CHAPTER 1
INTRODUCTION

1.1 Background

Composite floor systems have been used in buildings for over thirty years (Salmon and Johnson 1996). Composite construction typically consists of a concrete slab placed upon a rolled steel beam or girder and interconnected with shear connectors, generally welded shear studs. These shear connectors are designed to resist the horizontal shear that develops during bending.

Composite joists are relatively new in the field of structural engineering. In composite joist construction, open-web steel joists are used as an alternative to rolled steel sections. Joists are standardized parallel chord trusses economically fabricated using established techniques and standards. The chords are typically light double angle sections and the web members are usually round bars or small angles. The web openings in joists accommodate ductwork, electrical conduit and piping. Joists are also much lighter in weight than rolled steel sections.

Many types of shear connectors have been developed since composite construction began, but welded shear studs are by far the most popular. This is due to the fact that shear stud strength is the same in all directions and that they are easy to attach with a welding gun. Shear studs provide economical shear connection in longer spans. However, shear studs are often impractical in short-span (25 ft.-35 ft.) composite joists. The thin double-angle top chord section on short-span composite joists makes it difficult to satisfy the requirement that the stud diameter to flange thickness ratio be no greater than 2.5 (*Load and* 1993). In addition, it is difficult to obtain a good-quality weld on a thin base material. Often burn-through can occur, leaving a hole in the top chord of the joist (Hankins 1994). Poor-quality welding through the profiled steel deck has caused some contractors to revert to manual welding with electrodes. Another problem is that atmospheric conditions are not always favorable for welding. Obtaining a source of electricity is also a problem on some construction sites (Crisinel 1990).

These problems have led to the development of many non-welded shear connection alternatives. These non-welded shear connectors are generally attached with shot-fired nails or studs or with self-drilling, self-tapping screws. The main advantage of the use of these methods is that it removes the need for expensive and bulky welding equipment and skilled welders on smaller construction sites (Crisinel 1990, Jolly et al 1986).

The standoff screw is a non-welded shear connector that is currently being studied at Virginia Tech. This device is a self-drilling, self-tapping screw with a variable length standoff shank. A typical standoff screw is shown in Figure 1.1. Standoff screws are placed with a screw gun and can be installed in the top chord of joists through profiled steel deck. The standoff shank is embedded in the concrete, effectively resisting the horizontal shear between the concrete slab and the steel joist. The small diameter of the standoff screw and the means of attachment, screwing rather than welding, make it a promising alternative to welded shear studs in short-span composite joists.

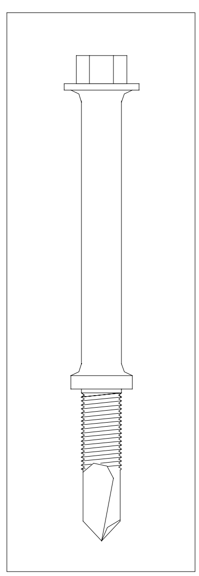

Figure 1.1 Typical Standoff Screw

1.2 Literature Review

The subject of standoff screws functioning as shear connectors is quite new. As a result, there has been little research in this area and the possibilities of using standoff screws in the field have not been fully investigated.

The first significant research performed on a type of standoff screw is that of Strocchia at Virginia Tech (1990). Strocchia conducted 13 series of pushout tests, totaling 36 tests, to evaluate the strength and behavior of six different types of deck fasteners. Of these six types, three were a type of standoff screw. These are shown in Figure 1.2. The standoff screws that Strocchia tested were self-tapping screws with standoff sleeves around the shaft of the screw to prevent the screws from being driven flush to the deck during installation. The standoff lengths that were investigated were 1.25 in., 1.75 in., and 2.25 in. The diameter of the screws varied from 0.215 in. to 0.240 in. Vulcraft 1.5VL 22 ga. deck was used in all tests although the concrete slab dimensions varied. The base material for the pushout tests was initially fabricated from WT 5x11 sections. However, to accurately model the behavior of the top chord of a steel joist, this was replaced by 2L-1.5x1.5x0.113 welded to a steel plate measuring 0.5 in. thick by 6 in. wide by 44 in. long.

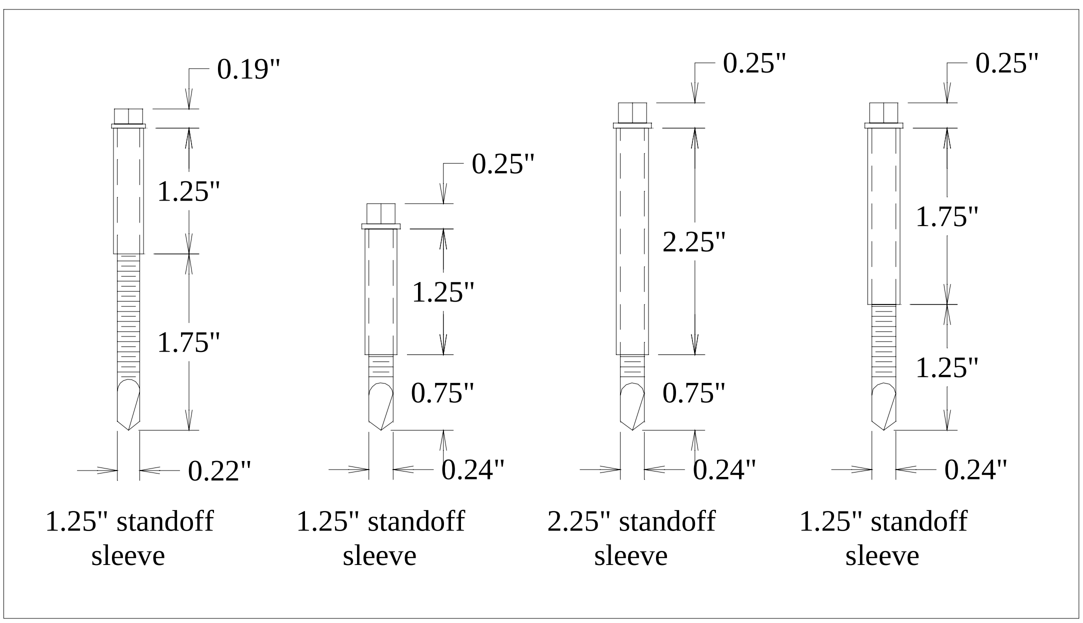

Figure 1.2 Self-tapping screws with standoff sleeves (Strocchia 1990)

Six of the 13 series of tests were conducted to investigate the standoff screws. Significant rotation of the base material and bending of the screws was observed in specimens with thin base angles. With this rotation, many screws did not fail by direct shear, but by bending and tensile stresses. Strocchia also observed that the embedment depth of the screws influenced the failure mode of the system. To realize the full strength of the screws, it was determined that the screws must be embedded in the concrete so that the top of the screw is higher than the top of the deck profile. This was evident in comparing the results of the tests with 2.25 in. standoff screw lengths with the results of tests utilizing 1.25 in. standoff screw lengths. The tests with the shorter screws failed primarily by shearing of the concrete ribs, while the tests with the longer screws failed by the more ductile failure of the screws. The smaller width of the specimens in comparison to more recent pushout test specimens also contributed to failure of the ribs. Another modification that was implemented to resist concrete rib failure was to invert the deck to increase the potential concrete rib failure plane. This change was effective in that the specimens with the inverted deck exhibited a ductile screw failure and failed at a higher load.

Strocchia concluded that the test series with the greater screw embedment depth or inverted Vulcraft 1.5VL deck displayed ideal behavior for a shear connection system in that these series provided enough strength to achieve composite action while exhibiting sufficient ductility. He also recommended modifications to the standoff screws used in his tests. He proposed increasing the diameter of the standoff shaft to stiffen the screws to resist bending inside the concrete ribs. He noted that the slit in the standoff sleeve may have reduced the stiffness of the standoff screw and he also suggested further tests of standoff screws in other deck types. These recommendations would influence further testing of standoff screws at Virginia Tech.

Lauer (1994) conducted eight full size composite joist tests at Virginia Tech with six different types of shear connectors, including the Buildex 5/16 in. diameter by 2 in. long standoff screw and the Elco Grade 8 standoff screw with the same dimensions. The standoff screws used in Lauer’s tests were similar to those used in this study, shown in

Figure 1.1. Two of the eight tests utilized the Buildex screws and one used Elco screws. Each test utilized profiled steel deck oriented perpendicular to the joist span direction. The two tests that utilized Buildex standoff screws, CSJ-1 and CSJ-2, were identical with the exception of the deck type used: CSJ-1 had Vulcraft 1.0C 26 ga. deck and CSJ-2 had Vulcraft 1.5VL 22 ga. deck. Both tests used two 8-in. deep joists that spanned 24 ft.-3 in. and were spaced 40 in. apart. The concrete slabs were 3 in. in total depth and 80 in. wide. The screws were staggered and placed in every third rib in CSJ-1 and every other rib in CSJ-2, 14 in each half span. Test CSJ-8 consisted of two 18-in. deep joists spanning 29 ft.-7.5 in. and spaced 40 in. apart. Nine Elco Grade 8 standoff screws were used in each half span. Vulcraft 1.0C 26 ga. deck was used with a 4 in. total thickness concrete slab.

In tests CSJ-1 and CSJ-2, Lauer noticed that "although overall behavior of the system was ductile, failure of individual standoff screw shear connectors was somewhat brittle." He noted extensive screw rotation causing the top chord angles to which they were connected to distort. Some screws tore through the deck without shearing off as the slip between the slab and joist increased. Screws did not begin to rupture until "member deflection and interface slip had progressed considerably." None of the standoff screws were pulled out of the top chord angles. In test CSJ-8, none of the Elco screws ruptured or pulled out of the top chord.

Lauer used pushout test data from Hankins' study (1994) to estimate the shear connector capacity in CSJ-1, CSJ-2, and CSJ-8. Although the test parameters of slab thickness, concrete compressive strength, and base metal thickness in CSJ-1 and CSJ-2 did not exactly match those of the pushout tests, the full-scale tests did not significantly differ from the pushout test configurations. The test parameters of base metal thickness, deck type, and slab thickness for CSJ-8 matched those of the pushout test exactly. In each case, the pushout test values were used without modification. Lauer calculated the experimental shear connection force in each test from equilibrium equations that represent a flexural model of a composite joist. These equations are:

$$\sum Q_{ae} \cdot e \pm N_a \cdot e' = M_{ae} \tag{1.1}$$

$$\sum Q_{ae} \cdot e_t + T_a \cdot e' = M_{ae} \tag{1.2}$$

where:

ΣQ_{ae} = measured shear connection

N_a = top chord force due to applied load

T_a = bottom chord force due to applied load

M_{ae} = experimental midspan moment under applied load

e = distance between bottom chord centroid and resultant concrete force, in.

e' = distance between centroids of top and bottom chords, in.

e_t = distance between top chord centroid and resultant concrete force, in.

Knowing the experimental applied moment on the joist and assuming a top chord force based on the value measured during testing, the experimental shear connection force was calculated. Then the remaining bottom chord force could be found from equilibrium. For the calculated shear connection force to be acceptable, the top and bottom chord forces had to be close to the actual chord forces. The experimental shear connection force in CSJ-1, CSJ-2, and CSJ-8 compared very favorably to the values obtained from Hankins' pushout test data, as can be seen in Table 1.1. The shear connection force from test CSJ-8 was the closest to its assumed value, possibly due to the fact that the full-scale test parameters of slab thickness, deck type, and base metal thickness were identical to those found in the corresponding pushout tests.

Table 1.1 Experimental Shear Connection (Lauer 1994)

	N_a (kips)	N_{ae} (kips)	T_a (kips)	T_{ae} (kips)	**Q_{ac} (kips)**	**Q_{ae} (kips)**	**Q_{ae}/Q_{ac}**
CSJ-1	36.8	36.8	82.0	81.8	**41.0**	**45.2**	**1.10**
CSJ-2	40.4	40.4	83.7	87.5	**41.0**	**43.3**	**1.06**
CSJ-8	21.8	21.8	54.1	62.3	**33.2**	**32.3**	**0.97**

N_a Top chord force due to applied loading, assumed value for use in Eq. 1.1

N_{ae} Experimental top chord force due to applied load

T_a Bottom chord force due to applied load, found from horizontal force equilibrium

T_{ae} Experimental bottom chord force due to applied load

Q_{ac} Calculated shear connector capacity

Q_{ae} Experimental shear connector capacity, back calculated using Eq. 1.1

The most extensive research performed to date on standoff screws is that of Hankins at Virginia Tech (1994). Hankins research was a companion study to the full-scale tests performed by Lauer, and consisted of 65 double-sided pushout tests on the Elco Grade 8, 5/16 in. diameter standoff screw, shown in Figure 1.1. Hankins research also included a preliminary series of nine pushout tests to compare three different types of screws. In each test, the standoff screws were fastened to a base material consisting of back to back angles separated by filler plates, to simulate the top chord of an open web steel joist. The varied test parameters included steel deck geometry, base angle thickness, embedment depth, slab width, and slab thickness. The 74 total tests were divided into six series: a preliminary series and series 1 through 5.

The preliminary series consisted of nine pushout tests and was conducted to compare the performance of three types of standoff screws, Buildex, Elco Grade 5, and Elco Grade 8. Three pushout tests were performed on each type of standoff screw. The top chord section was fabricated from 2L-1.5x1.5x0.123 and Vulcraft 1.0C, 26 ga. deck was used in all nine tests. As in all Hankins' tests that utilized profiled steel deck, the deck ribs were positioned perpendicular to the direction of applied load. The concrete slabs were 4 in. thick and measured 36 in. wide and 36 in. long. Three screws were used in each half test specimen, placed in a staggered position. It was found in the preliminary series that the Elco screws were much more ductile than the Buildex screws, failing at a much higher average slip. Accordingly, the Elco screws were determined to be more promising for use in composite joists. The Elco Grade 8 connector was chosen for further consideration over the Elco Grade 5 due to the higher theoretical shear and tensile strength of the Grade 8 screw. Pushout test series 1 through 5 were then conducted to further evaluate the performance of the Elco Grade 8 standoff screw.

Pushout test series 1 consisted of 20 tests that were conducted to study the performance of the Elco Grade 8 screw with a 2 in. standoff shank in specimens with solid slabs and varying top chord thicknesses. As in all the tests in series 1 through 5, four standoff screws were placed in each specimen half in a staggered pattern and welded wire fabric was used as reinforcement. The slab length and width were both 36 in. and the slab thickness varied. Pushout series 2 consisted of 12 tests to evaluate the performance of the Elco Grade 8, 1.5 in. standoff screw in Vulcraft 0.6C 28 ga. deck. The concrete slabs measured 36 in. long, 24 in. wide and 2 in. thick. Pushout series 3 consisted of 12 tests to evaluate the performance of the Elco Grade 8, 2 in. standoff screw in Vulcraft 1.0C 26 ga. deck. Series 3 slabs measured 36 in. long, 36 in. wide and 2.5 in. thick. Series 4 and 5 used Vulcraft 1.5VL 22 ga. deck and consisted of 12 and 9 tests, respectively. The slabs measured 36 in. by 36 in. with varying thicknesses. Screw height in series 4 was 2 in. and for series 5, it was 2.5 in.

Hankins observed five different failure modes in the pushout tests: screw shear, cone pullout, screw pullout from base angles, longitudinal splitting of the slab, and angle buckling in the top chord section. Buckling of the top chord section was preceded by substantial rotation at the base of the standoff screws and considerable deformation of the base angle. This deformation, shown in Figure 1.3, caused the development of a plastic hinge, allowing the top chord section to buckle laterally above the upper-most screws. Hankins also observed that base angle thickness greatly influenced the strength of the standoff screws. As base angle thickness increased, screw strength also increased. However, when the base angle thickness becomes too great, it acts relatively fixed and screw strength decreases. Therefore, Hankins concluded that greater connector strength could be achieved with a somewhat flexible base material, approximately 0.2 in. thick. Large rotation of the shear connector causes the connector to be loaded primarily in tension. Little or no rotation of the shear connector in a thicker base angle causes primarily shear loading of the connector. The drop in the strength of the standoff screws associated with a larger base thickness suggests an interaction between shear and tension. Hankins also noticed the great ductility of the Elco Grade 8 standoff screw, as evidenced in his plots of shear load vs. slip.

Hankins' analysis of the pushout test data consisted of applying several existing shear stud models to the test geometries of the 65 tests in series 1 through series 5. He then compared the analytical results to the experimental results to determine the predictability of the shear strength of the Elco Grade 8 standoff screw.

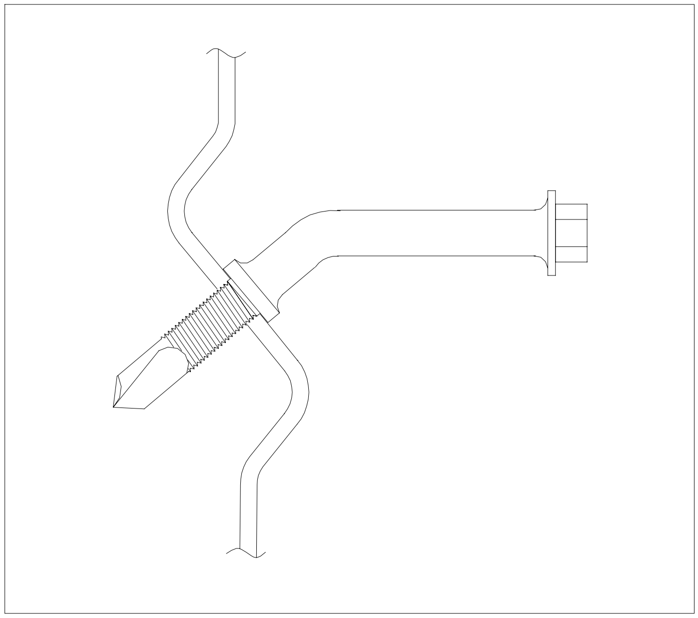

Figure 1.3 Typical Screw Rotation (Hankins 1994)

Hankins concluded that the concrete splitting model developed by Oehlers (1989) predicts the strength of the standoff screw in solid concrete slabs with acceptable accuracy, provided that the indirect tensile strength of concrete is taken as $6\sqrt{f'_c}$. Oehlers concrete splitting model is based on 50 reinforced and unreinforced pushout tests that failed by longitudinal cracking. He developed a method of predicting the occurrence of splitting due to the local action of individual connectors and global action of groups of connectors. The following equation was developed to predict a splitting crack in the concrete (Oehlers 1989):

$$P_s = \frac{0.6 h_a f_t b \pi}{\left(1 - \frac{d_s}{b}\right)^2} \tag{1.3}$$

where:

P_s = concentrated connector force resulting in concrete splitting, N

h_a = effective connector height, $1.8 \times d_s$, mm

f_t = indirect tensile strength of concrete, N/mm^2

b = pushout specimen slab width, mm

d_s = connector diameter, mm

Oehlers (1989) also investigated the effects of transverse reinforcement on the concrete splitting and determined that transverse reinforcement does not prevent splitting but does limit the severity of the split, thus preserving some of the interaction and shear connection. Hankins questioned the applicability of using Eq. 1.3 for standoff screws, since standoff screws would be used, in most instances, in applications utilizing steel deck. When steel deck is used, failure by concrete tensile pullout is more common than longitudinal splitting of the concrete.

Attempting to predict the strength of the standoff screw in configurations utilizing profiled steel deck, Hankins examined the stud strength model proposed by Lloyd and Wright (1990). Lloyd and Wright's model was based on a wedge shaped concrete failure surface that was a modification to a similar model developed by Hawkins and Mitchell. The Lloyd and Wright model, shown in Figures 1.4 and 1.5, is based on a shear path of least resistance.

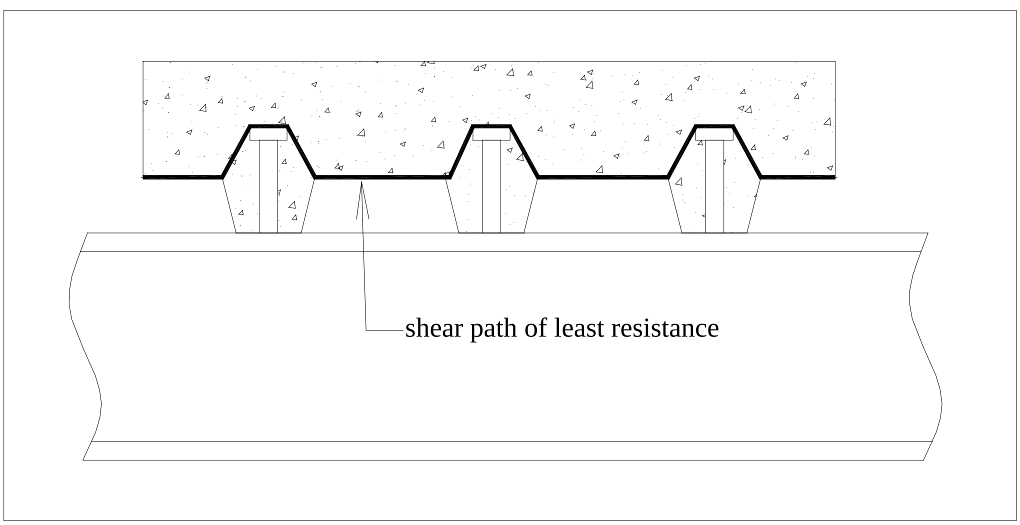

Figure 1.4 Longitudinal Shear Path (Lloyd and Wright 1990)

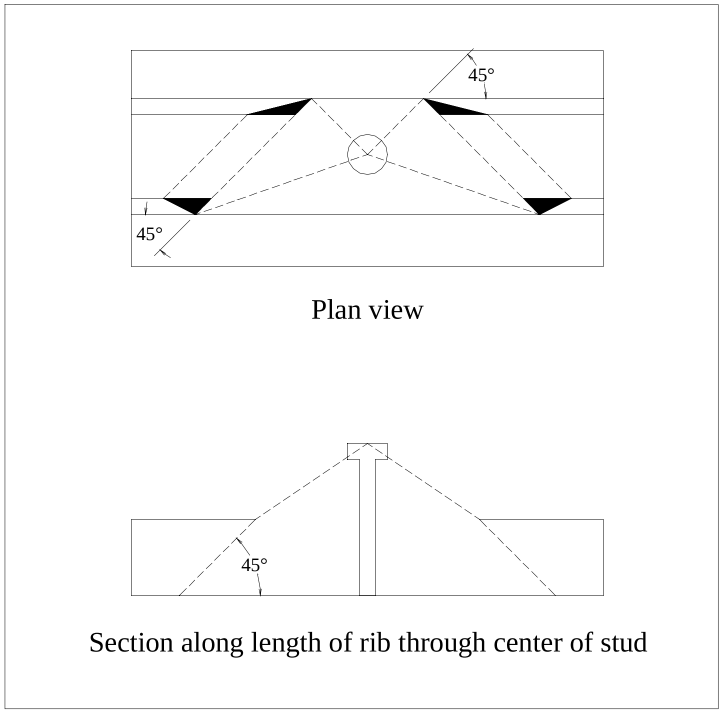

Figure 1.5 Wedged Shear Cone (Lloyd and Wright 1990)

Although the Lloyd and Wright model did not accurately predict the strength of the standoff screw, the wedged failure surface was consistent with the pullout failures observed in Hankins tests. Because no existing models accurately predicted the strength of the standoff screw in slab geometries utilizing profiled steel deck, Hankins developed an equation based on a rederivation of the Lloyd and Wright model which does so with acceptable accuracy. The equation is given by:

$$V_{wc} = 0.11\sqrt{A_{wc}\sqrt{f'_c}} \tag{1.4}$$

where:

V_{wc} = connector strength, kips

A_{wc} = surface area of wedge shaped tensile concrete pullout cone, in.^{2}

$$= 2w_{r2}\sqrt{\frac{w_{r2}^2}{4}+(H_s-h_r)^2}+w_{r2}\sqrt{w_{r2}^2+2(H_s-h_r)^2}+2w_{r1}\sqrt{3h_r^2}$$

f'_c = concrete compressive strength, psi

h_r = nominal rib height of steel deck, in.

H_s = total length of shear connector, in.

w_{r1} = concrete rib width at bottom of flange of steel deck, in.

w_{r2} = concrete rib width at top of flange of steel deck, in.

Hankins concluded that the Elco Grade 8 standoff screw provided effective shear connection and could be used in lightweight joist applications where conventional headed shear studs cannot be used.

Although there has been limited research performed on standoff screws, there has been extensive research performed on welded shear studs. Much of this data and information can be used to formulate hypotheses on the behavior of standoff screws.

Lawson (1997) investigated the influence of the shape of the deck profile on the resistance of shear studs. He proposed a concrete failure cone model that lead to a strength reduction factor that can be used for single studs or pairs of studs placed in the deck transverse to the applied force. This formula was then compared to pushout test results.

Lawson found that pairs of studs lead to much reduced resistances and often to lower deformation capacity. Eurocode 4 (EC4) modified the traditional strength reduction formula proposed by Grant, Fisher and Slutter (1977) as it was observed that this formula is unconservative at times. The modified strength reduction equation is:

$$r_p = \frac{0.7}{\sqrt{N}} \frac{b_a}{D_p} \frac{(h - D_p)}{D_p} \leq 1.0 \tag{1.5}$$

where:

r_p = EC4 strength reduction factor

h = stud height ($\geq D_p$+35 mm)

D_p = deck profile height

b_a = average rib width

N = number of studs per rib

The upper limit on r_p is reduced to 0.8 when studs are used in pairs in each deck rib. Lawson proposes his own strength reduction factor, observing that the EC4 reduction factor is less accurate when studs are placed in pairs. The equation is

$$r_p = \frac{h - D_p + 0.4b_a}{h} \leq 1.0 \qquad (1.6)$$

The following limitations apply to Lawson's proposed equation:

$b_a \geq 0.5h$

$h \leq 120$ mm

$h \geq D_p$+35 mm

stud diameter, $\phi \leq 19$ mm

If $b_a > 2h$, then $b_a = 2h$

For pairs of shear connectors, the same concrete cone failure model can be used and a further reduction factor applied:

$$r_n = \frac{0.5s + h}{2h} \leq 0.8 \qquad (1.7)$$

where s = distance between adjacent shear connectors ($\geq 3\phi$). For pairs of studs located in-line with the deck ribs, b_a should be taken as (h+e), where e=0.5 b_a when the stud is located in the center of the rib. These formulae were derived for 19 mm diameter studs. The equations are generally to be used with decks with relatively wide ribs and not for large diameter studs and decks with narrower or deeper ribs.

Jayas and Hosain (1987) examined the longitudinal spacing of the headed studs and the rib geometry of the metal decks in conducting 18 push-out tests. Five of the tests utilized solid concrete slabs, five used a metal deck with ribs parallel to the steel beam, and in the remaining eight the ribs were perpendicular to the steel beam. They found that stud shear was the primary failure mode in solid slabs and parallel ribbed slabs, if the studs were placed sufficiently apart. When the longitudinal stud spacing was not greater than six times the stud diameter, concrete related failures were observed. Stud pull-out failure, resulting in drastic reduction in connector strength, i.e., about 40% reduction, occurred in nearly all the specimens with perpendicular ribbed decks. Jayas and Hosain also noted that the model developed by Hawkins and Mitchell for predicting stud capacity in pull-out failures underestimates the stud capacity for 38 mm decks and overestimates the stud capacity for 76 mm decks. In addition, LRFD formulations produced higher shear strength values than were observed in specimens with perpendicular ribbed metal decks. As a result, Jayas and Hosain developed two separate empirical equations to determine the shear capacity in pull-out failures.

For a 76 mm deck: $$V_c = 0.35\lambda\sqrt{f'_c}\,A_c \leq Q_u \tag{1.8}$$

and for a 38 mm deck: $$V_c = 0.61\lambda\sqrt{f'_c}\,A_c \leq Q_u \tag{1.9}$$

where:

V_c = shear capacity due to concrete pull-out failure, N

f'_c = concrete compressive strength, Mpa

A_c = area of concrete pull-out failure surface, mm^2

λ = a factor which depends on the type of concrete used

Q_u = ultimate shear capacity (Ollgaard et al 1971)

$= 0.5A_s\sqrt{f'_c\,E_c}$

A_s = cross sectional area of the stud connector

E_c = concrete modulus of elasticity

Tests conducted on four full-size composite beams and two full-size push-off specimens confirmed the use of the above equations in determining shear capacity in pull-out failures (Jayas and Hosain 1989). The ribbed metal deck was placed perpendicular to the beam span and concrete pull-out was the principal mode of failure.

In addition to the concrete cone failures discussed earlier, Lloyd and Wright (1990) also encountered the failure mode of rib shear failure in several of their tests. Rib shear failure, or simply rib failure, is similar to a wedge-shaped concrete cone pullout failure, but extends the full width of a particular concrete slab in a pushout test specimen. This type of failure is highly dependent on the width of the pushout specimen and may be of concern in the design of edge beams. Based on their wedged-shaped concrete cone model, Lloyd and Wright presented the following equation to estimate the area of the failure surface in cases of rib shear:

$$A_{rs} = w_{r2}\sqrt{\frac{b^2}{4} + \left(H_s - h_r\right)^2} + b\sqrt{\frac{w_{r2}^2}{4} + \left(H_s - h_r\right)^2} \tag{1.10}$$

where:

A_{rs} = rib shear failure surface area, in.^{2}

b = width of pushout specimen, in.

h_r = nominal rib height of steel deck, in.

H_s = height of shear connector, in.

w_{r2} = concrete rib width at top flange of steel deck, in.

Lloyd and Wright concluded that rib shear failure strength can not be accurately determined from any existing models.

1.3 Scope of Research

This study is a continuation of the research on standoff screws that was begun by Hankins and Lauer at Virginia Tech. Hankins performed 74 pushout tests on Buildex and Elco standoff screws to evaluate the performance of standoff screws in varying deck geometries and concrete slab dimensions. Lauer conducted eight full scale composite joist tests, three of which utilized standoff screws as mechanical shear connectors. The research performed for this study consisted of 106 pushout tests on the Elco Grade 8 standoff screw shear connector and is intended to supplement the data and results obtained in Hankins' pushout tests. The testing procedure and setup for this study are virtually identical to that of Hankins. The primary differences in this study are the greater quantities of screws per test specimen and the investigation of additional screw heights.

This report is divided into five chapters. The details of the experimental program are provided in Chapter 2. These include the fabrication of the test specimens, the configurations of the various test series, the test setup and instrumentation, and the testing procedure. In Chapter 3, the results of each pushout test are given, as well as a comparison of the results from each test series. Chapter 4 consists of the analysis and interpretations of the data. The applicability of Hankins' proposed formula for predicting the shear strength of the Elco Grade 8 standoff screw will be investigated in Chapter 4 and the effects of grouping the connectors will be discussed. Chapter 5 will contain a summary of the research, conclusions, and recommendations for further research. The complete data for all 106 tests is contained in a separate volume entitled <u>Data Report for Standoff Screws Used in Composite Joists</u> (Alander, et al 1998b).