

CHAPTER G
REVIEW OF LITERATURE

The standoff screw, illustrated in Fig. G.1, 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 reflect 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.

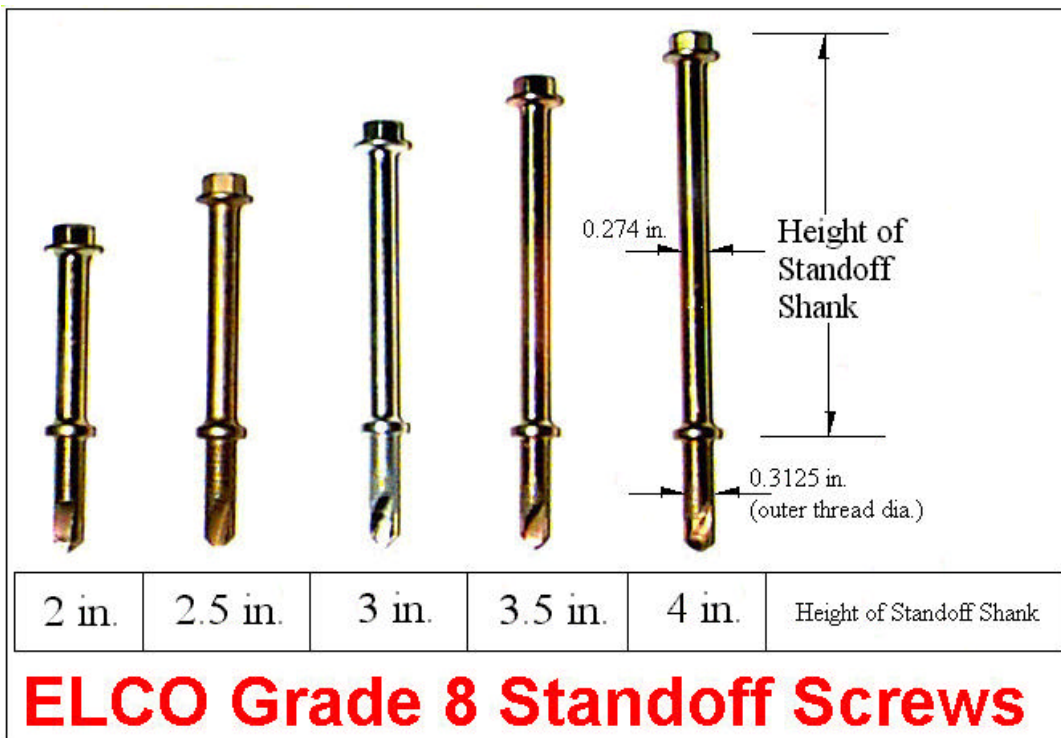


Fig. G.1 ELCO Grade 8 Standoff Screws

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

Many authors have discussed the significance of ductility of shear connections in composite floor structures. Some of the highlights of that will be reviewed herein as well.

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

G.1 Standoff Screws

The initial work in this area at Virginia Tech was done by Strocchia et al. (1990). Strocchia conducted a total of 36 push-out tests, organized in 13 test series. Among the six different types of deck fasteners investigated, four were standoff screws. These screws and their dimensions are shown in Fig. G.2. The type of deck used in all the tests, shown in Fig. G.3, was 1.5VLga22 manufactured by Vulcraft.

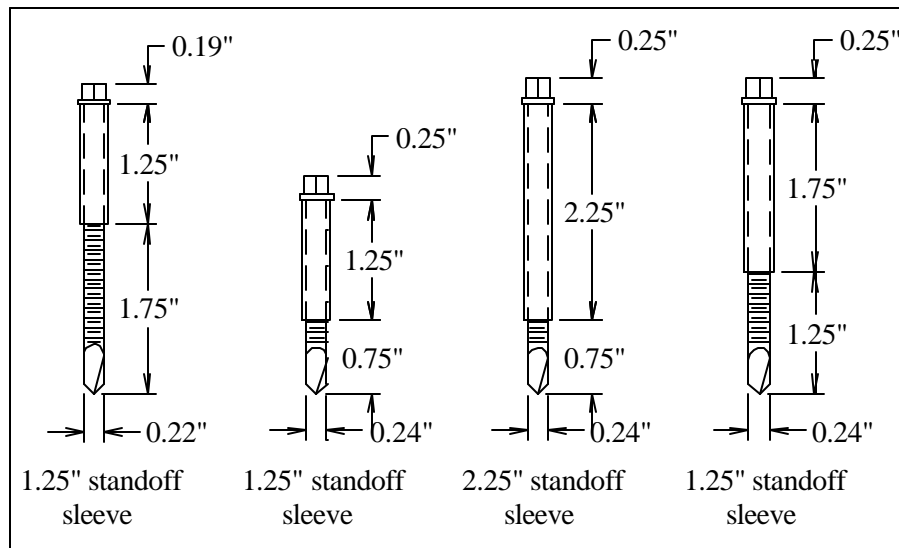


Fig. G.2 Self-tapping Screws with Standoff Sleeves (Strocchia et al. 1990)

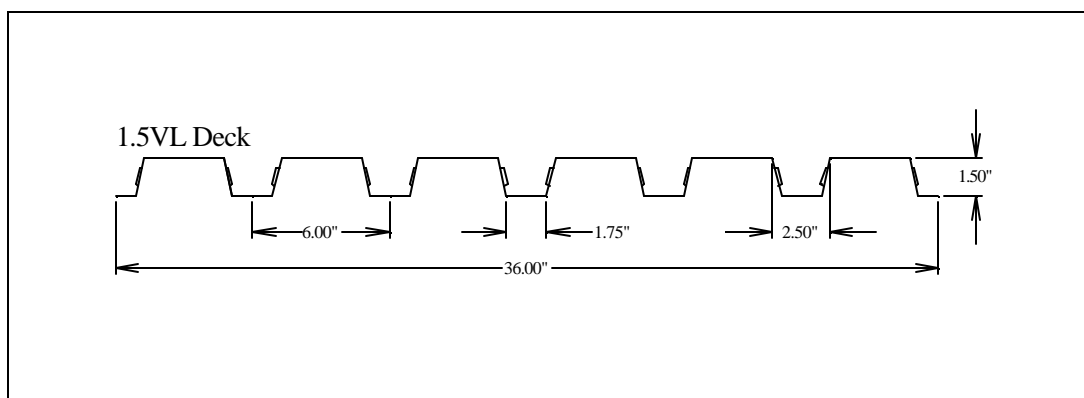


Fig. G.3 Vulcraft Deck Profile Used by Strocchia et al. (1990)

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. (1990) 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 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 may 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. (1996) 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 Fig. G.4, were used in 68 of the tests. The remaining six tests were conducted using ELCO Grade 5 and Buildex screws, respectively.

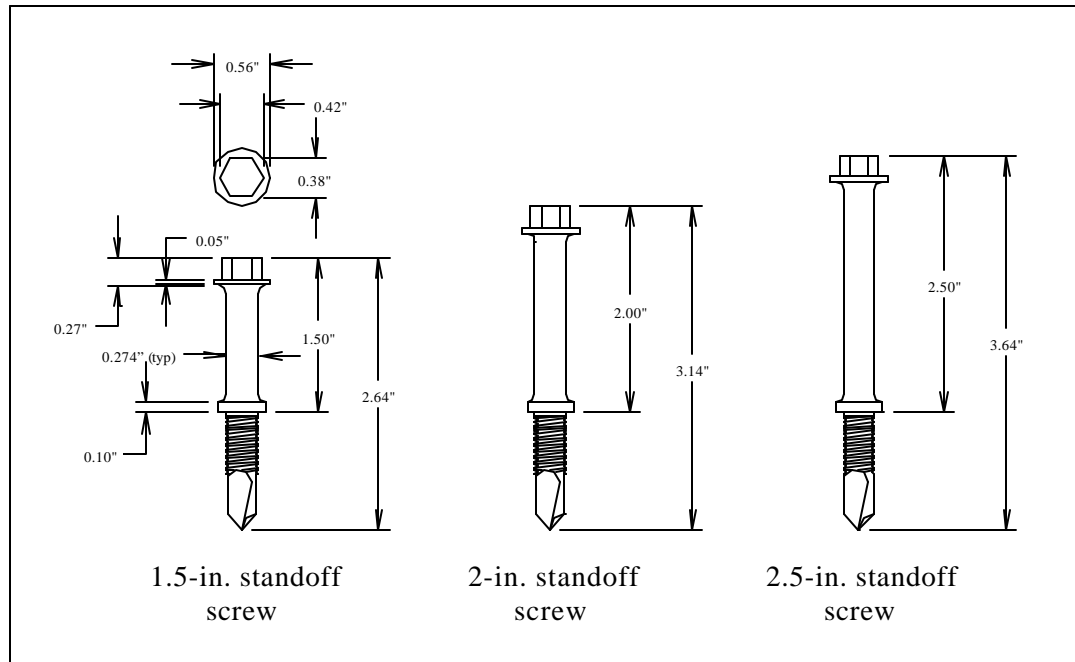


Fig. G.4 Standoff Screws Investigated by Hankins et al (1994)

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, Lauer et al. were able to predict the capacity of the 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 Eq. G.1, and G.2, which represent an equilibrium state of a composite flexural member, to back-calculate the strength of the shear connectors.

$$\sum Q_{ae} \cdot e \pm N_a \cdot e^l = M_{ae} \quad (\text{G.1})$$

$$\sum Q_{ae} \cdot e_t + T_a \cdot e^l = M_{ae} \quad (\text{G.2})$$

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^l = 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 Eqs. G.1 and G.2 and compared those to the values obtained from push-out tests (Table G.1).

Table G.1 Experimental Shear Connection (Lauer et al. 1996)

	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 Eq. (G.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 Eq. (G.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 nominal 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 Fig. G.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 Fig. G.5.

Hankins et al. observed and identified five different failure modes in the push-out tests they 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;

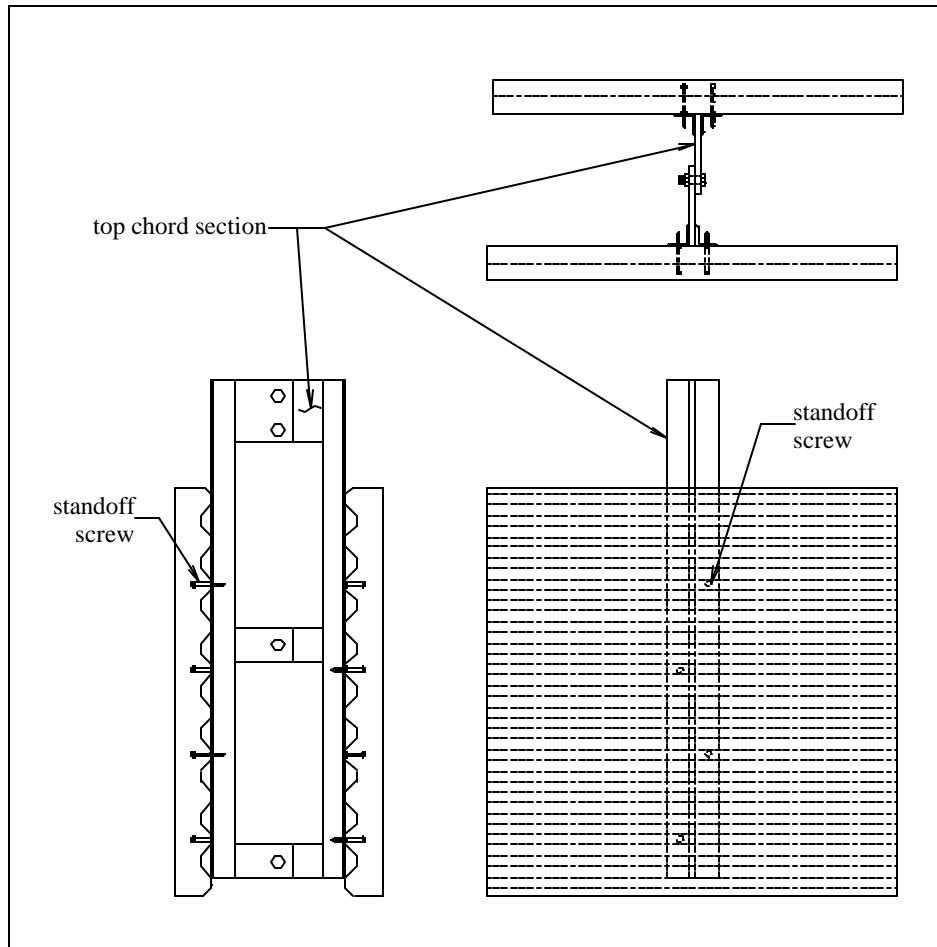


Fig. G.5 Typical Push-out Specimen Used by Hankins et al. (1994)

Hankins et al. concluded that the screw strength is related to the thickness of the 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 angles 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 in the tests performed by Hankins et al., as shown in Fig. G.6, caused significant angle deformations, which ultimately caused the top chord to buckle.

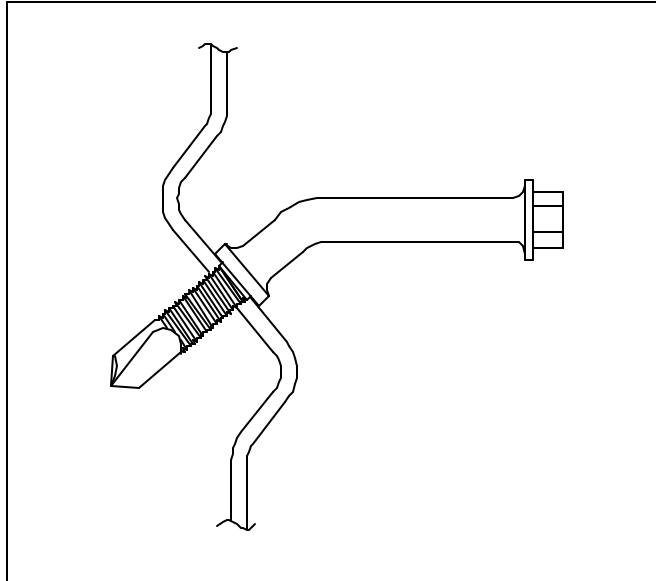


Fig. G.6 Typical Screw Rotation (Hankins et al. 1994)

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 Eq. G.3 predicted the strength of shear connection in solid slabs, as found by Hankins et al., with acceptable accuracy.

$$P_s = \frac{0.6h_a f_t b p}{\left(1 - \frac{d_s}{b}\right)^2} \quad (\text{G.3})$$

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 G.7 shows the generated shear load path, and Fig. G.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. The shear path of least resistance, as defined by Lloyd and Wright, however, was in agreement with the path of concrete cone failures that Hankins et al. observed in their push-out tests. In the absence of other adequate models, Hankins et al. developed their own equation (Eq. G.4) that 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.

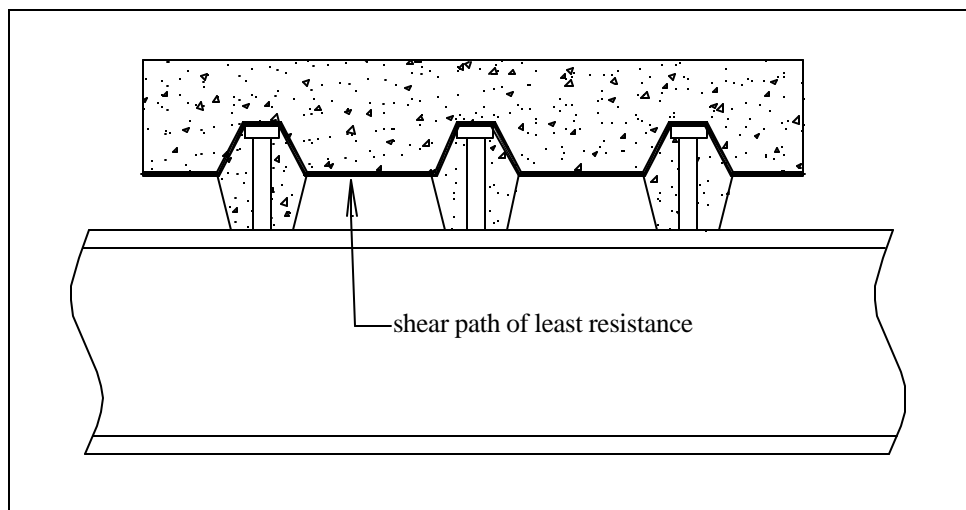


Fig. G.7 Longitudinal Shear Path (Lloyd and Wright 1990)

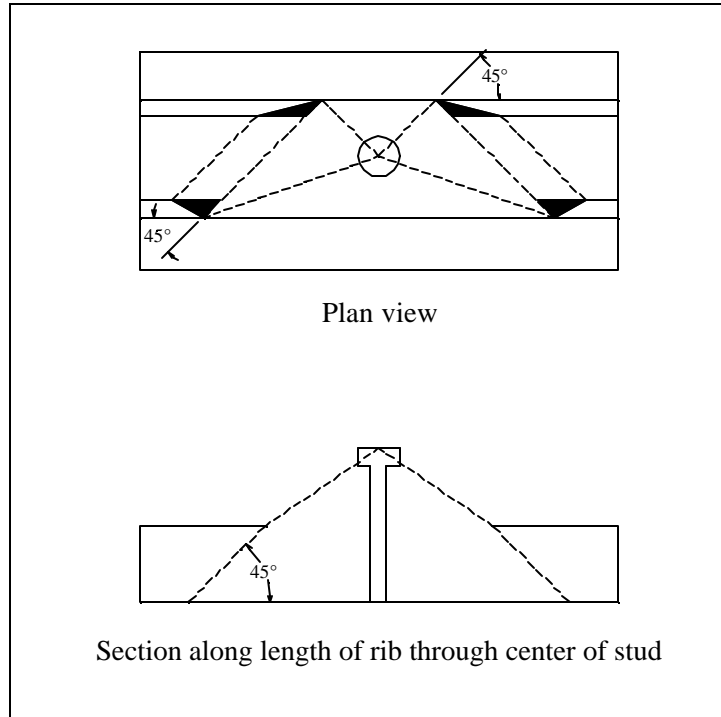


Fig. G.8 Wedged Shear Cone (Lloyd and Wright 1990)

$$V_{wc} = 0.11\sqrt{A_{wc}}\sqrt{f'_c} \quad (G.4)$$

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 the type of 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 Fig. G.9. Deck profiles used in each of the series are shown in Fig. G.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 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.

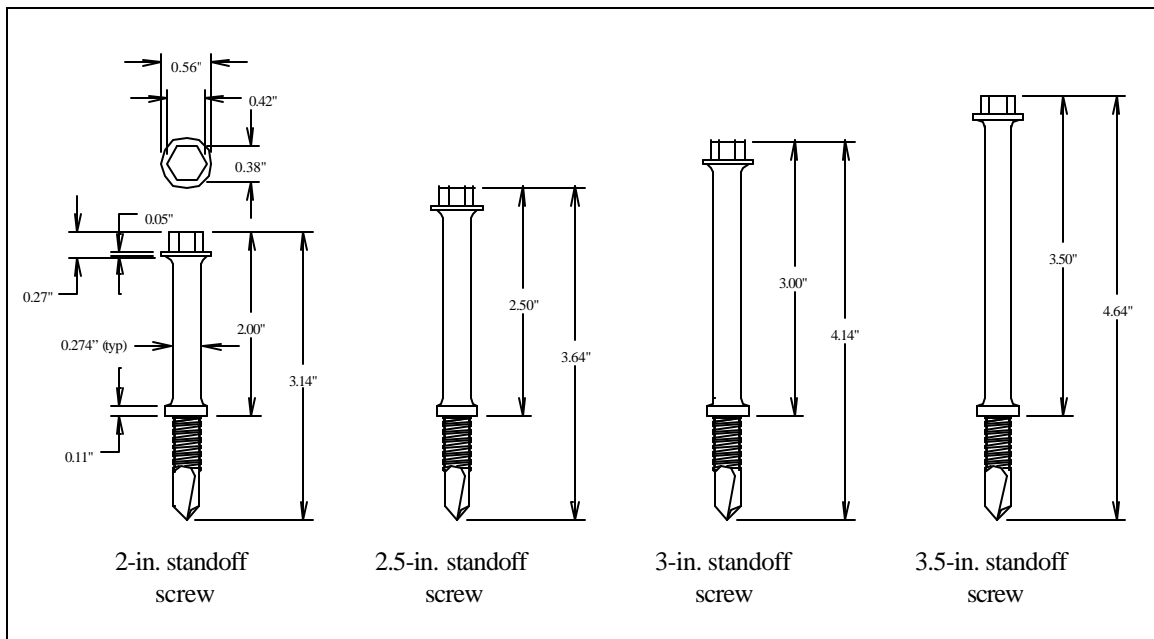


Fig. G.9 Standoff Screws Investigated by Alander et al. (1998)

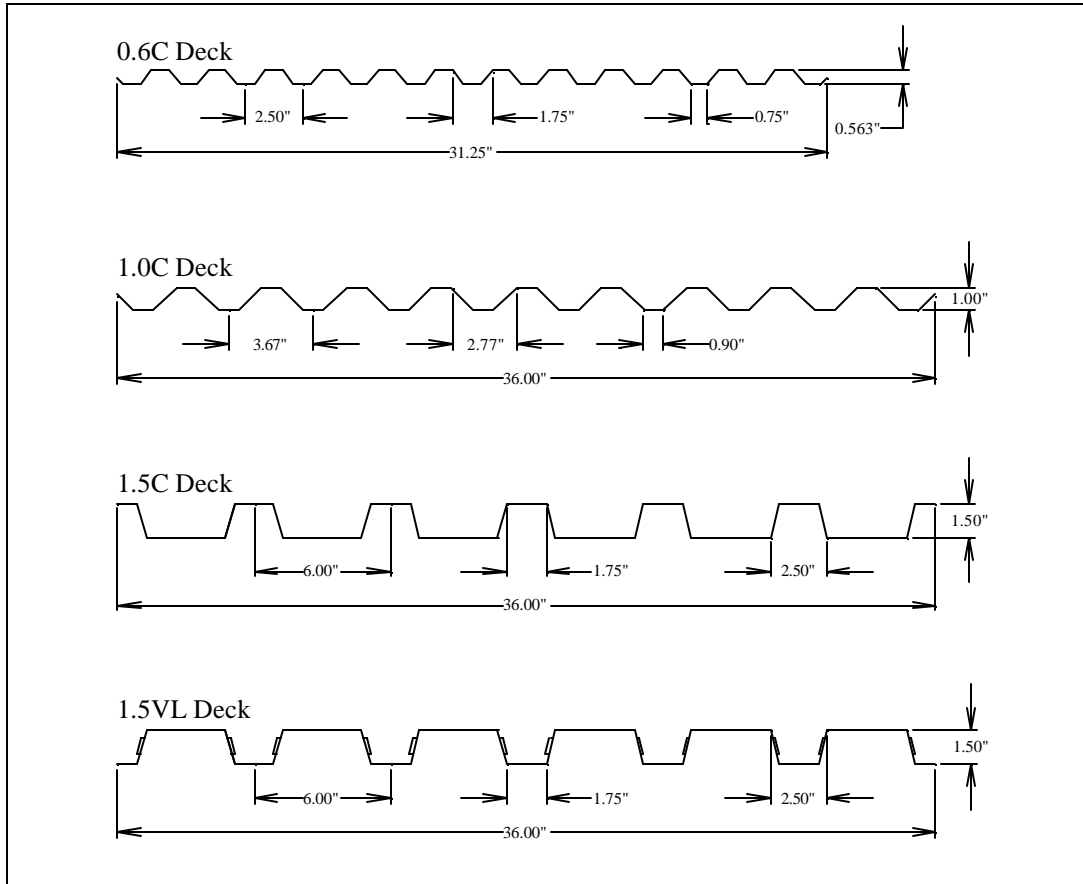


Fig. G.10 Vulcraft Deck Profiles Used by Alander et al. (1998)

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 (Eq. G.5), to predict the strength of shear connection using ELCO Grade 8 standoff screws to acceptable accuracy.

$$V_s = \sqrt{f_c'}(0.034 + 0.0012A_r + 0.068t_{TC}) \quad (G.5)$$

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, Eq. G.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 rib failure (Eq. G.6). Equation G.6, however, also has limitations, being only applicable to sections featuring 1.5VLga22 deck and more than two ELCO Grade 8 screws per rib.

$$V_{rs} = 0.11\sqrt{A_{rs}}\sqrt{f'_c} \quad (\text{G.6})$$

where:

- V_{rs} = rib shear strength, in.
- A_{rs} = rib shear failure surface area, in.² (Lloyd and Wright 1990)
= $w_{r2}\sqrt{\frac{b^2}{4} + (H_s - h_r)^2} + b\sqrt{\frac{w_{r2}^2}{4} + (H_s - h_r)^2}$
- 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 (Eq. G.7) proposed by The British Steel Construction Institute (Commentary and Load 1990).

$$v_r = 0.03hf_{cu}A_{cv} + 0.7A_{sv}f_y \leq 0.8hA_{cv}\sqrt{f_{cu}} \quad (\text{G.7})$$

where:

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

h = 1.0 for normal weight concrete

= 0.8 for lightweight concrete

f_{cu} = cube strength of concrete, ksi $\approx 1.25 f'_c$

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 Eq. G.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 (Eq. G.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.

$$V_s = 2.5 - 0.3N_r + 0.18A_r + 3.7t_{tc} \quad (G.8)$$

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, Eq. G.9 predicted the screw strength where screw shear failure occurred.

$$V_u = 0.5A_sF_u \quad (G.9)$$

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 (Eq. G.10) using regression techniques.

$$V_{wc} = 0.0024A_{wc}\sqrt{f'c} + 2.16 \quad (G.10)$$

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 Eqs., G.9 and G.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 Eq. G.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 Eq. G.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 et al. (1996). As none of the existing models were adequate in predicting the 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 further presented and dealt with in the first manuscript of this dissertation. 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.

Mujagic et al. (2001a) conducted a comprehensive study with the goal of deriving a strength prediction model based on available data from all tests performed to date at Virginia Tech. The strength prediction model was derived based on the results of 254 push-out tests. The results from six available full-scale short-span joist tests that featured standoff screws as shear connectors were used to evaluate the performance of the strength prediction model.

This study was based on the realization that the failures of shear connection in composite floors occur in three different modes: screw shear, concrete rib failure, and screw pullout. Each of the failures was analyzed separately, and the final strength prediction model represents a synthesis of those analyses. Unlike in previous studies, the intent was to derive a strength prediction model that adequately predicts the ultimate strength of the connector,

rather than that at certain value of slab to joist slip. Due to the principal interest in the ultimate strength, it is not uncommon to exclude slip measuring devices all together and simply test the push-out specimen to failure, with the strength at failure being the data point of foremost concern (Davies 1975).

In addition to deriving the strength prediction model, a reliability study aimed at deriving a strength reduction factor for the ELCO Grade 8 standoff screws was also conducted. It was shown that the derived strength reduction factor should ideally be applied to the strength of the standoff screw and that this reduced strength should be used in the calculation of composite member moment capacity. The proposed ELCO Grade 8 standoff screw strength prediction model is shown below as Eq. G.11.

When the rib is perpendicular to the joist:

$$\phi R_n = \phi \left[\begin{array}{l} 36.71 (t_{tc})^{1.61} \left(\frac{H_s}{h_r} \right)^{0.75} (1 - 0.15w_{r1}^2 + 0.98w_{r1}) \\ \frac{0.18(\ln f'_c)L_{sp}w_{r1}^{0.13}}{N^{0.74}} \leq CA_{sc}F_{ut} \end{array} \right] \quad (G.11a)$$

When the rib is parallel to the joist and for solid slabs:

$$\phi R_n = \phi 0.45A_{sc}F_{ut} \quad (G.11b)$$

where:

- ϕ = strength reduction factor = 0.85
- R_n = shear strength per screw, kips
- t_{tc} = top chord thickness, in.
- H_s = screw height, in.
- h_r = rib height, in.
- w_{r1} = bottom rib width, in.
- f'_c = concrete compressive strength, psi
- L_{sp} = length of the shear plane, in.

$$L_{sp} = 2\sqrt{\left(\frac{w_{r2} - l_s}{2}\right)^2 + (H_s - h_r)^2} + l_s$$

- w_{r2} = top rib width, in.

- N = number of screws per rib ≤ 12
- C = top chord thickness adjustment factor
 $= (2/5)$ for $t_c \leq 0.205$ in., $(2/3)(1-2t_c)$ for $t_c > 0.205$ in.
- A_{sc} = nominal cross-sectional area of the screw, in.²
- F_{ut} = screw tensile strength stress, ksi
- l_s = vertical distance between screws in a rib, in.

The model shown above has the following limitations:

- The top chord thickness shall be in the range of 0.109 in. to 0.250 in. The lower limit is the thinnest tested thickness and the upper limit is governed by both the self-tapping characteristics of the screw and ductility;
- The yield strength of the top chord angle shall nominally be 50 ksi. At this point, it is unknown, how a significantly lower or higher top chord angle yield strength would affect the strength of shear connection;
- The number of screws per rib shall in no instance exceed 12, as that is the highest number of screws per rib experimentally evaluated. Most of the time this number will be governed by practical considerations as the space available for the installation of screws in narrower ribs is fairly limited.

Mujagic et al. concluded that standoff screws are a reliable alternative to welded shear studs in light-weight composite floor joists. Their overall ductility, relatively simple installation, and the elimination of welding are major advantages. A relatively high number of screws required in larger joists and a lack of ductile behavior when used in joists with thicker top chords are the most prominent disadvantages.

As a result of their analysis, Mujagic et al. also concluded the following:

- Longitudinal rib splitting, the occurrence of which is not characteristic for full-scale specimens, was happening routinely in push-out specimens that either contained a relatively large total number of screws per specimen, or were not sufficiently reinforced against rib splitting.
- The effect of top chord angle yield strength on the strength of shear connection failing by screw pullout could not be fully realized from the available data due to the small range in the angle yield strengths.

- ELCO Grade 8 standoff screws do not exhibit a satisfactory level of ductility when used in joists with relatively thick top chords. The results of CSJ-13 indicated that no redistribution of horizontal shear force occurs. Screws closer to the supports therefore failed first. At the same time, screws closer to the mid-span carried significantly smaller load, or no load at all, at the time of the failure of the first shear connectors.
- The results of the study suggested the need for a different design procedure for composite joists than for composite beams in terms of how the strength reduction factor, ϕ , is applied. It appeared that the moment capacity of composite joists is directly proportional to the strength of the shear connector used, and thus, the statistical characteristics of the strength of the shear connector have a direct effect on the strength of the composite section itself. The situation is markedly different with respect to composite beams; their moment strengths are not directly proportional to the strength of the shear connectors used. Also, statistical characteristics of the shear connectors affect the moment capacity of a composite beam differently at different percentages of composite action. These differences result from neglecting the contribution of the top chord to the moment capacity of composite joists. The resulting approach for composite joists would be to apply a separate strength reduction factor to the strength of the shear connector, and to then use that reduced strength in the calculation of the composite joist moment capacity. Corresponding reliability studies were aimed at finding a new strength reduction factor for the moment capacity of composite joists.

Mujagic et al. (2001b) presented a simplified strength calculation model for finding the strength of standoff screws in composite joists (Equation G.12). This model was based on that shown as Eq. G.11. It stems from the attempt to make the standoff screw strength calculations simpler and user-friendlier without a significant negative impact on accuracy.

Although less liberal in strength prediction of the standoff screw, the model was shown to be less safe due to loss of accuracy, as its strength reduction factor was computed at somewhat lower reliability from the original model. Comparison of statistical characteristics of the original and the simplified model are shown in Fig. G.11.

When the rib is perpendicular to the joist:

$$\phi R_n = \phi \min \left[\begin{array}{l} 130 t_{tc}^{1.6} \\ \frac{A_{ts} F_{ut}}{\sqrt{3}} \\ \frac{0.5 L_{sp}^2}{N} \end{array} \right] \quad (G.12a)$$

When the rib is parallel to the joist and for solid slabs:

$$\phi R_n = \phi \frac{A_{ts} F_{ut}}{\sqrt{3}} \quad (G.12b)$$

where:

$$A_{ts} = \text{tensile stress area of the screw, in.}^2 \\ = 0.7854(D - 0.9743/n)^2$$

D = nominal screw diameter, in.

n = number of threads per inch

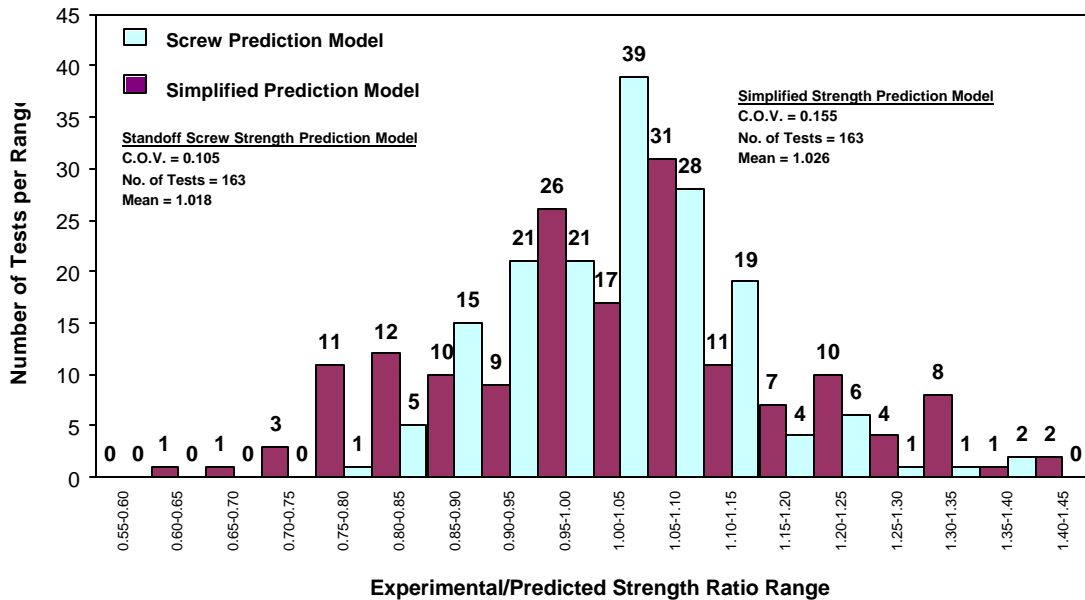


Fig. G.11 Comparison of Strength Ratios for Standoff Screw Calculation Models

The study on standoff screws was continued at Virginia Tech by Mason et al. (2002). Mason et al. conducted 44 push-out tests. A large number of these contained a scope of variables not used to develop strength prediction models by Mujagic et al. (2001a,b). This is especially true with respect to the thickness of top chord. Mason et al. concluded that the models developed by Mujagic et al. need to be adjusted so that the range of values of

different variables for which the model is valid can be widened. This primarily concerns higher top chord thicknesses.

G.1.1 Other Similar Shear Connectors and Related Analyses

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 influence on the connector strength;
- Thickness of the beam flange had no effect on the strength of the connector;
- The 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 material. Specifically, the metal thicknesses used were 9.70 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) studied the effect of the deck geometry on the strength of shear studs and found that pairs of studs lead to lower shear strengths per stud and lower deformation capacities. Lawson proposed a concrete failure cone model and consequently a strength reduction factor (Eq. G.13), 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.

$$r_p = \frac{h - D_p + 0.4b_a}{h} \leq 1.0 \quad (\text{G.13})$$

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:

$$b_a \geq 0.5h$$

$$h \leq 120 \text{ mm}$$

$$h \geq D_p + 35 \text{ mm}$$

$$\phi, \text{ stud diameter, } \leq 19 \text{ mm}$$

$$\text{If } b_a > 2h, \text{ then } b_a = 2h$$

Equation G.13 can be used where pairs of studs are used, and should be multiplied by the following reduction factor:

$$r_n = \frac{0.5s + h}{2h} \leq 0.8 \quad (\text{G.14})$$

where:

$$s = \text{distance between adjacent shear connectors } (\geq 3\phi), \text{ mm}$$

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 (CEN 1992a), as given by Eq. G.15, was not very accurate when predicting the strength of shear connections in which studs were in pairs.

$$r_p = \frac{0.7}{\sqrt{N}} \frac{b_a}{D_p} \frac{(h - D_p)}{D_p} \leq 1.0, \leq 0.8 \text{ (for studs in pairs)} \quad (\text{G.15})$$

where:

$$r_p = \text{Eurocode 4 strength reduction factor}$$

- h = stud height ($\geq D_p + 35$ 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 specifications of the American Institute of Steel Construction (AISC 1999). The model proposed by Grant et al. (1977) was modified in this specification as it was shown to be unconservative.

Summary of current AISC requirements (AISC 1999) for computation of strength of headed studs in composite slabs is given in Eqs. G.16, G.17, and G.18. The nominal strength of one shear stud embedded in solid concrete slab is given by:

$$Q_n = 0.5A_{sc}\sqrt{f'_c E_c} \leq A_{sc}F_u \quad (\text{G.16})$$

where:

- Q_n = nominal strength of one stud, kips
- A_{sc} = cross-sectional area of stud shear connector, in.²
- f'_c = concrete compressive strength stress, ksi
- E_c = modulus of elasticity, ksi
- F_u = specified minimum tensile strength of stud shear connector, ksi

When composite section of interest features formed deck, with ribs oriented perpendicular to steel beam, the reduction factor shown in Eq. G.17 is to be applied only to the first term of Eq. G.16.

$$SSRF = \frac{0.85}{\sqrt{N}} \frac{w_r}{h_r} \left[\left(\frac{H_s}{h_r} \right) - 1.0 \right] \leq 1.0 \quad (\text{G.17})$$

where:

- $SSRF$ = stud strength reduction factor
- N = number of studs per rib, not larger than three in computations

- w_r = average width of concrete rib or haunch, in.
- h_r = deck profile height, in.
- H_s = stud height after welding, not larger than $h_f + 3$ in. in computations, in.

The SSRF given by Eq. 17 is limited to 0.75 if only one stud is placed in a rib oriented perpendicular to the steel beam (AISC 1999). This is a change from the 1993 AISC Specification, which did not require this reduction. The reduction was proposed by Easterling et al. (1993), as it was shown that Eq. G.16 is unconservative in situations where only one stud per rib is used.

Finally, where studs are present in deck ribs oriented parallel to steel beam where w_r/h_f is less than 1.5, 1999 AISC Specifications require the following stud strength reduction factor:

$$SSRF = 0.60 \frac{w_r}{h_r} \left[\left(\frac{H_s}{h_r} \right) - 1.0 \right] \leq 1.0 \quad (\text{G.18})$$

Rambo-Roddenberry et al. (2002) proposed a new strength prediction model (Eq. G.19) for headed shear studs in composite beams. The authors reported 117 push-out tests and three composite beam tests. In conjunction with these, and other previously available results, the authors evaluated various models for prediction of shear stud strengths. It was decided that all the existing models, including the one stipulated by AISC either give unconservative results, or are impractical for use in routine design calculations. The model proposed by Rambo-Roddenberry et al. was derived based on results of available push-out tests, and its performance was evaluated using the results of full-scale tests. The new model also includes stud position within the rib as a parameter, while concrete strength parameters were excluded as variables. The authors recommend the phi-factor of 0.88 be applied to the moment strength of the composite beam, assuming that the stud strength is calculated using their model and beam flexural strength is computed using the present AISC method. Most of the research on which these recommendations are based was performed using 3/4-in. diameter studs. The authors point out that more research is required to evaluate performance of smaller and larger studs.

For studs in 2 in. and 3 in. deck with $d/t \leq 2.7$:

$$Q_{sc} = R_p R_n R_d A_s F_u \quad (\text{G.19a})$$

$$\begin{aligned} R_p &= 0.68 \text{ for } e_{mid-ht.} \geq 2.2'' \text{ (strong position studs)} \\ &= 0.48 \text{ for } e_{mid-ht.} < 2.2'' \text{ (weak position studs)} \\ &= 0.52 \text{ for staggered position studs} \\ R_n &= 1.0 \text{ for one stud per rib or staggered position studs} \\ &= 0.85 \text{ for two studs per rib} \\ R_d &= 1.0 \text{ for all strong position studs} \\ &= 0.88 \text{ for 22 gauge deck (weak studs)} \\ &= 1.0 \text{ for 20 gauge deck (weak studs)} \\ &= 1.05 \text{ for 18 gauge deck (weak studs)} \\ &= 1.11 \text{ for 16 gauge deck (weak studs)} \end{aligned}$$

For studs in 1 in. and 1 1/2 in. deck with $d/t \leq 2.7$:

$$Q_{sc} = R_n 3.08 e^{0.048 A_s F_u} \quad (\text{G.19b})$$

$$\begin{aligned} R_n &= 1.0 \text{ for one stud per rib} \\ &= 0.85 \text{ for two studs per rib} \end{aligned}$$

For studs in 2 in. and 3 in. deck with $d/t > 2.7$:

$$Q_{sc} = R_p R_n R_d A_s F_u - 1.5 \left(\frac{d}{t} - 2.7 \right) \quad (\text{G.19c})$$

For studs in 1 in. and 1 1/2 in. deck with $d/t > 2.7$:

$$Q_{sc} = R_n 3.08 e^{0.048 A_s F_u} - 1.5 \left(\frac{d}{t} - 2.7 \right) \quad (\text{G.19d})$$

where:

$$\begin{aligned} d &= \text{stud diameter, in.} \\ t &= \text{flange thickness, in.} \end{aligned}$$

A_s = stud cross-sectional area, in.²

F_u = stud fracture strength stress, ksi

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 (G.20 & G.21) to address the issues of inaccuracy of the current models.

$$\text{For a 76 mm deck:} \quad V_c = 0.35I\sqrt{f'_c}A_c \leq Q_u \quad (\text{G.20})$$

$$\text{and for a 38 mm deck:} \quad V_c = 0.61I\sqrt{f'_c}A_c \leq Q_u \quad (\text{G.21})$$

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, mm²

I = 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 (1987) 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.

G.1.2 Steel Anchors in Concrete

Aspects of steel anchorages used in concrete are reviewed here, given the similarity to standoff screws embedded in concrete and subject to combined shear and tensile forces.

Muratli et al. (2001) and Shirvani et al. (2001) conducted two separate studies at the University of Texas evaluating various models to calculate the concrete cone breakout strength of shear and tensile steel anchors in concrete. These studies are analogous to the problem of standoff screws embedded in a concrete slab and are therefore reviewed. Muratli et al. (2001) evaluated shear anchors, in which three possible predictive models were considered: Concrete Cone (CC) Method, the 45-Degree Cone Method, and a variation on the CC Method obtained by regression analysis. Shirvani et al. (2001) evaluated anchors in tension, in which four predictive models were evaluated: CC Method, 45-Degree Cone Method, variation on the CC Method, and a “Theoretical Model” related to the CC Method.

With respect to design of steel anchors in concrete, the ACI has traditionally focused on recommendations pertaining to nuclear energy related structures. These recommendations can be found in ACI 349 documents. Up until recently, the ACI 318 Building Code has been silent on the issue (Breen et al. 2001). ACI 349 has traditionally specified the 45° Cone Approach as a tool to be used in calculating the strength of steel anchorage in concrete. This has now been replaced with the new Concrete Capacity Design (CCD) Method, as specified by ACI 318-02 (ACI 2002).

Shear breakout capacity based on the 45-Degree Cone Method is defined by Eq. G.22. The failure is assumed to occur on the surface of a half-cone with inclination of 45 degrees (Fig. G.12) and at a stress of $4\sqrt{f_c}$ (CEB 1991, ACI 1997).

$$V_{no} = 2p c_1^2 \sqrt{f_c} \quad (G.22)$$

where:

- V_{no} = single anchor nominal shear breakout capacity, lbs
- c_1 = edge distance in loading direction, in.

f_c = concrete compressive strength, psi

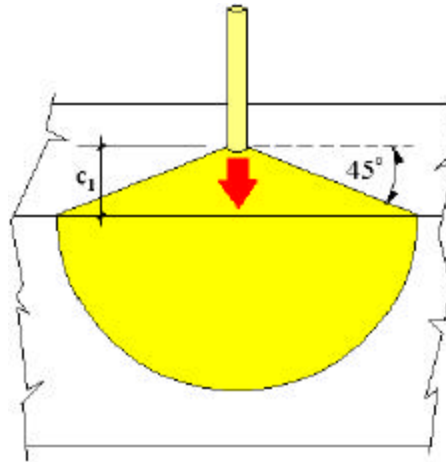


Fig. G.12 Breakout Body in 45-Degree Cone Method (CEB 1991, Muratli et al. 2001)

In cases where the depth of the concrete member is smaller than the edge distance, anchor spacing is less than $2c_1$, or member width is less than $2c_1$, the following modification to the shear breakout capacity equation is required:

$$V_n = (A_v / A_{v0}) V_{no} \quad (G.23)$$

where:

V_n = nominal shear breakout capacity, lbs

A_v = actual projected area of semi-cone on side of concrete member, in.²

$$= \left(p - \frac{pq}{180} + \sin q \right) \frac{c_1^2}{2}$$

q = $2 \cos^{-1} \left(\frac{h}{c_1} \right)$

A_{v0} = projected area of one fastener in thick member without influence of spacing, in.²

Similarly, tensile breakout capacity is reached at the stress of $4\sqrt{f_c}$ (CEB 1991, ACI 1997). The tensile breakout capacity is defined by Eq. G.24. The failure plane is assumed to have a 45-degree conical shape, as shown in Fig. G. 13.

$$T_o = 4 \mathbf{p} h_{ef}^2 \sqrt{f_c} \left(1 + \frac{d_h}{h_{ef}} \right) \quad (G.24)$$

where:

- T_o = nominal tensile breakout capacity of a single cone, lbs
- h_{ef} = effective embedment, in.
- f_c = concrete compressive strength, psi
- d_h = diameter of anchor head, in.

If the anchor is near an edge of concrete surface (i.e., $c_1 < h_{ef}$) or near an adjacent cone, the tensile capacity should be adjusted as follows:

$$T_n = \frac{A_N}{A_{N0}} T_o \quad (G.25)$$

where:

- T_n = nominal tensile capacity of a cone or a group of cones, in.²
- A_N = actual projected area of failure cone or cones, in.²
- A_{N0} = projected area of a single cone, in.²
- = $\mathbf{p} h_{ef}^2 \left(1 + \frac{d_h}{h_{ef}} \right)$

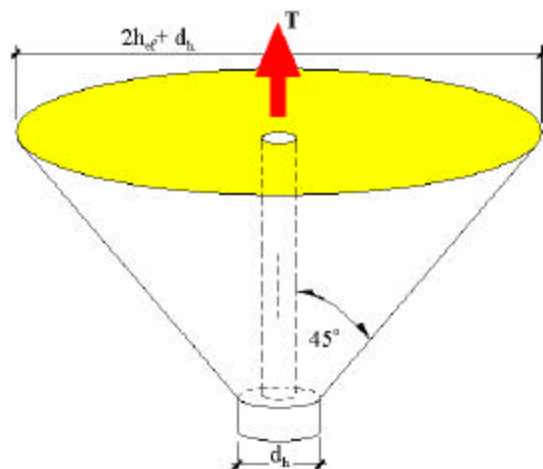


Fig. G.13 Tensile Breakout Cone in 45-Degree Cone Method (CEB 1991, Shirvani et al. 2001)

Code committees considering this aspect were looking for a new approach that would be more compatible with the LRFD format of the present code, more accurate, and better accommodating of brittle as well as ductile failure modes (Breen et al. 2001). In response to these needs, ACI 318 adopted an approach based on the CC Method. Shear breakout capacity using the CC Method (CEB 1991, Muratli et al. 2001) is given in Eq. G.26. Observed and simplified planes of failure are shown in Fig. G.14.

$$V_{no} = 13 (d_o f_c)^{0.5} \left(\frac{l}{d_o} \right)^{0.2} c_1^{1.5} \quad (G.26)$$

where:

- V_{no} = shear breakout capacity, lbs
- d_o = outside diameter of anchor, in.
- l = activated load-bearing length of anchors $\leq 8d_o$
- f_c = specified compressive strength of concrete, psi
- c_1 = edge distance in the direction of load, in.

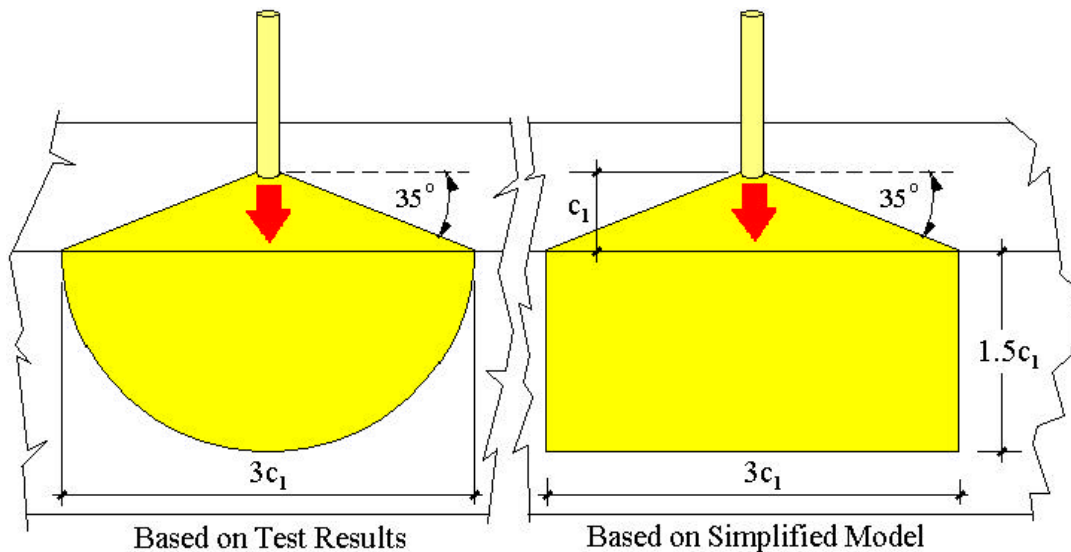


Fig. G.14 Idealized Breakout Model for a Single Shear Anchor by CC Method (CEB 1991, Muratli et al. 2001)

Equation G.26 is valid for as long as member thickness is not less than $1.4h_{ef}$. In case of smaller members, or where adjacent anchors are present, the shape of the failure cone

should be reduced as shown in Figure G.14. In cases where reduction is needed, Eq. G.27 applies.

$$V_n = \frac{A_v}{A_{vo}} \gamma_4 \gamma_5 \gamma_6 V_{no} \quad (G.27)$$

where:

- A_v = actual projected area at the side of concrete member, in.²
- A_{vo} = projected area of one fastener in thick member without influence of spacing and member width, in.²
- γ_4 = modification factor for shear strength to account for fastener groups that are loaded eccentrically
- γ_5 = modification factor to consider the disturbance of symmetric stress distribution caused by a corner
 - = 1, if $c_2 \geq 1.5c_1$
 - = $0.7 + 0.3c_2/c_1$, if $c_2 \leq 1.5c_1$
- c_1 = edge distance in loading direction, in.
 - = maximum of: $c_{2,max}/1.5$, $h/1.5$, if anchors are in a thin and narrow member with $c_{2,max} < 1.5c_1$ and $h < 1.5c_1$; else, actual dimension
- c_2 = edge distance perpendicular to loading direction, in.
- h = thickness of concrete member, in.
- γ_6 = modification factor for shear strength to account for absence or control of cracking

The tensile strength of a single anchor connection using the CC Method is determined using Eq. G.28. The breakout plane is idealized as a pyramid with its planes being inclined at 35° with respect to its base (Fig. G.15). In situations in where the failure cone is near an edge, the capacity is calculate by Eq. G.29.

$$T_o = k \sqrt{f'_c} h_{ef}^{1.5} \quad (G.28)$$

where:

- T_o = tensile breakout capacity, kips
- k = constant for anchor type; ranges from 13.48 to 35;

- f'_c = specified concrete compressive strength, psi
 h_{ef} = effective embedment depth, in.

$$T_n = \frac{A_N}{A_{N0}} \psi_2 T_o \quad (G.29)$$

where:

- A_{N0} = projected area of a single anchor at concrete surface without influence of edges, in.²
 A_N = actual projected area at the concrete surface, in.²
 ψ_2 = factor to account for disturbance of the circularly symmetric stress distribution caused by an edge,
= 1, if $c_1 \geq 1.5h_{ef}$
= $0.7 + 0.3 \frac{c_1}{1.5h_{ef}}$, if $c_1 \leq 1.5h_{ef}$;
 c_1 = edge distance to the nearest edge, in.

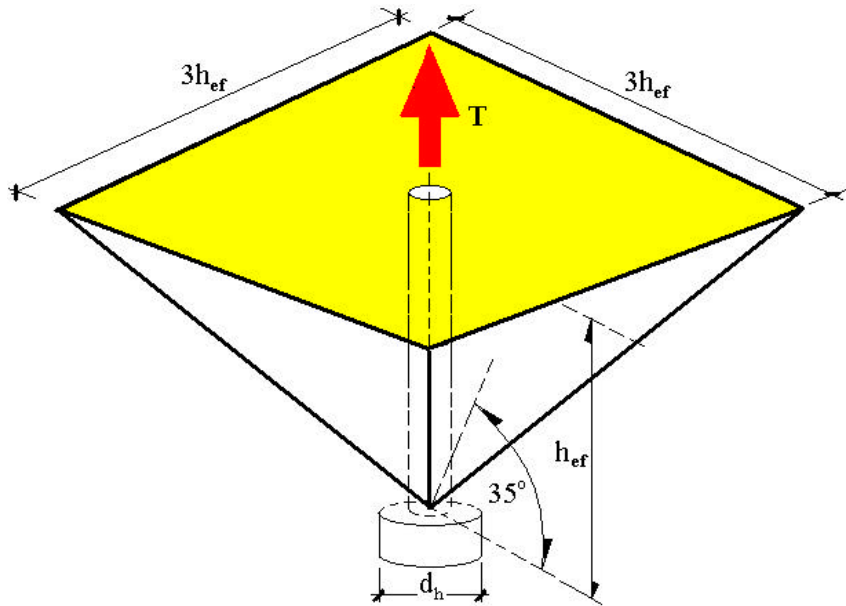


Fig. G.15 Tensile Concrete Breakout Body by CC Method (CEB 1991, Shirvani et al. 2001)

Based on inaccuracies they observed in the CC Method, Muratli et al. (2001) used regression analyses to derive an alternative method for concrete shear breakout strength (Eq. G.30). This alternative method is essentially a statistically improved CC Method. Variables

shown in Eq. G.30 are identical to those shown in Eq. G.26. In statistical analysis they performed, Muratli et al. found CC Method to be more reliable than 45-Degree Cone Method. They also confirmed that the type of anchor used can have significant impact on overall connection strength (Muratly et al. 2001).

$$V_{no} = 2.7l^{0.1} d_o^{0.3} f_{cc}^{0.5} c_1^{1.4} \quad (G.30)$$

Similarly, Shirvani et al. (2001) evaluated the Theoretical Model (Eq. G.31) used to predict tensile breakout capacity. This method is based on linear elastic fracture mechanics. Shirvani et al. also studied the so called the Variation on the CC Method, which is obtained by statistical manipulation of coefficients in existing CC Method model for prediction of cone tensile strength. They concluded that the CC and Theoretical Methods have a lower probability of failure. Also, they found the CC Method to have somewhat lower probability of failure than Theoretical Method. Finally, they determined that the variation on the CC Method resulted in a higher systematic error and probability of failure (Shirvani et al. 2001).

$$N_n = \frac{k\sqrt{f_{cc}}h_{ef}^2}{\left(1 + \frac{h_{ef}}{50}\right)^{0.5}} \quad (G.31)$$

where:

- N_n = predicted concrete tensile breakout capacity, kN
- f_{cc} = actual tested strength of 200-mm concrete cube, MPa
- h_{ef} = effective embedment, mm
- k = 2.75 for undercut and cast-in-place anchors, 2.5 for expansion and sleeve anchors

Appendix D of ACI 318-02 incorporates the CC Method in its fairly thorough procedure for prediction of strength of steel anchors in concrete. ACI stipulates higher strength reduction factors for ductile, and lower for brittle failures. It incorporates several limit states for tensile failures (concrete breakout strength of anchor in tension, pullout strength of anchor in tension, and concrete side-face blowout strength of a headed anchor in

tension). For failures in shear, the following limit states are applicable: steel strength of anchor in shear, concrete breakout strength of anchor in shear, and concrete pry-out strength of anchor in shear. The combined loading state is defined by ACI as shown in Eq. G.32.

$$\frac{N_u}{\phi N_n} + \frac{V_u}{\phi V_n} \leq 1.2 \quad (\text{G.32})$$

where:

- N_u = factored tensile load, lbs
- ϕ = strength reduction factor per D4.4 and D4.5
- N_n = nominal tensile strength, lbs
- V_u = factored shear load, lbs
- V_n = shear tensile strength, lbs

Other studies on steel anchorage in concrete were performed by Farrow and Klingner (1996), Michler and Curbach (2001), Rodriguez (1995), Wollmershauser et al. (2001), and Zhang (1997). Additional available strength computation methods and research project results by various authors are reviewed in ACI SP-103 publication *Anchorage in Concrete*. Studies on numerical modeling of steel anchors in concrete are reviewed in section G.2.2.

G.2 Finite Element Analysis of Composite Steel-Concrete Structures

Modeling of composite structures using finite element analysis is rather complex. This is due to the fact that as many as five interrelated materials may be involved. Further, behavior of concrete in its post-elastic state and its behavior in the state of tri-axial stress are neither fully resolved issues and still a subject of significant investigation in the research community. This section will review relevant previous studies of finite element modeling of composite steel-concrete structures.

G.2.1 Finite Element Analysis of Composite Steel-Concrete Beams

Nonlinear analysis, and finite element analysis in particular, is commonly used in studying various problems related to composite steel-concrete beams. One of the most prominent problems in this field, around which there is much controversy in the research

community is the definition of concrete behavior under the state of triaxial stress. The problem is further magnified by uncertainties in concrete inelastic and post-cracking behavior. Nonlinear finite element methods are seldom used in day-to-day design of composite steel-concrete structures. However, one example of where this was done is the design of 72-story InterFirst Plaza building in Dallas, TX (Viest et al. 1997). Most of applications, however, are limited to research. Literature describing finite element analysis of composite steel-concrete beams is reviewed herein.

Baskar et al. (2000) performed a nonlinear finite element (FE) analysis of composite plate girders subject to combined shear and negative bending. To do this, they utilized general purpose FE software ABAQUS. In the 3D ABAQUS model used, web, flanges and all stiffeners were idealized as 8-node thin shell elements. The concrete slab was modeled with 20-node quadratic brick elements. Reinforcing steel bars were modeled with the general beam element with circular section. Also, the concrete slab was modeled such that a small gap was left in between the top girder flange and the slab so that the shear stud parameter can be controlled. Studs were also modeled with general beam elements with circular sections.

Baskar et al. (2000) modeled steel as an elastic-perfectly plastic material in both compression and tension. Concrete was assumed to be an isotropic material. It was modeled as elastic-plastic material, as ABAQUS did not support modeling of large cracks in concrete under tension at the time of this study. To describe the material behavior for purpose of modeling, the authors used the model proposed by Carreira and Chu (1985). This model is shown as Eq. G.33. This model, often called The Cauchy Elastic Model, is the simplest way of introducing non-linearity into strain-stress relationship for concrete (Chen et al. 1991).

$$\frac{\mathbf{s}}{f'_c} = \frac{\mathbf{b}(\mathbf{e} / \mathbf{e}'_c)}{\mathbf{b} - 1 + (\mathbf{e} / \mathbf{e}'_c)^{\mathbf{b}}} \quad (\text{G.33})$$

where:

- s = $f(e)$, stress in concrete at a given strain value, psi
- f'_c = ultimate concrete compressive strength stress, psi
- e'_c = the strain corresponding to f'_c

$$\beta = \frac{1}{1 - \frac{f'_c}{e'_c E_0}}$$

Baskar et al. (2000) conducted analysis on both steel plate girders and composite steel-concrete plate girders. They then compared the load vs. deflection plots from the analysis to those obtained from experiments. They observed that the nonlinear finite element analysis fairly accurately predicted the ultimate load and behavior of steel-concrete composite plate girders.

Haufe et al. (1999) conducted a study with a goal of generating an efficient model that adequately represents material non-linear behavior of composite steel-concrete structures. The authors assumed small strains and applied rate-independent elastoplasticity to each of the three materials used (steel, concrete and interface layer). For the steel beams, Von Mises elastoplasticity with linear kinematic and isotropic work-hardening is applied. As for the constitutive model for concrete, it involves a combination of several widely accepted models for defining the behavior plain concrete, reinforcement and tension stiffening. The authors modeled concrete using the expanded Feenstra Model. This is essentially a hybrid model in which concrete behavioral curve is described using series of Drucker-Prager functions. For the reinforcement, perfect bond was assumed, and tension-stiffening was taken as extra stress in the direction of rebar length. An interface layer was inserted so bond behavior and geometric discontinuities can be modeled. The total stress (s) in a cracked state is defined as sum of the plain concrete stress (s_c), reinforcement stress (s_s), and stress due to tension stiffening (s_{ia}).

Haufe et al. (1999) performed a parametric study on a particular composite beam discretizing steel and concrete into two-dimensional 9-node elements and interface into 6-node elements. The results of analysis in shape of a load vs. slip curve for the beam were compared to the results of experimental program. They found good agreement between the two. They also noted that this approach lends itself as easily extendable for use in more sophisticated applications, i.e. concrete under tri-axial compression.

Wendel and McConnel (2000) developed and presented a computer program for advanced nonlinear finite element (FE) analysis of steel-concrete composite structures. The

program uses a layering technique, which allows consideration of stress redistribution through the member due to progressive cracking in the slab and yielding in the steel. Concrete is represented as a nonlinear elastic isotropic material before cracking and nonlinear material orthotropic thereafter. The basis for modeling the concrete behavior in the program is the nonlinear elastic isotropic model proposed by Cedolin et al. (1997). The model was tailored to suit 2D applications. Steel is considered to be elastic prior to yielding with strain hardening considered afterwards. Modeling of either full or partial shear connection is allowed by inserting a narrow stub element at the steel-concrete interface. Physical properties of the shear connection are defined by empirical load vs. slip curves (Wendel and McConnel 2000).

The authors reported great accuracy in the program’s ability to predict the failure behavior of general composite steel-concrete structures. Further work by the authors has been under way to extend the program’s ability to model and predict the effect of slip between the profiled steel sheeting and concrete slab (Wendel and McConnel 2000).

Salari et al. (1998) conducted a study with the goal of establishing a nonlinear analytical model, capable of considering the effect of deformable shear connectors on the overall performance of composite floors. The authors utilized a “distributed spring” at the interface of concrete slab and steel element to model connector deformation at the interface. They presented the basic governing equations for a composite beam with deformable shear connectors under small displacements. Based on those equations and using the force method of analysis, Salari et al. developed a new composite beam element, as illustrated in Fig. I.16. This model was then compared to a previously developed displacement-based composite beam element that is illustrated in Fig. G.17. This comparison showed the superior performance of the force-based element over the displacement-based element.

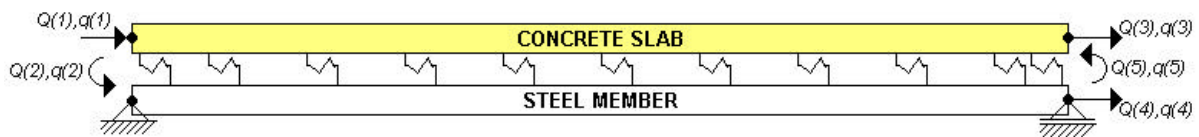


Fig. G.16 Model of a Force-Based Composite Beam Element (Salari et al. 1998)

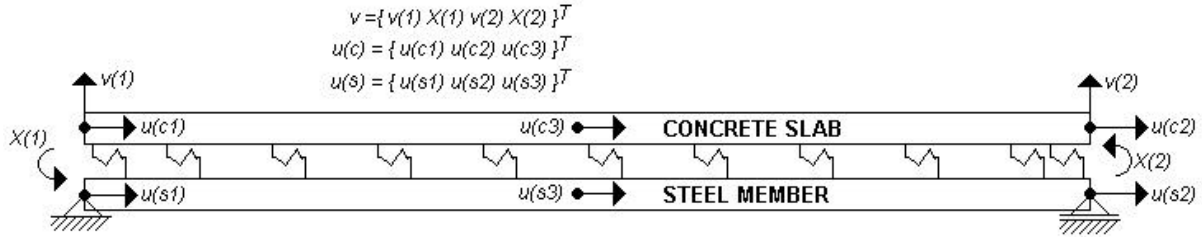


Fig. G.17 Model of a Displacement-Based Composite Beam Element (Salari et al. 1998)

Salari and Spacone (2001) extended the abovementioned study to frame elements. As in the study with beam elements, the authors were able to demonstrate the superiority of force-based elements to displacement-based elements. They also conducted full-scale experimental evaluations of composite frames and found a high degree of agreement between the results of the experimental and analytical programs, thus further substantiating the analytical method developed by Salari et al. (1998).

Finite-element modeling of composite steel-concrete components is generally done with the purpose of modeling the global behavior of the member. The success of such models is typically evaluated by comparison of experimental load-deflection curves with those obtained through numerical modeling. The studies mentioned above fit this profile. Studies aimed at numerical modeling of localized behavior of shear connection at the layer interface appear less frequently in the literature. One such study was performed by El-Lobody and Lam (2002). They utilized the ABAQUS Standard finite element package to model the behavior of shear connection in flexural composites made out of steel beams and precast hollow core slabs. The authors modeled all materials, including concrete, as elastic-perfectly plastic. They utilized simplified concrete model was given in BS8110. Also, the authors considered that for modeling purposes the concrete material response in tension is the same to that in compression. This simplification was used given the limitations that exist in the ABAQUS concrete material model, as well as limitations pertaining to the modeling of post-peak behavior. The load was applied to the model via the modified RIKS method that is generally used to predict unstable and non-linear structural collapses (El-Lobody and Lam 2002). El-Lobody and Lam performed a parametric study and showed the effect of various parameters on the strength of composite connection. Strength prediction models were not presented based on the results of the parametric study. The authors provided comparison of load-slip response curves from the representative tests with those obtained through numerical

modeling. The high level of agreement between the response curves of various samples indicates the usefulness of the proposed numerical model.

Other studies related to finite element modeling of composite beams and their connections were performed by Amadio et al. (2000), Ayoub and Filippou (2000), Dall'Asta and Zona (2000), El-Tawil and Derlein (2001), Faella et al. (2000), and Huber (2000).

G.2.2 Finite Element Analysis of Steel Anchorage in Concrete

As noted in section G.1.2, given the similarity to standoff screw embedded in concrete and subject to combined shear and tensile forces, steel anchors in concrete are very relevant to this study. When investigating the behavior of steel anchors in concrete, various authors have performed finite element analyses of the connections. The purpose of such analyses was generally to provide a theoretical basis for postulated behavioral patterns and design methods, such as the previously reviewed CC-Method.

Wollmershauser et al. (2001) used finite element analysis while investigating the anchor strength under loads that are not perpendicular to the concrete edge. Load angles were varied between 0 and 180 degrees. The authors do not provide any details of how the material properties were represented in the model, or what software was employed in the analysis. The analysis was utilized in the study to complement results of a limited number of tests and to predict cracking patterns of concrete near the edge.

Ožbolt (2001) explored a smeared crack approach in the finite element analysis of reinforced concrete structures. Ožbolt provided a number of application examples. One of these was a model of pull-out of a headed stud. The author utilized MASA, which is a finite element code intended for use in the nonlinear smeared fracture analysis. The concrete was represented by a microplane model, in which it is defined by a stress-strain relationship on planes (microplanes) of various orientations that are determined in advance (Ožbolt 2001). The author obtained very good agreement between crack patterns from the analysis and those from the tests. Such cracks were also consistent with the angle of failure cone prescribed by the CC-Method. The author notes that the method can accurately predict the behavior of rather complex structures. However, he also warns that the method yields results that are very sensitive to mesh refinement and an insufficiently refined mesh can lead to unrealistic results.

Asmus and Ožbolt (2001) used finite element techniques to investigate the splitting failure mode of anchorage connections. This type of failure is typical for narrow concrete members, and those in which anchors are too close to the edge. Asmus and Ožbolt investigated the influence of a variety of geometrical parameters of the concrete member (member thickness, member width, and edge distance) and anchor (embedment depth and drilled hole diameter) on the connection strength. Their model featured 3D solid finite elements and typical concrete mechanical properties. The analysis employed a micro-crack approach that facilitates an advanced mesh refinement. Asmus and Ožbolt found a good agreement between the finite element models and the tests.

Boussa et al. (2001) conducted a study in which they performed a general parametric study of steel anchors in concrete loaded in shear. Their motivation was the idea that most typical specification provisions for this kind of connection are based on empirical evidence, and even though the results obtained are generally on the safe side, problems could arise from incorrect prediction of how various variables affect strength. The study considered both material and geometry based non-linearity, although the concrete was modeled as a perfectly elastic material in regions far away from the anchor. For modeling of concrete, the authors used a smeared crack approach that accounts for progressive degradation of stiffness due to appearance of cracks. The model was meshed using 3D hexahedral elements. The exception was concrete in regions far away from the anchor, which was meshed with tetrahedral elements. The shear force was simulated by incremental application of horizontal displacement. Comparison of load-displacement curves from the tests and the modeling indicate a low level of agreement between the two, with experimental response yielding more conservative results. The authors believe that such modeling could serve as a basis for future code developments and replacement for time-consuming, and often costly testing.

Other studies in this area were conducted by Bruckner et al. (2001), Cervenka (2000), Hofmann et al. (2001), Li et al. (1999), and Nienstedt et al. (1999).

G.3 Ductility of Composite Floor Shear Connection

Shear connectors in composite steel-concrete beams can in some cases exhibit low ductility and suddenly fail before the composite section reaches its full plastic strength.

Some of the literature dealing with this problem, its causes, consequences and recommendations is reviewed herein.

Ductility of shear connectors has no universal definition or criteria. Johnson (1994) defines shear connector ductility, or *slip capacity*, as “*the maximum slip at which the a connector can resist 90% of its characteristic shear resistance, as defined by the falling branch of a load-slip curve obtained in a standard push test.*” He states that practically all shear connectors have some ductility, and that it is therefore possible to use uniform spacing of connectors along the beam span. Further, slip facilitates redistribution of longitudinal shear between connectors before any of them fail. In beams with a low degree of partial connection and in longer beams, the slip required for redistribution increases. This means that in some short spans the connectors may be ductile (have required slip capacity), while in some long beams the very same connector may be non-ductile. Also, the author states that somewhat higher ductility was found in single shear connectors placed in the middle of a concrete rib, noting the significance of effect of slab geometry, placement and number of connectors per rib on their ductility (Johnson 1994).

Figure G.18 illustrates the definition of ductility of headed shear studs per Eurocode 4. In Fig. G.18, N denotes the number of studs in a beam, while N_f denotes the number of studs in a beam required to develop the full plastic strength of beam under normal loading. The figure illustrates that for full composite connection, the design is the same regardless of whether the connector is ductile or not. However, for partial connection design, the ductility of connectors should be taken into account. With the exception of standard 19-mm headed studs, which have a ductility of 6 mm, most types of shear connectors are classified as non-ductile in Eurocode 4 (CEN 1992b). The reason for this is that until recently most push-out type tests were not carried past the maximum load, which is required to determine ductility, as defined above (Johnson 1994). It should be noted that Eurocode 4 sets 6 mm as the required ductility displacement requirement (Johnson et al. 1993).

Figure G.19 further illustrates the design philosophy with respect to connector ductility per Eurocode 4. If the shear connectors used are ductile, one can either use the less conservative plastic method (path AHC), also known as equilibrium method, or the interpolation method (path AKC). Either way, deformation capacity of shear connectors is exploited. Both methods are specified in clause 6.2.1.2 of Eurocode 4. If non-ductile

connectors are used, design is performed per clause 6.2.1.3. The path of composite action (PUQC) for non-ductile connectors is shown in Fig. G.19. There, P is the point of non-composite moment strength. Q is a point determined through elastic analysis of the section, while C is obtained by plastic analysis of the section. The Eurocode 4 design philosophy with respect to non-ductile connectors is based on the fact that larger slip at the steel-concrete interface increases curvature, but decreases longitudinal shear. Consequently, if connectors are non-ductile, slips must be kept small. If they are kept small, they can often be disregarded in analysis. In this respect, Eurocode 4 clause 6.2.1.3(1) refers to stress distribution based on “*full continuity*.” If non-ductile connectors are uniformly spaced, clauses 6.1.2 and 6.1.3 of Eurocode 4 effectively set the minimum limit of ratio of the number of studs in the beam to the number of studs required to develop full composite strength (N/N_f) to 0.4 (Johnson et al. 1993).

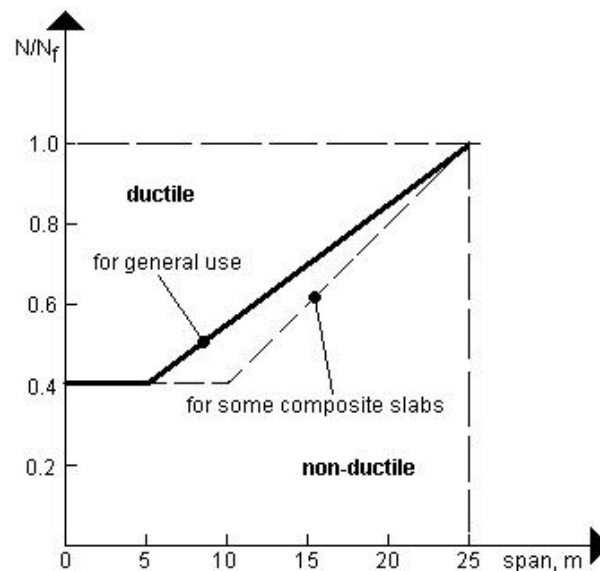


Fig. G.18 Definition of “Ductile” for Welded Studs and Other Connectors (Johnson 1994)

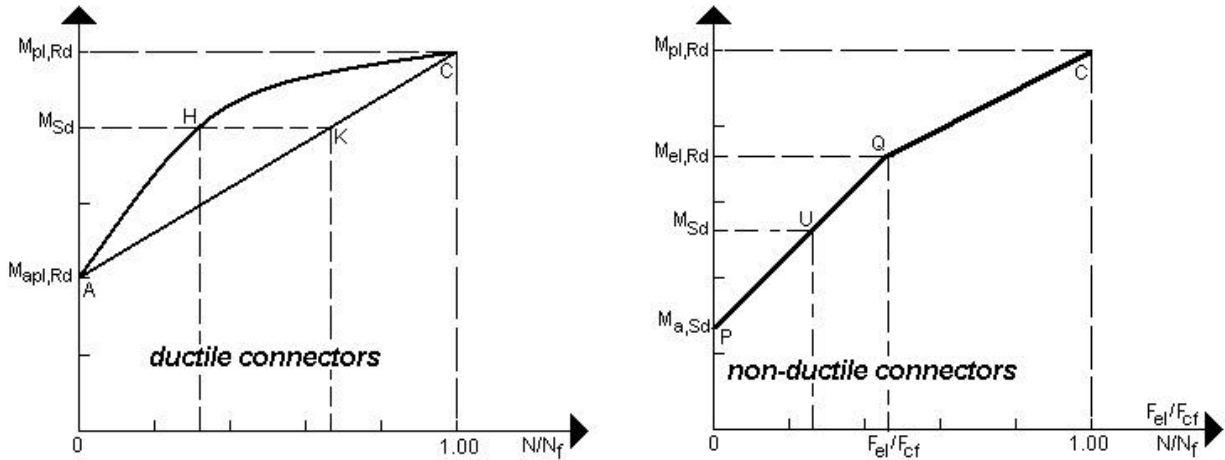


Fig. G.19 Design Methods for Partial Shear Connection (Johnson et al. 1993)

Mullett (1998) describes the attention that is given to the issue of shear connector ductility in BS5950. Like other authors, he points out that the horizontal shear force is unequally distributed to shear connectors in the elastic range. As shear connectors exceed their elastic range and undergo inelastic deformations, the horizontal shear force distribution among connectors becomes more even. BS5950 sets the minimum stud elongation at 15%, and requires the stud design strength to be taken as 80% of their characteristic strength (Mullett 1998). The two requirements ideally result in a fully redistributed force among ductile connectors and flexural failure of the beam, rather than failure of the connection. Similar to Eurocode 4, BS5950 sets the minimum of 0.4 on degree of shear connection, K. It also accounts for the effect of member length on shear connector ductility, placing further limitations on K as shown in Eq. G.34. In BS5950, ductility is also accounted for in strength analysis, as illustrated in Fig. G.20. Similar to Eurocode 4, it specifies an exact procedure (AHC) and an interpolative procedure (AC) for ductile connectors, as well a procedure for rigid connectors in which the shear force in connectors is linearly distributed among the connectors. In Fig. G.20, M_s denotes the moment strength of bare steel member, while M_{pc} denotes the full plastic moment strength, and $M_{K=0.4}$ is the strength at minimum percent composite (Mullett 1998).

$$K \geq \frac{L-6}{10} \geq 0.4 \quad (G.34)$$

where:

K = degree of shear connection

L = member length, m

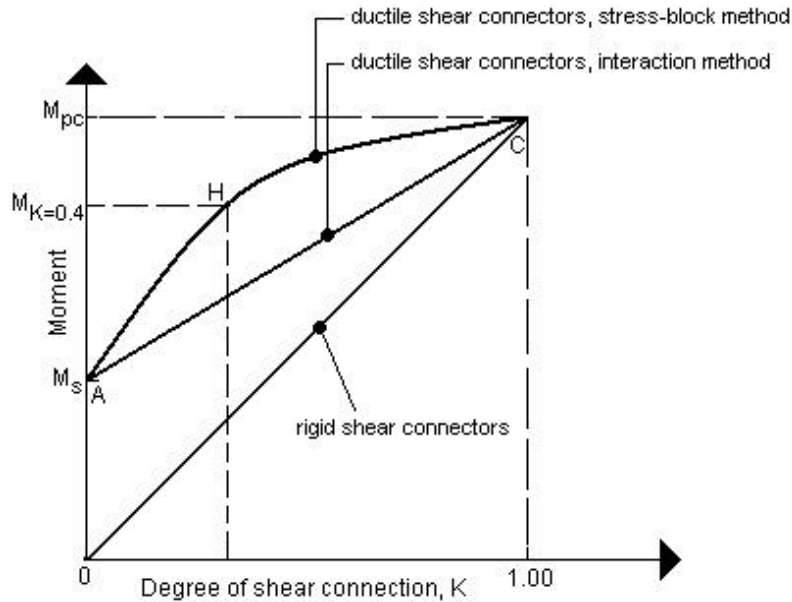


Fig. G.20 Interaction Between Moment and Degree of Shear Connection (Mullett 1998)

Davies (1975) notes that prior to the 1965 publication CP 117: Part 1, the British design practice was to design composite structures for service loads. With that approach, it was simply necessary to design shear connectors to resist the longitudinal shear per unit length based on elastic concept of complete interaction between the connected surfaces (Eq. G.35). From 1965, the British practice was generally to use the load-factor method, effectively invalidating Eq. G.35. Collins points out, however, that the spacing between connectors is of little consequence since, as he notes, the flexibility of connectors is such that they allow for redistribution of the longitudinal shear such that all connectors between the support and the maximum moment point become equally loaded.

$$Q = \frac{FAy}{I_{comp}} \quad (G.35)$$

where:

Q = longitudinal shear, kips/in.

F = vertical shear force, kips

A = transformed area of concrete above the interface, in.²

y = distance between center of area A and neutral axis, in.

I_{comp} = moment of inertia of transformed composite section, in.⁴

Aside from Eurocode 4 and BS5950, most specifications for composite steel-concrete beams set the minimum composite action to anywhere from 25 to 50%. This is because some ductility in the member is desirable after it reaches its design capacity (Viest et al. 1997). The AISC Specification (AISC 1999) does not specify minimum composite action, but the design aids in the AISC Manual of Steel Construction (AISC 2002) incorporate the lower limit of 25%.

Oehlers and Bradford (1999) point out that the response of shear studs when loaded in shear is not unlike that of a typical steel stress-strain curve, except that typical steel fractures at values of strain about 30 times higher than slip at which studs fracture. Hence, the possibility of the studs fracturing due to excessive slip is of a major concern (Oehlers and Bradford 1999). Figure G.21 shows typical relationships between moment strength and degree of composite action. Path DCBA is based on rigid plastic analysis. Oehlers and Bradford contend that this path is truly attainable only if shear connectors had unlimited slip capacity (ductility). Because this can never be the case, it is essential to insure that connectors do not fracture due to excessive slip prior to reaching the design moment strength. Oehlers and Bradford further say that path GH, as shown in Fig. G.19, is a representation of fracture of shear connectors at ultimate slip, S_{ult} (defined later in this review). If the beam has more shear connectors than required for full shear connection (range 1- N_1), slip will be in its elastic region, and beam strength (M_{fsc}) will be governed by the smaller of the plastic strength of steel or concrete. If a partial shear connection is provided (N_2), the moment strength (M_{psc} , M_2) will be governed by the strength of shear connection (P_{sh}), and corresponding slip will be on its plastic plateau, but less than S_{ult} . Finally, if the degree of shear connection is very low (N_3), the slip will correspond to S_{ult} , $M_{frac} = M_3$, which is less than the rigid plastic strength, M_4 (Oehlers and Bradford 1999).

To predict whether or not the premature failure due to shear connector fracture will happen or not, Oehlers and Bradford compare the slip capacity of shear connector (Eq. G.36) to the calculated maximum slip in the composite beam. If the former is less than the latter, shear connector fracture can happen, in which case geometrical and mechanical property adjustments to the composite beam can be made to avoid this type of failure. To compute the maximum slip in beam, Oehlers and Bradford recommend either *Parametric Study Approach* model by Johnson and Molenstra 1991, which is given by Eq. G.37 and is based on defining

the degree of interaction at the transition point C (Fig. G.21), or *Mixed Analysis Approach* model (Eq. G.38), based on defining the path GH in Fig. G.21. The former model is appropriate to be used where the degree of shear connection is 50 or more percent, while the latter is applicable to lower levels of shear connection (Oehlers and Bradford 1999).

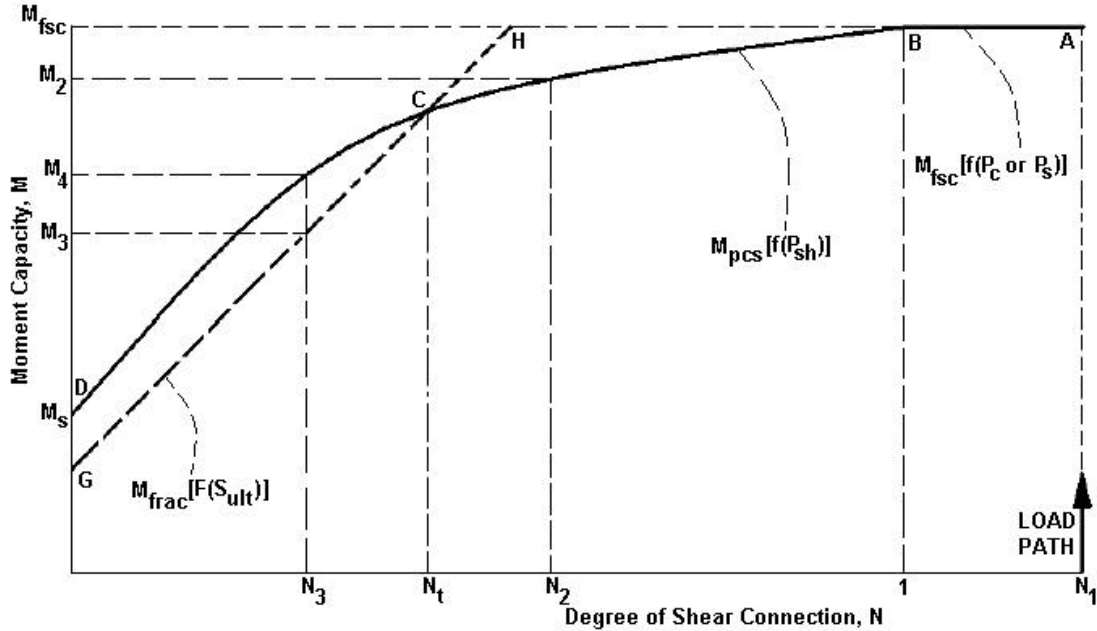


Fig. G.21 Fracture of Shear Connectors in Composite Beams (Oehlers and Bradford 1999)

With some geometric limitations, 5% characteristic slip capacity is as shown in Eq. G.36. If 0.42 is replaced with 0.48, one can compute the mean slip capacity.

$$S_{ult} = (0.42 - 0.0042f_c)d_{sh} \quad (G.36)$$

where:

S_{ult} = slip capacity of shear connector, mm

f_c = compressive cylinder strength of concrete, N/mm²

d_{sh} = diameter of the shank of shear connector, in.

$$S_{ult} = \left(\frac{M_s L h_s}{6 E_s I_s} \right) \left(\frac{L}{D} \right)^a \left(\frac{M_{psc} - M_s}{M_s} \right) \quad (G.37)$$

where:

for $\beta_{max} = 0.5$, $a = -0.13$, and $\beta = 1.03$

for $\eta_{\max} = 0.75$, $a = -0.24$, and $\beta = 1.70$

S_{ult} = maximum slip in the beam, mm

M_s = rigid plastic moment capacity of steel, kNm

M_{psc} = partial shear connection moment capacity of composite beam, kNm

h_s = height of steel component, mm

D = depth of composite beams, mm

E_s = modulus of elasticity of the steel component, kN/mm²

I_s = steel component moment of inertia, mm⁴

L = beam span length, m

$$S_{\max} = \frac{M_{\max} L}{3} K_1 - \frac{P_{sh} L}{4} K_2 \quad (\text{G.38})$$

where:

$$K_1 = \frac{h_{\text{cent}}}{E_c I_c + E_s I_s}$$

$$K_2 = \frac{h_{\text{cent}}^2}{E_c I_c + E_s I_s} + \frac{1}{E_c A_c} + \frac{1}{E_s A_s}$$

M_{\max} = moment due to applied load, kNm

h_{cent} = distance between concrete and steel component centroids, mm

E_c = concrete modulus of elasticity, kN/mm²

A_s = steel component cross-sectional area, mm²

I_c = concrete component moment of inertia, mm⁴

Mujagic et al. (2001a) observed that some composite joist tests featuring standoff screws exhibit a linear distribution of slip and longitudinal shear force among shear connectors at the failure load. Such joists exhibited brittle fracture of screws, low ductility, and sudden failure. The authors stated the need for further research aimed at deriving a design procedure that would take into account the shear connector ductility, and enable a user to design composite joists with standoff screws for ductile failure.

Other works on the subject are published by Aribert et al. (1999), Bode and Däuwel (1999), Dai et al. (1965), Dall'Asta et al. (2000), and Yam (1966).

G.4 Probability Based Design: General Concept 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, falls below the magnitude of load only within a predetermined acceptable margin, which has been adjusted for the possibility of overload. This criteria is summarized in Eq. G.39, which was presented by Ravindra and Galambos (1978).

$$\Phi R_n \geq \sum_{k=1}^j \gamma_k Q_{km} \quad (G.39)$$

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 others, 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 this model proposed by Cornell, based on both normal and lognormal distribution, are presented in Eq. G.40.

$$p_f = P[(R-Q) < 0] \quad \text{or} \quad p_{f,\ln} = P[\ln(R/Q) < 0] \quad (\text{G.40})$$

where:

$p_{f,\ln}, p_f$ = probability of failure of a structural element

R = resistance

Q = load effect

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

From Eq. G.40, it is apparent that failure is imminent if the load effects exceed resistance. However, the probability of failure will theoretically never be zero.

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

Galambos and Ravindra (1973) introduced an expression (Eq. G.41) which defines the uncertainty in resistance. Further, they simplified the expression to what is shown here as Eq. G.42. As variations of material, fabrication, and theory are generally less than 0.2, they noted that this simplification would result in an error of less than 5% in the calculated value of the coefficient of variation (C.O.V.) for resistance, V_R .

$$V_R = \sqrt{V_M^2 + V_F^2 + V_P^2 + V_M^2 V_F^2 + V_M^2 V_P^2 + V_F^2 V_P^2 + V_M^2 V_F^2 V_P^2} \quad (\text{G.41})$$

$$V_R = \sqrt{V_M^2 + V_F^2 + V_P^2} \quad (\text{G.42})$$

where:

V_R = C.O.V for resistance

V_M = C.O.V. for material

V_F = C.O.V. for fabrication

V_P = C.O.V. for design theory

In absence of more precise data, Galambos and Ravindra assumed that V_F is relatively small, typically up to 5%. With respect to V_P , they found that it may be represented by normal, lognormal, or Weibull distributions. The V_P is also known as professional coefficient of variation, 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 V_P 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.” On a probability density chart it represents a distance between the magnitude of safety margin and the dividing line between failure and survival. This idea is communicated through Fig. G.22, and Eq. G.43.

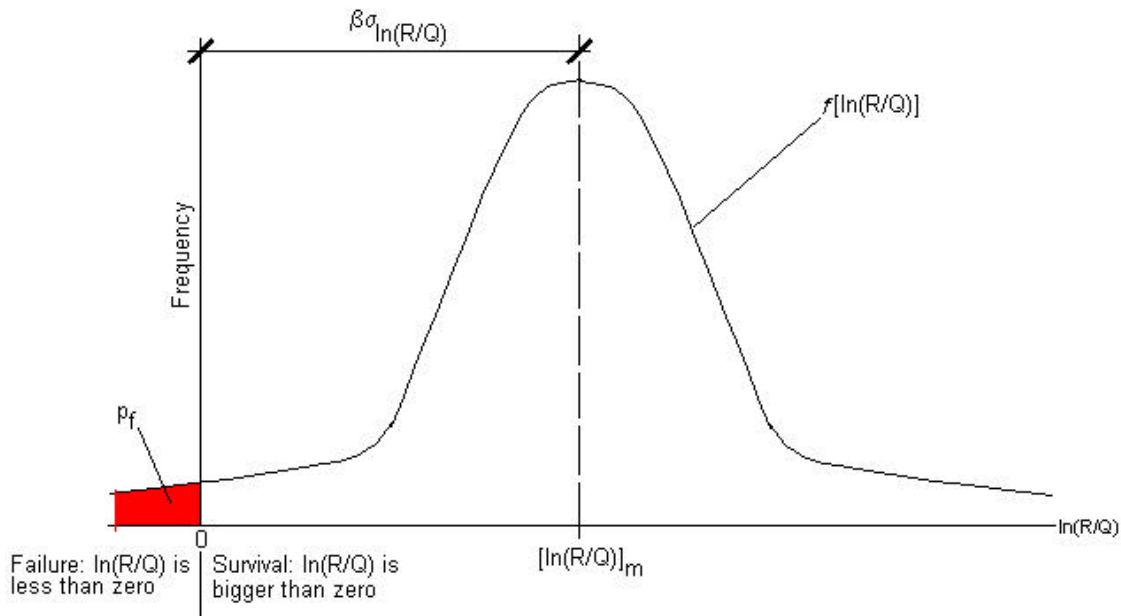


Fig. G.22 Representation of Safety Index, β , per Galambos and Ravindra (1973)

$$b = \frac{[\ln(R/Q)]_m}{S_{\ln(R/Q)}} \quad (G.43)$$

where:

β = safety index or reliability

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

$[\ln(R/Q)]_m$ = mean value 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 a new specifications (i.e. LRFD) is approximately

the same as that in an 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 the 1978 AISC specifications at the live to dead load ratio of about 3 (Gaylord 1992).

Next, Galambos and Ravindra, adopting the findings of Ang and Cornell (1972), defined the central safety factor, shown as Eq. G.44. This quantity, as they indicate, considers the variability, or uncertainties, of both the resistance and the load effects.

$$\theta = \exp [\beta(V_R^2+V_Q^2)^{1/2}] \quad (G.44)$$

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 this 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 Eq. G.44.

$$\Phi = \exp(-\alpha \beta V_R)(R_m/R_n) \quad (G.44)$$

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 indices suggested for connectors. Fisher et al. (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 indices that may be appropriate for connections. As a result of this study, a reliability index of 1.5 was suggested for all friction-type connections, while 4.5 was the magnitude of the reliability index stipulated for all other types of connections.

When developing the limit states based design procedure for composite flexural members, Galambos and Ravindra (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 variability Galambos and Ravindra considered while investigating the effect of shear connector on the strength factor was that of concrete. As for flexural members, Galambos and Ravindra utilized various written sources to obtain the statistical information for steel shapes, concrete, and reinforcement. Experimental results utilized in their study mostly came from the tests performed at Lehigh University. As mentioned earlier, a model by Rambo-Rodenberry et al. (2002) entails changing the factor of 0.85 to 0.88.

The philosophy behind European limit states design practice is not unlike that in the U.S.; however, the formulation of the design process is somewhat different. Relevant information regarding this is generally contained in the Eurocodes (Johnson 1994). Loads are grouped into three main groups: permanent, G (i.e. dead load of structure), variable Q (i.e. live, wind or snow load), and accidental A (i.e. impact loads). Design situations, also affecting the design, are grouped into three categories: persistent (normal use), transient (duration of construction), and accidental (fire or earthquake). Another categorization affecting the design is related to whether an action is fixed (typically permanent actions), or free (occurring only in a part of the structure or a member). In principle, loads G are assigned their partial factor of safety γ_G . These are typically bigger than 1.0 and are specified by Eurocode 1 (CEN 1992a). Variable loads Q are, in addition to γ_G , also assigned an appropriate combination factor ψ . These are also specified by Eurocode 1, and are typically less than 1.0 (Johnson et al. 1993).

Design resistances, are obtained by dividing the corresponding material property by an appropriate partial safety factor, γ_M , for resistance. The material property is defined as *5% lower fracture strength*. Where statistical information about a material property is not available, then nominal strength of the material can be used. Naturally, the goal of a design procedure is to insure that the design actions are met by corresponding design resistances (Johnson 1994).

With respect to composite beams, there is a major distinction between the Eurocode and AISC Specification. Namely, unlike AISC, Eurocode 4 requires appropriate partial safety factors to be applied to both load on shear connectors, as well as its resistance. With respect to statistical characteristics, beam flexural strength is computed separate from the stud, with its own partial factors of safety for both load and resistance (Johnson 1993).

Zeitoun (1984) conducted a study with the goal of deriving strength reduction factors for shear studs, with the strength being determined using Eq. G.16. Based on this equation and experimental work of the original authors, Zeitoun found two strength reduction factors:

- $F_s = 0.76$ when the left side of Eq. G.16 governs;
- $F_t = 0.94$ when the right side of Eq. G.16 governs.

The above given coefficients were computed assuming a reliability index of 3.0. Zeitoun makes no recommendation as to how these values would be implemented in design, and whether or not the overall strength reduction beam would need to be adjusted.

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.

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