Concrete breakout capacity of cast-in-place concrete anchors in early-age concrete

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- Precast concrete elements are often stripped at strengths lower than the minimum 2500 psi (17 MPa) required by ACI 318-08 appendix D, raising a concern as to whether the ACI equations are applicable for early-age concrete.
- Seventy-eight pullout tests were conducted on headed stud assemblies in concrete as young as 12 hours.
- The results indicate that the tensile strength of early-age concrete rises faster than the compressive strength, and the pullout strength of the inserts exceeds the capacity predicted by ACI 318-08 and the sixth edition of the *PCI Design Handbook*: *Precast and Prestressed Concrete* for compressive strengths as low as 1000 psi (7 MPa).

Prediction models for the breakout strength design of inserts were developed using mature concrete with a minimum concrete compressive strength of 2500 psi (17 MPa). Inserts used in stripping precast and prestressed concrete and tilt-up construction may see loads applied as early as 18 hours after casting, and architectural elements may have stripping and handling loads applied even earlier. There are anecdotal reports of failure of stripping inserts at these early ages. There are multiple potential reasons for these failures. First, the failure can occur due to the intricacies of the individual lifting device. Second, if the tensile capacity of the concrete develops more slowly than the compressive strength and decisions are based on compressive strength, a premature failure may occur. Third, if the insert is designed for the specified concrete strength rather than the stripping strength, there may be insufficient embedment. Fourth, stripping requirements may not have incorporated suction or dynamic loads due to stripping and handling. Increases in loads from stripping and handling are not considered in this study but must be considered in plant operations.

Research significance

The objective of this research is to determine, theoretically and experimentally, whether the current concrete breakout strength models based on concrete compressive strength and embedment depth for mature concrete can be applied to early-age concrete, specifically, whether at early age the breakout capacity is properly modeled using the compressive strength or whether an early-age correction factor is needed. Breakout failure is influenced by the tensile and compressive strength of the concrete, the variability of the concrete strength at early age, and the possible additional loads imposed in stripping the concrete from the form and dynamic effects during handling.

Background

Early-age concrete tensile versus compressive strength

The correlation between tensile and compressive strength is critical to this project because of the emphasis on early-age performance. While the concrete tensile strength affects breakout capacity of the anchor, compressive strength is most commonly measured and reported. In the literature review, the compressive strength is the variable most commonly used in predicting breakout strength and the concrete tensile strength is often not reported.

Although sometimes it is unclear in the published research whether it is the mature compressive strength or the strength at the age of the concrete testing that determines a concrete's tensile strength, the relationship between the two appears to be that as the compressive strength increases so, too, does the tensile strength, but at a decreasing rate. Many of the equations for comparing tensile and compressive strength are presented and used for comparison with the values obtained in this research.

Neville[1,2] explains the difficulties in developing a direct relationship between concrete compressive and tensile strengths. Neville lists at least six different factors that affect the concrete tensile strength–to–compressive strength ratio f_t/f_c. These factors include the strength, coarse and fine aggregates, age, curing, air entrainment, and density of the concrete. Aggregate can affect strength because crushed coarse aggregate increases flexural strength. The fine aggregate affects the ratio based on the aggregate grading, and possibly because of the difference in surface-to-volume ratio in the specimens used to measure compressive and tensile strength when modulus of rupture is taken as the tensile strength. The effects of age are only discussed beyond an age of one month, at which point the tensile strength increases more slowly than the compressive strength, which is similar to the overall tendency for the ratio of f_t/f_c to decrease as compressive strength increases. Curing affects the f_t/f_c ratio because tensile strength is more sensitive to shrinkage during dry curing in flexure test beams. Air entrainment lowers the compressive strength more than the tensile strength. Lightweight concrete may have high ratios of f_t/f_c at low strength, but at higher strengths the ratio is similar to that of normalweight concrete. Drying may reduce the ratio for lightweight concrete by 20%.

Mindess, Young, and Darwin[3] present the same factors affecting the ratio of concrete compressive strength to tensile strength as Neville. In addition, they explain how different tensile test methods produce different ratios. The ratio of splitting tension to compressive strength is usually in the range of f_{sp}/f_c' equal to 0.08 to 0.14 (where f_{sp} is the splitting tensile strength, and f_c' is the specified concrete compressive strength at 28 days). However, the ratio of direct tensile strength to compressive strength is about 0.07 to 0.11, and the ratio of modulus of rupture to compressive strength is about 0.11 to 0.23.

Equation (1) is used by ACI 318-08[4] as a lower bound.

$$f_{sp} = 6\sqrt{f_c'} \text{ psi } (f_{sp} = 0.48\sqrt{f_c'} \text{ MPa}) \quad (1)$$

The following equation is proposed by ACI Committee 363.[5]

$$f_{sp} = 7.4\sqrt{f_c'} \text{ psi } (f_{sp} = 0.59\sqrt{f_c'} \text{ MPa})$$

The following best fit of the data is proposed by Mindess, Young, and Darwin.

$$f_{sp} = 4.34 f_c'^{0.55} \text{ psi } (f_{sp} = 0.305 f_c'^{0.55} \text{ MPa})$$

This best fit equation is in general agreement except the best fit exponent is larger than the ½ proposed by ACI 318-08.

Oluokun et al.[6,7] state that the ACI 318-08 exponent of ½ is not valid for early-age concrete. Oluokun et al. tested three laboratory-prepared test mixtures and one sample from a precast, prestressed concrete producer. The 28-day compressive strengths ranged from 4000 to 9000 psi (28 to 62 MPa) for the four mixtures. Standard 6 × 12 in. (150 × 300 mm) cylinders were cast from a single batch for each series of testing. The coarse aggregate for all mixtures was 90% to 100% retained on a ¾ in. (19 mm) sieve with 100% less than 1 in. (25 mm). The fine aggregate was a manufactured crushed limestone aggregate. Oluokun et al. concluded that crushed aggregate produced a tensile strength about 25% higher than smooth aggregate. Equation (2) is the recommended formulation for tensile strength.

$$f_t = 29.8 f_c'^{0.79} \text{ psi } (f_t = 0.584 f_c'^{0.79} \text{ MPa}) \quad (2)$$

Khan et al.[8] selected modulus of rupture as the measure of the tensile strength. Three different curing conditions were investigated, including temperature-matched curing, sealed curing, and air-dried curing. The three concretes consisted of a nominal 4300; 10,150; and 14,500 psi (30, 70, and 100 MPa) compressive strength at 28 days. Khan et al. concluded that ACI 318-08 overestimates the

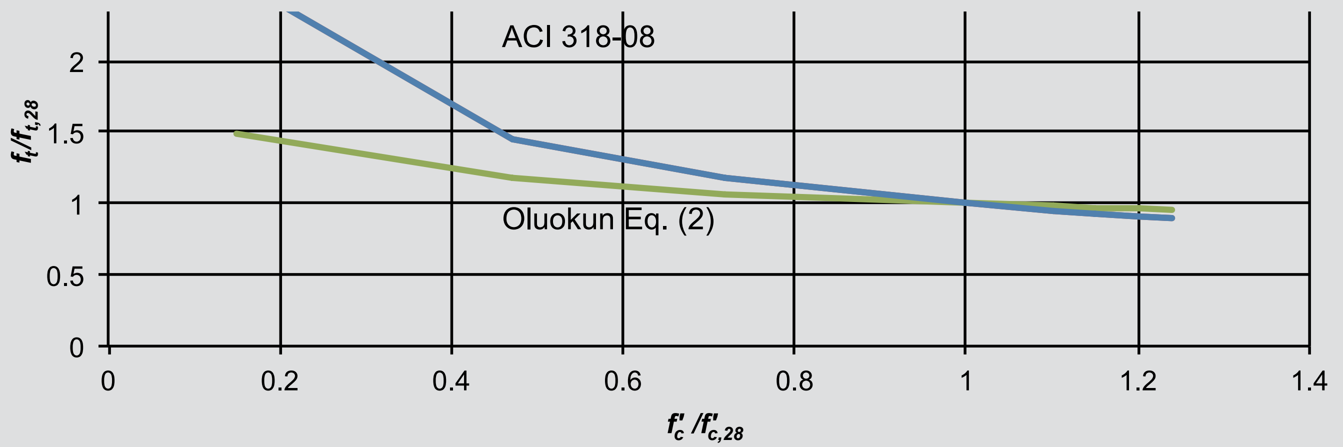

Figure 1. Normalized strength gains for early-age concrete. Note: f_t = concrete tensile strength; $f_{t,28}$ = 28-day tensile strength of concrete.

modulus of rupture for concrete compressive strengths less than 2180 psi (15 MPa) and underestimates it for strengths above 2180 psi (15 MPa). Further, Khan suggests that ACI 363R-92 overestimates the modulus of rupture for nearly all types of concrete.

One hundred and eighteen 4 × 4 × 20 in. (100 × 100 × 500 mm) specimens were cast using various concrete mixture proportions.[9] Twenty specimens were made for each of the ordinary portland cement concretes and nine specimens for each of the 70% slag cement replacement concrete and the 30% fly ash replacement concrete. Fifteen of the twenty specimens were tested at the ages of 1, 3, 7, 14, and 28 days at a strain rate of 5 με/min. The remaining five were tested at 28 days at different strain rates. Three were tested at a strain rate of 1 με/min and two at 30 με/ min. The nine specimens of fly ash and slag-cement concretes were tested at 7, 14, and 28 days at a strain rate of 5 με/ min. The compressive strength was measured using three 4 in. (100 mm) cubes at each age. Embedded steel bars were used to apply the load. Khan et al. concluded that the tensile-to-compressive strength ratio decreases as concrete matures and the tensile strength gain is similar for a wide variety of mixtures.

From this review, it is concluded that the tensile strength increases faster than compressive strength at early age when compared with the corresponding strength gains of mature concrete. This is determined from the higher slope of the tensile-to-compressive strength graph at early ages. Prediction methods used by ACI 318-08, and by extension the *PCI Design Handbook: Precast and Prestressed Concrete*,[10] underestimate modulus of rupture at compressive strengths greater than 2180 psi (15 MPa). Many of the models presented in the literature review were used for comparison with the data presented in this report.

Figure 1 examines the ratio of tensile strength gain to compressive strength gain. Both the tensile and compressive strengths are normalized to the 28-day strength for a 5000 psi (35 MPa) concrete. The ACI 318-08 equation (Eq. [1]) and the Oluokun equation for early-age concrete (Eq. [2]) are compared. Oluokun predicts a lower initial tensile strength than the ACI 318-08 formulation, which is consistent with Khan's findings. In both cases, the initial tensile capacity gain is higher than the compressive capacity. Thus, experimental validation should result in tensile strength gains on the order of 30% to 50% more than compressive strength gains based on Oluokun' s hypothesis. Theoretically, then, the inserts should perform well at early age. **Table 1** summarizes the model equations evaluated for tensile capacity.

A full analysis of this research complete with the proposed strength equations for both concrete tensile strength and insert breakout capacity is presented in *Concrete Breakout Capacity of Cast-in-Place Anchors in Early Age Concrete*.[11]

Breakout strength of headed studs

A review of the development of the breakout strength of headed stud inserts provides an evolution of predictive models over time, ultimately leading up to models currently used in practice. The models used for comparison in this paper are the models presented by Anderson, Tureyen, and Meinheit[12] and in the sixth edition of the *PCI Design Handbook*.

The *PCI Design Handbook* and ACI 318-08 present characteristic capacities based on a 5% fractile. A 5% fractile is defined as a 90% confidence that there is a 95% probability of the actual strength exceeding the nominal strength. This fractile is calculated by Eq. (3):

Table 1. Summary of tensile strength models

Source	f_t, psi	f_t, MPa
CEB	$f_t = 9.5\sqrt{f_c'}$	$f_t = 0.79\sqrt{f_c'}$
Oluokun	$f_t = 1.4f_c^{0.7}$	$f_t = 0.2f_c^{0.7}$
ACI 318-08	$f_{sp} = 6\sqrt{f_c'}$	$f_{sp} = 0.48\sqrt{f_c'}$
ACI 363-92	$f_{sp} = 7.4\sqrt{f_c'}$	$f_{sp} = 0.59\sqrt{f_c'}$
Mindess, Young, and Darwin best fit	$f_{sp}' = 4.34f_c'^{0.55}$	$f_{sp}' = 0.305f_c'^{0.55}$
Oluokun, > 6 hours and > 5 MPa	$f_t = 29.8f_c'^{0.79}$	$f_t = 0.584f_c'^{0.79}$
Oluokun, < 5 MPa	$f_t = 18.4f_c'^{0.6}$	$f_t = 0.928f_c'^{0.6}$
Khan (open)	$f_r(t) = 12.3f_c'(t)$	$f_r(t) = 0.085f_c'(t)$
Khan (sealed)	$f_r(t) = 11\left[f_c'(t)\right]^{2/3}$	$f_r(t) = 0.4\left[f_c'(t)\right]^{2/3}$
Khan (dry cured)	$f_r(t) = 10.5\left[f_c'(t)\right]^{2/3}$	$f_r(t) = 0.38\left[f_c'(t)\right]^{2/3}$

Note: Winters and Dolan 2013 provide a complete comparison of tensile strength comparisons. f_c = concrete compressive stress; f_c' = specified concrete compressive strength at 28 days; f_r = modulus of rupture; f_{sp} = splitting tensile strength; f_t = concrete tensile strength; t = time.

$$F_{5\%} = F_m(1 - K\nu) \tag{3}$$

where

$F_{5\%}$ = 5% fractile or characteristic capacity

F_m = mean failure capacity

K = factors for one-sided tolerance limits for normal distributions

ν = coefficient of variation

This approach is used for comparison with the equations presented in ACI 318-08 and the *PCI Design Handbook*.

Design models for connections in precast and cast-in-place concrete

Courtois[13] described problems with testing using small blocks that resulted in flexural splitting failure of the block before the ultimate capacity of the insert was reached. In addition, tests conducted at this time showed both concrete compressive strength and embedment depth to be important parameters for determining pullout capacity.

A shear cone breakout failure occurs where a concrete cone defined by the depth of embedment of the insert fails in shear. This is the type of breakout failure that was presented as a simple model and was used in early editions of the *PCI Design Handbook*. This approach is similar to punching shear calculations for a slab around a column.

Courtois identifies split cylinder tests as more informative than compression cylinder tests. He suggested that the breakout strengths might be more closely predicted when the concrete tensile strength is known than when only the concrete compressive strength is known. Courtois lists an area of future research: "In mass concrete structures, we must learn more about the ultimate tensile strength of concrete at very early ages. Forms are usually reanchored to a previous lift at ages of 48 to 72 hr and safe anchorage must be assured."

Sattler[14] reports the pure tension strength of connectors having headed studs based on a conical failure mode model. Sattler proposes a global safety factor equivalent to a load factor divided by a corresponding strength reduction factor of 2.0 to derive an allowable load. Sattler's work did not address spacing requirements of groups, edge-distance allowances, or anchoring to concrete in the tensile zone of a member where cracks could exist.

Bode and Roik[15] recommend design formulas for single studs loaded in tension based on cube strengths and the square root of the embedment length. They also note that for shorter studs, 2 in. (50 mm) in total length after welding, the standard deviation is greater than for longer studs because of the nonhomogeneous composition of the surrounding concrete and the close distance between the stud head and the concrete surface. Bode and Roik recommend reducing the strength by 20% for shorter studs. No further recommendations on other lengths are discussed.

Hawkins[16] conducted 12 tests on 1 in. (25 mm) diameter anchor bolt breakout specimens in 20 MPa (3000 psi) concrete. Embedment depth varied among 3, 5, and 7 in. (75, 125, and 175 mm). The washer diameter below the bolt varied among 2, 4, and 6 in. (50, 100, 150 mm). The thickness of this washer also varied as either ⅝ or ⅞ in. (16 or 22 mm). Nine specimens were 18 in. wide × 18 in. long × 9 in. deep (450 × 450 × 225 mm) and reinforced near the edges. The other three specimens were 46 in. wide × 46 in. long × 7 in. deep (1150 × 1150 × 175 mm) and also reinforced near the edges.

Hawkins's loading frame reacted against the concrete with 18 in. long × 2 in. wide (450 mm × 50 mm) steel beams with 16 in. (400 mm) center-to-center spacing for the smaller blocks and 30.5 in. long × 5 in. wide (760 mm × 125 mm) steel beams with 41 in. (1025 mm) center-to-center spacing for the larger block. Load was applied through a 100-ton (996 kN) center-hole ram positioned over a loading rod attached to the bolt.

Only three specimens showed conical breakout failures: one from the smaller block tests and two from the larger block tests. The reason presented for this is that the moment generated by the testing frame induces flexural cracking in the concrete, causing radial cracking failure before conical breakout failure can be reached. This is similar to the problems listed by Courtois, that is, a majority of the failures were splitting of the concrete.

From Hawkins's conclusions, an embedment depth of 8 to 10 times the bolt diameter is required for ductile behavior. Splitting failure is likely to occur when the embedment depth–to–bolt diameter ratio exceeds 4. Also, anchor bolts are likely to have ultimate capacities 20% to 30% less than comparable sized headed stud connectors.

Headed anchor breakout behavior in tension

A database was assembled on tension testing when the concrete capacity design method was in development.[12] Most of the data used 200 mm (8 in.) cube crushing strength $f_{cube,200}$. Using this information, they concluded that no additional tension testing was needed to describe the behavioral characteristics of welded headed stud anchors loaded in direct tension.

The tensile breakout strength N prediction takes the general form of the following equation:

$$N = af_c^{'\alpha} b h_{ef}^{\beta}$$

where

a = curve-fitting coefficient for concrete strength effect

b = curve-fitting coefficient for effective embedment depth

α = breakout strength coefficients determined by testing

h_{ef} = effective embedment depth of insert

β = breakout strength coefficient determined by testing

Regression analysis shows an excellent concrete breakout prediction equation using the variables $f_c^{'}$ and h_{ef}. The correlation coefficient R^2 value is nearly 0.98. The regression analysis shows the magnitude of the breakout strength coefficient α to consistently be about $^1/_2$ for $f_c^{'}$ and β to be $^3/_2$ for h_{ef}. Adding the variables for stud head diameter and stud shaft diameter did not significantly increase the R^2 value. These equations were developed using $f_c^{'}$ of the cylinder equal to $0.85f_{cube,200}$ as the correlation between cylinders and cubes.

ACI 318-08 appendix D assumes an average prediction equation for headed cast-in-place anchors in uncracked concrete $N_{u,ACIheaded}$ (Eq. [4]):

$$N_{u,ACIheaded} = 40\sqrt{f_c^{'}}\left(h_{ef}\right)^{1.5} \text{ lb}$$

$$(N_{u,ACIheaded} = 16.8\sqrt{f_c^{'}}\left(h_{ef}\right)^{1.5} \text{ N}) \quad (4)$$

This equation is used throughout this paper for comparison with the data collected and is equivalent to the equation presented in ACI 318-08 and the sixth edition of the *PCI Design Handbook*. This gives a test-to-predicted ratio of 0.992. However, when the cube strength conversion factor[3] of 1.11 is used, the test-to-predicted ratio increases to 1.11. Using the concrete strength conversion values from CEB-FIB, Eq. (5) is the *PCI Design Handbook* average prediction equation for headed cast-in-place anchors in uncracked concrete $N_{u,PCIheaded}$.

$$N_{u,PCIheaded} = 44\sqrt{f_c^{'}}\left(h_{ef}\right)^{1.5} \text{ lb}$$

$$(N_{u,PCIheaded} = 18.5\sqrt{f_c^{'}}\left(h_{ef}\right)^{1.5} \text{ N}) \quad (5)$$

Another ACI 318-08 alternative equation is presented for anchors with deep embedment, that is, h_{ef} greater than 12 in. (300 mm):

$$N_{u,ACIheaded} = 28\sqrt{f_c^{'}}\left(h_{ef}\right)^{5/3} \text{ lb}$$

$$(N_{u,ACIheaded} = 6.83\sqrt{f_c^{'}}\left(h_{ef}\right)^{5/3} \text{ N})$$

The fifth edition of the *PCI Design Handbook*[17] still used information based on Courtois's work, that capacity is proportional to h_{ef}^{2}, which overpredicts the test data by 16% to 30% depending on the strength conversion.

From regression analysis of tension data assuming f_c' equals $0.85f_{cube,200}$, Eq. (6) is the best fit equation.

$$N_{u,PCIheaded} = 35.57\left(f_c'\right)^{0.523}\left(h_{ef}\right)^{1.504} \text{ lb}$$

$$N_{u,PCIheaded} = 18.5\sqrt{f_c'}\left(h_{ef}\right)^{1.5} \text{ (N)} \qquad (6)$$

Anderson, Tureyen, and Meinheit note that a conversion factor for cubes of different sizes was taken as:

$$f_{cube,150} = \tfrac{1}{0.95} f_{cube,200} = 1.05 f_{cube,200}$$

Figure 2. Example stud assembly.

Headed steel stud anchors in composite structures: Tension and interaction

As the use of composite construction increases, conditions that lead to tension and combined shear and tension in headed studs are more prevalent. Examples include infill walls, coupling beams, connections to composite columns, or composite column bases. Pallares and Hajjar[18] note that the most advanced information on headed studs is included in the sixth edition of the *PCI Design Handbook* and ACI 318-08.

Composite design research considered only tests with no edge effects and concrete strength greater than 3 ksi (20 MPa). The 3 ksi limit is the minimum strength permitted by the American Institute of Steel Construction for composite structures. Based on the work presented by Pallares and Hajjar, concrete breakout is prevented if h_{ef} is greater than 7.5*d* (where *d* is the diameter of split cylinder or headed stud). This gives values similar to the 8 to 10 values proposed by Hawkins.[16]

Anchor strength test program

The equations in the sixth edition of the *PCI Design Handbook* for concrete breakout of headed studs are based on mature concrete values. In the precast concrete industry, loads for stripping, lifting, and handling are applied before concrete reaches full maturity. There is little research on the effects of early concrete age on breakout strength as opposed to 28-day low-strength concrete. The test protocol follows ASTM E488[19] and is conducted in uncracked concrete, as would be expected for stripping and handling of newly cast precast concrete panels. ACI 355.2-07[20] criteria are for testing inserts in cracked concrete and are not applicable to this test program.

Test specimens

Headed stud assemblies with a nominal length of 3 in. (75 mm) and a shank diameter of $^1/_2$ in. (13 mm) conforming to ASTM A1044[21] were used throughout testing. These specimens had a h_{ef}/d ratio of approximately 6, less than the 7.5 value predicted to assume steel yielding. Thus the specimens were predicted to fail in concrete breakout behavior. The studs were resistance welded directly to a 3 × 3 × ¼ in. (75 × 75 × 6 mm) plate meeting the requirements for plate thickness set forth in the *PCI Design Handbook.* On the other side of the plate, a $^5/_8$ in. (16 mm) diameter threaded rod 3 in. (75 mm) long was conventionally welded to the plate (**Fig. 2**). The threaded rod allowed for a means of attachment to apply a tensile load. A foam covering was glued around each threaded rod to keep the rod centered in the holes in the bottom of the forms and allow for easy form removal even if the studs shifted during the placing of the concrete.

The effective embedment depth h_{ef} was nominally 75 mm (3 in.) but had to be corrected for the weld burnoff and plate thickness. These corrections led to a h_{ef} value of 2.81 in. (71.5 mm). Approximately half of the assemblies were measured and found to be within 3 mm (0.12 in.) of this value. This actual effective embedment length h_{ef} of 2.81 in. (71.5 mm) is used for data reduction. The final length is consistent with the guidelines in the sixth edition of the *PCI Design Handbook.*

The stud assemblies were fabricated to replicate assemblies used in practice. The studs were resistance welded to the plates, and the threaded rods were conventionally welded to the opposite side of the plates. This led to some of the studs having an offset with respect to the threaded rod, not being perpendicular to the plate, or both (Fig. 2). To ensure weld and assembly integrity during testing, each stud assembly was preloaded to 85% of its calculated yield capacity in a universal testing machine. **Figure 3** shows a test assembly with the top and bottom attachments. Only one specimen failed during this testing. The failure resulted from an incomplete weld around the threaded rod.

Figure 3. Stud assembly loaded in the Instron.

Figure 4. Tripod loading frame.

In addition to proof testing of the assemblies, a single test was conducted on the shaft of a stud. The head and plate were cut off of an assembly from a 12-hour test from the first series. The stud shaft was then milled down to a constant cross section to be loaded in grips in a universal testing machine. A strain gauge was attached to the stud shaft and the shaft was loaded to failure.

Three direct tension tests were conducted on representative stud assemblies. The assemblies were loaded to failure using a universal testing machine. The assemblies were loaded to failure in the same manner as the stud quality control tests and had an average tensile strength of 77 ksi (530 MPa).

Concrete blocks Six test blocks were cast measuring 27 × 66 in. (690 × 1675 mm) on the bottom face and 29 × 66 in. (740 × 1675 mm) on the top face by 10 in. (250 mm) thick for the first series of tests. A side draft allowed for easy form removal. The concrete was a standard Wyoming Department of Transportation bridge deck mixture provided by the local ready-mixed concrete plant and had a 28-day specified compressive strength of 5000 psi (35 MPa). The mixture used ASTM C33[22] size 57 and size 67 crushed coarse aggregate. Ten studs were placed in the bottom of each form before placing the concrete.

ASTM E488 recommends that studs be spaced more than $2h_{ef}$ from the edge of the structural member. For spacing between studs, this minimum is doubled to $4h_{ef}$. The studs were spaced 7 in. (180 mm) from edges and 13 in. (330 mm) or $(2^{1}/_{3})h_{ef}$ and $(4^{1}/_{3})h_{ef}$, respectively, from each other, thus meeting or exceeding the ASTM guidelines. In addition, each form had two members across the top to hold lifting inserts. Each insert was spaced 13.5 in. (340 mm) from each end to minimize the moment created during stripping and to reduce the possibility of cracking the block while moving it into position for testing. The blocks were reinforced with a single no. 6 (M19) reinforcing bar in the center of each block to provide a means of rotating the block once it had been cast. While casting the blocks, thirty-four 4 × 8 in. (100 × 200 mm) and twenty-six 6 × 12 in. (150 × 300 mm) cylinders were also cast to determine the concrete strength during testing. After casting, the specimens and cylinders were then covered with plastic to cure.

In a second set of tests, the number of studs per block was reduced to four to increase the stud spacing and edge distance. The minimum edge distance was increased to 8.25 in. (210 mm) and the stud spacing increased to 16.5 in. (420 mm), or $(2^{3}/_{4})h_{ef}$ and $(5^{1}/_{2})h_{ef}$, respectively. Twenty 6 × 12 in. (150 × 300 mm) and twenty 4 × 8 in. (100 × 200 mm) cylinders were cast to determine the strength of the blocks at the time of testing as well as the 28-day strength. The concrete mixture was the same as used for the first tests. The blocks and cylinder were again covered with plastic to cure.

Loading

A primary difference between the first series of tests and the second is the use of a different loading frame. ASTM E488 recommends that the minimum distance from the center of the stud to the nearest point of contact on the loading frame be no less than twice the effective embedment depth h_{ef} of the stud. Using this criterion, a tripod frame was constructed with a distance from the center of the tripod to the nearest point of contact of 6 in. (150 mm), or $2h_{ef}$ (**Fig. 4**). The frame contacted the concrete with three 2 × 2 in. (50 × 50 mm) steel plates. This frame was used throughout the first round of testing. Many of the breakout segments flared out and extended to the frame contact points, which could affect the breakout strength.

For the second round of testing, a frame with an inside spacing of 23 in. (580 mm) was used. The frame's reaction beams were two 12 in. long (300 mm) 2 in. wide (50 mm) steel tubes that were placed along the long edge of the block for each test (**Fig. 5**). In all cases in the second round of testing, the support points were at least $6h_{ef}$ from the load point.

In both series of tests the load was applied by a 12-ton (107 kN) center-hole ram. The ram was attached to a 20 kip (80 kN) load cell and was then placed on top of the loading frame. The load cell was connected to the threaded rod on the stud assemblies by two eye bolts and a clevis. This arrangement acted as a means of aligning the rod and anchor so there was minimal bending in the loading system (Fig. 5).

Displacement of the stud relative to the concrete was measured by two linear potentiometers placed on either side of the insert threaded rod. The potentiometers were placed on a bridge so that they would not be affected by the breakout surface or the deflections of the loading frame.

Figure 5. Beam loading frame.

Circles were drawn marking the theoretical ACI 318-08 breakout diameter, and a picture was taken of the block before the breakout tests were conducted. The predicted breakout load was calculated using Eq (5) and an effective embedment length of 2.81 in. (71.5 mm). This value was used to determine the rate of loading and initial load values as set forth by ASTM E488.

Testing program

During the first series of tests, each block was removed from the mold, moved into position for testing, and rotated so that the studs were on the top side of the block two hours prior to the breakout test. One hour prior to testing, two cylinders were tested in compression following ASTM C39[23] and two cylinders were tested in splitting tension in accordance with ASTM C496.[24] On the second compression cylinder, a load was applied equal to 40% of the failure load of the first cylinder. The cylinder was then tested with an impact hammer in five locations along the cylinder. The compressive load and stress, tensile load, and impact hammer values were recorded. After testing the cylinders, the block was evaluated along the side and top with the impact hammer and the values were recorded.

The breakout tests began by moving the testing frame into position over the stud, attaching the loading rod, and positioning the potentiometer bridge around the threaded rod. The initial load of approximately 200 lb (900 N) was applied, then the data acquisition system was started, and the stud was pulled to failure using a constant displacement hydraulic pump. After failure the pump was shut off, the data acquisition system stopped, and the process repeated for the next stud until all of the studs on the block were tested. The breakout tests were performed at concrete ages of 12, 16, and 20 hours and 3, 8, and 28 days.

Following all breakout tests on a block, the impact hammer was used again to test the top of the block. Another three cylinders were tested in compression and three were tested in split tension. The last two compression cylinders were also tested using the impact hammer in the same manner as the cylinder hammer test before the breakout tests. The maximum breakout diameters parallel to both the short side and the long side of the block were measured as well as the depth to the top of the head on the stud.

The second set of testing was conducted in a similar manner. Because there were fewer studs per block, two blocks were tested at each of the concrete ages 12, 16, and 20 hours. To be able to test two blocks at once, all blocks were removed from the molds at a concrete age of 9 hours and placed into position to be tested. Supplementing the cylinder tests at the time of the breakout testing, five 6 ×

Figure 6. Typical breakout failure pattern at 16 hours.

12 in. (150 × 300 mm) and five 4 × 8 in. (100 × 200 mm) cylinders were tested in compression and splitting tension, respectively, at 28 days to establish the 28-day strength. **Figure 6** shows a typical series of breakout failures.

Results

The data collected from each stud test include the loading and displacement history. The failure type was recorded, and when the failure mode was concrete breakout, the maximum length and width of the breakout plane was measured at the original concrete surface. The failures were all expected to be in concrete breakout, but in the first round of testing, steel failure began to occur at a concrete age of 3 days.

For each compressive cylinder, both the ultimate load and ultimate stress were recorded. On the split tensile cylinders the ultimate load was recorded and the tensile stress T was calculated using Eq. (7) per ASTM C496.

$$T = \frac{2P}{\pi l d} \quad (7)$$

where

T = tensile splitting capacity of a split cylinder

P = force applied in split cylinder test

l = length of a split cylinder

In addition to these cylinder tests, three of the compressive cylinders were tested with an impact hammer with 40% of the expected ultimate load used to hold the cylinder in place.

Breakout results

During the first stage of testing, all failures occurred in concrete breakout until the concrete reached the age of three days. For the 28-day block, all failures occurred in the steel stud. Using Eq. (5) for comparison, the average test-to-predicted ratios for the concrete breakout failures ranged from 1.3 to 1.8 (**Tables 2** and **3**). These equations underpredicted the strength in all cases. **Figure 7** presents the data from test series 1 and a comparison with Eq. (5) and (6). Test series 2 has similar results.

Table 2. Comparison of predicted breakout strength and test results for test series 1

Test	12 hours*		16 hours†		20 hours‡		3.29 days§		8.54 days‖		28 days#	
	P_u, lb	P_u/N_{cb}	P_u, lb	P_u/N_{cb}	P_u, lb	P_u/N_{cb}	P_u, lb	P_u/N_{cb}	P_u, lb	P_u/N_{cb}	P_u, lb	P_u/N_{cb}
1	8067	1.46	10,105	1.42	14,176	1.84	11,498	1.19	7868	0.77	12,292	1.21
2	8878	1.60	12,077	1.69	13,961	1.82	**11,416**	1.18	**16,206**	1.59	**17,222**	1.69
3	8629	1.56	9566	1.34	15,962	2.08	14,001	1.44	**17,597**	1.73	**15,141**	1.48
4	7800	1.41	10,804	1.51	n.d.	n.d.	11,695	1.21	**12,105**	1.19	**14,875**	1.46
5	9667	1.75	9736	1.36	13,314	1.73	**15,583**	1.61	10,931	1.08	**11,160**	1.09
6	10,637	1.92	9379	1.31	12,783	1.66	**14,099**	1.45	**12,243**	1.20	**16,762**	1.64
7	11,696	2.11	10,504	1.47	15,018	1.95	**17,161**	1.77	**11,875**	1.16	**15,012**	1.47
8	10,981	1.98	10,618	1.49	12,510	1.63	13,823	1.43	**13,289**	1.30	**14,807**	1.45
9	9110	n.d.	10,123	n.d.	11,923	n.d.	**12,978**	1.34	**12,507**	1.23	**13,254**	1.30
10	n.d.	n.d.	n.d.	n.d.	n.d.	n.d.	n.d.	n.d.	**12,249**	1.20	**17,366**	1.70
Average	n//a	1.73	n/a	1.45	n/a	1.82	n/a	1.41	n/a	1.25	n/a	1.44

* f'_c = 860 psi; N_{cb} = 5533 lb.
† f'_c = 1430 psi; N_{cb} = 7135 lb.
‡ f'_c = 1660 psi; N_{cb} = 7687 lb.
§ f'_c = 2640 psi; N_{cb} = 9694 lb.
‖ f'_c = 4070 psi; N_{cb} = 10,200 lb.
f'_c = 5120 psi; N_{cb} = 10,200 lb.

Note: Values in bold represent tensile failures of insert assembly. Effective embedment of insert h_{ef} = 2.81 in. (71.5 mm) for all predictions. f'_c = specified concrete compressive strength at 28 days; n/a = not applicable; N_{cb} = predicted tensile breakout strength; n.d. = no data; P_u = maximum force recorded on load cell in pullout test. 1 lb = 4.448 N.

Table 3. Comparison of predicted and test results for test series 2

Test	12 hours		16 hours		20 hours	
	P_u, lb	P_u/N_{cb}	P_u, lb	P_u/N_{cb}	P_u, lb	P_u/N_{cb}
1	9933	1.65	14,267	1.91	19,878	2.37
2	8399	1.39	13,418	1.79	20,732	2.48
3	8295	1.38	14,012	1.87	>20,000	n.d.
4	9561	1.59	11,007	1.47	20,297	2.42
1	4942	0.82	12,483	1.67	20,923	2.50
2	7026	1.17	11,690	1.56	>20,000	n.d.
3	7075	1.17	10,128	1.35	>20,000	n.d.
4	6239	1.04	n.d.	n.d.	20,852	2.49
Average	n/a	1.28	n/a	1.66	n/a	2.45

* f'_c = 1020 psi; N_{cb} = 6026 lb.
† f'_c = 1570 psi; N_{cb} = 7476 lb.
‡ f'_c = 1970 psi; N_{cb} = 8374 lb.
Note: f'_c = specified concrete compressive strength at 28 days; n/a = not applicable; N_{cb} = predicted tensile breakout strength; n.d. = no data; P_u = maximum force recorded on load cell in pullout test. 1 lb = 4.448 N.

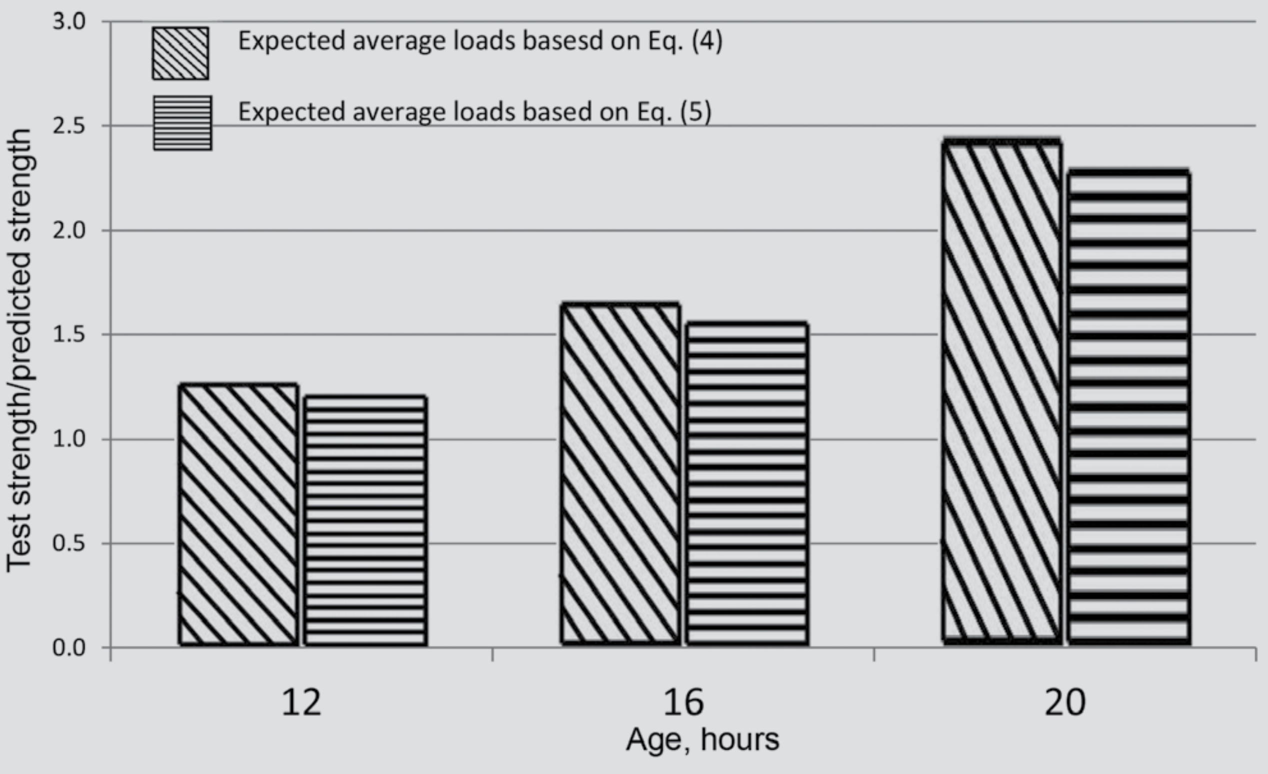

Figure 7. Test-to-predicted ratios of first series of tests.

Table 2 requires additional amplification. For the 8- and 28-day tests, the predicted failure mode changes from concrete breakout to steel yield. Thus, only the data for specimens less than 8 days old is appropriate for analysis of concrete breakout capacity. Second, the material specifications for the study provide only the minimum yield and ultimate tensile stress. Loads exceeding these minimums are possible. Overstrength conditions occurred in numerous tests, evidenced in Table 1 where both breakout loads and steel yield loads exceed the lower bound of 10.2 kip (45.4 kN) yield and 13 kip (57.8 kN) ultimate. This is discussed in more detail later. All tests were terminated at about 20 kip (89 kN), the limit of the load cell.

The second series of tests was conducted to determine whether the compressive field caused by the loading frame may have caused the test-to-predicted ratio to be higher than 1.0. In the second series, once again the average test-to-predicted ratios were all greater than 1.0. At 20 hours, three of the studs exceeded the 20 kip (89 kN) capacity of the load cell and the test was stopped to avoid damaging the data acquisition equipment. These test results are not included in the test-to-predicted calculations.

As an additional means of comparison with the current design equations, the 5% fractile was calculated for each of the different curing times from each series using Eq. (3) and K values from ACI 355.2-07. The calculated 5% fractile from this research exceeds the 5% fractile from ACI 318-08 in all cases except the 12-hour tests from the second series. However, when the 12-hour tests from both series are combined to develop a single 5% fractile, the value exceeds the 5% fractile from ACI 318-08.

Concrete strength results

The compressive cylinders show a 28-day average compressive strength of 5320 psi (36.7 MPa) for the first series of tests and 5120 psi (35.3 MPa) for the second series of tests. The cylinder's compressive and splitting tensile strength increased with age as was to be expected. The strengths were then compared with the equation presented by ACI 318-08 (**Fig. 8**).

Average tensile strengths were consistently higher than the expected tensile strengths based on the equations presented in the literature review comparing the concrete compressive strength with the splitting tensile strength. The split tensile strength prediction is increased on average by 25% when crushed coarse aggregate is used as suggested in the literature review. Applying this increase to the equations more closely matches the data collected (**Fig. 9**). A power fit is also plotted as a thin solid line for comparison and also has a power coefficient greater than the 0.5 in the ACI 318-08 equation, which is consistent with the earlier literature reviews.

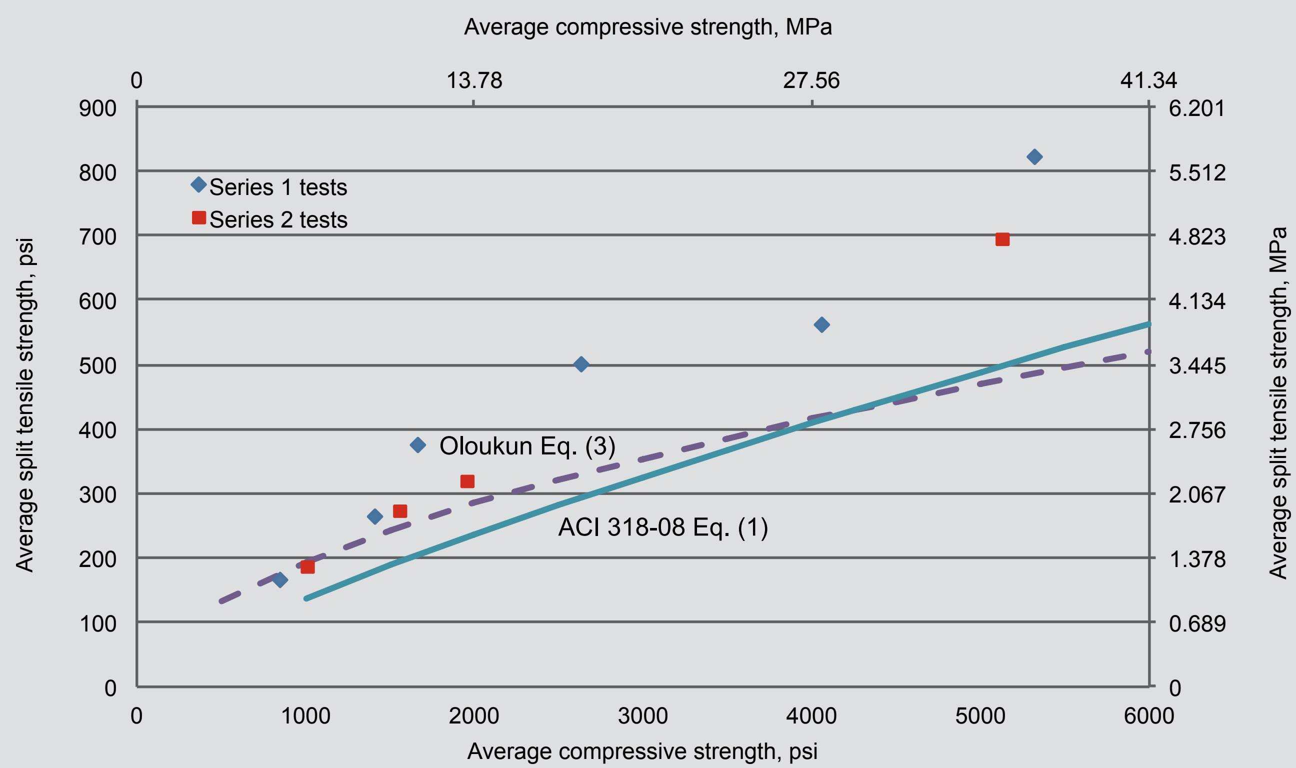

Figure 8. Split tensile to compressive strength comparison.

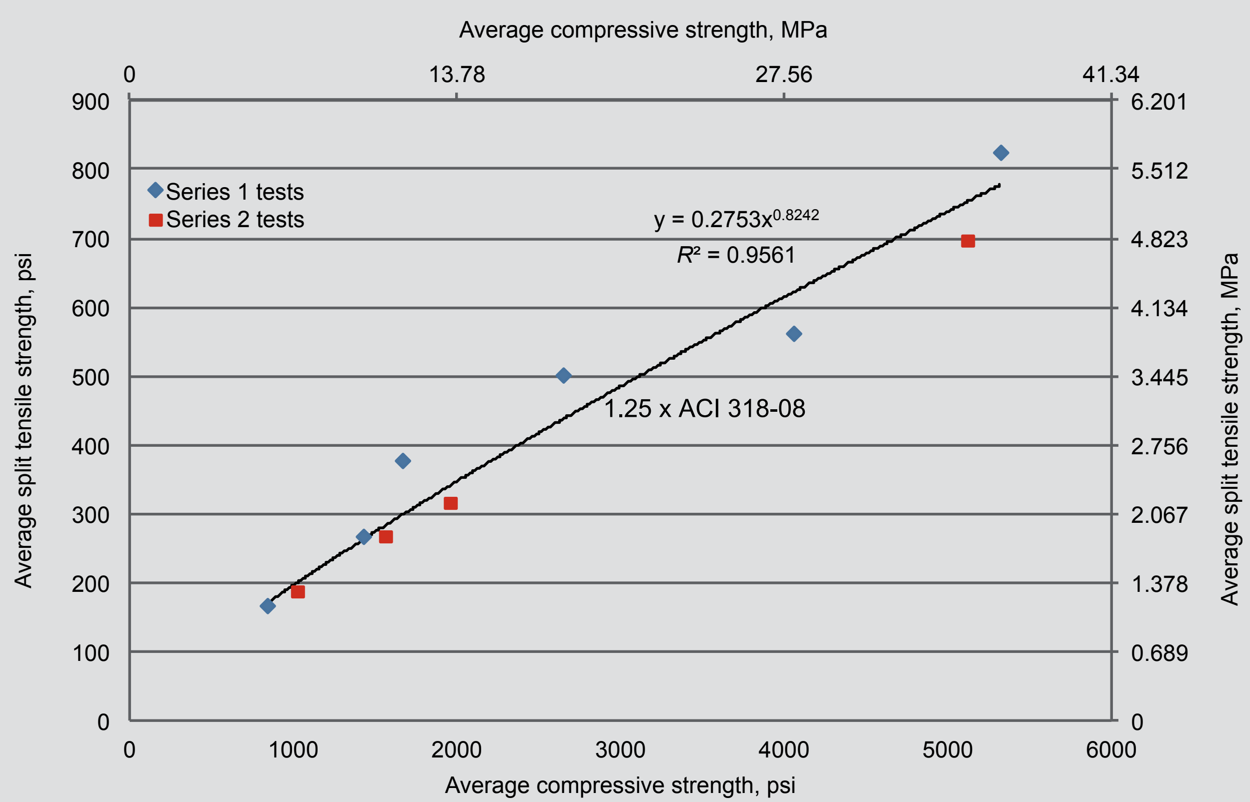

Figure 9. Split tensile to compressive strength comparison using Teychenné (1954). Note: *R* = correlation coefficient.

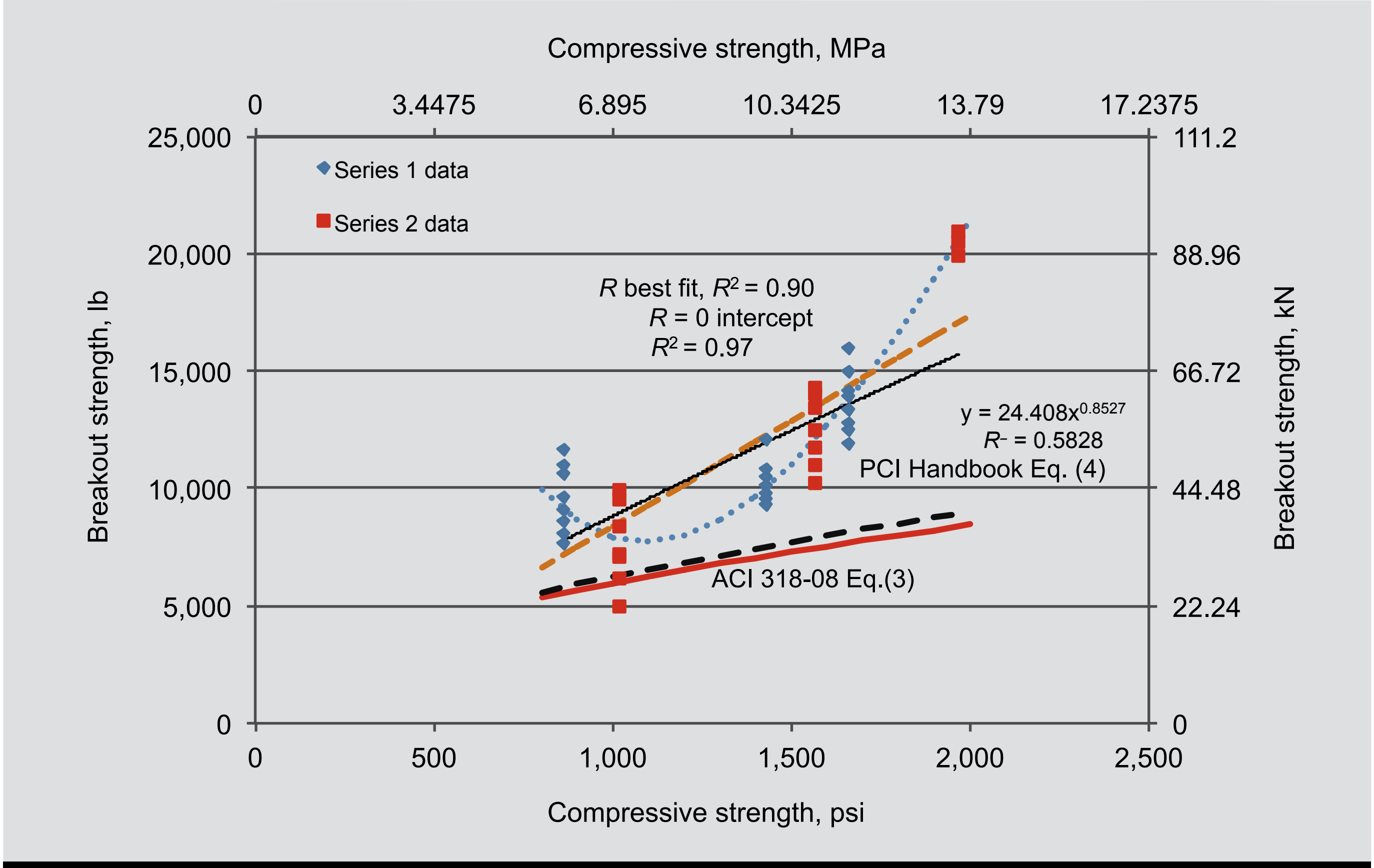

Figure 10. Comparison of best fit and current equations. Note: R = correlation coefficient.

Statistical analysis

Several variables were chosen for a possible model for early-age breakout strength. The data were limited to the breakouts from the tests with a concrete age of 20 hours and less. This was done so that only breakout failures are modeled. The variables for developing a model include the series of the test, the compressive strength, the splitting tensile strength, the age of the concrete, and the square root of compressive strength. When these variables were used to fit the breakout data, only compressive strength, square root of compressive strength, and the intercept are statistically significant based on a P-value of 1%. This statistical analysis agreed with a model selection analysis using both Cp and adjusted R squared. Cp is a calculation within the statistical analysis program to prevent underfitting the data, to ensure enough variables adequately describe the data, and to reduce bias. **Figure 10** plots the best fit line using the square root of compressive strength, compressive strength, and an intercept as the R best fit line.

This statistical analysis model has several problems. Over this range of compressive strengths, multicollinearity between the square root of compressive strength and compressive strength makes it difficult to determine how the variables relate to one another and their meaning in the regression equation. There is also a problem with the intercept and the slope of the line below about 1000 psi (7 MPa) compressive strength. There is no physical reason why the breakout strength should be greater than zero with a compressive strength of zero, and the strength should not increase as the compressive strength decreases.

Considering these statistical modeling problems, other models were investigated to develop best-fit equations. An examination of the current models used for prediction shows that the intercept is set to zero. The same variables that were used for the best fit equation produced the R 0-intercept line (Fig. 10). This gives an R squared value of nearly 0.97, about the same as the 0.98 listed by Anderson, Tureyen, and Meinheit. However, this high value is due to the poor model used for comparison and should not be used as a means of determining how well the equations fit the data. A power fit using only compressive strength is also used in an attempt to develop a relationship similar to the code equations. The exponent that produces the best fit of the data is equal to 0.85. This value is larger than the exponents of about 0.5 used in the code equations.

Figure 10 separates test 1 and test 2 data. Both tests use the same concrete mixtures, and concrete strength is reported at the time of testing, for example 20 hours. The strength gain of test 2 data is slightly less than for test 1. Consequently, the statistical data are examined singly and in combination. The chart indicates that for all combinations of modeling and loading, the breakout model for mature concrete is conservative for the prediction of breakout capacity of early-age concrete.

Statistical significance tests Statistical significance tests were conducted using guidance from ACI 335.2-07 in which the minimum compressive strength for which a statistical significance test is valid is 2500 psi (17 MPa). Although the compressive strengths were lower than the ACI 355.2-07 limit, the significance tests were conducted as a means of comparison.

Nine statistical tests were conducted to compare each of the tests conducted with a concrete age of less than 20 hours from each series. The means were normalized to the square root of the lower compressive strength in all cases. The means of the breakout data were statistically similar except for the cases involving the 16-hour tests from both series and the 20-hour test from the second series. The second-series 20-hour tests had a low coefficient of variation due to the limitations of the testing equipment. This led to the apparent statistical difference in the mean from the other tests.

Steel stud tests

Many of the steel failures exceeded the expected failure load based on the mechanical properties listed in the *PCI Design Handbook*. The single stud shaft tests also exceeded the expected yield and ultimate stresses. The yield stress from the steel stud shank test was approximately 66 ksi (455 MPa) and the ultimate stress was 82 ksi (565 MPa).

Table 4. Stud assembly failure loads

Stud	Failure load, lb
1	77,670
2	75,830
3	77,690

Note: 1 lb = 4.448 N.

This leads to a stud failure value of 16 kip (71 kN) on a 0.5 in. (13 mm) diameter stud.

The tension tests of the entire stud assemblies yielded similar results to the single stud shaft test. The average yield stress of the three tests was approximately 54 ksi (375 MPa), and the average ultimate tensile stress of the three tests was 77 ksi (531 MPa) (**Table 4**). All failures were cup and cone tensile failures away from the weld (**Fig. 11**).

The headed stud tensile test results indicate that the studs have higher strength than the minimum specified values. The tests do not, however, explain the unusually high pullout test data where the required tensile capacity of the stud assembly would have to be in excess of 105 ksi (725 MPa). The excess capacity is attributed to the misalignment of the studs and the plates in some assemblies (Fig. 2). The total tensile force is the resultant of the yield

Figure 11. Headed studs after testing.

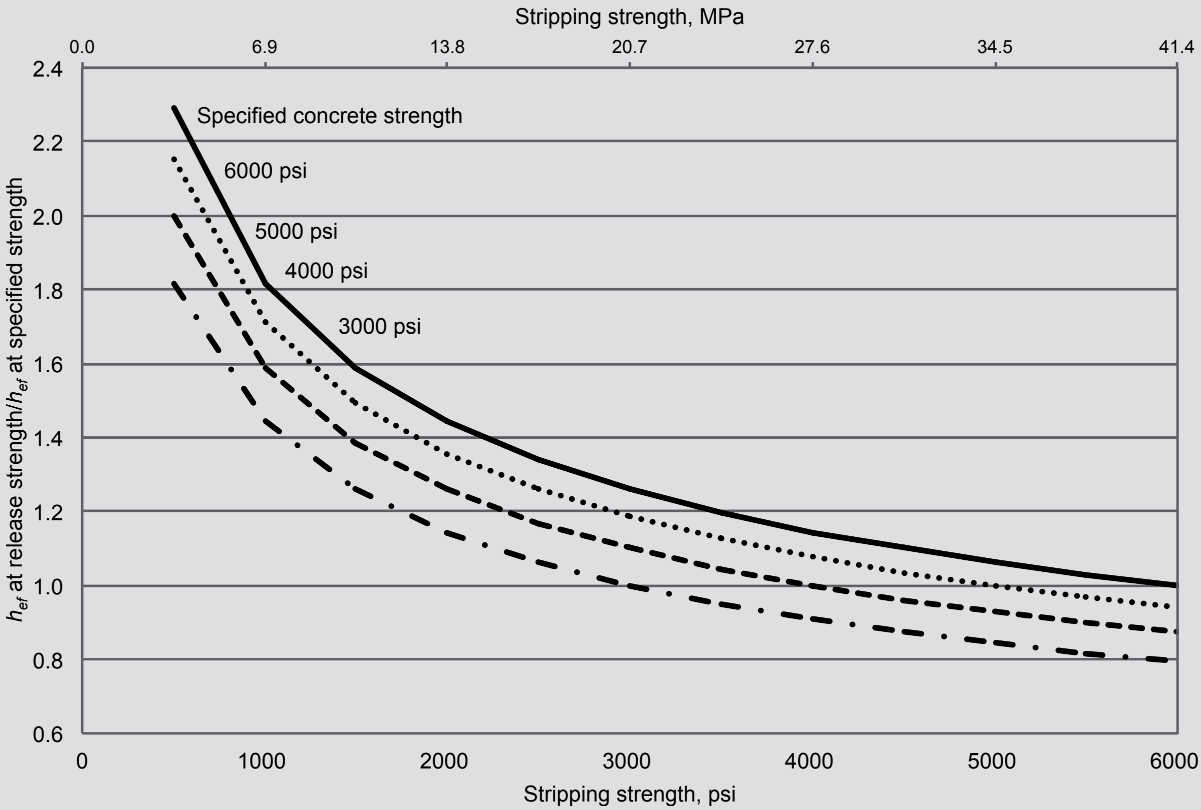

Figure 12. Comparison of minimum of embedment depth at various compressive strengths. Note: h_{ef} = effective depth of insert. 1 psi = 6.895 kPa.

capacity of the stud and the prying action of the plate. Even with the prying, the stud failed in tension, indicating that the breakout capacity exceeded the full tensile breakout capacity.

Design and production considerations

Breakout strength sensitivity The compressive strength of a specimen when the load is applied remains an important variable. At lower strengths a greater embedment depth is needed to develop the stud tensile capacity. Using this information, comparison charts can be generated relating the effective embedment depth required at the specified concrete stripping strength and the necessary effective embedment depth at different stripping strengths. **Figure 12** provides an example plot for a single stud.

For this example, if a precast concrete panel is to have load applied at 3000 psi (21 MPa) but the strength at the time of stripping is 2000 psi (14 MPa), the effective embedment length must be approximately 15% greater to avoid a breakout failure. Therefore, effective embedment depth for early stripping must be based on the specified strength at the time of load application, and that strength must be verified.

Insert strength considerations

The greater-than-expected steel strength can lead to another design problem. In the case of overload, most designs are made to result in a ductile failure in the steel. If the ultimate steel stress is greater than expected, there is a possibility of a brittle concrete breakout failure.

The increase in effective embedment depth needed is even greater with multiple studs and larger-diameter studs. To prevent this situation, a provision can be developed to specify a maximum yield stress to ensure ductile failure.

Other considerations of concrete tensile stress

Precast concrete production pieces are commonly stripped once the cylinder compressive strength is at or above the specified stripping strength for a member. The data presented in this report show that using only the compressive strength as a means of determining the expected breakout capacity is sufficient for a wide range, but not all, concrete mixtures. Regardless, the true breakout capacity of an insert is based on the tensile strength of the concrete. Therefore, if unexpected breakout failures occur, the loads may be higher than assumed, the insert may have a more complex interaction, or the mixture proportion may have a lower-than-average tensile-to-compressive strength ratio. If a plant is using a mixture proportion with lower-than-average tensile to compressive strength, that is, f_t is less than $6\sqrt{f_c'}$, it may be prudent to either conduct in-plant tests to determine the true breakout strength of the specific anchors used in their construction or to adjust the specified stripping strength to account for the lower tensile capacity.

Conclusion

The average breakout values in Tables 2 and 3 exceeded the expected breakout values based on Eq. (5) and (6) by the amount suggested in Fig. 1. Based on this test program and the theoretical tensile strength gain, the capacity predictions in the *PCI Design Handbook* are sufficient for the design of inserts or lifting inserts for concrete compressive strengths as low as 1000 psi (7 MPa) in uncracked concrete. This is consistent with the findings in the literature review that tensile strength increases faster than compressive strength at early age.

Although the age of the concrete does not need to be corrected for compressive strength, the strength at release or stripping remains an important factor. Low-strength concrete is more sensitive to breakout; thus the strength specified for stripping and handling must be verified at the time of loading.

Last, to reiterate important initial assumptions, headed studs were used as a proxy for all lifting inserts. Specialty inserts are projected to behave similarly to the studs. That is, if the insert behaves as calculated in normal-strength concrete, it should also behave as predicted in lower-strength concrete. Additional testing of inserts that have unusual geometry or display erratic behavior is highly recommended. No shear or side breakout strength tests were performed to validate this assumption.

Acknowledgments

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Notation

a = curve fitting coefficient for concrete strength effect

b = curve fitting coefficient for effective embedment depth

Cp = calculation within the statistical analysis program to prevent underfitting the data, to ensure enough variables adequately describe the data, and to reduce bias

d = diameter of split cylinder or headed stud

f_c = concrete compressive strength

f_c' = specified concrete compressive strength at 28 days

$f_{cube,150}$ = compressive strength of a 150 mm (6 in.) cube

$f_{cube,200}$ = compressive strength of a 200 mm (8 in.) cube

f_r = modulus of rupture

f_{sp} = splitting tensile strength

f_{sp}' = compressive splitting stress

f_t = concrete tensile strength

$f_{t,28}$ = 28-day concrete tensile strength

$F_{5\%}$ = 5% fractile or characteristic capacity

F_m = mean failure capacity

h_{ef} = effective depth of insert

K = factors for one-sided tolerance limits for normal distributions

l = length of a split cylinder

N = tensile breakout strength

N_{cb} = predicted tensile breakout strength

$N_{u,ACIheaded}$ = tensile breakout strength in uncracked concrete determined by ACI 318-08 appendix D

$N_{u,PCIheaded}$ = tensile breakout strength determined by *PCI Design Handbook: Precast and Prestressed Concrete*

P = force applied in split cylinder test

P_u = maximum force recorded on load cell in pullout test

R = correlation coefficient

t = time

T = tensile splitting capacity of a split cylinder

α = breakout strength coefficients determined by testing

β = breakout strength coefficient determined by testing

ν = coefficient of variation

About the authors

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Abstract

CI 318-08 appendix D, "Anchoring to Concrete," requires a minimum concrete strength of 2500 psi (17 MPa). Precast concrete elements are often stripped at strengths lower than 2500 psi, raising a concern as to whether the ACI equations are applicable for early-age concrete. This paper provides theoretical and experimental validation of the use of inserts in concrete strengths as low as 1000 psi (7 MPa). Theoretical validation is made by examining the gain in tensile and compressive strength of early-age concrete. Experimental validation comprised 78 pullout tests on headed stud assemblies in concrete with an age as young as 12 hours. The work concluded the following:

- The tensile strength of early-age concrete rises faster than the compressive strength.

- The pullout strength of the inserts exceed the theoretical capacity predicted by ACI 318-08 appendix D and the *PCI Design Handbook: Precast and Prestressed Concrete.*

Keywords

Breakout, concrete, early age, insert, strength.

Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute's peer-review process.

Reader comments

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