BEHAVIOUR OF C-SHAPED ANGLE SHEAR CONNECTORS IN HIGH STRENGTH CONCRETE

ALI SHARIATI

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This thesis is kindly devoted to my adorable brother MOJTABA. Much obliged to you for your sponsorship and support without which this couldn't be conceivable. I adore you for all that you are and everything that you have accomplished for me.

ABSTRACT

The most commonly used types of shear connectors in steel-concrete composite structures are headed shear stud and Perfobond shear connectors. Due to the limitations in the performance of headed studs and Perfobond shear connectors, the use of C-shaped shear connectors as an alternative has been recommended. Furthermore, manufacturing of the C-shaped shear connectors is easier compared to the other connectors since in most steel shops, commercial standard sizes for hot rolled steel profiles of C-shaped shear connectors are available. Moreover, by simply cutting in their long steel profiles, these types of connectors can be easily prepared and the manufacturing cost and time for making C-shaped connectors are significantly low. C-shaped connectors show higher shear capacity and with the use of the conventional reliable welding system, it could be welded easily to steel beam.

The angle connectors, as compared to channel connectors, could be cheaper and more economical due to the absence of bottom flange which ultimately saves more steel. The results of investigation of behavior of C-shaped angle shear connector is not included in previous research yet. In this thesis, the results of twenty four experimental push-out tests on angle shear connectors are presented. The connectors were embedded in a solid slab with High Strength Concrete (HSC) and were tested under monotonic and low cyclic fatigue loading. Based on the results, angle shear connectors showed good behavior in terms of shear strength but not in ductility as the main criterion for the behavior of shear connectors. Connector fracture types of failure was seen and less sensitivity to flange thickness and height of connectors was reported for angle connectors in terms of shear strength. Longer connectors exhibit more ductility compared to shorter connectors. The details of angle shear connector behavior have been discussed in detail as well.

ABSTRAK

Jenis-jenis penyambung ricih yang biasa digunakan dalam keluli-konkrit struktur komposit yang diketuai stud ricih dan Perfobond penyambung ricih. Oleh kerana keterbatasan dalam pelaksanaan diketuai kancing dan Perfobond penyambung ricih, penggunaan penyambung ricih berbentuk C sebagai alternatif telah disyorkan. Tambahan pula, peughasilan penyambung ricih berbentuk C adalah lebih mudah berbanding penyambung lain sejak di kedai-kedai yang paling keluli, saiz standard komersial untuk profil keluli tergelek panas penyambung ricih berbentuk C boleh didapati. Selain itu, dengan hanya memotong dalam profil keluli panjang mereka, ini jenis penyambung boleh dengan mudah disediakan dan kos pengeluaran dan masa untuk membuat penyambung C-berbentuk ketara rendah. Penyambung C berbentuk menunjukkan kapasiti yang lebih tinggi dan dengan menggunakan sistem kimpalan dipercayai konvensional, ia boleh dikimpal mudah kepada rasuk keluli.

Penyambung sudut, berbanding dengan penyambung saluran, boleh menjadi lebih murah dan lebih menjimatkan kerana ketiadaan bahagian bawah bebibir yang akhirnya menjimatkan keluli lebih. Hasil siasatan ke atas tingkah laku C berbentuk sudut penyambung ricih tidak termasuk dalam penyelidikan sebelumnya. Dalam tesis ini, hasil daripada dua puluh empat eksperimen menolak keluar ujian ke atas penyambung ricih sudut dibentangkan. Penyambung telah tertanam dalam papak yang kukuh dengan kekuatan konkrit Tinggi (HSC) dan telah diuji di bawah ekanada dan rendah kitaran keletihan muatan. Berdasarkan keputusan, penyambung ricih sudut menunjukkan tingkah laku yang baik dari segi kapasiti ricih tetapi tidak dalam kemuluran sebagai kriteria utama untuk tingkah laku penyambung ricih. Jenis patah penyambung kegagalan dilihat dan sensitiviti yang kurang untuk ketebalan bebibir dan ketinggian penyambung dilaporkan untuk penyambung sudut dari segi keupayaan ricih. Penyambung lagi mempamerkan lebih kemuluran berbanding penyambung pendek.

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Ali Shariati

University of Malaya

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LISTS OF ABBREVIATIONS

Q_n	Shear strength of one channel shear connector (N)
t_f	Flange thickness of channel shear connector (mm)
t_w	Web thickness of channel shear connector (mm)
L_c	Length of channel shear connector (mm)
f'_c	Specified 28 days cylinder strength of concrete (MPa)
E_c	Modulus of elasticity of concrete (MPa)
f_c	Specified compressive strength of concrete (MPa)
Q_u	Nominal strength of one channel shear connector (N)
w^2	Web thickness of the channel (mm)
L	Length of the channel (mm)
H	Height of the channel
$\frac{w_d}{h_d}$	The $\frac{w_d}{h_d}$ is the width to depth ratio of the rib of the metal deck
P	Load carrying capacity of the connector (kgf)
V_u	Ultimate shear force in the shear failure mode or by concrete crush (N)
h	Height of shear connector (mm)
K_i	i: Segment member
S_0	Initial scale factor
k_{dflt}	Default stiffness
r	Over closure factor
L_{elem}	Element lengt
d	Over closure measure
S	Geometric scale factor
P	Shear strength of the connector (kgf)
t_w	Web thickness of connector (cm)
L_c	Length of connector (cm)
f_c	Concrete compressive strength (kgf/cm^2)

LIST OF APPENDIXES

1 INTRODUCTION

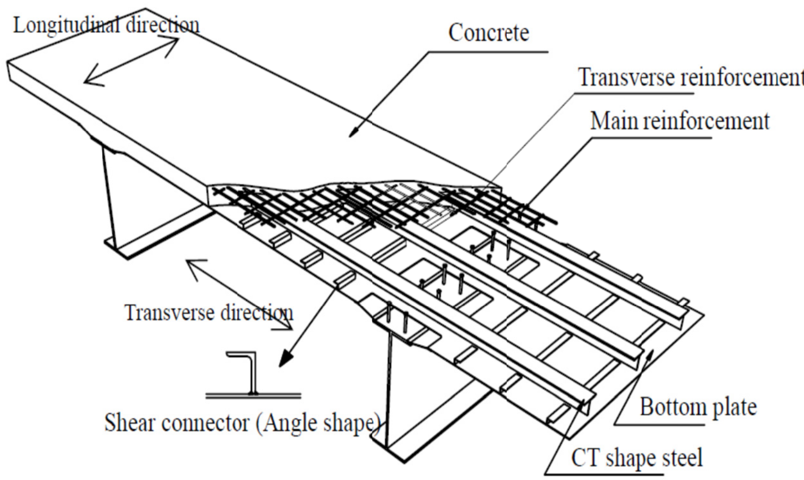
1.1. Preface

Shear connectors between concrete slabs and steel beams in composite construction can play an important role in the seismic response of a structure. They provide the necessary shear connection for composite action in flexure, and can be used to distribute the large horizontal inertial forces in the slab to the main lateral load resisting elements of the structure as shown in Figure 1.1.

Connecting the existing concrete slab and steel girders is a potential economic way to strengthen these floor systems as it allows for composite action to be developed. As opposed to the original non-composite condition, composite action allows the existing steel girder and the concrete slab to act together more efficiently. In non-composite girders, the steel girders and the concrete slab act separately in flexure. Hence, by using shear connectors, girders can be increased by more than 50% as compared to that of non-composite girders (Peiris & Harik, 2011). By connecting the steel girder to the concrete slab, it allows the transmission of shear forces at the steel concrete interface, thus, enabling the benefits of composite actions to be achieved. Prior to casting of the concrete slab, shear connectors are welded to the top of the steel girder in order to develop composite action in the construction of new bridges (Kwon et al., 2009).

Figure 1.1. Shear connectors in a composite beam

Headed stud and Perfobond shear connectors are the most common types of shear connectors while the application of C-shaped shear connectors (Figure 1.2) is increasing because of some limitations for the use of headed studs and Perfobond shear connectors.



a) C-shaped angle shear connector

b) C-shaped channel shear connector

Fig. 1.2 Typical C-shaped channel and angle shear connectors

To achieve the behavior of angle shear connectors in composite beam with high strength concrete slab, primary experimental push-out test on these connectors have been conducted. The results of these tests in case of shear capacity and ductility were compared and discussed.

1.2. Problem statements

C-shaped channel and angle shear connectors were used in previous research as reviewed in literature review when they were embedded in different kinds of concrete. The application of angle shear connectors in High Strength Concrete (HSC) is not well studied up to date. Consequently, the behavior of these shear connectors in high strength concrete is of interest.

The objective of this thesis was aimed at better understanding of the behavior of angle shear connectors in HSC and fulfil the recommendation of previous researchers. Evaluation of the reliability of the existing equations in predicting the shear capacity of angle shear connectors in high strength concrete is another objective of this study.

1.3. Objectives of the study

The objective of the study is listed as below:

1. To assess the behavior of C-shaped angle shear connectors embedded in high strength concrete slab under monotonic and fully reversed cyclic loading.
2. To investigate several parameters affecting the behavior of C-shaped angle shear connectors such as connector's geometric properties and concrete strength.
3. To evaluate the reliability of the existing equations in predicting the shear capacity of the C-shaped angle shear connectors in high strength concrete.

1.4. Scope of work

The scope of this study is to improve the design of C-shaped angle shear connectors in composite beam. Available guidelines are limited to the design of the isolated angle shear connectors. No equation is currently available for the design of angle shear connectors embedded in high strength concrete slab. There is a definite need to develop new formulations for the design of angle shear connectors in slabs with high strength concrete. The results of monotonic push-out tests are used to calibrate a proposed nonlinear finite element model were used to evaluate the accuracy of available equations for the prediction of angle shear connectors in HSC.

There are only a few references available to estimate the shear capacity of angle shear connectors in reinforced concrete slab. Based on the results of this thesis, the existing design equations seem to be inaccurate. The evaluation of these equations was the scope of this thesis as well. Future research on angle shear connectors were proposed as well.

1.5. Outline of the thesis

In this thesis, a literature review on C-shaped angle shear connectors are discussed in Chapter Two. After the literature review of existing studies on C-shaped shear connectors, the objective of this study was proposed.

Chapter Three describes the design and the method of experimental push-out test and parametric finite element analysis of push-out tests on angle shear connectors. The test setup, experimental test specimens and procedures of Finite Element Analysis (FEA) are presented in this chapter. Twenty four push-out test specimens were selected for experimental testing. The finite element analysis was employed to show the behavior of the experimental push-out tests, for obtaining more details on different size of the connectors and different types of concrete. In addition, the FEA result can be used for a wider investigation of the connectors' behavior for the future works after verification with the test results.

In Chapter Four, the experimental test and finite element results in the behavior of the angle shear connectors are presented and discussed. The behavior of shear connectors in case of shear capacity under monotonic loading, strength degradation under low cyclic fatigue loading and their relative ductility was discussed.

Chapter Five summarizes the findings of the study and suggests a few recommendations for future works.

2 LITERATURE REVIEW

2.1. Preface

In this chapter, an attempt has been made to review different types of shear connectors including the commonly used one and their main alternatives means C-shaped shear connector used in composite structures. This review tries to identify the C-shaped shear connectors that are the most relevant to composite structures and reviews their representative journal publications that are related to this topic. Comparative studies, which have been conducted by several researchers, were covered to address the applicability and the efficiency of C-shaped shear connectors. The representative shear connectors for headed stud connectors which are commonly used as shear connectors in composite structures were discussed and a summary of their behavior was included.

2.2. Headed Studs

To resist horizontal shear and vertical uplift forces in composite steel-concrete structures, the most commonly used types of shear connector is the head stud, known as the Nelson stud (Figure 2.1). This type of connector contributes to the shear transfer and prevents uplift. It is designed to work with an arc welding electrode, and, simultaneously, after the welding, it then acts as the resisting connector with a suitable head.

Figure 2.1. Headed stud shear connector

Many researches have been carried out on headed stud connectors and various equations have been proposed to estimate the strength of headed studs (Gelfi and Marini, 2002; Lee et al., 2005; Ollgaard et al., 1971; Viest, 1956a). Viest (1956a) carried out the initial studies on headed stud shear connectors, where full-scale push out specimens were tested with various sizes and spacing of the headed studs. The push-out and composite beam tests were used to evaluate the shear capacity of the headed stud. In order to investigate the behavior of headed stud connectors in solid slabs, an accurate nonlinear finite element model was developed by Ellobody (2002), and Lam and Ellobody (2005).

Validations against test results and comparison with data specified in the current Codes of Practice, such as BS5950 (1994), and AISC (2005), was carried out using the effective numerical model by Lam and Lobody (2001). The results of the experiment conducted by these authors are comparable with the results obtained from the finite element analysis. The finite element model offered accurate predictions on the capacity of the shear connection, the load slip behavior of the headed studs and the failure modes.

Ellobody (2002) conducted another finite element model by considering the linear and non-linear behavior of the materials in order to simulate the structural behavior of headed stud connectors. The use of the model in examining variations in concrete strength and headed stud diameter in parametric studies was also presented. Consequently, it was found that the finite element results suggested by BS5950 (1994) may overestimate the headed stud's shear capacity.

The experimental tests to assess the behavior of the shear connection between the steel and lightweight concrete that were carried out at the University of Minho were described in another work by Valente and Cruz (2004). The behavior of headed stud connectors embedded in ECC (Engineered Cementitious Composites) was investigated by Li et al. (2006), while, in order to examine the capacity of large headed stud shear connectors embedded in a solid slab, an accurate nonlinear finite element model of the push-out specimen was performed by Nguyen and Kim (2009).

The AISC (2005), CSA (2001) and Eurocode (2004) standards currently provide design equations for the calculation of the resistance of a headed stud shear connector. The investigation of the headed stud's capacity has been conducted thoroughly and tabulated values can be found in BS 5950: Part 3 (1990) and BS 5400 : Part 5 (1983) as well.

The headed stud's root is functioned to transmit the horizontal shear force acting at the steel-concrete interface, while the head is provided for preventing uplift of the slab. The cross-sectional area of a headed stud connector is directly proportional to its shear strength and its ultimate shear strength is influenced largely by the concrete's compressive strength and modulus of elasticity.

2.3. Perfobond Ribs

In the late 1980s, Leonhardt et al. (1987) developed a new type of connector called the Perfobond rib (1987). This connector was introduced in recognition of the unsatisfactory behavior of headed studs in which resulting from fatigue problems caused by live loads on composite bridges. Developed in Germany, this connector consists of a welded steel plate, with a number of holes (Figure 2.2) (Lee et al., 2010). The flow of concrete through the rib holes formed dowels that provided resistance in both the vertical and horizontal directions. This shear connector is a viable alternative to the headed stud connector, as signified in the experimental studies conducted previously (Ahn, et al., 2010) and recently (Jumaat et al., 2011; Kisa et al., 2011), This connector was initially used in building structures (Ferreira et al., 1998). The fact that not only it ensures the concrete steel bond but also enabled a better anchorage of the internal columns hogging moment has encouraged its adoption. By passing through the

Perfobond web holes or simply by being superimposed to the transverse reinforcing bars would allow these bars to be anchored. A study, which had been done by Zellner (1987) indicated that a one meter length of Perfobond connector was comparable to eighteen 22 mm diameter's headed studs which were disposed in two lines or twenty four 19 mm diameter's headed studs which were disposed in three lines.

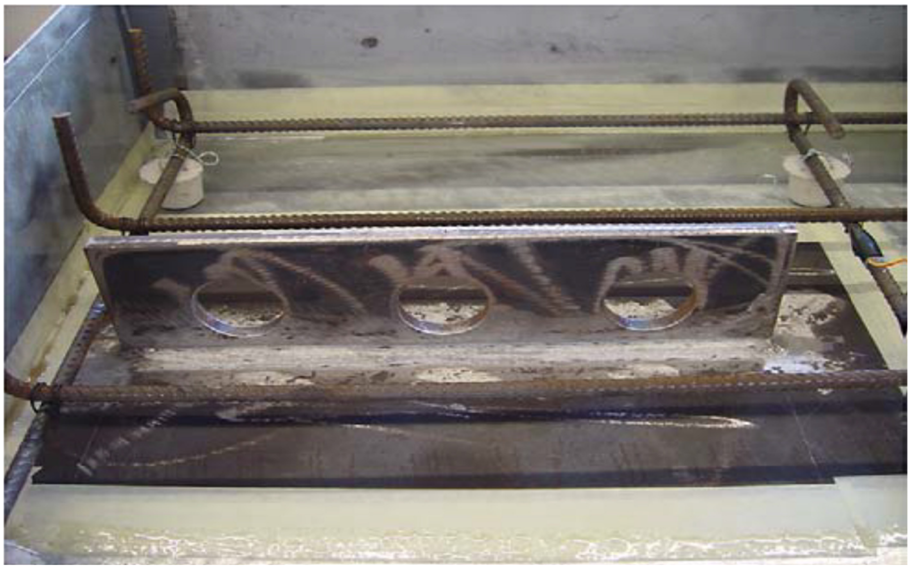


Figure 2.2. Perfobond Ribs shear connector

2.4. T-rib connector

In the scope of a study on Perfobond connectors, Vianna et al. (2009) presented an alternative connector for headed studs, called the T-Perfobond (Figure 2.3). The researcher also provided a comparative study between the behavior of these connectors and a limited number of T-Perfobond connectors. By adding a flange to the plate, which acted as a block, the derivation of this connector from the Perfobond connector was created. The need to combine the large strength of a block type connector with some ductility and uplift resistance arising from the holes in the Perfobond connector web was a motivating factor for the development of this T-Perfobond connector.

In order to prevent a premature loss of stiffness in the connection, the T-rib connector detail should minimize the prying action effect (Ferreira, 2000). As leftover rolled sections can be used to produce the T-rib connectors, it can reduce cost and minimize welding work. The four steps involved in the fabrication process of the T-rib connectors are: (i) initial profile, (ii) web holes, (iii) flange holes, (iv) opposite flange saw cut are as shown in Figure 2.3.

For similar longitudinal plate geometries, the resistance and stiffness of T-Perfobond connectors are higher than that of Perfobond connectors. In addition to this advantage, the use of T-Perfobond connectors offers benefits in terms of saving material and labor, as they are produced by ordinary laminated I or H sections.

Figure 2.3. T-Rib shear connector

2.5. T-Connectors

This connector is a section of a standard T-section welded to the H or I section with two fillet welds Figure 2.4. T-connectors evolved from the observation by Oguejiofor and Hosain (1997) they stated that a large part of the bearing capacity of a Perfobondstrip was the result of the direct bearing of the concrete at the front end of the (discontinuous) Perfobondstrip. Therefore, a T section, which has a larger cross section than a single strip, and its shape, was potentially able to prevent vertical separation between the steel-section and the concrete, seemed to be a good alternative.

The behavior of the T-connector is very favorable. The bearing stress on the front of the T is very high, which is a result of the relatively small area. Local concrete crushing occurs, which results in a quasi-plastic performance (Zingoni, 2001). The load capacity for T-connectors are similar to that of the oscillating Perfobondstrip, however, the ductility of these connectors is much larger (Rodera, 2008). When theses connectors are used in concrete with fibers, lightweight concrete or a higher strength concrete, there is a notable increase in the load capacity and ductility of this type of connector. In the case of the T-shape connectors, the strength of the connector itself is vital and the concrete is no longer decisive. Disregarding the Perfobondstrip, the resistance characteristic of the T-shape connectors is considered as the highest and its failure mode varies according to different concrete strengths.

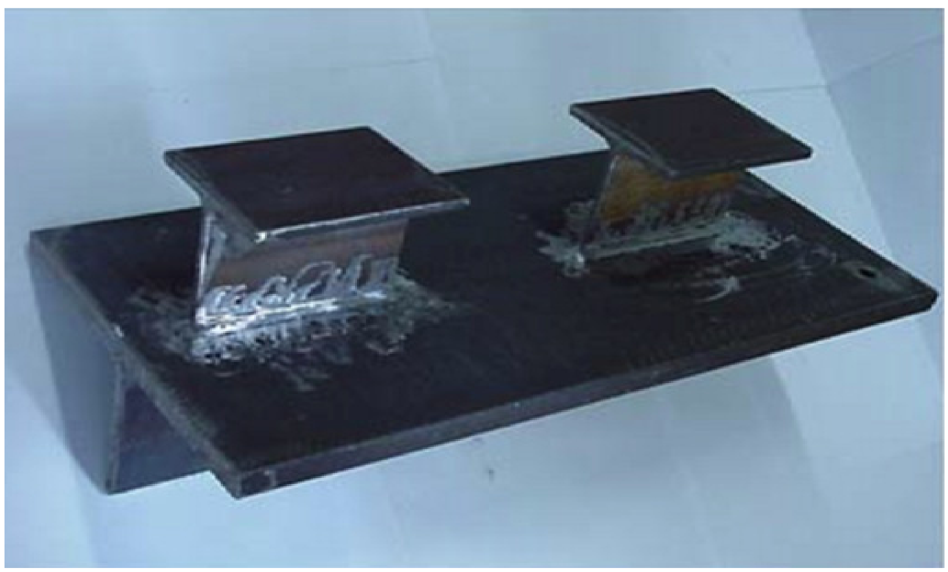


Figure 2.4. T shear connector

2.6. C-shaped shear connector

Due to the limitations for the use of headed studs and Perfobond shear connectors in composite construction, the use of C-shaped shear connectors may be a recommended alternative, especially in developing countries. Some restrictions on the fatigue behavior of headed studs have been reported such as the commencement of fatigue cracks in welds under cyclic loading, the necessity of specific welding equipment and high power generators on site (Chromiak et al., 2006). Also for the Perfobond shear connector, problems will appear when the steel bars need to cross the connector openings and it is difficult to position the slab for lower reinforcement (Ver´ıssimo et al., 2006).

In addition, the manufacturing of headed studs and Perfobond shear connectors is not as easy as C-shaped shear connectors due to the special shape of the headed studs and the need of making holes in Perfobond shear connectors, which is a time consuming and an expensive procedure.

There are commercially available standard sized hot rolled steel profiles of C-shaped shear connectors mostly in the steel industry. It is also easy to prepare these types of connectors by simply cutting their plain steel profiles. One may also notice that manufacturing cost and time to manufacture C-shaped connectors are significantly lower compared to the headed stud and Perfobond connectors.

Also, the C-shaped connectors have high load carrying capacity and can be welded to steel beam by using the conventional reliable welding system. Some inspections, like bending test which is needed for headed stud connectors, are not necessary for these types of shear connectors and also positioning the slab for bottom reinforcement, may not be a challenge when C-shaped shear connectors are employed (Ciutina & Stratan, 2008). Generally speaking, C-shaped shear connectors are preferred as they overcome the constraints and difficulties of using the headed studs and Perfobond shear connectors in composite beams.

The C-shaped shear connectors can be made with both angle and channel profiles as shown in Figures 2.5 to 2.7. The angular profiles can also include L-shaped shear connectors in addition to the C-shaped one as well. Since angle connectors in the absence of bottom flange in comparison to channels, they could be cheaper and more economical than channel connectors. A hoop reinforcement should be provided for L-shaped angle connector to prevent uplift of concrete in the composite system (Eurocode 4, 2004), a similar problem also occurs in the case of Perfobond connectors when the steel bars need to cross the connector openings. Therefore, it is better to use the C-shaped angle shear connector than the L-shaped connector in composite beams.

Figure 2.5. Typical angle shear connector

Figure 2.6. Typical channel shear connector

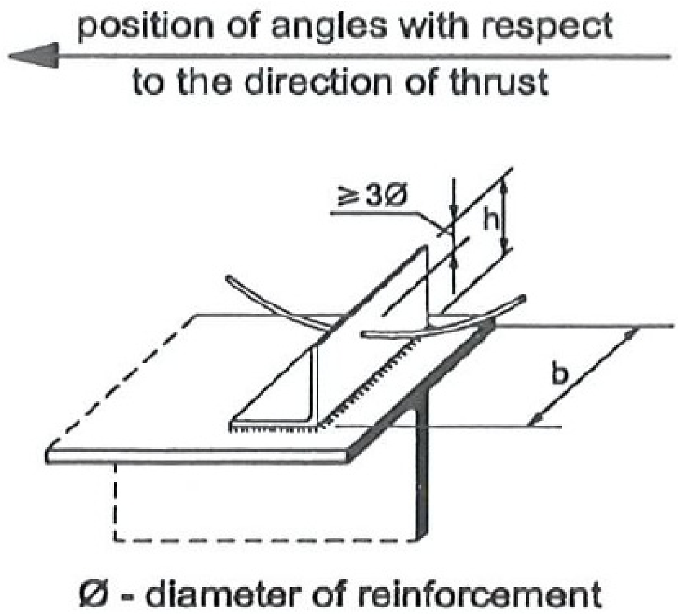


Figure 2.7. Typical L-shaped angle shear connector [Eurocode 4; CEN 2001]

Although channel shear connectors, as one of the popular C-shaped shear connectors, are used more in structures because of their accepted well-behaved performance. Angle shear connectors without bottom flange could be cheaper and more economical than channel shear connectors by saving more steel material in composite beams. The convenient welding process of angle connectors compared to channel connectors is a further advantage.

2.6.1. C-shaped channels

Channel shear connectors were used in the scale-model of composite bridges and initially tested at the University of Illinois by Viest et al. (1952). The test results of a preliminary study of channel shear connectors were presented by Slutter and Driscoll (1965) and Pashan (2006) to identify their behavior and assess the possibility of using this steel profile as a shear connector. From the above studies, some equations were derived to calculate the capacity of channel shear connectors in a solid concrete slab.

Those equations were adopted into building codes, such as the National Building Code of Canada (NBC) (2005) and the American Institute of Steel Construction (AISC) (2005).

In order to assess the accuracy of the design code equations for the strength of channel shear connectors, an experimental study using specimens with different channel sizes and lengths under monotonic loading was conducted by Pashan (2006). Several researchers presented the behavior of channel shear connectors embedded in a solid concrete material slab based on an experimental study conducted under monotonic and low-cycle fatigue loading and proposed an effective numerical model using the finite element method to simulate the push-out test of channel shear connectors as described in following.

A series of tests were carried out on push-out specimens made of plain concrete, reinforced concrete (RC), fiber reinforced concrete (FRC) and engineered cementitious composite (ECC) by Maleki and Bagheri (2008a, 2008b). Based on the results, the reversed cyclic shear strength of most specimens is lower than their monotonic strength by about 10% – 23%. The results also indicated that the shear strength and load-displacement behavior of the specimens was slightly affected by the use of the polypropylene fibers (FRC specimens). However, a considerable increase in the ultimate strength and ductility of channel shear connectors was achieved by the use of the polyvinyl alcohol fibers (ECC specimens) (Maleki & Bagheri, 2008a).

A validation against experimental test results and a comparison with the data given in North American design codes was carried out for additional research on the shear capacity of channel shear connectors embedded in a solid reinforced concrete slab under monotonic loading by using the finite element analysis. To investigate the variations in concrete strength, channel dimensions and the orientation of the channel, parametric studies by using nonlinear finite element analysis were performed. It was found that to determine the ultimate strength of channel shear connectors, the significant parameters included the strength of concrete, the web and flange thicknesses of the channel and the length of the channel, whereas the height of the channel section was regarded else. Moreover, a change in the stiffness and the ultimate strength of the shear connector could be caused by changing the orientation of the channel (Maleki & Bagheri, 2008b).

Other similar push-out tests have been performed by Shariati et al. (2011) for channel connectors after embedding in high strength concrete (HSC).

In other research, Maleki and Mahoutian (2009) investigated the shear strength of channel shear connectors embedded in normal and polypropylene concrete both experimentally and analytically. Before a prediction for the shear capacity of channel connectors in polypropylene concrete could be reached, an extensive parametric study was performed. An equation was suggested for the shear strength of these connectors when used in polypropylene concrete, which was included in design codes.

Generally, Viest et al. (1952), Pashan (2006), Ollgaard et al. (1971), Viest (1960) and Johnson (1970) reported on a literature review of composite beam research from 1920 to 1958 and 1960 to 1970 for headed stud and channel shear connectors that are embedded in normal concrete. Their results of the push-out test showed that the strength of the composite system could be affected by other factors apart from the concrete strength, which include the flange thickness, web thickness and channel length. Several equations for obtaining the channel shear connector capacity were proposed based on these investigations.

Years later, building codes adopted some of these equations. The current Canadian code (2005), for instance, suggests the use of the following equation for the calculation of the shear capacity of a channel shear connector embedded in a solid concrete slab.

In order to calculate the nominal strength, Q_n, for a channel shear connector embedded in a concrete slab, the following equation has been provided by the current American Standards (AISC) (2005).

$$Q_n = 0.3(t_f + 0.5t_w)\, L_c \sqrt{E_c f'_c} \qquad (2.1)$$

Where:

Q_n = Shear strength of one channel shear connector (N)

E_c = Modulus of elasticity of concrete (MPa).

t_f = Flange thickness of channel shear connector (mm)

t_w = Web thickness of channel shear connector (mm)

L_c = Length of channel shear connector (mm)

f'_c= Specified 28 days cylinder strength of concrete (MPa).

Meanwhile, the National Building Code (NBC) (2005) of Canada suggests the following equation which can be implemented for calculating the shear capacity of a channel shear connector embedded in a solid concrete slab,

$$Q_n = 36.5(t_f + 0.5t_w)L_c\sqrt{f_c} \qquad (2.2)$$

Where:

Q_n = Nominal strength of one channel shear connector (N)

t_f = Flange thickness of channel shear connector (mm)

t_w = Web thickness of channel shear connector (mm)

L_c = Length of channel shear connector (mm)

f'_c = Specified compressive strength of concrete (MPa)

In addition, modified equations for the prediction of the shear capacity of channel shear connectors embedded in polypropylene (PP) concrete were suggested by Maleki and Mahoutian (2009).

$$Q_n = 27.2(t_f + 0.5t_w)L_c\sqrt{f'_c} \qquad (2.3)$$

Pashan and Hosain (2009) used the test results of the eighteen push-out specimens with solid concrete slabs to perform a regression analysis for the development of a new equation. They believed that the original equation, which provided excellent correlation with the test results, was deemed to be too complex. The following simplified version of this equation was developed at the expense of some accuracy:

$$Q_u = (336w^2 + 5.24LH)\sqrt{f_c'} \tag{2.4}$$

Where:

Q_u = Nominal strength of one channel shear connector (N)

L = Length of the channel (mm)

w = Web thickness of the channel (mm)

f_c' = Specified compressive strength of concrete (MPa)

H = Height of the channel.

The next equation was proposed by them for the shear capacity of channel connectors in metal deck slabs as well.

$$Q_u = \left(1.7LH\frac{w_d}{h_d} + 275.4\,w^2)\sqrt{f_c'}\right) \tag{2.5}$$

Where:

The $\frac{w_d}{h_d}$ is the width to depth ratio of the rib of the metal deck

2.6.2. C-shaped angles

The primary results of the push out tests on specimens with several shear connectors including channel and L-shaped angle shear connectors were reported by Rao (1970). The results indicated that the C-shaped channel shear connectors provided considerable flexibility and showed greater load carrying capacity than other shear connectors.

In a research carried out by Ciutina and Stratan (2008), five different types of shear connectors which comprised of L-shaped angle shear connectors subjected to cyclic and monotonic loading, were investigated through a limited number of push-out tests. It was concluded that the cyclic loading makes 10% – 40% reductions in shear resistance for all connectors including L-shaped angle shear connector compared to the corresponding monotonic loading.

L-shaped angle shear connectors can be used in some other structural members too. For example, in extended connections, the L shaped angle connector can be applied as single or double angle bolted shear connections. A research conducted by Higgins (2005) discussed on the design of bolted extended double angle, single angle, and tee shear connections and covered the design of the extended connections by using these connectors.

Being a shear connector embedded in concrete foundation can be considered as another application of this connector. An equation for shear capacity of angle shear connector embedded in concrete foundation is suggested by ASCE (2000).

Also a few studies have been conducted on the behavior of C-shaped angle shear connectors. Kiyomiya and Yokota (1987) investigated the ultimate strength and deformation of various kinds of shear connectors, including C-shaped angles, channels, and T-shaped shear connectors, in composite members. It was concluded that the shapes and directions of shear connectors and concrete strength greatly affect the mode of failure of specimens in the push-out test.

In a research by Choi et al. (2008), the fatigue strength of a welded joint between C-shaped angle shear connectors and bottom plate in steel-concrete composite slabs was investigated through fatigue tests and finite element analysis. The results confirmed that the stress level at the welded joint was low and considerably less than the fatigue limit of this connector based on Eurocode 4 (Eurocode 4, 2004).

Another research, done by Fukazawa et al.(2002), undertook the wheel trucking test on the composite slab, applying C-shaped angle shear connectors, in order to explain their applicability to continuous composite steel girder bridges and their performance under moving load conditions. Their results showed that the composite slab has the sufficient fatigue durability and stiffness.

In a research carried out by Saidi et al. (2008), the relationship between transfers shear force and relative displacement on the C-shaped angle shear connectors and T-shaped shear connectors, utilized in steel-concrete sandwich beam, was studied and a numerical model was presented. In this model, the rotation at the edge of the shear connector and the horizontal movement of it were presumed to be the boundary condition of the angle and the T shaped shear connectors.

A new test method was developed by Ros and Shima (2009), in order to examine the shear load-slip relationship of C-shaped angle shear connectors. The conclusions of their study showed that the direction of the shear force of the shear connector influences the shear strength of the shear connector.

Empirical equations were developed by Kiyomiya and Yokota (1987) that predicted the load-carrying capacity of some shear connectors including of angle connectors.

$$P=65\sqrt{t_w}\,L_c\sqrt{f_c} \tag{2.6}$$

Where:

P = Load carrying capacity of the connector (kgf)

t_w = Web thickness of connector (cm)

L_c = Length of connector (cm)

f_c = Concrete compressive strength (kgf/cm^2)

Another equation was also proposed by Ros (2011) that can predict the ultimate shear capacity of angle shear connectors based on either the connector failure or concrete crush.

$$V_u = k \times \sqrt{f_c} \times L_c \times h \tag{2.7}$$

$$k = 63\,t_w/h + 160 \tag{2.8}$$

Where:

V_u = Ultimate shear force in the shear failure mode or by concrete crush (N),

L_c = Length of connector (mm),

h = Height of shear connector (mm),

t_w= Thickness of shear connector (mm),

f_c= Concrete compressive strength (MPa),

2.7. Concluding Remarks

C-shaped shear connectors is recommended, as an alternative, especially in developing countries. Because of some advantages. There are commercial standard sizes for hot rolled steel profiles of C-shaped shear connectors in most steel shops. It is also easy to prepare these types of connectors by simply cutting in their profiles. One may also notice that their manufacturing cost and time for C-shaped connectors are much lower compared to other common connectors. Generally speaking, C-shaped shear connectors are preferred as they overcome the restraints and difficulties of using common shear connectors like the headed studs and Profobond shear connectors in composite beams.

The C-shaped shear connectors can be made with both angle and channel. Since the use of C-shaped shear connector could be efficient in composite beams and the authors could not find any relevant investigation on the behavior of C-shaped angle shear connectors under fully reversed cyclic loading embedded in high strength concrete, current research was conducted to investigate the efficiency of the C-shaped angle shear connectors in the above mentioned conditions.

3 RESEARCH METHODOLOGY

3.1. General

The experimental push-out tests provide current knowledge of the load-slip behavior of the shear connectors in composite beams. Numerous researches on push-out tests, also called composite beam tests, have been conducted. Full-scaled experimental tests are widely known as very expensive and time-consuming option for investigation. Analytical methods to predict the nonlinear reaction and the ultimate shear capacity of the shear connectors in composite beams are definitely valuable alternatives. The results of the analytical methods should be validated against accurate experimental results.

The three-dimensional nonlinear behavior of push-out specimens and the interaction between steel shear connectors and concrete slab make the mathematical modelling of such specimens very complex. Some limited studies are available for modelling the push-out specimen on C-shaped shear connectors. On the other hand, the finite element method has become, in recent years, a powerful and useful tool for the analysis of a wide range of engineering problems. A comprehensive finite element model permits a considerable reduction in the number of experiments.

Nevertheless, in a complete investigation of any structural system, the experimental phase is essential. Taking into account that numerical models should be based on reliable test results, experimental and numerical/theoretical analyses complement each other in the investigation of a particular structural phenomenon. The methodology of this research has been divided into experimental and analytical studies. Push-out tests using specimens of angle connectors in high strength concrete under monotonic and cyclic loading completed the experimental investigations.

In analytical study, a three-dimensional finite element model using ABAQUS environment used to simulate the behaviour of angle shear connectors. The push-out test arrangement is modelled and all linearity and nonlinearity behavior of all the components is taken into consideration to establish the modes of failure, the ultimate strength, and the load-slip behavior of the shear connectors. The results of the finite element model are compared with selected push-out test results and the tabulated values given in the current equation of practice. Parametric studies using this model were carried out to investigate the variations in concrete strength and shear connector's dimensions.

3.2. Experimental test procedure

3.2.1. Introduction

The experimental program involved the testing of twenty four push-out specimens with angle shear connectors embedded in high strength concrete with different geometric properties under monotonic and cyclic loadings.

Twelve specimens were tested under monotonic loading and a similar number of specimens were tested under low cyclic loading. All specimens consist of the connector with a height of 100 and 75 mm with different thickness of flange and web as shown in Table 3.1.

Table 3.1. The angle geometric properties

Specimen	Height (h) (mm)	Length (Lc) (mm)	Web (tw) thickness(mm)	Flange thickness (tf)(mm)
A10050	100	50	6	8.5
A7550	75	50	5	7.5
A10030	100	30	6	8.5
A7530	75	30	5	7.5

Using HSC in composite construction enables more slender structures and increases their load carrying capacity where durability and strength are important concerns (Portolés et al., 2011). The use of HSC along with using shear connectors in composite beams is becoming more common. Some results from push-out tests of headed studs in normal and high strength concrete are presented (An & Cederwall, 1996; Bullo &

Marco, 2004) but there is no study on the behavior of angle shear connectors in HSC. Therefore, it is interesting to study the behavior of the angle shear connectors in HSC.

The aim of this thesis is to investigate the behavior and effects of different sizes of angle shear connectors in HSC under monotonic and low cycle fatigue loading. Ultimately, the load capacities of angle connectors in this type of concretes can be determined and evaluated with current design codes.

There has been limited study of the behavior of angle shear connectors in composite beams. Choi et al. (2008) and Choi et al. (2011) investigated the fatigue strength of welded joints between angle shear connectors and the bottom plate in steel-concrete composite slabs through fatigue tests. The research confirmed that the stress level at the welded joint was small and much lesser than the fatigue limit.

Empirical equation 2.6 was developed by Kiyomiya and Yokota (1987) and suggested by Kiyomiya and Yokota (1986) to predict the load-carrying capacity of the C-shaped shear connectors including angle connectors (Figure 3.1).

In this study, the suggested equations which predict the shear capacity of channel and angle connectors are also evaluated.

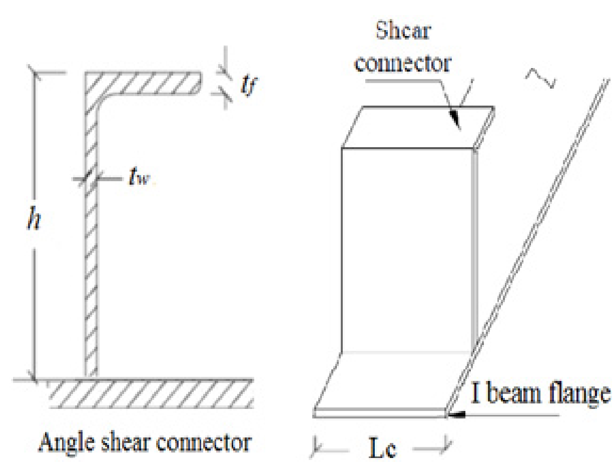


Figure 3.1. Details of connectors' specifications

3.2.2. Concrete materials

Air-dry condition aggregates were used in all the concrete mixes. The fine aggregate was graded silica sand with a maximum nominal size of 4.75 mm and the coarse aggregate was crushed granite with a maximum nominal size of 10 mm. The cement used in all mixes was Ordinary Portland Cement (OPC) corresponding to ASTM C150 (2005). To attain acceptable workability, super plasticizer (SP) was used in the concrete mix. The slump test results for different concrete mix designs were achieved to be between 75-100 mm for H_1 to H_3 mix designs (Figure. A.2. in APPENDIX A). The slump records were measured before the concrete pour.

Both concrete slabs were cast in the horizontal position, as is done for composite beams in practice. A reliable quality of concrete for both sides of the specimen slabs was assumed. All specimens were cured in water at least for 28 days before testing.

Using high strength concrete elements became a common trend in modern construction (Shah & Ribakov, 2011). Using HSC in composite construction enables more slender structures and increases their load carrying capacity where durability and strength are important concerns (Portolés et al., 2011). Also, the use of high strength concrete substantially changes the ratio between maximum slip requirement and connection deformation capacity and demands direct connection ductility checking (Bullo & Di Marco, 2004). In respect of the enhanced material properties of HSC, which increases service loads, it is obvious that the fatigue limit state becomes more significant compared to the common material strength. In addition, use of HSC makes more cost-effective products and offer a feasible technical solution or a combination of both. For the application of HSC to composite beams, the properties of specific ductility and its load carrying capacities are of particular interest as well as its fatigue behavior. The use of HSC along with using channel and headed stud shear connectors in composite beams is becoming more common. Some results from push-out tests of headed studs in normal and high strength concrete are presented (An & Cederwall, 1996; Bullo & DI Marco, 2004) but there is only limited study of behavior of channel shear connectors in HSC. Therefore, it is interesting to study the behavior of the channel shear connectors in HSC.

Based on the ACI 363 (1984), the compressive strength of HSC varies between 41 to 82 MPa. Compression strength level of 63 ,80 and 82 MPa as representative of this type of concrete were used in this study. The chosen levels of HSC are from the middle and end of this range where they almost cover the total range. Therefore, these two levels of HSC can be defined as the representative of the high strength concrete in this study. The mix properties of HSC materials are presented in Table 3.2.

Table 3.2. Mix proportions of high strength concrete materials by weight

Mix no	Cement (kg/m^3)	Coarse aggregate (kg/m^3)	Fine aggregate (kg/m^3)	Water (kg/m^3)	Silica fume (kg/m^3)	SP (%)	W/C	Modulus of Elasticity (GPa)	Compressive Strength (MPa)
H_1 Series	460	910	825	168	40	0.5	0.37	39	82
H_2 Series	460	910	825	168	40	0.5	0.37	38	80
H_3 Series	360	940	870	180	-	1	0.50	32	63

3.2.3. Concrete compression test

The compressive strength of concrete is basically obtained from testing the cube and cylinder samples. The compressive strength of normal concrete obtained from the cube samples is higher compared to the cylinder samples (Neville, 1995). According to British standard code, 100 mm concrete cubes are employed as standard specimens for measuring the compressive strength. On the other hand, 150×300 mm concrete cylinders are used in the American standard code. Because of the differences in the geometry of the samples (aspect ratio) and the end effect provided by the machine plates, the cube samples show a higher compressive strength (Islam, 2002). Generally speaking, the compressive strength of the cylinder samples is 5% to 25% lower compared to the cube samples for a given concrete mix. It is worth noting that the difference between the cube and cylinder samples decreases with an increase in the concrete strength (Neville, 1995). A factor of 1.2 (BS 1881: Part 120 (1983)) is usually applied to convert the compressive strength of the cylinder samples to the cube samples for normal strength concrete. In this study, both cylinder and cube concrete samples were tested based on the ASTM C39 (2005) and BS 1881: Part 120 (1983). The presented results of the compressive strength in this thesis are achieved by taking the average of compressive strength of both types of samples where the shape factor 1.2 was implemented for the cylinder samples.

All the push-out specimens and the relevant standard cylinder and cubic specimens were cured in water for at least 28 days before testing. Push-out test and concrete compression test specimens were tested on the same day for accurate results.

3.2.4. Test setup

The push-out specimens vary based on the size of the connector in the concrete slabs. As many specimens were considered under monotonic loading, also the similar ones then were considered under fully reversed low cycle fatigue loading. Push-out specimens consist of a steel I beam with two slabs attached to each flange of the beam (Figure 3.2). One connector was welded to each beam flange and for those specimens with reinforced concrete slabs, two layers of steel bars with four 10 mm diameter steel bar hoops were applied in two perpendicular directions for all slabs. All the details of push-out specimens followed those of Maleki and Bagheri (2008) and Maleki and Mahoutian (2009). Four types of angle connectors with 75 and 100 mm in height and 30 and 50 mm in length were used.

The preparation of specimens for the test and the procedure of push-out test from the specimen's concrete casting to the fracture of specimens in this study is shown in Appendix A.

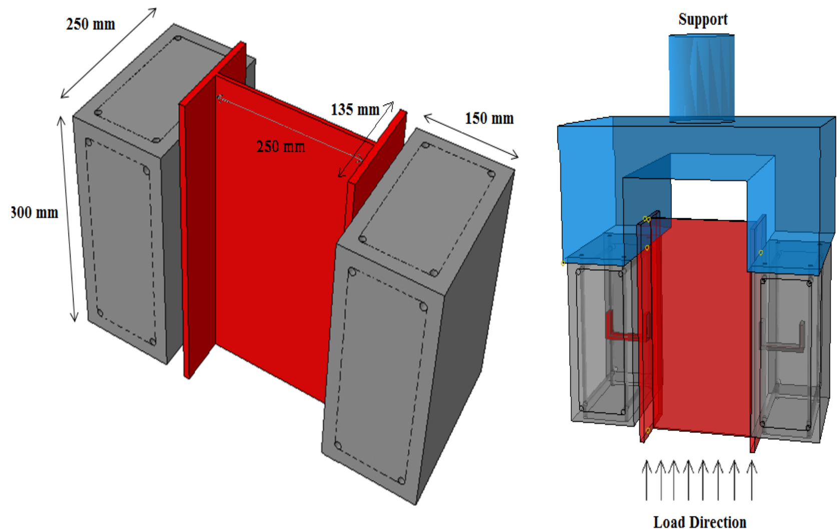


Figure 3.2. Details of typical push-out test specimen

3.2.5. Specimens' symbolization

In the symbolization of the specimens, the first letter/s indicates the type of concrete referring to its series. The first two/three digits indicate the height and the last two/three digits indicate the length of the shear connector in the concrete slabs. The letters M and C specify monotonic and cyclic loading. Figure 3.2 shows the details of a typical specimen used in push-out test.

3.2.6. Loading and test procedure

Shear connectors are used to develop composite action in a beam. The connectors should be able to transfer shear forces even under severe load reversals that can take place in the beam during strong earthquakes. Consequently, it is expected that shear connectors experience extreme cyclic shear reversals (McMullin et al., 1993). In this case low cycle fatigue load is important.

The interested objective of this thesis is in order to develop additional data and to understand the performance of angle shear connectors subjected to low cycle fatigue loading. The load was applied using a 600 kN capacity universal testing machine (Figure 3.3). Specific support was applied for loading the slabs (Figure 3.4) and the cyclic loading procedure is similar to Civjan and Singh (2003), and also Maleki and Bagheri (2008a).

Due to the unidirectional nature of the load test frame, specimens were rearranged earlier to every loading procedure when turning the specimen and loading the upper concrete faces. Monotonic loading involved slow increment of the loading until failure. Pseudo-dynamic loading involved of three cycles at ±1/3 M, ±2/3 M, and ± M, where M is the static yield capacity of the control specimen, as determined from the load-slip curve of monotonic loading. The static yield capacity of each specimen was used to compare with the results of reversed cyclic loading after three cycles. This phenomenon helps to understand the amount of shear strength reduction after cyclic tests.

Generally two methods of loading can employ for applying mechanical test conditions: load-control and displacement-control. It is not possible to precisely maintain test conditions using the load-control method because in these situations the displacements can change with no change in the load input. The most important cases in which this occurs are at lower loads that the slope of the load-slip curve is small and for some specimens may approach zero. Also at the other extreme, the fracture of a specimen results in a sudden decrease in load with increasing displacement (slope of the load - slip curve becomes negative).

Figure 3.3. Universal testing machine used to apply loading

Figure 3.4. Specific support used for loading the slabs

The displacement-control method provides the means for testing these extreme conditions. This is currently not possible to load-control methods since the required loads cannot be measured and the appropriate combinations of loads are, therefore, undefined (Goel et al., 1998). In this thesis, the displacement-control was used throughout with a rate of 0.04 mm/s similar to other researches (Maleki & Bagheri, 2008a; Maleki & Mahoutian, 2009).

The steel I beams were placed on universal test machine deck. Since varying the connector orientation makes a variation in the ultimate strength of the connector and relative stiffness (Maleki & Bagheri, 2008b), this matter was considered in the push-out test and then, the same orientation for connectors was considered in the first half cycle loading of all specimens (Figure 3.2). The practical load and relative slip between the I beam and the concrete block was automatically recorded at each time step by the universal test machine. To obtain a hysteresis loop record for the low cycle fatigue test, the load-slip behavior was carefully recorded in each half cycle of the test during reversing the specimen. The practical load and relative slip between the I beam and the concrete slab were automatically recorded at each time step by the universal test machine in each half cycle of the test. Using these data a hysteresis loop record for the low cycle fatigue test was achieved.

Due to the unidirectional nature of the load test frame, specimens were rearranged earlier to every loading procedure when turning the specimen and loading the upper concrete faces.

3.3. Finite element analysis

3.3.1. Introduction

In this section an effective numerical model is proposed using finite element method to simulate the push-out test of angle shear connectors. The focus is on the shear strength of angle shear connectors embedded in high strength concrete slab under monotonic loading. The models have been validated against test results presented in the experimental tests. The models then compared with data given by equations for the prediction of shear capacity of shear connectors. Parametric studies using this nonlinear model are performed to investigate the variations in concrete strength and connector dimensions.

The detailed Finite Element Analysis (FEA) also used to evaluate and compare the results of the push-out tests. Actually, the purpose of this part is to find the difference between reality and the simulation of push-out test results in the first stage and to make the shear strength of the push-out test as the results of the parametric study in the second stage. Consequently, the results of shear strength for shear connectors can be estimated by changing the different effective parameter of connector's capacity without doing expensive push-out tests. In addition, the parametric modelling could help to achieve certain results that would not be observed through experimental testing. This objective is achieved once the accurate modelling considers the parameters like nonlinear materials and geometries (i.e. concrete crushing and cracking, contact interaction), suitable elements to model the interaction between the steel and concrete. In addition, the modelling must provide appropriate solutions to overcome the convergence problem.

The specimens modelled in finite element analysis were chosen from the specimens in the experimental tests and developed in the finite element program (ABAQUS). A good finite element model can predict the shear strength close to the values obtained from tests. Therefore, by using this model, the effects of various parameters such as flange and web thickness, height and length of shear connectors and different concrete properties could be effectively predicted as well.

In this parametric study, evaluation of the available equations for the prediction of angle shear connector's capacity was aimed. At first, the finite element models matching the properties of experimental specimens. Then they were developed and the extended results were compared to the push-out test results. After adopting the appropriate material model, mesh sizing and interface boundary conditions, the model is considered accurate enough to predict the shear strength of angle shear connectors embedded in their relative concrete. In the next step, the evaluation of the equations for estimation of shear capacity of angle shear connector embedded in concrete slab was aimed. This phenomena helps to realize the accuracy of the equations for estimation of angle's capacity in high strength concrete.

3.3.2. General descriptions

Nonlinear three-dimensional finite element analysis of push-out test was engaged. In addition, the effect of some parameters, such as the flange and web thickness, height and length of shear connectors and different concrete properties was investigated to develop the connection behavior. General contact using the ABAQUS explicit program was proposed to model the interaction between the shear connector and concrete. Satisfactory agreement was achieved between the results in terms of the load-slip relationship and shear strength of the connectors. The details of finite element modelling for specimens are described in the following sections.

3.3.3. Material properties

3.3.3.1. Steel

The kinematic bilinear stress-strain relationship with full plastic stress was considered for the shear connectors and steel reinforcing bar in the concrete slab. Figure 3.5 shows the stress-strain relationship for the steel materials. The von Mises yield criterion was used to define the material yield surface, and an associated flow rule was used to determine the plastic deformation. For all steel materials, the elasticity modulus (E_s), density (γ) and Poisson ratio (υ) were assumed to be 205 GPa, 7800 kg/m^3 and 0.3, respectively. The yield and ultimate strength of steel components were achieved from the steel coupon testing as described in Appendix B.

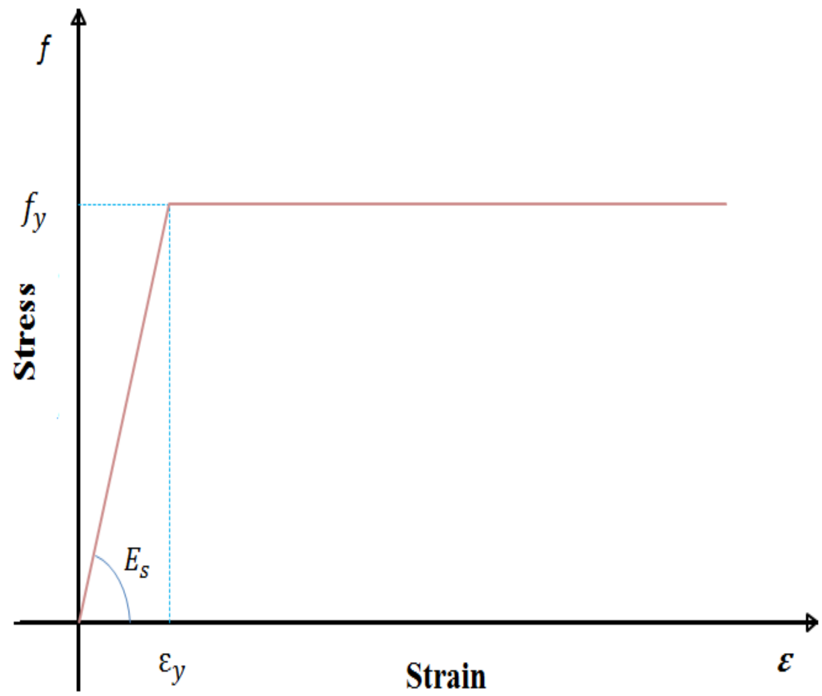


Figure 3.5. Stress-strain relationship for steel materials (BS 5950, 1990)

3.3.3.2. Concrete

The stress-strain relationship, according to the EC2 (Eurocode 2, 2002), was used to define the behavior of the concrete material. Figure 3.6 shows an equivalent uniaxial stress-strain curve to consider the nonlinear behavior of concrete in compression. The compression curve is divided into three parts including the elastic range, the nonlinear parabolic portion and the descending slope. The value of the first part is the proportional limit stress of 0.4 f_{ck} (Eurocode 2,2002), where f_{ck} is defined as the concrete strength of the cylinder specimen and is equal to 0.8 f_{cu}, while f_{cu} is the concrete strength of the cubic specimen. The strain (ε_{c1}) in relation to f_{ck} is equal to 0.0022. The stress for nonlinear parabolic part can be obtained as given in Equations 3.1 and 3.2 (Eurocode 2, 2002),:

$$\sigma_c = \left(\frac{kn - n^2}{1 + (k-2)n}\right) f_{ck} \tag{3.1}$$

Where:

$$n = \frac{\varepsilon_c}{\varepsilon_{c1}}$$

$$k = 1.1\, E_{cm} \times \frac{\varepsilon_{c1}}{f_{ck}}$$

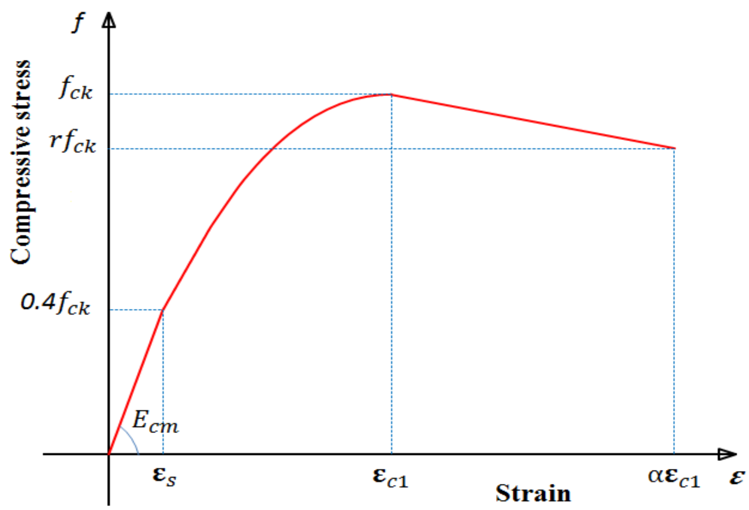


Figure 3.6. Stress-strain relationship for compression behavior of concrete (BS 5950, 1990)

The descending part can be used to define the post failure for concrete compression behavior in specimens in which concrete crushing occurred. The descending slope ceases at a stress value of $\mathrm{r}f_{\mathrm{ck}}$. Here r is the reduction factor obtained by Ellobody et al., 2006). The range of r can be between 1 to 0.5 equivalents to the concrete cube strength with a range of 30 to 100 MPa. The ultimate strain of concrete at failure ($\mathcal{E}_{\mathrm{cu}}$) is equal to $\alpha\mathcal{E}_{\mathrm{c1}}$. According to EC2 (Eurocode 2, 2002) and BS 8110 (1997) $\mathcal{E}_{\mathrm{cu}}$ is equal to 0.0035, which means that here α is equal to 1.75 and Poisson ratio (υ) were assumed to be, 2,350 kg/m^3 and 0.2, respectively. The elasticity module (E_{cm}), was obtained from EC2 (Eurocode 2,2002) given in Equation 3.2

$$E_{cm} = 9.5(f_{ck} + 8)^{\frac{1}{3}} \qquad E_{cm} \text{ in GPa and } f_{ck} \text{ in MPa} \qquad (3.2)$$

The nonlinear behavior of concrete in tension using the uniaxial stress-strain curve is shown in Figure 3.7. The tensile stress in concrete increases linearly to the strain before the concrete cracks and decreases to zero once the concrete cracks. Cracking can be defined in three ways, i.e. linear, bilinear and exponential function when there are no reinforcing bars or only a few reinforcing bars in the concrete (ABAQUS). The exponential function type (Cornelissen et al., 1986) was used to define the tension softening in this analysis. Figure 3.8 shows the tension stress-crack displacement relationship. In the models, damage for concrete cracking was defined for the specimens.

The concrete damage plasticity model presumes a non-associated potential plastic flow. The Drucker-Prager hyperbolic function was used as the flow potential. The material dilation angle (Ψ) and eccentricity ($\boldsymbol{\varepsilon}$) were taken as 15^{o} and 0.1, respectively. The ratio of biaxial compressive strength to uniaxial compressive strength ($\frac{f_{b0}}{f_{c0}}$) was taken as 1.16.

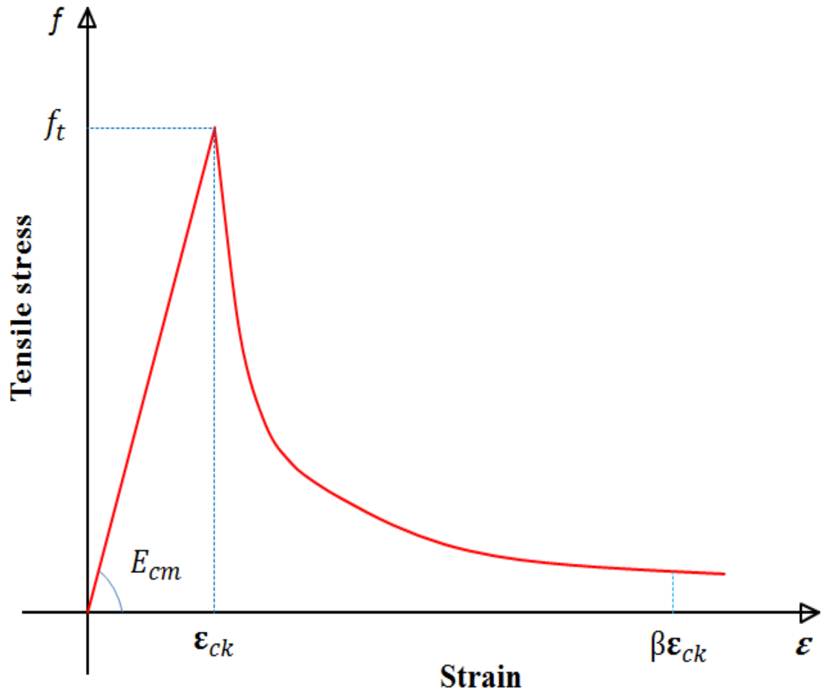


Figure 3.7. Stress-strain relationship for tensile behavior of concrete (Cornelissen, et al., 1986)

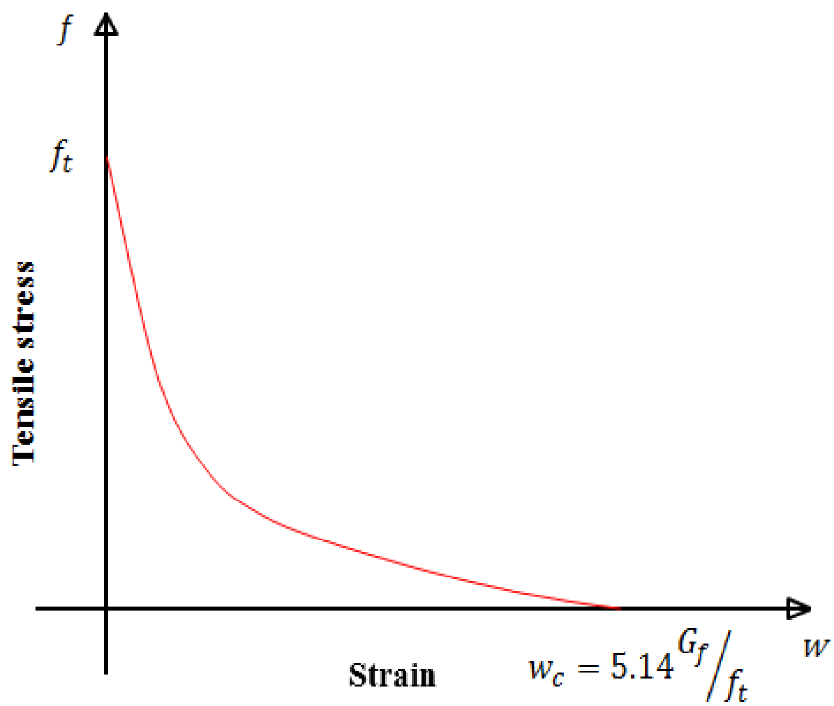


Figure 3.8. Exponential function of tension softening model (Cornelissen, et al., 1986)

3.3.4. Modelling of the specimens

To obtain appropriate results using the ABAQUS software, all components including the steel I beam, shear connectors, reinforcing bars and concrete slab were modelled. To model the interaction between the components, general contact with the ABAQUS explicit program was employed. This is a very important part in the analysis and needs more attention as inappropriate interaction may cause a convergence problem. The geometry of the specimens was described earlier. The geometric models consist of three main parts: 1) Steel includes steel I beam and the shear connector, which are merged together and made one part, 2) Concrete slab and 3) Reinforcing bars.

The geometry of the parts in a typical specimen is shown in Figure 3.9. Figure 3.10 shows the steel and concrete parts modelled in software separately and Figure 3.11 displays the placement of steel angle connectors in concrete slab for reference. Figures 3.12 to 3.13 show the details of angle shear connectors used in this study.

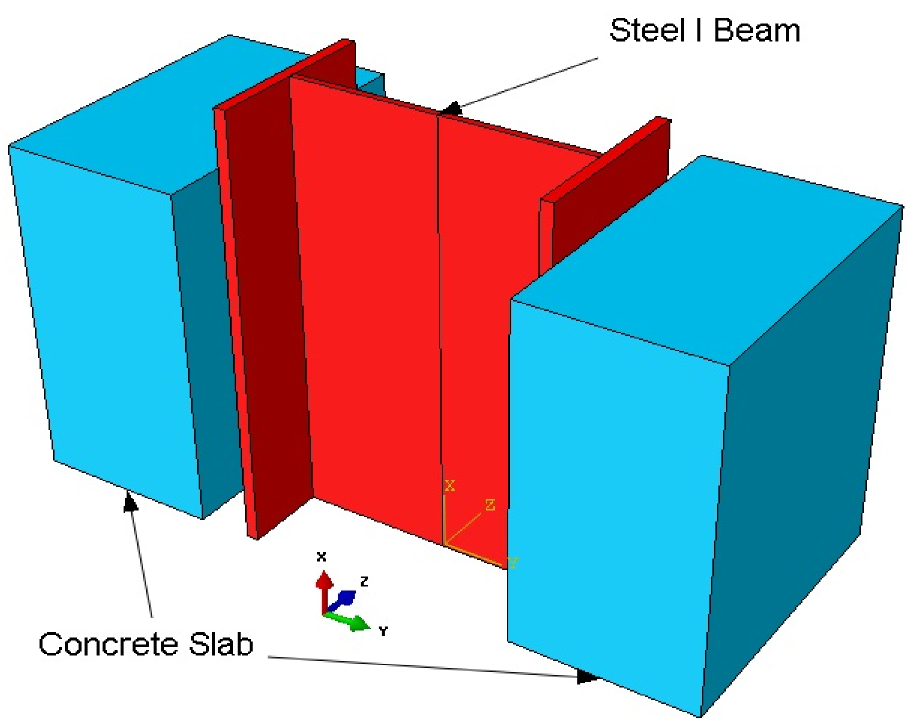


Figure 3.9. Typical view of the specimens in the experimental test

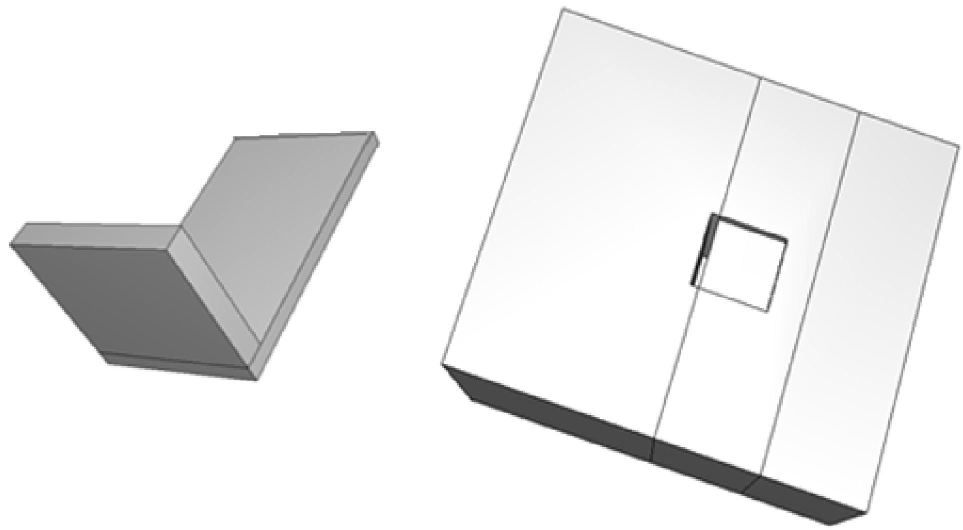


Figure 3.10. Separate parts of assembly (angle and concrete slab)

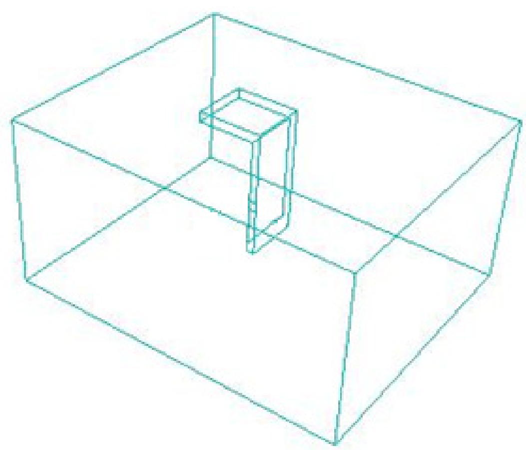


Figure 3.11. Placement of steel connector in concrete slab

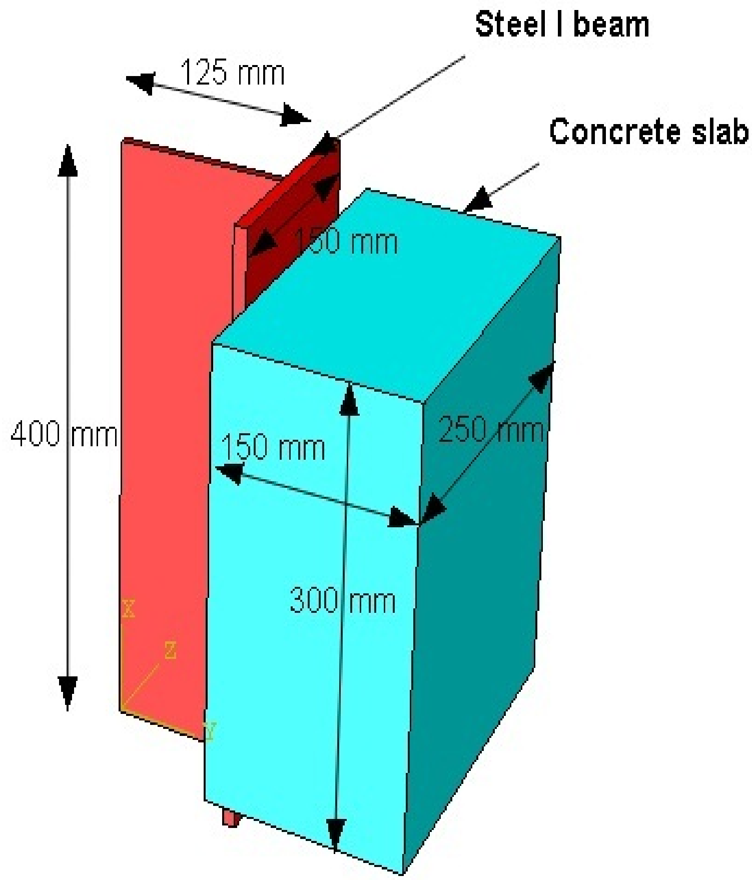


Figure 3.12. Typical geometric modeling of the specimens

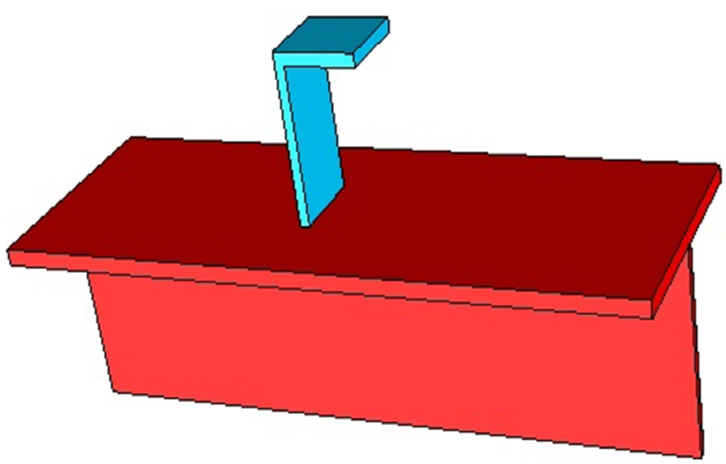


Figure 3.13. Typical steel part with angle in specimen

3.3.5. Element type

Figures 3.14 to 3.16 show the typical finite element mesh of the specimens to model the geometry of the test specimen. The mesh size was selected such that it satisfies a good accuracy and a reasonable computational time. The types of element used in the finite element modelling are presented as follows.

1. The eight-node solid element (C3D8R) was used to model the steel beam and the shear connector and the concrete slab, which has three translational degrees of freedom at each node. This element can consider concrete cracking and crushing in three orthogonal directions at each integration point.
2. The truss element (T3D2) was used for modelling the reinforcing bars. This element has three translational degrees of freedom (translation in the x, y and z directions) at each node.

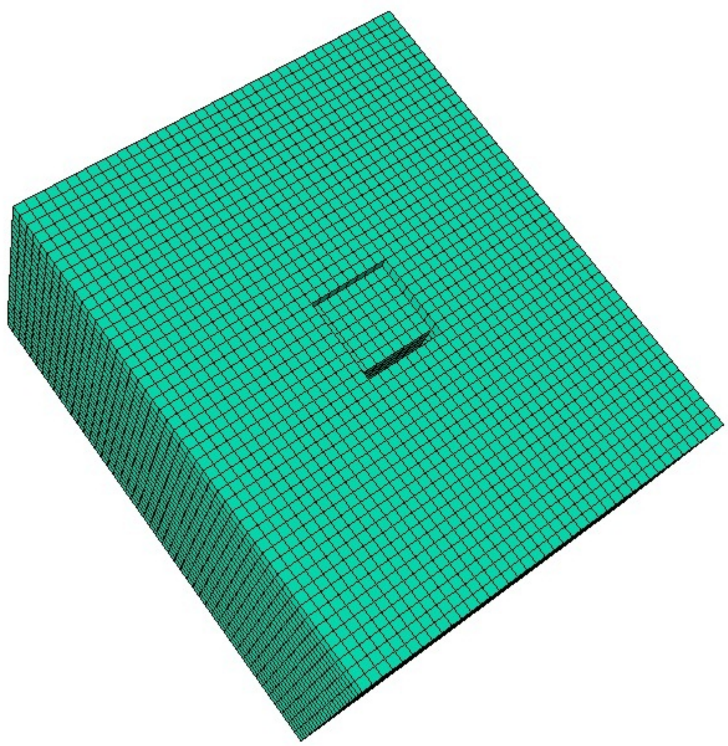


Figure 3.14. Typical meshing of concrete part in specimen

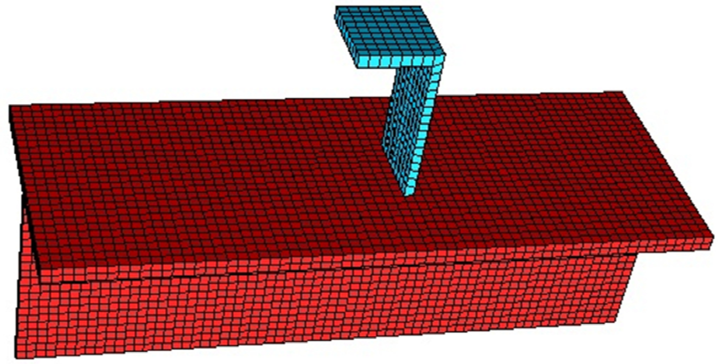


Figure 3.15. Typical meshing of steel part in specimen with angle shear connector

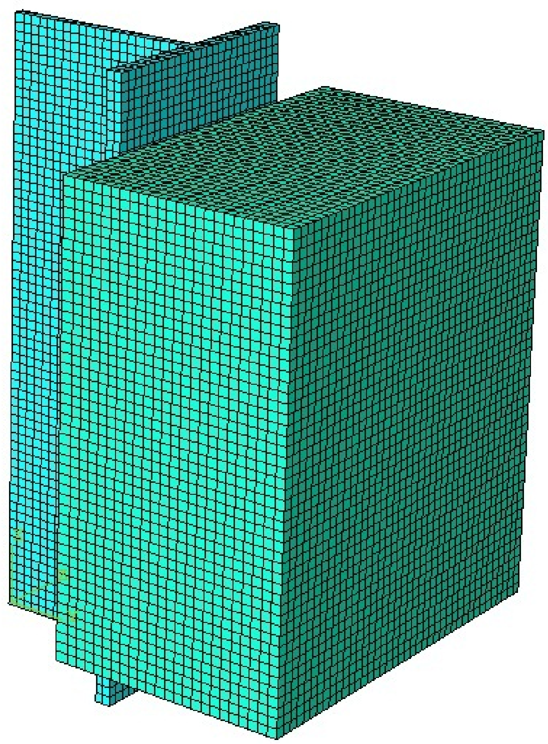


Figure 3.16. Typical meshing of whole the half push-out specimen

3.3.6. Element Interaction

The finite element analysis behaviour is dependent on an accurate definition of the relation between the parts. Two steps were considered to define the interaction between steel, concrete and reinforcing parts, as below:

3.3.6.1. Embedded element

The interaction between the reinforcing bars and the concrete slab was assumed to be full bond with no slip between them. Consequently, the embedded element was used for this purpose. For assigning the embedded element, the part of the concrete slab, which was close to the reinforcing bars, was defined as the embedded.

3.3.6.2. Contact interaction

General contact was chosen to define the contact interaction between the steel and concrete parts. To define general contact, some text comment, such as *CONTACT CLEARANCE ASSIGNMENT and *CONTACT CONTROL ASSIGNMENT were defined in the ABAQUS explicit program. Figure 3.17 shows the connector's surfaces contacted to concrete surface for angle shear connectors.

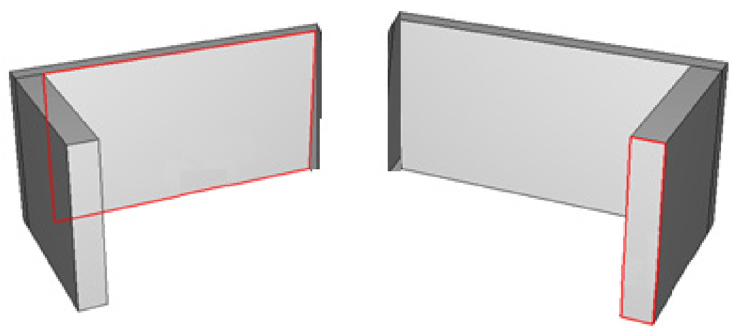


Figure 3.17. Angle surfaces which are contacted to concretes

3.3.7. Contact properties

Both the normal and tangential contacts were considered in the contact interaction. Geometry scaling, which is a softened pressure-over closure relationship was used to model the normal contact between the steel and concrete. This model provides a simple interface to increase the default contact stiffness when a critical penetration is exceeded as shown in Figure 3.18. The penetration measure, d, is defined either directly or as a fraction, r, of the minimum element length, L_{elem} in the contact region. Each time the current penetration exceeds a multiple of this penetration measure, the contact stiffness are scaled by a factor, S. The initial stiffness K_i is given in Equation 3.3

$$K_i = S_0 . k_{dflt} \tag{3.3}$$

Where:

i: Segment member

S: Geometric scale factor

k_{dflt}: Default stiffness

r: Over closure factor

L_{elem} : Element length

d = r.L_{elem}: Over closure measure

S_0: Initial scale factor

In this analysis, parameters like S_0, S and d are considered as below:

$3< S_0 <10$ $\quad$ $1.05 < S <1.1$ $\quad$ $1\times10\text{-}6 < d <1\times10^{-7}$m

Tangential contact was used to define the friction contact between the steel and the concrete. The coulomb friction model was used to model the friction contact between the steel and concrete. The friction coefficient between the steel and the concrete was assumed to be 0.3.

Angle surfaces contact separately with the corresponding surface of the concrete slab in shape of surface to surface contact. In all cases, the contact surfaces of the slab act as master and Angle surface act as slave.

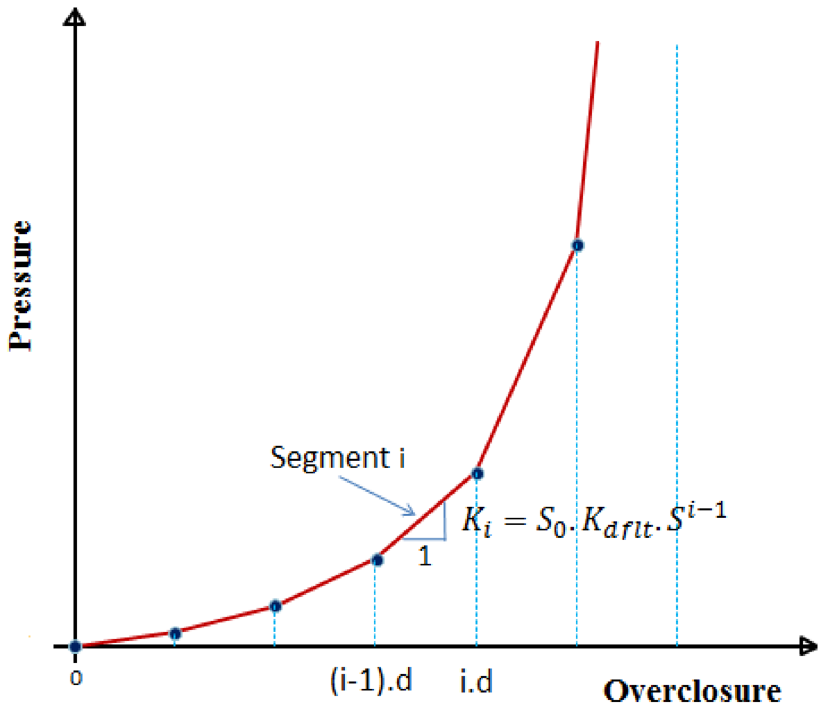


Figure 3.18. Softened pressure-over closure relationship – Geometry scaling (ABAQUS manual, 2011)

3.3.8. Loading and boundary condition

The similar loading of the test specimen was considered for the loading in the finite element analysis. A vertical velocity loading was applied to the concrete slab. The load direction is downward. Velocity controlled loading provides a more stable system in the nonlinear stage rather than force controlled loading. A special 0.04 mm/s rate of velocity was monotonically used for the loading rate. This rate was achieved by comparing the kinematic energy with the internal energy when the effect of dynamic analysis could be neglected. The real rate, which is much smaller than this rate, results in longer time analysis with little effect on the accuracy of the results. As shown in Figure 3.19 all nodes of I beam section on the symmetric plan were constrained against displacement on the X, Y and Z direction axis to simulate boundary conditions for the specimens.

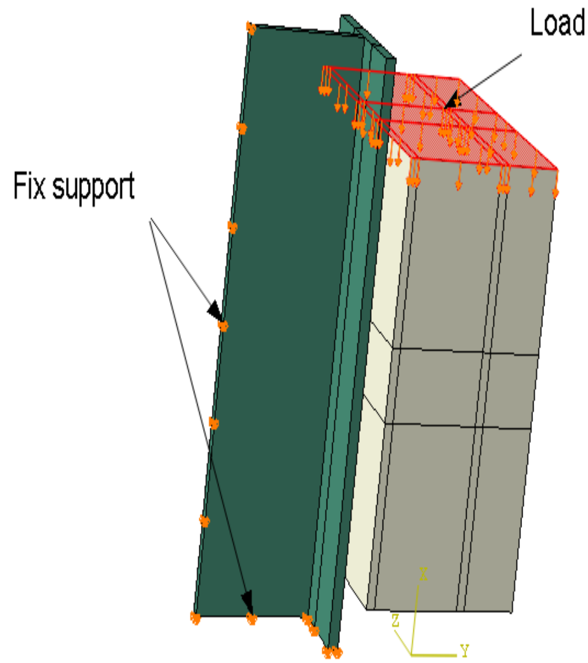


Figure 3.19. Boundary condition in the specimen

The loading step is the next procedure after the assembling of the parts. In this analysis, linear loading was considered. This loading is similar to the experimental loading procedure in push-out test when the it was applied instantly. Since the ultimate load cannot be realized in parametric analysis, 1.5 times of real shear strength of any specimens considered for maximum loading.

The next step is obtaining the output results of the software such as stress, strain, and boundary conditions.

3.3.9. Analysis solution

Available analysis, such as general static in the ABAQUS Standard was used for initial analysis; however, it caused a convergence problem and stopped the analysis even at the beginning of the analysis. The results of the RIKS method also resulted in a convergence problem (Kim & Nguyen, 2010). Consequently, ABAQUS explicit was employed for analysis, which is suitable for nonlinear materials and geometry, large deformation and concrete damage and discontinuous parts (ABAQUS). Reduced integration, second accuracy and enhanced control were considered in the analysis for solid element.

4 RESULTS AND DISCUSSION

4.1. Preface

This chapter is composed of two parts to present the results and discussion of this study. Results and discussion on the experimental test of the specimens is described in the first part. In the second part, results and discussion on the finite element analysis are provided to verify the finite element analysis results against experimental test results where ever applicable.

4.2. Experimental test results

This section comprises of the experimental results related to the push-out specimens along with a description of observed failure mechanisms. The main test results are shown in the form of load-slip curves. In all the load slip graphs shown in this section, the abscissa represents the average slip in mm at the interface of the steel section and the concrete slab. The ordinate represents the load per connector in kN.

4.2.1. Load-slip analysis for monotonic loading

Static strength is necessary for shear connector design and ductility is an essential assumption in the design which is confirmed throughout ultimate slip (Shim, 2004). The load-slip curve for one shear connector is used to extract the mechanical properties of the shear connector. The slip occurs between the I beam and the concrete block. The static curve is used to see if there is sufficient ductility for shear connectors as well.

In the result of universal testing machine under monotonic loading, the applied load had a slip increment record on non-linear stage. The reason for this testing was to define the load - slip curve more accurately and to record the distortion of the specimens at failure. It is hardly possible to record readings for many examples with connector rupture failure that cause the sudden failure of shear connector. Failures of concrete specimens, were accompanied by a significant amount of load beyond the ultimate load level.

The load-slip curve for all specimens with angle shear connector embedded in reinforced concrete high strength slab was drawn and presented in Figure 4.1. The load-slip curve of specimens with the failure of the connector fracture type came to a sudden end. Table 4.1 presents the results of the push-out tests for angle shear connectors embedded in high strength reinforced concrete under monotonic loading.

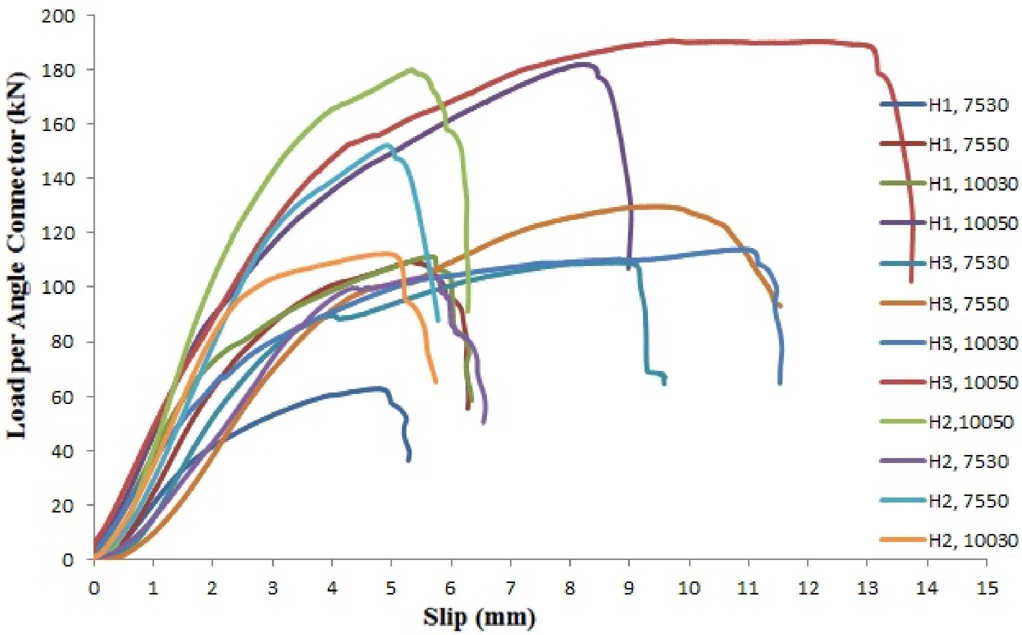


Figure 4.1. Load-slip curves of angle connectors embedded in in high strength reinforced concrete under monotonic loading

Table 4.1. Monotonic test results for angle connectors embedded in high strength reinforced concrete

Specimen	Failure Load (kN)	Maximum Slip (mm)	Failure type
$H_1$10050-M	178.3	0.5	Connector fracture
$H_1$7550-M	152.9	1.0	Connector fracture
$H_1$10030-M	112.7	2.0	Connector fracture
$H_1$7530-M	103.7	2.0	Connector fracture
$H_2$10050-M	180.0	1.0	Connector fracture
$H_2$7550-M	145.9	1.0	Connector fracture
$H_2$10030-M	92.7	2.0	Connector fracture
$H_2$7530-M	63.7	2.0	Connector fracture
$H_3$10050-M	160.5	1.0	Connector fracture
$H_3$7550-M	130.7	2.0	Connector fracture
$H_3$10030-M	83.1	1.5	Connector fracture
$H_3$7530-M	44.8	1.5	Connector fracture

Analyzing the load-slip curves of specimens with angle shear connectors show that when they are subjected to the monotonic load, the connector yielded in the maximum slip and shear load capacity. The load-slip curve for one shear connector was employed to extract the mechanical properties of that connector. The slip occurred between the steel I-beam and the concrete block. Based on Eurocode 4 (Eurocode 4, 2004), a connector may be taken as ductile if the characteristic slip capacity is at least 6 mm. It can be concluded that angle shear connectors are not sufficiently ductile in the peak load for the monotonic loading while they are embedded in HSC since the relative slip was measured as 0.5 – 2.0 mm, for all connectors. In all specimens, a sudden termination of the load-slip curve was observed. Almost all angle specimens do not experience a yield plateau, which results in an increase in the slip while the load reaches its peak.

The load-slip curves provide the opportunity to assess the ductility factor of shear connectors. Considering these curves, the ductility of the connectors can be evaluated by using a ductility factor, μ, which is defined in terms of an equivalent elastoplastic load-deflection curve, as shown in Figure 4.2. The load-deflection curve of a shear connector was idealized by two straight lines representing the initial stiffness and the maximum strength. The intersection of those two lines was taken as the equivalent yield displacement, Δ_y.

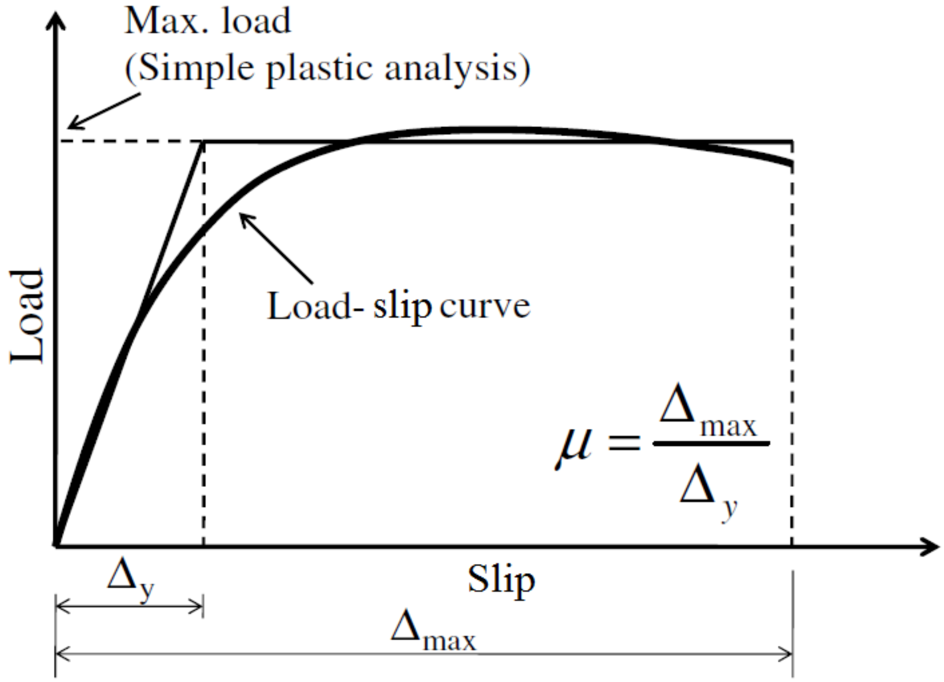


Figure 4.2. Definition of ductility factor

The maximum deflection, Δ_{max}, was defined as the deflection at the condition which the shear connector failed. The corresponding ductility factor was then calculated as Δ_{max}/Δ_y. A higher ductility factor shows a more ductile composite system. In general, the greater the ductility factor of a particular system, the greater the amount of inelastic redistribution of applied load (Kwon et al., 2011). Ductility enhances safety in structures by both providing warning of impending before failure and by allowing redistribution of loads to adjacent beams. The calculated ductility factor for each specimen is presented in Table 4.2.

The results of the ductility factor show that longer connectors exhibit more ductility compared to while for shorter connectors.

Table 4.2. Ductility factor for angle specimens embedded in HSC slab

Specimen	Δ_{max}	Δ_y	μ
$H_1$10050-M	13.0	4.0	3.3
$H_1$7550-M	8.0	2.0	4.0
$H_1$10030-M	10.0	3.0	3.3
$H_1$7530-M	7.5	3.0	2.5
$H_2$10050-M	10.0	2.5	4.0
$H_2$7550-M	4.5	1.5	3.0
$H_2$10030-M	5.0	2.0	2.5
$H_2$7530-M	4.0	2.5	1.6
$H_3$10050-M	9.0	2.0	4.5
$H_3$7550-M	5.0	1.0	5.0
$H_3$10030-M	10.5	4.0	2.6
$H_3$7530-M	3.5	0.5	7.0

The results indicate that although the angle shear connector shows good behavior in terms of the ultimate shear strength, but the ductility criteria of the Eurocode 4 (Eurocode 4, 2004) which define a ductile connector when it has a slip of more than 6.0 mm, is not satisfied. This is due to the resistance of the angle connectors falls below 90% of the maximum load for a slip of less than 2.0 mm. The resistance would drop more intensely and suddenly with the increase of height in angle connectors.

4.2.2. Load-slip analysis for reversed cyclic loading

Generally speaking, by checking the monotonic tested specimens' results against the cyclic tested specimens, it was observed that the capacities are reduced when fully reversed cyclic loading is introduced after a certain number of load cycles. To prevent failure of the connector, this number of load cycles should be used as the shear strength under cyclic loading. That is why cyclic testing of these levels of stresses is needed to ensure sufficient fatigue life under reversed load cycles (Maleki & Bagheri, 2008a). The load-slip curves of the cyclic loadings of the push-out test specimens are examined in this part.

Figure 4.3 shows the typical load-slip curves, for a specimen when subjected to the cyclic loading. A summary of the results can be also seen in Table 4.3.

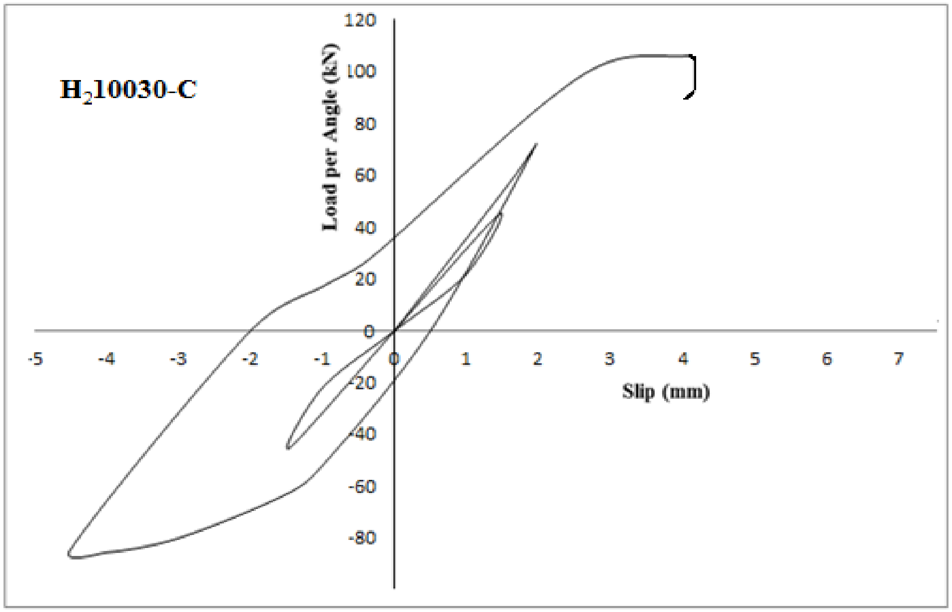


Figure 4.3. Cyclic load-slip curves of angle shear connectors specimen $H_2$10030-C embedded in HSC slab

Table 4.3. Cyclic test results for similar pair to pair specimens

Pair to pair similar specimen	Failure Load (kN)	Strength degradation (%)	Maximum Slip (mm)
$H_1$10050-M	178.3	1.4	0.5
$H_1$10050-C	175.8		1.0
$H_1$7550-M	152.9	1.4	1.0
$H_1$7550-C	150.8		2.5
$H_1$10030-M	112.7	0.4	2.0
$H_1$10030-C	112.3		1.5
$H_1$7530-M	103.7	0.1	2.0
$H_1$7530-C	103.8		2.0
$H_2$10050-M	141.0	0.6	1.0
$H_2$10050-C	140.2		1.5
$H_2$7550-M	109.6	0.6	1.0
$H_2$7550-C	109.0		2.5
$H_2$10030-M	77.9	0.5	2.0
$H_2$10030-C	78.3		1.5
$H_2$7530-M	69.6	0.1	2.0
$H_2$7530-C	69.5		2.5
$H_3$10050-M	111.3	1.3	1.0
$H_3$10050-C	109.9		1.5
$H_3$7550-M	106.1	1.1	2.0
$H_3$7550-C	104.9		2.0
$H_3$10030-M	105.2	0.3	1.5
$H_3$10030-C	104.9		1.5
$H_3$7530-M	99.0	1.4	1.5
$H_3$7530-C	97.6		2.0

All push-out specimens resisted for the first and the second half cycles of loading, at 1/3M and 2/3M (M is considered to be the failure load of the same specimen, under monotonic loading) and then fractured, at the first half cycle of the whole load (M). This failure is indicated as the cyclic failure in Table 4.2. In summary, angle shear connectors resisted 98.6% – 99.9% of their monotonic capacity when embedded in a reinforced concrete slab and subjected to the fully reversed cyclic loading. A decrease in capacity is understood to occur, after a certain number of load cycles.

One may conclude that all angle shear connector specimens, under reversed cyclic loading embedded in HSC, showed 0.1% – 1.4% strength degradation compared to monotonic cases where the failure type did not change. It can be clearly noticed that the capacities of specimens when subjected to the fully reversed cyclic loading were reduced. So under cyclic load conditions, there is no significant change on the failure loads as compared to the monotonic loading.

The strength degradation can be described due to the crushing of the concrete, on the compressive face of the slabs and due to the large slip and the plastic yielding at this region.

4.2.3. Failure mode of shear connectors

Basically two types of failure are defined in the push-out specimens. The first type is connector fracture (Figure 4.4) and the second type is concrete crushing splitting (Figure 4.5) (Maleki & Bagheri, 2008a; Shariati et al., 2010). In some test results of previous researches a combination of both failure modes was reported as well (Figure 4.6). In all tests reported in this thesis, the connector fracture was observed.

The type of failure greatly influences the strength, ductility and loading response. After the testing, the specimens were crushed in order to observe the effect of loading on the concrete slab, connector and reinforcement.

Figure 4.4. Connector fracture of failure

Figure 4.5. Failure of concrete crushing splitting

Figure 4.6. Combined failure mode of concrete and connector

All the push-out specimens experienced connector fracture mode of failure, under both monotonic and low cyclic fatigue loads (Fig. 4.7 and 4.8). Although the same type of failure was seen in monotonic and cyclic loading for all specimens, the failure in the low cycle test was less ductile compared to the monotonic loading.

Figure 4.7. Typical fracture of angle shear connector for specimen with high strength concrete (Fractured angle in slab)

Figure 4.8. Typical fracture of angle shear connector for specimen with high strength concrete (Fractured angle attached to steel I beam)

4.2.4. Effect of connector dimensions on shear capacity

It can be seen that the behavior of the 30 mm long angles differs from that of the 50 mm long. For specimens with longer angle connector, concrete cracking occurred on the sides of the slabs while such cracking was not observed in specimens with shorter angle. Hence, it can be concluded that longer angle connector contributes to more cracking in concrete. The same phenomenon was also observed for the channel shear connectors as reported by Maleki and Bagheri (2008), Maleki and Mahoutian (2008) and Shariati et al. (2011).

In this study, the height of the connectors varied from 75 mm to 100 mm. Considering the load slip curves of monotonic loading for specimens with the angle length of 50 mm, it can be observed that the specimens with 100 mm high angle connectors carried slightly higher load compared to the specimens with 75 mm high angle connectors.

This corresponds to an increase in the ultimate load capacity of 16.6%. Similarly, for specimens with the angle length of 30 mm, the angle with 100 mm height carried higher load as compared to that specimen by the 75 mm high angles. In this case, a 33% increase in the angle height raised the ultimate load capacity by 8.7%.

The fact that angle connectors with, lower height have a tendency of concentrating the applied load on a smaller area might be the cause of this conclusion. The curves also showed that the specimens with 75 mm high angles demonstrated higher flexibility compared to those with 100 mm high angles. A 2.0 mm slip at the ultimate load level was recorded for the 75 mm high angles as compared to 0.5 – 1.0 mm slip for angles with 100 mm height.

4.2.5. Evaluation of design codes for the shear capacity of angle connectors based on experimental tests

The actual shear strength of angle shear connectors obtained from push-out tests was compared with the Equation 4.1 suggested by Kiyomiya and Yokota (1986) which was modified by Kiyomiya and Yokota (1987) and Equation 4.2 proposed by Ros (2011).

$$P=65\sqrt{t_w}\, L_c\sqrt{f_c} \tag{4.1}$$

Where:

P = Shear strength of the connector (kgf)

t_w = Web thickness of connector (cm)

L_c = Length of connector (cm)

f_c = Concrete compressive strength (kgf/cm^2)

$$V_u= k \times \sqrt{f_c}\times L_c\times h \quad \text{and} \quad k=63\times\left(\frac{t_w}{h}\right) + .60 \tag{4.2}$$

Where:

V_u = Ultimate shear force in the shear failure mode or by concrete crush (N),

L_c = Length of connector (mm),

h = Height of shear connector (mm),

t_w= Thickness of shear connector (mm),

f_c= Concrete compressive strength (MPa),

The relative amount of strength of the connectors based on the push-out results and the standard codes are presented in Table 4.4.

Table 4.4. Comparison of test results with code prediction for angle connectors in HSC

Specimen	Failure load (kN)	Eq 4.1 (kN)	Test/ Eq. 4.1	Eq 4.2 (kN)	Test/ Eq. 4.2
$H_1$10050-M	178.3	73.5	2.4	243.6	0.7
$H_1$7550-M	152.9	44.1	3.5	146.2	1.0
$H_1$10030-M	112.7	67.1	1.7	197.0	0.6
$H_1$7530-M	103.7	40.3	2.6	118.2	0.9
$H_2$10050-M	180.0	72.6	2.5	240.6	0.7
$H_2$7550-M	145.9	43.6	3.3	144.4	1.0
$H_2$10030-M	92.7	66.3	1.4	194.5	0.5
$H_2$7530-M	63.7	39.8	1.6	116.7	0.5
$H_3$10050-M	160.5	64.4	2.5	213.5	0.8
$H_3$7550-M	130.7	38.7	3.4	128.1	1.0
$H_3$10030-M	83.1	58.8	1.4	172.6	0.5
$H_3$7530-M	44.8	35.3	1.3	103.6	0.4

In the Table 4.4, the ratio of strength of shear connector based on test results and available equations for their shear capacity is given to compare the strength of connectors. It is seen that the equation recommended by Kiyomiya and Yokota (1987) (Equation 4.1) is conservative while the equation suggested by Ros (2011) (Equation 4.2) gives unconservative capacities. Based on the results of angle shear connector's capacity, more research for obtaining further accurate equation for capacity of angle shear connectors is needed.

4.2.6. Summary of experimental results

In this chapter the experimental investigations of the push-out specimens with angle shear connectors embedded in the HSC in composite beams was conducted. The test experimental program involved the testing of twenty four push-out specimens under monotonic and low cyclic fatigue loading when using slab with HSC. The behavior of the angle shear connectors embedded in solid concrete slabs with high strength concrete was the objective of the experimental tests to understand the effect of several parameters on the shear capacity and ductility of these types of shear connectors.

Generally, there are three different types of failure mechanisms mean, crushing-splitting of concrete, connector fracture and a combination of these two failures. Only the connector fracture of failure was observed in the experimental investigations. The shear capacity and the ductility of angle shear connectors as main parameters in the behavior of shear connectors were discussed as well.

The details of the experimental results were fully-described in summary as follows;

1. The connectors in reinforced concrete have a relative slip of 0.5–2.5 mm under monotonic loading embedded in HSC.
2. Based on Eurocode 4, the angle shear connectors are not sufficiently ductile in the peak load for the monotonic loading.
3. The connectors in reinforced concrete under low cyclic fatigue loading, all push-out specimens fractured at the first half cycle of the whole load (M) and resisted 98.6% – 99.9% of their monotonic capacity.
4. One may conclude that all angle shear connector specimens, under reversed cyclic loading, showed 0.1% – 1.4% strength reduction compared to those monotonic cases where the failure type did not change
5. All the push-out specimens experienced connector fracture mode of failure, under both monotonic and low cyclic fatigue loads.
6. Although the same type of failure was seen in monotonic and cyclic loading for all specimens, the failure in the low cycle test was less ductile compared to the monotonic load.
7. One may conclude that higher angle connectors resist less shear once the length of the connector increases compared to lower connectors once their length increases.
8. For specimens with the higher length of connectors, cracks were observed at the top surface of the slab when the maximum load was applied.

9. For specimens with shorter connectors in the slabs, small cracks at the top surface had a tendency to develop around the center of the slabs; however, overall these specimens did not experience severe cracking.
10. The equation recommended by Kiyomiya and Yokota (1987) (Equation 4.1) is unconservative while the equation suggested by Ros (2011) (Equation 4.2) gives conservative capacities.

4.3. Finite element results

Effective numerical analyses were proposed using finite element method to simulate the push-out test of angle shear connectors. The shear strength prediction of connectors under monotonic loading embedded in HSC is the aim of this study. The models have been validated against experimental test results. Parametric studies using these nonlinear models are performed to investigate the variations in concrete strength and connector dimensions. Generating an evaluation for accuracy of the available equations for the prediction of shear capacity of angle connectors in high strength concrete is the target.

4.3.1. Verification of finite element results

The finite element results should be verified using experimental test results to ensure the accuracy of the modelling. Comparison of the load-slip relationship between the finite element results and the experimental results are shown in Figures 4.9 to 4.11. The von misses stress and plastic strains of the specimens are shown in the Figures 4.12 to 4.14 as well. According to the results, the finite element analysis is able to exhibit the accurate elastic and inelastic behavior of the connections.

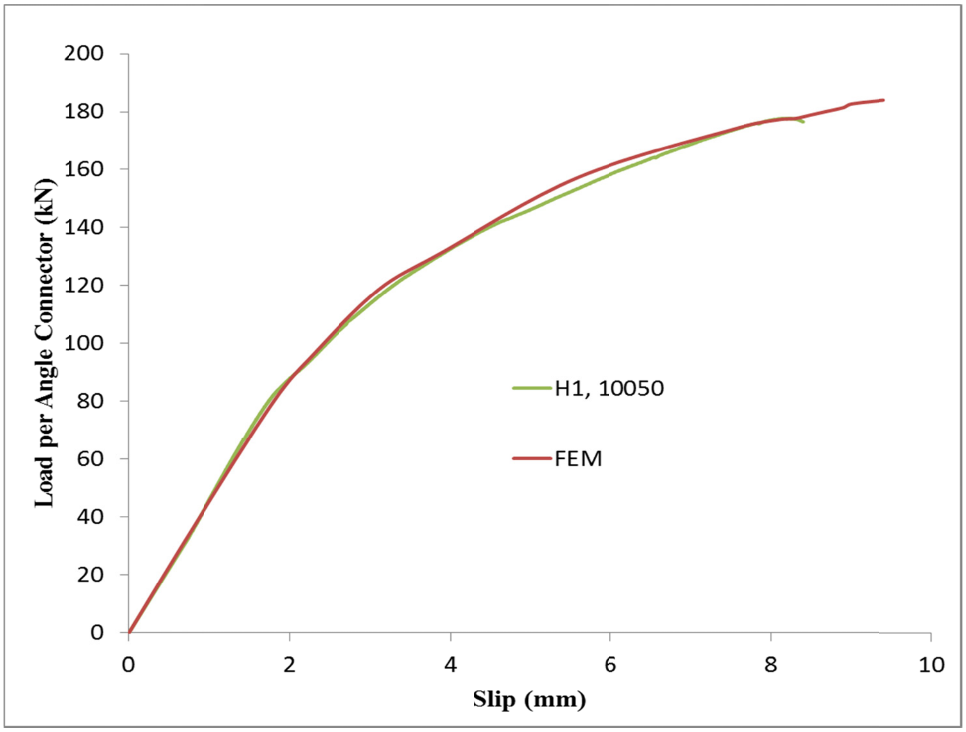


Figure 4.9. Typical comparison of load-slip between FEA and experimental test for angle specimen $H_1$10050 in HSC

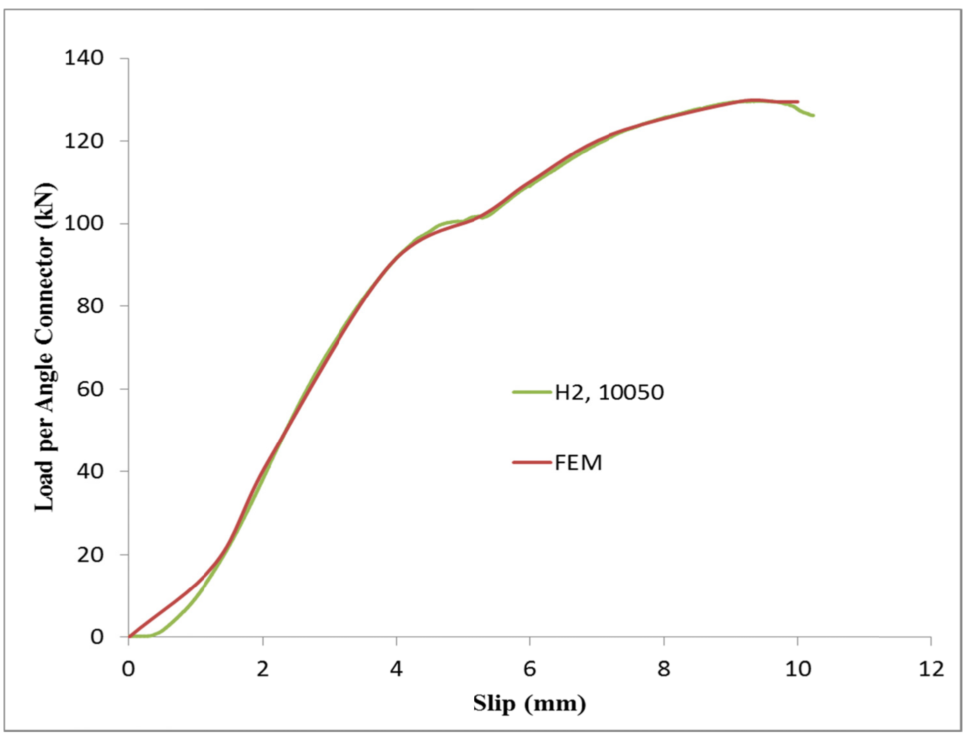


Figure 4.10. Typical comparison of load-slip between FEA and experimental test for angle specimen $H_2$10050 in HSC

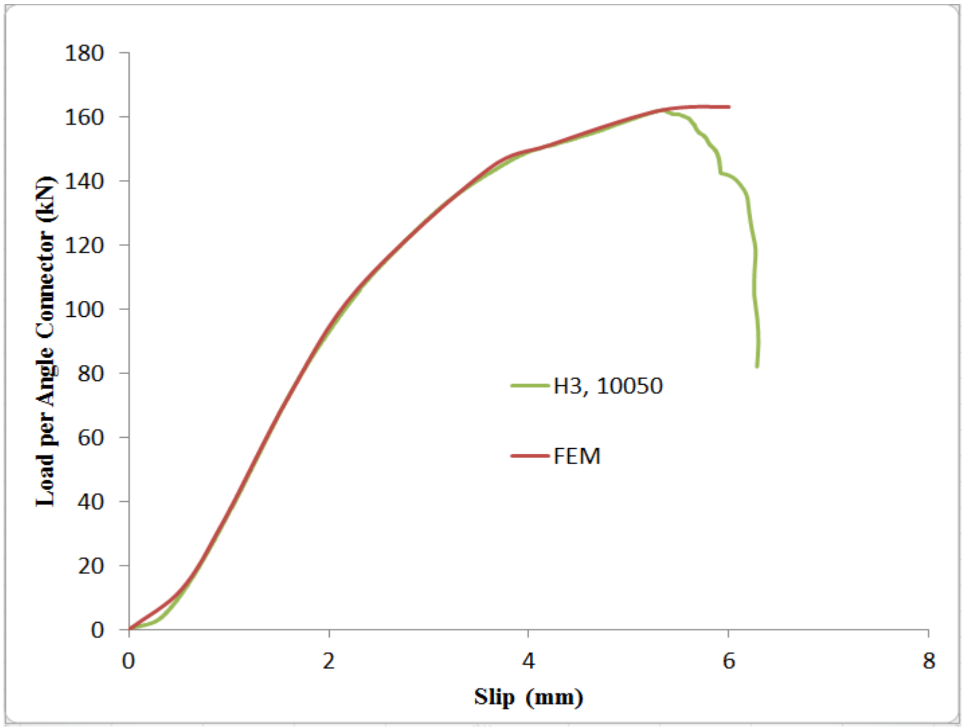


Figure 4.11. Typical comparison of load-slip between FEA and experimental test for angle specimen $H_3$10050 in HSC

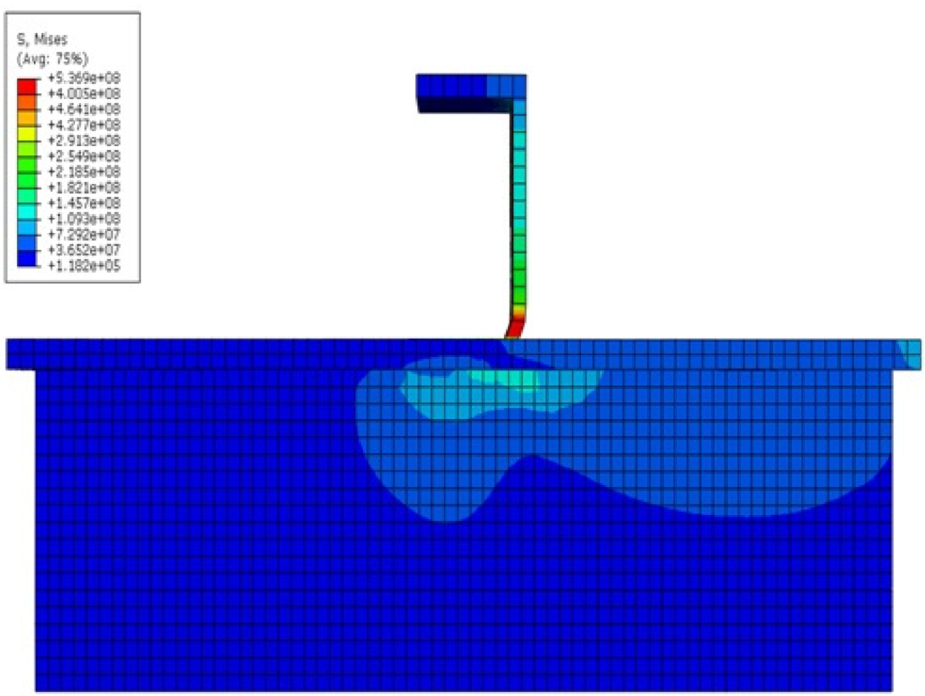


Figure 4.12. Typical Von Mises stress for specimen $H_1$10050

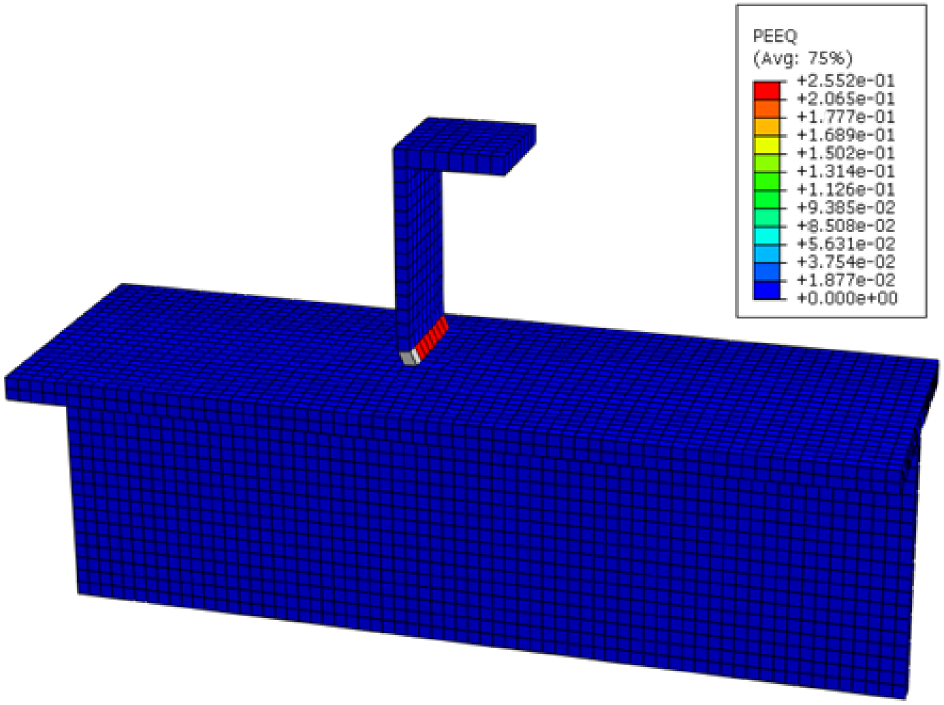


Figure 4.13. Typical plastic strain for specimen $H_1$10050 (Perspective)

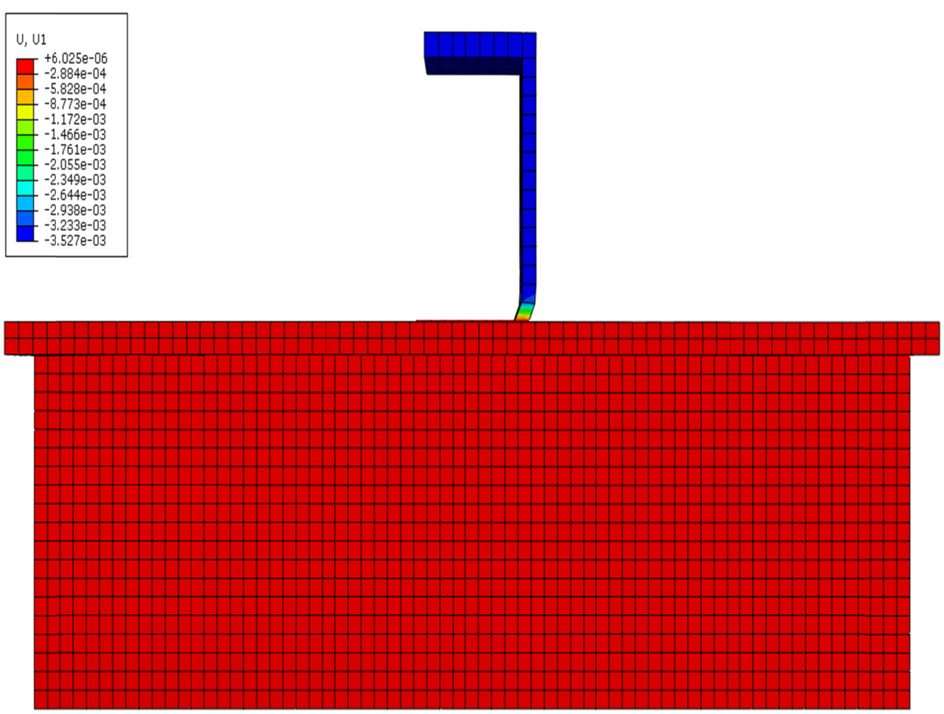


Figure 4.14. Typical plastic strain for specimen $H_1$10050

4.3.2. Parametric investigations based on finite element method

4.3.2.1. Preface

The finite element model was used to perform a parametric study by using concrete with various compressive strengths and various dimensions of connectors. After verification of the finite element results against the experimental test, the shear capacities of angle shear connectors obtained from finite element analysis were evaluated and discussed. The discussion deliberated mostly on the shear capacity alterations under monotonic loading while different dimensions of shear connector and compression strength of high strength concrete were used.

As mentioned earlier, the shear strength for the angle shear connectors embedded in reinforced concrete slab can be estimated by using Equations 4.1 by Kiyomiya and Yokota (1987) and 4.2 by Ros (2011). The results of these equations were compared with the nonlinear finite element results to check the accuracy of these formulae for the HSC.

Table 4.6 presents the results of finite element analysis for specimens with angle shear connectors embedded in high strength concrete under monotonic loading. Table 4.6 compares the ultimate strengths obtained from finite element analysis against the Equations 4.1 and 4.2 to check the accuracy of these equations as well. From the ratio achieved from the last column of the table, it can be concluded that for those specimens which have ratios of more than 1, the shear capacity estimated by the equation is conservative and for those with ratios less than 1, the capacity estimation is unconservative.

These results confirm that there is a need for more research to find a comprehensive formula for the prediction of shear capacity for angle shear connectors in composite slab.

Table 4.5. The results of finite element and experimental study for angle connectors embedded HSC.

Specimens with HSC mix	f_c (MPa)	Angle Connector Specification (mm)				Shear Capacity (kN)	
		Length	Height	Thickness		FE Analysis	Experimental
				Web	Flange		
H_1 series	82	50	75	5	7.5	150.2	152.9
		50	100	6	8.5	174.4	178.3
		50	120	7	9	189.3	-
		50	140	7	10	201.2	-
		50	160	7.5	10.5	217.5	-
H_1 series	82	30	75	5	7.5	107.0	103.7
		30	100	6	8.5	116.2	112.7
		30	120	7	9	160.8	-
		30	140	7	10	166.8	-
		30	160	7.5	10.5	188.1	-
H_2 series	80	50	75	5	7.5	145.5	145.9
		50	100	6	8.5	180.6	180.0
		50	120	7	9	188.2	-
		50	140	7	10	199.3	-
		50	160	7.5	10.5	201.1	-
H_2 series	80	30	75	5	7.5	63.1	63.7
		30	100	6	8.5	91.9	92.7
		30	120	7	9	108.9	-
		30	140	7	10	127.0	-
		30	160	7.5	10.5	143.8	-
H_3 series	63	50	75	5	7.5	131.3	130.7
		50	100	6	8.5	158.8	160.5
		50	120	7	9	167.1	-
		50	140	7	10	170.7	-
		50	160	7.5	10.5	183.8	-
H_3 series	63	30	75	5	7.5	44.9	44.8
		30	100	6	8.5	80.6	83.1
		30	120	7	9	86.9	-
		30	140	7	10	97.1	-
		30	160	7.5	10.5	105.1	-

Table 4.6. Comparison of the ultimate strengths obtained from the finite element against the equation in reinforced concrete.

Specimens with HSC mix	Shear Capacity (kN)			Comparison	
	FE Analysis	Equation (1)	Equation (2)	FE/Eq1	FE/Eq2
H_1 series	150.2	82.2	329.8	1.8	0.5
	174.4	79.4	301.1	2.2	0.6
	189.3	79.4	286.6	2.4	0.7
	201.2	73.5	243.6	2.7	0.8
	217.5	67.1	200.6	3.2	1.1
H_1 series	107.0	49.3	197.9	2.2	0.5
	116.2	47.6	180.7	2.4	0.6
	160.8	47.6	172.0	3.4	0.9
	166.8	44.1	146.2	3.8	1.1
	188.1	40.3	120.3	4.7	1.6
H_2 series	145.5	81.2	325.8	1.8	0.4
	180.6	78.4	297.4	2.3	0.6
	188.2	78.4	283.1	2.4	0.7
	199.3	72.6	240.6	2.7	0.8
	201.1	66.3	198.1	3.0	1.0
H_2 series	63.1	48.7	195.5	1.3	0.3
	91.9	47.1	178.4	2.0	0.5
	108.9	47.1	169.9	2.3	0.6
	127.0	43.6	144.4	2.9	0.9
	143.8	39.8	118.9	3.6	1.2
H_3 series	131.3	72.0	289.1	1.8	0.5
	158.8	69.6	263.9	2.3	0.6
	167.1	69.6	251.2	2.4	0.7
	170.7	64.4	213.5	2.7	0.8
	183.8	58.8	175.8	3.1	1.0
H_3 series	44.9	43.2	173.5	1.0	0.3
	80.6	41.8	158.3	1.9	0.5
	86.9	41.8	150.7	2.1	0.6
	97.1	38.7	128.1	2.5	0.8
	105.1	35.3	105.5	3.0	1.0

From the results, one may conclude that the equation recommended by Kiyomiya and Yokota (1987) (Equation 4.1) is conservative while the equation suggested by Ros (2011) (Equation 4.2) gives unconservative shear capacities. Meanwhile, it is believed that further investigations are inevitable in order to propose new equations for estimating the shear capacity of angle shear connectors.

4.3.3. Summary of finite element results

Effective numerical analyses were proposed by using the finite element method to simulate the push-out test of angle shear connectors. The finite element results were verified by using experimental test results to ensure the accuracy of the modelling. The von misses stress and plastic strains of the specimens were shown. According to the results, the finite element analysis is able to exhibit the accurate elastic and inelastic behavior of the connections.

Using the finite element model by concrete with various compressive strengths of HSC and various dimensions of connectors, a parametric study was conducted. After verification of the finite element results against the experimental test, the shear capacities of angle shear connectors obtained from finite element analysis were evaluated.

The finite element results confirmed the results of experimental tests and show that:

1. The equation recommended by Kiyomiya and Yokota (1987) (Equation 4.1) is conservative while the equation suggested by Ros (2011) (Equation 4.2) gives unconservative shear capacities.
2. The angle shear connectors are not sufficiently ductile in the peak load for the monotonic loading.
3. Higher angle connectors resist less shear once the length of the connector increases compared to lower connectors once their length increases

5 CONCLUSION

5.1. Introduction

This thesis presents the experimental and parametric investigations of push-out specimens with angle shear connectors embedded in high strength concrete in composite beams. The experimental test program involved the testing of twenty four push-out specimens. The specimens were designed to study the effect of several parameters on the shear capacity of angle shear connectors under monotonic and low cyclic fatigue loading.

The only type of failure mechanisms were observed in experimental investigations was connector fracture. The shear capacity and the ductility of angle shear connectors as main parameters in the behavior of shear connectors were discussed as well.

Design equations (Equations 4.1 and 4.2) are the general equaions for the shear capacity of angle shear connector in normal reinforced concrete. These equations were evaluated for estimation of the shear capacity of angle connectors in high strength concrete using the parametric study.

Generally, by referring to the results, angle shear connectors showed good behavior in terms of shear capacity but exhibited limited ductility behavior. The details of the results were described as follows.

5.2. Investigation of behavior of angle shear connectors

Based on Eurocode 4, the angle shear connectors are not sufficiently ductile in the peak load for the monotonic loading in HSC. These results have been confirmed by finite element results as well.

The angle connectors in high strength reinforced concrete under low cyclic fatigue loading, fractured at the first half cycle of the whole load (M) in all push-out specimens.

In all cases, the failure mode of angle shear connectors was connector fracture.

5.3. Investigation of parameters affecting the connector’s behavior

Limited ductility was observed for angle shear connectors, where a slip of less than 2.5 mm was recorded.

Higher angle connectors resist less shear compared to lower connectors once their length increases. These results have been confirmed by finite element results as well. For specimens with the higher length of angle connectors, cracks were observed at the top surface of the slab when the maximum load was applied. For angle specimens with shorter connectors in the slabs, small cracks at the top surface had a tendency to develop around the center of the slabs.

5.4. Comparison between experimental and finite element analysis

The shear capacity obtained from the finite element results for specimens with angle connector embedded in high strength concrete have a considerable difference with the results of experimental tests. The experimental test results and finite element results for channels are in perfect agreement for the specimens embedded in HSC slab.

5.5. Evaluation of design equations

From the results it was concluded that the Equation 4.1 proposed by Kiyomiya and Yokota (1987) is more conservative than the Equation 4.2 proposed by Ros (2011) for estimation of the angle shear capacity in HSC. For the specimens with high strength concrete, the strength prediction in Equation 4.2 is underestimated for the test results of all specimens. In the case of HSC, this formula is unconservative since the experimental results of angle shear capacity is less than or equal to the capacities obtained from the design equation.

5.6. Recommendations for future research

Based on research presented in this thesis, the following recommendations can be made:

1. It is believed full scale tests for investigation of the behavior of angle shear connectors in composite beam can complete the investigation. Experimental and parametric studies using finite element analysis of full scale test for angle shear connectors are suggested.
2. Since it was understood that both the available equations for estimation of shear capacity for angle shear connectors (Equations 4.1 and 4.2) are not accurate, it is recommended that further research on angle connectors with different dimensions should be carried out in order to develop a numerical equation which would be able to predict the shear capacity of the these connectors with different dimensions.

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AWARDS

1. Silver Medal (2012)

Mahdi Shariati, N.H. Ramli Sulong, Meldi Suhatril, Ali Shariati, "Innovative Angle-Shape Shear Connector in Steel-Concrete Composite Beam", International Trade Fair, iENA 2012, Nuremberg, Germany, International Federation of Inventors Associations, 2012.

LIST OF PUBLICATION

Shariati, A., Shariati, M.; Ramli Sulong, NH; Suhatril, M.; Arabnejad Khanouki, M.M.; Mahoutian, M. (2014) Experimental assessment of angle shear connectors under monotonic and fully reversed cyclic loading in high strength concrete. *Construction and Building Materials 52 276-283 Elsevier*.

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1. **Shariati, A.**, Ramli Sulong, N.H. Behavior of steel-concrete composite beam using shear connector under fully reversed cyclic loading, IPPP grant no PV083-2012A, *University of Malaya, Kuala Lumpur, Malaysia.*

APPENDIX A

SPECIMENS PREPARATION AND TEST PROCEDURES

This appendix presents the preparation of specimens for the test and the procedure of push-out test on angle shear connectros embedded in high strength concrete.

Figure. A.1. Concrete preparation

Figure. A.2. Slump test for concrete

Figure. A.3. Preparation of cylindrical and cubic specimens

Figure. A.4. Modulus elasticity test

Figure. A.5. Typical specimens ready for concrete casting

Figure. A.6. Specimens ready for push-out test

Figure. A.7. Settelment of special support on specimen's slabs

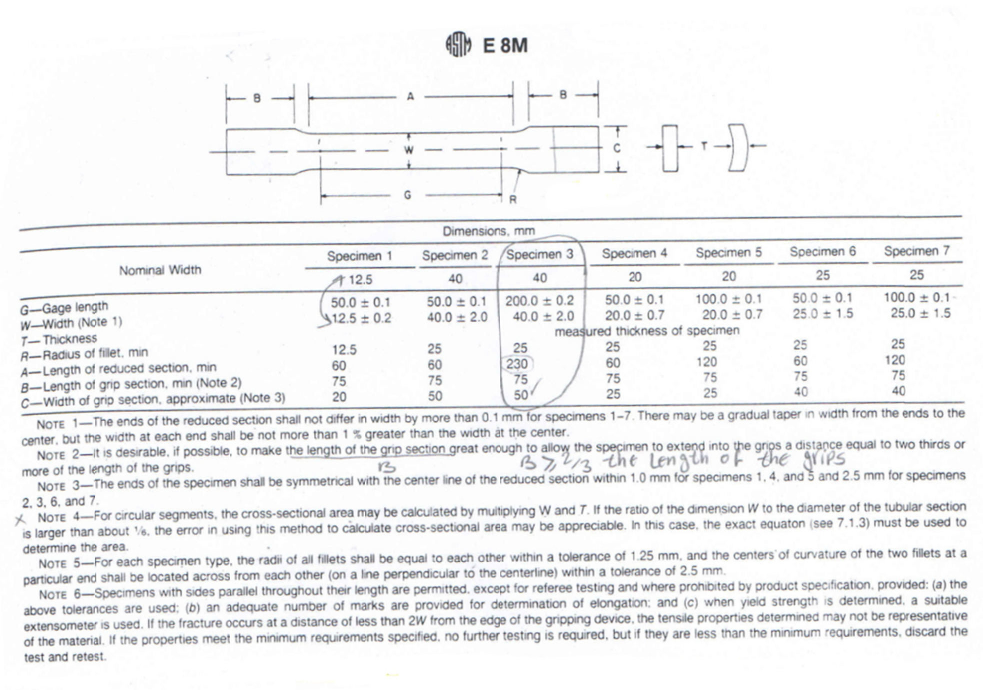
Figure. A.8. Specimen ready for loading

Figure. A.9. Typical specimens after fracture

APPENDIX B

STEEL COUPON TEST

This appendix presents the steel coupon testing of steel component in this study. ASTM 370 specification was used in preparing and testing of the specimens. Three specimens for each test were tested from flange. and web of channel and angle steel profiles Following figures and tables shows the procedure of the steel coupon testing.



E 8M

Nominal Width	Specimen 1	Specimen 2	Specimen 3	Specimen 4	Specimen 5	Specimen 6	Specimen 7
	12.5	40	40	20	20	25	25
G—Gage length	50.0 ± 0.1	50.0 ± 0.1	200.0 ± 0.2	50.0 ± 0.1	100.0 ± 0.1	50.0 ± 0.1	100.0 ± 0.1
W—Width (Note 1)	12.5 ± 0.2	40.0 ± 2.0	40.0 ± 2.0	20.0 ± 0.7	20.0 ± 0.7	25.0 ± 1.5	25.0 ± 1.5
T—Thickness	measured thickness of specimen						
R—Radius of fillet, min	12.5	25	25	25	25	25	25
A—Length of reduced section, min	60	60	230	60	120	60	120
B—Length of grip section, min (Note 2)	75	75	75	75	75	75	75
C—Width of grip section, approximate (Note 3)	20	50	50	25	25	40	40

Dimensions, mm

NOTE 1—The ends of the reduced section shall not differ in width by more than 0.1 mm for specimens 1–7. There may be a gradual taper in width from the ends to the center, but the width at each end shall be not more than 1 % greater than the width at the center.

NOTE 2—It is desirable, if possible, to make the length of the grip section great enough to allow the specimen to extend into the grips a distance equal to two thirds or more of the length of the grips.

B   B ≥ 2/3 the length of the grips

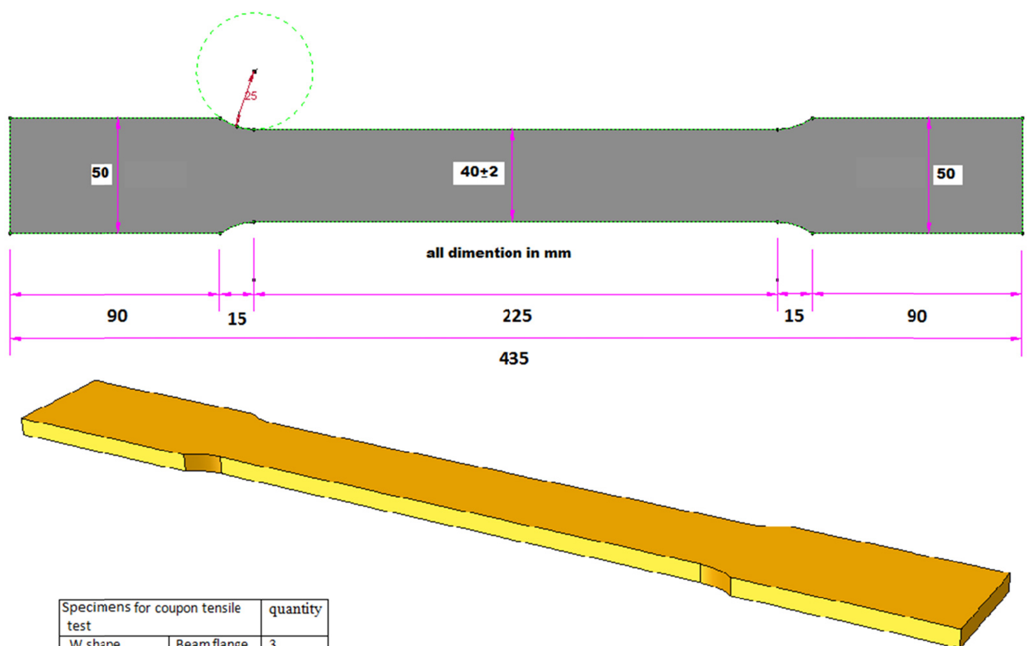
NOTE 3—The ends of the specimen shall be symmetrical with the center line of the reduced section within 1.0 mm for specimens 1, 4, and 5 and 2.5 mm for specimens 2, 3, 6, and 7.

NOTE 4—For circular segments, the cross-sectional area may be calculated by multiplying W and T. If the ratio of the dimension W to the diameter of the tubular section is larger than about 1/6, the error in using this method to calculate cross-sectional area may be appreciable. In this case, the exact equaton (see 7.1.3) must be used to determine the area.

NOTE 5—For each specimen type, the radii of all fillets shall be equal to each other within a tolerance of 1.25 mm, and the centers of curvature of the two fillets at a particular end shall be located across from each other (on a line perpendicular to the centerline) within a tolerance of 2.5 mm.

NOTE 6—Specimens with sides parallel throughout their length are permitted, except for referee testing and where prohibited by product specification, provided: (a) the above tolerances are used; (b) an adequate number of marks are provided for determination of elongation; and (c) when yield strength is determined, a suitable extensometer is used. If the fracture occurs at a distance of less than 2W from the edge of the gripping device, the tensile properties determined may not be representative of the material. If the properties meet the minimum requirements specified, no further testing is required, but if they are less than the minimum requirements, discard the test and retest.

Figure. B.1. Standard method according to ASTM



Specimens for coupon tensile test		quantity
W shape 406*140	Beam flange	3
	Beam web	3
W shape 353*127	Beam flange	3
	Beam web	3
Column plate	-	3

Figure. B.2. Specimen’s dimension

Figure. B.3. Universal testing machine

Figure. B.4. Universal testing machine specification

Figure. B.5. Coupon test samples before tensile testing

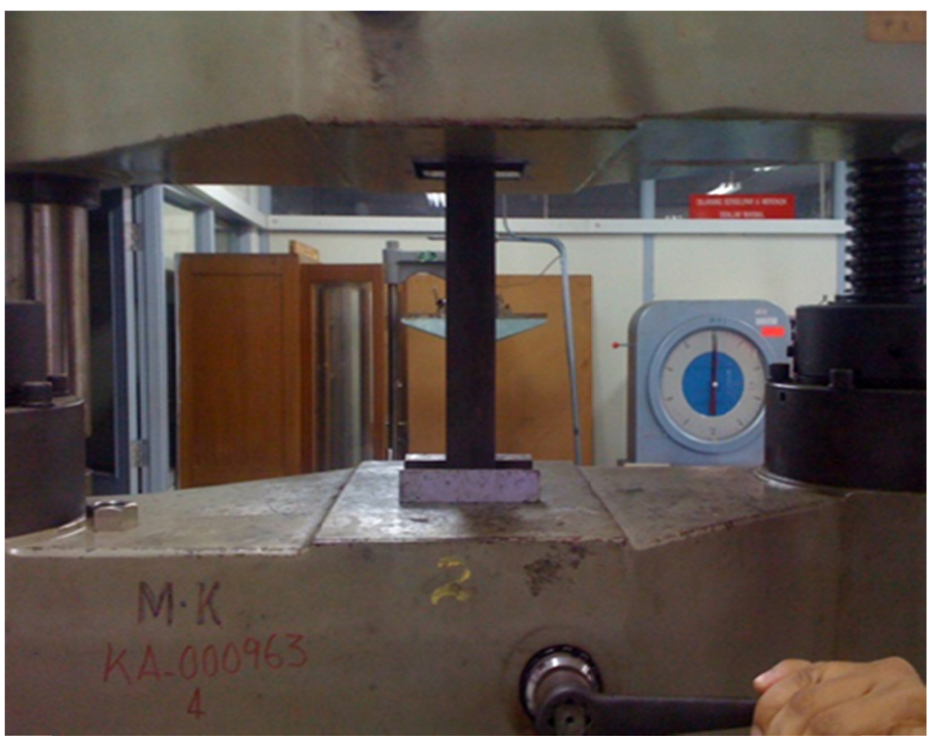


Figure. B.6. Specimen's installation for tensile testing

Figure. B.6. Coupon test samples after tensile testing

Table B.1. Coupon tensile test result

Specimens	1	2	3
F_y (MPa)	338.8	341.85	340.00
F_u (MPa)	493.86	495.03	495.77
Elongation (%)	24.23	23.74	24.16
F_y Average (MPa)	340.22		
F_u Average (MPa)	494.89		
Elongation Average (%)	24.04		
$\frac{F_u}{F_y}$	1.45		