CHAPTER 1
INTRODUCTION

1.1. Background

The first documented example of steel-concrete composite construction dates back to 1877, when rolled steel beams encased in concrete were used in completion of the Ward House, a private residence in the state of New York (Viest et al. 1997). Since then, the application of steel-concrete composites has been rapidly spreading to many different structural parts, from composite columns to composite lateral force resisting systems. Steel-concrete composite structural components have become commonplace because of their functionality, simplicity, economy, and applicability to many different types of structures.

Although many different composite structural components may be encountered at construction sites all over the country, one particular application of composite steel-concrete construction is by far the most widespread, namely, composite floor systems. A composite floor system typically consists of a steel beam or joist and a cold-formed steel deck composite slab. Typically, this system is held together by welded shear studs, which are connected to the top of the steel member on one end and embedded in concrete on the other.

The moment capacity of a steel-concrete non-composite flexural section can often be doubled by the addition of adequate shear connection, making the section composite. Advantages of composite flexural members over non-composite members include increased stiffness, increased allowable span length, reduction in weight and depth of the steel section required, and decreased overall weight of the floor (Salmon and Johnson 1996).

Along with improvements and innovations in composite construction in general, specific improvements in composite floors have been made as well in recent years. One development that continues to gain popularity is the composite open web joist. Open web joists are prefabricated steel trusses consisting of top chord, bottom chord, and web members. The top and bottom chords usually consist of double angle sections, and web members are most often either double angle sections, single (crimped) angle sections, or continuous round bars.

Composite open-web joists have several advantages. They are able to span large openings, thus providing large column free areas. The open web structure also permits

HVAC ducts and utilities (plumbing, electricity, telecommunications) to be placed within the depth of the joist.

In contrast to their functionality and numerous advantages, using welded shear studs can be problematic in certain open-web joists. This is especially true for the shorter-span joists, which generally use relatively thin angles in the top chord. Specifically, problems can arise with welding of headed shear studs to the double angle top chords. The double angle section is narrow and thus presents a target that is difficult for the stud installer to hit when welding the studs. The requirement used for limiting the ratio of stud diameter to base metal thickness is given in the AISC specifications (Load and 1993) as 2.5. This criteria sometimes limits the use of composite joists because the top chords must be increased in size to accommodate the welded shear stud, resulting in a loss of economy. This circumstance warrants the need for a different shear connector that satisfies the strength requirements, is functional and easy to install, and at the same time does not govern the size of the joist chord to the extent that welded shear connectors do.

The development of a ductile shear connector, which can be screwed into the top chord, would solve the problem of strict welding requirements and also eliminate the need for sizeable welding equipment and the necessary energy sources at the job site. This new shear connector would also be simple to install, not requiring the presence of trained and expensive welding personnel. The new connector would potentially be perfect for small- and medium-sized construction jobs where lighter joists with thinner top chords could be used in composite floors.

In response to this need, there has been ongoing research at Virginia Polytechnic Institute and State University for the last decade. The researchers have focused on the development of a shear connector that would make the open web joist even more practical and appealing for use in composite floors. The ongoing investigation has narrowed the possible choices down to ELCO Grade 8 standoff screw, different sizes of which are shown in Figure 1.1. The screw has a self-drilling, self-tapping point. The screw material is ASTM Grade 8 and has a minimum specified tensile strength stress of 150 ksi.

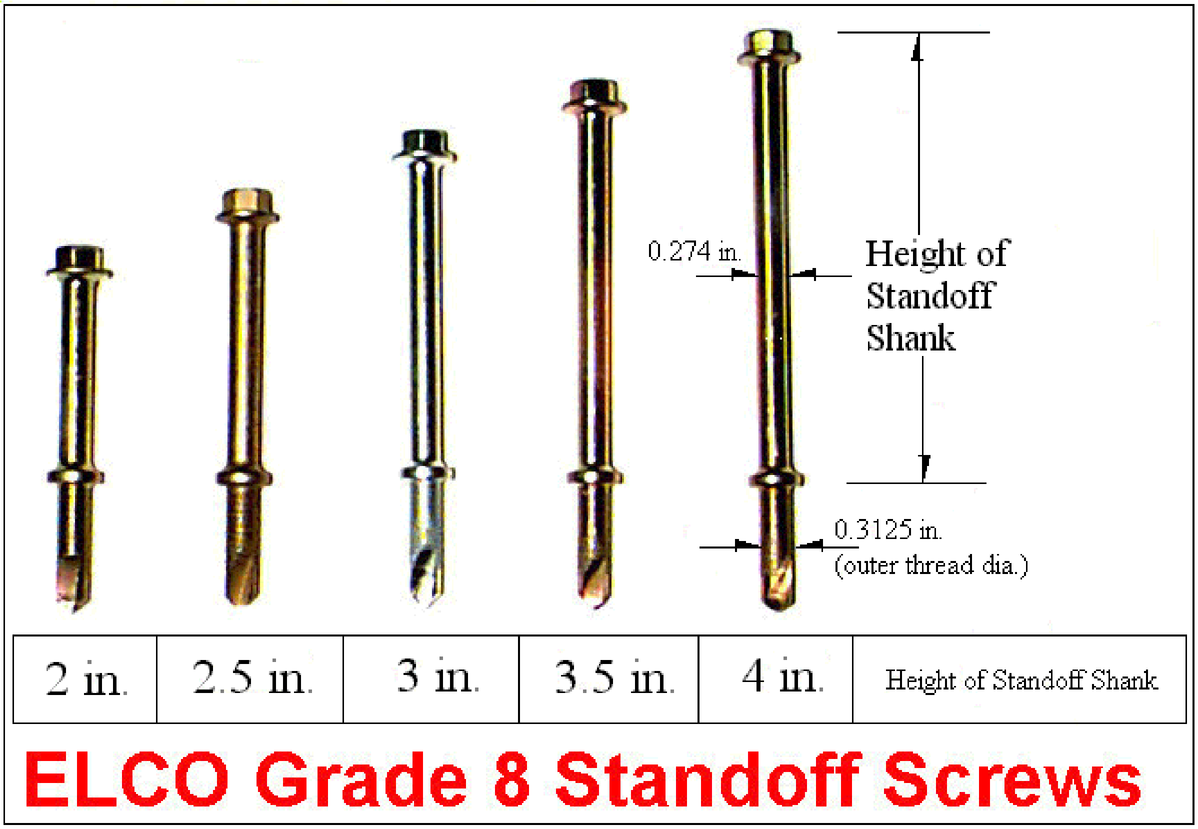

Figure 1.1 ELCO Grade 8 Standoff Screws

1.2. Literature Review

The standoff screw used in composite joists is a new product that is still in research and development. Other ideas utilizing drilled shear connectors have been presented in the past (El-Shihy 1986), (Moy et al. 1987). Some of them reflected the concept contained in the idea of standoff screws. However, most of the literature regarding the use of standoff screws as shear connectors is limited to the academic publications originating at Virginia Polytechnic Institute and State University where the bulk of the research on the subject has been done.

There is a wealth of research and publications available on other similar shear connections. Relevant aspects of that literature are reviewed here as some of the aspects of that analytical work may be helpful in analyzing the behavior of standoff screws.

An abundance of literature is also available on Load Resistance Factor Design and probability-based structural design. Applicable publications from this area are reviewed as well.

1.2.1. Standoff Screws

The initial work in this area at Virginia Tech was done by Strocchia et al. (1990). Strocchia et al conducted a total of 36 push-out tests, organized in 13 test series. Among the six different types of deck fasteners investigated, six were the standoff screws. These screws and their dimensions are shown in Figure 1.2, and the type of deck used in all the tests, shown in Figure 1.3, was 1.5VLga22 manufactured by Vulcraft.

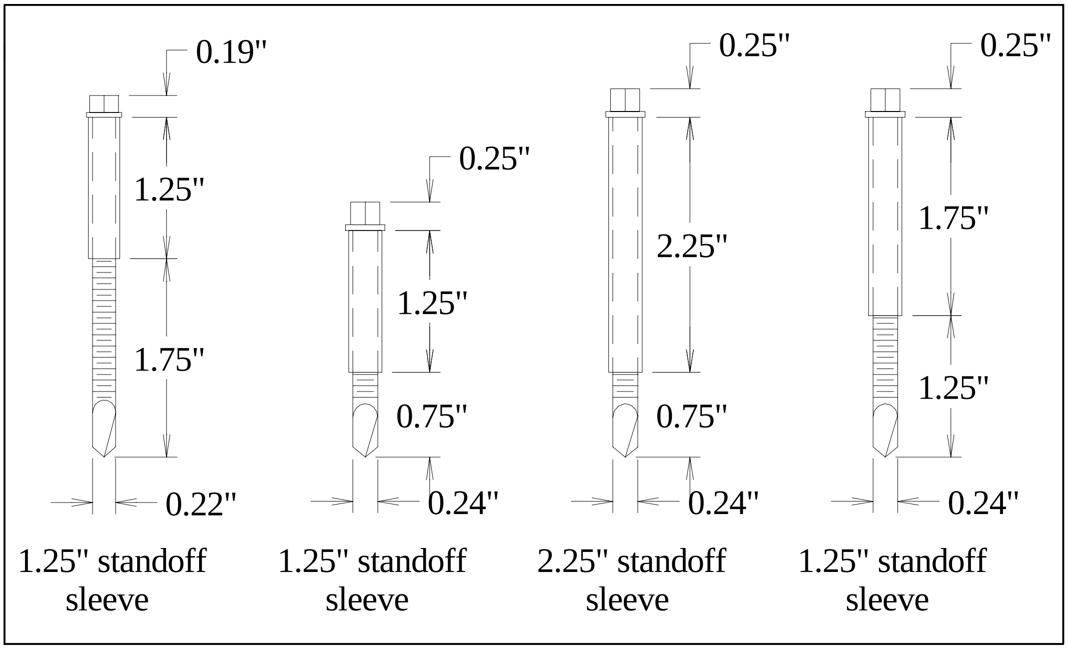

Figure 1.2 Self-tapping Screws with Standoff Sleeves (Strocchia et al. 1990)

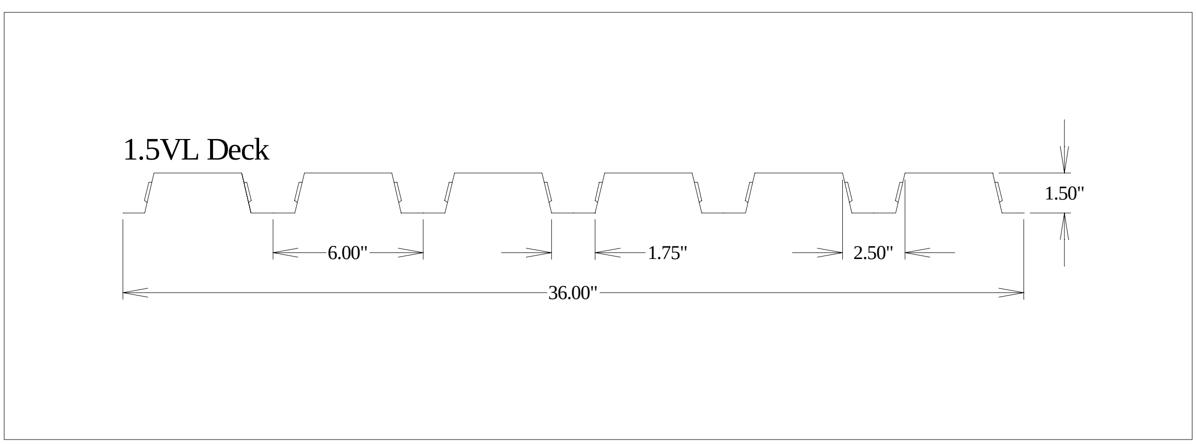

Figure 1.3 Vulcraft Deck Profile Used by Strocchia et al. (1990)

Structural steel tee sections (WT 5x11) were used in the first tests as base members. To simulate the actual top chord of an open web joist, the tees were later replaced by double angle sections of various sizes, welded to a steel plate that was ½ in. thick, 6 in. wide, and 44 in. long. Strocchia et al. observed significant rotation of screws in thinner angles. As a result of this rotation, screws did not fail by direct shear, but by the combined effect of shear and tension to which they were subjected after the screw had rotated. Strocchia et al. also observed the effect of screw embedment on the capacity of shear connection. In that respect, he noticed that the full strength of the screws were not taken advantage of unless the screws were embedded in concrete so that their height exceeded the height of the rib. Specimens where screws were embedded above the height of the rib failed by screw shear, while those where screws were not embedded above the rib height experienced concrete rib-related type of failure. Push-out specimen slabs used in these tests were only 12 or 24 in. wide, which could increase the possibility of the specimen failing by concrete related failure since the specimen ribs were not long enough to absorb the force transferred through the screws. In later specimens, the width was increased to 36 in., which helped avoid concrete-related failures. Another change was the inversion of the steel deck, which resulted in an increase in shear failure plane, which in turn resulted in the increase in the strength of the shear connection and consequently, a decrease in the number of rib failures.

Strocchia et al. concluded that the inverted deck enabled an ideal behavior and ductile failure mode with the use of standoff screws because it limited concrete-related rib failures and allowed the concrete to sustain loads high enough to fail the screws in a ductile fashion. He also noted that increasing the diameter of the standoff screw shank increases the screw stiffness and capacity by allowing it to resist bending inside the concrete due to the loads applied. Strocchia et al. also suggested that the effect of deck type on the capacity of the shear connection be investigated as well. Further studies of standoff screws at Virginia Polytechnic Institute and State University were based on his suggestions.

Almost simultaneously Lauer et al. (1994) and Hankins et al. (1994) conducted two separate studies on standoff screws used as shear connectors. As a part of their study, Lauer et al. conducted six full-scale tests containing standoff screws of different types. Four of these tests used ELCO Grade 8 screws, while two of them used Buildex screws. The deck was oriented perpendicular to the joist. Deck types used in these tests were 1.0C and 1.5VL.

At the same time, Hankins et al. performed 74 push-out tests. ELCO Grade 8 standoff screws, as shown in Figure 1.4, were used in 68 of the tests. The remaining six tests were conducted using ELCO Grade 5 and Buildex screws, respectively.

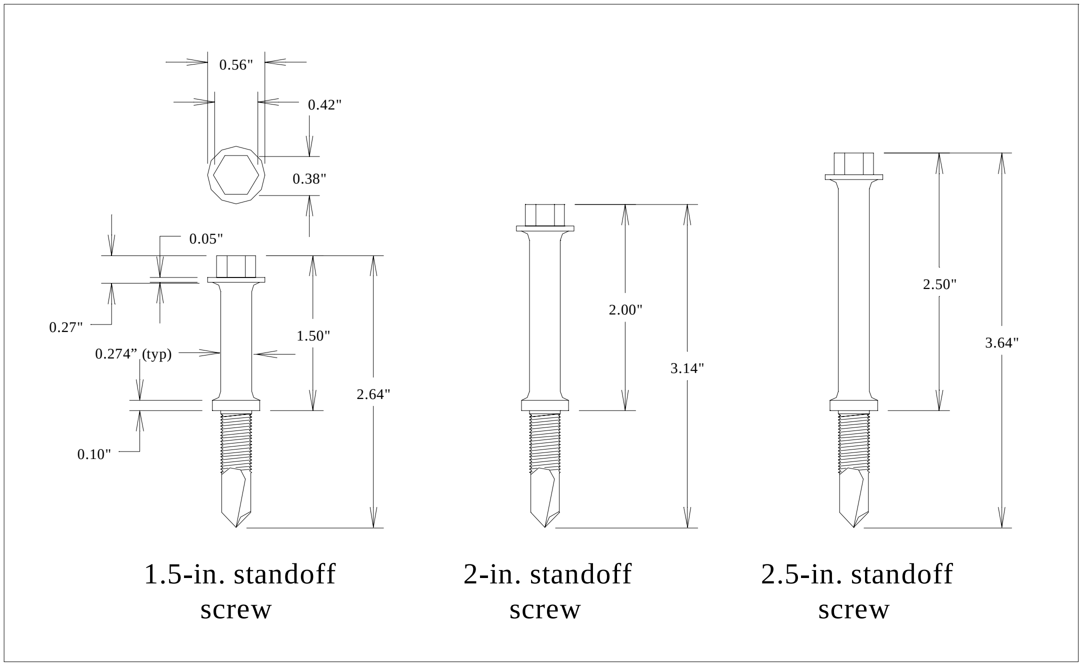

Figure 1.4 Standoff Screws Investigated by Hankins et al (1994)

Lauer et al stated that the failure of each individual shear connector in tests CSJ-1, and CSJ-2 (which used Buildex screws) was somewhat brittle, even though the overall behavior of composite joists was generally ductile. Lauer et al also observed distortion in the top chord angles due to screw rotation. They noted that the screws did not begin to rupture until significant member deflections and slab to joist slips were visible. At larger slip values, some screws tore through the deck without shearing off.

In specimen CSJ-8 none of the screws sheared off. The top chord buckled before the screws reached their capacity.

In specimens CSJ-9 and CSJ-10, the bottom and the top chord, respectively, failed. Lauer et al. list shear connection failure as the controlling failure mode for CSJ-11, but further description of the screws' behavior during the test is unavailable.

In no instance during these six full-scale tests did the screw pull out of the angle. Even though not all the parameters from push-out tests of Hankins et al. matched those of full-scale tests of Lauer et al., Lauer et al. were able to predict the capacity of shear connection in specimens CSJ-1, CSJ-2, and CSJ-8 to acceptable accuracy. Top chord thickness, slab thickness, and type of deck used were the parameters that did not differ between the push-out and full-scale tests. Lauer et al., however, made no further adjustments for the parameters that were not used, such as screw height, or number of screws per rib. Instead, they used the values from corresponding push-out tests as they were reported.

Lauer et al. used Equations 1.1, and 1.2, which represent an equilibrium state of a composite flexural member, to back-calculate the strength of the shear connectors.

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

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

where:

ΣQ_{ae} = back-calculated experimental shear connection strength

N_a = top chord force due to applied load

T_a = bottom chord force due to applied load

M_{ae} = experimental mid-span 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.

Using the experimental values of maximum applied moment, and using the top chord force measured during the test, Lauer et al. calculated the experimental values of shear connection strength using Equations 1.1 and 1.2 and compared those to the values obtained in push-outs (Table 1.1).

Table 1.1 Experimental Shear Connection (Lauer et al. 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**

where:

N_a = Top chord force due to applied loading, assumed value for use in Equation (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 strength

Q_{ae} = Experimental shear connector capacity, back-calculated using Equation (1.1)

Based on the review of the results of Lauer et al., several possible reasons for the difference between the calculated and experimental values of shear connection strength are:

- Differences between configurations of full-scale tests and corresponding push-out tests;
- Unaccounted for friction between the deck and the joist, which adds to total shear connection strength;
- Statistical scatter.

The research of Hankins et al. involved performing 74 push-out tests, including a preliminary series of nine tests with the task of comparing the three available standoff screws and their performance as shear connectors. These three connectors included Buildex, ELCO Grade 5, and ELCO Grade 8 screws. Three push-out tests were performed with each type of screw. The top chord sections were 2L-1.5x1.5x0.123, and Vulcraft 1.0Cga26 deck, with ribs oriented perpendicular to the top chord, was used in all the preliminary tests. All the slabs were 4 in. deep and otherwise measured 36 x 36 in. Each slab was connected to the steel member by three screws. In comparing the results from these preliminary tests, Hankins et al. concluded that ELCO screws were more promising for further investigation and potential use since Buildex screws were brittle and exhibited little ductility. The mode of failure obtained in tests done with Buildex screws was screw shear. Shear load capacity values per screw *(average shear strength of 2.93 kips)* for Buildex screws were significantly lower than those of ELCO screws *(average shear strength of 3.97 kips for ELCO Grade 5* *screws, and 3.69 kips for ELCO Grade 8 screws),* which all failed by screw pullout. The slip tolerated by Buildex screws was on average only 0.216 in., compared with 0.693 in. using ELCO Grade 5, and 0.690 in. using ELCO Grade 8 screws. Average secant stiffness of Buildex screws was 963 kips/in., which is more than what was obtained with ELCO Grade 5 (716 kips/in.), but less than ELCO Grade 8 (997 kips/in.). Given their higher strength and ductility ELCO screws, clearly were superior to Buildex screws.

In addition, Hankins et al. noted that ELCO Grade 5 and Grade 8 screws could not be compared based on results of preliminary tests only, because all the screws pulled out of top chord material, and it was hard to predict the differences in behavior of the two types of screws that would occur at higher loads where screws would reach their ultimate strengths and pull out.

Finally, Hankins et al. suggested that ELCO Grade 8 screws were the most suitable for future consideration and were preferred to ELCO Grade 5 screws due to their higher theoretical shear and tensile strength. The cost of producing an ELCO Grade 8 screw is only minimally higher than that of the Grade 5 screw.

The remaining 65 tests were organized in five series. Based on conclusions reached after performing the preliminary test series, Hankins et al. utilized ELCO Grade 8 screws in all five series. Profiles of the screws investigated are shown in Figure 1.4. Hankins et al. varied different parameters, such as deck type or slab thickness, throughout the series. A typical push-out specimen and screw configuration used by Hankins et al. is shown in Figure 1.5.

Hankins et al. observed and identified five different failure modes in the push-out tests he performed:

- Screw shear-off;
- Concrete cone pull-out;
- Screw pull-out from base angles;
- Longitudinal splitting of slabs;
- Angle buckling in the top chord section;

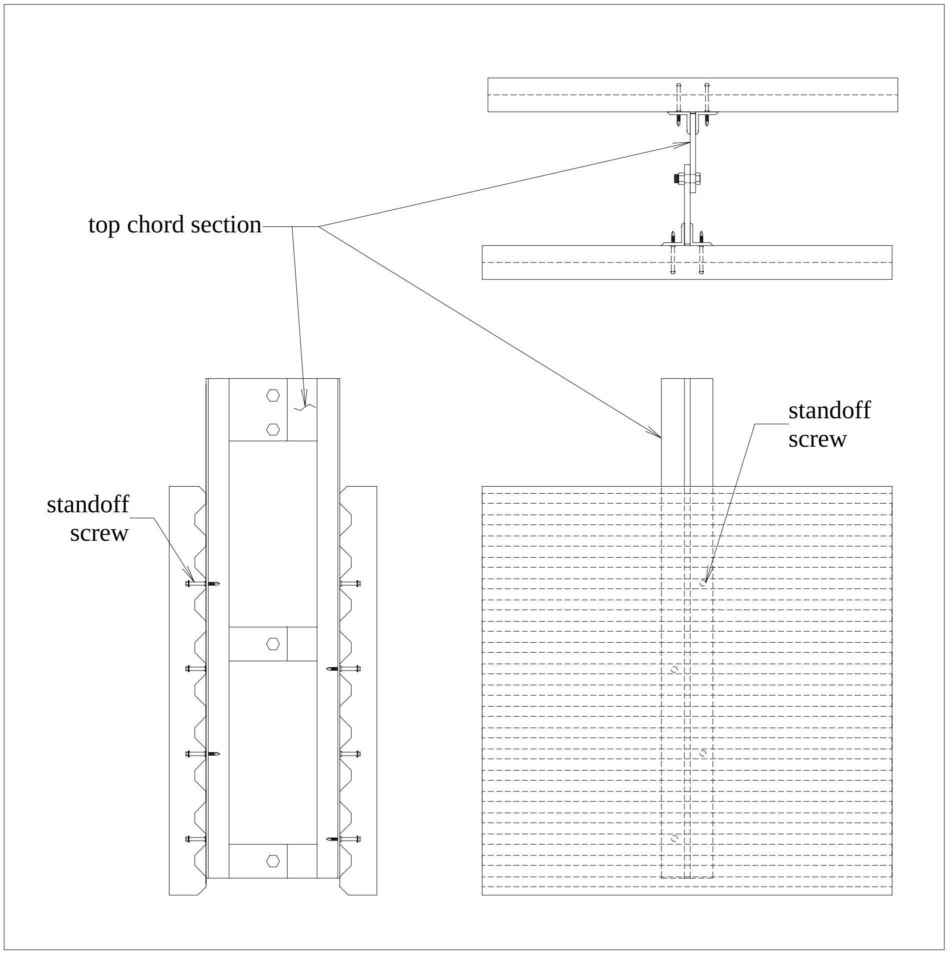

Figure 1.5 Typical Push-out Specimen Used by Hankins et al. (1994)

Hankins et al. concluded that the screw strength is related to the thickness of base material. Namely, the screws embedded into thinner angles were able to rotate. This decreased the extent to which they were loaded in shear and increased their load in tension. Given the fact the screw strength in tension is more than that in shear, the screws exhibited higher strength in thinner angles. On the other hand, the screws embedded in thicker angles were not able to rotate, and they exhibited lower strength as they were loaded mainly in shear. Very thin base angle resulted in screw pull-out mode of failure. Hankins et al. concluded that the greatest possible strength of a standoff screw could be achieved with base material being approximately 0.200 in. thick. Hankins et al. again witnessed a high level of ductility of standoff screws, which is discernable from their load vs. slip plots. Significant screw rotation, as shown in Figure 1.6, in the tests performed by Hankins et al. caused significant angle deformations, which ultimately caused the top chord to buckle.

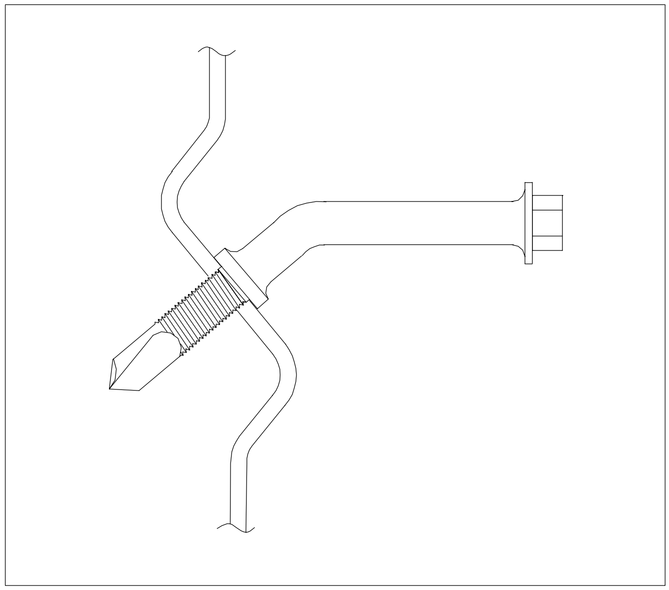

Figure 1.6 Typical Screw Rotation (Hankins et al. 1994)

Using existing models applicable to shear studs, Hankins et al. calculated theoretical values of shear strength and compared them to the results of push-out tests.

As long as indirect tensile strength of concrete is taken as $6\sqrt{f'_c}$, the concrete splitting model developed by Oehler (1989) and shown as Equation 1.3 predicted the strength of shear connection in solid slabs, as found by Hankins et al., with acceptable accuracy.

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

where:

P_s = concentrated connector force resulting in concrete splitting, N

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

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

b = push-out specimen slab width, mm

d_s = connector diameter, mm

Oehlers' model is based on 50 push-out tests, some of which were reinforced and some not. The model Oehler developed predicts the slab splitting due to local and individual effects of shear connectors, and due to the effect of a group of shear connectors on the entire slab. Although Oehlers' equation seems to be useful in predicting longitudinal slab splitting, Hankins decided that the equation might have very limited applicability in situations where standoff screws are used. Oehlers' also found that transverse reinforcement in composite slabs does not prevent the slab splitting but does lessen magnitude of the split, which in effect prevents the loss of some of the shear connection.

The model proposed by Lloyd and Wright (1990), which Hankins used to predict the strength of shear connection featuring standoff screws in slabs with formed steel deck, was based on previous models developed by Hawkins and Mitchell (1984). Specifically, it assumes the existence and shape of a wedge shaped concrete failure surface, which would occur under the applied load at the path of least resistance. Figure 1.7 shows the generated shear load path, and Figure 1.8 shows the wedged shear cone that comes as a result of shear path. The model by Lloyd and Wright failed to accurately predict the strength of shear connection. Shear path of least resistance, as defined by Lloyd and Wright, however, was in agreement with the path of concrete cone failures which Hankins et al. observed in their push-out tests. In the absence of other adequate models, Hankins et al. developed their own equation (Equation 1.4) which was used to predict the strength of standoff screw connection in slabs utilizing formed deck. In essence, the model Hankins et al. developed is a modified version of what was proposed by Lloyd and Wright.

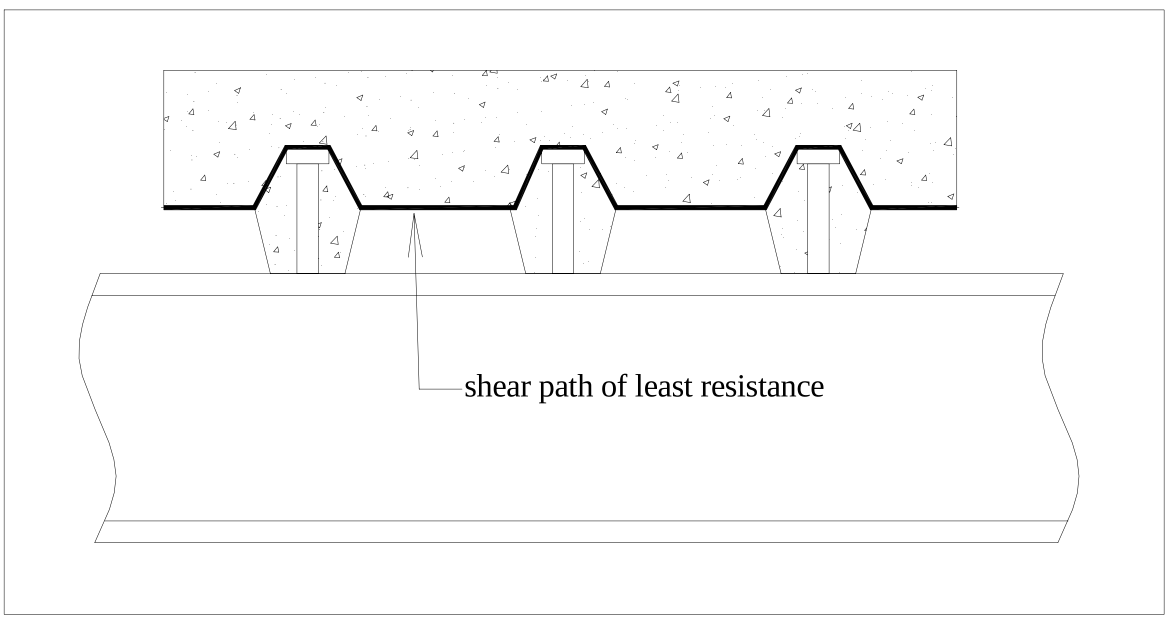

Figure 1.7 Longitudinal Shear Path (Lloyd and Wright 1990)

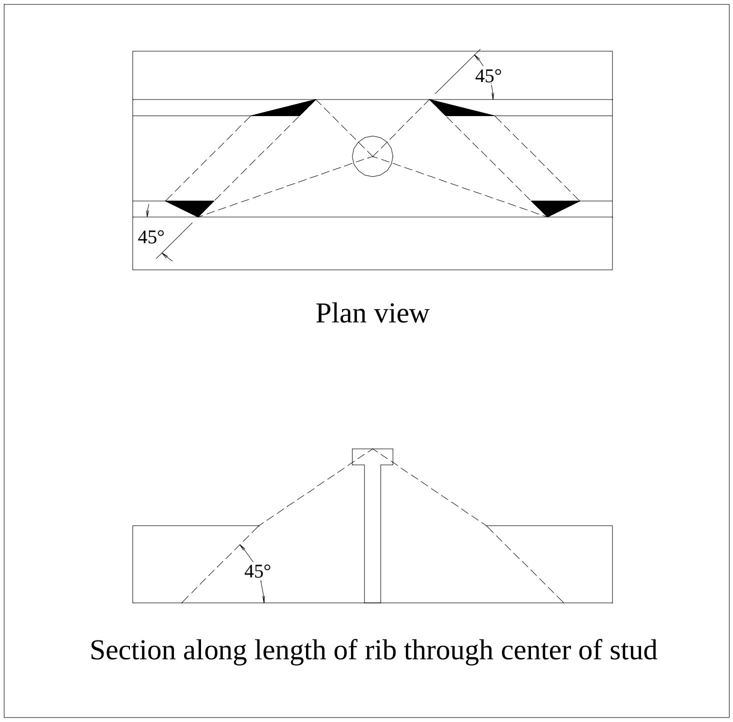

Figure 1.8 Wedged Shear Cone (Lloyd and Wright 1990)

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

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 et al. noted that their model predicted the strength of standoff screw shear connection with acceptable accuracy. Hankins et al. reported that the 5/16 in. diameter ELCO Grade 8 standoff screw can be used as an effective shear connector in composite lightweight joist sections.

Continuing the study done by Hankins et al., Alander et al. (1998) performed 106 push-out tests using ELCO Grade 8 stand-off screws. The tests of Alander et al. were organized into five series, one of which was preliminary. The common parameter for each of the rest of the series is type of the deck used. Other parameters, such as top chord thickness, screw height, and number of screws used per rib, varied from test to test and from series to series. Profiles of the screws investigated by Alander et al. are shown in Figure 1.9. Deck profiles used in each of the series are shown in Figure 1.10. Most of the features characteristic to the tests of Hankins et al. were repeated by Alander et al. One of the major differences was the introduction of the full-length steel plates between the angles to which they were welded end to end without interruptions. The goal of this adjustment was to eliminate top chord buckling that occurred occasionally in the tests of Hankins et al. and that would not be an applicable mode of failure in a regular composite open web joist setup, since the joist is continuously braced by the deck, protecting the top chord against buckling.

The slabs contained different amounts of reinforcement, and in some tests were not reinforced at all. Reinforcement consisted either of no. 4 rebar, welded wire mesh, or a combination of both. Top chord profiles ranged from 2L 1.25x1.25x0.109 to 2L 3.0x3.0x0.313.

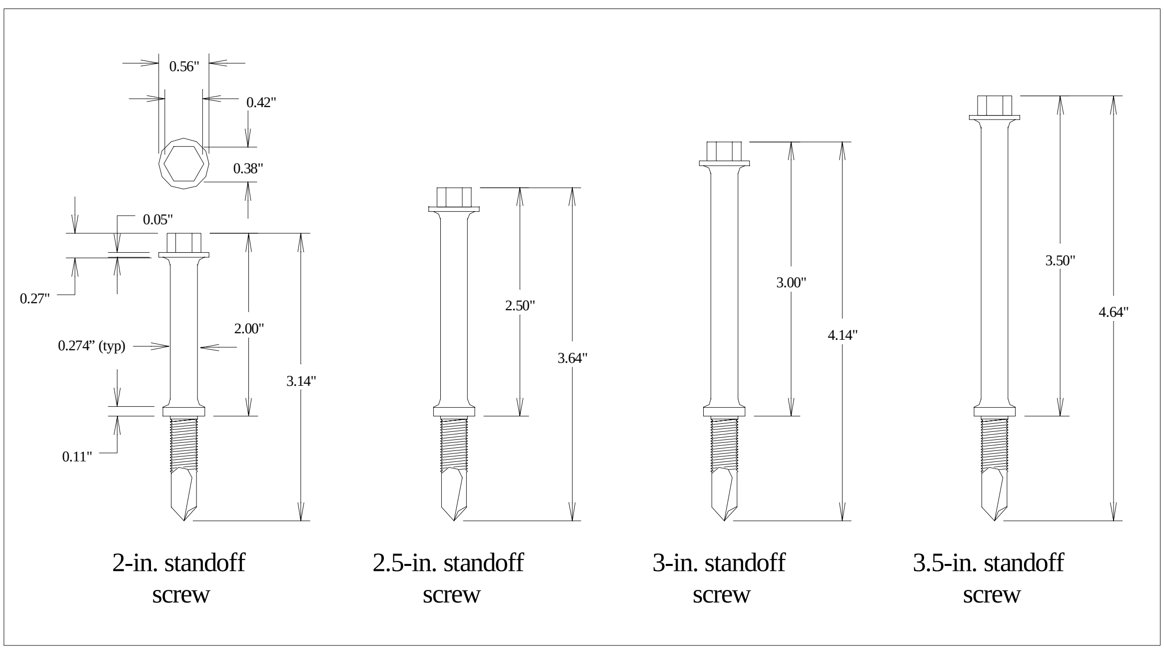

Figure 1.9 Standoff Screws Investigated by Alander et al. (1998)

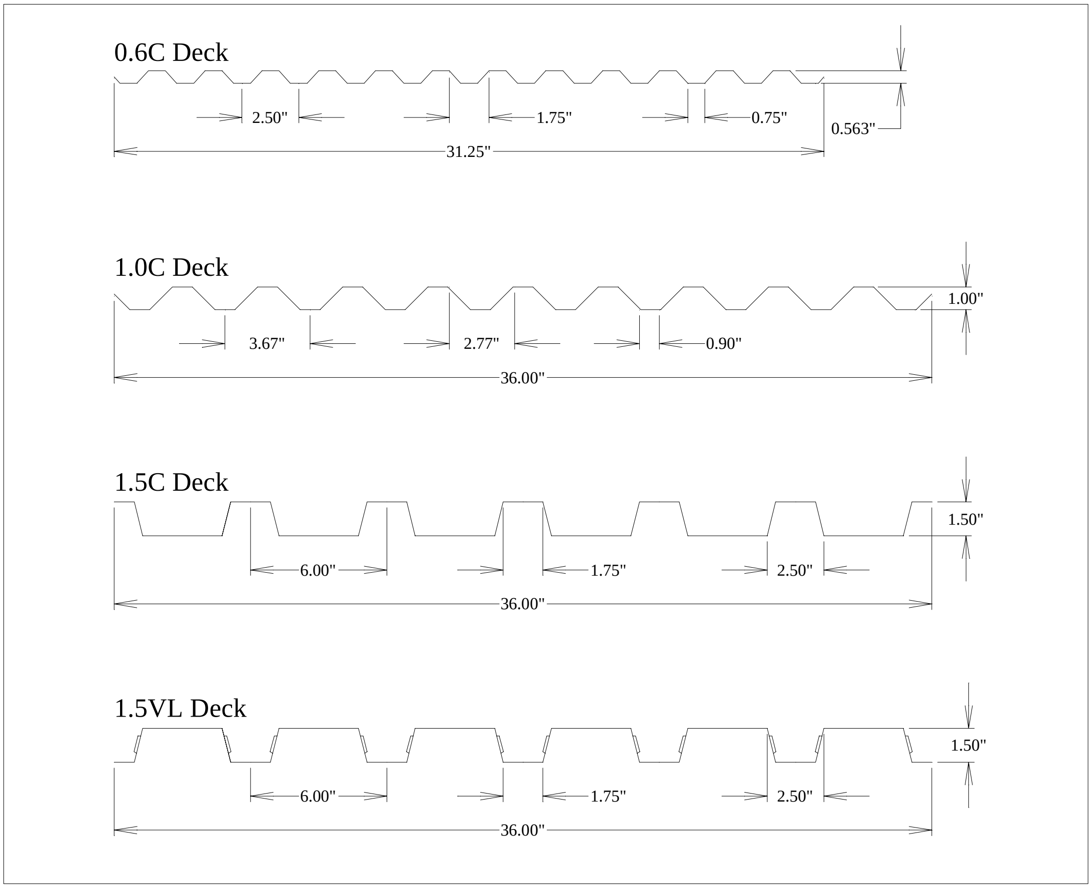

Figure 1.10 Vulcraft Deck Profiles Used by Alander et al. (1998)

Alander et al. focused on investigating the applicability of the model of Hankins et al. on other configurations. Alander et al. attempted to eliminate the effect of different concrete strengths on the strength of screw shear connection by normalizing the compressive concrete strength to 4000 psi. Alander et al. chose to evaluate the strength of standoff screws at the slip of 0.200 in. Specifically, their goal was to represent the shear connection of the screw at regular serviceability conditions. As top chord buckling was not an applicable mode of failure in composite joist sections, Alander et al. excluded from their analysis all the tests that had exhibited this sort of failure.

As a result of linear regression, Alander et al. obtained an equation (Equation 1.5), to predict the strength of shear connection using ELCO Grade 8 standoff screws to acceptable accuracy.

$$V_s = \sqrt{f_c'}\left(0.034 + 0.0012A_r + 0.068t_{TC}\right) \tag{1.5}$$

where:

V_s = shear strength per screw, kips

f_c' = concrete compressive strength, psi

A_r = rib area, in.^{2}

= average rib width $\times$ nominal rib height

t_{TC} = top chord thickness, in.

The equation Alander et al. developed, however, has certain limitations:

- Screw embedment must be at least 1.5 in. above the deck form;
- If 0.6C deck is used, there may be no more than 1 screw per rib;
- If 1.0C deck is used, there may be no more than two screws per rib;
- If 1.5C deck is used, there may be no more than 4 screws per rib;
- If 1.5VL deck is used, there may be no more than two screws per rib.

Alander et al. observed a brittle rib failure in six of the test series they investigated. In cases where this failure occurs the Equation 1.6 is not applicable. Alander et al. did not find a way of predicting the mode of failure for a particular configuration, but they did further modify the equation derived by Hankins et al. (1994), to predict the rib failure (Equation 1.6). Equation 1.6, however, also has limitations, being only applicable to sections featuring 1.5VLga22 deck and more than two ELCO Grade 8 screws per rib.

$$V_{rs} = 0.11\sqrt{A_{rs}\sqrt{f'_c}} \tag{1.6}$$

where:

V_{rs} = rib shear strength, in.

A_{rs} = rib shear failure surface area, in.^{2} (Lloyd and Wright 1990)

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

b = width of concrete rib, in.

h_r = nominal rib height of steel deck, in.

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

f'_c = concrete compressive strength, psi

Further, to predict the capacity of the standoff screw in solid slabs, Alander calculated the resistance of the slab to longitudinal shear. To do this, Alander used a model (Equation 1.7) proposed by The British Steel Construction Institute (Commentary and Load 1990).

$$v_r = 0.03\eta f_{cu} A_{cv} + 0.7 A_{sv} f_y \leq 0.8\eta A_{cv} \sqrt{f_{cu}} \tag{1.7}$$

where:

v_r = shear resistance per unit length of each shear plane, kips/in.

η = 1.0 for normal weight concrete

= 0.8 for lightweight concrete

f_{cu} = cube strength of concrete, ksi ≈ 1.25 f'_c

A_{cv} = cross-sectional area of concrete per unit length of each shear plane, in.^{2}/in.

A_{sv} = amount of steel reinforcement crossing each shear plane, in.^{2}/in.

f_y = yield strength of steel reinforcement, ksi

In the absence of actual data, Alander et al. assumed the yield strength of steel reinforcement to be 65 ksi for all the reinforcement used. The equation essentially predicts the strength of a section per unit length of shear plane. To obtain the total strength of a section, one would have to multiply the value obtained by Equation 1.7 by the total number of shear planes in the section and by the length of the slab in the direction of the applied load.

Finally, Alander et al. concluded that the application of standoff screws, as an alternative to classical welded shear connectors, is possible and that the prospect of future standoff screw use seems promising. They recommended that further experimental programs be conducted, focusing on different and as yet uninvestigated configurations. Finally, Alander et al. recommended full-scale tests as a verification process for the results of push out tests.

Webler et al. (2000) continued the study on ELCO Grade 8 standoff screws by performing an additional 59 push-out tests. With the results of their tests, Webler et al. confirmed several earlier claims, primarily the ones concerning the effects of screw rotation and angle thickness on the strength of standoff screws. Webler et al. pointed out the importance of sufficient transverse slab reinforcement, which can significantly limit the possibility of longitudinal slab splitting as a mode of failure.

Using the approach of multiple linear regression, Webler et al. derived a model (Equation 1.8), which predicted the screw strength at the value of 0.200 in. of slab to joist slip, independent of the mode of failure at the ultimate load.

$$V_s = 2.5 - 0.3N_r + 0.18A_r + 3.7t_{tc} \tag{1.8}$$

where:

V_s = shear strength per screw, kips

N_r = number of screws per rib

A_r = rib area, in.^{2}

= average rib width x normal rib height

t_{tc} = top chord thickness, in.

Webler et al. also developed several equations, based on the mode of failure, to predict the screw capacity at the ultimate load. In particular, Equation 1.9 predicted the screw strength where screw shear failure occurred.

$$V_u = 0.5A_sF_u \tag{1.9}$$

where:

V_u = ultimate shear strength per screw, kips

A_s = effective tensile area, in.^{2}

F_u = tensile stress, ksi

For configurations that use steel deck and fail by concrete rib failure, Webler et al. developed another equation (Equation 1.10) using regression techniques.

$$V_{wc} = 0.0024A_{wc}\sqrt{f'c} + 2.16 \tag{1.10}$$

where:

V_{wc} = shear strength per effective rib, kips

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

f'_c = concrete compressive strength, psi

Both Equations, 1.9 and 1.10, are valid only if screws are embedded in concrete at least 1.5 inches above the deck profile. As for solid slabs, Webler et al. suggested that the Equation 1.7 be used, regardless of the mode of failure, which could be either screw shear or longitudinal slab splitting. They based this recommendation on finding that Equation 1.7 yielded values similar to those obtained from the tests for both kinds of failure.

Further investigation of the ELCO Grade 8 standoff screw was done by Mujagic et al. (2000a). It included 23 push-out tests, and two full scale short span tests, CSJ-12 and CSJ-13, which complement the previous CSJ test series done by Lauer, Gibbings et al. (1996). As none of the existing models was adequate in predicting strength of shear connection in the full- scale tests, further analytical work on development of adequate predictive models is needed. Mujagic et al. also noted the difference in the extent to which screws closer to supports of a full-scale test are loaded as compared to those closer to the mid-span. This could potentially affect the design procedure significantly, as the distribution of screws over the span of the joist could have a major impact on strength of the entire section. This issue is presented further in Section 3.9. Mujagic et al. also pointed out that a prototype of the screw installation gun developed by ELCO was used on his full-scale tests for the first time ever. The goal of developing this new tool is to decrease the amount of labor and length of time required to install the standoff screws in regular field conditions.

1.2.2. Other Similar Shear Connectors

Research done by El-Shihy (1986) at the University of Southampton, UK, was one of the pioneer projects that originated the idea of standoff screws used in composite joists. El-Shiny investigated three types of unwelded shear connectors. Two of the considered connectors were functionally very similar to standoff screws investigated at Virginia Tech. Those two connectors were self-drilling and tapping screw (TEK) and self tapping fixed shear stud, and they were evaluated through push-out specimens. El-Shiny introduced several conclusions based on his results:

- The type of concrete used had no major on the connector strength;

- Thickness of the beam flange had no effect on the strength of connector;
- Larger number of connectors used was more likely to cause separation of concrete and deck, as well as concrete related failures;
- Rib orientation and type of deck used had no significant effect on screw strength.

Connectors investigated by El-Shiny, however, were screwed through fairly thick base metals. Specifically, the metal thicknesses used were 9.7 and 12.8 mm, or 0.382 and 0.504 in., respectively.

An abundance of research has been performed on welded headed studs, which are the most popular shear connectors used in composite sections. Some of the analytical models derived from this body of research is useful in describing the behavior of standoff screws. Lawson (1997), who studied the effect of the deck geometry on the strength of shear studs, found that pairs of studs led to lower shear strengths per stud and lower deformation capacities. Lawson proposed a concrete failure cone model and consequently a strength reduction factor (Equation 1.11), which can be used to calculate the strength of a single stud or a pair of studs in decks oriented transverse to the applied force.

$$r_p = \frac{h - D_p + 0.4 b_a}{h} \leq 1.0 \tag{1.11}$$

where:

r_p = strength reduction factor

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

D_p = deck profile height, mm

b_a = average rib width, mm

with the following limitations:

$b_a \geq 0.5h$

$h \leq 120$ mm

$h \geq D_p$+35 mm

ϕ, stud diameter, ≤ 19 mm

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

Equation 1.11 can be used where pairs of studs are used, and should be multiplied by the following reduction factor:

$$r_n = \frac{0.5s + h}{2h} \leq 0.8 \tag{1.12}$$

where:

s = distance between adjacent shear connectors (≥3ϕ), mm

When pairs of studs are located in-line with the deck ribs, b_a should be taken as (h+e), where e = 0.5b_a for a stud located in the center of the rib. These models were derived for 19 mm diameter studs. They are generally applicable to decks with relatively wide ribs, smaller diameter studs, and not for decks with narrower or deeper ribs.

Lawson's research was based on his observation that the stud strength reduction factor used in Eurocode 4 (Equation 1.13) was not very accurate when predicting the strength of shear connections in which studs were in pairs.

$$r_p = \frac{0.7}{\sqrt{N}} \frac{b_a}{D_p} \frac{(h - D_p)}{D_p} \leq 1.0,\ \leq 0.8\ \text{(for studs in pairs)} \tag{1.13}$$

where:

r_p = Eurocode 4 strength reduction factor

h = stud height (≥D_p+35 mm), mm

D_p = deck profile height, mm

b_a = average rib width, mm

N = number of studs per rib

The equation shown above and featured in Eurocode 4 is basically a modified version of the model originally proposed by Grant et al. (1977), which is the basis for the current AISC specifications (AISC 1993). The model proposed by Grant et al. (1977) was modified because it was shown to be unconservative.

Through 18 push-out tests Jayas and Hosain (1987) investigated the influence of longitudinal spacing of the headed studs and the deck geometry on the strength of shear connection. One of their findings was that in solid slabs and in slabs with ribs parallel to the direction of the load, the prevalent mode of failure was stud shear, provided the studs were spaced sufficiently apart. When this spacing was less than or equal to six times the diameter of the stud, concrete related failures occurred. When the deck ribs were oriented perpendicular to the member, stud pull-out failure, resulting in huge losses of strength, almost always occurred. Jayas and Hosain also established that the model derived by Hawkins and Mitchell (1984) used to predict the shear stud strength in pull-out failures is inaccurate for 38 and 76 mm decks. Specifically, it underestimates the stud capacity when used with the former, and it overestimates it with the latter. Jayas and Hosain developed two equations (1.14 & 1.15) to address the issues of inaccuracy of the current models.

For a 76 mm deck: $$V_c = 0.35\lambda\sqrt{f'_c}A_c \leq Q_u \tag{1.14}$$

and for a 38 mm deck: $$V_c = 0.61\lambda\sqrt{f'_c}A_c \leq Q_u \tag{1.15}$$

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

The equations developed by Jayas and Hosain that predict the shear capacity in pull-out failures were proven reliable through 4 full-scale composite beam tests and two full-scale push-out specimens.

1.2.3. Probability Based Design: General Idea and Aspects Applicable to This Study

Load and Resistance Factor Design (LRFD) is a probability based design procedure that takes into account the possibility of overload and understrength of the structure in designing the structural members for a desired factor of safety based on the probability of these effects occurring. Load factor design (LFD), which has been utilized in Europe for years, is the forerunner of LRFD. The design procedure used in Canada that is based on similar philosophy is known as Limit States Design (LSD) (Geschwindner et al. 1994).

The basics of LRFD philosophy insure that the magnitude of resistance, after being adjusted for understrength, never falls below the magnitude of load, which has been adjusted for the possibility of overload. This criteria is summarized in Equation 1.16, which was presented by Ravindra and Galambos (1978).

$$\Phi R_n \geq \sum_{k=1}^{j} \gamma_k Q_{km} \qquad (1.16)$$

where:

R_n = nominal resistance, force or moment units

Φ = resistance factor ≤ 1.0

Q_m = mean load effect, force or moment units

γ = corresponding load factor

In cooperation with other experts, depending on a particular field of expertise, Ravindra and Galambos presented a series of papers concerning LRFD procedures for many structural members in the ASCE Journal of Structural Division (September 1978). These papers represent a basis of further development of LRFD in the United States.

The paper establishing the basic and general concepts of LRFD is based on a research project conducted at Washington University from 1969 to 1976 (Ravindra and Galambos 1978). Ravindra and Galambos chose to adopt the probabilistic model proposed by Cornell (1969), due to its simplicity and ability to account for all the uncertainties consistently. The basics of the model proposed by Cornell are presented in Equations 1.17.

$$p_f = P[(R-Q) < 0] \quad \text{or} \quad p_f = P[\ln (R/Q) < 0] \qquad (1.17)$$

where:

p_f = probability of failure of a structural element

R = resistance

Q = load effect

R-Q, ln(R/Q) = safety margin

It is obvious from Equation 1.17, that failure is imminent every time resistance is exceeded by load effects. However, the probability of failure will theoretically never be zero.

This review will further concentrate on the resistance side of LRFD based procedure, as that is compatible with the objectives of this study.

Galambos and Ravindra (1973) introduced an expression (Equation 1.18) which defines the variance in resistance. Further, they simplified the expression to what is shown here as Equation 1.19. As variances of material, fabrication, and theory are generally less than 0.2, they noted that this simplification would result in the total resistance variance error of less than 5%.

$$V_R = \sqrt{V_M^2 + V_F^2 + V_P^2 + V_M^2 V_F^2 + V_M^2 V_P^2 + V_F^2 V_P^2 + V_M^2 V_F^2 V_P^2} \qquad (1.18)$$

$$V_R = \sqrt{V_M^2 + V_F^2 + V_P^2} \qquad (1.19)$$

where:

V_R = variation in resistance

V_M = variation in material

V_F = variation in manufacturing

V_P = variation in design theory

In absence of more precise data, Galambos and Ravindra assumed that variance in fabrication is relatively small. Variance in material, as they found, may be represented by normal, lognormal, or Weibull distributions. Variance in design theory is also known as professional variance, hence “P.” This quantity, in absence of reliable data, is unknown, as the assumptions such as perfect plasticity, perfect elasticity, or homogeneity cannot be theoretically evaluated, but the variance in design theory might be evaluated if the test data are available and compared with a mathematical or statistical model describing a behavioral pattern.

For the purposes of this analysis, Galambos and Ravindra also introduced the quantity of β, which is commonly known as the "safety index," and on a probability distribution chart it represents a distance between the magnitude of safety margin and the dividing line between failure and survival. This idea is communicated through Figure 1.11, and Equation 1.20.

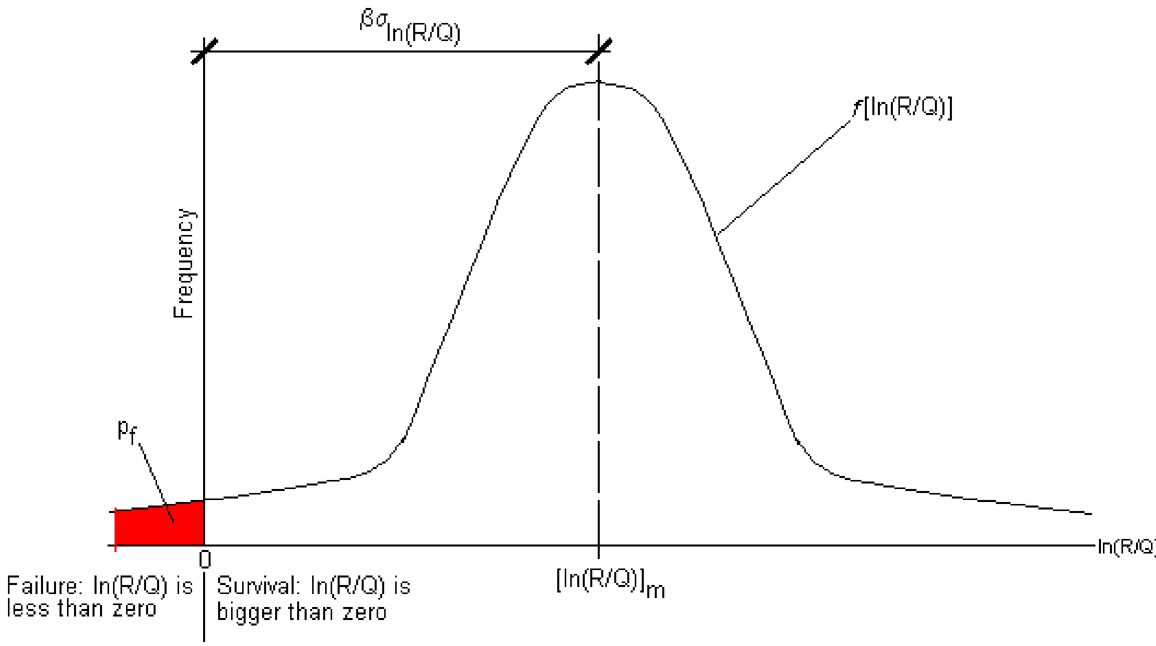

Figure 1.11 Representation of Safety Index, β, per Galambos and Ravindra (1973)

$$\beta = \frac{[\ln(R/Q)]_m}{\sigma_{\ln(R/Q)}} \tag{1.20}$$

where:

β = safety index or reliability

$\sigma_{\ln(R/Q)}$ = standard deviation of natural logarithm of the (R/Q) ratio

[ln(R/Q)] = mean value of of natural logarithm of the (R/Q) ratio

Reliability is a chosen value. It represents the desired level of safety (reliability) a particular structural component is to have. It can be chosen by virtue of code calibration, in which it is set so that the level of safety in the new specifications (i.e. LRFD) is approximately the same as that in the old specifications (i.e. ASD). The second possibility is that the writers of a structural design code agree upon the appropriate level of reliability for a certain structural component (Geschwindner et al. 1994). The latter method is more characteristic for newly researched and developed structural elements, that were not covered under the provisions of current and previous structural codes. As for the code calibration, LRFD specifications are calibrated so that the reliability achieved for the newly designed structures is similar to those designed by 1978 AISC specifications, and at the live to dead load ratio of about 3 (Gaylord 1992).

Next, Galambos and Ravindra, adopting the findings of Ang and Cornell (1974), defined the central safety factor, shown as Equation 1.21. This quantity, as they indicate, considers the variances, or uncertainties, of both the resistance and the load effects.

$$\theta = \exp\left[\beta(V_R^2+V_Q^2)^{1/2}\right] \tag{1.21}$$

where:

θ = central safety factor

Finally, Galambos and Ravindra completed an analysis procedure to find a way in which to evaluate resistance factors independently, regardless of load effects. They did it by essentially splitting the central factor of safety into two factors, one for the load, and one for the resistance. Details of that analysis are not presented here, but the resulting equation that can be used for calculating the resistance factor, Φ, is shown as Equation 1.22.

$$\Phi = \exp(-\alpha\,\beta\,V_R)(R_m/R_n) \tag{1.22}$$

where:

R_n = nominal resistance

R_m = mean measured resistance

α = 0.55, a factor determined through error minimization procedure

Of special interest to this study are the reliability indexes suggested for connectors. Fisher, Galambos, Kulak, and Ravindra (1978) suggested that the reliability factor of 3.0 used for flexural members may not be satisfactory for connections because they should desirably be stronger and thus safer than the members. They conducted a calibration procedure similar to that done for beams and columns to arrive at new reliability indexes that may be appropriate for connections. As a result of this study, a reliability factor of 1.5 was suggested for all friction-type connections, while 4.5 was the magnitude of the safety index stipulated for all other types of connections.

When developing the limit states based design procedure for composite flexural members, Galambos et al. (1976) did not derive separate strength reduction factors for the shear connector, and the flexural member. Instead, they established one single strength reduction factor to be applied to the strength of the flexural member only. Nonetheless, this factor of 0.85 was to some extent adjusted for the influence of the strength of shear connectors. The only material variance Galambos et al. considered while investigating the effect of shear connector on the strength factor was that of concrete. As for flexural members, Galambos et al. utilized various written sources to obtain the material variances for steel shapes, concrete, and reinforcement. Experimental results utilized in their study mostly came from the tests performed at Lehigh University.

The current trends lean towards the idea of separating the statistical characteristics of the shear connector and the flexural member and either evaluate separate load factors for both, or improve the way in which the statistical variance of the shear connector is affecting the strength of the flexural member.

1.3. Research Objectives

The objective of the research reported herein is to present a comprehensive evaluation of all available experimental data from push-out, full-scale, and simple shear and tensile tests utilizing the ELCO Grade 8 standoff screws. The goal is to develop a strength prediction equation and determine reliability parameters compatible with the Load Factor Resistance Design (LRFD) procedure that would allow the use of this shear connector in design of composite floor systems.

The study considers results from push-out tests featuring this type of screw reported by Hankins et al. (1994), Alander et al. (1998), Webler et al. (2000), and Mujagic et al. (2000). Further, this study identifies the limitations in earlier approaches aimed at predicting the strength of standoff screws. An improved strength prediction model is developed that considers all applicable limit states and determines maximum strength of a connector. A reliability study, as a part of this research, was also conducted to derive strength reduction factors to be used in design. Parameters considered in development of the model include deck type and geometry, screw height, concrete compressive strength, top chord angle yield strength, and stand-off screw rupture strength. Results from strength prediction model were compared with results from composite joist tests.

The model, which can successfully and to an acceptable degree of accuracy predict the strength of the shear connection between the slab and steel joist featuring ELCO Grade 8 standoff screws, as well as the mode of failure, is evaluated by comparing its predictions to the results of existing full scale short span composite joist tests that utilized this screw.

The model derived predicts the ultimate screw strength, rather than its strength at 0.200 in. of slip, as was done in previous studies.

Several additional issues were identified during this study. They are addressed in the chapters that follow and should be subjects of subsequent studies. These issues pertain to ductility of shear connections with ELCO Grade 8 standoff screws, and differences in design procedure of composite beams and composite joists from the standpoint of the LRFD method.