Shear Strength of Stud Connectors in Lightweight and Normal-Weight Concrete

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STEEL-CONCRETE composite construction using normal-weight concrete has been used since early in the 1920's. Substantial use of composite construction began mainly for bridge structures in the 1950's as a result of the work done by Viest.[16–18] Its primary growth in building construction during the last decade was a result of the simplified design provisions introduced into the 1961 AISC Specification. The development of these provisions were based on studies reported by Slutter and Driscoll.[5,11]

The type of shear connectors has changed substantially during the past 20 years. Bridge construction made extensive use of spiral connectors in the early 50's. These were replaced by the flexible channel and stud connectors. Today, headed studs are used extensively for both bridge and building construction. The first studies on stud shear connectors were undertaken by Viest, who tested full scale pushout specimens with various sizes and spacings of the studs.[16] Later studies on bent and headed studs were initiated at Lehigh University by Thurlimann.[15] A series of beam and pushout tests were reported by Slutter and Driscoll, who developed a functional relationship between the shear connector strength and the concrete compressive strength.[5,11] The mathematical model was comparable to the useful capacity proposed earlier by Viest.[17]

Since 1961, several investigations of composite beams using lightweight concretes have been made. Studies at the University of Colorado[3,14] and at Lehigh University[6,12,13] evaluated the strength of stud connectors in a number of different types of lightweight aggregate concretes using pushout specimens. Investigators at University of Missouri[1,2,4] examined various sizes of stud shear connectors, the effect of haunches, and the behavior of beams. These studies showed that the strength of a shear connector embedded in lightweight concrete was 5 to 40% lower than the strength of connectors embedded in normal-weight concrete. Considerable variation was apparent in the pushout data because of variation in specimen geometry, slab reinforcement, and experimental techniques. Also, the tensile strength of the stud connectors varied (from 62 to 82 ksi) and in many instances was unknown. Because of these variations and the limited data, it was not possible to provide rational design recommendations.

The purpose of this investigation was to determine the strength and behavior of connectors embedded in both normal-weight and lightweight concretes so that design recommendations could be made. A series of pushout specimens were constructed and tested to assist with the evaluation. The tests with normal-weight concrete provided directly comparable data under the same controlled conditions. The ultimate loads found from tests of pushout specimens provide a lower bound to the strength of connectors in beams.[5]

A companion study on the behavior of composite beams with lightweight concrete slabs was undertaken at the University of Missouri.[8]

TEST SPECIMENS, PROGRAM, AND PROCEDURES

The test program was developed after the controlled variables were selected. The variables considered included the basic material characteristics as determined by standard control tests (i.e., concrete compressive strength f'_c, split tensile strength f'_{sp}, modulus of elasticity E_c, and density w), the stud diameter, type of aggregate, and number of connectors per slab. The stud connector tensile strength, slab reinforcement, and geometry were considered in the experiment design as one-level factors.

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Table 1. Pushout Results and Average Concrete Properties

Aggregate	Individual Specimen Average Connector Ultimate Load, kips			Average Concrete Properties			
	Spec. No. 1	Spec. No. 2	Spec. No. 3	Compressive Strength f'_c(ksi)	Tensile Strength f'_{sp}(ksi)	Density w(pcf)	Concrete Modulus E_c(ksi)
A	29.3	32.5	30.6	5.08	0.51	148.1	3740
LA*	24.5	26.5	24.7	3.64	0.43	147.6	3510
SA**	19.5	20.8	19.9	4.01	0.43	147.4	3580
B	27.4	25.4	25.4	4.78	0.47	140.5	3180
LB*	18.3	18.1	17.3	2.67	0.32	138.6	2190
SB**	18.2	16.9	18.8	4.03	0.46	142.6	3170
2B†	26.1	25.5	25.0	4.78	0.47	140.5	3180
C-‡	19.9	21.3	21.0	4.69	0.24	89.1	1510
C	21.6	21.5	22.2	4.28	0.35	108.2	2060
D-‡	24.1	23.0	22.7	4.72	0.32	99.2	2430
D	21.6	23.3	24.4	4.92	0.36	113.4	2530
E-‡	19.6	19.2	17.8	3.60	0.30	97.7	1840
E	23.1	22.5	21.6	4.30	0.37	111.1	2190
LE*	18.7	19.5	19.7	3.22	0.32	111.4	1880
SE**	15.7	15.7	17.0	4.00	0.33	112.3	2060
2E†	21.2	23.1	22.7	4.40	0.39	111.1	2210

* L indicates series with lower compressive strength.
** S indicates series with $\frac{5}{8}$-in. connectors; all other tests on $\frac{3}{4}$-in. connectors.
† 2 indicates series with 2 connectors per slab.
‡ Specimens with lightweight aggregate and fines.

Description of Specimens—Most of the specimens had four connectors embedded in each slab, as illustrated in Fig. 1. However, several specimens with a single row of two studs, located at mid-height of the slabs, were also tested. All specimens had the same slab reinforcement.

The specimens were cast with the beam vertical and in an inverted position, to assure that voids would not form under the studs on their bearing side. A common form was fabricated so that three specimens could be cast simultaneously.

Test Program—Forty-eight pushout specimens were tested during this investigation. The program consisted of groups of two slab specimens with three specimens in each group (see Table 1), to provide replication and permit the variability to be evaluated.

The normal-weight concrete was manufactured from two types of coarse aggregate. Type A was a crushed limestone and Type B was a natural river gravel.

Three different types of lightweight aggregates were used (Types C, D, and E). Each type of lightweight aggregate was combined with either lightweight fine aggregate or with natural sand. A description of the lightweight coarse aggregate is given in Table 2.

The experiment design considered the stud diameter, number of stud connectors per slab, type of concrete, and the concrete properties. The stud tensile strength and type specimen were considered as one-level factors. This permitted the direct evaluation of the various types of aggregates and concrete properties on the connector shear strength.

Table 2. Description of Coarse Lightweight Aggregates

Material	Expanded Shale (C)	Expanded Shale (D)	Expanded Slate (E)
Color	Brown	Gray to Black	Gray to Black
Max. Size	$\frac{1}{2}$-in.	$\frac{3}{4}$-in.	$\frac{3}{4}$-in.
Shape	Rounded	Cubical to irregular	Cubical to irregular
Production Meth.	Rotary kiln	Rotary kiln	Rotary kiln
Loose Unit Wt. (Approx.)	35 pcf	47 pcf	45 pcf

Control Tests—The characteristics of the concrete slab in which the connectors were embedded were determined by control tests. Standard 6 in. x 12 in. control cylinders were cast along with the pushout specimens to assist in determining the characteristics of the concrete slabs. Sixteen cylinders were cast for each group of specimens. The cylinders were moist cured for 5 to 7 days, along with the pushout specimens. They were then stripped and air cured until the day of testing, along with the pushout specimens.

The modulus of elasticity was obtained during the compression test of the cylinders. An averaging compressometer with a 6-in. gage length was mounted on the cylinder. The dial gage was read at each 10 kip load increment. The modulus of elasticity was calculated from the difference in readings at 10 and 50 kips. Often the modulus of elasticity is taken as the tangent modulus at zero load. Obviously, this would result in slightly higher values than the secant modulus deter-

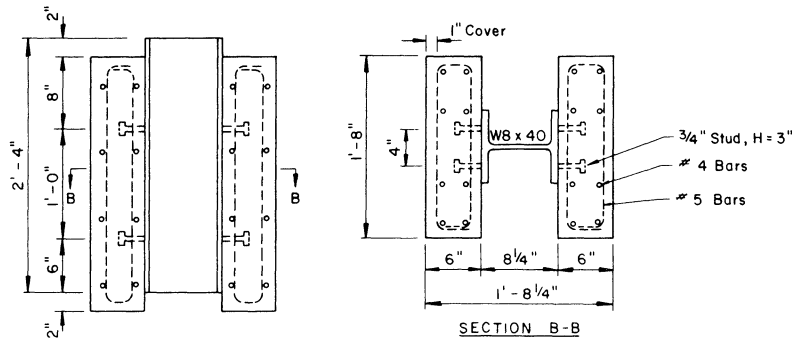


Fig. 1. Details of pushout specimen.

mined from the deformations at 10 and 50 kips. The concrete tensile strength was obtained from split cylinder tests, and the density of the concrete was determined from the weight and volume of the cylinders.

All stud shear connectors were provided from the same lot. The physical properties of the connectors were determined from standard tension tests. The average ultimate strength was 70.9 ksi for the $\frac{3}{4}$-in. studs and 70.2 ksi for the $\frac{5}{8}$-in. studs.

Pushout Tests—The pushout specimens were tested in a 300-kip capacity hydraulic testing machine. The specimens were placed on sheets of 0.5-in. homosote in order to obtain a uniform load distribution on the bearing surface of the slabs.

Testing was usually conducted on the 28th day after casting. Loads were in 10-kip increments, maintained constant at each load level while the vertical slips between the slab and beam were measured.

One specimen from each group was loaded to ultimate load without unloading. The remaining two pushout specimens were loaded to approximately the working load level for the connectors, then unloaded, and reloaded to their ultimate load.

TEST RESULTS

The average properties of the cylinders that correspond to the pushout specimen are listed in Table 1. This includes the concrete compressive strength, f'_c, the split tensile strength, f'_{sp}, the modulus of elasticity, E_c, and the concrete density, w.

All lightweight concrete mixes, except C, satisfied the requirements of ASTM C330. The C-mix was composed of lightweight coarse and fine aggregates and did not yield a satisfactory level of split tensile strength as proportioned and used. ASTM C330 requires an average split tensile strength of 290 psi for structural lightweight concrete. The C-concrete provided a strength of 244 psi.

Typical load-slip curves for a normal weight and a lightweight concrete specimen with two slabs are shown in Fig. 2a. Both types of concrete exhibited substantial inelastic deformation before failure. At ultimate load, there was no sudden failure evident. After further deformation accompanied by a decrease in load, failure was evidenced by a shearing off of the stud connectors or by failure in the concrete slab.

The average load-slip curves for a group of three specimens are compared in Fig. 2b for normal-weight and lightweight concrete pushout tests. It is apparent that the average curves are nearly the same for each specimen group. Two specimens from each group were unloaded after reaching an average load of 10 kips per connector. Subsequent reloading did not change the shape of the overall load-slip relationship (Fig. 2b).

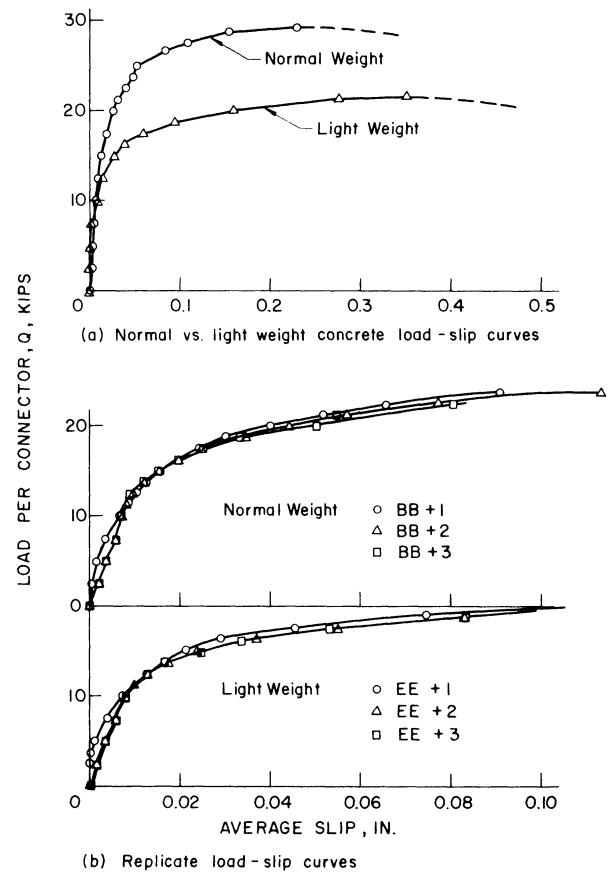


Fig. 2. Typical Load-Slip curves.

The ultimate load per shear connector for each pushout specimen is listed in Table 1. The ultimate loads did not vary much between the replicate specimens of a test group. Very seldom did the standard deviation exceed 1 kip. It is apparent that the connector strengths were decreased significantly (from 15 to 25%) when the connectors were embedded in lightweight concrete. The sanded lightweight concretes provided slightly higher shear strengths than did the all lightweight concrete mixes.

In this study the tensile strengths of all the $\frac{5}{8}$-in. and $\frac{3}{4}$-in. connectors were the same (approximately 70.7 ksi). Hence, the results of the tests on different diameter connectors provided direct information on the influence of connector diameter. Stud connectors of both sizes were embedded in the two normal-weight concretes and one lightweight concrete. The results show that the connector shear strength is nearly proportional to the cross-sectional area of the stud.

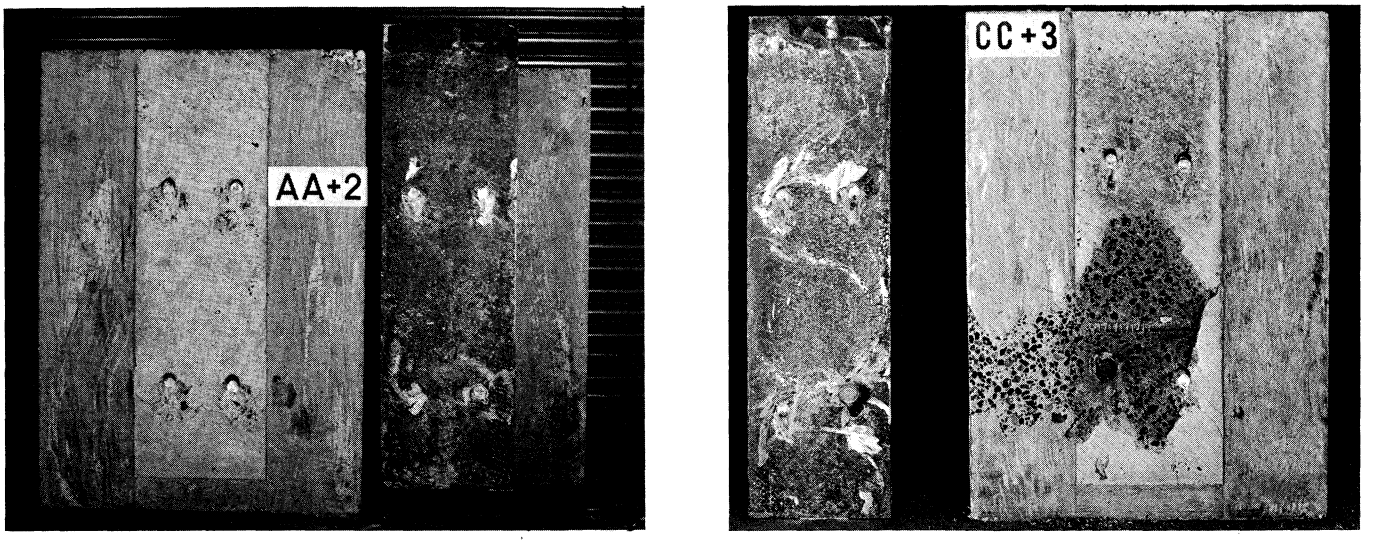
Failure Modes—Most specimens were subjected to additional loading and deformation after the ultimate load was reached. Often, slab cracks were visible just after ultimate load was reached. The loading was normally continued until one or both slabs separated from the steel beam. This occurred at large slips. There were basically two separation modes observed. In one, the studs were sheared off the steel beam and remained embedded in the slab after unloading occurred. In the other, the concrete failed in the region of the shear connectors. In many tests both types of failures were observed in the same specimen.

Specimen A2, which had normal-weight concrete slabs, exhibited the typical stud shear failure. Figure 3a shows the four studs that were embedded in one slab which sheared off. The other slab was still connected to the steel beam. The photograph also indicates that the studs did not shear off at the same slip levels since the gaps between the studs and the slab are not the same size indicating that different amounts of plastic deformation occurred.

A typical specimen which exhibited concrete failure is shown in Fig. 3b. The connectors were pulled out of the slab together with a wedge of concrete. Both normal-weight and lightweight concrete slabs had wedges of similar shape pulled out of the slab. The cracks in the slabs were more numerous and larger in lightweight concrete than in the normal-weight concrete specimens.

The pushout specimens with only one pair of connectors in each slab all failed by shearing off the studs. One reason for this observation could be that the distance from the studs to the end of the slab was greater and the slab force smaller. Also, since the reinforcement in the slab was identical to that used in the other specimens, more reinforcement would be available per connector. However, the ultimate shear strength per connector did not increase for this type of specimen.

The observed mode of failure after slab separation was not applicable to the ultimate load. In order to evaluate the failure mode and determine the state of deformation and type of failure, two specimens were sawed longitudinally through the slab and connectors. One specimen had a normal-weight concrete slab and the second had a lightweight concrete slab. Loading was discontinued just after the ultimate loads were reached in these two specimens and unloading started to occur.



(*a*) *Studs sheared off.* (*b*) *Studs and concrete failure.*

Fig. 3. Typical failure views after slab separation.

The slabs were cut using a diamond disk saw. The cuts were placed so one side of the disk saw would match the center line of the studs. To avoid cutting through the entire length of the steel beam flange, the flange was burned off so that only two small plates remained. The cross section of the sawed test specimens are shown in Fig. 4.

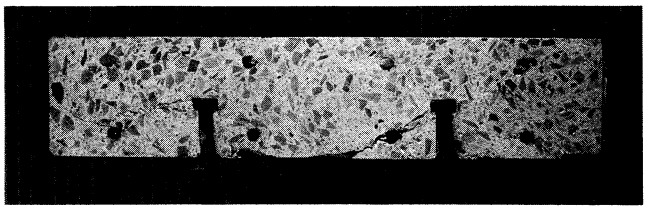
The crack pattern in the concrete slabs is very similar for both specimens. The cracks near the head of the studs are different for the upper and lower connectors. At the upper studs, the crack is nearly vertical to the free end. The crack at the lower stud propagated toward the surface of the steel beam at about a 45° angle. This could result in a lower ultimate strength for the upper pair of studs. The specimens containing only one row of two connectors appeared to have crack patterns similar to the lower pair of studs, because the distance to the free end was greater. Since the ultimate loads per connector were the same for one or two pairs of connectors, the connector shear strength for both the upper and lower studs was about the same.

The deformed shape of the studs was different in the normal-weight and lightweight concrete specimens, as is apparent in Fig. 4. In the normal-weight concrete, greater restraint of the stud is apparent from the curvature (see Fig. 4c). In the lightweight concrete slab the stud was nearly straight (see Fig. 4d). In both slabs the studs were rotated through a large angle at the weld. It is also apparent that the concrete in front of the studs is crushed.

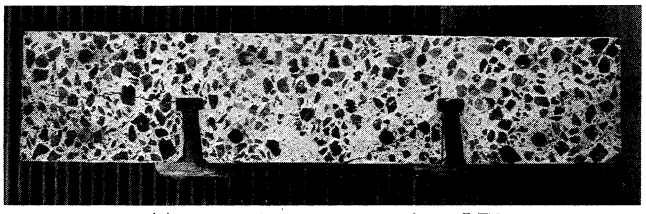
The observed behavior at ultimate load confirmed that the concrete is the controlling medium. For this reason, variation in the tensile strength of the shear connector would not be as critical a parameter as is sometimes believed. It also appears reasonable to assume that smaller diameter connectors would be more dependent on the stud tensile strength, since the concrete forces would not be as great.

ANALYSIS OF RESULTS

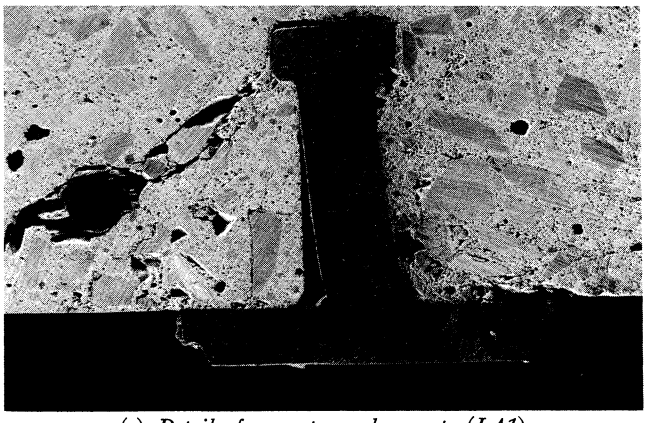
In order to compare the ultimate loads from all the specimens, including different connector sizes, the average shear strength (Q_u/A_s) was used. An examination of the data obtained in this study indicated that the average shear strength was proportional to the cross-sectional area of the studs for specimens having comparable concrete properties; for example, series LA vs. SA, series B vs. SB and series C vs. SE. This observation was also confirmed by statistical tests which indicated that the mean strengths (Q_u/A_s) of two of the three combinations were not significantly different. Earlier studies also considered the average shear strength.[11] The $\frac{3}{4}$-in. connectors used in this study were all furnished from the same lot and had an average tensile strength of 70.9 ksi. The $\frac{5}{8}$-in. connectors were also furnished from one lot and had about the same tensile strength (70.2 ksi).



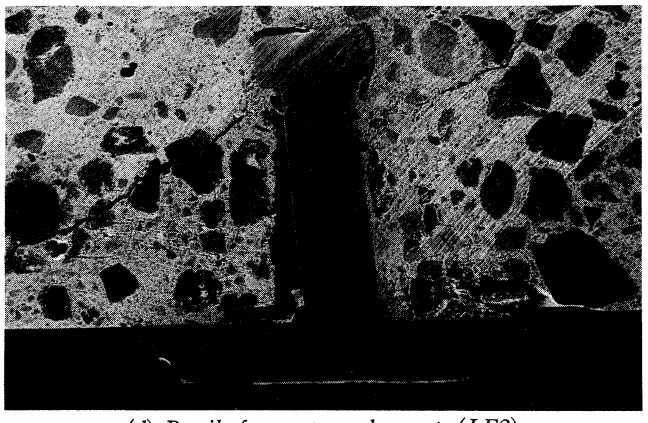
(a) *Normal weight concrete specimen LA1.*



(b) *Lightweight concrete specimen LE2.*



(c) *Detail of connector and concrete (LA1).*



(d) *Detail of connector and concrete (LE2).*

Fig. 4. Sawed sections of lightweight and normal weight slabs and connectors.

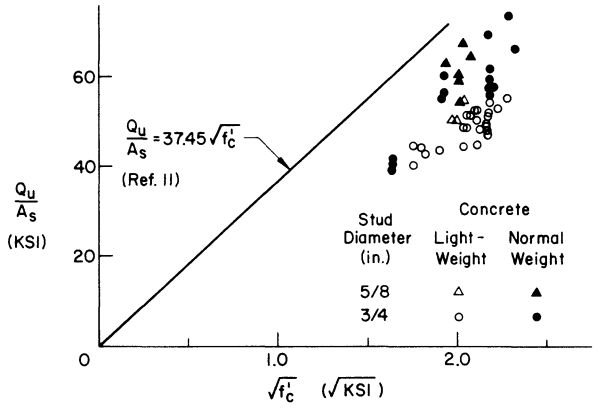


Fig. 5. Connector strength as a function of concrete compressive strength.

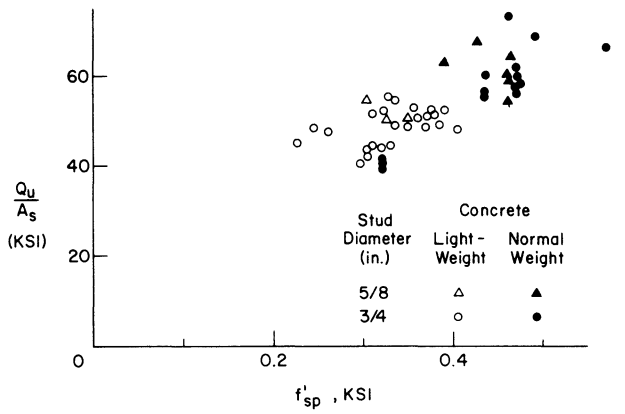


Fig. 6. Connector strength as a function of concrete tensile strength.

Influence of Concrete Properties—Since the material characteristics of the concrete were carefully determined throughout this study, it was desirable to determine whether or not the connector strength and the measured concrete and stud shear connector properties could be correlated. The properties of concrete considered included the compressive strength, the split tensile strength, the modulus of elasticity, and the unit weight.

Earlier studies by Slutter and Driscoll[11] had related the connector shear strength to the compressive strength of normal-weight concrete. The relationship suggested by Viest[17] for useful capacity was modified and used. This resulted in the relationship

$$Q_u = 37.45 A_s \sqrt{f'_c} \text{ (kips)} \qquad (1)$$

where A_s is the nominal area of the stud shear connector, in in.^{2}, and f'_c is the compressive strength of the concrete, in ksi.

The results of this study are plotted as a function of the square root of the compressive strength of concrete in Fig. 5 to ascertain whether or not this relationship was applicable to this study. It is visually apparent that the relationship is not in agreement with the results of this study and does not account for the difference between normal-weight and lightweight concrete.

Equation (1) was based on limited data from beams and pushoff tests.[5,11] The expression was only intended to be valid for concrete strengths up to 4 ksi. It was noted that the beam test results yielded higher values, because of friction and redistribution of the connector forces. In addition, the data was taken from several sources and experimental techniques as well as other uncontrolled variables all contributed to the higher values predicted by Eq. (1).

A study of the test data does indicate a decrease in connector strength when the concrete strength decreases substantially. However, no definite trend is apparent for the concrete strengths between 3.5 and 5.0 ksi for either normal-weight or lightweight concrete.

Figure 6 compares the average shear strength of the stud connectors with the split tensile strength of the concrete. No trends are apparent for the lightweight aggregate concretes. The normal-weight concrete specimens do indicate a decrease in connector shear strength with a decrease in split tensile strength. Taken together, all data provide a trend of decreasing shear strength with a decrease in tensile strength. The variability of the test data is large.

Figure 7 compares the connector shear strength as a function of the concrete density. The density was determined from the concrete control cylinders. The weight of concrete varied from 89 to 148 pcf. Although there is no trend within the various types of concrete, the overall tendency, is, again, a decreasing shear connector strength with a decrease in concrete density.

The relationship between the shear strength and the measured concrete modulus of elasticity is summarized in Fig. 8. Good correlation is evident for both tne normal-weight and lightweight concrete data. Since the concrete modulus of elasticity was in reasonable agreement with the value suggested by ACI, the compressive strength and density of concrete could also be used to determine the modulus and provide a comparable relationship.

Connector Shear Strength and Concrete Properties—In order to obtain a mathematical relationship between the ultimate shear strength of a stud connector and the material properties of the concrete, multiple regression analyses (least squares fit) were made. All 48 two-slab pushout specimens were used. The shear strength (Q_u/A_s) was used as the dependent variable, and the measured concrete properties were considered as independent variables.

A general exponential model given by Eq. (2), which considered all concrete properties, was initially selected.

$$Q_u/A_s = e f'^a_c f'^b_{sp} E^c_c w^d \qquad (2)$$

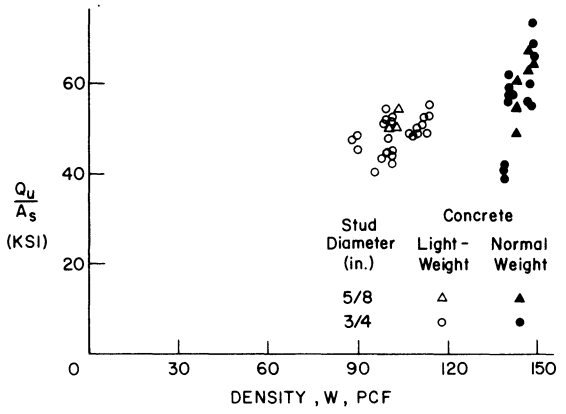


Fig. 7. Connector strength as a function of concrete density.

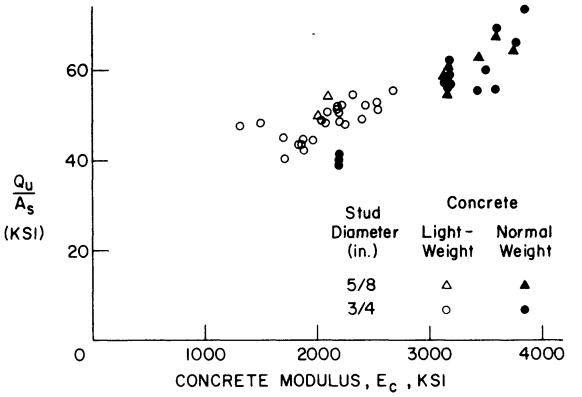


Fig. 8. Connector strength as a function of Modulus of Elasticity of concrete.

In order to obtain linear equations for the regression analysis, the model was linearized by making a logarithmic transformation.

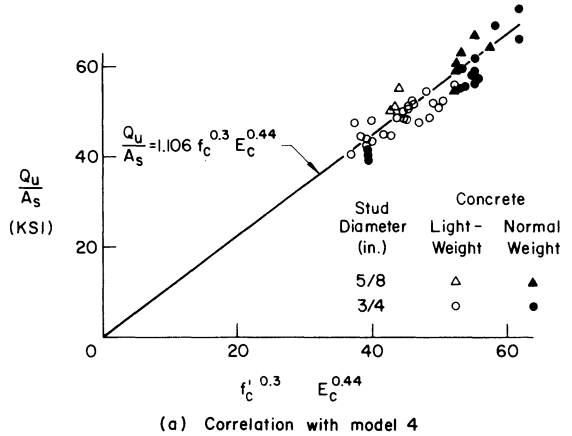
Results from regression analyses, using all possible combinations of the four concrete properties as independent variables, are summarized in Table 3. The results are listed in order of fit. The largest coefficient of correlation was obtained with Model 1, which considered all variables. However, the first four models provided about the same fit. Models 3 and 4, which ignored the split tensile strength, f'_{sp}, provided nearly identical values of the coefficient of correlation. It is also apparent that including the concrete density had a negligible effect of the correlation coefficient, since Model 4 yielded about the same correlation as Model 3.

When only two variables were considered, as with Models 4 and 6, the combination of compressive strength and modulus of elasticity provided a better fit than the combination of compressive strength and density. The test data are compared with Model 4 in Fig. 9a. It is apparent that the compressive strength and modulus of elasticity of concrete provide a reasonable estimate of the ultimate strength of stud shear connectors embedded in both normal- and lightweight concrete.

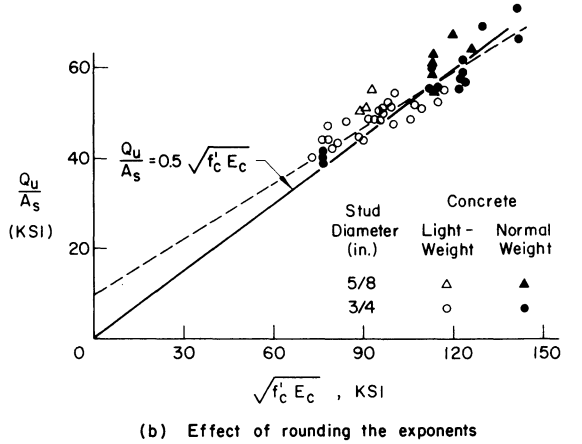
Table 3. Results of Regression Analyses Using Logarithmic Transformations

General model: $Q_u/A_s = ef'^a_c f'^b_{sp} E_c^c w^d$

Obtained Exponents				Coefficient of Correlation	Model Number
a	b	c	d		
0.435	−0.229	0.395	0.306	0.90	1
0.325	−0.148	0.527	—	0.89	2
0.334	—	0.385	0.092	0.89	3
0.304	—	0.439	—	0.89	4
0.640	−0.211	—	0.887	0.87	5
0.542	—	—	0.675	0.86	6
—	—	0.706	−0.413	0.85	7
—	0.019	0.698	−0.418	0.85	8
—	−0.041	0.509	—	0.83	9
—	—	0.484	—	0.83	10
0.301	0.470	—	—	0.75	11
—	0.389	—	0.244	0.70	12
—	0.551	—	—	0.68	13
—	—	—	0.612	0.64	14
0.469	—	—	—	0.50	15



(a) Correlation with model 4



(b) Effect of rounding the exponents

Fig. 9. Comparison of connector strength with concrete strength and Modulus of Elasticity.

Effect of Rounding Off Exponents—Since it is desirable to use more convenient exponents, analyses were made to determine the effect of rounding the exponents obtained for Models 4 and 6. Several sets of exponents were examined for each model. Rounding the exponents decreased the coefficient of correlation by less than 1.7%. Hence, the exponents can be rounded off without significantly affecting the overall fit to the test data.

The test data are compared with the modified Model 4 in Fig. 9b. The dashed line is the least squares fit to the test data when both exponents were rounded to 0.5. The solid line was determined by forcing the model to conform to the origin. It is apparent that the fit to the data is not appreciably affected when the intercept is ignored.

As noted earlier, the modulus of elasticity for the concrete can be determined from the concrete compressive strength and density by use of the ACI formula. Hence Model 6, which includes concrete compressive strength and density, can be transformed into Model 4, which considers compressive strength and the modulus of elasticity of concrete. For design purposes, Eq. (3) provides a reasonable estimate for both Models 4 and 6

$$Q_u = 0.5A_s\sqrt{f'_cE_c} \qquad (3)$$

This relationship provides a good estimate of the ultimate strength of shear connectors embedded in both normal-weight and lightweight concrete slabs. Equation (3) expresses the shear connector strength as a function of the stud connector area and concrete properties. The influence of the type of aggregate is reflected in the modulus of elasticity.

Comparison with Earlier Studies—Test data are available from a number of investigations that were made prior to this study. Driscoll and Slutter[5] observed that the height-to-diameter ratio (H/d) for studs embedded in normal-weight concrete should be equal to or larger than 4 if the full capacity of the connector is to be developed. Specimens which did not satisfy this requirement were not considered.

Only specimens which had one or two connectors per row were considered, since increasing the number of connectors has been shown to influence the shear strength per stud when the slab width and reinforcement are not changed.[16]

A number of haunched specimens were tested at the University of Missouri.[2] Shear strength per connector for this type of specimen was lower than other solid slab specimens. They are not included in the comparison.

Investigators at the University of Sydney[10] examined small scale lightweight concrete specimens with ⅜-in. studs. Most of the concrete slabs were not reinforced. The shear strength (Q_u/A_s) for the specimens without reinforcement were in the range of data from other investigations of specimens with reinforced slabs. When the slabs were reinforced, the ultimate shear strength was substantially higher than for larger studs. These specimens were not considered due to their small scale.

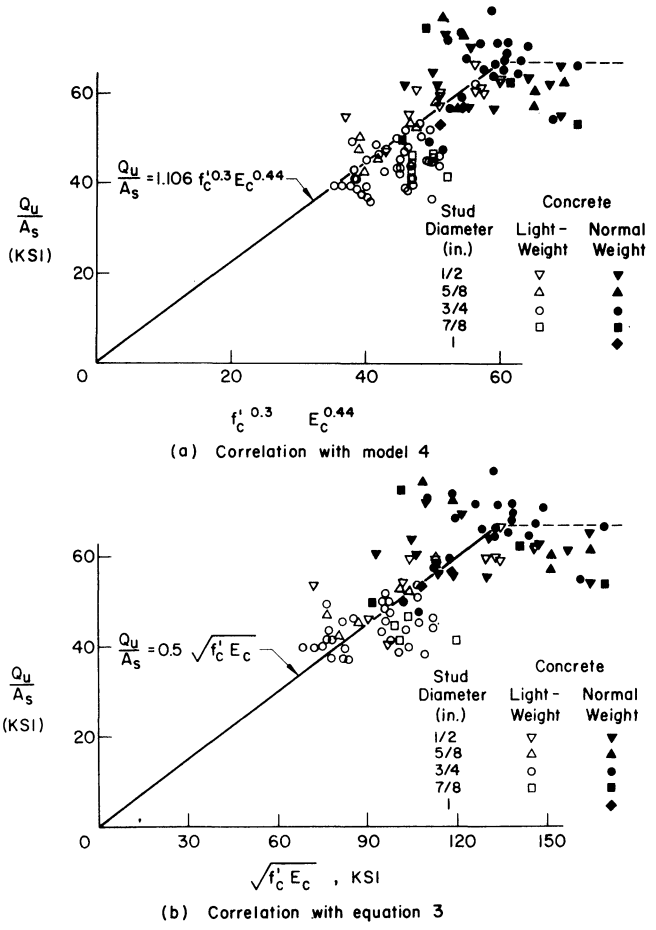


Fig. 10. Comparison of earlier studies with Model 4 and Equation 3.

Other tests were also ignored when the welds were bad or the loading eccentric. The moduli of elasticity was not reported in a number of studies. For such tests, the moduli were estimated from the compressive strength and the density of the concrete using the ACI formula.

The test data from other investigations[2,4,6–9,12–14,16] are compared with Model 4 and Eq. (3) in Fig. 10. It is apparent that both Model 4 and Eq. (3) are in reasonable agreement with the test data, although the scatter is greater for the test results from other investigations. The mean regression line for all test data was not appreciably different from the mean relationship developed from this study. The coefficient of correlation was decreased 18% to 0.72 and the standard error of estimate increased 90% to 8.46 ksi.

An examination of Fig. 10 also suggests that an upper bound to the connector strength is approached when $\sqrt{f'_cE_c} \cong 130$, as the test data tends to plot along a horizontal line. This corresponds to a value of $Q_u/A_s \cong 65$ ksi. This appears reasonable and is probably

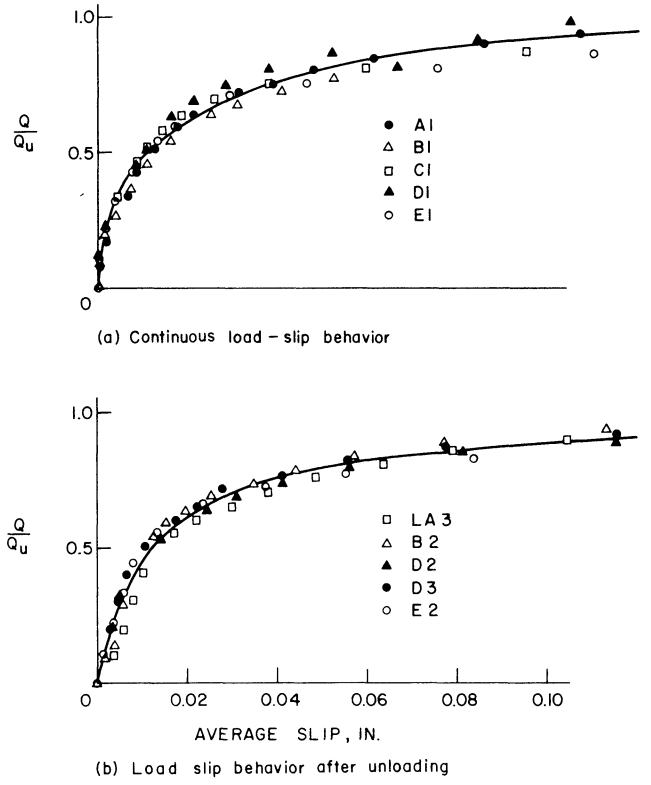


Fig. 11. Load-Slip relationships.

related to the tensile strength of the connector. Many of the specimens in Fig. 10 that exhibited higher shear strengths at lower concrete strengths are those with ½-in. diameter connectors. As noted earlier, the concrete in which the smaller connector is embedded is not likely to control when smaller forces exist. Hence, the ultimate shear strength would be more sensitive to the physical properties of the connector. Often the smaller diameter connectors have a higher tensile strength. The one ⅞-in. stud plotted in Fig. 10 that also produced a high strength was an 8½-in. anchor, which had the highest tensile strength of all studs tested. These two conditions should also permit development of higher apparent shear strength.

Comparison with Current Specifications—In the 1969 AISC Specification, the allowable loads for stud shear connectors embedded in normal weight concrete are based on the model suggested by Slutter and Driscoll,[11] given by Eq. (1). Design loads were obtained from this relationship by dividing by 2.5. The ratio between Eq. (3) and the design loads given in the AISC Specification varied from 1.93 to 2.08 for concrete compressive strengths between 3 and 4 ksi. The concrete modulus of elasticity, E_c, was determined from the ACI formula assuming the density, w, for normal concrete to be 145 pcf. Since the shear strength obtained from pushout specimens is a lower bound to the shear strength of stud connectors in beams, the factor of safety for the connectors in beams is somewhat larger.[5,11]

Structures designed to the AISC Specification have performed satisfactorily and all known beam tests with normal-weight concrete slabs and connectors proportioned according to the AISC provisions have developed their full flexural capacity. Hence, it seems reasonable to use a factor of safety against failure for pushout specimens equal to about 2. Design loads can be obtained from Model 4 or Eq. (3) on that basis, since the predicted strengths for concrete compressive strengths of 4 ksi are all less than the upper bound.

Load-Slip Relationships—The load-slip curves for the specimens within a group were almost identical, as was illustrated in Fig. 2. Unloading of the specimens did not affect the envelope of the curves, and the reloading was reasonably linear until the maximum load prior to unloading was reached.

Curves from various types of concrete were compared by non-dimensionalizing the load by the ultimate strength of the specimen, as illustrated in Fig. 11.

The maximum load was reached at slips varying from 0.23 to 0.42 in. It is apparent that the curves form a narrow band over the entire range of slip. Since the specimens in Fig. 11a were not unloaded, the curves provide an envelope for a continuous load-slip relationship that includes the initial bond condition. Figure 11b provides similar non-dimensionalized curves for specimens which were unloaded. The first loading cycle was not considered and only the reloading portion is shown.

Since all the pushout specimens had similar load-slip curves, an empirical formula for the load-slip relationship of continuously loaded specimens was determined as:

$$Q = Q_u(1 - e^{-18\Delta})^{2/5} \tag{4}$$

This function is compared with the test curves in Fig. 11a. The function has a vertical tangent at zero load. This was also observed for the measured load-slip curves due to the bond acting between the concrete slab and the steel beam. Equation (4) approaches Q_u as the slip increases. For a slip equal to 0.2 in., the function yielded 99% of the ultimate load.

The load-slip relationship for the reloading condition was similar to one suggested by Buttry.[2] The function

$$Q = Q_u \frac{80\Delta}{1 + 80\Delta} \tag{5}$$

was found to provide a reasonable fit to the test data. The load-slip relationship defined by Eq. (5) is dependent on the level of preloading and slip. Equation (5) provides an estimate of the reloading load-slip relationship for preloads of 10 kips per connector. Equation (5) is plotted in Fig. 11b for comparison. The slope at zero is 80 Q_u (kips/in.) and the function approaches Q_u at larger slip values. For slips of 0.2 to 0.4 in. the equation yields 94 to 97% of the ultimate load.

SUMMARY AND CONCLUSIONS

This study summarizes the results of tests on 48 two-slab pushout specimens. The main purpose of the investigation was to evaluate the capacity and behavior of stud shear connectors embedded in lightweight concrete. Two different types of normal-weight aggregates and three types of lightweight aggregate were examined. The lightweight concretes were made with both lightweight coarse aggregate with natural sand and with lightweight fines.

The following conclusions were drawn from this study:

1. The shear strength of stud connectors embedded in normal-weight and lightweight concrete was primarily influenced by the compressive strength and the modulus of elasticity of the concrete. The following empirical function described the test results:

$$Q_u = 1.106 A_s f'_c{}^{0.3} E_c^{0.44}$$

while the following simplified equation is satisfactory for design purposes:

$$Q_u = \tfrac{1}{2} A_s \sqrt{f'_c E_c}$$

where f'_c is the concrete compressive strength (ksi), E_c the modulus of elasticity (ksi), and A_s the cross-sectional area of the stud shear connector (in.^{2}).

2. Other concrete properties including the concrete tensile strength and density did not significantly improve the fit to the test data.

3. Pushout specimens with either one or two rows of studs per slab exhibited the same average strength per stud.

4. The shear strength was approximately proportional to the cross-sectional area of the studs.

5. The load-slip relationship for continuous loading can be expressed as:

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

where Q is the load and Δ is the slip in inches.

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