - e^{-c_1 d}\right)^3$$

where,

$Q$ is the force of the shear connector

$Q_u$ is the ultimate force

$c_1$ and $c_2$ are the parameters of the model

$d$ is the slip capacity of the shear connector

Due to $Q_u, c_1$ and $c_2$ are the changing parameters in the equation, the relationship of these values with respect to $\alpha$ is shown in Equation 8.2. Each of the parameter values with respect to $\alpha$ can be obtained in Table 8.3 and plotted in Figures 8.12 to 8.14.
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\[ Q = f_1(\alpha)(1 - e^{-f_2(\alpha)d})^{f_3(\alpha)} \]  

(8.2)

8.4.3 G2 Profiled

The second group of parametric studies, the author varied the strain regimes which included \( \alpha = 1, 0.5, 0, -0.5 \) and -1 for G2 profiled steel sheeting. Figure 8.15 shows the force-slip relationship of G2 profiled steel sheeting with the variation of \( \alpha \) value. The observation of the shear strength and ductility is similar to that mentioned in Section 8.4.2 above. When the structure is subjected to pure shear and pure bending, the maximum shear connector capacities are 94 kN and 125 kN, respectively, which is a 25 % shear capacity increment. The definition of \( f_1, f_2 \) and \( f_3 \) with respect to \( \alpha \) is determined in Table 3 and plotted in Figures 8.16 to 8.18.

8.4.4 G3 Profiled

The third group of parametric studies, G3 is used to verify Hicks’ trapezoidal profiled steel sheeting. Figure 8.19 illustrates the force-slip relationship of G3 trapezoidal profiled steel sheeting with the variation of \( \alpha \) value. The results illustrate that the shear strength and ductility is similar as mentioned in Section 8.4.2 above. When the structure is subjected to pure shear and pure bending, the maximum shear connector capacity is 91 kN and 128 kN respectively which is a 29 % increase in shear capacity. The definition of \( f_1, f_2 \) and \( f_3 \) with respect to \( \alpha \) is determined in Table 3 and plotted in Figures 8.20 to 8.22.
8.4.5 Hicks (2007) Experimental Studies

An example of Hicks (2007) experimental study was used to verify the accuracy of these equations. Figure 8.23 shows the cross-section through the beam, and from Figure 8.23, the calculation to determine the $\alpha$ value is as below:

\[ f'_c = 28 \text{ N/mm}^2 \]
\[ f'_y = 378 \text{ N/mm}^2 \]
\[ N_{\text{concrete}} = 0.85 f'_c BD_n \]
\[ N_{\text{concrete}} = 0.85(28)(2500)(80) \]
\[ N_{\text{concrete}} = 4760 \text{kN} \]
\[ N_{\text{steel}} = 2N_{\text{flange}} + N_{\text{web}} \]
\[ N_{\text{steel}} = 2b_f d_f f'_y + b_w d_w f'_y \]
\[ N_{\text{steel}} = 2(10.4)(167)(378) + 7.3(289.2)(378) \]
\[ N_{\text{steel}} = 2111 \text{kN} \]

From the above equations,

\[ N_{\text{concrete}} > N_{\text{steel}} \]

Therefore the neutral axis, N.A. falls in the concrete element.

Compression, C = Tension, T

\[ 0.85(28)(2500)d_n = 2111 \]
\[ d_n = 35.5 \text{mm} \]

In order to find the value of $\alpha$

\[ \alpha = \frac{d_n}{D} \]
\[ \alpha = -\frac{35.5}{140 - 35.5} \]
\[ \alpha = -0.34 \]
From Figures 8.22 to 8.24, the $Q_u$, $c_1$ and $c_2$ can be determined and the force-slip relationship is derived as shown in Equation 8.3.

$$Q = 115.98 \left(1 - e^{-0.92d}\right)^{0.41}$$ (8.3)

Equation 8.3 is plotted in Figure 8.24, and from Figure 8.24, it is verified that when the correct $\alpha$ value of -0.34 is determined, the discrepancy of the result is only 0.3% when compared with the initial assumption of $\alpha = -1$ where the discrepancy is 4%.

### 8.5 PROFILED SLABS COMPARED WITH AVAILABLE INTERNATIONAL STANDARDS

A comparison was made to evaluate the finite element solutions from G1, G2 and G3 with three existing international standards. They included the Australian Standard AS 2327.1 (2003), British Standards Institution (2004) and American Standard AISC (1999).

Two failure modes can be observed to investigate the ultimate shear capacity of the shear connector in the push tests. They include shear failure and concrete failure. The fracture of studs will often govern when the concrete strength is relatively high and it is given by Equations 8.4(a, b and c).

$$P_{AS} = 0.63d^2 f_{us}$$ (8.4a)

$$P_{EC} = 0.628d^2 f_{us}$$ (8.4b)
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\[ P_{\text{ASC}} = 0.785d^2 f_{us} \]  \hspace{1cm} (8.4c)

where,

- \( E_{sc} \) is the modulus of elasticity of the shear connector,
- \( f_c \) is the compressive strength of the concrete,
- \( f_{us} \) is the ultimate strength of the shear connector.

However, when the shear connector is stronger than concrete strength, Equations 8.5(a, b and c) will tend to govern.

\[ P_{\text{AS}} = 0.31d^2 \sqrt{f_{ck} E_c} \]  \hspace{1cm} (8.5a)

\[ P_{\text{EC}} = 0.29d^2 \sqrt{f_{ck} E_c} \]  \hspace{1cm} (8.5b)

\[ P_{\text{ASC}} = 0.39d^2 \sqrt{f_{ck} E_c} \]  \hspace{1cm} (8.5c)

where,

- \( f_{ck} \) is the characteristic compressive cylinder strength of the concrete
- \( E_c \) is defined as the mean modulus of elasticity of the concrete.

When the steel profiled slab is taken into consideration, equations from 8.4 and 8.5 above are multiplied by a reduction factor \( k \). The value of \( k \) is defined in the equations below:

\[ k_{\text{AS}} = 1.148 - (0.18 / \sqrt{n}) \]  \hspace{1cm} (8.6a)

\[ k_{\text{EC}} = \left( \frac{0.7}{\sqrt{n}} \right) \left( \frac{h_o}{h_p} \right) \left( \frac{h}{h_p} - 1 \right) \leq k_{i,\text{lim}} \]  \hspace{1cm} (8.6b)

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\[ k_{ASC} = \left( \frac{0.85}{\sqrt{n}} \right) \left( \frac{b_o}{h_p} \right) \left( \frac{h}{h_p} \right) - 1 \leq 1 \]  \hspace{1cm} (8.6c)

where,

- \( n \) is the number of studs
- \( b_o \) is the width of the steel profiled rib
- \( h_p \) is the height of the decking profile
- \( h \) is the height of the stud

A summary of the ultimate loads for the shear connector, \( P \) which are calculated using the Australian Standard, Eurocode and American standards, is given in Table 8.4. Moreover, the author analysed the finite element model using different concrete strength which includes \( f'_c = 20, 25, 32, 35, 40, \) and \( 50 \) N/mm\(^2\). Table 8.4 verifies that when the concrete strength is lower concrete failure will govern whilst when the concrete strength is higher, stud fracture tends to govern.

For concrete strength with \( f'_c = 20 \) to \( 35 \) N/mm\(^2\), the finite element model demonstrated that the concrete stress reached a maximum value before the stress in the shear studs reached maximum. Therefore, this verified that the push tests failed due to concrete failure. The failure modes predicted by existing standards were also similar to the finite element models. From Table 8.4, the finite element model shows a more conservative value when compared with the Australian and American standards. On the other hand, the Eurocode seems to be overly conservative.

When the concrete strength increases, the finite element model shows that the shear stud reaches its maximum stress before the concrete slab. This proved that the
failure mode is governed by stud fracture. When compared with the existing standards, the calculated theoretical values for both the Australian and American standards seem to provide higher values when compared with the finite element models. It is confirmed that the finite element model is more conservative than the two existing standards. On the contrary, the Eurocode seems to be very conservative, where the calculated theoretical value is lower than the finite element predicted value.

From the analyses above, the author are also able to determine the relationship of ultimate load capacity of headed stud shear connectors with varying concrete properties and for different types of profiled sheeting. Equations 8.7, 8.8 and 8.9 shown are for G1, G2 and G3 profiled sheeting, respectively, and are illustrated in Figure 8.25. Figure 8.25 enables engineers to input the concrete properties and ascertain the ultimate capacities of headed stud shear connectors.

\[
F_u = -0.0168 f_c^2 + 2.56 f_c + 28.50 \quad (8.7)
\]

\[
F_u = -0.0097 f_c^2 + 2.11 f_c + 31.92 \quad (8.8)
\]

\[
F_u = -0.0069 f_c^2 + 1.86 f_c + 32.22 \quad (8.9)
\]

where,

- \( F_u \) is the ultimate load capacity of the headed stud shear connector
- \( f_c \) is the concrete property
8.6 DESIGN MODELS AND RECOMMENDATIONS

An extensive parametric study of fifteen push test specimens with different profiled steel sheeting geometries was performed using the established finite element models. The comparison of shear connector capacities obtained from the finite element models proved that they depend significantly on the width and rib types of profiled steel sheeting.

The introduction of the parameters such as $\alpha$, $Q_{ul}$, $c_1$ and $c_2$ are the key characteristics of the proposed method because they account for the behaviour of the shear connection nonlinearity effects. When accurate parameter values were applied to the force-slip relationship equations, it seems to suggest that the proposed method has less discrepancy when compared with the experimental study, and it is also useful for design engineers using these profiles in industry.

When the Australian, Eurocode and American Standards are compared with the finite element results, the Australian and American standards appeared to overestimate the shear connector capacity, whilst the Eurocode grossly underestimated the strength of shear connectors for profiled slabs. From the three existing standards, the American Standard seems to be less conservative where higher ultimate loads were observed.

Graphs based on the numerical analyses were produced and are suitable for practising engineers to estimate the shear capacity of headed stud shear connectors for design purposes.
8.7 SUMMARY OF CHAPTER

Shear connection nonlinearity always results in significant changes in the strength and ductility of composite steel-concrete beams. In order to highlight such phenomena, a non-linear procedure with different strain regimes implemented in the concrete element has been presented to account for both the shear connection strength and ductility.

A comparison with experimental results allows the validation of such a procedure with reference to both solid and profiled slabs for both push and beam tests. The finite element analysis that has been undertaken in this chapter demonstrated that the push test to determine the shear connection capacity for solid slabs is reliable and accurate. However, this is not the case for slabs with profiled steel sheeting.
Table 8.1 Profiled Steel Sheeting Dimensions for Parametric Study

<table>
<thead>
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<th>Group</th>
<th>Specimen</th>
<th>Dimensions</th>
<th>Strain Profiles</th>
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<tr>
<td></td>
<td></td>
<td>Profiled Sheeting</td>
<td>Stud</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$b_0$ (mm)</td>
<td>$h_p$ (mm)</td>
</tr>
<tr>
<td>G1</td>
<td>G1-1</td>
<td>136</td>
<td>60.9</td>
</tr>
<tr>
<td></td>
<td>G1-2</td>
<td>136</td>
<td>60.9</td>
</tr>
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<td></td>
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<td>G3-3</td>
<td>144</td>
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<td>G3-4</td>
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### Table 8.2 Dimensions and Profiled Steel Sheeting Details of Parametric Study with Different Concrete Strength Property

<table>
<thead>
<tr>
<th>Group</th>
<th>Concrete Property (N/mm²)</th>
<th>Dimensions</th>
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<td></td>
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<td>Profiled Sheeting</td>
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<tr>
<td></td>
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<td>G2</td>
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<td>G3</td>
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</table>
Table 8.3 Parameter of Force Slip Relationship with Respect to $\alpha$.

<table>
<thead>
<tr>
<th>Profiled Type</th>
<th>$f_1(\alpha) = a\alpha^1 + b\alpha^2 + c\alpha + d$</th>
<th>$f_2(\alpha) = e\alpha^4 + f\alpha^3 + g\alpha^2 + h\alpha + j$</th>
<th>$f_3(\alpha) = k\alpha^4 + l\alpha^3 + m\alpha^2 + n\alpha + p$</th>
</tr>
</thead>
<tbody>
<tr>
<td>G1</td>
<td>$a = -17.94$, $b = 117.2$, $c = -0.06$, $d = 0.13$, $e = 0.12$, $f = 1.2$, $g = -0.07$, $h = -0.03$, $j = 0.17$, $k = 0.03$, $l = 0.03$, $m = 0.4$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>G2</td>
<td>$a = 13.30$, $b = -1.51$, $c = -28.60$, $d = 111.5$, $e = 0.15$, $f = 0.15$, $g = 0.35$, $h = 0.96$, $j = 0.02$, $k = -0.07$, $l = -0.03$, $m = 0.07$, $n = 0.4$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>G3</td>
<td>$a = -18.36$, $b = 109.74$, $c = 0.67$, $d = 0.2$, $e = 0.27$, $f = 0.3$, $g = 1.05$, $h = -0.07$, $j = -0.03$, $k = 0.17$, $l = 0.03$, $m = 0.4$</td>
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## Table 8.4 Comparison of Ultimate Load for Shear Connector between Finite Element Models and International Standards

<table>
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<tr>
<th>Profiled Type</th>
<th>Concrete Property, $f'_c = 20 \text{ N/mm}^2$</th>
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<th>Eurocode</th>
<th>American Standard</th>
<th>FEM/Standard</th>
<th>FEM Result</th>
<th>AS</th>
<th>EC</th>
<th>AISC</th>
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<td>FEM Result (kN)</td>
<td>SF</td>
<td>CF</td>
<td>SF</td>
<td>CF</td>
<td>SF</td>
<td>CF</td>
<td>SF</td>
<td>CF</td>
</tr>
<tr>
<td>G1</td>
<td>73.1 (CF)</td>
<td>116.1</td>
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<td>80.5</td>
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<tr>
<td>G2</td>
<td>70.3 (CF)</td>
<td>116.1</td>
<td>74.6</td>
<td>113.5</td>
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<td>111.8</td>
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<td>74.6</td>
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<td>CF</td>
<td>SF</td>
<td>CF</td>
<td>SF</td>
<td>CF</td>
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<td>89.6</td>
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<td>89.6</td>
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### Concrete Properties, $f'_{c} = 35$ N/mm$^2$

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<td>CF</td>
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<td>G3</td>
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<td>110.7</td>
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### Concrete Properties, $f'_{c} = 40$ N/mm$^2$

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<th>American Standard</th>
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<tr>
<td></td>
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<td>SF</td>
<td>CF</td>
<td>SF</td>
<td>CF</td>
</tr>
<tr>
<td>G1</td>
<td>103.7 (SF)</td>
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<td>82.4</td>
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### Concrete Properties, $f'_{c} = 50$ N/mm$^2$

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<tbody>
<tr>
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<td>CF</td>
<td>SF</td>
<td>CF</td>
</tr>
<tr>
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<td>141.7</td>
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<td>110.1 (SF)</td>
<td>116.1</td>
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<td>85.1</td>
<td>97.6</td>
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</table>

* SF denotes stud failure and CF concrete failure and
* Shaded area denotes failure mode govern
*All dimensions in mm

Figure 8.1 Push Tests for Both Solid and Profiled Slab
Figure 8.2 The Value of $\alpha$ and Location of Neutral Axis for the Concrete Element
Figure 8.3 Behaviour of Composite Steel-Concrete Beams Subjected to Modified Axial Force
Figure 8.4 Lam and El-Lobody (2005) Experiment Compared with Different Strain Regimes
$\alpha = 1$, Stud Failure

$\alpha = -1$, Concrete Failure Before Stud Reached Maximum Capacity

Figure 8.5 Stress Contours and Failure Modes for Solid Slabs
Figure 8.6 Lam and El-Lobody’s Modified Push Test with Different Strain Regimes
CHAPTER 8 – Effects of Strain Regimes

\[ \alpha = 1, \text{Concrete Failure at Trough} \]

\[ \alpha = -1, \text{Stud Failure Before Concrete Reached Maximum Stress} \]

Figure 8.7 Stress Contours and Failure Modes for Profiled Slabs
Figure 8.8 Comparison of Experimental Data and Finite Element Result for Hicks (2007)
Figure 8.9 Dimensions of Profiled Steel Sheeting
Figure 8.10 Comparison of Force Slip Relationship for Different Types of Profiled Steel Sheeting
Figure 8.11 Force Slip Relationship for G1 Profiled Steel Sheeting with Variations of $\alpha$ Values
Figure 8.12 Ultimate Load with Respect to $\alpha$ for G1 Profiled Steel Sheeting
Figure 8.13 $c_1$ with Respect to $\alpha$ for G1 Profiled Steel Sheeting
Figure 8.14 $c_2$ with Respect to $\alpha$ for G1 Profiled Steel Sheeting
Figure 8.15 Force Slip Relationship for G2 Profiled Steel Sheeting with Variations of $\alpha$ Values
Figure 8.16 Ultimate Load with Respect to $\alpha$ for G2 Profiled Steel Sheeting
Figure 8.17. $c_1$ with Respect to $\alpha$ for G2 Profiled Steel Sheeting
Figure 8.18 $c_2$ with Respect to $\alpha$ for G2 Profiled Steel Sheeting
Figure 8.19 Force Slip Relationship for G3 Profiled Steel Sheeting with Variations of $\alpha$ values
Figure 8.20 Ultimate Load with Respect to $\alpha$ for G3 Profiled Steel Sheeting
Figure 8.21 $c_1$ with Respect to $\alpha$ for G3 Profiled Steel Sheeting
Figure 8.22 $c_2$ with Respect to $\alpha$ for G3 Profiled Steel Sheeting
Figure 8.23 Beam Cross Section for G3 Profiled Steel Sheeting
Figure 8.24 Comparison between Experimental Data and Finite Element Model for Hick’s (2007) Trapezoidal Profiled Steel Sheeting
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Figure 8.25 Ultimate Load Versus Concrete Properties for Different Profiled Steel Sheeting
CHAPTER 9

EFFECTS OF THE COMBINATION OF AXIAL AND SHEAR LOADING ON HEADED STUD SHEAR CONNECTORS FOR COMPOSITE STEEL-CONCRETE BEAMS

9.1 INTRODUCTION

Under extreme loading situations, the performance of a structure is largely related to the inelastic behaviour of its critical members and components. In multi-storey buildings subjected to extreme conditions, such as severe impact, blast or seismic effects, some members are expected to undergo deformation levels which are well beyond those tolerated under normal load combinations. Depending on the configuration of the structure and the type of loading, realistic assessment of the overall performance of the structure necessitates a detailed evaluation of the response of such key elements. In addition to an appropriate determination of the capacity of these members, it is also imperative to examine and quantify their ductility and residual strength characteristics for headed stud shear connectors.

In the last few years, numerical modelling especially has been used to investigate the behaviour of composite steel and concrete beams. Citipitioglu et al. (2002) studied the effect of bolt pre-tension and the friction coefficient of adjacent surfaces on the connections behaviour in detail using the finite element modelling approach.
The authors used brick elements to model all connection components. Kishi et al. (2001) also investigated the behaviour of the similar type of connection using the finite element method in order to estimate the behaviour of semi-rigid connections. Ahmed et al. (2001) examined the prying action of shear connectors in top and seat angle connections using finite element model. They concluded that the prying force is related to the shear connectors’ diameter, spacing, and top and seat angle thickness. Pirmoz (2006) also applied the finite element method in studying the connection behaviour and dynamic parameters of semi-rigid frames.

Furthermore, the behaviour of bolted and welded double angle connections under shear and tension using the finite element method, together with an experimental work, has been studied by Yang et al. (2000) and Hong et al. (2001). These studies showed that these types of connections, under a combination of axial and shear force, behave like simple shear connections.

There is no current research on the effect of the combination of axial and shear loading on the behaviour of headed stud shear connectors. It is necessary to analyse the headed stud shear connector behaviour before designing composite steel and concrete structures.

In order to increase the strength and stiffness of composite steel-concrete structures, the slip between the structural steel beam and concrete slab has to be minimised. One way of achieving this is by welding the headed stud shear connectors on the upper flange and embedded into the concrete slab. As mentioned in Chapter 2, the increased use of headed stud shear connectors under a combination
of axial and shear loading is due to the development of composite wall systems. For wall systems, the headed studs are subjected to a combination of shear and tensile axial loading.

9.2 EXPERIMENTAL SPECIMEN DETAILS

9.2.1 Experimental Investigation for Push Tests

Experimental studies undertaken by Saari et al. (2004) were compared with the finite element analysis developed herein which address the application of combined axial tensile and shear loading in push tests. Figure 9.1 depicts a modification of the standard push test setup, made to accommodate the application of combined axial tensile and shear loading. Moreover, there are two different types of steel reinforcement configurations that were used. They consist of confined reinforcement around the shear studs and cage reinforcement to provide ample confinement around the shear studs. Figures 9.2 and 9.3 demonstrate the confined and cage reinforcement configurations, respectively. The concrete reinforcement ends are 38 mm from the head of the shear stud connector to make sure that the region is prone to concrete damage from cracking when axial tensile force being applied to the structure.

The specimen was designed to represent the local behaviour of the headed stud shear connectors when tensile axial load is implemented. The spacing of headed stud shear connectors was selected to be large enough, such that there would be no overlap in the vertical distribution of stresses due to a 45 ° failure cone forming for each headed stud shear connector when a tension axial force was applied. The thickness of the wall was based on realistic construction details, and no edge effects
would impede the headed stud shear connector behaviour. Lastly, the width of the wall was wide enough to ensure that the horizontal distribution of stresses at each wall would not affect the behaviour of the opposite interface.

The push tests performed by Becher (2005) on shear connection of solid slabs was used to consider the combination of axial tension and shear loading. The author also used experimental studies undertaken by Wu (2006) to consider a profiled slab in order to evaluate the differences between the solid and profiled slabs.

9.2.2 Parametric Studies for the Push Tests

The author also have undertaken parametric studies by considering different thicknesses of concrete slab for both the solid and profiled steel sheetings, and their effects on the headed stud shear connector behaviour under combined tensile axial and shear loading. Lastly, the author also analysed the models of Becher (2005) and Wu (2006) by using different tensile axial loading factors to develop an interaction diagram for the push test model.

9.3 NUMERICAL ANALYSIS: COMPARISON WITH EXPERIMENTAL PUSH TESTS

9.3.1 General

To evaluate the accuracy of the finite element modelling approach, finite element models are created according to Saari et al. (2004) experimental studies and the results are compared with experimental results. It can be seen that the results
obtained from finite element models are in good agreement with the experimental study data. Differences between the numerical simulation and experimental results may be the result of several causes, like numerical modelling simplification, test specimen defects, residual stress and shear connector pre-tension.

9.3.2 Saari (2004) Experiments

A modified push test specimen was developed to examine the behaviour of headed shear stud connectors when they are subjected to a combination of tensile axial and shear loading. From the experimental studies, the push test was exposed to two different types of reinforcement schemes. They included confined reinforcement and cage reinforcement. The experimental studies undertaken by Saari et al. (2004) were compared against finite element models developed by the author.

9.3.2.1 Confined Reinforcement Scheme

Figure 9.4 gives the value of ultimate load achieved when the push test is subjected to shear loading. The experiments yielded at a maximum load of 101 kN shear capacity for the push test, whilst the finite element model was 100 kN. The discrepancy was 1%. This proved that the finite element model was accurately modelled. For shear loading, the failure mode was governed by shear stud failure.

When the push test was subjected to both shear and axial loading, the failure modes observed include shear and tension failures of the concrete around the headed stud shear connectors and also of the headed shear stud connectors themselves. The failure mode for concrete in tension was only seen in the specimen due to the
confined reinforcement scheme where minimal reinforcement scheme was anticipated. The experimental study showed a maximum shear capacity of 60 kN, whilst the finite element result was 59 kN. The discrepancy was 1.7%. In Figure 9.4, it is observed that the ultimate load capacity of the headed stud shear connectors has a capacity reduction of 40%. This is due to the effect of tension failure for both the concrete and headed stud shear connectors when axial tension is taken into consideration.

Figures 9.5 and 9.6 compare the stress distribution of push tests under shear loading and the combination of shear and axial tension loading, respectively. Figure 9.5 indicates that both the structural steel beam and headed stud connectors reach their maximum stress value. Therefore, the failure is governed by the headed stud shear connectors. When tensile axial load is applied to the structures, Figure 9.6 shows the stress distribution in the concrete element reaches a maximum value followed by the stress in the headed stud shear connectors reaching a maximum. This illustrates that when the concrete starts cracking, the stress is transferred to the headed stud connectors until they have failed under axial tensile loading.

9.3.2.2 Caged Reinforcement Scheme

Figure 9.7 shows that when a cage reinforcement scheme was implemented in the push test, the initial behaviour was similar to that for a confined reinforcement scheme. When the push-test specimen was subjected to shear loading, the experimental studies resulted in a maximum shear capacity of 126 kN, whilst the
finite element result was 123 kN. The discrepancy is 2.4 %, and this illustrates that the finite element model was able to accurately model the experimental behaviour.

For shear loading, the failure mode was governed by yielding/or fracture of the shear stud failure, and this can be attributed to the same factor since the concrete for both specimens was initially undamaged. Besides, there was less stiffness degradation when compared with the confined reinforcement scheme. This was due to the fact that there was no observable concrete cracking present in the cage reinforcement scheme, and it is believed that the headed stud connectors had fully yielded in tension.

When the push-test specimen was subjected to combined shear and axial loading, the failure mode that governed the behaviour was cracking of the concrete, and this was due to the axial loading which caused the concrete to be subjected to a tensile force. However, the cage reinforcement scheme greatly reduced crack propagation. The experimental study showed a maximum shear capacity of 76 kN, whilst the finite element result was 78 kN. The discrepancy of 2.6 % meant that the cage reinforcement scheme showed a higher shear capacity when compared with the confined reinforcement scheme. It also prevented concrete cracking and the pull out of the headed stud from the concrete when axial force is subjected.

Figures 9.8 and 9.9 reveal the stress distributions of the push tests under shear loading and the combination of shear and axial tension loading, respectively. Figure 9.8 illustrates that the failure mode is governed by fracture of the headed stud connectors where the headed stud connectors reach their maximum stress value.
When the tensile axial load is applied to the structure, the failure mode changes to concrete failure as illustrated in Figure 9.9.

9.4 INTERACTION DIAGRAM COMPARED WITH AVAILABLE EQUATIONS

The primary objective of this investigation of headed stud shear connectors is to understand the behaviour under combined shear and axial tensile loading. In order to develop an interaction diagram and design criteria for headed stud connectors subjected to combined shear and axial tensile loading, the headed stud shear connectors were analysed under different loading conditions. The headed stud connectors subjected to pure shear and pure axial tension were analysed. Three types of failure modes were determined. They include failure of the headed stud shear connectors, severe concrete cracking failure, and concrete pullout failure.

The strength of specimens tested by Becher (2005) and Wu (2006) are compared with the interaction formulas for headed stud shear connectors subjected to combined shear and tension according to McMackin et al. (1973), and given in Equation 9.1. All the values for shear and axial tension are normalised by $A_s F_{u}$.

$$\left( \frac{P_{sd}}{A_s} \right)^{\mu_1} + \left( \frac{N_{sd}}{N_d} \right)^{\mu_2} = 1$$  \hspace{1cm} (9.1)

where,

- $P_{sd}$ is the applied shear load
- $P_d$ is the shear capacity of the headed shear stud = $1.106A_s f_c^{0.3} E_c^{0.44} \leq N_d$

$$P_d = 1.106A_s f_c^{0.3} E_c^{0.44}$$
\(N_{ad}\) is the applied axial tensile load

\(N_d\) is the axial tensile capacity of the headed shear stud = \(\sigma_s A_s\)

\(A_s\) is the area of the headed shear connector

\(\mu_1\) and \(\mu_2\) is 5/3 for solid slab and 3/2 for profiled slab

\(f'c\) is the 28 days compressive strength of concrete

\(E_c\) is the modulus of elasticity of the concrete

\(\sigma_u\) is the ultimate tensile stress of the headed stud shear connector

Figure 9.10 summarises the results of the combined shear and axial tensile loading for solid slabs. The loading configuration for both of these forces was monotonic. The finite element model and Equation 9.1 give nearly identical results. However, the finite element model seems to be more conservative when compared with Equation 9.1. Under a combination of loading the increased tensile capacity was not a significant factor and only slightly increases in the tensile and shear components were observed.

Figure 9.10 also specifies the comparison between solid slab from Becher (2005) model and profiled slab from Wu (2006) model. From Figure 9.10, it is indicated that the increment of axial tensile loading will cause the shear capacity to decrease. When compared with the solid slab, the profiled slab loses its shear capacity higher than the solid slab. If the axial tensile load is increased to 50 \%, Figure 9.10 reveals that the profiled slab reduces its shear strength capacity twice as much as the solid slab due to concrete cone failure. From the finite element analysis the author proposed that \(\mu_1\) and \(\mu_2\) is equivalent to 3/2 for profiled slab. One improvement that
could be made is to modify the reinforcement scheme to prevent the concrete slab from concrete cone failure.

9.5 PARAMETRIC STUDIES

9.5.1 Becher (2005) Experiments

Initially, the finite element result was compared with the experimental study undertaken by Becher (2005). Figure 9.11 shows that the finite element result is in good correlation with the experimental values. The experimental studies illustrate a maximum shear capacity of 105 kN for the push test, whilst the finite element result was 102 kN. The discrepancy was 2.9 %. This proves that the finite element model was accurate in predicting the behaviour.

A parametric study was undertaken where the variable parameter chosen was the slab thickness. The slab thicknesses chosen were 120, 150 and 200 mm. The stiffness was different between shear loading and combined axial and shear loading. The push test subjected to shear loading seems to have a higher stiffness when compared with the combination of axial and shear loading.

For shear loading, the 200 mm slab developed a higher shear stud capacity than the 120 mm slab. The shear capacities were 103, 104 and 114 kN for the slabs with 120, 150 and 200 mm thicknesses, respectively. For combined shear and axial loading, the reductions of the shear capacities were 25.4, 31.3 and 36.1 % for the 120, 150 and 200 mm slabs, respectively. For the finite element analyses, it is verified that the thicker slab will have a greater reduction in shear capacity. This is
because the thicker slab will cause more concrete cracking failure when subjected to axial loading.

In order to increase the shear stud capacity, the author adapted cage reinforcement scheme. When cage reinforcement was embedded into the concrete element, the shear stud capacity increased to 123, 125 and 137 kN for concrete slab of 120, 150 and 200 mm, respectively. When a combination axial and shear loading is implemented, Figure 9.12 shows the ultimate capacity of headed stud shear connector according to reinforcement scheme. When cage reinforcement scheme is implemented, the 120 and 150 mm slab changed their failure mode from concrete cracking failure to shear stud failure.

The ultimate load which relates to different concrete thickness of the push tests can be determined from Equations 9.2, 9.3, 9.4 and 9.5 for solid cage reinforcement with shear loading, solid confined reinforcement with shear loading, solid cage reinforcement with combination of axial and shear loading and solid confined reinforcement with combination axial and shear loading, respectively, and they are illustrated in Figure 9.12. The figure also shows that the ultimate load increases by approximately 20% when confined reinforcement scheme is used compared with cage reinforcement scheme.

\[ F_u = 0.002t^2 - 0.45t + 149 \]  \quad 100 \leq t \leq 200 \text{ mm} \quad (9.2)

\[ F_u = 0.002t^2 - 0.38t + 124 \]  \quad 100 \leq t \leq 200 \text{ mm} \quad (9.3)

\[ F_u = 0.005t^2 - 1.67t + 229 \]  \quad 100 \leq t \leq 200 \text{ mm} \quad (9.4)

\[ F_u = 0.002t^2 - 0.77t + 136 \]  \quad 100 \leq t \leq 200 \text{ mm} \quad (9.5)

Where,
$F_u$ is the ultimate load in kN

$t$ is the thickness of the concrete slab in mm

### 9.5.2 Wu (2006) Experiments

Similar to Becher (2005), the finite element result was compared with experimental studies undertaken by Wu (2006). Figure 9.13 shows that the finite element result is in good correlation with the experimental study. The experimental results show a maximum value of 49 kN in the shear capacity for the push test, whilst the finite element result was 51 kN. The discrepancy is 4.1%. This confirms that the finite element model was accurately modelled.

A parametric study was undertaken where the variable parameter was the slab thickness. The slab thicknesses used were 120, 150 and 200 mm. The stiffness was different for shear loading and combined axial and shear loading. The push test specimens subjected to shear loading seems to have a higher stiffness when compared with those under both axial and shear loading. Figure 9.14 gives the ultimate capacity of headed stud shear connectors according to the reinforcement scheme.

For shear loading, the 200 mm slab had a higher shear stud strength capacity than the 120 mm slab. The shear capacities were 51, 59 and 63 kN for slabs with a 120, 150 and 200 mm in thickness, respectively. For the combination of shear and axial loading, the shear capacities were 31 kN and 40 kN for 120 mm and 150 mm, respectively. Unfortunately, due to axial tensile loading, the 200 mm thick profiled slab failed at 32 kN during the analysis. This proves that when subjected to a
combination of shear and axial tensile loading, the profiled slab with a thickness of more than 200 mm is not safe. The reason is because the thicker slab will cause the concrete to crack more when axial force is subjected. Hence, a possible improvement to this problem is to modify the reinforcement to cage reinforcement.

When cage reinforcement embedded into the concrete element, the shear stud capacity increased to 62, 72 and 76 kN for concrete slab of 120, 150 and 200 mm, respectively for shear loading. From Figure 9.14, it can be observed that the failure mode transformed from concrete cracking failure to stud failure. This proved otherwise for combination of axial and shear loading. Even though the cage reinforcement scheme was implemented, the concrete cracking failure still applied, but with higher capacity strength. From the finite element models, it indicates that less cracking occurred when compared with confined reinforcement scheme.

Equations 9.6, 9.7, 9.8 and 9.9 determine the ultimate load with respect to concrete thickness of the push tests for profiled cage reinforcement with shear loading, profiled confined reinforcement with shear loading, profiled cage reinforcement with combination of axial and shear loading and profiled confined reinforcement with combination axial and shear loading, respectively, and they are illustrated in Figure 9.14. It shows that the ultimate load increases by about 21 % for confined reinforcement scheme compared with cage reinforcement scheme under shear loading conditions. For a combination of axial and shear loading, the slab thickness of 120 mm showed an increment of 52 % in the ultimate load, whilst for slab thickness of 150 mm and 200 mm show only 32 % of increment. It can be
explained that cage reinforcement plays an important role to prevent concrete from cracking.

\[
F_u = -0.003t^2 + 1.2t - 36.49 \quad 100 \leq t \leq 200 \text{ mm} \quad (9.6)
\]

\[
F_u = -0.002t^2 + 0.88t - 22.19 \quad 100 \leq t \leq 200 \text{ mm} \quad (9.7)
\]

\[
F_u = -0.003t^2 + 0.85t - 19.85 \quad 100 \leq t \leq 200 \text{ mm} \quad (9.8)
\]

\[
F_u = -0.005t^2 + 1.66t - 95.48 \quad 100 \leq t \leq 200 \text{ mm} \quad (9.9)
\]

where,

- \(F_u\) is the ultimate load in kN
- \(t\) is the thickness of the concrete slab in mm

### 9.5.3 Parametric Studies for Interaction Diagram

A parametric study was carried out by varying the push-test slab thickness from 100 to 200 mm. The reason the author carried the parametric study is to create an interaction diagram which corresponding to the axial versus shear loading for different slab thicknesses. The spacing of stud shear connectors and the size remain the same for all cases.

Figure 9.15 illustrates the interaction diagram for Becher’s solid slab according to slab thickness. From Figure 9.15, it can be observed that the thicker the slab, the higher shear stud capacity. Bear in mind that the author used cage reinforcement scheme for the interaction diagram in order to prevent concrete cracking when the push test is subjected to axial tension load. It also can be perceived that the headed
stud shear capacity increase approximately 3.1\% and 6.6\% for 150 mm and 200 mm thickness, respectively when compared with 120 mm slab.

Figure 9.16 demonstrates the interaction diagram for Wu’s profiled slab according to slab thickness. From Figure 9.16, it can be noticed that the stud shear capacity only increased 29\% from 120 mm slab to 150 mm slab. When 200 mm thick slab was analysed, the studs shear capacity was reduced. The shear stud capacity reduced 12\% when compared with 150 mm slab. The main reason is that the 200 mm thick slab cracks easily when axial load was subjected to the push tests specimens. Finite element stress contours in Figure 9.17 illustrates that the thinner slab starts to crack when axial force is loaded. Therefore, it is not advisable to use a profiled slab of more than 200 mm thick.

9.6 DESIGN MODELS AND RECOMMENDATIONS

Parametric study push test specimens with different loading conditions for both solid and profiled slabs were performed using the established finite element models. The comparison of shear connector capacities obtained from the finite element models proved that not only do they depend on the type of profiled steel sheeting but also depend significantly on the loading conditions, reinforcement layout scheme and thickness of the slabs.

The introduction of the Equations 9.2 to 9.6 is a key characteristic of the proposed method because it accounts for the behaviour of the shear connection for composite steel-concrete structures. The relationship of ultimate shear capacity according to
slab thickness is useful for design engineers to estimate the behaviour of shear connectors in practice for design purposes.

The findings of interactions diagram in Figures 9.15 and 9.16 give an indication of the behaviour and strength of headed stud shear connectors under variety of loading conditions. The results provide reasonable estimates of the capacity of headed stud shear connectors in tension, shear and combined tension and shear loading.

9.7 SUMMARY OF CHAPTER

Shear connection nonlinearity always results in significant changes in the strength and ductility of composite steel-concrete beams. A modified push-test specimen used to implement the axial tensile load in the concrete element has been presented to account for both the shear connection strength and ductility. A comparison with experimental results allows the validation of such a procedure with reference to both solid slabs and profiled slabs for both push tests and beam tests.

The finite element analysis that has been undertaken in this chapter demonstrates that the axial tensile load has a pronounced impact on the shear behaviour where it greatly reduces the strength and deformation capacity of the studs. However, this problem can be solved by changing the reinforcement scheme from confined to cage where full strength of the headed stud connectors can be attained.
When solid slabs are taken into consideration, the finite element analysis proves that the thicker the slab, the higher the shear capacity is shown. On the other hand, the axial tensile capacity is reduced with an increase in slab thickness. The main reason is that the thicker slab will encourage higher concrete cracking and concrete cone failure caused by the axial tensile loading. For profiled slab, the result is proved to be otherwise where the thicker slab shows a higher shear and axial tensile capacity. This is because the thicker slab reduces concrete cone failure.

For slabs of more than 150 mm thickness, a modification of reinforcement scheme is required to prevent concrete cone failure and headed stud connector pull-out failure. The results of combined shear and axial tensile loading show that the design interaction diagram gives reasonable predictions of the headed stud connector capacities for both solid and profiled slabs. The interaction diagram is very useful for design engineers when the headed shear connectors are subjected to various loading conditions.
Figure 9.1 Modified Push-Test Experimental Specimen for Saari et al. (2004)
* All reinforcements are 12mm diameter

Figure 9.2 Confined Steel Reinforcement Scheme for Saari et al. (2004)
Figure 9.3  Caged Steel Reinforcement Scheme, Saari et al. (2004)

* All reinforcements are 12mm diameter
Figure 9.4 Comparison of Force Slip Relationship of Saari et al. (2004) Experimental Data and Finite Element Models for Confined Reinforcement
Figure 9.5 Stress Distribution for Saari’s Confined Reinforcement Scheme Under Shear Loading
Figure 9.6 Stress Distribution for Saari’s Confined Reinforcement Scheme Under Shear and Tensile Axial Loading
Figure 9.7 Comparison of Force Slip Relationship of Saari et al. (2004) Experimental Data and Finite Element Models for Caged Reinforcement
Figure 9.8 Stress Distribution for Saari’s Caged Reinforcement Under Shear Loading
Figure 9.9 Stress Distribution for Saari’s Caged Reinforcement Scheme Under Shear and Tensile Axial Loading
Figure 9.10 Comparison of Interaction Diagram for Combined Axial Tension and Shear Loading for Both Solid Slab and Profiled Slab
Figure 9.11 Comparison of Force Slip Relationship of Becher (2005) Experimental Data and Finite Element Models
Figure 9.12 Comparison of Ultimate Load According to Slab Thickness with Different Reinforcement Scheme for Becher’s Solid Slab
Figure 9.13 Comparison of Force Slip Relationship of Wu (2006) Experimental Data and Finite Element Models
Figure 9.14 Comparison of Ultimate Load According to Slab Thickness with Different Reinforcement Scheme for Wu’s Profiled Slab
Figure 9.15 Interaction Diagram for Combine Axial Tension and Shear Loading for Solid Slab According to Slab Thickness
Figure 9.16 Interaction Diagram for Combine Axial Tension and Shear Loading for Profiled Slab According to Slab Thickness
Concrete tensile stress reached and caused concrete to crack

Figure 9.17 Stress Distribution of Wu’s Profiled Slab at 200mm Thick
10.1 SUMMARY

The primary objective of this thesis was to study the behaviour of headed stud shear connectors on composite steel-concrete structures under various conditions. The headed stud shear connectors under mechanical behaviour as well as various loading conditions were evaluated in order to better understand the performance and behaviour of headed stud shear connectors. This research was divided into two major parts:

Part I. Material behaviour.

Part I emphasises the issues concerning the effects of steel fibres, elevated temperature and long-term effects on the behaviour of headed stud shear connectors for composite steel-concrete structures which are mainly concerned with material behaviour. These are discussed in Chapter 5, 6 and 7.

Part II. Various loading conditions.

Part II highlights the problems relating to the effects of strain regimes and the combination of axial tension and shear loading on the behaviour of headed stud shear connectors for composite steel-concrete structures which essentially effect the loads
imposed on the headed stud shear connectors. These are discussed in Chapter 8 and 9.

This thesis used specimens based on experimental investigations undertaken by various researchers, in order to calibrate with the finite element model using ABAQUS, Karlsson and Sonrensen (2006a), Karlsson and Sonrensen (2006b), Karlsson and Sonrensen (2006c) and Karlsson and Sonrensen (2006d). Two types of slabs were studied for comparison with the finite element model. They included solid slabs and profiled slabs.

Two types of experimental investigations were studied where steel fibres were considered. They include Becher (2005) for the solid slabs and Wu (2006) for the profiled slabs. When elevated temperatures were taken into account, there are three different experimental series results which were compared with the finite element analysis mentioned. The push tests performed by Lam and El-Lobody (2005) and El-Lobody and Young (2006) considered shear connection of a solid and profiled slab, respectively. The experimental investigation undertaken by Zhao and Aribert (1992) at elevated temperatures with different loading conditions were also compared with the finite element analysis.

Experimental investigations undertaken by Lam and El-Lobody (2005) for solid slabs were used and compared with the finite element analysis when time-dependent behaviour and variation in strain regimes were taken into consideration. The author modified Lam’s solid slab to consider a profiled slab in order to evaluate the differences between the solid and profiled slabs. Furthermore, the author also used
the experiments conducted by Hicks (2007) for both push and beam tests as a comparison against the finite element analysis for profiled slabs when strain regimes were taken into consideration.

Finally when combined axial tension and shear loading were analysed, the author used experimental studies undertaken by Saari et al. (2004) to compare with the finite elements results. At the same time, the author modified experimental studies carried out by Becher (2005) and Wu (2006) for the purpose of consideration of both solid and profiled slabs.

10.2 CONCLUDING REMARKS

Analysis results derived from the finite element model were compared with the existing experimental results reported herein. The accurate comparisons with the full behaviour of various configurations demonstrated the capability and robustness of the model. The various modes of failure were also traced satisfactorily, highlighting in particular the reliability of behaviour and strength of the headed stud shear capacity adopted in the model. The shear connector load-slip characteristics obtained from the push-out finite element models were found to be conservative and were able to predict the slip response in the beams with sufficient accuracy.

Using the abovementioned models, a set of logical parametric studies which examined the influence of a diverse range of parameters was undertaken. The results have been thoroughly appraised in the thesis. The slab thickness, types of profiled sheeting, concrete strength properties and reinforcement layout scheme were
established to be among the most influential parameters. The predicted models which define the characteristics strength, stiffness and ductility of the headed stud shear connectors were also thoroughly reviewed, and their suitability to be incorporated in design practice was discussed.

Generally, there are three differences between the profiled steel sheeting and solid slab. One of them is the failure mode. For the solid slab, the failure mode is shear connection failure whereas for the profiled slab, failure is dominated by concrete failure. Stresses in the shear connector and concrete are lower compared with those in the solid slab, due to the addition of the steel profile. It also can be observed that the solid slab generally has a higher ultimate load compared with that of the slab with profiled steel sheeting.

The steel and concrete interface plays a significant role in the apparent strength of the headed stud shear connectors. If profiled steel sheeting is placed between the steel and concrete, the headed stud shear connectors strength is about 42% of the strength of studs in a solid slab specimen without profiled steel sheeting. This decrease in strength is probably due to the reduction of friction at the steel and concrete interface.

Composite steel-concrete beams with the inclusion of steel fibres showed an improvement in cracking load. Studies considering continuous beams in hogging moment regions with the inclusion of steel fibres to look at the cracking behaviour and ductility will be subjected to further research in this project. Moreover, the
reduction factor for the profiled slab requires augmentation for all the international standards.

When the Australian, British and American Standards are compared, both experimental and finite element results seem to be in good agreement with the Australian and British Standards, however the American Standard seems to be less conservative for solid slabs. Consequently, for profiled slabs, this issue should be considered carefully by the designer.

The positioning of the shear connectors in composite beams has been discussed in this thesis. It was concluded that both the strong and middle positions have similar stiffness, whilst the stiffness for the weak position of shear connectors is roughly 10% lower. The ultimate loads for the shear connectors reduced by 22% and 11% for both the weak and middle positions, respectively.

When steel fibres of $F=0.3$ were added to the specimens, the ultimate load was reduced by 15% and 8%. When compared with concrete without steel fibres, the load reduction value is less significant. On the other hand, when steel fibres with a factor of $F=0.6$ were added to the specimens, the ultimate strength showed reductions of 18% and 8% for both the weak and middle positions, respectively.

As a conclusion, steel fibres are introduced to provide desirable characteristics such as increased strength and ductility in composite steel and concrete beams, but there is an optimum value for the steel fibre dosage included in the concrete. Figures
5.19 and 5.20 could clarify the optimum concrete strength property used to acquire the ultimate shear capacity when an optimum value of steel fibres was used.

When long-term effects are taken into account, the reduction in stiffness and ductility for both the solid and profiled slabs was observed in the finite element analysis. It is concluded that creep and shrinkage caused by slip of the shear connectors was noticeable in the first 400 days. After that, they did not have a major influence on the mechanical behaviour of composite steel and concrete structures.

Composite steel-concrete beams with long-term effects taken into account show a reduction in stiffness, shear and slip capacity. The purpose of obtaining Figure 6.12 is for designers to determine the slip capacity according to time. Moreover, Figures 6.13-6.16 are also valuable for designers to attain the ultimate load of headed stud shear connectors capacity according to concrete strength properties for both solid and profiled slabs.

As established, the ultimate load of a solid slab is higher than that of a profiled slab, but Figures 7.14 and 7.18 have shown that the profiled steel sheeting acted as a protective layer for the concrete slab. The profiled steel sheet slabs can withstand a maximum of 60% of their ultimate load at ambient temperature compared with only 40% attained in the solid slab under fire conditions. This is useful for structural efficiency purposes. When designers use the ultimate limit state for strength, different load combination factors are used. Figures 7.12 and 7.16 are useful to estimate the fire exposure time before the structure fails.
The reason for employing different strain regimes in composite steel-concrete beams is to properly simulate the behaviour of shear connectors in composite beams where trapezoidal slabs are used. The pertinent results obtained from the finite element analysis were verified against independent experimental results and existing design standards. Based on the finite element analysis and experimental results, it is evident that the strength and the load-slip behaviour of composite steel-concrete beams are greatly influenced by the strain regimes exist in the concrete element.

The introduction of parameters, such as $\alpha$, $Q_U$, $c_1$ and $c_2$ for G1, G2 and G3 profiled slab, can be obtained from Figures 8.12-8.14, 8.16-8.18 and 8.20-8.22, respectively. These are useful for design engineers using these profiles in industry. Furthermore, Figure 8.25 is helpful for design practitioners to estimate the ultimate load according to the given concrete strength properties.

The findings of the combination of axial tension and shear loading provide an indication that both the existing experimental studies and finite element models give reasonable estimates of the capacity of the headed stud connectors. Shear connection nonlinearity always results in significant changes in the strength and ductility of composite steel-concrete beams. Figures 9.15 and 9.16 show the interaction diagram of axial tension load versus shear load for solid and profiled slabs, respectively. In these figures, the thickness of slab ranges from 100 to 200 mm thick. The interaction diagram is practical for design engineers when the headed steel anchors are subjected to various loading conditions.
It should be borne in mind that the designs generated are only valid for the use of the solid slab profiled steel sheeting mentioned herein. Important assumptions are made in deriving this data regarding product tolerances, materials and components used, that may not be valid if other products are used. However, the approaches in this thesis may be of use for further studies.

10.3 RECOMMENDATIONS FOR FURTHER RESEARCH

Based on the research presented in this thesis, the following points highlight several pertinent issues which may be useful for further investigation. They offer the possibilities to extend and improve on the work carried out in the present study.

1. Composite steel-concrete beams with the inclusion of steel fibres show the improvement in the cracking load. Studies considering continuous beams in hogging moment regions with the inclusion of steel fibres to look at the cracking behaviour and ductility will be subjected to further research in this project.

2. In this study, the use of three different mixtures of fibre volume, $F=0$, $F=0.3$ and $F=0.6$ have been examined. Studies considering other sizes, types and quantity of fibres could be useful for determining the most cost effective product that does not sacrifice the strength capacity of headed shear studs.
3. A study analysing the effect of fibres on the mechanical bonding between the concrete and profiled steel sheeting could be beneficial and advantageous for such products.

4. Experimental studies considering the long-term effects of composite steel-concrete beams with steel fibres on the stiffness and ductility, is the subject of further research in this ongoing project.

5. Further studies based on various types of concrete mix design parameters, variation in effective thickness of members and various humidity should be considered for the long-term effects on composite steel-concrete structures.

6. For the elevated temperature, further experimental study is considered necessary in order to understand in detail the fire resistance for composite steel-concrete structures. Furthermore, the inclusion of steel fibres is also recommended when structures are subjected to elevated temperatures.

7. Concrete elements in push tests undergo a phase change once they reach 100°C. The effect of this occurrence on temperature distributions is not captured in the model proposed in the thesis herein. If a phase change were incorporated, the model would better simulate the fire performance of the concrete element.

8. Further experimental research is considered necessary in order to validate the results to enhance an understanding of modified strain regimes.
9. For the combination of axial tension and shear loading, in the present research, the author only considered monotonic loading. It is beneficial to input cyclic loading behaviour for further studies.

10. It is recognised that standard push-out tests are conservative to represent the slip deformation behaviour of shear connectors in composite steel-concrete beams. Although considerable work has been conducted to investigate the shear connector behaviour via push-out tests, very limited work has been carried out to study the localised behaviour of connectors in a concrete medium which is susceptible to cracking. It may be of interest to evaluate these effects. In addition, finite element studies to investigate the behaviour of shear connectors may also be performed.
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