EXPERIMENTAL STUDY OF BEHAVIOUR AND STRENGTH OF SHEAR STUDS IN COMPOSITE BRIDGE DECK CONSTRUCTION

by

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Signature of Author

To my parents and beloved wife

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ABSTRACT

Cast-in-place concrete in composite with steel sections is commonly used in bridge deck constructions. The shear transfer between the concrete and steel section is achieved by shear connectors and the strength calculation of conventional shear connectors, i.e. shear studs, is provided in various design codes in North America. Due to the fact that the strength equation is largely based on experimental results, the applicability of the equation is only warranted where the design matches the experimental configuration of the test specimens. Thus, the codes specify detailing requirement for the stud height and the elevation of the reinforcement mesh in relation to the stud height. However, these requirements, in particular, the elevation of the reinforcement mesh, may be difficult to meet accurately in construction practice. The implications of not meeting the mesh requirement to the strength of the shear stud and the remedy solutions are examined in this study.

An experimental program involving the test of thirty-three push-out specimens was designed and conducted with a focus on the shear studs' performance. Testing parameters included reinforcement mesh position, shear stud height, presence of stud head, shear stud spacing, and steel flange surface treatment. In addition, the performance of a new type of shear studs, referred to as adjustable studs, was also studied experimentally. The ultimate load and load vs. slip curves were presented and discussed in the forms of tables and graphs. The failure modes were noted and the relationship between the failure modes and the ultimate capacity was discussed. Ultimate loads obtained from specimens were then used to assess the efficacy of code suggested values.

Results showed that depending on the elevation of reinforcement mesh, three failure modes were observed including concrete related failure, combined concrete failure and bent studs and stud shear-off from the steel flange. The elevation of the reinforcement mesh had a significant effect on the ultimate load of the specimen. As the mesh elevation increased from intercepting the stud to being in flush with the top of the stud to above the stud, the ultimate load decreased. Specimens with unheaded shear studs had lower ultimate load than specimens with headed shear studs. Flange treatment had an impact on the ultimate load, where the coating on flanges resulted in a decrease in the ultimate load. Test results also showed that the close placement of the shear studs result in a reduction on the ultimate load when the other parameters were kept the same. In the comparison between conventional and adjustable shear studs, specimens with adjustable studs shared similar failure mode to those with conventional studs, but attained on average lower load capacity. The comparison with the code suggested values showed that the code suggested value is only ensured when double-layer reinforcement mesh is used and placed at code specified elevation. A single layer mesh intercepting the studs resulted in the ultimate load slightly lower than the code value. The code values for adjustable studs are markedly higher than the experimental value, which raises the question whether the code equation for conventional studs is directly transferrable to adjustable studs.

LIST OF ABBREVIATIONS AND SYMBOLS USED

ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
AISC-LRFD	American Institute of Steel Construction Load and Resistance Factor Design
ASTM	American Society for Testing and Materials
CSA-S06	Canadian Standards Association Bridge and Highway Design Code
CSA-S16	Canadian Standards Association Limit State Design of Steel Structures
HSC	High strength concrete
LVDT	Linear Variable Differential Transducer
NS-TIR	Nova Scotia Department of Transportation and Infrastructure Renewal
NSC	Normal strength concrete

SYMBOLS

$\boldsymbol{\lambda}$	Constant
$\boldsymbol{\phi_{sc}}$	Performance factor and is taken as 0.8
$\mathbf{A_{sc}}$	Area of the shank of the stud in mm^2
$\mathbf{d}$	Diameter of the shank of the stud in mm
$\mathbf{E_c}$	Modulus of Elasticity of concrete in MPa
$\mathbf{E_s}$	Modulus of Elasticity of steel in MPa
f'_c	Concrete compressive strength in MPa
$\mathbf{F_u}$	Specified minimum tensile strength of the stud, specified as 450 MPa in the Canadian bridge code
$\mathbf{h}$	Height of shear stud in mm
$\mathbf{h_a}$	Height of stiff connector or effective height of flexible connector and equal to 1.8 times the diameter of a shear stud in mm
$\mathbf{h_r}$	Elevation of reinforcement in the concrete slab in mm
$\mathbf{K}$	Constant
$\mathbf{n_r}$	Number of connectors that can be assumed to fail as a group
$\mathbf{P_u}$	Ultimate load of the specimen in kN
$\mathbf{s}$	Average slip obtained at the ultimate load in mm
$\mathbf{t_f}$	Thickness of the flange in mm

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CHAPTER 1 INTRODUCTION

1.1 Background of Composite Construction

Commonly used in bridge and building construction, the composite structure is a type of construction where a concrete slab and steel section act together to resist load. In building construction composite slabs often consist of a thin concrete slab poured over corrugated or ribbed steel sheets used as formwork, resting on top of steel beams. The first composite slab with a ribbed deck was used in France in 1962 and was called the Robinson slab (Ahn, 2009). In the bridge construction application, composite sections are used where a solid slab is poured on top of formwork rather than corrugated sheets. However, bridges with corrugated steel sheet are present but are not common. Steel girders in composite construction are lighter and shallower than non-composite construction (Badie et al., 2002). In both construction methods, the transfer of the forces between concrete and steel sections is achieved by shear connectors. Welded on top of the steel beams and embedded into the concrete, these connectors are intended to prevent horizontal movement and separation between the two materials which allows them to act as one unit. End welded shear studs are the most commonly used and researched shear connectors, although many other types of connectors have been proposed and used in various applications as well. Shear studs are used for their convenient construction and non-directional behavior (Wang et al. 2011).

1.2 Behaviour of Shear Studs

The capacity and failure mechanism of shear connectors is often studied by means of push-out tests. A typical push-out specimen consists of a steel section with shear connectors on both flanges embedded in concrete slabs. The web of the steel section is loaded until the specimen fails. The ultimate load obtained is then divided by the number of shear connectors to calculate the strength of each connector (Driscol and Slutter, 1961). The load is assumed to be transmitted from the steel section to the slabs only through the studs, and the load is distributed evenly between the studs (Viest, 1956). This method was first adopted in the 1930's in Switzerland to test the shear capacity of spiral shear connectors (Davies, 1967). The strength calculation of conventional shear studs is provided in various design codes in North America. Research has shown that the reinforcement mesh in the slab confines the concrete around the stud and thus increases the concrete strength. Although this effect is not quantified in the design guidelines, the codes do specify the detailing requirement for the stud height and the elevation of the reinforcement mesh in relation to the stud height. However, these requirements, in particular the elevation of the reinforcement mesh, may be difficult to meet accurately in construction practice. While considerable research has been conducted in the general area of composite section and shear connectors, limited scientific information is available in reported literature on the effect of not meeting the detailing requirement on the strength of the shear stud and no provisions are provided in the current design codes to address the related issues.

1.3 Research Objectives

The main objective of this research is to examine the effect of reinforcement mesh detailing requirement on the strength of the shear stud and the potential remedy solutions. Several parameters are considered in the design of specimens. The detailed objectives of this research are listed as follows:

1. To conduct an extensive literature review on the research on the topic of shear stud strength in general and that related to the effect of detailing requirement on its strength.

2. To conduct push-out tests to study the performance of conventional shear studs with the focus on the effect of varying elevations of reinforcement mesh. Other parameters including stud height, stud spacing, the presence of stud head, and the steel flange treatment are considered.

3. To examine the performance of adjustable studs for its potential application in the remedy solution.

4. To assess the validity of the Canadian code suggested stud strength equations for both stud types.

1.4 Scope of Research

A literature review is presented in Chapter 2, which includes the description of constructional issues pertaining to the problem, previous research conducted by others most relevant to this research, and the strength equation and detailing requirement specified in various codes. A detailed description of the experimental setup and specimens is presented in Chapter 3. Chapter 4 presents test results, discussion, and analysis of push-out specimens and auxiliary tests as well as comparison between the experimental and code values. The summary and conclusion of the research are presented in Chapter 5.

CHAPTER 2 LITERATURE REVIEW

2.1 Design Practice and Research Issues

Considerable research has been conducted in the behaviour of composite sections and shear studs in the past 6 decades and the design of them, as a result, has been well established in the various codes and standards. The current Canadian Bridge and Highway Design Code CSA-S06 (2006) and its American counterpart AASHTO (2005) governs the design of composite sections in their specific application in bridge construction whereas the Canadian Steel Design Code CSA-S16 (2010) and AISC LRFD Specifications for Steel Buildings (2005) provide the design guidelines for their application in building construction. While the design principles are the same, different codes may have different detailing requirements specific to the intended application. The following section reviews the design requirements of the above mentioned codes and discusses the issues relevant to this research.

The Canadian steel design code CSA-S16, CSA-S06, and AASHTO state the minimum height to diameter ratio of installed shear studs to be 4 as expressed in the following:

$$h/d \geq 4 \tag{2.1}$$

where h is the height of the stud, and d is the diameter of the shank of the stud. This ratio should be the minimum limiting factor for studs applied in bridges and buildings. Based on the work by Ollgaard et al. (1971), Eqn (2.2) is currently used in the CSA-S16, the CSA-S06, and AASHTO to estimate the shear capacity of headed studs in a composite slab:

$$q_r = 0.50\phi_{sc}A_{sc}\sqrt{f_c'E_c} \leq \phi_{sc}F_uA_{sc} \tag{2.2}$$

where A_{sc} is the area of the shank of the stud in mm^2, f'_c is the concrete compressive strength in MPa, and E_c is the Modulus of Elasticity of concrete in MPa, and F_u is the specified minimum tensile strength of the stud, specified as 450 MPa in the Canadian bridge code. ϕ_{sc} is a performance factor and is taken as 0.8. The left-hand side term to the inequality estimates the shear stud strength as affected by the compressive strength and modulus of elasticity of concrete whereas the right-hand side represents the stud strength governed by the tensile strength of stud as the stud bends over and finally fails in tension (Jayas and Husain, 1988).

Since Eqn (2.2) was derived based on test results, the strength is, strictly speaking, only warranted where the design matches the experimental configuration of the test specimens on which the equation is based. The design of the slab itself usually results in two layers of steel reinforcement mesh for both capacity and shrinkage control and the codes provide detailing requirement on the placement of studs with respect to the reinforcement mesh. The detailing requirement by CSA-S06 clause 8.11.2.2 is shown in Figure 2.1. The minimum cover to the bottom and top reinforcement is 50 mm and 70 mm respectively. Furthermore, CSA-S06 clause 10.11.8.2 contains requirements for placement of the slab reinforcement relative to the head of the shear stud and shear stud spacing. The clear distance between the head of the stud to the bottom transverse reinforcement should be at least 25 mm. CSA-S16 and AASHTO both limit a minimum distance of 6 stud diameters but CSA-S06 specifies 4 stud diameters. CSA-S16 clause 17.7.2.4 specifies a maximum 1000 mm longitudinal distance whereas AASHTO clause 6.10.10.1.2 and CSA-S06 clause 10.11.8.3.1 specifies a maximum of 600 mm. It is desirable to fit the shear stud in between the two layers of mesh. For a typical 200 mm thick concrete slab, the shear studs need to be placed within an 80 mm space. This is a small and space and it is not surprising that in many cases, the studs may not be at the specified elevation, being either below the bottom layer of the mesh or above the top layer of the mesh.

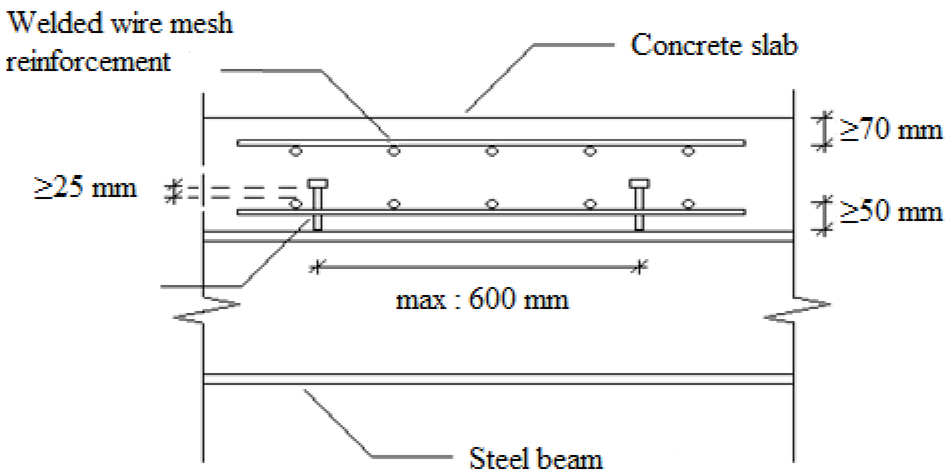


Figure 2.1 Detailing requirement for composite bridge decks

Other design equations have been proposed including the equation proposed by Oehlers and Johnson (1987) that takes into account the interaction of the concrete and stud as expressed in Eqn (2.3).

$$q_r = KF_u A_{sc} \left[\frac{E_c}{E_s}\right]^{0.40} \left[\frac{f_c'}{F_u}\right]^{0.35} \tag{2.3 a}$$

$$K = 4.3 - 1.1 n_r^{-0.5} \tag{2.3 b}$$

where K is a constant and can be calculated using Eqn (2.3 b), E_s is the Modulus of Elasticity of the steel in MPa, and n_r is the number of shear connectors that can be assumed to fail as a group (have similar displacements as in a shear span). When the number of shear connector to fail reaches infinity the K factor is 4.3, and is 3.2 for the characteristics of a single connector. Xue et al. (2008) modified Eqn (2.3) to include the effect of stud heights and the equation is expressed below.

$$q_r = 3\lambda F_u A_{sc}\left[\frac{E_c}{E_s}\right]^{0.40}\left[\frac{f_c'}{F_u}\right]^{0.2} \tag{2.4 a}$$

$$\lambda = \begin{cases} 6 - \dfrac{h}{1.05d} & (h/d \leq 5) \\ 1 & 5<h/d<7 \\ \dfrac{h}{d} - 6 & h/d \geq 7 \end{cases} \tag{2.4 b}$$

where λ is a factor dependent on the stud height. The remaining terms are as defined before.

Many factors may result in the studs being not at the specified height. Cambering of the steel beam may be one. According to CSA-S6 any beam that spans longer than 25 m shall at least be cambered against dead load deflections. Some errors in the cambering could arise from poorly drawn shop drawings, inaccurate calculations or improper cambering methods. The beams may not deflect as calculated by the designer where the steel could be more or less stiff than the value used in calculations. Another potential reason is misalignment of formwork for concrete. The piers, supporting the beams may not be aligned at the same height and although haunches (see Figure 2.2) in the concrete deck are used to keep the concrete deck level, errors in construction may still occur where the formwork may be placed too high or too low causing the slab to be either elevated or lowered.

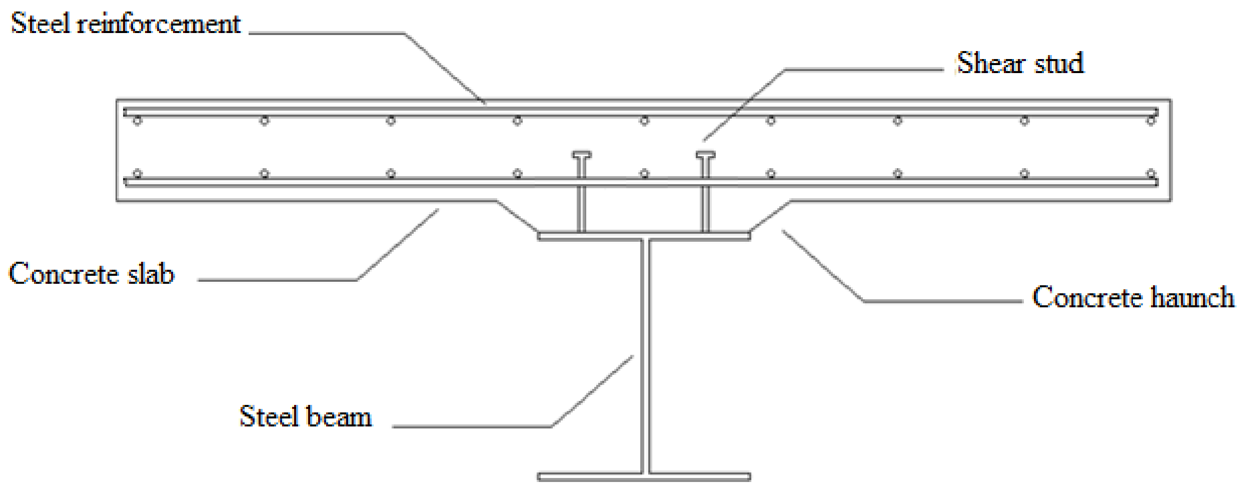


Figure 2.2 Transverse view of haunches in concrete slabs

The concrete pour sequence could also result in misalignment of steel mesh in relation to the stud height. The concrete deck is often poured in sections where the ends are poured first. The formwork setting may cause the slab to be elevated or lowered in some sections. Some metal fabricators base the stud height on the screed elevations of the bridge by anticipating the differential settlement and deflection of the bridge deck from the concrete, results in the uneven stud heights along the length of the bridge on the fabricated steel beam. The shear studs may be shorter at two ends and longer in the middle to also accommodate the cambering of the steel beam.

In practice, if the shear stud is found to be at the wrong elevation through the site inspection, it will be removed by cutting through the bottom of the shank and another stud with the correct height is welded instead with a special welding machine. For studs higher than the top layer of reinforcement mesh, some length of the stud will be cut off to achieve the desired height but the head of stud will be lost. These processes are both labour intensive and time consuming. One potential remedy to the problem is the adjustable stud which is essentially a threaded bar with a nut as shown in Figure 2.3. This stud can be welded to the flange of steel section in the same fashion as conventional steel studs and the nut can be twisted to the required stud height and the excess length can be cut-off or grinded-off if required. This system offers the flexibility of adjusting the height and also preserves the head of the stud. But this adjustable system is more expensive than a conventional stud. Considering that a bridge has hundreds of studs, use of adjustable studs will result in a marked increase in costs. Alternatively, they could be used in the last section of the steel beam where the screed elevations are not very accurate.

Figure 2.3 Proposed adjustable stud system

There is little technical information available in the literature on the strength and behaviour of the adjustable studs and whether they are comparable to the conventional studs in performance.

Having errors in shear stud heights is not the only concern steel fabricators have. Steel beams have no specified shelf life in the bridge code and fabricators tend to ship the beams to the job site as fast as they can. However, in some cases, beams can be left lying in the fabricators field for months before being shipped to the job site. Depending on the weather, by the time it is delivered, the beams most likely have developed varying degrees of rust. According to Nova Scotia Transportation and Infrastructure Renewal (NS-TIR) specifications, the top flange of the beam used in composite construction must not be coated. But some fabricators want to avoid rust issues by applying a protective coating on the flange. The effect of the coating on the shear transfer of the composite section is not fully examined.

The following sections provide a review on previous studies of shear connectors that is deemed to be most relevant to this study.

2.2 Research Background

Viest (1956) tested 12 push-out specimens with varying stud diameters to determine if round headed studs could be used as shear connectors. Specimens also included shear studs at different spacing by increasing the number of studs in the same size steel section. Inelastic deformations in the concrete and yielding of the steel were observed. Test results also showed that the load capacity of shear studs at constant slip increased with increasing strength of concrete and that increase was approximately proportional to $\sqrt{f_c'}$. In addition, an increase in the stud diameter resulted in an increase in the stud load capacity.

Driscol and Slutter (1961) examined the effect of stud height on shear strength of studs. They tested push-out specimens with stud diameters varying from 12.7 mm (0.5 inch) to 19.1 mm (0.75 inch). It was found that when the ratio of the stud height-to-diameter was below 4.2, the concrete failed before the stud sheared-off. When the ratio was greater than 4.2, failure was by stud pulling out. Concrete failure resulted in a greater reduction of the connector strength and should be avoided. They found that the shear connectors can be spaced evenly and do not have to follow the shear diagram of a member in distribution.

Davies (1967) tested 20 push-out specimens with varying spacing and arrangements of shear studs. All specimens had the same 9.525 mm (3/8 in) diameter and 50.8 mm height. Shear stud spacing ranged from 88.9 mm to 6.35 mm. Specimens contained two shear studs that are at right angle to the load, a series shear studs parallel to the line of loading, and a cluster of shear studs arranged as the shape of a square. Test results showed that specimens with two shear studs perpendicular to the line of loading had the highest shear strength of all specimens, and gave 25% more shear strength than specimens with parallel arrangements. He found that when there are three studs per flange arranged longitudinally, the strength per stud is more sensitive to variations in spacing than when there are only two per flange. Adding a second row of shear studs in close proximity (38.1 mm to 6.35 mm) to the other studs reduces the shear strength per stud by almost 70%, than having a single row of shear studs at right angle to the line of loading.

Gobble (1968) tested the strength of shear connectors on beams of varying flange thickness. He tested 41 push-out specimens containing studs of diameter 12.7 mm (1/2 inch), 15.9 mm (5/8 inch), 19.1 mm (3/4 inch) and flange thickness varying from 3.25 mm (0.128 inches) to 11.23 mm (0.442 inches). He found that in order for the stud to fail by shear-off at the connection the following ratio should be attained:

$$d_s / t_f \leq 2.7 \tag{2.5}$$

where d_s is the diameter of the stud and t_f is the thickness of the flange. If the ratio is greater than stated the connection will fail by shear stud pull-out ripping a piece of the flange with it.

Davies (1969) examined 7 simply supported composite beams on the effect of connector spacing and amount of transverse reinforcement. Specimens had connectors spacing ranging from 95.3 mm (3.75 in) to 38.1 mm (1.5 in), an equivalent of 10 times to 4 times diameter of stud. Beam spans were reduced from 3.05 m (10 ft) to 1.2 m (4 ft) as the spacing decreased. All beams had identical 32 shear studs placed in one line in the middle of the steel flange. He found that connector spacing had little or no effect on deflection, steel and concrete strains, or ultimate moment resistance. However, the decrease in stud spacing increased the amount of slip but was questionable if the loss of interaction affects the behaviour of the beam as a whole. The amount of transverse wire mesh in the slab was varied from 0.118 to 0.94% of the slab cross sectional area. Results showed that a beam with 0% transverse reinforcement will only achieved 50 to 60% of the ultimate load in comparison to a beam with proper reinforcement as described by the author. He found that adequate reinforcement adds to the ultimate strength of a beam by preventing longitudinal cracking along the line of shear connectors. He also found that 0.5% of transverse reinforcement is the minimum amount that should be required in composite slabs, and 1.0%. is the maximum. When transverse reinforcement was less than 0.5%, the ultimate moment would be very sensitive to changes in reinforcement amounts while the amount of reinforcement higher than 1.0% had little effect to increase the ultimate moment. However, the minimum amount to prevent longitudinal concrete cracking was 0.82%.

Johnson (1970) investigated the effect of amount of transverse reinforcement on shear capacity of composite slabs. He concluded that regardless of the elevation of the reinforcement mesh, it resulted in an increase in the longitudinal shear capacity of the slab.

Ollgaard et al. (1971) tested 48 push-out specimens containing light-weight and normal-weight concrete. All specimens were made with a solid slab with same reinforcement of top and bottom mesh, of # 4 bars (129 mm^2) parallel to the line of loading and #5 (200 mm^2) bars perpendicular. The variables they tested were concrete properties, stud diameters, and number of connectors per slab. Failure modes included studs sheared-off but stayed imbedded in the concrete or the concrete failed in the areas around the studs. They found that the shear capacity of a stud was almost proportional to its cross-sectional area. The following equation was generated from the least square fit of the test results to estimate the stud strength:

$$q_r = 1.106 A_{sc} f_c'^{0.3} E_c^{\ 0.44} \tag{2.6}$$

where A_{sc} is the area of the shank of the stud, f'_c is the concrete compressive strength, and E_c is the modulus of elasticity of concrete. This equation formed the basis for stud strength equation (Eqn (2.2)) used in the North American codes.

Dorton et al. (1977) investigated the use of high strength bolts fitted in the predrilled holes on the steel flange as shear connectors. High strength bolts with 22.225 mm diameter were used as the shear connectors with oversized holes in the flanges to incorporate concrete slab shrinking without stressing the steelwork. The authors tested push-out specimens with an H-pile section containing 2 bolts in each flange. The bolts failed in shear at an average load of 205 kN each with a concrete strength of 29.7 MPa. At the tested concrete strength the calculated ultimate strength of shear studs is 167 kN using CSA-S06 equations. The tested ultimate load was 22.8% higher than the estimated ultimate load. Authors concluded that bolts can replace shear studs of that size.

Jayas and Husain (1988) examined the validity of Eqn (2.2) for ribbed slabs. Jayas and Husain analysed 18 push-out specimens containing solid slabs and ribbed slabs with ribs both parallel and perpendicular to the steel section. One layer of reinforcement mesh was used in each slab with 10 M parallel and 15 M perpendicular to the line of loading. It was found that Eqn (2.2) accurately calculated the shear strength of the connectors in solid and parallel ribbed slabs when failure occurred due to stud shear-off when studs are spaced more than six-times the diameter of stud apart. However, the code overestimated the strength in perpendicular ribbed slabs. With slabs that have longitudinal stud spacing of less than six-times the diameter of the stud concrete failed before the studs were sheared-off which resulted in a decrease of 7% and 14% in shear strength in solid and parallel slabs respectively being compared to specimens with spacing of six-times the stud diameter.

Lloyd and Wright (1990) conducted push-out tests on specimens with profiled steel sheets and headed shear connectors to study the effects of varying the reinforcement position and the slab dimensions. The authors found that increasing the slab width in a tested specimen had little effect on the on the ultimate load of shear connectors. The authors observed a similar failure cone around the shear studs in all specimens with varying reinforcement positions, from placed right above the profile sheeting, to being placed flush with the head of the stud, to being placed above the head of the stud. They concluded that the variations in quantity and position of the reinforcement had little effect on the connection strength.

Oehlers and Park (1992) tested 25 push-out specimens to test the effect of transverse reinforcement on the ultimate strength of shear connectors in longitudinally cracked slabs. They varied the reinforcement elevation from 25 mm from the surface of the flange to 95 mm, in a 130 mm thick concrete slab. They also varied the amount of transverse reinforcement in the slab by increasing the diameter of the bars. Some specimens contained a double line of shear studs at a spacing of 67 mm while others had one line of studs at a spacing of 100 mm. All specimens had 100 mm long shear studs with a 19 mm diameter. The authors stated that an increase in the yield strength of the reinforcement by up to 56% had no effect on the stud ultimate load. Test data showed that the double line shear connector arrangements had a decrease of 20% in ultimate load when compared to a specimen with a single line of shear studs. After examination of specimens with elevating the reinforcement in the slab the authors found that for the reinforcement to have effective confinement around the stud, it must abide by the following equation:

$$h_r \leq 1.7 h_a \tag{2.7}$$

where h_r is the elevation of the reinforcement in the concrete slab in mm and h_a is the height of stiff connector, or effective height of flexible connector, which is equal to 1.8 times the diameter of a shear stud in mm. They stated that the reinforcement adds to the concrete confinement around the stud rather than contribute to strength.

An and Cederwall (1996) researched the effect of concrete strength on strength of shear studs. They tested 8 push-out tests with concrete strengths varying between 30-40 MPa for normal strength, and between 80-100 MPa in high strength concrete. Reinforcement varied between a single and double-layer both under the head of the stud. In normal strength concrete specimens, concrete failure predominated while in high strength concrete specimens, failure was more due to fracture of the shear studs. It was observed that the transverse bars in the bottom layer of reinforcement had the highest strain in both layers of reinforcement. This was true for all strengths of concrete. This supports the theory that the reinforcement mesh helps confine the concrete around the studs to increase its capacity. For normal strength concrete, specimens with double layers of reinforcement were able to withstand an average 6% more loads and had higher slip than specimens with a single layer of reinforcement. However, having two layers of reinforcement did not add strength over specimens with a single layer of reinforcement in high strength specimens.

Badie et al. (2002) carried out tests on large size studs in bridge decks. They used 31.8 mm (10/8 inch) studs as opposed to the commonly used 19.1 mm (3/4 inch) and 22.2 mm (7/8 inch) studs. Specimens included specimens with double rows of 7/8 inch studs to be compared with a single row of 10/8 inch studs, and specimens with alternating headed and headless studs along the length of the flange. The authors reported that almost all the studs failed at their tensile capacity as demonstrated by Eqn 2.2 (AASHTO LRFD 1998). The 10/8 inch studs demonstrated a load capacity twice that of the 7/8 inch studs. The larger studs also showed 30% less slippage at ultimate load than the smaller studs, thus creating a more rigid structure. The authors found that replacing half the headed studs with headless studs reduced the shear capacity by 17%.

In bridges composed of steel beams and precast concrete slabs, the shear studs are often arranged in a group arrangement to fit in the holes made in the precast concrete slab which will be grouted to make a fixed connection between the steel and concrete. Okada et al. (2006) tested the effect of group stud arrangements in precast slabs. A number of specimens had transverse reinforcement run in between the studs to be compared with specimens that did not have any reinforcement between the studs as shown in Figure 2.4. They found that the influence on the strength between the grouped arrangement and the normal arrangement, is small enough to be negligible, but if the concrete strength was low the influence will be greater. Slip in grouped arrangements was 25% lower than normal arrangement. No difference was observed in the results between specimens with and without transverse reinforcement between the studs, which results in ease of construction.

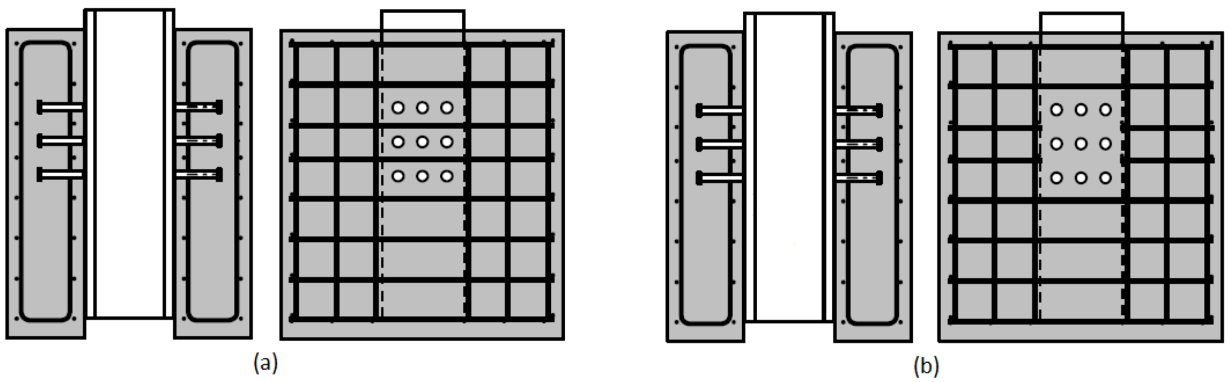


Figure 2.4 Test specimens: (a) Specimens with transverse reinforcement between shear studs. (b) Specimens without transverse reinforcement between shear studs (adapted from Okada et al. 2006)

Xue et al. (2008) investigated the effects of stud diameter and height, concrete strength, and the amount of transverse reinforcement that was varied by increasing the layers of reinforcement in the slab from two to four having a concrete cover of 15 mm. They stated that push-out test results provided lower shear transfer capacity than that obtained in beam test result; and that transverse reinforcement had a negligible effect when using high strength concrete but more effect when using normal strength concrete. The results indicated the behaviour of shear studs in bridges is affected by concrete strength and amount of transverse reinforcement. An increase in the stud diameter resulted in an increase in the ultimate shear capacity but they had a negligible effect on the load-slip curve shape. An increase in the stud height decreased its ultimate strength by 16.3% in 13 mm studs. The maximum load of specimens increased when the amount of transverse reinforcement increased especially for studs with larger diameters.

Smith and Couchman (2010) examined the effect of reinforcement mesh position in the ribbed slab on shear strength. They tested 27 push-out specimens having both top and bottom reinforcement, where the top reinforcement was placed at minimum cover (70 mm) from the top, and the bottom reinforcement was placed on top of the profile sheeting. They noticed that placing the mesh on top of the ribbed deck increases the shear strength of studs by 30% than placing the mesh in the top layer. When the mesh placed on top of the profile sheeting intercepting the stud, failure was by the concrete pull-out in a shape of a cone below the head of the stud.

Wang et al. (2011) tested 12 push-out specimens to study the effect of increasing the stud diameter and its tensile capacity in specimens on their ultimate strength. The authors examined studs with 200 mm heights, and diameters of 22 mm, 25 mm, and 30 mm. The studs had tensile strength ranging from 430 MPa to 675 MPa. All specimens had top and bottom layers of reinforcement with 50 mm concrete cover. Specimens were tested at concrete strength of 70.3 MPa. All specimens failed by studs shearing off with no obvious concrete cracks around the studs. Results showed that studs with 675 MPa tensile strength reached 15.3% and 15.6% higher loads and rigidity when compared with studs having 465 MPa tensile strength. The authors also found that the shear capacity and shear rigidity of the 30 mm studs were 39.6% higher and 82.2% higher than specimens with 22 mm studs.

Prakash et al. (2012) modified the conventional push-out test specimens by increasing the concrete confinement around the studs using hoop type transverse reinforcement to test the effect of shear strength and stiffness of high strength steel (HSS) stud connectors. The HSS studs used in this study had ultimate tensile and yield strengths of 900 MPa and 680 MPa respectively. The studs had a 20 mm shank diameter and 30 mm head diameter. During loading the specimens had cracks that extended parallel to the shear stud position on the top of the slab at almost 30% of the ultimate load. The authors found that the added hoop reinforcement had yielded during loading and resulted in an increase in the ultimate load of the push-out specimens. The authors also found that the HSS shear studs achieved on average 132 kN per stud which was 23.4% more than the 107 kN anticipated using the Eurocode (2004).

2.3 Summary

In this chapter, the research issues and background are explained and a general literature survey on the state-of-the-research on the shear studs is provided. The literature review shows that there is little information in either reported research or the current code practice regarding guidelines for issues specific to this study.

CHAPTER 3 EXPERIMENTAL PROGRAM

3.1 General

The push-out tests were designed to study the composite action between shear studs and concrete with the focus on the effect of elevation of steel reinforcement mesh. Other parameters including the types and the arrangement of the shear studs and the treatment of steel flange surfaces were also considered. Detailed descriptions of test specimens, experimental setup, and testing procedures are given in the following sections.

3.2 Test Specimens

3.2.1 Testing Standards

The ASTM standards does not specify the dimensions and detailing of push-out specimens like the Eurocode standard (2004). The Eurocode specifies the specimens to have four shear connectors welded on each flange as shown in Figure 3.1. All dimensions shown in Figure 3.1 are in mm. The code specifies ribbed bars reinforcement of 10 mm diameter with 15 mm concrete cover. The concrete slabs should be poured in the horizontal position and left to air cured to simulate field conditions. The bond between the concrete and steel is prevented by greasing the flanges. For each concrete mix design a minimum of four concrete cylinders must be poured and cured alongside the push-out specimens. The push-out specimens should be tested at a concrete strength of 70% ± 10% of the designed concrete strength which can be accomplished by testing the specimens earlier than 28 days from casting. The material properties including tensile strength, yield strength, and the maximum elongation of the shear connector should be determined.

During testing the specimens should first be loaded 25 times in increments between 5% 40% of the expected failure load. After the load cycling, the specimen testing can begin where subsequent increments should be imposed such that failure does not occur in less than 15 minutes. The slip between should be measured between both concrete slabs and the steel continuously during load increments until the load drops to 20% below the maximum load. The transverse separation between the steel section and each slab should be measured as close as possible. If multiple specimens are tested of the same parameters, the mean should be calculated for the specimens given that the loads do not exceed 10% deviations then divided by the number of shear connectors reduced by 10%. If the loads of the specimens exceed 10%, then at least three different specimens must be tested again.

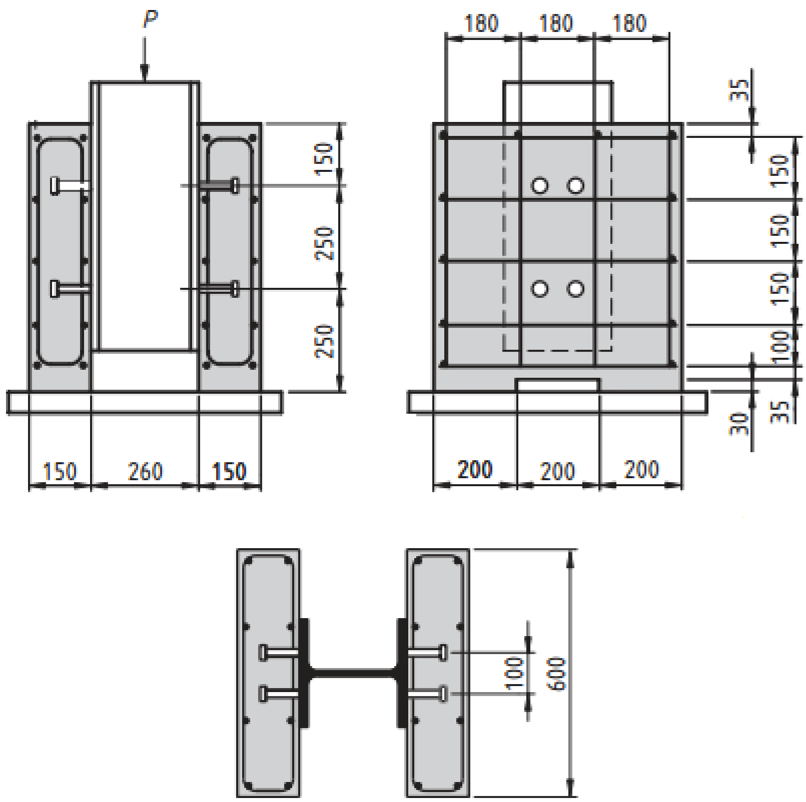
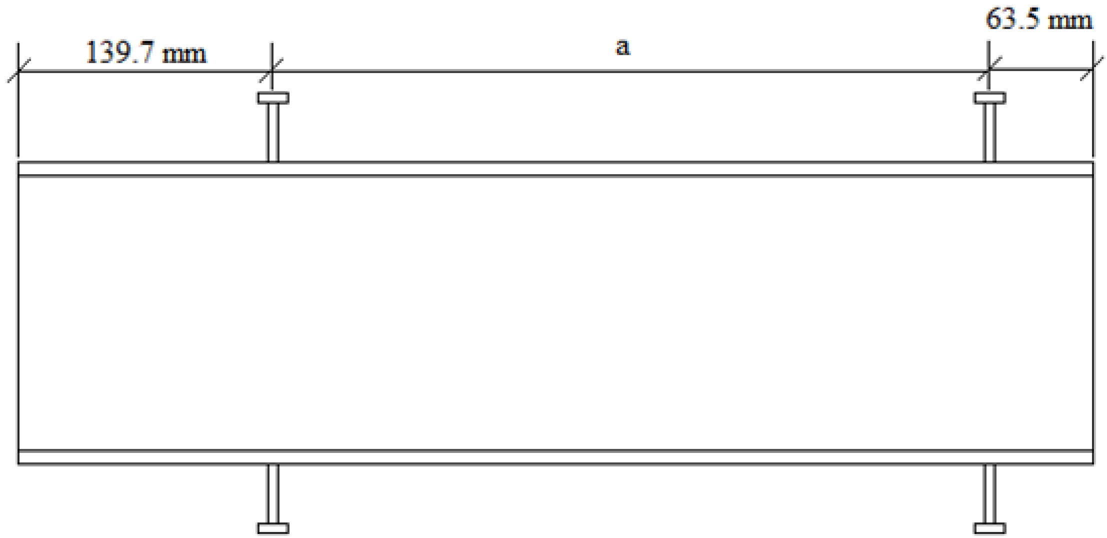


Figure 3.1 Eurocode push-out test standard specimen

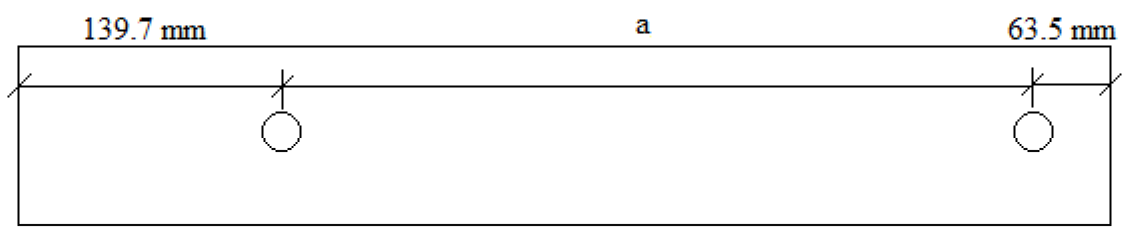
3.2.2 Description of Test Specimens

A total of thirty-three push-out specimens were fabricated and tested with various parameters. Figure 3.1 shows the arrangement of steel studs on the W250x28 steel section used in the test. Dimension (a) in the figure was 300 mm for most specimens.

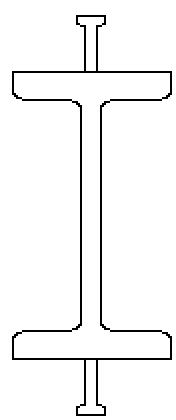
However, dimension (a) was changed to 200 mm and 100 mm in specimens to decrease the shear stud spacing. Two shear studs of 22.2 mm (7/8 inch) in diameter were welded on each flange of the section. Figure 3.2 shows a schematic view of a push-out test specimen with concrete cast around the steel section. Each specimen had two concrete slabs on each side of the steel section attached to the flanges. Each concrete slab measures 355.6 mm wide by 228.6 mm deep. The height of the slab was varied depending on the stud spacing with the 508 mm being the height for most of the specimens. The steel section and the concrete slab were offset by 76.2 mm.



(a) Side view

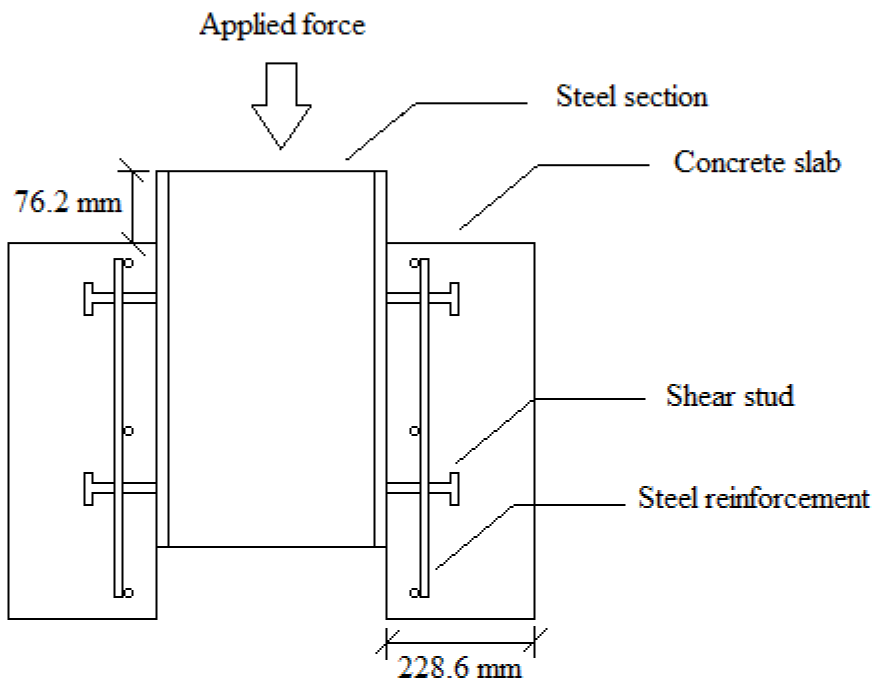


(b) View of flange

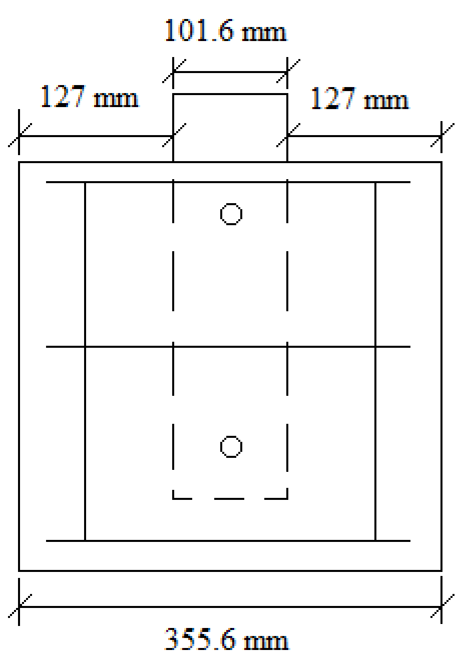


(c) Plan view

Figure 3.2 Illustration of steel section with welded shear studs



(a) Cross sectional view



(b) Side elevation view

Figure 3.3 Illustration of push-out specimen

Thirty-three specimens were divided into two series according to the type of studs used in the study. Series 1 contained the conventional Nelson shear studs with a diameter of 22.2 mm shown in Figure 3.3, whereas Series 2 contained adjustable shear studs of the same diameter shown in Figure 3.4. Series 1 studs had an average ultimate tensile capacity and yield strength of 478.5 MPa and 368.6 MPa respectively; whereas Series 2 studs had 594.1 MPa and 470.0 MPa. Among Series 1 specimens, some were fabricated with unheaded shear studs, seen in Figure 3.3 (b). Some specimens in Series 2 had the extra portion of the shear stud cut-off after the nut was twisted to the required height as in Figure 3.4 (b), while the others had a 50 mm protruding tip as in Figure 3.4 (a). The specified concrete strength used in the test is 45 MPa for all specimens.

(a) (b)

Figure 3.4 Series 1 Stud types (a) headed stud (b) unheaded stud

(a) (b)

Figure 3.5 Series 2 Adjustable studs (a) with protruding tip (b) cut-off tip

Table 3.1 provides details of twenty-eight specimens in Series 1. Specimens S1-C-1 and S1-C-2 are specimens where no steel reinforcement mesh was used. Specimens S1-D-1 to S1-D-3 are specimens having double layers of reinforcement mesh with various positions in relation to the studs (the position of layers indicated in the bracket). Specimens S1-4 to S1-22 are used to study the effect of the elevation of reinforcement mesh in relation to the stud height on the capacity of the specimen with either headed or unheaded studs. In general, three positions of reinforcement mesh including below the head of studs, in flush with the top of studs and above the studs were investigated in combination with three stud heights including 100, 150, and 200 mm (4, 6 and 8-inch). S1-8 together with S1-4 were used to study the effect of the friction between the flange and the concrete by having an ungreased flange in specimen S1-8. Specimens S1-9 and S1-10 were tested to study the effect of stud spacing where the spacing was varied to 200 and 100 mm respectively while 300 mm was used for the rest of specimens. S1-23 and S1-24 are used to investigate the effect of the flange surface treatment. The steel flanges for these two specimens are shown in Figure 3.5.

(a)

(b)

Figure 3.6 Surface treatment for steel sections (a) SP10 blast left outdoors to rust (b) SP10 blast coated with inorganic zinc primer

Table 3.1 Description of Series 1 specimens

	Concrete strength (MPa)	Stud height (mm)	Headed studs	Steel mesh position (mm)	Surface preparation	Stud spacing (mm)
S1-C-1	45	100	NO	-	Greased	300
S1-C-2	45	100	YES	-	Greased	300
S1-D-1	45	100	YES	Double layer (150,200)	Greased	300
S1-D-2	45	100	YES	Double layer (50,90)	Greased	300
S1-D-3	45	100	YES	Double layer (50,160)	Greased	300
S1-4	45	100	YES	70	Greased	300
S1-5	45	100	YES	100	Greased	300
S1-6	45	100	YES	150	Greased	300
S1-7	45	100	YES	200	Greased	300
S1-8	45	100	YES	70	Un-Greased	300
S1-9	45	100	YES	70	Greased	200
S1-10	45	100	YES	70	Greased	100
S1-11	45	150	YES	70	Greased	300
S1-12	45	150	YES	150	Greased	300
S1-13	45	150	YES	200	Greased	300
S1-14	45	200	YES	70	Greased	300
S1-15	45	200	YES	150	Greased	300
S1-16	45	200	YES	200	Greased	300
S1-17	45	150	NO	70	Greased	300
S1-18	45	150	NO	150	Greased	300
S1-19	45	150	NO	200	Greased	300
S1-20	45	200	NO	70	Greased	300
S1-21	45	200	NO	150	Greased	300
S1-22	45	200	NO	200	Greased	300
S1-23(2 specimens)	45	100	YES	70	SP10 blast left outdoors to rust	300
S1-24(2 specimens)	45	100	YES	70	SP10 blast coated with inorganic zinc primer	300

Table 3.2 gives details of Series 2 specimens used to study the adjustable studs. Three specimens in this series are fabricated with adjustable shear studs with cut-off tip, and two with protruding tip. Specimen S2-C-1 is the control specimen with adjustable studs but without reinforcement mesh.

Table 3.2 Description of Series 2 specimens

	Concrete strength (MPa)	**Stud height (mm)**	**Cut-off tip**	**Steel mesh position (mm)**	**Surface preparation**	**Stud spacing (mm)**
S2-C-1	45	100	YES	-	Greased	300
S2-1	45	100	YES	70	Greased	300
S2-2	45	100	NO	70	Greased	300
S2-3	45	150	YES	70	Greased	300
S2-4	45	150	NO	70	Greased	300

3.2.3 Fabrication of Test Specimen

All specimens were constructed in the heavy structural laboratory at Dalhousie University where temperature and moisture are well controlled. The formworks for all specimens were built according to the specified dimensions. Straight edges and levels were used throughout the construction process to ensure the formworks square and level. The pieces of the formwork can be seen in Figure 3.6 and the final product can be seen in Figure 3.7. The formwork was built around a fixed steel section in the middle, forming two boxes to be filled with concrete. The steel section was fixed from the top and bottom to prevent tilting of the steel section during pouring.

Figure 3.7 Formwork pieces

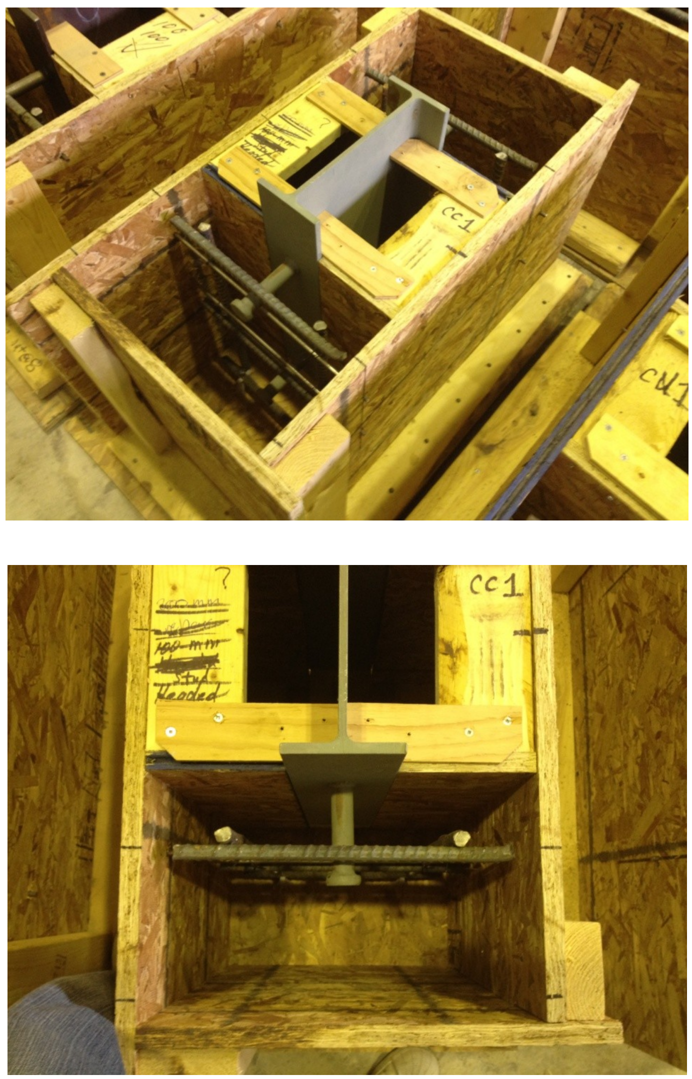


Figure 3.8 Wooden formwork for specimen S1-24-1

All specimens containing layer reinforcement had the same steel mesh consisting of two 15 M bars in the direction of the loading and three 15 M bars in the transverse direction as shown in Figure 3.8. The two bars placed in the direction of the loading are 470 mm long with 220 mm spacing between them. The three bars in the transverse direction are 320 mm long with 235 mm spacing between them. The bars were welded together to form a mesh for ease of placement.

Figure 3.9 Reinforcement mesh

The reinforcement mesh was placed on two 25 mm chairs as shown in Figure 3.9 for positioning.

Figure 3.10 Reinforcement placed on 25 mm chair

The reinforcement was held in position horizontally by tying the reinforcement mesh to two thin bars drilled through the formwork extending to the outside at the required position as shown in Figure 3.10.

Figure 3.11 Tying of reinforcement

The concrete used was a 45 MPa high performance concrete mix approved by Nova Scotia Department of Transportation and Infrastructure Renewal (NS-TIR). Concrete was poured into the formwork and gently vibrated to reduce the potential honey combing and trapped air voids. After pouring, the top of the specimen was levelled and smoothed. Specimens were wet cured for 7 days using wet burlap and polyethylene covers according to ASTM C192 (2007). After 48 hours of curing, the formwork was stripped from the specimens. Cylinders were cast along with the specimens, cured and tested according to ASTM C39 (2011) at 7 and 10 days to determine the compressive strength of concrete.

3.3 Test Procedure and Data Acquisition

An Instron universal testing machine was used to apply the compressive load to the specimens as shown in Figure 3.11. Each specimen was first placed inside the Instron and aligned in the centre of the loading head in the vertical and two transverse directions. Two linear variable differential transformers (LVDT) were mounted on each side of the web of the steel section to measure relative slip between the steel and the concrete during loading. The LVDTs were held by magnetic clamps mounted 250 mm from the top of the flange. A 12.7 mm steel plate and a thin piece of rubber sheet were placed on top of the steel section to ensure uniform distribution of the loading. Steel shims, if necessary, were used to level the steel plate on top of the specimen. Prior to each test, the LVDTs were checked for their functionality and the specimen was cycled twice with a 10% of the estimated capacity to make sure the specimen was "settled in" the test position. A displacement controlled loading scheme was used where the compressive load was applied at an average rate of 1.5 *mm/min* until the specimen lost capacity to sustain any additional load and the load vs. deflection curves showed an irreversible drop in the load. The load and deflections were monitored and recorded at an interval of 0.1 second throughout the loading history for each test using the data acquisition system in the Instron. Appearance of cracks and signs of failure were noted and recorded for each test.

Figure 3.12 Experimental test set-up

3.4 Auxiliary Tests

3.4.1 Concrete Cylinders

Push-out specimens were poured in two batches. The first batch contained 17 specimens and 15 concrete cylinders were cast. The second batch contained 16 specimens and 15 cylinders were cast. Concrete cylinders were 203.2 mm tall and 101.6 mm in diameter and were casted and tested in accordance to ASTM C39 (2011). Cylinders were cast at the same time as the specimens. After casting, concrete cylinders were wrapped in plastic sheets for the first 24 hours, and then were placed in a curing room. Cylinders were tested at 7 and 10 days to determine the strength of the concrete. Since the concrete mix used is a high performance concrete mix, strength of 45 MPa was reached on the 10^{th} day. Specimens were tested in a span of 3 days. During the testing, cylinders were caped using a sulphur compound to make sure that cylinder surface is level and smooth as seen in Figure 3.12.

Figure 3.13 Caped concrete cylinders

3.4.2 Steel Coupons

Steel coupons were randomly cut from the steel sections from both the flanges and webs. Coupons were also machined from both adjustable and conventional shear studs. Coupons cut from the steel section were rectangular, where coupons machined from the stud were round. Dimensions of round coupons and rectangular coupons can be seen in Figure 3.13 and 3.14 respectively. Rectangular coupons cut from the flange were 10 mm thick were coupons cut from the web were 6.4 mm thick.

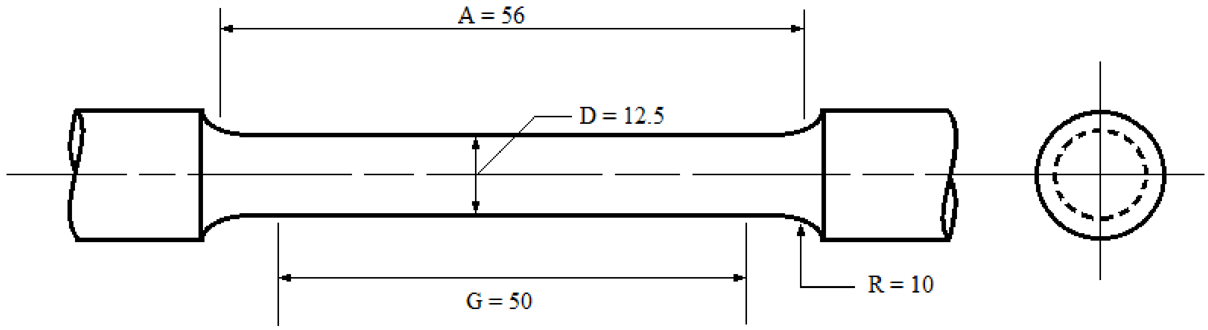


Figure 3.14 Dimensions of round coupons

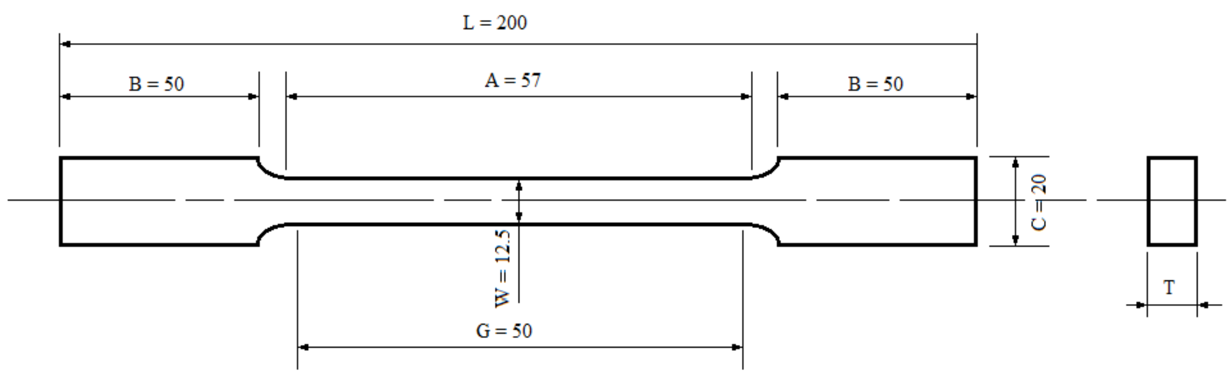


Figure 3.15 Dimensions of rectangular coupons

Specimens were tested in an Instron machine according to ASTM E8 (2012) to determine the ultimate tensile strength, yield strength, and modulus of elasticity of steel. Specimens were loaded at a rate of 1 *mm/minute* untill failure.

CHAPTER 4 EXPERIMENTAL RESULTS

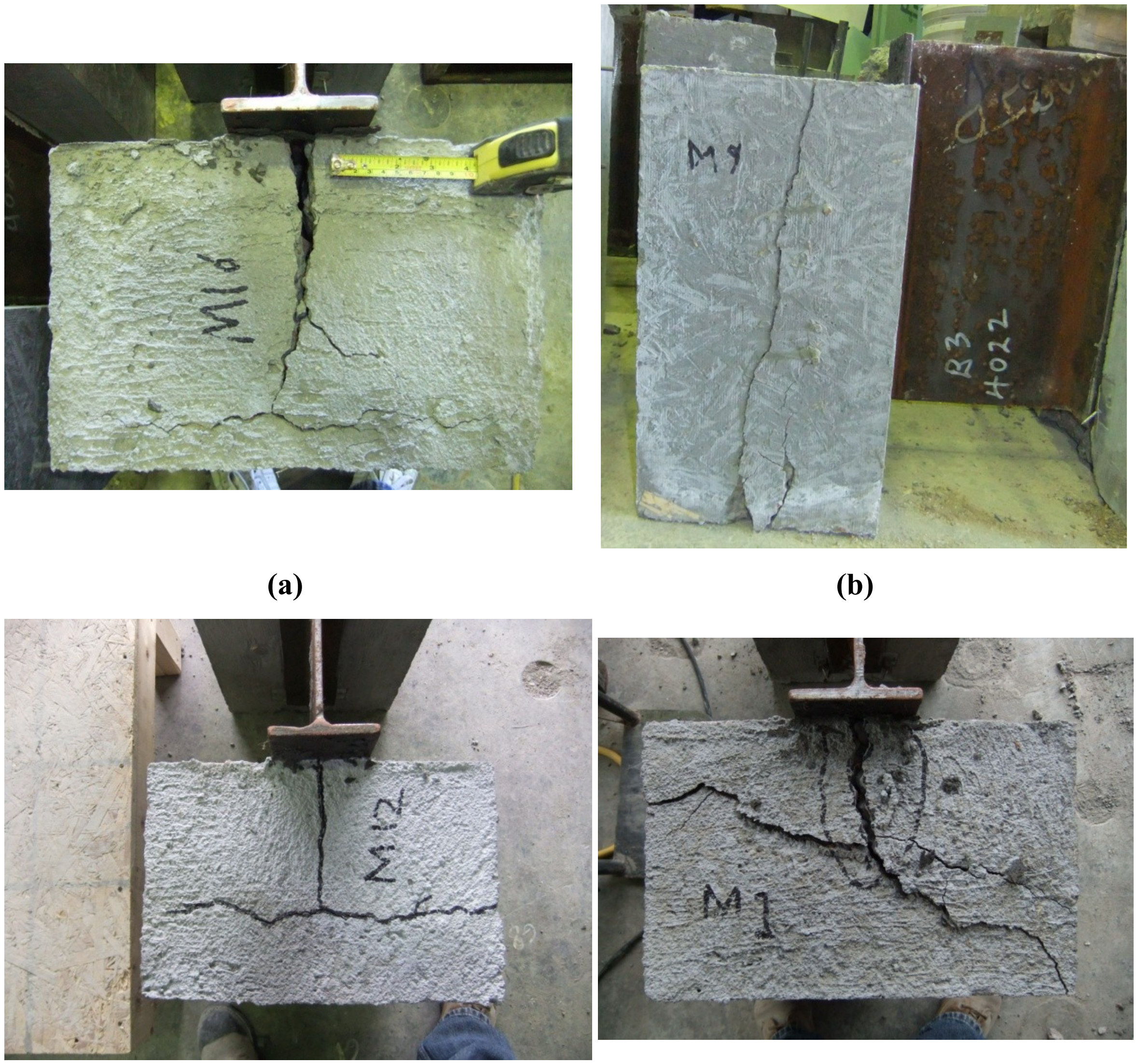
4.1 General

In this chapter the experimental results of 33 push-out specimens and auxiliary specimens of concrete cylinders and steel coupons are presented in the forms of tables and graphs. The effects of various parameters considered on the behaviour and capacity of composite action are discussed. The comparison between the test results and code suggested values is also presented.

4.2 Failure Mode

Depending on the elevation of the reinforcement mesh, failure modes differed. As the elevation of the mesh increased from intercepting the studs to in flush with the stud head and to above the stud, the failure mode shifted from stud shear off to combined concrete failure and bent studs to concrete related failure. Overall, for all specimens tested, at about 60 to 70% of the ultimate load, signs of concrete cracking were observed as seen in Figures 4.1. For specimens with a single layer reinforcement mesh intercepting the studs, initial cracks were formed in the direction parallel to the stud at the top of the concrete slab and then grew into cracks perpendicular to the stud direction at the position of reinforcement forming a T shape as shown in Figure 4.1 (a) and (c). For specimens with double layers of reinforcement mesh, cracks were formed crossing both layers with an angle as seen in Figure 4.1 (d). As the load increased, some cracks developed through the vertical face of the slab column as seen in Figure 4.1 (b). At failure, some specimens had a slab shear-off from the steel section with the shear studs remaining embedded inside the slab as seen in Figure 4.2. This failure was accompanied with a sudden bang noise. Other specimens had both slab columns remained attached with no apparent shear stud failure. However, after removal of the concrete around the shear studs, it was discovered that one stud had been sheared off in some of these specimens and the remaining stud kept the slab in place. Concrete related failure was characterized by initial concrete cracking and concrete crushing at failure as seen in Figure 4.3. The bent suds were shown in Figure 4.4

where some studs experienced plastic deformation during loading but stayed attached to the steel section. Figure 4.4 (a) shows a shear-like deformation form commonly seen in 100 mm shear studs whereas a bending-like deformation form seemed to occur for 150 mm, and 200 mm studs seen in Figure 4.4 (b).



(a) **(b)**

(c) **(d)**

Figure 4.1 Concrete cracking in tested specimens (a) Crack parallel to shear stud, (b) Longitudinal crack through slab, (c) Crack parallel to shear stud extending perpendicularly over position of reinforcement, (d) Crack pattern of a specimen with a double-layer reinforcement

(a) **(b)**

Figure 4.2 Sheared off specimen after testing (a) Sheared off flange, (b) Sheared off slab

Figure 4.3 Concrete crushing

(a) (b)

Figure 4.4 Deformed studs (a) 100 mm stud, (b) 200 mm stud

4.3 Ultimate Capacity

For ease of discussion, test results were divided into eight groups based on one common parameter. For each group, a brief description of specimens is given and followed by the discussion on the ultimate load, slip at ultimate load and the observed failure mode of each specimen. Note that all specimens exhibited concrete cracking and crushing to some degree, specimens with slab shear-off are specially identified under "failure mode" in the summary tables. A complete description of the specimens is found in Table 1 in Chapter 3.

4.3.1 Group 1 - Specimens Without Reinforcement Mesh

Group 1 consists of specimens without reinforcement in the slabs and contained 100 mm long studs. A summary of results is shown in Table 4.1 where P_u is the ultimate load of the specimen and s represents the average slip obtained at the ultimate load. Column 5 shows the increases in capacity of specimens using the lowest tested load as the reference value. The load vs. slip diagram is seen in Figure 4.5.

Table 4.1 Experimental results of Group 1 specimens

Specimen ID	Description	P_u (kN)	s (mm)	% Difference	Failure mode
S1-C-1	Headed stud	567	2.30	18.2	Concrete failure/ no bending in stud
S1-C-2	Unheaded stud	479	2.07	-	Concrete failure/ no bending in stud
S2-C-1	Adjustable stud/ no tip	492	2.31	2.6	Concrete failure/ no bending in stud

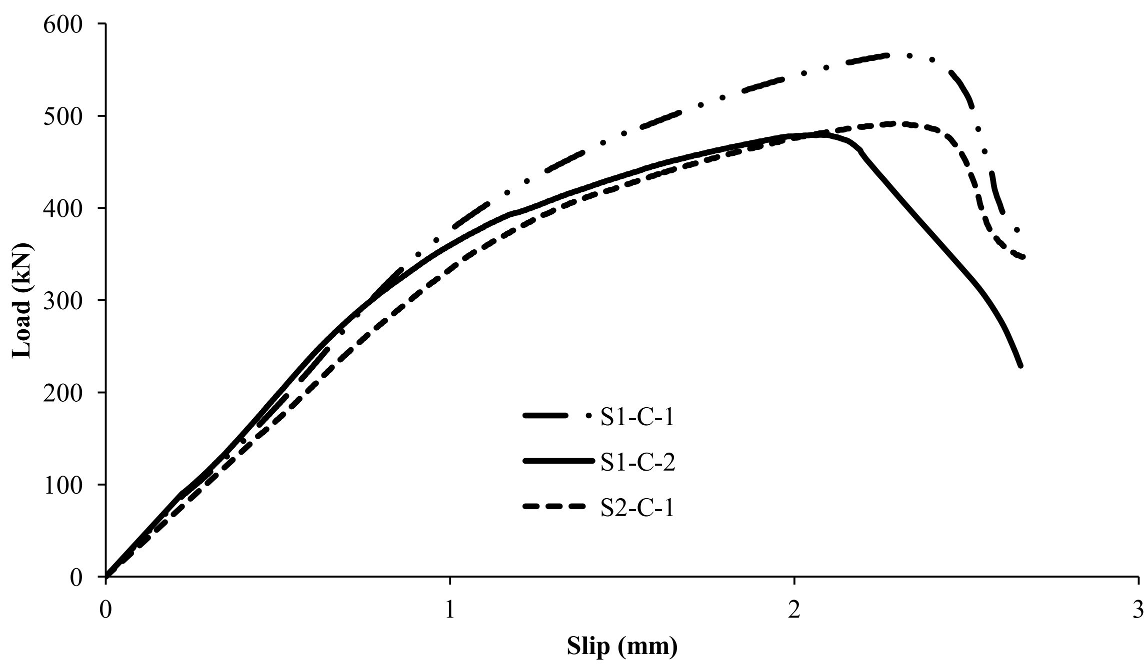
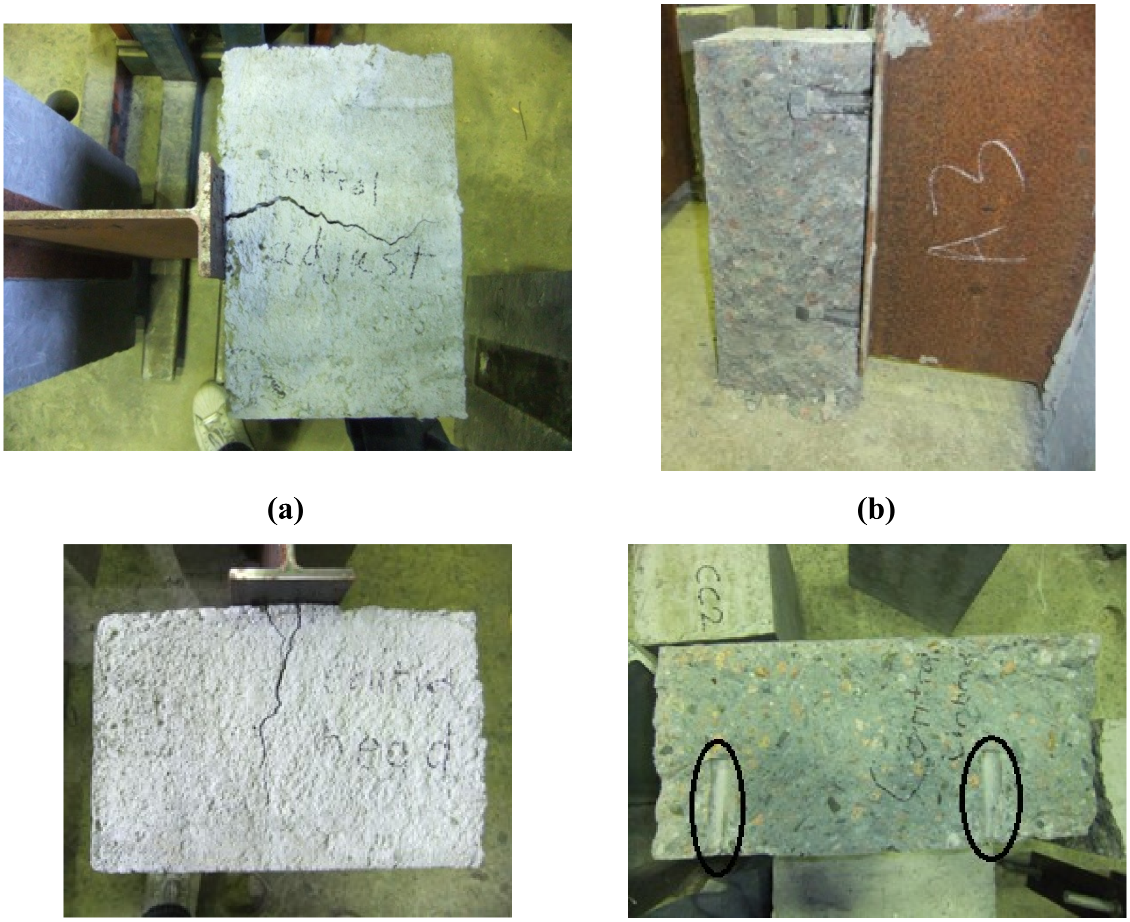


Figure 4.5 Load vs. slip graph for Group 1 specimens

Table 4.1 shows that specimen with headed conventional studs attained the highest ultimate load followed by specimen with adjustable studs and the specimen with unheaded studs had the lowest ultimate load. With all other parameters being constant, test results show that the head of the stud resulted in an 18.2% increase in ultimate load from that for the unheaded specimen and that the use of the conventional stud led to a 15.3% higher ultimate load in specimen than the adjustable stud. As shown in Figure 4.5, all three specimens had similar behaviour and similar slippage at ultimate load. The failure appeared to be concrete related as there was no sudden drop in the curve. This is further confirmed in Figure 4.6 showing the concrete cracking failure. Cracks formed parallel to the shear stud and due to lack of reinforcement, no cracks perpendicular to the shear stud were observed. The studs of all three specimens did not undergo any significant deformation as seen in Figure 4.7. This is attributed to the lack of reinforcement in the slab and the concrete did not have any effective confinement around the studs to allow the studs to reach their tensile strength.



(a) (b)

(c) (d)

Figure 4.6 Failure mode of Group 1 specimens (a) Adjustable stud specimen concrete cracking, (b) Adjustable stud specimen cross sectional view, (c) Concrete cracking in control headed specimen, (d) Unheaded stud slab specimen cross sectional view

(a) **(b)**

(c)

Figure 4.7 Stud deformation-Group 1 specimens (a) Adjustable studs, (b) Headed studs, (c) Unheaded studs

4.3.2 Group 2 - Specimens With Double-Layer Reinforcement Mesh

Group 2 consists of specimens with 100 mm long studs and double-layer reinforcement mesh in the slab. Results are summarized in Table 4.2 where positions of the two layers are indicated in the table. The percentage difference was calculated as an increase over specimen S1-C-1. The load vs. slip curves can be seen in Figure 4.8. Responses of S1-D-2 and S1-D-3 were similar with a sudden load drop at failure but both were distinctively different from that of S1-D-1 where the load fell more gradually. This difference is also reflected in their failure modes as indicated in Table 4.2.

Table 4.2 Experimental results of Group 2 specimens

Specimen ID	Mesh position (mm)	P_u (kN)	s (mm)	% Difference	Failure mode
S1-D-1	Both above stud (150, 200)	567	2.72	-	Concrete failure/ studs bent
S1-D-2	Both below stud (50, 90)	720	7.42	27.1	Slab shear-off/ 3 studs sheared-off
S1-D-3	Code requirement (50, 160)	769	8.33	35.6	Concrete failure/ 2 studs sheared off

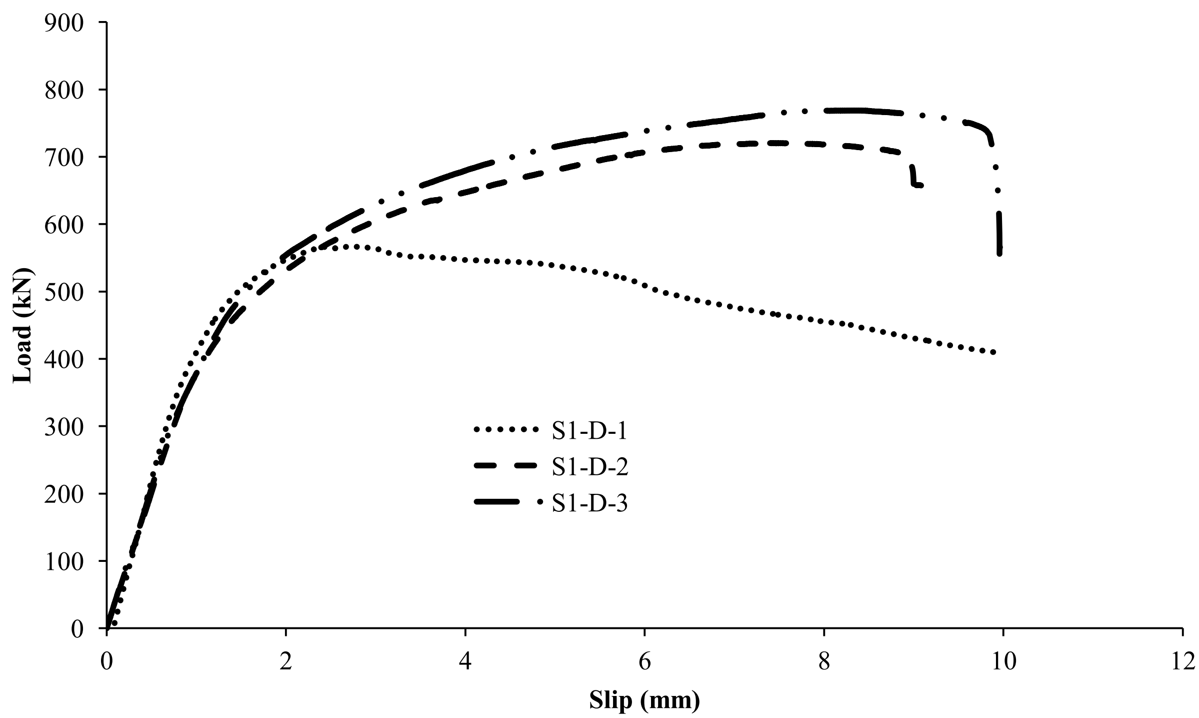


Figure 4.8 Load vs. slip graph for Group 2 specimens

Specimen S1-D-3 complying with the code reinforcement mesh requirement had the highest ultimate load and slippage in the group. Specimens S1-D-2 and S1-D-3 had respective increases of 27.1% and 35.6% in ultimate load over specimen S1-D-1 with both reinforcement layers well above the head of the studs. The comparison between S1-D-2 and S1-D-3 indicates that code required reinforcement mesh layout resulted in a 6.7% higher load than specimen with both layers below the head of the stud. This suggests that well-positioned reinforcement is crucial in ensuring the attainment of ultimate load. Specimen S1-D-1 had similar behaviour to the Group 1 specimen S1-C-1 in ultimate load but had an 18% increase in slippage. It suggests that when the reinforcement mesh does not intercept the stud, in this case, above the head of the stud, the reinforcement effect on the capacity is negligible. After further examination of the tested S1-D-1 specimen, it was found that the shear studs were bent as can be seen in Figure 4.9. Having both layers above the head of the stud prevented cracks from extending to the upper region of the slab and thus a premature failure, allowing the studs to deform.

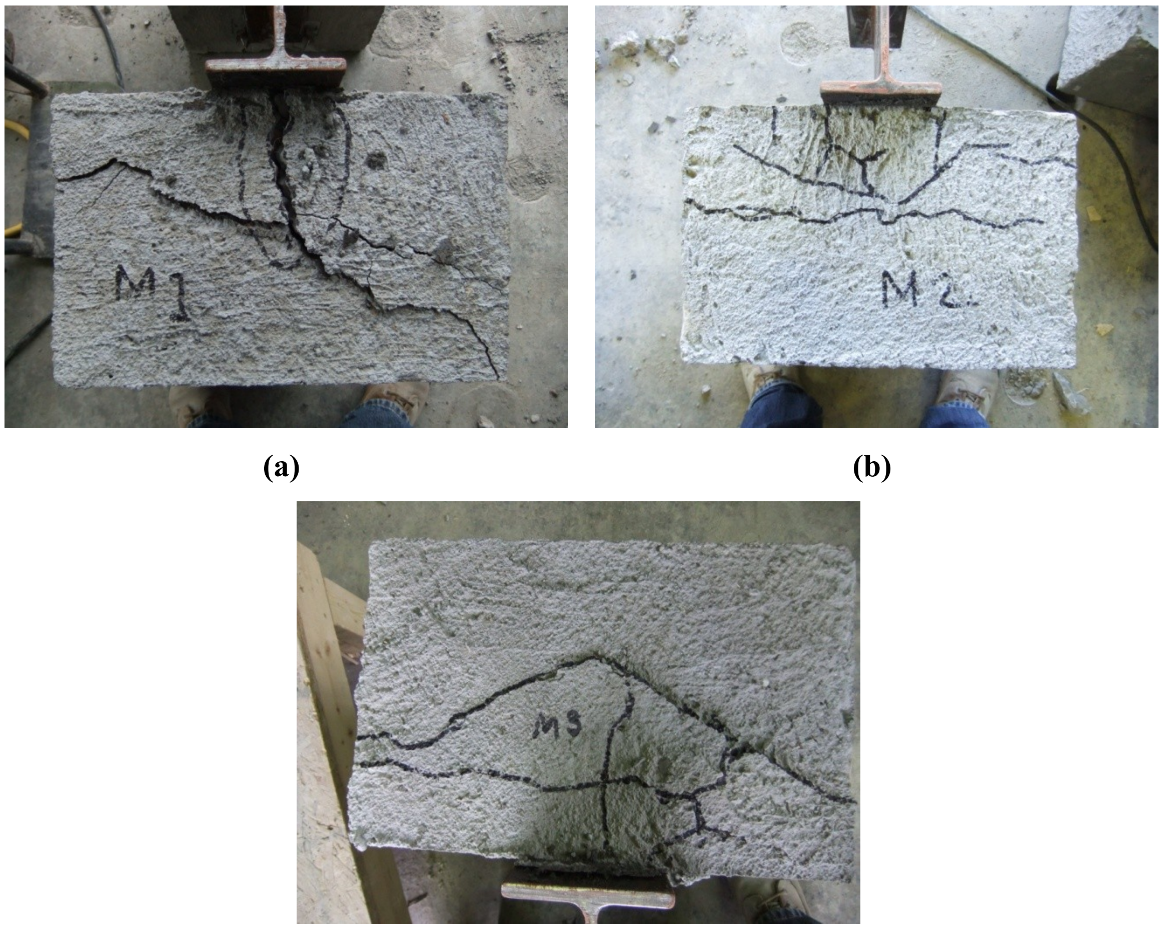
(a)

(b)

(c)

Figure 4.9 Stud deformation-Group 2 specimens (a) S1-D-1, (b) S1-D-2, (c) S1-D-3

The crack pattern for a specimen with double-layer of reinforcement mesh showed that the cracks were formed with an angle to the reinforcement. Figure 4.10 shows the cracking patterns for the specimens with double-layer of reinforcement mesh.



(a) (b)

(c)

Figure 4.10 Crack pattern for Group 2 specimens (a) S1-D-1, (b) S1-D-2, (c) S1-D-3

The layers of reinforcement in specimen S1-D-1 were able to increase the slippage by 18.3% over specimen S1-C-1 by confining the concrete above the shear studs, hence, preventing cracks from expanding into that region causing a premature failure, but were not able to increase the ultimate load. Specimen S1-D-2 failed by slab shear-off, where three studs were sheared off as seen in Figure 4.9 (b). Specimen S1-D-3 showed visible concrete failure but after the removal of the concrete it was found that two studs from each side were sheared off during loading as seen in Figure 4.9 (c).

4.3.3 Group 3 - Specimens With Various Stud Spacing

A summary of test results for specimens with different stud spacing are summarized in Table 4.3. All specimens in this group had 100 mm studs and one layer of reinforcement placed at 70 mm from the steel flange. A graph showing load vs. slip for the specimens is shown in Figure 4.11.

Table 4.3 Experimental results of Group 3 specimens

Specimen ID	Stud spacing (mm)	P_u (kN)	s (mm)	% Difference	Failure mode
S1-4	300	706	7.04	28.3	Concrete failure/ 1 stud sheared-off
S1-9	200	616	4.05	12.1	Concrete failure/ bent studs
S1-10	100	550	4.08	-	Concrete failure/ bent studs
S1-8	300 (Ungreased)	835	8.17	51.9	Concrete failure/ 1 studs sheared-off

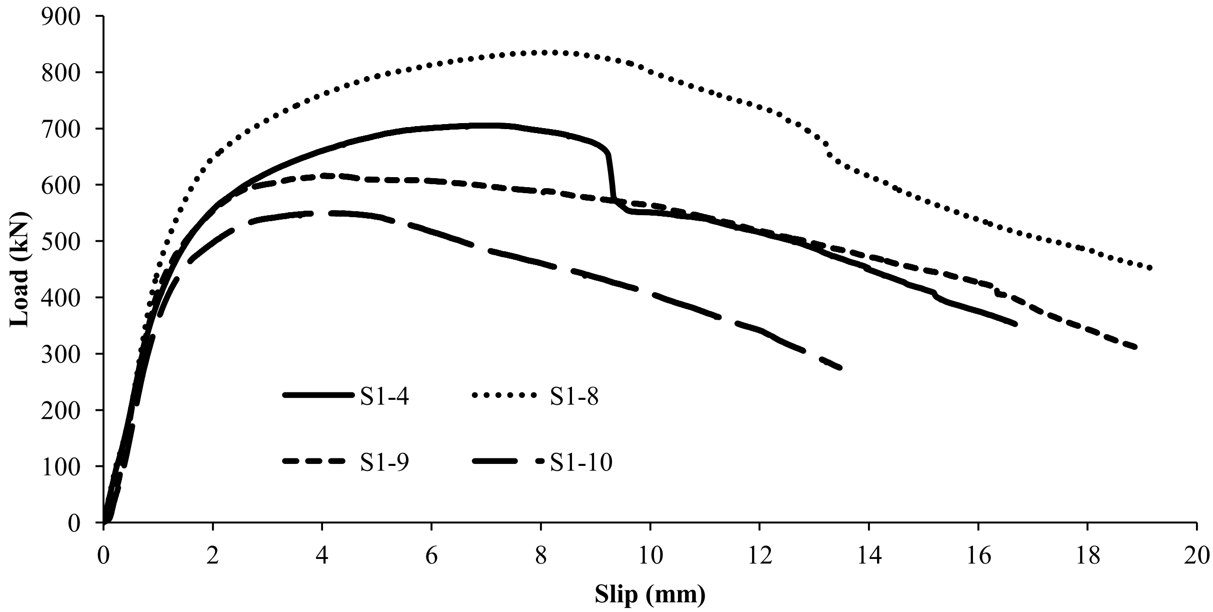


Figure 4.11 Load vs. slip graph for Group 3 specimens

Specimen S1-4 with the largest spacing had the highest ultimate load and slippage in the group with greased flanges. As shear stud spacing decreased so did the ultimate load. Specimens S1-4 and S1-9 had 28.3% and 12.1% higher ultimate loads respectively than specimen S1-10. The closer spacing may cause an overlapping of the stress field, which reduces the confining effect of the surrounding concrete and ultimately leads to a lower strength.

Specimen S1-8 with steel flange left ungreased had an 18.4% higher ultimate load than specimen S1-4 with greased flange while other parameters were kept the same. This increase is attributed to the friction between the concrete slab and the steel flange.

All specimens showed substantial concrete cracking. Pictures of the specimens after testing are shown in Figure 4.12 and 4.13 and the steel sections after the removal of concrete are shown in Figure 4.14. After further examination of the specimens, it showed that specimen S1-4 had a stud sheared-off as seen in Figure 4.14 (c) which is consistent with the sudden drop on the load vs. slip curve in Figure 4.11. Specimens S1-10 and S1-9 had bent shear studs.

Figure 4.12 Specimen S1-10 after testing

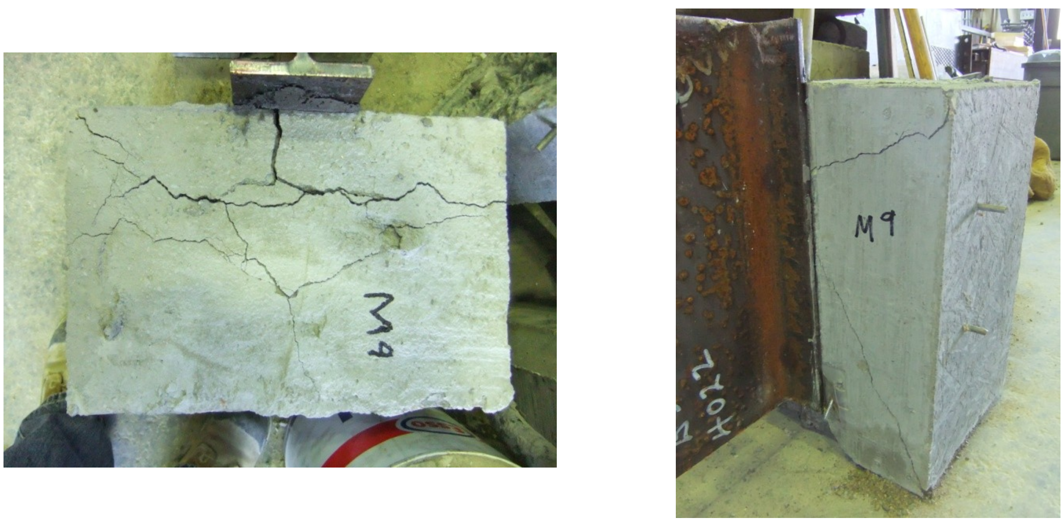


Figure 4.13 Specimen S1-9 after testing

(a) **(b)**

(c) **(d)**

Figure 4.14 Stud deformation- Group 3 specimens (a) S1-10, (b) S1-9, (c) S1-4, (d) S1-8

4.3.4 Group 4 - Specimens With Different Flange Treatment

Results of specimens that had different flange treatments are presented in Table 4.4. Specimens in this group had 100 mm studs and reinforcement mesh at 70 mm from the steel flange. A graph of load vs. slip for the specimens is shown in Figure 4.15. Specimen S1-23-2 had a premature failure due to tilting of the steel section during concrete pouring and was not used for comparison.

Table 4.4 Experimental results of Group 4 specimens

Specimen ID	Description	P_u (kN)	s (mm)	Average Values P_u (kN)	Average Values s (mm)	% Difference	Failure mode
S1-23-1	SP10 blast left outdoors to rust	740	8.93	740	8.93	5.9	Concrete failure/Slab shear-off
S1-23-2		543	3.89			-	Concrete failure/Slab shear-off
S1-24-1	SP10 blast coated with inorganic zinc primer	660	6.50	699	6.73	-	Concrete failure/ 3 studs sheared-off
S1-24-2		738	6.95			-	Concrete failure/ Slab shear-off

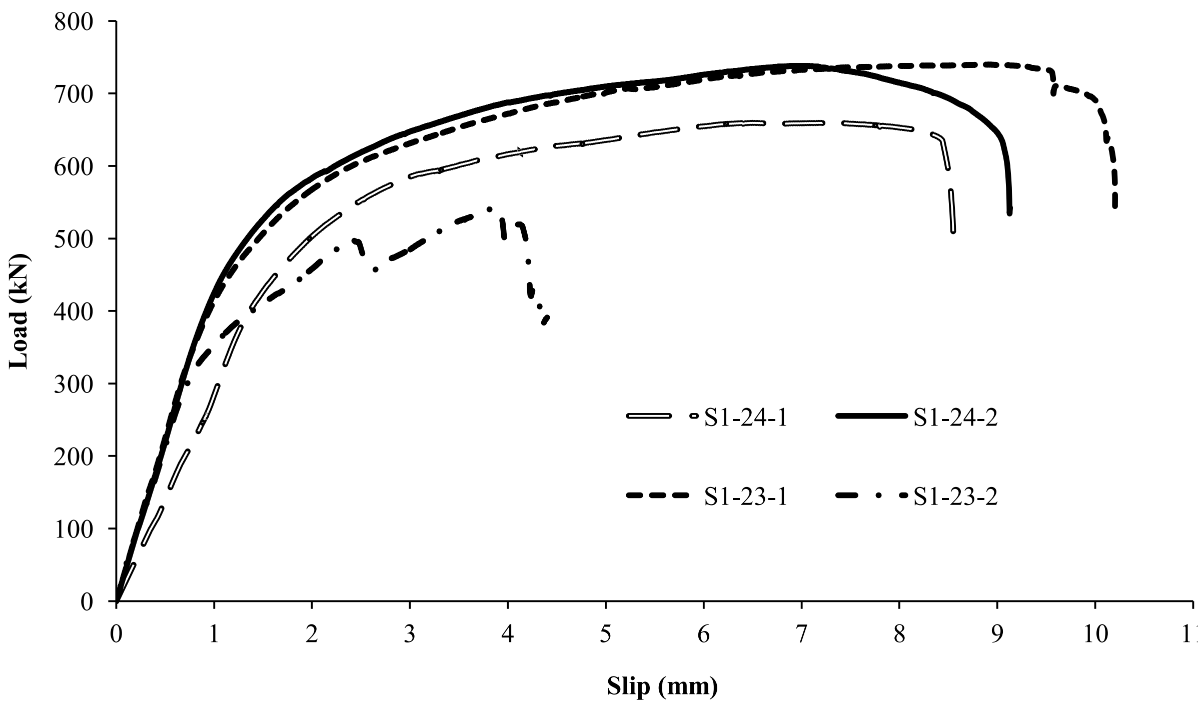


Figure 4.15 Load vs. slip graph for Group 5 specimens

For comparison purposes, the average ultimate load and slippage for specimens S1-24-1 and S1-24-2 are used. Specimen with rusted flanges (S1-23-1) attained a 5.9% higher load than the specimen with coated flanges. This increase is believed to be attributed to greater friction between the concrete and the rusted flange surfaces than the coated surfaces. This increase is not significant from the practical viewpoint. All three specimens (S1-23-1, S1-24-1, and S1-24-2) had the same failure mode by stud shear-off. Failure is shown for specimen S1-24-1 after the removal of the concrete in Figure 4.16. Comparison of specimens S1-24 and S1-4 (greased flange) from Group 3 showed that two specimens attained similar ultimate loads, indicating the effect of flange coating might be similar to greasing on the stud strength. However, more testing is required to verify these findings. Specimen S1-23-1 with rusted flanges attained 11.3% lower ultimate load than specimen S1-8 with ungreased flanges. This seemingly discrepancy is due to the fact that the rusted area on flanges of specimen S1-23-1 was only partial (Figure 3.5). The total area of the coated flange is 51,327 mm^2. The rusted flange 15 mm of coating encasing the rust, giving a total rusted area of 34,070 mm^2. The ratio of rusted flanges to coated flange is 0.7.

Figure 4.16 Stud deformation of Specimen S1-24-1 after testing

4.3.5 Group 5 - Specimens With 100 mm Studs and Varying Reinforcement Mesh Position

Test results of Group 5 specimens are summarized in Table 4.5. The load vs. slip graph for the specimens is shown in Figure 4.17. Specimens in Group 5 had 100 mm studs and the elevation of the single layer of reinforcement mesh in relation to the steel flange was varied from below the head of the stud to above. Percentage change of ultimate load of specimens is calculated as increase over specimen S1-7 which gave the lowest load.

Table 4.5 Experimental results of Group 5 specimens

Specimen ID	Mesh position (mm)	P_u (kN)	s (mm)	% Difference	Failure mode
S1-4	70	706	7.04	26.1	Concrete failure/ 1 studs sheared-off
S1-5	100	676	4.89	20.9	Concrete failure/ bent studs
S1-6	150	562	2.63	0.4	Concrete failure/ bent studs
S1-7	200	559	2.07	-	Concrete failure

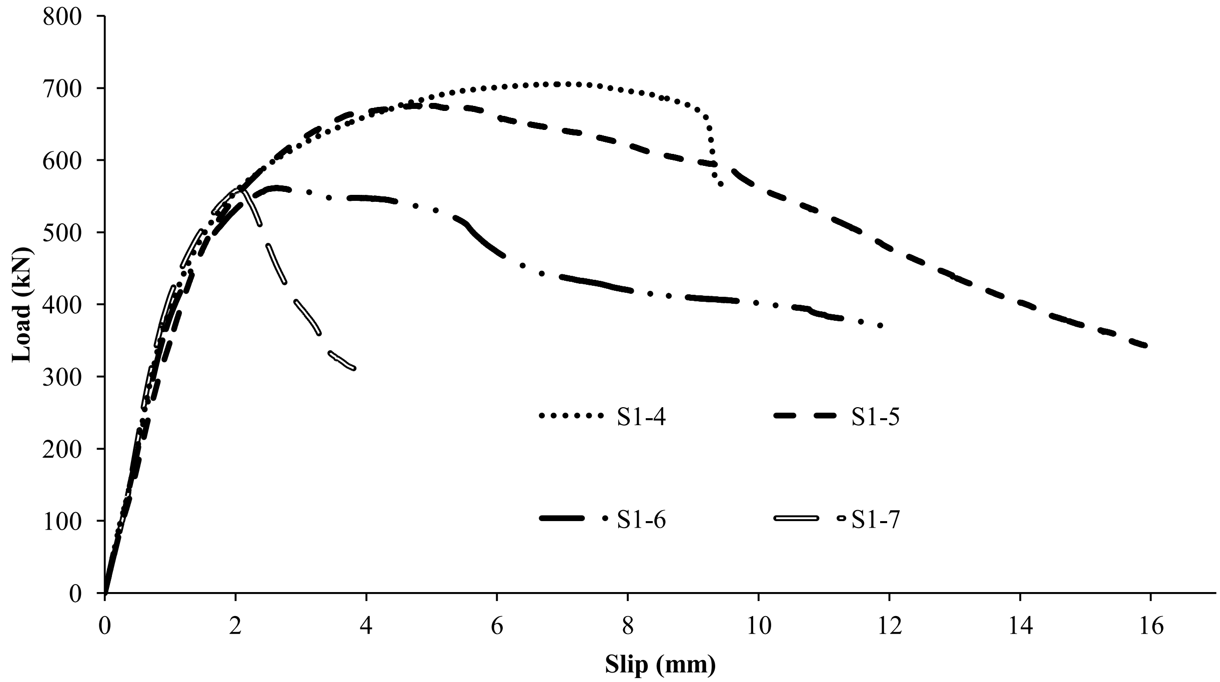
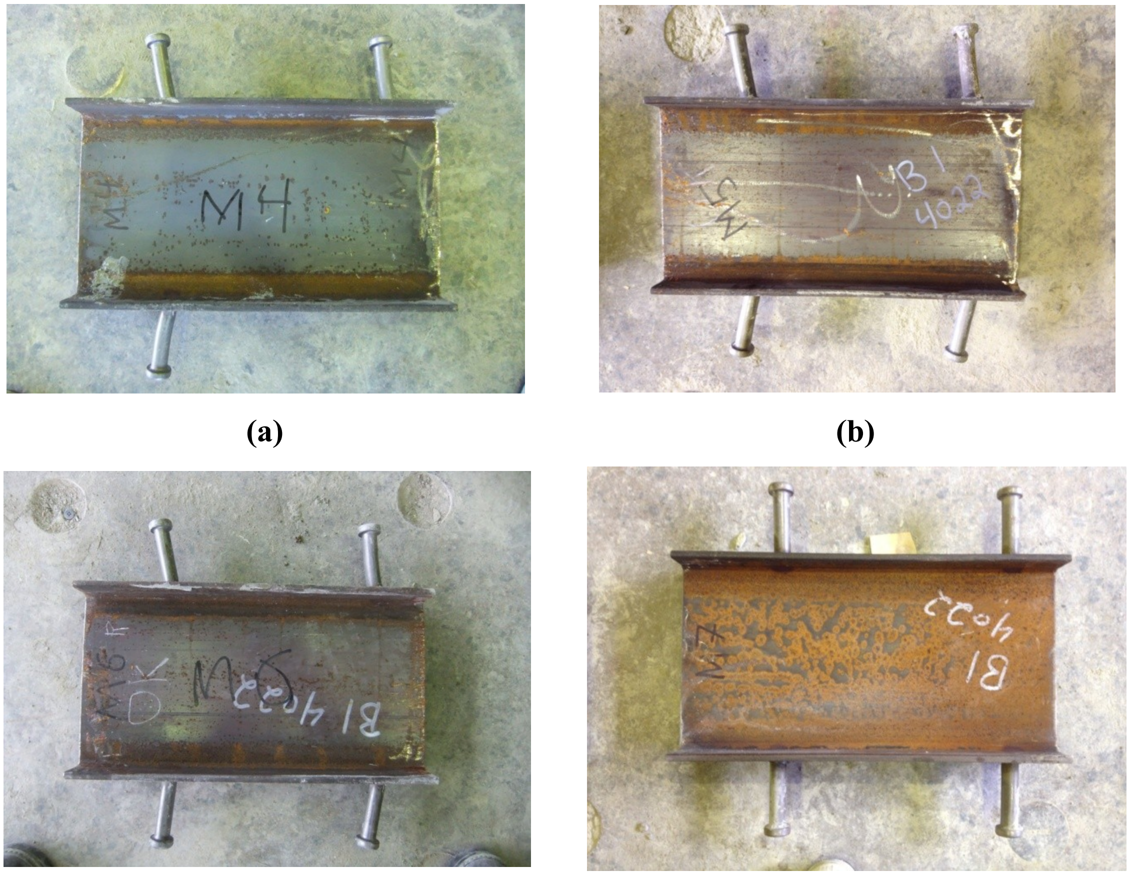


Figure 4.17 Load vs. slip graph for Group 5 specimens

The comparison of specimens S1-4 to S1-7 showed that as the elevation of reinforcement mesh increased, the ultimate load decreased. When the reinforcement is placed above the stud head, whether 50 or 100 mm above, the ultimate loads were practically the same and they were similar to the ultimate load attained by specimen S1-C-1 without reinforcement mesh, suggesting that the effect of the reinforcement is negligible. When the reinforcement is placed at about 70% of the stud height or in flush with the stud head, a significant ultimate load increase in the order of 23% on average was observed. Figure 4.18 shows pictures of the tested specimens after the removal of the concrete. Specimen S1-4 had a single stud sheared off during testing whereas the other three showed the concrete failure with studs remaining attached. Figure 4.19 shows the concrete cracking in specimen S1-7 after testing. The cracking pattern was similar to specimens S1-C-1 without reinforcement mesh. It again confirmed that the confinement of reinforcement mesh to concrete can be considered negligible when the reinforcement mesh is not intercepting the studs.



(a) (b)

(c) (d)

Figure 4.18 Stud deformation- Group 5 specimens (a) S1-4, (b) S1-5, (c) S1-6, (d) S1-7

(a) (b)

Figure 4.19 Specimen S1-7 cracking pattern

4.3.6 Group 6 - Specimens With 150 mm Studs and Varying Reinforcement Mesh Position

Test results of Group 6 specimens are summarized in Table 4.6. This group contains specimens with both headed and unheaded 150 mm studs and varying elevations of reinforcement from below the head of the stud to above. Specimens S1-11 through S1-13 were headed specimens and specimens S1-17 through S1-19 were unheaded specimens. Graphs for load vs. slip for both headed and unheaded specimens are shown in Figures 4.20 and 4.21 respectively.

Table 4.6 Experimental results of Group 6 specimens

Specimen ID	Mesh Position (mm)	P_u (kN)	s (mm)	% Difference	Failure mode
S1-11	Headed (70)	709	12.72	47.8	Slab shear-off/ remaining studs bent
S1-12	Headed (150)	525	5.18	11.0	Concrete failure/ 1 stud sheared- off
S1-13	Headed (200)	473	2.30	-	Concrete failure/ 1 stud sheared-off
S1-17	Unheaded (70)	675	8.61	42.4	Slab shear-off/ remaining studs bent
S1-18	Unheaded (150)	474	4.17	-	Concrete failure/ studs bent
S1-19	Unheaded (200)	477	2.14	0.6	Concrete failure

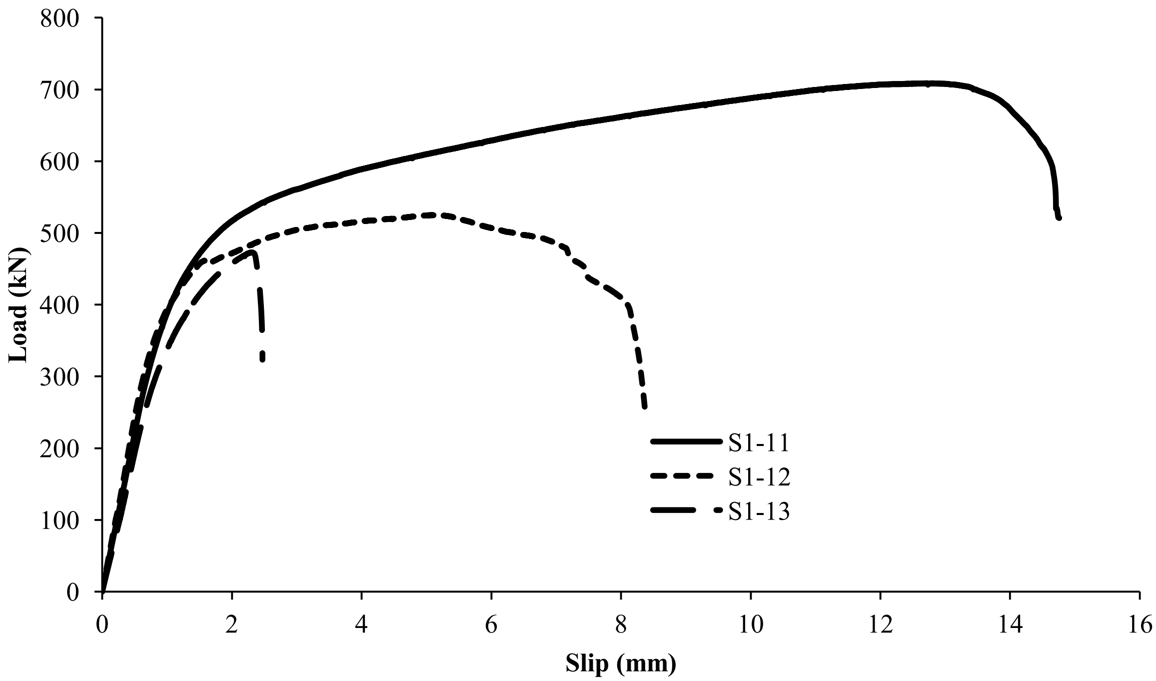


Figure 4.20 Load vs. slip graph for Group 6 headed specimens

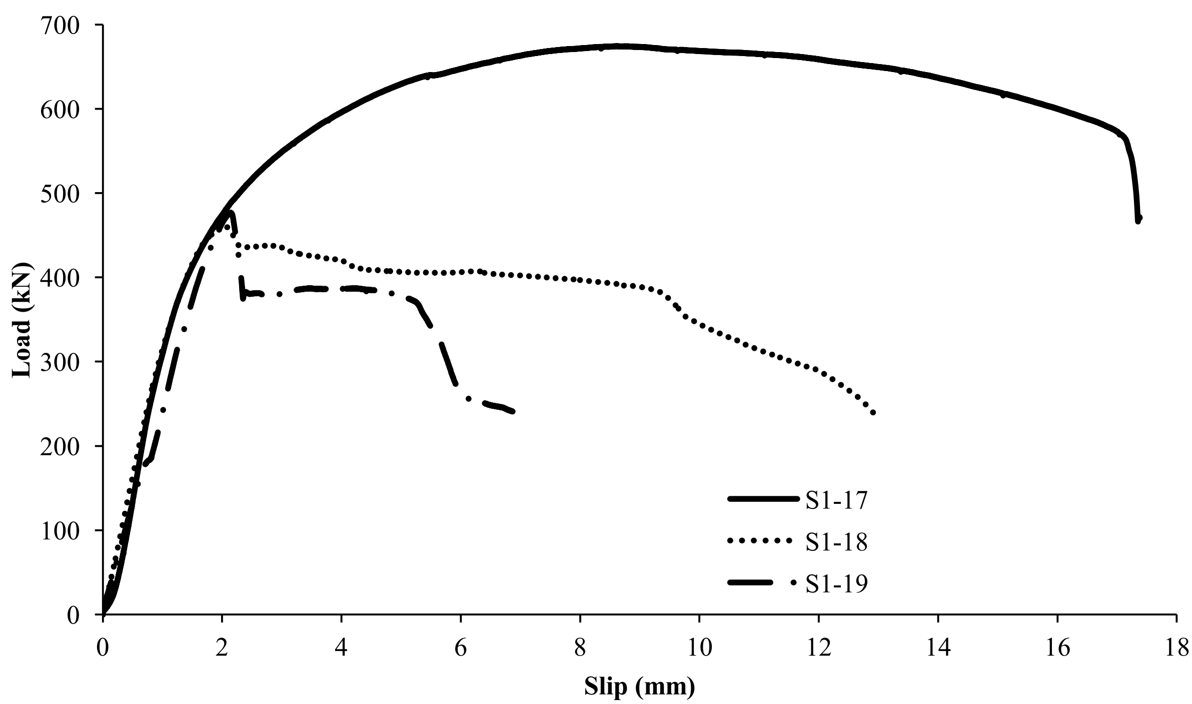


Figure 4.21 Load vs. slip graph for Group 6 unheaded specimens

Figure 4.22 shows the tested specimens after the removal of the concrete. Results obtained in this group for headed studs were similar to results observed in Group 5, where increasing the elevation of reinforcement reduced the ultimate load and slippage of the specimen. As the reinforcement mesh elevation increased from 70 to 150 to 200 mm, the reduction in ultimate load was 23.4% and 33.3% respectively. For unheaded specimens, specimen S1-17 with reinforcement mesh intercepting the studs attained the highest ultimate load. When the mesh was placed flush and above the stud head (Specimens S1-18 and S1-19), the ultimate loads were practically the same. From failure mode viewpoint, for specimens with unheaded studs, specimen S1-17 had two studs sheared off while the other two failures were predominately concrete related. This shows that regardless of headed or unheaded studs, the reinforcement needs to be intercepting the studs to have any effect in increasing the ultimate load of the specimen.

(a)

(b)

(c)

(d)

(e)

Figure 4.22 Stud deformation-Group 6 specimens (a) S1-12, (b) S1-13, (c) S1-17, (d) S1-18, (e) S1-19

4.3.7 Group 7 - Specimens With 200 mm Studs and Varying Reinforcement Mesh Position

Results of Group 7 specimens are summarized in Table 4.7. Group 7 specimens had both headed and unheaded 200 mm studs and varying elevations of reinforcement. Specimens S1-14 through S1-16 had headed studs whereas specimens S1-20 through S1-22 had unheaded studs. Load vs. slip responses for headed and unheaded shear stud specimens are shown in Figures 4.23 and 4.24 respectively.

Table 4.7 Experimental results Group 7 specimens

Specimen ID	**Mesh position (mm)**	**P_u (kN)**	**s (mm)**	**% Difference**	**Failure mode**
S1-14	Headed (70)	658	7.16	48.93	Concrete failure/ studs bent
S1-15	Headed (150)	587	4.34	32.80	Concrete failure/ studs bent
S1-16	Headed (200)	442	5.22	-	Concrete failure/ studs bent
S1-20	Unheaded (70)	650	11.80	27.14	Concrete failure/ 1 stud sheared-off
S1-21	Unheaded (150)	561	4.55	9.74	Concrete failure/ 1 stud sheared-off
S1-22	Unheaded (200)	511	2.57	-	Concrete failure

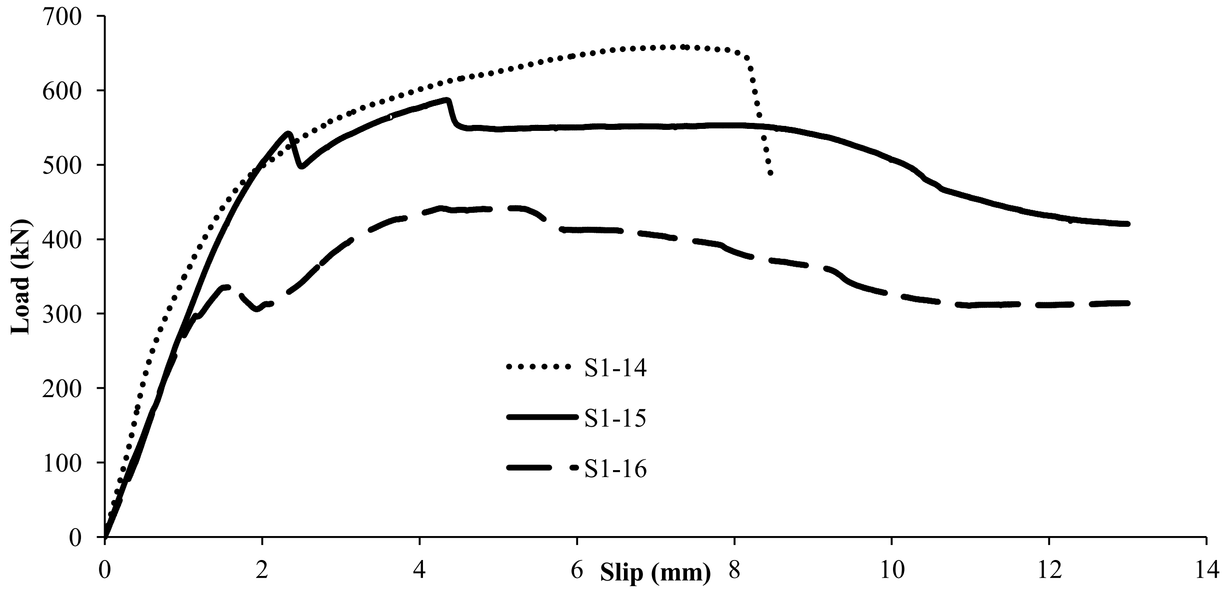


Figure 4.23 Load vs. slip graph for Group 8 headed specimens

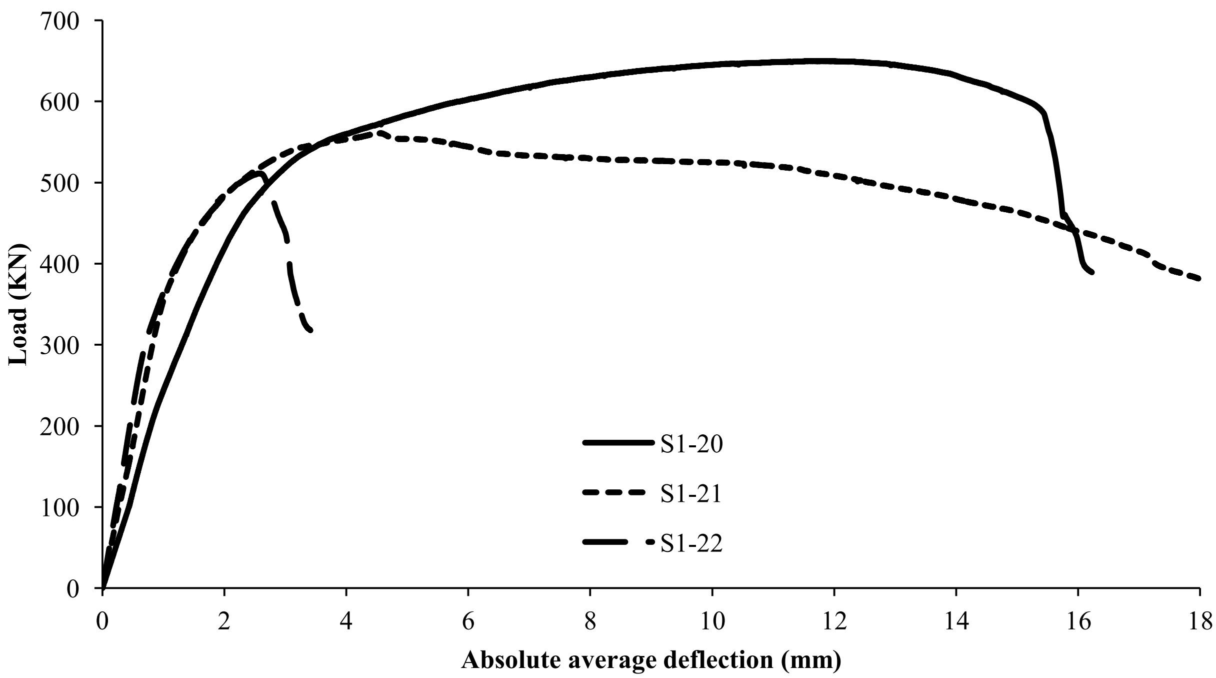


Figure 4.24 Load vs. slip graph for Group 8 unheaded specimens

Consistent with the previous findings, results of this group also showed that as the elevation of the reinforcement increased, the ultimate load of the specimen decreased for specimen groups with both headed and unheaded studs. Headed stud specimens all had the failure mode of concrete failure with deformed studs as can be seen in Figure 4.25. Specimens S1-22 and S1-20 in the unheaded stud specimens group had a similar failure mode where a single stud was sheared off. For a given height of stud, the reinforcement mesh placed at about 35% of the stud height resulted in a 15.8% higher ultimate load than the case where the mesh was placed at 75% of the stud height. It seems to suggest that the lower the reinforcement is, the higher the reached ultimate load of the specimen. Using the Eqn 2.7 set by Oehlers and Park (1992) h_a or the effective height of the shear stud is calculated as 40 mm and hence h_r or the reinforcement position should be 68 mm. This is in agreement with results obtained in this research.

(a)

(b)

(c)

(d)

(e)

Figure 4.25 Stud deformation-Group 7 specimens (a) S1-14, (b) S1-15, (c) S1-16, (d) S1-21, (e) S1-20

4.3.8 Group 8 - Specimens With Adjustable Studs

This group of specimens contains 100 and 150 mm adjustable studs with either protruding tips or nut in-flush with the shank end. The reinforcement mesh was placed at 70 mm from the steel flange for all specimens in this group. Test results are summarized in Table 4.8 while the load vs. slip responses are shown in Figure 4.26.

Table 4.8 Experimental results of Group 8 specimens

Specimen ID	Description	P_u (kN)	s (mm)	% Difference	Failure mode
S2-C-1	100 mm (no reinforcement)	492	2.31	-	Concrete failure/ no bending in stud
S2-1	100 mm	554	4.18	12.7	Slab shear-off/ 3 studs sheared-off
S2-2	100 mm/tip	537	3.99	9.2	Slab shear-off/ 3 studs sheared off
S2-3	150 mm	562	3.64	14.3	Slab shear-off/ 3 studs sheared-off
S2-4	150 mm/tip	497	3.61	1.1	Slab shear-off

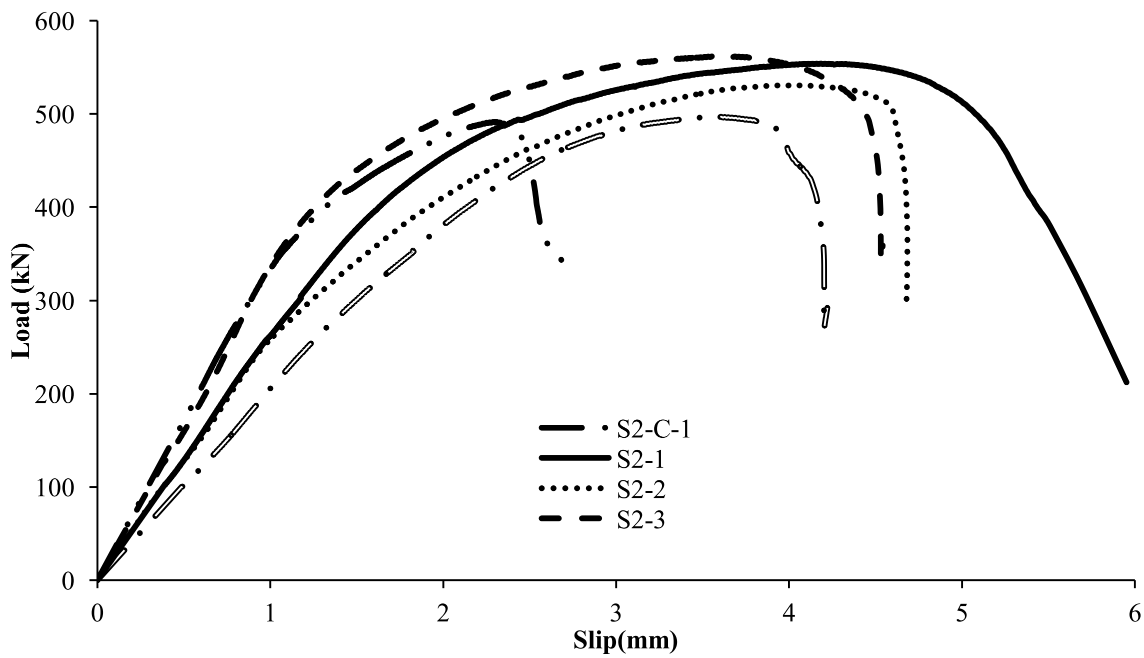


Figure 4.26 Load vs. slip graph for Group 8 specimens

For both stud heights tested, results showed that protruding tips resulted in a decrease in the ultimate load in comparison with specimens having the tip cut-off. For 100 mm adjustable studs, the specimen with nut in-flush with the shank end had a 3.2% higher capacity than its counterpart with a protruding tip. For 150 mm studs, this value was 13.1%. When the effect of stud length is concerned, the comparison of specimens without tip showed with 150 mm stud had a slightly higher ultimate load of 1.5% than the one with 100 mm stud. However, 150 mm stud with protruding tip resulted in an 8.0% lower ultimate load than the 100 mm counterpart. Figure 4.26 shows that all specimens with adjustable studs and reinforcement mesh exhibited similar behaviour where a slab was sheared off. Specimen S2-C-1 without reinforcement mesh failed by concrete crushing with a smaller slip at ultimate.

4.4 Discussion and Comparison

This section presents comparison and discussion of results focusing on the effects of parameters of stud height and reinforcement mesh elevation, headed vs. unheaded studs, and conventional vs. adjustable studs.

4.4.1 Stud Height and Reinforcement Mesh Elevation

Table 4.9 summarizes the effect of stud height on the ultimate load capacity in association with the position of the reinforcement mesh. It shows that for a given stud height, an increase in the elevation of reinforcement mesh resulted in a decrease in the ultimate load. The effect of stud height needs to be considered in combination with the steel mesh position. For the reinforcement mesh placed at 70 mm, an increase in the stud height from 100 to 150 mm did not show any marked difference in the ultimate load. When the stud height increased to 200 mm, the ultimate load showed a decrease in the ultimate load. Noting that reinforcement mesh intercepts the studs for all the mesh positions in this case, it seems to suggest that the increase in the stud height may result in a reduction in the ultimate load. When the reinforcement mesh is placed at 150 mm, a reduction in ultimate load is observed as the stud height increased from 100 to 150 mm.

Noting that reinforcement mesh does not intercept the stud in both cases, the stud height increase also results in a reduction in ultimate load. A higher ultimate load for 200 mm stud height is simply the result of the interception of the stud by reinforcement mesh. This general trend is reconfirmed for the case of reinforcement mesh placed at 200 mm, where an increase in stud height resulted in a decrease in ultimate load. Figure 4.27 shows a graph of specimens with reinforcement mesh positioned at 70 mm with varying stud heights and Figure 4.28 shows a graph of load vs. stud heights with different stud elevations. The numbers in bracket indicate the height of the stud and the reinforcement layer position in mm measured from the steel flange.

Table 4.9 Stud height and reinforcement mesh position comparison

Position of reinforcement mesh above the flange (mm)	**P_u (kN)**			
	Stud height (mm)	**100**	**150**	**200**
70		706	709	658
150		562	525	587
200		559	473	442

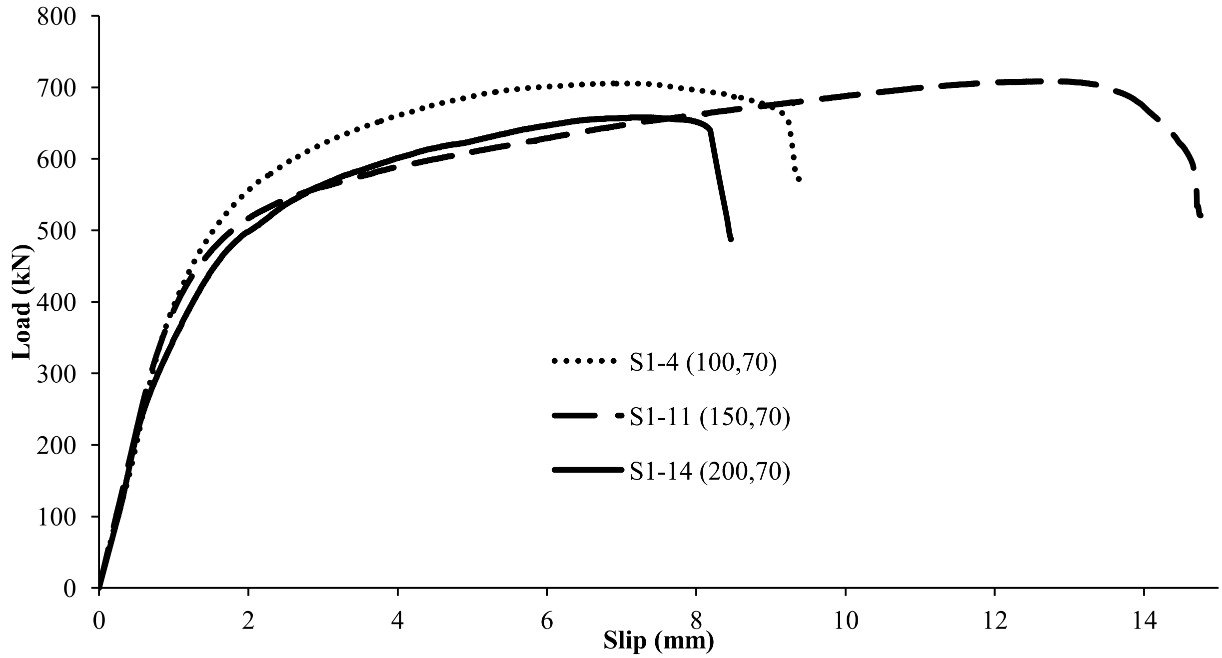


Figure 4.27 Load vs. slip graph for specimens with reinforcement mesh placed at 70 mm from flange surface

4.4.2 Headed vs. Unheaded Studs

Table 4.10 compares ultimate load results of conventional headed and unheaded shear studs for 150 and 200 mm long studs. Figures 4.28 and 4.29 compare load vs. slip graphs for headed and unheaded specimens. The first letter in bracket indicates whether it is a headed specimen (Y) or unheaded (N) which is followed by the height of the studs and the position of the reinforcement in mm. All specimens failed by concrete failure except for the 150 mm headed specimens with reinforcement at 70 mm where they had slab sheared off. When the mesh is placed at 70 or 150 mm from the flange, the headed shear studs resulted in slightly higher ultimate loads than unheaded studs provided that the mesh intercepts or is in flush with the stud head. This increase in the ultimate load is in average 3.7% for specimen with intercepting mesh. When the reinforcement mesh is placed at 200 mm above the flange, specimens with 150 mm long studs showed practically the same ultimate load with either headed or unheaded studs. However, for 200 mm long stud, the ultimate load attained by specimen with unheaded studs was higher than that with headed studs by 15.6%. This seeming abnormality may be a result of irregularity occurred in experimental testing. In general, it can be concluded that the headed studs can attain higher ultimate loads than unheaded studs with reinforcement mesh intercepting the stud height. This finding is consistent with observations made by Badie et al. (2002) where he attained a 17% higher load in a specimen containing fully headed shear studs studs in comparison with a specimen that contained alternating headed and unheaded studs.

Table 4.10 Headed and unheaded studs comparison

Position of reinforcement wire above the flange (mm)	Stud height (mm) (headed stud)		Stud height (mm) (unheaded stud)	
	150	200	150	200
70	709	658	675	650
150	525	587	474	561
200	473	442	477	511

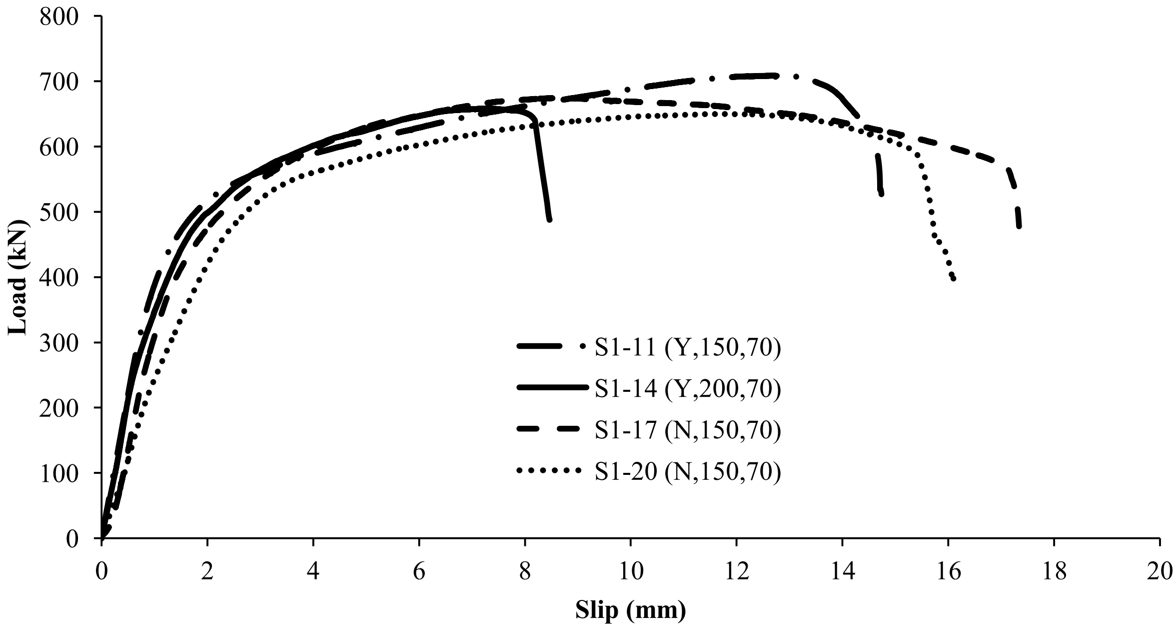


Figure 4.28 Load vs. slip graph for headed and unheaded specimens with reinforcement placed at 70 mm from the bottom

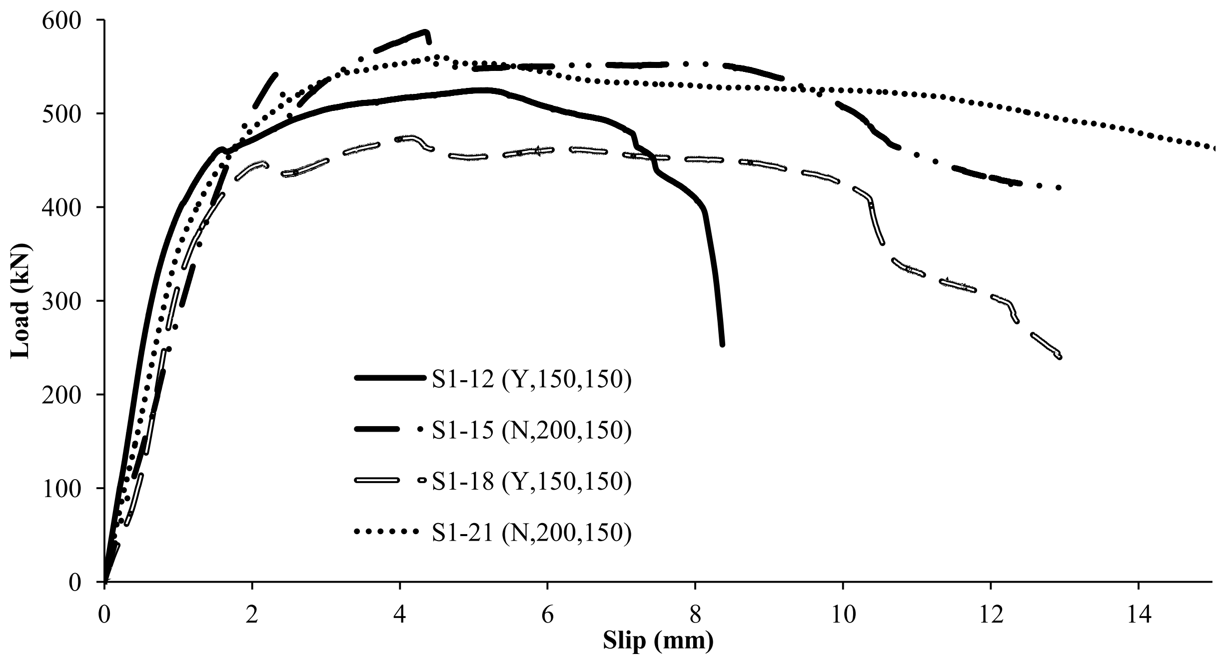


Figure 4.29 Load vs. slip graph for headed and unheaded specimens with reinforcement placed at 150 mm from the bottom

4.4.3 Conventional vs. Adjustable Studs

Table 4.11 compares the ultimate load of adjustable studs with conventional headed studs and Figure 4.30 compares the load vs. slip graph for adjustable and conventional studs. The specimens used in comparison all have reinforcement mesh placed at 70 mm from the steel flange. The specimens with adjustable studs having nut in-flush with the shank end were used in the comparison since they achieved a higher load than the specimens with studs having a protruding tip. All specimens failed by slab shear off except for the specimen with 100 mm studs, where it had a single stud shear-off and concrete failure with deformed studs.

Table 4.11 Comparison of conventional and adjustable studs

Stud height (mm)	**P_u (kN)**	
	Conventional headed stud	**Adjustable stud (Cut-off tip)**
100	706	554
150	709	562
Avg	708	558

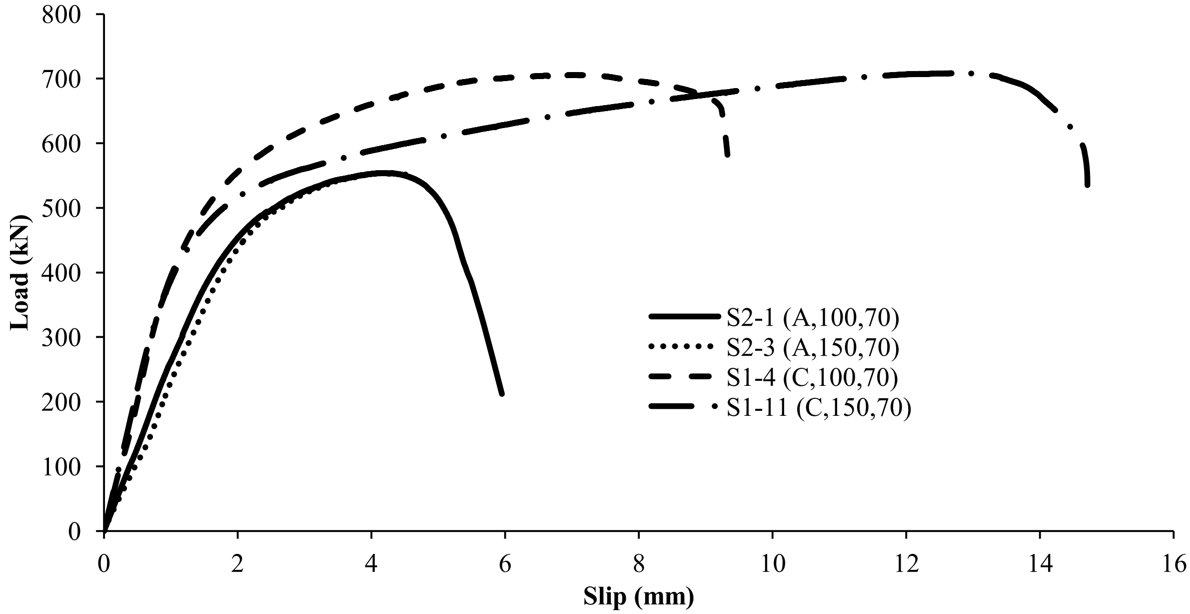


Figure 4.30 Comparison of load vs. slip graph for specimens with conventional and adjustable studs

The conventional studs achieved an average of 26.8% higher loads than their adjustable counterparts for both stud heights studied. The ratio of conventional to adjustable specimens ultimate loads is 1.27. This may be attributed partially to the fact that adjustable stud shank is threaded thus reducing the effective cross-sectional area. The diameter of the stud without considering the threads is 19.0 mm. The ratio of cross-sectional area between the conventional and adjustable stud is 1.37. The size of the head of the studs is also different. The nut used as the head of the adjustable studs was bigger and thicker than the conventional studs head. The nut has a maximum 38.8 mm width and 21.8 mm thickness as opposed to a 34.6 mm diameter and 10.0 mm thickness in the conventional studs.

Failed shear studs of the adjustable type did not show significant plastic deformation in comparison with conventional headed studs as shown in Figure 4.31 and 4.32. It can be seen from Figure 4.31 that the adjustable stud had concrete remains still embedded inside the threads.

(a) (b)

Figure 4.31 Sheared off studs (a) Headed stud, (b) Adjustable stud

(a) (b)

Figure 4.32 Stud shear off (a) Headed stud, (b) Adjustable stud

Steel coupons obtained from adjustable and conventional studs revealed distinctively different mechanical properties of two materials. The adjustable stud material showed 25.3% higher yield and 23% higher ultimate strengths than the conventional studs. The ratio of ultimate tensile strength of the adjustable studs to the conventional studs is 1.24.

Summary of steel coupons test results can be found in Section 4.5.2. The failure surfaces of the shear studs can be seen in Figure 4.33. The higher material strength was still offset by the reduction in the shank cross-sectional area of the adjustable stud. This has resulted in an overall lower strength of the specimens with adjustable studs.

(a) **(b)**

Figure 4.33 Shear studs failure surfaces (a) Adjustable studs, (b) Conventional studs

4.4.4 Comparison With the Code Value

4.4.4.1 Results from this Study

Code strength values were estimated for both conventional studs and adjustable studs using Eqn (2.2) as specified in CSA-S06 (2006). It is noted that since no design equation is available for adjustable studs, Eqn (2.2) for conventional studs was used in this case. In the calculation of stud tensile strength, a diameter of 22.2 mm was used for conventional studs whereas 19.0 mm was used for adjustable studs. Concrete properties f'_c and E_c were estimated as 45 MPa and 30.2 GPa respectively. Since the modulus of elasticity of the concrete was not measured in lab experiments for this research, a value was estimated using the equation found in the Concrete Handbook. The Modulus of Elasticity of the concrete will be $4500\sqrt{45}=30.2$ GPa. This value was used in Eqn (2.2) and Eqn (2.3). Data given from the Portland Cement Association and MichiganTech using ASTM C469 test method, established that high performance concrete has a minimum value of 40 GPa. F_u for the studs was taken as the average value obtained from coupon test. The average ultimate tensile strengths for the conventional and adjustable studs were evaluated as 478.5 MPa and 594.1 MPa respectively. For the adjustable type studs, the concrete strength governed. While tensile strength of stud governs the design strength for the conventional studs. Therefore, the design strengths for conventional and adjustable studs are determined to 742.8 and 660.8 kN without ϕ_{sc} term and 594.2 and 528.6 with ϕ_{sc} term.

The model given by Oehlers and Johnson in Eqn (2.3) was also used to estimate the failure load of the specimens. The n_r term was set as 4, assuming all four shear studs share the load equally in the specimen. This leads to a K factor of 3.75. Using the calculated K factor, the shear resistance of a specimen is estimated at 587 kN and 486 kN for conventional and adjustable studs respectively.

Table 4.12 shows a summary of comparison between test results and code design value along with Eqn (2.3) and Figure 4.35 illustrates $P_{u,exp}/P_{u,code}$ for all specimens. Specimens S1-23 and S1-24 represent the average of the two specimens tested. In Figure 4.34, the control specimens (S1-C-1 to S1-D-3) are indicated in the graph with a triangle and the adjustable stud specimens are indicated by a dash. The remaining specimens are indicated by a diamond shape. Out of 33 specimens, only two specimens, S1-D-3 and S1-8 attained loads higher than the code predicted value without ϕ_{sc} factor with $P_{u,exp}/P_{u,code}$ ratios being 1.04 and 1.12 respectively. Specimen S1-D-3 had two layers of reinforcement in the slab placed at code suggested position (70 mm from the top and 50 mm from the bottom); This shows that the code equation is only applicable for specimens with steel mesh present and placed at code suggested position. Specimen S1-D-2 having two layers of reinforcement below the head of the stud achieved 97% of the code required load. Comparing specimen S1-8 (ungreased flanges) and specimens S1-23 (rusted flange), the latter obtained about 99% of the code required value. Eqn (2.3) gives a better estimate of the specimens ultimate load than the code equation, especially the adjustable stud specimens. All adjustable stud specimens achieved between 1.1 to 1.14 higher loads than calculated. Out of 33 specimens, 18 specimens achieved higher loads than Eqn (2.3). Specimen S1-18 with 150 mm unheaded studs and reinforcement at 150 mm had the lowest ratio of 0.78. Specimens having a single layer of reinforcement below the head of the stud all achieved ultimate load higher than calculated by the theoretical model, except for specimens S1-10 and S1-21 which had 100 mm shear stud spacing and 200 mm unheaded shear stud height respectively. However, the two specimens still achieved ratios above 0.9. Adjustable stud specimens all achieved higher loads than the theoretical model anticipated including the specimen S2-C-1 which had no reinforcement in the concrete slab. All specimens that achieved loads lower than 0.9 had the reinforcement layer placed either flush or above the head of the stud, with the exception of specimen S1-C-2 which had an unheaded 100 mm shear stud with no reinforcement. On average, the specimens achieved 0.81 and 1.04 ratios using the code value and the theoretical model respectively. Both Equations resulted in a similar coefficient of variation of almost 16%. This shows that the theoretical model is more suited to estimate the ultimate load of the specimens.

Specimens with single layer reinforcement mesh intercepting the stud height achieved about 90% of the code value with a similar failure mode where one or more studs were sheared off. Those specimens with single layer reinforcement mesh placed above the studs achieved about 70% of the code value on average. It suggests that it is critical to have reinforcement mesh intercepting the stud height at the limit that Oehlers and Park (1992) suggested.

Table 4.12 Comparison of test results and code value

Specimen ID	$P_{u,exp}$ (kN)	$P_{u,exp}/P_{u,Code}$ with ϕ_{sc}	$P_{u,exp}/P_{u,Code}$ without ϕ_{sc}	$P_{u,exp}/P_{u,Eqn\ 2.3}$
S1-C-1	567	0.95	0.76	0.96
S1-C-2	479	0.81	0.65	0.82
S2-C-1	492	0.93	0.74	1.01
S1-D-1	567	0.95	0.76	0.97
S1-D-2	720	1.21	0.97	1.23
S1-D-3	769	1.29	1.04	1.31
S1-4	706	1.19	0.95	1.20
S1-5	676	1.14	0.91	1.15
S1-6	562	0.95	0.76	0.96
S1-7	559	0.94	0.75	0.95
S1-8	835	1.41	1.12	1.42
S1-9	616	1.04	0.83	1.05
S1-10	550	0.93	0.74	0.94
S1-11	709	1.19	0.95	1.21
S1-12	525	0.88	0.71	0.89
S1-13	473	0.80	0.64	0.81
S1-14	658	1.11	0.89	1.12
S1-15	587	0.99	0.79	1.00
S1-16	442	0.74	0.60	0.75
S1-17	675	1.14	0.91	1.15
S1-18	460	0.77	0.62	0.78
S1-19	477	0.80	0.64	0.81
S1-20	650	1.09	0.87	1.11
S1-21	561	0.94	0.76	0.96
S1-22	511	0.86	0.69	0.87
S2-1	554	1.05	0.84	1.14
S2-2	537	1.02	0.81	1.11
S2-3	562	1.06	0.85	1.16
S2-4	497	0.94	0.74	1.02
S1-23	740	1.25	0.99	1.26
S1-24	699	1.18	0.94	1.19
		Avg	0.81	1.04
		COV	16.2	16.1

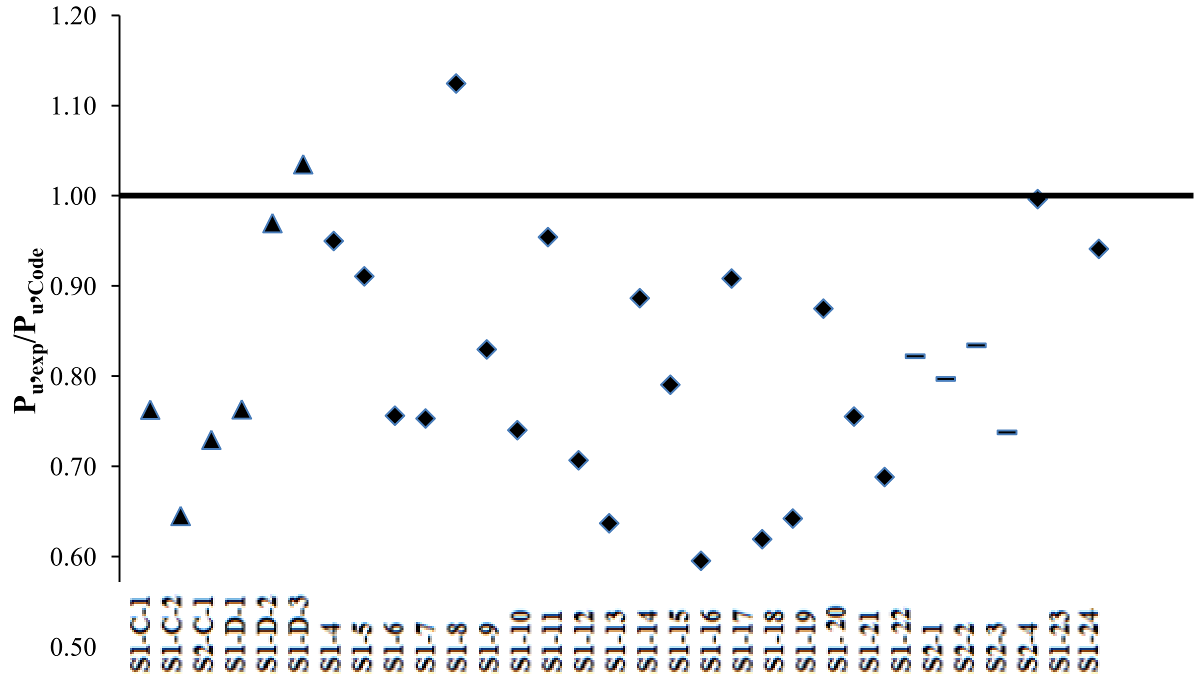


Figure 4.34 Results of ultimate loads in comparison with code estimated values without ϕ_{sc}

4.4.4.2 Results from Other Studies

An and Cederwall (1996) tested 8 specimens, including 4 with normal-strength (NSC) and 4 with high-strength concrete (HSC). The code values calculated were calculated for both concrete types using Eqn (2.2). The authors used 19 mm shear studs with ultimate strength of 519 MPa. The NSC had a compressive strength and modulus of elasticity of 31.0 MPa and 27.1 GPa respectively. The HSC had a compressive strength and Modulus of Elasticity of 85.0 MPa and 34.1 GPa respectively. A summary of the test results is shown in Table 4.13. When two layers of reinforcement are used they are both placed below the head of the stud as shown in Figure 4.35.

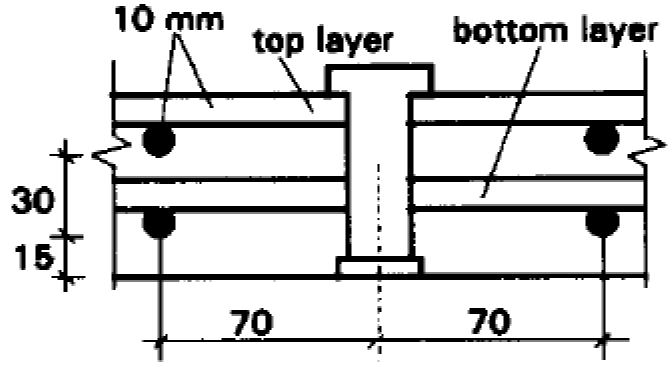


Figure 4.35 Reinforcement layers configuration of An and Cederwall (1996) test specimens

As can be seen from the results, none of the normal-strength concrete specimens reached ultimate loads higher than the code value. As stated by the authors, all the normal-strength concrete specimens failed due to concrete failure, where the high-strength concrete specimens failed by slab shear-off. All high strength specimens achieved at least 18% higher load than the code value. It is clear that the concrete strength has a high influence on the ultimate load of the specimen. This correlates with the results obtained from this research that the concrete failure led to the lowest ultimate load in the corresponding group and lower than the code value.

Table 4.13 Test results from An and Cederwall (1996)

Specimen designation	Description (number of reinforcement layers)	$P_{u,exp,}$ (kN)	$P_{u,\,code}$ (kN)	$P_{u,exp}/P_{u,code}$
NSC11	1	131.1	145.2	0.90
NSC12		127.1	145.2	0.88
NSC21	2	137.7	145.2	0.95
NSC22		135.8	145.2	0.94
HSC11	1	178.8	147.2	1.22
HSC12		180.8	147.2	1.23
HSC21	2	173.2	147.2	1.18
HSC22		183.5	147.2	1.25
			Avg	1.06

Table 4.14 shows a summary of test results obtained by Xue et al. (2008). The second column in the table shows description of some of the parameters used by the authors in their research. The plus sign indicates that the stud is lengthened and the stud height is larger than that of the corresponding standard stud as stated by the authors. Figure 4.36 shows the dimension of the studs used in mm.

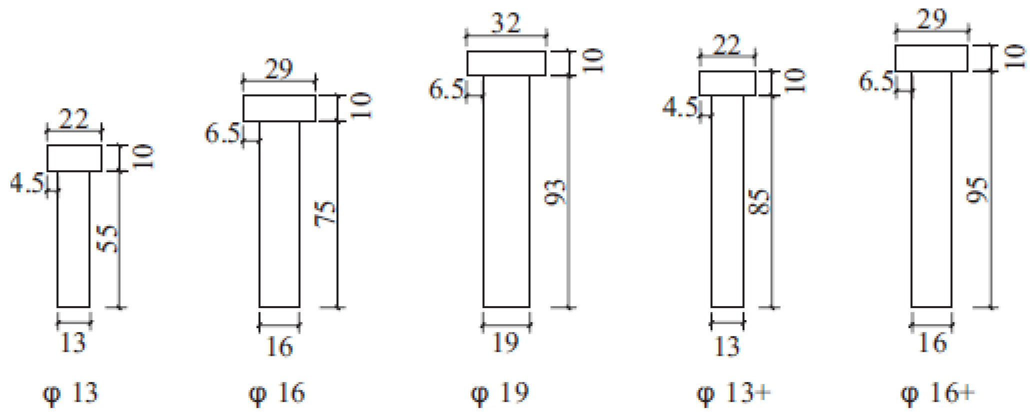


Figure 4.36 Dimensions of studs as used in testing by Xue et al. (2008)

All tested specimens failed due to a shear stud failure except specimen 18 which had a concrete slab failure and was placed separately in Table 4.14. Specimens 4-6 and 7-9 had different size steel sections. As stated by the authors that the lengthened studs were inferior to the normal studs as seen by comparing specimens 1-3 with 19-20 and 7-9 to 25-26. This is consistent with findings in this research regarding the effect of the stud height. The specimens reached an average 1.05 of the code required value ranging from 0.8 to 1.39. Test results show that the ultimate load was anticipated for studs using the code equation with smaller diameters and concrete strength of 50 MPa; But as the shear studs got bigger and the concrete strength was reduced from 50 MPa to 30 MPa the code value was higher than the lab results.

Table 4.14 Test results from Xue et al. (2008)

Specimen designation	Description (stud diameter in mm, concrete strength in MPa, number of transverse reinforcement layers)	$P_{u,exp,}$ (kN)	$P_{u,\ code}$ (kN)	$P_{u,exp}/P_{u,code}$
1-3	13,50,2	163.9	117.8	1.39
4-6	16,50,2	164.9	178.7	0.92
7-9	16,50,2	212.8	178.7	1.19
10-12	16,30,2	154.3	178.7	0.86
13-15	19,50,2	221.9	252	0.88
16-17	19,30,2	201.0	252	0.80
18	19,30,2	207.9	252	0.83
19-20	13+,50,2	147.7	117.8	1.25
21-22	13+,50,4	147.1	117.8	1.25
23-24	13+,30,2	137.4	117.8	1.17
25-26	16+,50,2	177.4	178.7	0.99
			Avg	1.05

Badie et al. (2002) tested push-off specimens including specimens with 31.8 mm (1.25 in) studs. Table 4.15 shows the achieved ultimate load of the specimens along with a ratio of the achieved load over the code value. Push-off tests include one concrete slab subjected to a shear force after fixing the beam horizontally as shown in Figure 4.37. Specimens with half headed and unheaded studs had a reduction of 17% of ultimate load over the fully headed specimens as seen from comparing Group 3 and Group 4 results.

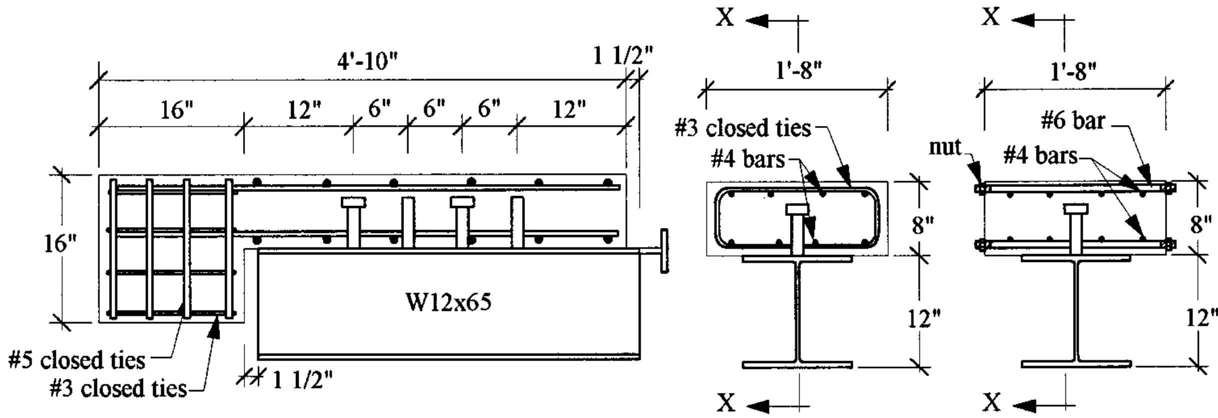


Figure 4.37 Group 5 specimens as tested by Badie et al. (2002)

It is seen from the table that none of Groups 1 to 4 reached ultimate loads higher than the code value. However, when the authors decided to change the close ties in the slab to the threaded bars in Group 5-b, they were able to reach exactly 100% of the code value although they were using alternating headed and unheaded studs. Groups 2 and 3 almost achieved double the ultimate load of Group 1 in spite of using half the number of shear studs in Group 1. Hence the conclusion could be drawn that one 31.8 mm stud can replace two 22.2 mm studs conservatively as was stated by the authors.

Table 4.15 Test results from Badie et al. (2002)

Specimen Designation	Description (# of studs, size of studs in mm)	$P_{u,exp,}$ (kN)	$P_{u,\ code}$ (kN)	$P_{u,exp}/P_{u,code}$
Group 1	8, 22.2	127.2	130.8	0.97
Group 2	4, 31.8	242	349.2	0.69
Group 3	4, 31.8	231	349.2	0.66
Group 4	4, 31.8 (alternating headed and unheaded studs)	198	349.2	0.57
Group 5-a	4, 31.8 (alternating headed and unheaded studs, with closed ties)	297	349.2	0.85
Group 5-b	4, 31.8 (alternating headed and unheaded studs, with threaded bars)	349	349.2	1.00
			Avg	0.75

In summary, the comparison between code values and test results from this study and other studies shows that in some cases, the code equation overestimates the stud strength. Noting that the code equation considers the concrete and stud strength separately, a model considering the interaction of concrete and stud may provide improved estimate of the stud strength.

4.5 Auxiliary Test Results

4.5.1 Concrete Cylinders

Push-out specimens were poured in two batches. The first batch contained 17 specimens and 15 concrete cylinders were cast. The second batch contained 16 specimens and 15 cylinders were cast. Three cylinders were tested after 7 days then at 10 and 14 days after the push-out specimens were tested. The compressive strengths of the cylinders are summarized in Table 4.16. The tested concrete cylinders can be seen in Figure 4.38. The increase in concrete compressive strength with time is graphed in Figure 4.39.

Figure 4.38 Tested concrete cylinders

Table 4.16 Cylinder tests results

Batch number	Time elapsed before test (days)	Compressive strength (MPa)			Avg
Batch 1	7	40.5	41.2	40.0	40.6
	10	45.3	47.9	43.9	45.7
	14	48.5	48.6	47.6	48.2
Batch 2	7	42.3	41.1	39.8	41.1
	10	46.6	45.2	44.3	45.4
	14	47.5	48.6	47.7	47.9

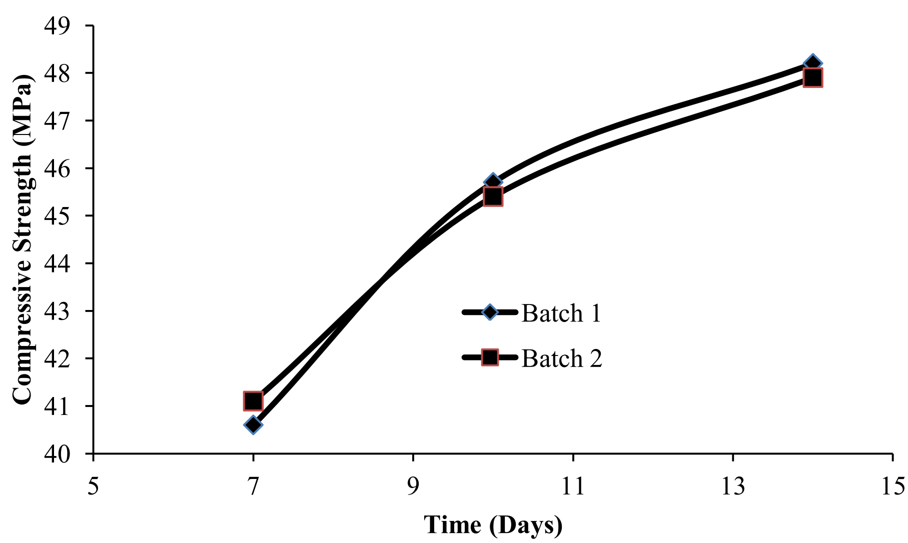


Figure 4.39 Compressive strength of concrete vs time

4.5.2 Steel Coupons

A total of 12 coupons were tested according to ASTM E8 (2012) standards. Six stud coupons were tested, where three were formed from the conventional studs and three were formed from the adjustable studs. Six rectangular coupons were cut from the flange and the web of the steel section, three from each. The test results are summarized in

Table 4.17 and stress vs. strain diagrams for the steel coupons can be seen in Figures 4.40 and 4.41.

Table 4.17 Summary of steel coupons test results

Coupon type	Specimens ID	Description	Ultimate Stress (MPa)	Yield Stress (MPa)	Modulus of Elasticity (GPa)
Round tensile coupons (Shear studs)	H-1-1	Conventional stud coupons	475.1	366.8	185.8
	H-1-2		492.3	379.0	181.3
	H-1-3		468.2	359.9	193.3
	Avg		478.5	368.6	186.8
	H-2-1	Adjustable stud coupons	593.0	467.1	206.0
	H-2-2		596.4	467.1	173.4
	H-2-3		592.9	475.7	205.6
	Avg		594.1	470	195.0
Rectangular tensile coupons (steel section)	E-1-1	Flange coupons	437.5	363.0	-
	E-1-2		442.8	360.9	-
	E-1-3		469.8	414.0	186.1
	E-2-1	Web coupons	521.7	429.3	-
	E-2-2		515.7	421.9	-
	E-2-3		418.5	308.7	183.4

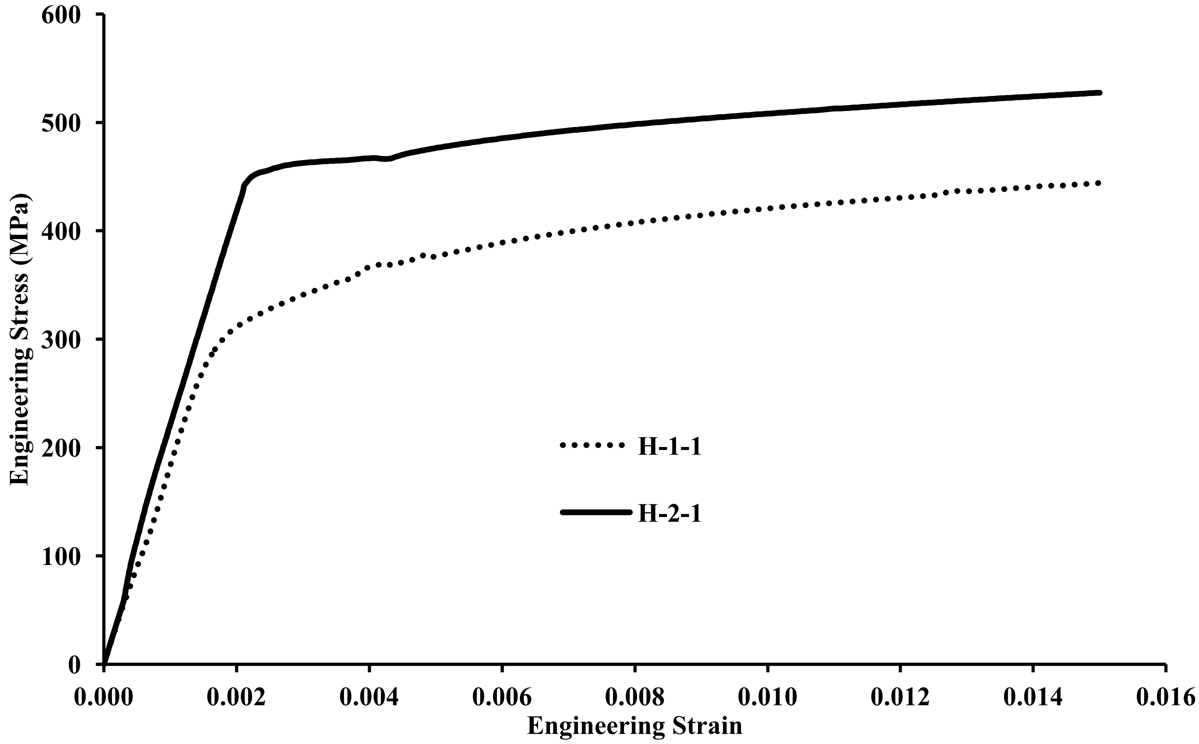


Figure 4.40 Stress vs. strain graph of specimens H-1-1 and H-2-1

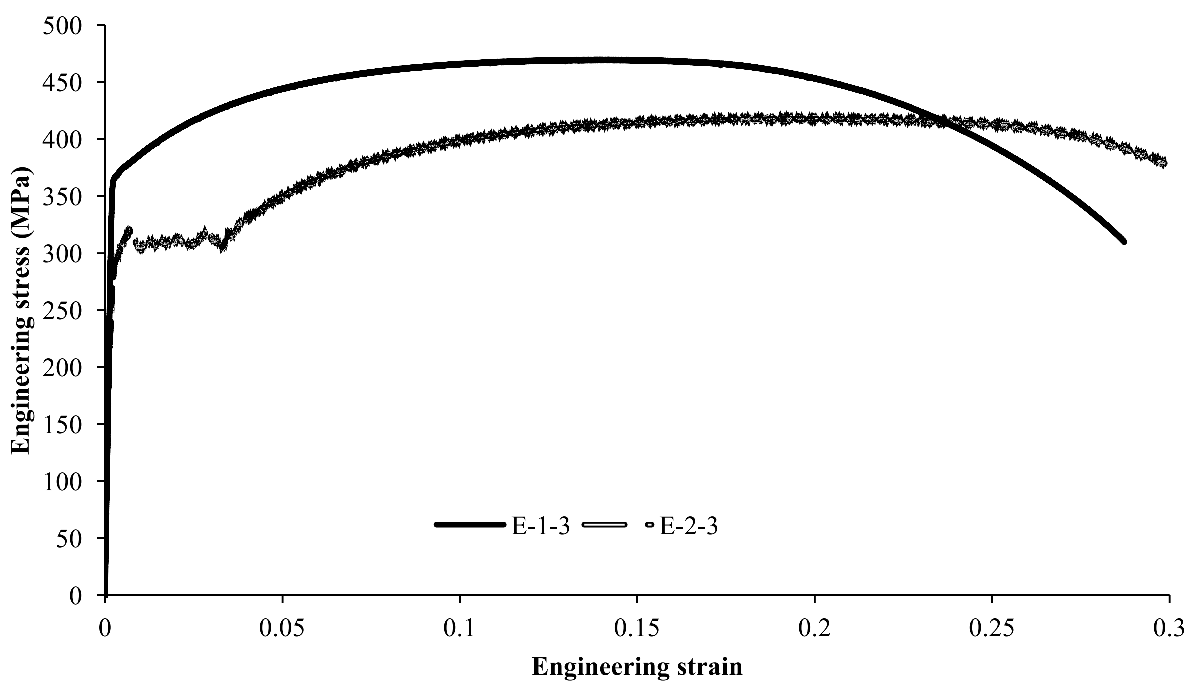


Figure 4.41 Stress vs. strain graph of specimens E-1-3 and E-2-3

CHAPTER 5 CONCLUSION

5.1 Summary

This study was carried out to investigate the behaviour of shear stud connectors in composite bridge deck application. The effects of several parameters including reinforcement position, shear stud spacing, shear stud height, beheading of shear studs, and steel flange surface treatment are the focus of the study. As an alternative to conventional studs, the performance of adjustable studs was also studied.

The experimental program involved the testing of 33 push-out tests. All specimens contained W250x28 steel sections with two 22.2 mm (7/8-inch) shear studs on each flange. The shear studs were embedded within 45 MPa concrete slabs. Except for control specimens, all specimens contained single or double layers of 15-M bars used as reinforcement mesh. Specimens were loaded by means of an Instron machine until the specimens failed to take any more load. During testing, the applied load and slip were measured and recorded up to failure. Specimens were compared by means of load vs. slip graph, ultimate load, and failure mode of each specimen.
Test results were used to assess the efficacy of the design equation for shear strength of studs as well as the detailing requirement specified in CSA S6 (2006).

5.2 Conclusions

Conclusions from this study are presented as follows:

1. Failure modes depended on the reinforcement position in the slab. Concrete failure was more common in specimens with reinforcement mesh placed above the head of the stud or no reinforcement at all. Stud shear-off failure and yielding of shear studs were more common in specimens with reinforcement intersecting the shear studs.

2. The ultimate load of the specimen is sensitive to the reinforcement mesh position. The reinforcement mesh has to intercept the shear stud to effectively confine the concrete around the studs and allow the shear studs to bend then fail by shear-off. Having a single layer of reinforcement above the head of the stud, does not increase the ultimate load or slip in comparison to having no reinforcement at all. However, when having two reinforcement layers, the top reinforcement is important to prevent cracks from propagating into the top region of the slab causing a premature failure while the bottom layer confines the concrete around the stud allowing it to reach its maximum shear capacity.

3. Testing show that decreasing the shear stud spacing results in lower ultimate load and more severe concrete failure. Where even with the addition of reinforcement mesh intercepting the shear studs they did not achieve significant bending and slip.

4. Different flange treatment has an impact on the ultimate load of a specimen. Specimens with a rusted flange had a 5.9% increase in ultimate load over specimens with a coated flange. Specimens with greased flanges had a reduced ultimate load of 15.5% in comparison to an ungreased flange.

5. Specimens with adjustable studs were weaker than conventional studs with an average of 26.5% decrease in the ultimate load. This reduction in capacity is attributed to the reduced cross-sectional shank area of the adjustable studs.

6. When reinforcement mesh intercepted the studs, specimens containing headed studs had 5.4% increase in ultimate load over specimens containing unheaded studs. However, the difference in ultimate load was almost negligible when the mesh was placed above the studs.

7. Test data show that as shear studs increased in height the shear stud capacity of the specimen decreased. This trend is also seen in the adjustable studs where

specimens with without a protruding tip led to an average increase of 8.2% over specimens with a protruding tip.

8. Out of 33 specimens, only two obtained higher loads than the code suggested value. It shows that having a double layer of reinforcement mesh placed at the code suggested elevation in the specimen is essential in achieving the design strength and the friction between the concrete and the steel flange contributes significantly to the stud strength.

9. The theoretical model suggested by Oehlers and Johnson (1987) gave a better estimate for the specimens than the code value found in CSA-S06.

5.3 Recommendations for Future Research

1. Additional experimental testing should be carried out in large-scale testing to further investigate the effect of reinforcement position, flange surface treatment, and unheaded shear studs on the ultimate load capacity of the beam and slip at ultimate loading.

2. Test the effect of adding fibres in the concrete mix design to prevent premature concrete failure in the cases of misplaced reinforcement mesh.

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