CHAPTER 1

INTRODUCTION

1.1 Background

Composite construction using steel and concrete has been used since the early 1920s. It gained widespread use in bridges in the 1950s and in buildings in the 1960s (Ollgaard et al 1971). In composite beams, the steel and concrete are joined by mechanical connections, the most popular form being welded headed shear studs. The shear studs are welded to the flange of the steel beam, generally through composite steel deck. A concrete slab is cast on top of the deck with the stud, functioning to tie the slab and beam together as a unit. A composite beam, which is shown in Fig. 1.1a, has greater strength and stiffness than if the beam and slab were behaving independently.

The use of formed steel deck in composite construction has been common since the 1960s. In the US, the shear studs are typically welded through the deck to the steel beam. The steel deck is useful because it acts as formwork for the concrete slab as well as tensile reinforcement. A composite beam using steel deck is shown in Fig. 1.1b.

When studs are used in design, one must be able to predict their ability to resist the longitudinal forces that arise between the steel and concrete. Hawkins and Mitchell (1984) expressed very well how difficult it is to predict the strength of studs by stating, "The analysis of the actions of an embedded stud shear connector near failure is very complex due to the inelastic deformations in the stud under the combined effects of shear, bending and tension and due to the inelastic deformations in the concrete surrounding the stud."

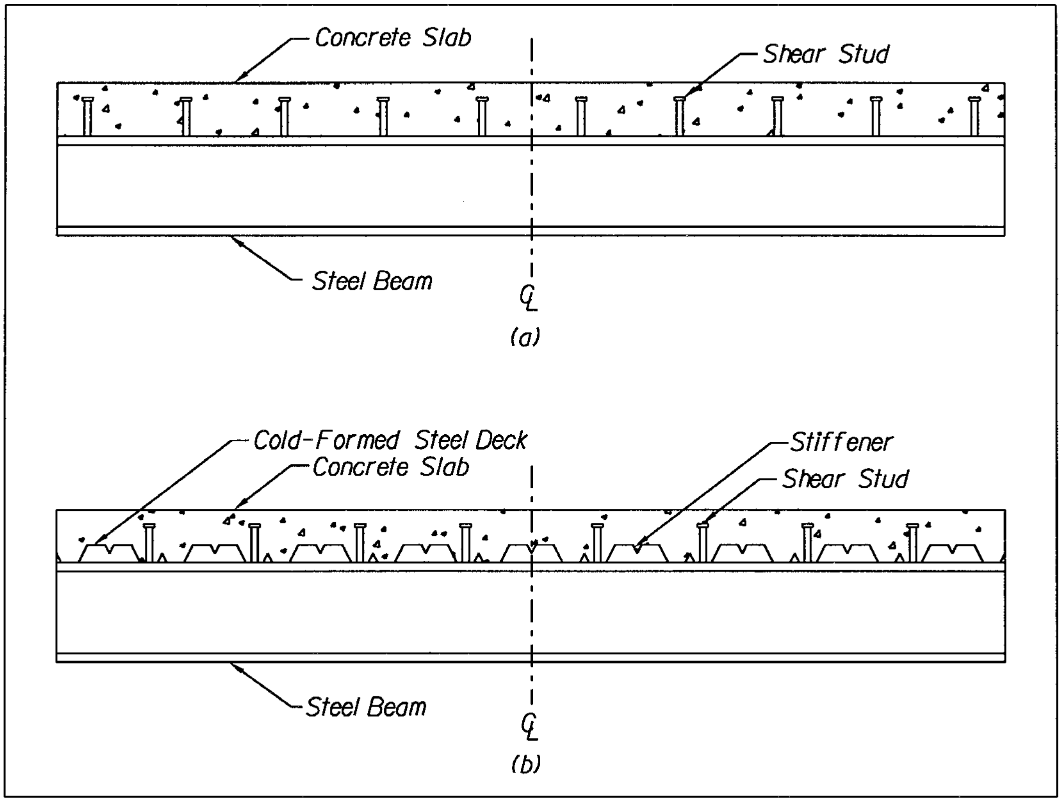

Fig. 1.1 Composite Beams Using Shear Stud Connectors:
(a) Non–Composite Solid Slab
(b) Composite Slab Using Cold–Formed Steel Deck

Strength prediction equations have predominantly been derived from empirical studies. Both push-out tests, which were first used in Switzerland in the 1930s (Davies 1967), and full-scale beam tests have been used to develop shear stud strength prediction expressions. Because of the large size and expense of beam tests, push-out tests are usually used to evaluate a wide array of parameters. A push-out test specimen is shown in Fig. 1.2. A photo of a push-out test is shown in Fig. 1.3. Beam tests are often used to verify the results of methods developed from push-out tests. It has been found that push-out test results can be used to accurately predict beam test results if the push-out tests are detailed similar to the beam test (Easterling et al 1993).

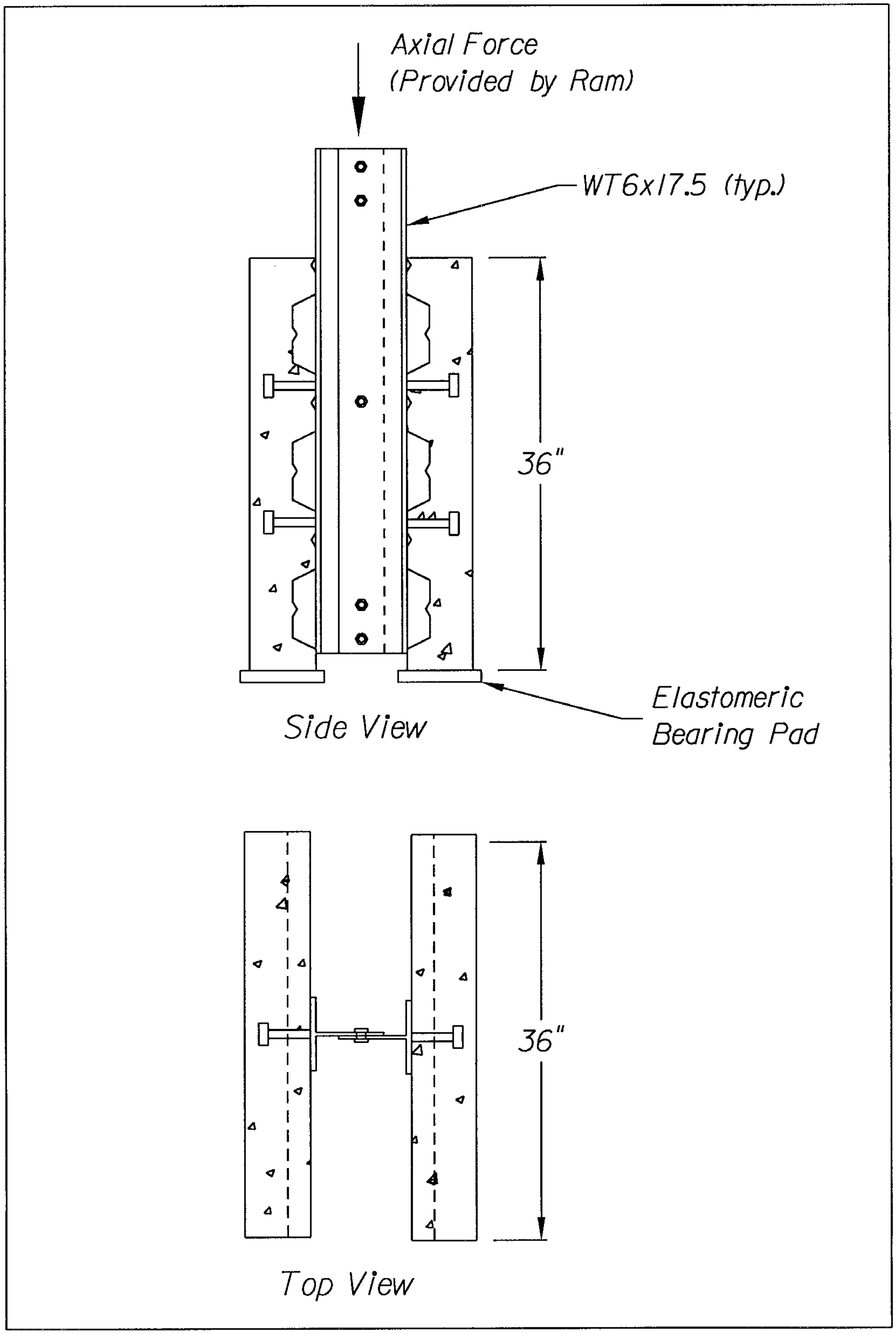

Fig. 1.2 Typical Dimensions and Details of a Push-Out Specimen at Virginia Tech

Fig. 1.3 Push-Out Test of a Composite Slab with Formed Steel Deck

Early shear stud strength prediction equations, developed in the 1960s and 1970s, were for solid slab construction. The equations were developed based on the results of push-out tests. The equations were modified for the use of steel deck in the late 1970s and were based on full-scale beam tests (Grant et al 1977).

Typical steel deck profiles used in the US have stiffeners in the center of the deck flanges. This results in the need for the studs to be placed off-center. The side of the stiffener that the stud is welded on (either toward the nearest end of the beam span or toward the centerline of the beam span) affects the strength of the stud. This is because the amount of concrete between the steel deck and the stud, on the stud's load-bearing side, is different, as shown in Fig. 1.4. The stud is deemed in the "strong position" if it is

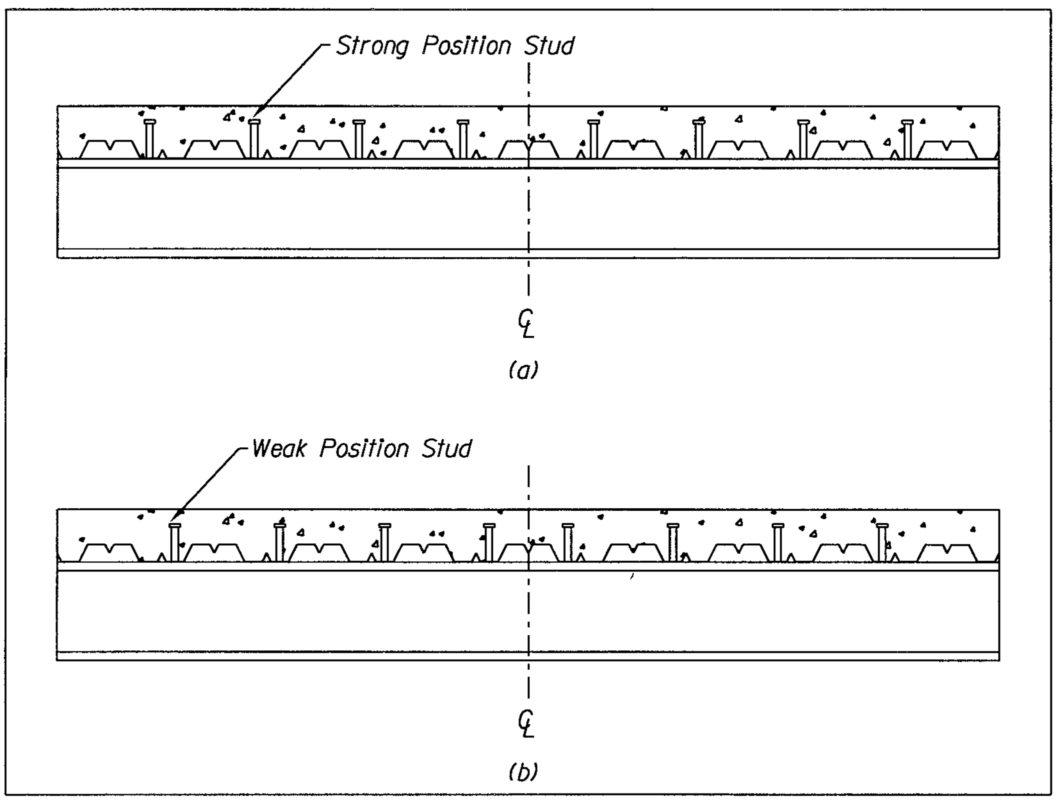

Fig. 1.4 (a) Strong Position Studs in a Composite Beam, (b) Weak Position Studs in a Composite Beam

placed nearer to the end of the beam span, and in the “weak position” if it is placed nearer to the middle of the beam span. The stud strength equations (Grant et al 1977) were developed from tests mostly using deck without stiffeners where the studs were welded in the center of the deck rib. The position of the stud in the rib is not considered in the present strength equations used in the American Institute of Steel Construction (AISC) specifications (*Load* 1993 and *Specification* 1989).

Experimental research performed around the world has shown that the existing shear stud strength prediction equations that are included in the AISC specification are unconservative. Also, there are several limitations that are placed on the use of these equations, such as deck height restrictions, that limit the use of composite constructions.

Issues related to the strength of welded shear studs have been studied at Virginia Polytechnic Institute and State University (VT). Over 300 push-out tests on welded headed shear studs have been performed, along with a limited number of beam tests. A brief summary of these tests, prior to the tests presented in Chapters 2 and 3, follows.

Easterling et al (1993) reported results from a series of push-out tests and composite beam tests. The authors showed that results from push-out tests can be used to accurately predict beam test results. All of the studs in the weak position failed by punching through the web of the steel deck. The authors concluded that weak position stud strength is a function of the steel deck strength. Weak position studs behaved in a more ductile manner than strong position studs. All of the studs in the strong position failed from concrete failure; none failed by shear. The results of the tests, which were all configured with no more than one stud in a rib, showed that the AISC equations for stud strength are unconservative. Strong position studs had strengths approximately 70% of predicted strengths using AISC specifications, while weak position studs had strengths only 60% of the predicted strength. The authors' review of previously published test data showed that the AISC equations were developed from tests primarily using pairs of studs. The authors recommended that, for single studs, the stud reduction factor not exceed 0.75; the studs should be placed in the strong position; and composite action should be at least 0.50. In the AISC specification (*Load* 1999), an upper limit of 0.75 was placed on the reduction factor for single studs as an interim solution to the unconservative nature of the equations. Note: Any reference to AISC in this study will be for the 1993 specification.

Lyons et al (1994) performed 48 solid slab push-out tests, to determine the effect of stud tensile strength and concrete properties on shear connections, and 87 push-out tests using steel deck. Variables included stud placement, height, and arrangement, as well as steel deck height and gage. The test results showed that weak position studs are very ductile and are influenced by the steel deck strength. The authors hypothesized that concrete strength and unit weight affect shear strength because they determine the stud deformation allowed before failure and that friction between the steel and concrete increases the apparent strength of the stud beyond the ultimate shear strength of the stud. The authors concluded that the current AISC equations are unconservative for shear connections with steel decks. The equations reported by Mottram and Johnson (1990) were found to be more conservative, but still insufficient for weak position studs. An upper limit on connector strength of $0.8A_sF_u$, where A_sF_u is the tensile strength of the stud, was recommended (Lyons et al 1994).

1.2 Literature Review

This review will begin with the strength prediction equations developed for welded headed studs in solid slabs and will include more recent research on welded headed studs in slabs with formed metal deck.

Chinn (1965) performed ten solid slab push-out tests and two beam tests using lightweight and normal weight concrete. His tests, which were similar to those by Viest (1956), used 1/2, 5/8, 3/4, and 7/8 in. diameter studs. Stud lengths were approximately four times the diameter. Flanges of the steel beams were greased before the concrete was placed to reduce the effect of friction. Chinn found that, based on two tests, cycling the load did not affect the slips, i.e., a specimen could be loaded to failure without unloading. Referring to Fig. 1.5, a "useful capacity" Q_{uc} was found at the intersection of the straight-

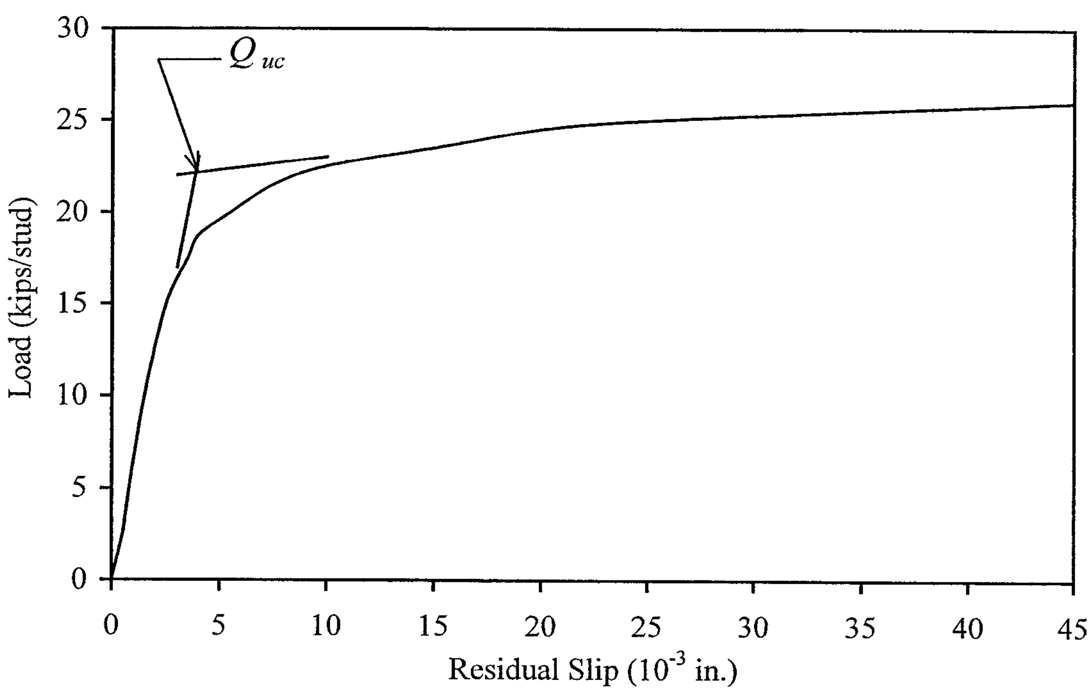

Fig. 1.5 "Useful Stud Capacity," Q_{uc}, as determined by Chinn (1965)

line lower part with the straight line projected backward, tangent to the upper part of the curve. The useful capacity, which occurred at a slip of about 0.015 in., followed the relationship

$$Q_{uc} = 6.5d^2 f_c' \sqrt{\frac{4000}{f_c'}} \qquad (1.1)$$

where Q_{uc} = useful stud capacity (k)

d = diameter of stud (in.)

f'_c = concrete strength (psi)

The failure mode was stud shearing in all specimens except the one with 7/8 in. diameter studs, which failed by slab cracking. The ultimate loads "did not appear to be affected by the concrete strength," and followed the relationship

$$Q_u = 39.22d^{1.766} \qquad (1.2)$$

where Q_u = ultimate stud capacity (k)

Chinn found that the ultimate stud strength from a push-out test was 18% to 43% higher than the direct shear strength. He concluded that "not all the force applied to the push-out specimen is transmitted through the section of the stud which shears. It is likely that some force is transferred by friction and some through the weld root below the section which shears." Chinn also performed two beam tests to confirm the push-out test results and found that the beams designed using results of the push-out tests behaved satisfactorily.

Slutter and Driscoll (1965) studied the ultimate strength design of composite beams. Their tests were evaluated using simple elastic theory and the theory of incomplete interaction between the concrete slab and the steel beam. Twelve 15 ft simple span composite beams, one two-span continuous beam, and nine push-out specimens were tested. Two of the beams were constructed without shear connectors to investigate the effect of the natural bond between the steel and concrete. One of these two beams and a beam with bent studs were loaded from below the steel beam. According to Chinn (1965), this prevented friction forces at the beam-to-slab interface from being developed. The push-out and beam test results were combined with results from two beam tests and 11 push-out tests from other sources to form an equation for the ultimate strength of stud shear connectors. This equation, which was developed for both bent and headed studs, is

$$q_u = 930 d_s^2 \sqrt{f_c^{'}} \tag{1.3}$$

where q_u = ultimate strength of stud shear connector (k)

d_s = diameter of shear connector (in.)

f'_c = concrete strength (psi)

This equation was deemed applicable for use with concrete strengths less than 4000 psi. Two beam tests without shear connectors and with identical parameters, except for the method of applying load, were conducted to investigate the role that bond and friction play in transmitting shear forces. Bond was not present in either test because concrete shrinkage caused bond failure. On one of the beams, the load was suspended from the steel beam so that only the concrete weight caused friction forces. The test was discontinued at a load of 20 k because the member separated. In the other beam test without shear connectors, the load was applied on top. Friction forces were probably developed because the beam carried a load of 41.5 k, giving it an ultimate moment of about 7% greater than its plastic moment. Tests on beams with shear connectors and loads suspended from the beam gave similar results when loads were applied on top of the beam. The authors concluded that the ultimate strength of a shear connector is related to the ultimate flexural strength of a beam; and that shear connectors can be uniformly spaced if there are an adequate number of connectors because the shear connection loads are redistributed.

Early tests by Davies (1967), in which 20 half-scale solid slab push-out tests were performed, included the number, spacing, and pattern of welded studs as variables. The specimens all used 3/8 in. x 2 in. welded studs. The flanges of the steel beam were greased. Standard studs, with two studs per flange placed perpendicular to the direction of the load, gave an ultimate load 25% more than those studs placed parallel to the direction of the load. The authors believed that "the (parallel) studs would have an adverse influence on one another due to the restriction imposed on the distribution of stress from the roots of the connectors." The authors found that, for the studs tested, the ultimate stud strength varies linearly with the longitudinal stud spacing. Furthermore, the ultimate strength increases with stud spacing at a greater rate for three studs per flange than for two studs per flange.

Goble (1968) investigated the behavior of thin flange push-out specimens. From 41 specimens with 1/2, 5/8, and 3/4 in. diameter shear studs, he determined when the failure mode of a shear stud shifts from stud shear to flange pull-out based on flange thickness. Flanges of several thicknesses were used, ranging from 0.128 in. to 0.442 in.

The concrete strengths varied among the tests. Goble found that flange pull-out occurs when the diameter-to-flange thickness ratio is above 2.7. He found that the strength of the stud is not reduced where the failure mode shifts. He also found that ductility is the same, regardless of the failure mode, although thinner flanges caused more flexibility in the lower load ranges. For the stud shear failure mode, the stud strength can be expressed

$$q_u = 882 d_s^2 \sqrt{f_c^{'}} \tag{1.4}$$

where q_u = stud failure load (k)

d_s = diameter of stud (in.)

f'_c = concrete strength (psi)

This is close to the equation by Slutter and Driscoll (1965), where the constant 882 is replaced by 930. However, Slutter and Driscoll's equation is limited to concrete strengths below 4000 psi; most of the tests by Goble had concrete strengths greater than 4000 psi. For the pull-out failure mode, the stud strength is expressed by Goble (1968) as

$$q_u = 4.70 t_f d_s^2 f_u \tag{1.5}$$

where t_f = flange thickness (in.)

f_u = flange ultimate strength (ksi)

Fisher (1970) drew several conclusions regarding the design of composite beams with formed metal deck. For working loads, rib height does not greatly influence beam stiffness. However, for ultimate loads, the shear stud strength decreases with increasing rib height. When the ratio of rib width to height is greater than 1.75, the flexural strength of the composite beam can be developed with full shear connection. An equation for stud connector strength is given as

$$Q_{u-rib} = 0.36 \frac{w}{h} Q_{u-sol} \tag{1.6}$$

where $Q_{u\text{-}rib}$ = ultimate shear connector strength in composite slab

w = average rib width

h = rib height

$Q_{u\text{-}sol}$ = ultimate shear connector strength in solid slab

Other conclusions were made: the shear stud strength increases when the reinforcing steel is attached to the stud; more efficient connections can be made with smaller diameter studs; and the strength of shear studs in lightweight concrete is less than in normal weight concrete by the square root of the ratio of the moduli of elasticity of lightweight to normal weight concrete. Fisher made recommendations for the design of composite beams with formed metal deck.

Ollgaard et al (1971) performed 48 solid slab push-out tests. Variables considered were concrete compressive strength, concrete split tensile strength, modulus of elasticity of concrete, density of concrete, stud diameter, type of aggregate, and number of connectors per slab. The stud tensile strength, slab reinforcement, and geometry were constant for all tests. Stud diameters tested were 5/8 in. and 3/4 in. Two types of normal weight concrete and three types of lightweight concrete were tested. The failure modes observed were stud shearing, concrete failure, and a combination of the two.

For the concrete failure mode, the lightweight concrete had more and larger cracks in the slabs than did the normal weight concrete. When one pair of connectors was in each slab, all failed by shearing off the studs. Specimens with one or two rows of studs per slab had the same strength per stud. The lightweight concrete tended to crush in front of the studs, causing the stud to remain straight when it deformed. When lightweight concrete was used, stud strengths decreased 15% to 25%.

The normal weight concrete provided greater restraint of the stud, so more curvature of the stud occurred. Studs in both types of concrete rotated a large amount at the weld. The tests showed that studs in both types of concrete exhibited considerable inelastic deformation before failure as the specimens did not fail suddenly at ultimate load.

The tests showed that the average shear strength of a stud is almost proportional to the cross-sectional area of the stud for specimens with similar concrete properties. The authors saw no "definite trend" for concrete strengths between 3.5 and 5.0 ksi. However, the stud strength decreases when the concrete strength decreases considerably. The data indicated that the stud strength is more influenced by the concrete compressive strength and modulus of elasticity than by the concrete split tensile strength and density. They also concluded that the concrete properties control at ultimate load, so shear connector tensile strength is not as critical; however, "smaller diameter connectors would be more dependent on stud tensile strength, because the concrete forces would not be as great." This, along with the fact that smaller diameter studs usually have a higher tensile strength, would probably result in higher strengths.

The authors showed that the equation proposed by Slutter and Driscoll (1965) is not valid for different types of concrete (namely, lightweight). This equation overpredicts the stud strengths obtained from tests. The authors performed multiple regression analyses using logarithmic transformations. Fifteen models were tested, containing all possible combinations of the four concrete properties described previously as independent variables. The average shear strength divided by the cross-sectional area, (Q_u/A_s), was the dependent variable. The equation below was shown to adequately represent the stud strength:

$$Q_u = 1.106 A_s f_c^{'0.3} E_c^{0.44} \tag{1.7}$$

where Q_u = ultimate stud strength (k)

A_s = area of stud (in^2)

f'_c = concrete compressive strength (psi)

E_c = concrete modulus of elasticity (psi)

The exponents were rounded, and the following expression was obtained:

$$Q_u = 0.5 A_s \sqrt{f_c^{'} E_c} \tag{1.8}$$

An upper limit on the stud strength occurs around $\sqrt{f_c^{'} E_c}$ =130, which corresponds to Q_u/A_s = 65 ksi. The authors attributed this to the stud tensile strength being reached.

The equations were based on the concrete modulus of elasticity obtained from cylinder tests. The authors showed that the equations work well when the ACI formula is used to obtain the modulus.

Lastly, reloading the specimen during testing caused the same overall load-slip behavior as continuous loading caused. The load-slip curve for continuously loaded specimens, which includes the initial bond, was expressed as

$$Q = Q_u \left(1 - e^{-18\Delta}\right)^{2/5} \tag{1.9}$$

where Q = load (k)

Δ = slip (in.)

The load-slip equation for specimens that were reloaded is similar to the one suggested by Buttry (1965) and is

$$Q = Q_u \frac{80\Delta}{1 + 80\Delta} \tag{1.10}$$

Grant et al (1977) investigated composite beams with formed steel deck to study shear connector strength and beam behavior. The authors performed 17 composite beam tests, and used the results of 58 tests from other researchers. For the 17 tests performed by the authors, the variables included yield strength of the steel beam, geometry of the deck, and degree of partial shear connection. The beams had simple spans of 24 ft or 32 ft, were composite with lightweight concrete slabs on formed steel deck, and used 3/4 in. shear studs. The authors justified the use of wide slabs, with widths 16 times the slab thickness, by stating that wide slabs result in a stronger connection than narrow slabs because the failure surface is larger and "propagates through the rib along a path of least resistance to a natural termination." The use of wide slabs, the authors say, more closely resembles an actual structure, and is the "reason the shear connectors did not fail prematurely." The other 58 tests included as variables the unit weight and strength of concrete, diameter and height of stud shear connectors, type of slab reinforcement, and type of loading.

The authors observed very large deflections in all the composite beams with metal deck, due to the forming of a plastic hinge near the beam midspan. The authors concluded that there was "ductile shear connection which permitted redistribution of the slab force along the span of the beam." This resulted in a large ductility of the beam. They also concluded that rib height, rib width-to-height ratio, and embedment length should be included in a model to predict connector strength.

A modification to the equation developed by Fisher (1970) was made to include the height of the stud shear connector. The strength of the stud in the ribs of composite beams with formed steel deck can be expressed as

$$Q_{rib} = \frac{0.85}{\sqrt{N}}\left(\frac{H-h}{h}\right)\left(\frac{w}{h}\right)Q_{SOL} \leq Q_{SOL} \qquad (1.11)$$

where Q_{rib} = strength of a stud in formed steel deck

N = number of studs in a rib

H = height of stud shear connector

h = height of rib

w = average rib width

Q_{SOL} = strength of the stud shear connector in a flat soffit slab (Eqn. 1.8)

Grant et al (1977) reported that the flexural strength of a composite beam with formed steel deck can be estimated more accurately if the slab force is assumed to act at the middepth of the solid portion of the slab above the ribs rather than at the centroid of the concrete stress block.

The test results showed that beams with partial shear connection are less stiff than beams with full composite action. An effective moment of inertia for beams with solid slabs or composite slabs was expressed as

$$I_{eff} = I_s + \sqrt{\frac{V'h}{Vh}}\left(I_{tr} - I_s\right) \tag{1.12}$$

where I_{eff} = effective moment of inertia

I_s = moment of inertia of the steel section

I_{tr} = moment of inertia of the transformed composite section

Vh = one-half the total force possible in the slab or steel beam

$V'h$ = the number of connectors in the shear span times the allowable load per connector

Johnson and Oehlers (1981) analyzed 125 push-out test results from 11 sources, performed 101 new push-out tests and four composite T-beam tests, and performed a parametric study. Based on a finite element model, the authors found that "a weld collar less than 5 mm high attracts 70% of the total shear and reduces the bending moment at the base of the stud to one-third of the value found for a stud without a collar." As the height of the weld collar increases, the stud shank failure strength increases. Also, the distance that the resultant force on the stud acts from the base increases as the stiffness of the concrete decreases. This increases the bending moment at the base of the stud and decreases the stud shear strength. Based on the model, the authors found that voids in the concrete significantly reduce the stud strength. Further, "stiff inclusions," such as reinforcing bars, significantly increase the stud strength when placed at the soffit of the slab but do not affect the strength when "typical bottom cover is provided." They also attributed variations in push-out test results to variations in the degree of compaction of the concrete and position of the aggregate. They stated that in push-out tests the top of the stud shank is subjected to axial tension, but the bottom is subjected to compression due to the downward frictional force on the shank and weld collar. In beam tests, however, uplift may cause the axial forces to be tensile.

Hawkins and Mitchell (1984) performed 10 push-out tests under reversed cyclic loading and 13 tests under monotonic loading to study the seismic response of shear connectors. Thirteen of the tests used metal deck (1 1/2 in. and 3 in.), and ten were solid slabs. Variables included the type of loading, presence of ribbed metal deck, geometry of metal deck, and orientation of metal deck. The authors observed four failure modes: stud shearing, concrete pull-out, rib shearing, and rib punching. The studs that failed in stud shearing had large slips at failure, were very ductile, had stable hysteresis loops, and had large energy absorbing capabilities. The shear strength of the stud when subjected to reversed cyclic loads is 17% lower than when monotonically loaded. Staggering the studs or using large stud spacings increases the stud shear strength. Also, decreasing the concrete strength increases the slip. Studs may also fail in concrete pull-out due to a tensile force in the stud caused by large deformations. This type of failure is very brittle, and has a "poor hysteretic response." It can cause a large decrease in strength and ductility compared to stud shearing failure. The reversed cyclic strength is 29% lower than the monotonic strength. The strength of the connection, based on three tests on tension specimens, can be expressed as

$$V_c = 5.4\sqrt{f_c^{'}}A_c \tag{1.13}$$

for concrete pull-out failure

where V_c = shear strength due to concrete pull-out (psi)

f'_c = concrete compressive strength (psi)

A_c = area of concrete pull-out failure surface (in^2)

The value of A_c can be found from equations that Hawkins and Mitchell developed based on a pyramidal cone-shaped failure surface in the concrete, propagating downward from the underside of the stud head at a 45 degree angle. The area of the concrete pull-out failure for single studs (ss) and double studs (ds) is

for $h_p < w_1/2$,

$$A_c(ss) = 2\sqrt{2}\left(H_s w_r\right) \tag{1.14}$$

$$A_c(ds) = 2\sqrt{2}H_s w_r + \sqrt{2}w_r s \tag{1.15}$$

and for $h_p > w_1/2$,

$$A_c(ss) = 2\sqrt{2}\left(2h_p^{\ 2} + h_r w_r\right) \tag{1.16}$$

$$A_c(ds) = 2\sqrt{2}\left[2\left(H_s - h_r\right)^2 + w_r h_r + \left(H_s - h_r\right)s\right] \tag{1.17}$$

where h_p = projected height of stud = H_s-h_r (in.)

w_1 = overall trough width (in.)

s = transverse spacing of studs (in.)

H_s = overall stud height (in.)

h_r = metal deck rib height (in.)

w_r = average rib width (in.)

Rib shearing failure usually occurs when studs are grouped together or the deck profile has narrow ribs or large rib heights. The strength and ductility are decreased significantly for this failure mode. Reversed cyclic loading causes S-shaped hysteresis loops and little energy absorption. Rib punching failure occurs when the concrete cover over the stud in the direction of the applied shear is limited.

Elkelish and Robinson (1986) studied six parameters that affect longitudinal cracking of composite beams with metal deck using experimental specimens as well as a finite element analysis. The parameters investigated were type of loading, concrete compressive strength, beam span-to-slab width ratio, thickness of the solid part of the slab, percentage of transverse reinforcement, and the existence of the metal deck. Three loading conditions were used: uniformly distributed load, single point load at midspan, and two point loads at the third points. Twenty-four simply supported beams were used in the analysis. Six experimental beams were used to verify the analysis method.

The results showed that a uniformly distributed load causes the longitudinal crack to start at the top of the slab; single point load and two point loads cause the crack to start at the bottom. The initial longitudinal crack is delayed with an increase of the span-to-width ratio, the steel beam yield strength-to-concrete strength ratio, the thickness of the solid part of the slab, and the transverse reinforcement ratio. Welded wire mesh did not increase the resistance to initial longitudinal cracking. The metal deck helps to resist cracking when point loads are applied, but it does not help for a slab that is uniformly loaded.

Oehlers and Coughlan (1986) analyzed 116 push-out tests to derive the shear stiffness of shear stud connections in composite beams. The authors stated that the flexibility of the shear connection is important because it indirectly affects the flexural strength and fatigue life of the beam. The tests showed that studs in strong concrete are stiffer than studs in weaker concrete. The slip at failure is about one-third of the stud diameter. The tests showed that there is large permanent set even at low loads. The authors explained that this occurs because “the weld collar embeds into the concrete; this is helped by the irregular shape of the collar which causes local crushing of the concrete.” Near the ultimate load, when the stud fractures, the stud has little permanent deformation and the concrete is crushed next to the bearing surface of the weld collar. The authors concluded that at low loads, the amount that the stud embeds into the concrete is affected by voids or dense aggregate particles and the roughness of the weld collar. At high loads, the embedment depends more on the cube strength of the concrete.

Oehlers and Johnson (1987) analyzed 110 solid slab push-out tests to derive an equation that predicts the strength of shear studs in beams. They allowed for variations in the number and position of the studs in the slab and for the “difference between the external restraints in push-tests and in beams.” The test parameters that were considered were the cross-sectional area of the stud shank, moduli of elasticity of the concrete and stud, compressive strength of concrete, and tensile strength of the stud. The tests were detailed so that the concrete slabs would not fail by splitting or shear. The functional relationship between the stud strength and the test parameters was formed by considering the stud connection to be like a steel beam on a concrete foundation with a concentrated load at midspan. The authors argued that “increasing the strength of the stud will allow the concrete to resist a greater shear load as the interface pressure can be distributed across a larger area before fracture of the stud, whereas increasing the strength of the concrete, and hence its modulus, will reduce the flexural forces on the stud and allow a greater shear load before fracture of the stud." The authors statistically analyzed tests from Ollgaard et al (1971) and Johnson and Oehlers (1981) and found that the static shank failure load of stud shear connections in composite beams P_p can be expressed as

$$P_p = KA\left(\frac{E_c}{E_s}\right)^{0.40} f_{cu}^{\ 0.35} f_u^{\ 0.65} \tag{1.18}$$

$$K = 4.1 - n^{-1/2} \tag{1.19}$$

where n = number of studs subjected to similar displacements

A = area of stud shank

E_c = modulus of elasticity of concrete

E_s = modulus of elasticity of stud material

f_{cu} = cube strength of concrete

f_u = tensile strength of stud material

The factor K "allows for the increasing probability of failure at a given load/stud as the number of studs in a shear span reduces." In other words, if fewer studs are used, the design strength should be reduced so that it has the same safety margin as a slab with many studs. The authors explained that there are differences between connection strengths in composite beams and in push-out specimens because of the normal force at the steel-concrete interface. The weight of the concrete slab and applied loads cause compressive forces at the interface in composite beams. The authors performed eight push-out tests where axial tensile and compressive forces were applied to the connections. They determined that the connection strength of studs with zero axial load is 81% of the strength of studs with axial load. The authors also mentioned that a smaller height weld collar reduces the strength of a stud.

Robinson (1988) performed 49 push-out tests on shear studs with 51 mm (2 in.) and 76 mm (3 in.) deep metal deck, as well as two composite beam tests with 76 mm (3 in.) deep metal deck. The push-out specimens simulated shear connections in interior composite beams, exterior composite beams, and girders. The composite beams tested simulated interior composite beams. The push-out tests had only one or one pair of studs per specimen half. The author argued that "push-off specimens having two and three headed studs spaced in the longitudinal direction of the push-off specimens gave rise to unequal shear forces at the roots of the headed studs as well as different transverse tensile forces along the longitudinal axes of the studs." Push-out tests on weak position studs, in which the minimum cover in the direction of the shear forces was given, were also done.

The beam and the exterior beam-type push-out specimens mostly failed by "cracking through the solid part of the concrete slab at the root of the concrete rib on both sides of the rib." The average normalized ultimate shear capacities of the exterior beam-type push-out specimens were much lower than for the interior beam-type push-out specimens. Push-out specimens with single studs gave different results than those with pairs of studs: slips at ultimate load with pairs of studs were about 1.36 times slips with single studs, and the strength of a pair of studs is only about 1.3 times stronger than a single stud. All push-out specimens failed at a load lower than that predicted by Grant et al (1977). The push-out test results showed good correlation with beam tests when the ultimate shear strengths from push-out tests were used to predict the ultimate flexural moment of the beam.

Jayas and Hosain (1988) performed 18 full-size push-out tests and 4 pull-out tests. Five of the push-out specimens had solid concrete slabs, five had the metal deck parallel to the steel beam, and eight had the metal deck perpendicular to the steel beam. The parameters were longitudinal stud spacing and the rib geometry of the metal decks. Both 38 mm (1.5 in.) and 76 mm (3 in.) deck and 16 mm x 76 mm (0.625 in. x 3 in.) and 19 mm x 125 mm (0.75 in. x 5 in.) headed studs were used.

The failure modes from the push-out tests were shearing off of studs, crushing of concrete near the stud, longitudinal shearing of the concrete slab, stud pull-out together with a concrete wedge, and rib shear. The first three failure modes occurred only in solid slabs and composite slabs with deck parallel to the beam. For these specimens, when the studs were far apart (spaced more than six diameters), the studs were most likely to shear off. Specimens with closely spaced studs (spaced about six diameters) failed in the concrete. When the studs were closely spaced, the stud strength reduced by 7% for solid slabs, and 14% for parallel ribbed slabs. Thus, when the longitudinal stud spacing is near or below six times the stud diameter, the authors recommend a reduction in stud strength.

Specimens with deck perpendicular to the beam failed mostly by stud pull-out. One of these specimens failed by rib shear. The solid slab test results showed that concrete failures for studs spaced less than six diameters should be checked in design. When the failure mode was stud shearing, the metal deck did not appear to affect the stud strength. Stud pull-out failure results in a much lower stud strength than stud shear failure. For perpendicular ribbed metal decks, stud pull-out failure was most likely to occur, which resulted in a 40% reduction in strength from a solid slab for a specimen with a wide rib profile, where the rib width-to-rib height ratio, W_r/h_r, is greater than four. The connector strength was reduced further for decks with narrow ribs. There appeared to be no effect of stud spacing on stud strength for specimens with perpendicular deck. The authors found that North American codes overestimate the stud strength when the deck is perpendicular to the beam. Also, the equation by Hawkins and Mitchell (1984) underpredicts the stud strength when 38 mm (1.5 in.) deck is used and overpredicts the stud strength when 76 mm (3 in.) deck is used. They performed a linear regression analysis and developed two separate equations, similar to the one from Hawkins and Mitchell (1984) for two deck heights:

For 76 mm (3 in.) deck,

$$V_c = 0.35\lambda\sqrt{f'_c}A_c \leq Q_u \tag{1.20}$$

and for 38 mm (1.5 in.) deck,

$$V_c = 0.61\lambda\sqrt{f'_c}A_c \leq Q_u \tag{1.21}$$

where V_c = shear strength due to concrete pull-out failure (N)

f'_c = concrete compressive strength (MPa)

A_c = concrete pull-out failure surface area (Hawkins and Mitchell 1984) (mm^2)

λ = factor dependent upon type of concrete
= 1.0 for normal density concrete
= 0.85 for semi-low density concrete
= 0.75 for structural low density concrete

Q_u = ultimate shear stud strength from Ollgaard et al (1971) (N)

Jayas and Hosain (1989) report the results of four composite beams and two push-off specimens with ribbed metal deck perpendicular to the beam that were tested to verify previously reported results (Jayas and Hosain 1988). The beam tests, all designed with partial shear connection, varied the deck profile and longitudinal stud spacing. The push-off tests were used as companions to two of the beam tests. The mode of failure observed most often was concrete pull-out. One of the beams failed by concrete pull-out and rib shearing. One of the push-out specimens failed by concrete pull-out and stud shearing. The authors concluded that stud strength for pull-out failures is dependent upon the deck geometry and stud layout. The authors concluded that the North American codes do not provide good results for the horizontal shear loads, given that strengths calculated with the CSA and LRFD specifications give higher shear strengths for the studs than the observed strengths. The equation from Jayas and Hosain (1988) for 76 mm (3 in.) deck accurately predicted (indirectly) the flexural strength of the beams in this test program.

Oehlers (1989) discussed the effect of slab splitting on stud strength. He described the shear studs as dowels that transfer longitudinal shear flow across the steel flange/concrete slab interface. He described three types of slab cracking that can occur due to the force that a stud induces in the slab. The first type is a lateral crack that extends from the sides of the stud caused by "ripping action." This cracking does not affect the dowel strength significantly. The second type is a "herringbone formation of shear cracks", which does not usually cause failure because transverse reinforcement is often used. The third type is splitting in front, and then in back, of the stud because of "large lateral tensile stresses in front of the triaxial compression zone." This causes dowel failure by concrete compressive failure.

Push-out tests were performed to determine the splitting strengths of concrete slabs. In unreinforced specimens, splitting caused a "rapid reduction in load." In reinforced specimens, splitting caused a more gradual reduction in load. Slab splitting reduces the shear connection strength to between 20% and 90% of the dowel strength. Transverse reinforcement does not appear to increase the connection strength after splitting or to increase the splitting strength; however, it can limit the length of the split. Looping the reinforcement around the base of the stud, however, can allow the strength to increase after splitting and to possibly exceed the dowel strength. Equations to predict the splitting resistance of slabs were given so that designers can avoid a loss of shear connection due to splitting. They can be applied to single or groups of connectors and to stiff or flexible connectors.

Mottram and Johnson (1990) performed 35 composite slab push-out tests using three types of steel deck, with ribs placed only transverse to the steel beam, and using both lightweight and normal weight concrete. Studs used were 19 mm x 95 mm (0.75 in. x 3.75 in.) or 19 mm x 120 mm (0.75 in. x 4.75 in.).

The tests showed that failure occurred in the concrete ribs, not in the studs, with the strength being proportional to $f_{cu}^{0.27}$. A decrease in transverse spacing from 76 mm (3 in.) to 50 mm (2 in.) resulted in a 6% reduction in strength. The resistance per stud for two studs per rib was less than for one stud per rib. The resistance per stud for two studs placed diagonally was less than for an unfavorable stud; however, the maximum slip was greatly reduced. Two studs in line were stronger than two diagonally placed, even though the diagonal studs were further apart. The authors recommend that off-center studs be placed on the "favorable" side away from the midspan of the beam (referred to as the “strong position” in the US); tests done on “unfavorable” (referred to as the “weak position” in the US) studs were 35% weaker than “favorable” studs. The “weakening effect” of an “unfavorable” stud was less for shallower deck. One stud per trough had a slip capacity of 7 mm (0.28 in.) or more; two studs per trough had a smaller slip capacity, half less than 5 mm (0.20 in.). This is a large difference in ductility. The authors recommend accounting for this loss of slip capacity with increasing longitudinal shear resistance in the design of long spans with partial shear connection. Increasing the slab thickness was shown to increase the connection resistance. The authors compared the test results with the predicted values from the equation below, which was being developed but was later modified and published by Lawson (1992), and the equation from Grant et al (1977). The strength reduction factor, *SRF*, is multiplied by the equation for Q_{SOL} (Eqn. 1.8) to obtain the strength of a stud in a composite slab.

$$SRF = \frac{0.75r}{\sqrt{N_R}}\left(\frac{H_s}{H_s + h_R}\right) \leq 1.0 \tag{1.22}$$

where r = factor to account for position of stud in rib
for central or strong position studs, r is the lesser or b_o/h_R and 2.0
for weak position studs, r is the least of b_o/h_R, $e/h_R + 1$, and 2.0

N_R = number of studs per rib

H_s = height of stud

h_R = depth of deck

b_o = average rib width

e = distance from center of stud to mid-height of deck web on loaded side

The equation above was found to be more consistent with test results than the equation proposed by Grant et al (1977). Unlike Grant's equation, this equation accounts for the position of the stud or studs in the rib.

Oehlers (1990), in a discussion of Jayas and Hosain (1988), suggested that design codes do not consider all of the failure modes that can occur in stud shear connections. Codes mainly consider the shank failure of the stud. Concrete failure is avoided by properly using transverse reinforcement or by limiting the concrete cover to the stud or the stud height. He believes more research needs to be performed to prevent concrete slab failure. The author believes that Jayas and Hosain (1988) should have included failure modes other than dowel failure and embedment failure of the concrete slab, such as the three modes of slab cracking failure from Oehlers (1989), to reduce the scatter of the test results. He also believes that the embedment failures observed by Jayas and Hosain (1988) were not the cause of failure but the effect. Otherwise, there would be no need for the Hawkins and Mitchell (1984) equation to be modified for different deck geometries because using the area of the concrete pull-out failure surface in the equation allows for this.

Oehlers indicates that a longitudinal crack in front of a stud causes more compressive failure of the concrete, which can cause one of two results. If the "zone of compressive failure" is large, the stud head will rotate, causing tensile forces behind the stud head and thus tensile cracking of the concrete "in a conical failure surface behind the stud." If the zone is small, the "concrete compressive failure at the base of the stud and in front of the stud will simply increase the lever arm of the resultant force normal to the stud," causing more flexural forces on the stud and thus dowel failure at a reduced shear load. The author also argues that specifying the stud spacing will not always prevent concrete failure.

Lloyd and Wright (1990) performed 42 composite slab push-out tests. Variables included slab width, slab height, and the amount and position of reinforcement. They also studied the effect of applying transverse loading to the slab as well as the effect of sheeting-joint details on connection strength. Two types of deck were tested: deck without stiffeners with the studs welded centrally and a re-entrant profiled deck with stiffeners with the studs welded in the strong position. The stud size used was 19 mm x 100 mm (0.75 in. x 3.94 in.) and the slab was 115 mm (4.5 in.) thick and consisted of normal weight concrete. The slab width varied from 450 mm to 1350 mm (17.7 in. to 53.1 in.). The amount and position of reinforcement and the number of profile ribs were varied. Transverse moment was applied to some tests, until a longitudinal crack occurred, to simulate hogging action of a slab over a beam. Three different sheeting details were tested.

In almost all of the tests, surface cracks appeared along with separation of the concrete from the deck just before ultimate load was reached. "After ultimate load, the slabs were seen to ride over the sheeting and cause extensive profile distortion." Wedge-shaped failure cones, not pyramidal-shaped cones as suggested by Hawkins and Mitchell (1984), occurred around the studs in all of the tests. This mechanism has been found to occur in a composite beam test. Some specimens also failed by rib shear. Tests with the deck parallel to the beam failed by longitudinal shear along the rib or by stud failure. The tests showed that the slip of the deck relative to the beam is half or less of the slip of the slab relative to the beam. Increasing the width of the specimen and varying the amount and position of reinforcement appear to have little effect on the connection strength. Applying a transverse moment to the specimen increased the ultimate strength "only marginally," but caused high loads to be maintained "long after ultimate load had been reached." Sheeting joints decreased the strength a small amount. The authors believe that a "full rib of concrete should be provided beyond the connector position in order that the full strength of the connection is obtained." The authors developed expressions to predict the connection strength, for the total surface area of the wedge-shaped cone for cone failures.

For single studs,

$$A_c(ss) = 2w_1\sqrt{w_1^2/4 + h_p^2} + w_1\sqrt{w_1^2 + 2h_p^2} + 2w_2\sqrt{3D_p^2} \tag{1.23}$$

For double studs,

$$A_c(ds) = A_c(ss) + 2s\sqrt{w_1^2/4 + h_p^2} \tag{1.24}$$

These expressions are more sensitive to the deck and stud geometry than those by Hawkins and Mitchell (1984).

For off-center studs, the following expression may be used

$$\begin{aligned} A_c &= \left(w_1/2 + x\right)\sqrt{\left(w_1/2 + x\right)^2 + h_p^2} \\ &+ \left(3w_1/2 + x\right)\sqrt{\left(w_1/2 - x\right)^2 + h_p^2} \\ &+ w_1\sqrt{4\left(w_1/2 + x\right)^2 + 2h_p^2} + 2w_2\sqrt{3D_p^2} \\ &+ s\left(\sqrt{\left(w_1/2 + x\right)^2 + h_p^2} + \sqrt{\left(w_1/2 - x\right)^2 + h_p^2}\right) \end{aligned} \tag{1.25}$$

where x is the positive to the front of the trough.

The connection resistance can then be found as

$$Q_K = 0.92\left(A_c\sqrt{f_{cu}}\right)^{0.349} \tag{1.26}$$

where f_{cu} = concrete strength

or for design,

$$Q_K = \left(A_c\sqrt{f_{cu}}\right)^{0.34} \tag{1.27}$$

The surface area of the rib shear failure, which occurred in all of the 450 mm wide specimens, may be expressed as

For single studs,

$$A_{cr}(ss) = w_1\sqrt{b^2/4 + h_p^2} + b\sqrt{w_1^2/4 + h_p^2} \tag{1.28}$$

For double studs,

$$\begin{aligned} A_{cr}(ds) = {} & w_1\sqrt{(b-s)^2/4 + h_p^2} + (b-s)\sqrt{w_1^2/4 + h_p^2} \\ & + 2s\sqrt{w_1^2/4 + h_p^2} \end{aligned} \tag{1.29}$$

When the rib-shear area becomes smaller than the wedge-shear cone area predicted by the above equations, the failure mode changes from a cone failure to a rib-shear failure. The more narrow the slab, the more likely rib shear failure will occur. The breadth of the slab at which this change occurs can be expressed as

$$b_{cr}(ss) = {}^{A_c(ss)}\!\!\diagup_{h_p^2}\sqrt{w_1^2/4 + h_p^2} - {}^{w_1}\!\diagup_{2h_p^2}\sqrt{A_c(ss)^2 + 4h^4} \tag{1.30}$$

$$b_{cr}(ds) = b_{cr}(ss) + s \tag{1.31}$$

Wright and Francis (1990) tested four full-scale composite beams with composite slabs along with three push-out specimens. All beams were designed for partial shear connection less than 50%. The deck type and stud size were kept constant; the number of studs used was varied. It was found that longitudinal cracking did not seem to affect beam behavior, and the "hogging moment may have been beneficial to the shear connection strength. This benefit would come as a result of the concrete at the stud base being put into compression and the resulting triaxial stress level increasing the stud strength." The beams and the push-off specimens failed in concrete shear around the studs.

Kitoh and Sonoda (1990) performed composite beam tests in which the thickness of the steel, diameter and height of the studs, and spacing of the studs were varied. Stud forces were measured during the tests. Three-dimensional finite analyses were also performed. The tests showed that in the elastic region, the natural bond between the steel and concrete reduced the forces in the studs. Almost all shearing forces were shown to be at the base of the studs. Also, the tensile forces in the studs were about 10% of the shearing forces in the elastic region and were 30% to 50% of the shearing forces at ultimate. The authors explained the mechanism behind the tensile forces in the studs: "The local bending moment *M* due to shearing force *S* caused separation in one side and contact in another side from the stud position, and thus the tensile force *N* was yielded on the stud as a reaction against the compressive force *C* transmitted on the contact area."

Sublett et al (1992) performed 36 push-out tests to determine the strength of studs in composite open web steel joists. Test parameters were base member thickness, deck rib geometry, slab thickness, stud position, and normal load application. The authors concluded that strength of concrete highly influences the stud ultimate strength: higher strength concrete may increase stud ultimate strength, and lower strength concrete may decrease stud ultimate strength. The stud strength may be assumed to vary approximately linearly with base member thickness. Studs in the strong position exhibited a larger stiffness than those in the weak position. The AISC strength predictions were unconservative. Weak position studs had an average strength of 52% of predicted strength using AISC and the strong position studs showed 72% of predicted strength. The authors recommend the Mottram and Johnson (1990) method over the current AISC specification, or its results are 30% more conservative than AISC, perhaps because Mottram and Johnson included the stud location as a test parameter. The authors recommended for future tests, that the concrete strength should be varied to include low strengths to test the validity of Eurocode 4 (*EN* 2001) specification. They also recommended that the AISC specification should include stud position as a variable affecting stud strength. Also, studs used with thin flange sections, having thicknesses less than 2 1/2 times the stud diameter, should have a reduced strength.

Lawson (1992) gave a thorough review of present design rules for calculating shear connection capacities. He explained the faults and limitations of these methods; based on this, he proposed a new method for calculating the strength reduction factor for shear connectors welded through deck.

1. For single or pairs of studs placed centrally in rib:

a. $$r_p = \frac{0.75}{\sqrt{N}} \frac{b_a}{D_p} \frac{h}{h + D_p} \leq 1.0 \text{ for } b_a \leq 2D_p \tag{1.32}$$

b. $$r_p = \frac{1.5}{\sqrt{N}} \frac{h}{h + D_p\left(2D_p / b_a\right)} \le 1.0 \text{ for } b_a \ge 2D_p \tag{1.33}$$

where r_p = reduction factor on shear stud strength

b_a = average rib width (or minimum width for re-entrant profiles)

D_p = deck height ≤ 100 mm

h = stud height ≥ D_p + 35 mm

N = number of studs per rib

2. For single studs placed off-center in rib:

a. Strong position studs:

Use equations above with N = 1 and b_a replaced by $2(b_a\text{-}e)$.

b. Weak position studs:

Use first equation above with $b_a = e + D_p$ and N = 1 or second equation above if $e > D_p$

where e = distance from center of stud to mid-height of deck web

c. Staggered studs (single strong and weak position studs in separate ribs):

Take average of cases 2a and 2b.

3. For pairs of studs placed off-center in rib:

a. In pairs (side by side):

Use cases 2a or 2b as appropriate with N = 2.

b. In pairs (in-line):

Use equations above with N = 2.

Johnson and Yuan (1997) developed equations based on theoretical models for seven modes of failure. Results of over 300 push out tests and 34 new ones were used to determine the accuracy of the models. This method, unlike many of the methods specified in design codes around the world, considers the position of the stud within the rib of the metal deck. Five modes of failure are considered for transverse sheeting. They are shank shearing (SS), rib punching (RP), rib punching with shank shearing (RPSS), rib punching with concrete pull-out (RPCP), and concrete pull-out (CPT). Theoretical models were developed for each failure mode. For shank shearing failure of studs in slabs that are reinforced so that splitting cannot spread, the shear strength is found from Eurocode 4:

$$P_{rs} = 0.37 A_s \left(f_c E_{cm} \right)^{0.5} \leq 0.8 A_s f_u \tag{1.34}$$

where P_{rs} = shear strength of stud in a solid slab

A_s = cross-sectional area of stud

f_c = cylinder strength of concrete

E_{cm} = modulus of elasticity of concrete

f_u = ultimate strength of stud

For other failure modes, the shear strength is defined by

$$P_r = k_t P_{rs} \tag{1.35}$$

where P_r = shear strength of stud

k_t = reduction factor for modes of failure other than SS

For concrete pull-out failure of studs in slabs with one stud per trough, in a central or favorable position, the strength is

$$P_r = k_{cp} P_{rs} \tag{1.36}$$

$$k_{cp} = \frac{\left[\eta_{cp} + \lambda_{cp}\left(1 + \lambda_{cp}^{\ 2} - \eta_{cp}^{\ 2}\right)^{0.5}\right]}{\left(1 + \lambda_{cp}^{\ 2}\right)} \leq 1.0 \tag{1.37}$$

$$\eta_{cp} = \frac{0.56 \nu_{tu} h^2 \left(b_o - \frac{h}{4}\right)}{h_p N_r P_{rs}} \leq 1.0 \tag{1.38}$$

$$\lambda_{cp} = \frac{e_r T_y}{h_p P_{rs}} \tag{1.39}$$

$$T_y \cong 0.8 A_s f_u \tag{1.40}$$

$$\nu_{tu} = 0.8 f_{cu}^{\ 0.5} \leq 5 \tag{1.41}$$

where k_{cp} = reduction factor for CPT failure mode

η_{cp} = non-dimensional group for CPT failure mode

λ_{cp} = non-dimensional group for CPT failure mode

ν_{tu} = shear strength of concrete

h = height of stud

b_o = average width of deck trough

h_p = height of steel deck

N_r = number of studs per rib

e_r = distance from center of stud to nearer wall of rib for favorable position studs

T_y = yield tensile strength of stud

f_{cu} = cube strength of concrete

If h>2h_p, use h=2h_p. If η_{cp}≥1.0, it should be taken as 1.0. This indicates failure type SS rather than CPT.

For rib punching failure of studs placed in the unfavorable position, the strength is

$$P_r = k_{rp} P_{rs} \tag{1.42}$$

$$k_{rp} = \frac{\left\lfloor \eta_{rp} + \lambda_{rp}\left(1 + \lambda_{rp}^{\;2} - \eta_{rp}^{\;2}\right)^{0.5} \right\rfloor}{\left(1 + \lambda_{rp}^{\;2}\right)} \le 1.0 \tag{1.43}$$

$$\eta_{rp} = \frac{1.8\left(e_f + h - h_p\right) t f_{yp}}{P_{rs}} \tag{1.44}$$

$$\lambda_{rp} = \frac{e_f T_y}{2 h_p P_{rs}} \tag{1.45}$$

$$T_y \cong 0.8 A_s f_u \tag{1.46}$$

where k_{rp} = reduction factor for RP failure mode

η_{rp} = non-dimensional group for RP failure mode

λ_{rp} = non-dimensional group for RP failure mode

e_f = distance from center of stud to nearer rib wall for unfavorable position studs

t = thickness of steel deck

f_{yp} = yield strength of steel deck

For combined rib punching and concrete pull-out failure of studs in slabs with two studs placed in series or diagonally in a trough, the stud placed on the unfavorable side is assumed to fail by rib punching. The stud placed on the favorable side is assumed to fail by concrete pull-out. The resistances of the two studs are added to get the combined resistance. For the rib punching failure mode, the equations are as follows:

$$P_r = k_u P_{rs} \tag{1.47}$$

$$k_u = \frac{\left\lfloor \eta_u + \lambda_u \left(1 + \lambda_u^2 - \eta_u^2\right)^{0.5} \right\rfloor}{\left(1 + \lambda_u^2\right)} \leq 1.0 \tag{1.48}$$

$$\eta_u = \frac{\left(e + h - h_p\right) t f_{yp}}{P_{rs}} \tag{1.49}$$

$$\lambda_u = \frac{eT_y}{2 h_p P_{rs}} \tag{1.50}$$

where k_u = reduction factor for rib punching in RPCP failure mode

η_u = non-dimensional group for rib punching in RPCP failure mode

λ_u = non-dimensional group for rib punching in RPCP failure mode

e = distance from center of stud to nearer wall of rib

For the concrete pull-out failure mode, the equations are as follows:

$$P_r = k_f P_{rs} \tag{1.51}$$

$$k_f = \frac{\left\lfloor \eta_f + \lambda_f \left(1 + \lambda_f^2 - \eta_f^2\right)^{0.5} \right\rfloor}{\left(1 + \lambda_f^2\right)} \leq 1.0 \tag{1.52}$$

$$\eta_f = \frac{0.56 v_{tu} h^2 \left(e + s_t - \frac{h}{4}\right)}{h_p P_{rs}} if\, 0.75h \leq \left(e + s_t\right) \tag{1.53}$$

$$\eta_f = \frac{v_{tu} \left(e + s_t\right)^2 \left(0.75h - \frac{\left(e + s_t\right)}{3}\right)}{h_p P_{rs}} if\, 0.75h > \left(e + s_t\right) \tag{1.54}$$

$$\lambda_f = \frac{eT_y}{h_p P_{rs}} \tag{1.55}$$

$$T_y \cong 0.8 A_s f_u \tag{1.56}$$

$$v_{tu} = 0.8 f_{cu}^{\ 0.5} \leq 5 \tag{1.57}$$

where k_f = reduction factor for concrete pull-out in RPCP failure mode

η_f = non-dimensional group for concrete pull-out in RPCP failure mode

λ_f = non-dimensional group for concrete pull-out in RPCP failure mode

s_t = spacing of studs

When $h > 2h_p$, assume $h = 2h_p$. If $\eta_f \geq 1.0$, k_f is taken as 1.0 and the failure mode is RPSS.

1.3 Scope of Research

Current codes, such as the AISC specification, give unconservative predictions of shear stud strength when steel deck is used. In most of the existing strength prediction equations, an upper limit is placed on shear stud strength. The American Institute of Steel Construction LRFD Specification (*Load* 1993) uses the stud tensile strength, A_sF_u, as the upper limit. Other codes use 80% of this value. The only apparent reason for using this upper limit is that the data seems to approach it. If one used a failure criterion, such as Von Mises' failure theory, one would estimate the shear stress at ultimate to be around 60% of the ultimate tensile stress. However, test data shows that a stud, especially in a solid slab, exhibits significantly more strength than indicated by the above. A stud that is placed in a composite slab and has a small amount of concrete between the stud and the deck rib in front of the stud (on the load-bearing side of the stud) has a strength less than 60% of its tensile strength.

The purpose of the research reported herein is to develop a new equation for predicting the strength of shear studs when steel deck is used. Once an adequate strength prediction method is developed from push-out tests, beam tests are used to verify the method. Thereafter, a reliability study is performed so that a new resistance factor may be applied to composite beams.

Chapter 2 describes the experimental study that uses push-out tests, which were performed to investigate the wide array of parameters that may be used in design. This study includes tests using deeper decks, longer studs, and smaller and larger diameter studs than previously tested. Chapter 3 presents the results and includes a discussion of these push-out tests. Chapter 4 compares these results with existing strength prediction methods. Chapter 5 presents the results of tests performed on bare studs not placed in concrete. Chapter 6 evaluates the results of these tests and push-out tests performed at VT and elsewhere. A new stud strength prediction model is proposed. Chapter 7 describes three composite beam tests and presents their results. Chapter 8 evaluates these results and results from many other beam tests performed by other researchers and compares them to the AISC strength model and the new model proposed in Chapter 6. Chapter 9 calculates a resistance factor for composite beams that are designed using the new strength prediction method. Chapter 10 is a summary of the findings of this study. Push-out and beam test data, as well as other information related to these tests, are reported in Rambo-Roddenberry et al (2002).