CONCRETE BREAKOUT CAPACITY OF CAST-IN-PLACE ANCHORS IN EARLY AGE CONCRETE

PRECAST/PRESTRESSED CONCRETE INSTITUTE

R and D RESEARCH REPORT

James Winters
Daniel P. Jenny Fellowship recipient

and

Dr. Charles W. Dolan

University of Wyoming

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EXECUTIVE SUMMARY

ACI 318-08 Appendix D Anchoring to Concrete requires a minimum concrete strength of 17.4 MPa (2500 psi) for use of the equations in Appendix D. Precast concrete elements are often stripped at strengths lower than 17.4 MPa (2500 psi). This practice raises a concern whether the ACI or PCI Design Handbook equations are applicable for in plant operations when members are stripped at an early age. The study was motivated in part by unconfirmed reports of failure of inserts during initial removal and storage of precast concrete elements at the plant.

This report provides theoretical and experimental validation of the use of ACI 318-08 for concrete strengths as low as 7 MPa (1000 psi). While the compressive strength of the concrete is specified for striping and handling precast elements, the pullout capacity of inserts is dependent on the tensile strength of concrete. Theoretical validation is made by examining the gain in tensile and compressive strength of early age concrete and comparing that strength gain to the relative ratio of tensile to compression strength of cured concrete. Experimental validation is accomplished by conducting 78 pullout tests on 13 mm (½ in.) diameter by 75 mm (3 in.) long headed stud assemblies with 16 mm (⅝ in.) threaded ends for attachment cast into the soffit of a 250 mm (10 in.) deep concrete block and tested at an age as low as 12 hours. The results are summarized below from Tables 4-1 and 4-2 in this report. In all cases, the concrete breakout capacity is greater than the predicted capacity. This is consistent with the finding that concrete tensile capacity rises faster than the compression strength at early ages.

An underlying assumption in this work is that the behavior of headed studs is a proxy for other lifting hardware and that if the headed stud assembly fails at a lower than predicted load, a striping insert would be assumed to fail at a lower load. Conversely, successful performance of headed stud assemblies is indicative that other striping inserts would perform adequately. Specialty lifting inserts were not tested and if performance of a particular insert is in question, additional testing is recommended.

The test specimens were cast upside down, consistent with the Anderson, Tueryen, and Meinheit tests used to develop the PCI Design Handbook recommendations. The test program does not address production issues such as consolidation around inserts, air entrapment under inserts, or “top bar” effects.
Notes: 1) Values in bold (red) represent tensile failures of insert assembly.  2) \( h_{f} = 71.5 \text{ mm (2.813 in.)} \) for all predictions

### Comparison of Predicted and Test Results for Test Series 2

<table>
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<th>Test</th>
<th>( P_u ) (lbs)</th>
<th>( P_u/N_{cb} ) (kN)</th>
<th>( P_v ) (lbs)</th>
<th>( P_v/N_{cb} ) (kN)</th>
<th>( P_{uv} ) (lbs)</th>
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<td>11007</td>
<td>49.0</td>
<td>20297</td>
<td>90.3</td>
</tr>
</tbody>
</table>

Average: 1.28 | 1.66 | 2.45
The report is presented in four parts. First, a review of the literature on both concrete breakout capacity and the relationship between tensile and compressive strength is provided as background. Second, the experimental program and test results are described in detail. Third, implications for plant operations in lifting and handling are discussed. Fourth, the detailed conclusions and recommendations are presented.

The overall findings of this work within the constraints listed above are 1) the tensile strength of early age concrete rises faster than the compressive strength and 2) the pullout strength of the inserts exceed the theoretical capacity predicted by the ACI 318-08 Appendix D and the PCI Design Handbook 6th Ed. equations. These findings imply that the strength prediction for lifting inserts is valid in concrete with a compressive strength as low as 1000 psi; however, attention to detail, plant stripping and handling practice, unique characteristics of specialty inserts, and validation that the actual strength of the concrete meets or exceeds the specified strength remain critical for safe operations.
Table of Contents

EXECUTIVE SUMMARY ........................................................................................................ i
Table of Contents .................................................................................................................. iv
List of Figures ....................................................................................................................... vi
List of Tables ....................................................................................................................... vi
List of Test Photographs after breakout ............................................................................ vii
Abstract ............................................................................................................................... viii
Key Words ............................................................................................................................ viii
1.0 INTRODUCTION ............................................................................................................... 1
  1.1 Background ................................................................................................................... 1
  1.2 Research Significance ................................................................................................. 2
  1.3 Organization ................................................................................................................. 2
2.0 Literature Review ............................................................................................................. 3
  2.1 Breakout Strength of Headed Stud Inserts ................................................................. 3
  2.1.1 Development of Breakout Strength Design for Connections in Precast and In-Situ Concrete ........................................................................................................... 3
  2.1.2 Strength in Shear and Tension of Cast-in-Place Anchor Bolts ................................... 4
  2.1.3 Headed Studs – Embedded in Concrete and Loaded in Tension ............................... 5
  2.1.4 Headed Anchor Behavior in Tension ...................................................................... 7
  2.1.5 Headed steel stud anchors in composite structures - Tension and interaction ....... 8
  2.2 Early Age Concrete Tensile vs. Compressive Strength ................................................ 9
  2.2.1 Properties of Concrete - Neville ............................................................................. 9
  2.2.2 Properties of concrete - Mindess, Young and Darwin ............................................ 12
  2.2.3 Splitting Tensile Strength and Compressive Strength Relationship at Early Ages .. 12
  2.2.4 Tensile Strength of Low, Medium, and High-Strength Concretes at Early Ages .... 13
  2.2.5 Direct tension test and tensile strain capacity of concrete at early age ............... 14
  2.2.6 PCI Design Handbook (2004) .............................................................................. 15
  2.3 Literature Review Conclusions .................................................................................... 16
3.0 Test Program .................................................................................................................. 18
  3.1 Test objective ............................................................................................................... 18
  3.2 Test Specimens .......................................................................................................... 18
  3.3 Loading ......................................................................................................................... 22
  3.4 Testing Program ......................................................................................................... 23
4.0 Results ............................................................................................................................ 26
List of Figures

Figure 2-1: Normalized strength gains for early age concrete 16
Figure 3-1: Stud Assembly 19
Figure 3-2: Stud assembly loaded in the Instron List of Tables 19
Figure 3-3: Top-loading attachment applied 19
Figure 3-4: Milled stud shaft loaded in grips 20
Figure 3-5: Schematic of concrete block dimensions 21
Figure 3-6: Concrete Form 21
Figure 3-7: Specimens during curing 21
Figure 3-8: Tripod Loading Frame 22
Figure 3-9: Beam Loading Frame 22
Figure 3-10: Cylinder hammer tests 23
Figure 3-11: Typical breakout failure pattern at 16 hours 25
Figure 4-1: Test-to-predicted ratios of first series of tests 28
Figure 4-2: Test-to-predicted ratios for second series of tests 29
Figure 4-3: Split Tensile to Compressive Strength Comparison 30
Figure 4-4: Split Tensile to Compressive Strength Comparison using Teychenne (1954) 31
Figure 4-5: Comparison of best fit and current equations 33
Figure 4-6: Headed studs after testing 34
Figure 4-7: Insert deflection as a function of time 36
Figure 4-8: Load application as a function of time 36
Figure 4-9: Insert deflection as a function of time 37
Figure 4-10: Insert load as a function of time 37
Figure 5-1: Comparison of Minimum of Embedment Depth at Various Compressive Strengths 38
Figure 5-2: Sensitivity to Steel Yield Stress 39

Conclusions or recommendations in this report are the opinions of the authors. PCI assumes no responsibility for the interpretation or application of the information contained herein.
List of Tables

Table-2-1: Portland Concrete Association Tensile and Compressive Strength 11
Table 2-2 Summary of Tensile Strength Models 17
Table 4-1 Comparison of Predicted Breakout Strength and Test Results for Test Series 1 27
Table 4-2 Comparison of Predicted and Test Results for Test Series 2 27
Table 4-3 Stud Assembly Failure Loads 34
Table A-1: Compressive Cylinder Strengths for First Series Tests 44
Table A-2: Splitting Tensile Cylinder Strengths for First Series of Tests 44
Table A-3: Compressive Cylinder Strengths for Second Series Tests 44
Table A-4: Splitting Tensile Cylinder Strengths for Second Series of Tests 44
Table A-5: Ultimate Failure Loads for First Series Tests 45
Table A-6: Ultimate Failure Loads for Second Series Tests 46
Table A-1: Impact Hammer Results for Series 1 12 Hour Tests 47
Table A- 2: Impact Hammer Results for Series 1 16 Hour Tests 47
Table A-3: Impact Hammer Results for Series 1 20 Hour Tests 47
Table A-4: Impact Hammer Results for Series 1 3 Day Tests 47
Table A-5: Impact Hammer Results for Series 1 3 Day Tests 48
Table A-11: Impact Hammer Results for Series 1 28 Day Tests 48
Table A-12: Impact Hammer Results for Series 2 12 Hour Tests 48
Table A-13: Impact Hammer Results for Series 2 16 Hour Tests 48
Table A-14: Impact Hammer Results for Series 2 20 Hour Tests 48
Table A-15: Impact Hammer Results for Series 2 28 Day Cylinder Tests 49
Table A-16: Impact Hammer Results for Series 2 28 Day Cylinder Tests 49

List of Test Photographs after breakout

Figure B-1: First Series 12 Hour Block 50
Figure B-2: First Series 16 Hour Block 50
Figure B-3: First Series 20 Hour Block 51
Figure B-4: First Series 3 Day Block 51
Figure B-5: First Series 8 Day Block 52
Figure B-6: First Series 28 Day Block 52
Figure B-7: Second Series 12 Hour Block 1 53
Figure B-8: Second Series 12 Hour Block 2 53
Figure B-9: Second Series 16 Hour Block 1 54
Figure B-1: Second Series 16 Hour Block 2 54
Figure B-2: Second Series 20 Hour Block 1 55
Figure B- 3: Second Series 20 Hour Block 2 55
Abstract

This report provides theoretical and experimental validation of the use of ACI 318-08 for concrete strengths as low as 7 MPa (1000 psi). While the compressive strength of the concrete is specified for lifting and handling precast elements, the pullout capacity of inserts is tied to the tensile strength of concrete. Theoretical validation is made by examining the gain in tensile and compressive strength of early age concrete and comparing that strength gain to the relative ratio of tensile to compression strength of cured concrete. Experimental validation is accomplished by conducting 78 pullout tests on 13 mm (½ in.) diameter by 75 mm (3 in.) long headed stud assemblies with 16 mm (⅝ in.) threaded ends for attachment cast into the soffit of a 250 mm (10 in.) deep concrete block and tested at an age as low as 12 hours. In all cases, the concrete breakout capacity is greater than the predicted capacity. The higher capacity is consistent with the findings that concrete tensile capacity rises faster that the compression strength at early ages.

The overall findings of this work within the constraints listed above are 1) the tensile strength of early age concrete rises faster than the compressive strength and 2) the pullout strength of the inserts exceed the theoretical capacity predicted by the ACI 318-08 Appendix D and the PCI Design Handbook 6th Ed. equations. These findings imply that the strength prediction for current inserts is valid in concrete with a compressive strength as low as 1000 psi; however, attention to detail, plant stripping and handling practice, unique characteristics of specialty inserts, and validating that the actual strength of the concrete exceeds the specified strength remain critical for safe operations.

Key Words

Early age, concrete, breakout, strength, inserts
CONCRETE BREAKOUT CAPACITY OF CAST-IN-PLACE ANCHORS IN EARLY AGE CONCRETE

1.0 INTRODUCTION

1.1 Background

Headed studs are defined in the *PCI Design Handbook 6th Ed.* as “… a smooth shaft with integral head. These devices are typically welded to a plate or another structural steel shape. The studs may be hand welded, but are more often attached using a stud gun.” Headed studs have been used in composite construction to transfer shear loads between elements and in many cases shear and tension or only tension loads are transferred by the studs. The shape of headed studs resembles structural hex bolts’ shape, but differs notably in their attachment to plates or other structural members. Headed studs are welded to their base plates, but bolts are placed through holes in plates and any fixity comes from tightening of the nut (Hawkins 1987).

*PCI Design Handbook 6th Ed.* provides minimum mechanical properties including tensile and yield strength, elongation and reduction of area; dimensions of headed studs; minimum plate thickness; as well as calculations for predicting failure loads of cast-in-place headed studs. The physical property values of the headed studs are derived from ASTM 1044. The tensile failure loads for the complete insert are based on four possible failure modes: steel yield, concrete breakout, concrete pullout, and concrete side-face blowout. *ACI 318-08* includes an additional splitting failure mode. This report focuses on the concrete tensile breakout failure mode of headed studs as representative of the failure mode strength behavior of inserts used in lifting and handling precast concrete pieces at early ages.

Prediction models for design were developed using mature concrete with minimum concrete compression strength of 17.4 MPa (2500 psi). Inserts used in lifting precast and prestressed concrete and tilt-up construction may see loads applied as early as 18 hours after casting the member while architectural pieces may have stripping and handling loads applied even earlier. There are concerns of failure of lifting 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 if decisions are based on compressive strength, then a premature failure may occur. Third, if the insert is designed for the specified concrete strength rather than the actual early stripping strength, there may be insufficient embedment. Other factors such as suction resulting from stripping and removal of a piece from the form and dynamic striping loads can contribute to early failures. Increases in loads from stripping and handling are not considered in this study but must be considered in plant operations.

1.2 Research Significance

The objective of this research is to determine, theoretically and experimentally, if the current concrete breakout strength models based on concrete compressive strength and embedment depth for mature concrete apply to early age concrete. Specifically, if at early age, the breakout capacity is properly modeled using the compressive strength or whether an early age correction factor is needed.

Concrete breakout failure of inserts in early age concrete is primarily a serviceability or cost issue but may also be a life-safety issue if the piece is suspended when the failure occurs. Breakout failure is influenced by the strength of the concrete, the variability of the concrete strength at early age, and the possible additional loads required to strip the concrete from the form and dynamic effects during handling.

1.3 Organization

The next chapter includes a literature review separated into two sections. The first section examines equations used to predict concrete breakout capacity of headed studs. The second explores the relationship between concrete tensile and compressive strengths. The information gathered in the literature review is used to compare to the data collected during testing.

The test program is discussed following the literature review. A description of the test specimens, testing methods, the instrumentation used during the testing, and references to relevant ASTM standards are presented.

The results of the testing follow the test program. This includes data collected during testing as well as data reduction. Conclusions and recommendations, including consideration for plant operations, are presented.
2.0 Literature Review

2.1 Breakout Strength of Headed Stud Inserts

A review of the published research into the breakout strength of headed studs provides several possible equations to predict breakout capacity. This section presents the evolution of predictive models over time; ultimately leading up to models currently used in the ACI Building Code and in the PCI Design Handbook. The models used for comparison in this paper are presented by Anderson, Tureyen, & Meinheit (2007) and in the PCI Design Handbook 6th Ed (2004).

The PCI Design Handbook 6th Ed and ACI 318 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 the equation:

\[ F_{5\%} = F_m(1 - K\nu) \]

where

- \( F_{5\%} \) is the 5% fractile or characteristic capacity
- \( F_m \) is the mean failure capacity
- \( K \) values are the factors for one-sided tolerance limits for normal distributions
- \( \nu \) is the coefficient of variation.

This information is also used for comparison to the equations presented in the ACI code and the PCI Handbook.

2.1.1 Development of Breakout Strength Design for Connections in Precast and In-Situ Concrete

Courtois (1969) described problems with insert testing using small blocks that resulted in flexural splitting failure of the block before the ultimate capacity of the insert was reached. Additionally, tests conducted at this time showed both concrete compressive strength and embedment depth to be important parameters for determining pullout capacity. Correlation is postulated on a shear cone breakout model.

A shear cone breakout failure occurs where a concrete cone defined by the depth of embedment of the insert fails in shear. This type of breakout failure was presented as a simple model
and was used in early editions of the PCI Handbook. An equation for strength of a shear cone breakout failure in direct tension is given as:

\[ P_I = (A_c)K\sqrt{f'_c} \quad N, \text{mm}^2, \text{MPa (lbf, in}^2, \text{psi)} \quad \text{Eq. 2-2} \]

where

\( K \) is a function of mix proportions and varies from 0.15 (1.8) for low strength concrete as used in lock and dam construction with 75 and 150 mm (3 and 6 in.) size aggregate, 112 to 167 kg cement per m\(^3\) (188 to 282 lb cement per cu. yd.) and various percentages of fly ash replacement to 0.35 (4.2) for normal strength concrete.

\( A_c \) is the pseudo-shear cone area on the concrete surface, (located at \( d/2 \) from the stud head where \( d \) is embedment depth).

This approach is similar to punching shear calculations for a slab around a column. The variation in the coefficient, \( K \), at various strengths seems to be for mature concrete of different strengths, but may be useful for early age concrete as well. However, Courtois identifies split cylinder tests as more informative than compression cylinder results and suggests 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.”

### 2.1.2 Strength in Shear and Tension of Cast-in-Place Anchor Bolts

Hawkins (1987) conducted 12 tests on 25 mm (1 in.) diameter anchor bolt breakout specimens in 20 MPa (3000 psi) concrete. Embedment depth varied between 75, 125, and 175 mm (3, 5 and 7 in.). The washer diameter below the bolt varied between 50, 100, 150 mm (2, 4, and 6 in.). The thickness of this washer also varied as either 16 or 22 mm (5/8 or 7/8 in.). Nine specimens were 450 mm wide by 450 mm long by 225 mm deep (18 in. by 18 in. by 9 in.) and reinforced near the edges. The other 3 specimens were 1150 mm wide by 1150 mm long by 175 mm deep (46 in. by 46 in. by 7 in.) and also reinforced near the edges.

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

Only three specimens showed cone breakout failures; one from the smaller block tests and two from the larger block tests. The reason presented for this is attributed to the moment generated by the testing frame inducing flexural cracking in the concrete causing radial cracking failure before cone breakout failure can be reached. This is similar to the problems listed by Courtois (1969), that is a majority of the failures were splitting of the concrete.

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

2.1.3 Headed Studs – Embedded in Concrete and Loaded in Tension

Sattler (1962) reported the following for pure tension strength of connectors and headed studs:

\[
\max T = 0.15 \cdot \pi \cdot d_2 \cdot h_s \cdot \beta_w \leq \frac{(\pi \cdot d_1^2)}{4} \cdot \sigma_y \quad N, \text{lb} \text{f} \quad \text{Eq. 2-3}
\]

where:

- \(d_1\) = diameter of shaft (mm, in.)
- \(d_2\) = diameter of stud head (mm, in.)
- \(h_s\) = embedded shaft length (mm, in.)
- \(\beta_w\) = cube strength of concrete after 28 days (N/mm\(^2\), psi)
- \(\sigma_y\) = guaranteed yield strength of stud material (which by ASTM A-1044 gives a guaranteed value of 350 N/mm\(^2\), (51,000 psi))

Sattler proposed a global safety factor, equivalent to a load factor divided by a corresponding strength reduction factor, of 2.0 to derive allowable load:

\[
T_{alt} = \frac{(\max T)}{2.0} = 0.15 \cdot \pi \cdot d_2 \cdot h_s \cdot \beta_w / 2 \quad N, \text{lb} \text{f} \quad \text{Eq. 2-4}
\]
Slater’s work did not address spacing requirements of stud groups, edge distance allowances, or anchoring to concrete in the tensile zone of a member where cracks could exist.

Bode and Roik (1987) recommend a formula for the breakout strength of a single stud loaded in tension:

\[ \max T = F \times \sqrt{h_s (h_s + d_2) / \beta_W} \quad N, lbf \]

where:

- \( max T \) is the average tension strength of the stud.
- \( F \) is calculated to be 145.0 \((N/mm)^{0.5}\) (10.96 \((lb/in)^{0.5}\)) for average values.
- \( h_s \) = embedded shaft length (mm, in.)
- \( d_2 \) = diameter of stud head (mm, in.)
- \( \beta_W \) = cube strength of concrete after 28 days \((N/mm^2, psi)\)

For design Bode-Roik recommend the following:

\[ \max T = F \times \sqrt{h_s (h_s + d_2) / \beta_{WN}} \quad N, lbf \]

where:

- \( \beta_{WN} \) = specified cube strength of concrete at 28 days \((N/mm^2, psi)\)
- \( F \) is 8.90 (117.8) for a 5% fractile.
- \( F \) is 8.25 (109.1) when taking into account that concrete is mixed, poured, and compacted better in the laboratory than on a construction site.
- \( F \) reduces to 2.75 (36.38) when a 3.0 global safety factor is applied to the breakout strength.

A maximum length of \( h = 175 \) mm (7 in.) is recommended due to lack of information on the strength of longer studs.

It is also noted that for shorter studs, 50 mm (2 in.) in total length after welding, the standard deviation is significantly greater than for longer studs because of the non-homogeneous 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 studs of shorter length. No further recommendations on other lengths are discussed.
To create a single solution where the head diameter is not available, Bode and Roik recommend the coefficient $F$ should be changed as follows:

$$\text{max} \, T = 15.60 \, h_s^{3/2} \cdot \sqrt{\beta_W} \quad N \quad \text{Eq. 2-7a}$$

$$T_{5\% \text{ fractile}} = 11.46 \, h_s^{3/2} \cdot \sqrt{\beta_{WN}} \quad N \quad \text{Eq. 2-7b}$$

$$T_{alt} = 3.80 \, h_s^{3/2} \cdot \sqrt{\beta_{WN}} \quad N \quad \text{Eq. 2-7c}$$

The coefficients are 206.4, 151.6, and 50.5 respectively in US customary units. The allowable design load, $T_{alt}$ does not require ductile behavior. However, making longer studs with thinner shafts is more desirable because the global safety factor for concrete failure is 3.0 vs. 1.7 for steel yielding.

### 2.1.4 Headed Anchor Behavior in Tension

The University of Texas at Austin assembled a database on tension testing when the concrete capacity design method was in development, (Meinheit, Anderson, and Tureyen 2007). Most of this data used 200 mm (4 in.) cube crushing strength, $f_{cc200}$. Using this information, the authors concluded that no additional tension testing was needed to be done to describe the behavioral characteristics of welded headed stud anchors loaded in direct tension.

The strength prediction takes the general form of:

$$N = a \cdot f'_c \cdot b \cdot h_{ef}^\beta$$

Regression analysis shows an excellent concrete breakout prediction equation using the variable $f'_c$ and $h_{ef}$. The correlation coefficient ($r^2$) value is nearly 0.98. The regression analysis shows the magnitude of the exponent $\alpha$ to consistently be about ½ for $f'_c$ and $\beta$ to be 3/2 for $h_{ef}$. Adding additional variables of stud head diameter and stud shaft diameter did not significantly alter the $r^2$ value. These equations were developed using $f'_{c_cylinder} = 0.85 \, f_{cc200}$ for a correlation between cubes and cylinders.

**ACI 318-08 Appendix D** assumes and average prediction equation for headed cast–in-place anchors in uncracked concrete to equal:

$$N_{u,ACI \text{ headed}} = 16.7 \, \sqrt{f'_c(h_{ef})^{1.5}} \quad N \quad \text{Eq. 2-9a}$$

$$N_{u,ACI \text{ headed}} = 40 \, f'_c(h_{ef})^{1.5} \quad \text{lb} \quad \text{Eq. 2-9b}$$
This equation is used throughout this paper for comparison to the data collected. This is equivalent to the equation presented in ACI 318-08 and PCI Design Handbook, 6th Ed. This gives a test to predicted ratio: 0.992. When the CEB-FIB MC90 strength conversion factor, $f'_{c} = \gamma f_{c200}$ where $\gamma = 0.85$, is used the test to predicted ratio increases to 1.108. Using the concrete strength conversion values from CEB-FIB the average prediction equation is:

$$N_{u,\text{WJE/PCI headed}} = 18.5 \sqrt{f'_{c}(h_{ef})^{1.5}} \ N \quad \text{Eq. 2-10a}$$

$$N_{u,\text{WJE/PCI headed}} = 44 \sqrt{f'_{c}(h_{ef})^{1.5}} \ lbf \quad \text{Eq. 2-10b}$$

Another alternative equation is presented for anchors with deep embedment, i.e., $h_{ef} > 300$ mm (12 in.):

$$N_{u,\text{ACI headed alternative}} = 6.83 \sqrt{f_{c}(h_{ef})^{5/3}} \ N \quad \text{Eq. 2-11a}$$

$$N_{u,\text{ACI headed alternative}} = 28 \sqrt{f_{c}(h_{ef})^{5/3}} \ lbf \quad \text{Eq. 2-11b}$$

The PCI Design Handbook, 5th Ed. still used information based on Courtois’ work, indicating that capacity is proportional to $h_{ef}^{2}$, which over predicts the test data by 16 to 30% depending on the strength conversion. At shallower depths, often used in the precasting industry, the PCI equation is more conservative than the ACI equation.

From regression analysis of tension data assuming $f'_{c} = 0.85 f_{c200}$ the best fit equation is:

$$N_{u,\text{WJE/PCI}} = 16.47(f'_{c})^{0.523}(h_{ef})^{1.504} \ N \quad \text{Eq. 2-12a}$$

$$N_{u,\text{WJE/PCI}} = 35.57(f'_{c})^{0.523}(h_{ef})^{1.504} \ lbf \quad \text{Eq. 2-12b}$$

Anderson, Tureyen, and Meinheit describe the conversion factor for cubes of different sizes as:

$$f_{cube,150} = \frac{1}{0.95} f_{cube,200} = 1.05 f_{cube,200} \quad \text{Eq. 2-13}$$

2.1.5 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 becoming more prevalent, such as infill walls, coupling beams, connections to composite columns, or composite column bases. Pallares and Hajjar
(2010) note that the most advanced information on headed studs is included in the *PCI Design Handbook, 6th Ed.* and *ACI 318-08*. Only tests with no edge effects and concrete strength greater than 20 MPa (3 ksi) were considered for composite construction. A 20 MPa (3 ksi) limit is the minimum strength permitted by AISC for composite structures. Pallares and Hajjar concluded concrete breakout is prevented if $h_{at} > 7.5d$. This gives values similar to the 8 to 10 values proposed by Hawkins (1987).

2.2 Early Age Concrete Tensile vs. Compressive Strength

Research into the correlation between tensile and compressive strength is important for 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.

The published research is inconsistent in reporting whether the mature compressive strength or the age of the concrete that determines a concrete’s tensile strength. The relationship between the two is that as the compressive strength increases so too does the tensile strength, but at a decreasing rate. This relationship is most commonly described as a ratio of strengths at a range of values or a curvilinear fit of data. Several alternative predictive models for determining tensile strength are compared to the values obtained in this research.

2.2.1 Properties of Concrete - Neville

A.M. Neville’s (1973, 1996) in *Properties of Concrete* explains the difficulties in developing a direct relationship between concrete compressive and tensile strengths. Neville lists at least six different factors that affect the ratio of concrete tensile stress to compressive stress ratio, $f_t/f_c$. These factors include the level of strength, the coarse and fine aggregate composition, the age of the specimens, the curing, air entrainment, and density of the concrete. The aggregate can affect the strength because crushed coarse aggregate increases flexural strength. The fine aggregate affects the ratio based on the aggregate grading and possibly 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. 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 very high ratios of $f_t/f_c$ at very low strength, but at higher strengths the ratio is similar to normalweight concrete. Drying may reduce the ratio for lightweight concrete by 20 percent.

The first edition of *Properties of Concrete* (Neville 1973) offers four alternative equations, Teychenné, the Comité Européen du Béton (CEB), University of Illinois, and PCA, for determining tensile strength from compressive strength along with a table showing ratios and a graph plotting tensile and compressive strengths.

The equation presented by Teychenné (1954) comparing 100 mm (4 in.) cube compressive strength and 100 X 100 X 400 mm (4 x 4 x 16 in.) beams loaded in three-point bending to determine tensile strength is as follows:

$$f_t = 0.69 \times f_c^{\frac{1}{2}} \quad \text{MPa} \quad \text{Eq. 2-15a}$$

$$f_t = 8.3 \times f_c^{\frac{1}{2}} \quad \text{psi} \quad \text{Eq. 2-15b}$$

The coefficient, 0.69, varies from 0.52 (6.2) for some gravels up to 0.86 (10.4) for crushed rock.

The Comité Européen du Béton assumes:

$$f_t = 0.79 \times f_c^{\frac{1}{2}} \quad \text{MPa} \quad \text{Eq. 2-16a}$$

$$f_t = 9.5 \times f_c^{\frac{1}{2}} \quad \text{psi} \quad \text{Eq. 2-16b}$$

to relate cylinder compressive strength to modulus of rupture.

The University of Illinois suggests

$$f_t = \frac{3000}{(4 + \frac{12000}{f_c})} \quad \text{Eq. 2-17}$$

where $f_c$ is determined from cylinders and $f_t$ is the modulus of rupture.

The Portland Cement Association gives a table to describe the relationship between tensile and compressive strength. The overall trend in the table indicates a higher tensile strength for lower strength concrete.
Table-2-1: Portland Concrete Association Tensile and Compressive Strength

<table>
<thead>
<tr>
<th>Compressive Strength of Cylinders</th>
<th>Strength Ratio</th>
<th>Modulus of rupture to compressive strength</th>
<th>Direct tensile strength to compressive strength</th>
<th>Direct tensile strength to modulus of rupture</th>
</tr>
</thead>
<tbody>
<tr>
<td>MN/m²</td>
<td>lb/in²</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>1000</td>
<td>0.23</td>
<td>0.11</td>
<td>0.48</td>
</tr>
<tr>
<td>15</td>
<td>2000</td>
<td>0.19</td>
<td>0.10</td>
<td>0.53</td>
</tr>
<tr>
<td>20</td>
<td>3000</td>
<td>0.16</td>
<td>0.09</td>
<td>0.57</td>
</tr>
<tr>
<td>30</td>
<td>4000</td>
<td>0.15</td>
<td>0.09</td>
<td>0.59</td>
</tr>
<tr>
<td>35</td>
<td>5000</td>
<td>0.14</td>
<td>0.08</td>
<td>0.59</td>
</tr>
<tr>
<td>40</td>
<td>6000</td>
<td>0.13</td>
<td>0.08</td>
<td>0.60</td>
</tr>
<tr>
<td>50</td>
<td>7000</td>
<td>0.12</td>
<td>0.07</td>
<td>0.61</td>
</tr>
<tr>
<td>55</td>
<td>8000</td>
<td>0.12</td>
<td>0.07</td>
<td>0.62</td>
</tr>
<tr>
<td>65</td>
<td>9000</td>
<td>0.11</td>
<td>0.07</td>
<td>0.63</td>
</tr>
</tbody>
</table>

A graph is also provided in Neville showing the relations found by Walker and Bloem. This graph plots both moduli of rupture as well as splitting tensile strength to compressive strength. The graph is useful for quick comparison, but the exact relationship proposed is not provided.

The difficulty with using the values presented in the first edition of Properties of Concrete (Neville 1973) is the need for an additional conversion to determine the ratio $f_t/f_c$ when comparing cylinder compressive strength and split cylinder strength as the measure of tension strength. The Fourth Edition of Neville’s book (1996) provides more values for this correlation that are similar to current forms in the ACI Code. Two equations are provided to compare splitting tensile strength to compressive strength.

Raphael (1984) suggests:

$$f_t = 0.3(f_c)^{2/3} \quad MPa$$  \hspace{1cm} Eq. 2-18a

$$f_t = 1.7(f_c)^{2/3} \quad psi$$  \hspace{1cm} Eq. 2-18b

Oluokun (1991) suggests:

$$f_t = 1.4(f_c)^{0.7} \quad MPa$$  \hspace{1cm} Eq. 2-19a

$$f_t = 0.2(f_c)^{0.7} \quad psi$$  \hspace{1cm} Eq. 2-19b

and the British Code of Practice BS 8007:1987 gives:

$$f_t = 0.12(f_c)^{0.7} \quad MPa$$  \hspace{1cm} Eq. 2-20a
\[ f_t = 0.84(f_c)^{0.7} \text{ psi} \]  \hspace{1cm} \text{Eq. 2-20b}

where compressive strength is determined on cubes and \( f_t \) is direct tensile strength.

### 2.2.2 Properties of concrete - Mindess, Young and Darwin

Concrete by Mindess, Young, and Darwin (2003) present the same factors that affect 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 (\( f'_{sp} \)) to compressive strength (\( f'_{c} \)) is usually in the 0.08 to 0.14 \( f'_{c} \) range. While the ratios of direct tensile strength to compressive strength is about 0.07 to 0.11 \( f'_{c} \) and the ratio of modulus of rupture to compressive strength is about 0.11 to 0.23 \( f'_{c} \).

The equations used by ACI Committee 318 are presented as a lower bound

\[ f'_{sp} = 0.48 \sqrt{f'_{c}} \text{ MPa} \]  \hspace{1cm} \text{Eq. 2-21a}

\[ f'_{sp} = 6 \sqrt{f'_{c}} \text{ lb/in}^2 \]  \hspace{1cm} \text{Eq. 2-21b}

And the equations proposed by ACI Committee 363

\[ f'_{sp} = 0.59 \sqrt{f'_{c}} \text{ MPa} \]  \hspace{1cm} \text{Eq. 2-22a}

\[ f'_{sp} = 7.4 \sqrt{f'_{c}} \text{ psi} \]  \hspace{1cm} \text{Eq. 2-22b}

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

\[ f'_{sp} = 0.305 * f'_{c}^{0.55} \text{ MPa} \]  \hspace{1cm} \text{Eq. 2-23a}

\[ f'_{sp} = 4.34 * f'_{c}^{0.55} \text{ psi} \]  \hspace{1cm} \text{Eq. 2-23b}

The ACI Building Code uses an exponent of 0.5. This best fit equation is in general agreement with the exponent proposed by ACI.

### 2.2.3 Splitting Tensile Strength and Compressive Strength Relationship at Early Ages

The prior studies are unclear whether the low strength concrete is fully cured or if the low strength data is at an early age. Oluokun et al. (1991) claim that the ACI exponent of \( \frac{1}{2} \) on \( f'_{c} \) is not valid for early age concrete. The mixes tested included 3 laboratory prepared test mixes and one precast prestressed sample from a concrete producer. The 28-day compressive strengths ranged...
from 4000 to 9000 psi for the four mixes. Standard 150 x 300 mm (6 x 12 in.) cylinder molds were cast from a single batch for each series of testing. The course aggregate for all mixes was 90 to 100% retained on a 19 mm (¾ in.) sieve with 100% less than 25 mm (1 in.). The fine aggregate was a manufactured crushed limestone.

The cylinders were covered with wet burlap and polyethylene for 20 to 24 hours. The specimens were then stripped and transferred to a standard moist room. Tests were conducted at 6 and 12 hr, and 1, 2, 3, 7, and 28 days. Compressive capacity was measured following standard procedures and the tensile capacity was measured using the split-cylinder test.

For specimens tested at an age greater than 6 hours and if \( f'_{c} \) is greater than 5 MPa (1000 psi) the relationship between tensile splitting strength and compressive strength is found to be:

\[
f_t = 29.8 \times f'_c^{0.79} \quad \text{MPa} \quad \text{Eq. 2-24a}
\]

\[
f_t = 0.584 \times f'_c^{0.79} \quad \text{psi} \quad \text{Eq. 2-24b}
\]

This equation includes the fit for the data presented in the research as well as values obtained from 5 other research projects.

For compressive strengths less than 5 MPa (1000 psi) the relationship was found to be:

\[
f_t = 18.4 \times f'_c^{0.6} \quad \text{MPa} \quad \text{Eq. 2-25a}
\]

\[
f_t = 0.928 \times f'_c^{0.6} \quad \text{psi} \quad \text{Eq. 2-25b}
\]

2.2.4 Tensile Strength of Low, Medium, and High-Strength Concretes at Early Ages

Khan et al. (1996) selected modulus of rupture as the measure of tensile strength. Three different curing conditions were investigated including temperature-matched curing, sealed curing, and air-dried curing. The three concretes tested consisted of a nominal 30, 70, and 100 MPa (4300, 10150, 14500 psi) compressive strength at 28-days.

The concretes use ASTM Type I cement. The 70 and 100 MPa mixes contain 7-8 and 8-9 percent silica fume, respectively. The coarse aggregate was a crushed limestone with 19 mm (3/4 in.) maximum for the 30 MPa, 13 mm (½ in.) maximum for the 70 MPa, and 6 mm (¼ in.) maximum for the 100 MPa concretes. The fine aggregate was river sand with a fineness modulus of 2.55 for all three mixes.
A control cylinder, 15 additional cylinders, and 11-100 x 100 x 400 mm (4 x 4 x 16 in.) flexural beam specimens were cast. Modulus of rupture values were calculated by using a 4-point bending test with a span length of 300 mm (12 in.) and 100 mm (4 in.) constant moment section. The specimens were tested at frequent intervals over 3 days.

The concretes subjected to temperature-matched curing typically resulted in much higher flexural strengths than the sealed and air-dried specimens. The sealed specimens showed higher flexural strength than the air-dried specimens.

Khan et al. (1996) concludes ACI 318 overestimates modulus of rupture strength for concrete compressive strengths less than 15 MPa (2175 psi) and underestimates above 15 MPa. ACI 363 overestimates the modulus of rupture strength for nearly all types of concrete.

For temperature matched curing, the suggested equation for relating modulus of rupture to compressive strength is given by:

\[
f_r(t) = 0.085 \cdot f'_c(t) \quad MPa
\]

Eq. 2-26a

\[
f_r(t) = 12.3 \cdot f'_c(t) \quad psi
\]

Eq. 2-26b

For sealed curing the equation is presented as:

\[
f_r(t) = 0.4 \cdot (f'_c(t))^{2/3} \quad MPa
\]

Eq. 2-27a

\[
f_r(t) = 11 \cdot (f'_c(t))^{2/3} \quad psi
\]

Eq. 2-27b

For air dried curing the equation is presented as:

\[
f_r(t) = 0.38 \cdot (f'_c(t))^{2/3} \quad MPa
\]

Eq. 2-28a

\[
f_r(t) = 10.5 \cdot (f'_c(t))^{2/3} \quad psi
\]

Eq. 2-28b

High strength concrete (100 MPa) has an initial retardation period, in which the flexural strength gain is lower than low strength concrete. After this initial retardation period, the flexural strength gains proportionally with compressive strength. The high strength concrete was also found to be more sensitive to drying methods.

2.2.5 Direct tension test and tensile strain capacity of concrete at early age

One hundred eighteen 100 x 100 x 500 mm (4 x 4 x 20 in.) tensile specimens were cast using various concrete mix designs (Swaddiwudhiponga et al. 2003). Twenty specimens were made for
each of the ordinary Portland cement concretes (OPC) and 9 specimens for each of the 70% ground granulated blast furnace slag replacement concrete (GGBFS) and the 30% pulverized fly ash replacement concrete (PFA). Fifteen of the 20 specimens were tested at different 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 2 were tested at 30 με/min. The 9 specimens of PFA and GGBFS concretes were tested at 7, 14, and 28 days at a strain rate of 5 με/min, three at each age. The compressive strength was measured using three 100-mm (4 in.) cubes at each age. Embedded steel bars were used to apply the tensile loading.

The tensile to compressive strength ratio decreases as concrete matures. This implies that the gain on tensile strength is smaller than the increase in compressive strength. The tensile to compressive strength ratio varies from 0.04 to 0.10 \( f'_c \). This is in general agreement with Neville (1973) when the cube strength is converted to cylinder strength. The overall finding of this study is that the tensile strength increases faster than the compressive strength at early ages for a wide variety of concrete mixtures.

The tensile to compressive strength ratio is higher at lower compressive strength and decreases as the compressive strength of OPC concrete rises. The different mix proportions of OPC do not affect this relationship at constant compressive strength. The PFA concrete shows a similar relationship, but GGBFS concrete has a smaller ratio of tensile to compressive strength.

2.2.6 PCI Design Handbook (2004)

From ACI 318-02 the tensile strength is described by the modulus of rupture as:

\[
f_r = 90.4 \lambda \sqrt{f_c} \quad \text{MPa} \quad \text{Eq. 2-29a}
\]

\[
f_r = 7.5 \lambda \sqrt{f'_c} \quad \text{psi} \quad \text{Eq. 2-29b}
\]

Additionally, if the splitting tensile strength, \( f_{st} \), is specified, ACI 318 permits the value of \( f_{st}/80.8 \) \( (f_{st}/6.7) \) to be substituted for \( \lambda \sqrt{f_c} \).

Conclusions or recommendations in this report are the opinions of the authors. PCI assumes no responsibility for the interpretation or application of the information contained herein.
2.3 Literature Review Conclusions

The current PCI and ACI equations for predicting breakout strength of headed anchors work well for mature concrete. This prediction model along with the model developed by Anderson, Tureyen and Meinheit (2007) is used for comparison in this report.

Figure 2-1 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 35 MPa (5000 psi) concrete. The ACI 318 (Eq. 2-21) and the Oluokun equation for early age concrete (Eq. 2-24) are compared. Oluokun predicts a lower initial tensile strength than the ACI formulation, which is consistent with Khan’s findings. In both cases, the tensile capacity gain is higher than the compressive capacity. Thus, experimental validation should indicate strength gains on the order of 30-50 percent based on Oluokun’s hypothesis.

Tensile strength gains faster than compressive strength at early concrete age when compared to the corresponding strength gains of mature concrete. Further, the behavior is consistent for a wide range of mixtures, admixtures and cementitious materials. Prediction methods used by ACI, and by extension PCI, reflect the increased tensile strength at compressive strengths greater than 15 MPa (2175 psi). The model equations are summarized in Table 2-2.

![Figure 2-1: Normalized strength gains for early age concrete](image-url)
Table 2-2 Summary of Tensile Strength Models

<table>
<thead>
<tr>
<th>Source</th>
<th>Equation</th>
<th>$f_t$ (MPa)</th>
<th>$f_t$ (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Teychenné (beams)</td>
<td>2-15</td>
<td>$f_t = 0.69 \times f_c^{1/2}$</td>
<td>$f_t = 8.3 \times f_c^{1/2}$</td>
</tr>
<tr>
<td>CEB</td>
<td>2-16</td>
<td>$f_t = 0.79 \times f_c^{1/2}$</td>
<td>$f_t = 9.5 \times f_c^{1/2}$</td>
</tr>
<tr>
<td>Univ. of Ill.</td>
<td>2-17</td>
<td>$f_t = 3000/(4 + \frac{12000}{f_c})$</td>
<td></td>
</tr>
<tr>
<td>Raphael</td>
<td>2-18</td>
<td>$f_t = 0.3(f_c)^{2/3}$</td>
<td>$f_t = 1.7(f_c)^{2/3}$</td>
</tr>
<tr>
<td>Oluokun</td>
<td>2-19</td>
<td>$f_t = 0.2(f_c)^{0.7}$</td>
<td>$f_t = 1.4(f_c)^{0.7}$</td>
</tr>
<tr>
<td>BS 8007</td>
<td>2-20</td>
<td>$f_t = 0.12(f_c)^{0.7}$</td>
<td>$f_t = 0.84(f_c)^{0.7}$</td>
</tr>
<tr>
<td>ACI 318</td>
<td>2-21</td>
<td>$f_{sp} = 0.48 \sqrt{f_c}$</td>
<td>$f_{sp} = 6 \sqrt{f_c}$</td>
</tr>
<tr>
<td>ACI 363</td>
<td>2-22</td>
<td>$f_{sp} = 0.59 \sqrt{f_c}$</td>
<td>$f_{sp} = 7.4 \sqrt{f_c}$</td>
</tr>
<tr>
<td>Mindess, Young and Darwin best fit</td>
<td>2-23</td>
<td>$f_{sp} = 0.305 \times f_c^{0.55}$</td>
<td>$f_{sp} = 4.34 \times f_c^{0.55}$</td>
</tr>
<tr>
<td>Oluokun&gt;6hr and &gt;5MPa</td>
<td>2-24</td>
<td>$f_t = 0.584 \times f_c^{0.79}$</td>
<td>$f_t = 29.8 \times f_c^{0.79}$</td>
</tr>
<tr>
<td>Oluokun&lt;5MPa</td>
<td>2-25</td>
<td>$f_t = 18.4 \times f_c^{0.6}$</td>
<td>$f_t = 0.928 \times f_c^{0.6}$</td>
</tr>
<tr>
<td>Khan (open)</td>
<td>2-26</td>
<td>$f_r(t) = 0.085 \times f_c'(t)$</td>
<td>$f_r(t) = 12.3 \times f_c'(t)$</td>
</tr>
<tr>
<td>Khan (sealed)</td>
<td>2-27</td>
<td>$f_r(t) = 0.4 \times (f_c'(t))^{2/3}$</td>
<td>$f_r(t) = 11 \times (f_c'(t))^{2/3}$</td>
</tr>
<tr>
<td>Khan (dry cured)</td>
<td>2-28</td>
<td>$f_r(t) = 0.38 \times (f_c'(t))^{2/3}$</td>
<td>$f_r(t) = 10.5 \times (f_c'(t))^{2/3}$</td>
</tr>
</tbody>
</table>
3.0 Test Program

3.1 Test objective

The equations in the PCI Design Handbook, 6th Ed. for concrete breakout of headed studs are based on mature concrete values. In the precast industry loads are applied before concrete reaches full maturity. There is little research on the effects of early concrete age on breakout strength. This report evaluates early concrete age, to determine if the development of tensile capacity is sufficient to capture the lower bound strength and if the mature concrete equations can be applied directly to early age concrete. The test protocol follows ASTM E 488 (2009) and is conducted in uncracked concrete, as would be expected for striping and handling of precast concrete panels. ACI 355.2 criteria are for testing inserts in cracked concrete and are not applicable to this test program.

3.2 Test Specimens

3.2.1 Stud Assemblies

Headed studs with a nominal length of 75 mm (3 in.) and a shank diameter of 13 mm (½ in.) were used throughout testing. These studs were stud welded directly to a 75 x 75 x 6 mm (3 x 3 x ¼ in.) plate meeting the requirements for plate thickness set forth in the PCI Design Handbook, 6th Ed. On the other side of the plate a 16 mm (5/8 in.) diameter threaded rod 75 mm (3 in.) long was conventionally welded to the plate, Figure 3-1. The threaded rod allows 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 form and allow for easy form removal even if the studs shifted during the placing and consolidation of the concrete.

The embedment depth $h_{ef}$ is nominally 75 mm (3 in.) but must be corrected for the weld burnoff and the plate thickness. These corrections led to a $h_{ef}$ value of 71.5 mm (2.813 in.). Approximately half of the assemblies were measured and found to be within 3 mm (0.12 in.) of this value. A nominal actual embedment length of 71.5 mm (2.81 in.) is used for data reduction. The measured burnoff lengths are consistent with the recommendations of the PCI Design Handbook.

3.2.1.1 Quality Control of Stud Assemblies

The stud assemblies were fabricated by Rocky Mountain Prestress to resemble assemblies used in practice. This led to some of the studs having either an offset with respect to the threaded rod, not being perpendicular to the plate, or both, Figure 3-1. To assure weld and assembly integrity
during testing, each stud assembly was preloaded to 85% of its specified yield capacity in an Instron 1332 Universal Test machine. Figure 3-2 and Figure 3-3 show an assembly loaded into the Instron without the top attachment and with the top attachment, respectively. Only one assembly failed during this testing. The failure resulted from an incomplete weld around the threaded rod.

Figure 3-1: Stud Assembly

![Image](image1.png)

Figure 3-2: Stud assembly loaded in the Instron

![Image](image2.png)

Figure 3-3: Top loading attachment applied

In addition to proof testing of the assemblies, a single test was conducted on the shaft of one stud. The head and plate were cut off of an assembly from a 12 hour test specimen. The stud shaft
was then milled down to a constant cross section to be loaded in grips in an Instron 1332, Figure 3-4. A strain gauge was attached to the stud shaft and the shaft was loaded to failure.

![Figure 3-4: Milled stud shaft loaded in grips](image)

Three direct tension tests were also conducted on representative stud assemblies. The assemblies were loaded to failure using an Instron 1332 machine. The assemblies were placed in the test machine in the same manner as the stud quality control tests and loaded to failure.

### 3.2.2 Concrete Blocks

For the first set of tests, six blocks were cast measuring 690 x 1675 mm (27 x 66 in.) on the bottom face and 740 x 1675 mm (29 x 66 in.) on the top face by 25 mm (10 in.) thick, Figure 3-5 and Figure 3-6. The side draft allowed for easy form removal. The concrete was a standard Wyoming DoT bridge deck mix provided by the local ready mix plant and had a 28 day specified compressive strength of 35 MPa (5000 psi) and used ASTM C33 size 57 and size 67 crushed coarse aggregate. Ten studs were set in the bottom of each form before placing the concrete.

ASTM E 488 recommends that studs be spaced more than 2·h_{ef} from the edge of the structural member. For spacing between studs, this minimum is doubled to 4·h_{ef}. The studs were spaced 180 mm (7 in.) from edges and 330 mm (13 in.) or 2 ⅜ and 4 ⅝·h_{ef} respectively from each other, thus meeting or exceeding the ASTM guidelines. Additionally, each form had two members across the top to hold lifting inserts. Each insert was spaced 340 mm (13.5 in.) from each end to minimize the moment created during striping and to reduce the possibility of cracking the block while moving it into position for testing, Figure 3-5. The blocks were reinforced with a single #19 (#6) rebar in the center of each block to provide a means of rotating the block once it had been cast. While casting the blocks, 34 - 100 x 200 mm (4 x 8 in.) and 26 - 150 x 300 mm (6 x 12 in.) cylinders
were also cast to determine the concrete strength during testing. After casting, the specimens and cylinders were then covered with plastic to cure, Figure 3-7.

For the 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 210 mm (8.25 in.) and the stud spacing increased to 420 mm (16.5 in.), or 2.75 \( h_{ef} \) and 5.5 \( h_{ef} \), respectively. Twenty 150 x 300 mm (6 x 12 in.) and 20 - 100 x 200 mm (4 x 8 in.) cylinders were cast to determine the strength of the blocks at the time of testing as well as the 28 day strength. The same concrete mixture was used for both sets of specimens. The blocks and cylinders were again covered with plastic to cure.

![Figure 3-5: Schematic of concrete block dimensions](image)

Conclusions or recommendations in this report are the opinions of the authors. PCI assumes no responsibility for the interpretation or application of the information contained herein.
3.3 Loading

Based on the initial calculations of breakout strength an 89 kN (20 kips) load cell was selected to maximize the data collection range. The selection of this load cell required that testing be terminated at 89 kN (20 kips) to prevent damage to the instrumentation. The load cell selection additionally assured that at least one quarter of the full range would be used to record breakout strengths.

A primary difference between the first series of tests and the second series of testing was that a different loading frame was used to conduct the tests. ASTM E 488 recommends the minimum distance from the center of the stud to the nearest point of contact on the loading frame should be no less than twice the 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 150 mm (6 in.), or \( 2 h_{ef} \), Figure 3-8. The frame contacted the concrete with 3-50 x 50 mm (2 x 2 in.) steel plates. This frame was used throughout the first round of testing. Many of the breakout cones extended to the frame contact points, which could interfere with the breakout cone. This led to the design of a larger frame with increased spacing for the second round of testing.

For the second round of tests, a frame used an inside spacing between the reaction supports of 580 mm (23 in.). The frame’s reaction beams were two 300 mm (12 in.) long 50 mm (2 in.) wide steel tubes that were placed along the long edge of the block for each test, Figure 3-9. In all cases in the second round testing the support points were at least \( 6 h_{ef} \) from the insert.

Figure 3-8: Tripod Loading Frame

Figure 3-9: Beam Loading Frame
In both series of tests the load was applied by a 107 kN (12 ton) center-hole ram. The 89.0 kN (20 kips) load cell was attached to a threaded rod which was passed through the center-hole of the ram, which in turn was placed on top of the loading frame. Displacement of the stud relative to the concrete was measured by two potentiometers placed on either side of the insert. The potentiometers were placed on a bridge so that the potentiometers would not be affected by the failure cone or deflection of the loading frame. Two eye bolts and a clevis were used to connect the threaded rod and the insert so there is minimal bending in the loading system.

### 3.4 Testing Program

During the first series of tests, each block was individually de-molded, moved into position for testing and rotated so that the studs were on the top surface of the block two hours prior to the breakout test. One hour prior to testing two cylinders were tested in compression following ASTM C39 and two cylinders were tested in splitting tension following ASTM C496 in an ELE International concrete compression machine. 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 a Swiss impact hammer in five locations along the cylinder, Figure 3-10. The compressive load and stress, tensile load, and impact hammer values were recorded.

![Cylinder hammer tests](image)

**Figure 3-10: Cylinder hammer tests**
After testing the cylinders, the block was tested along the edge of the block with the impact hammer and the values were recorded. Circles were drawn marking the ACI 318 breakout diameter and a picture was taken of the block before the breakout tests were conducted. The predicted breakout load was then calculated using Equation 2-12. This value was used to determine the rate of loading and initial load values as set forth by ASTM E 488.

The breakout tests were performed according to ASTM E 488. 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. An initial load of approximately 900 N (200 lb) was applied, then the data acquisition system was started, and the stud was pulled to failure. After failure the pump was shut off, the data acquisition system stopped and the process began again for the next stud. This was repeated until all of the studs on the block were tested. The breakout tests were performed at concrete ages of 12, 16, 20 hours and 3, 8, and 28 days.

After the breakout tests on the block were completed, the impact hammer was used again to test the edge 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 carried out in a similar manner. Due to the fewer number of 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 de-molded 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 150 x 300 mm (6 x 12 in.) and five 100 x 200 mm (4 x 8 in.) cylinders were tested in compression and splitting tension respectively at 28 days to establish the 28 day strength. Figure 3-11 shows a typical series of breakout failures.
Figure 3-11: Typical breakout failure pattern at 16 hours
4.0 Results

The results collected from each stud breakout test included 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 three 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 was calculated using:

\[ T = \frac{2P}{\pi ld} \]  

Eq. 4-30

from ASTM C496. The detailed data from the cylinder compressive strength tests and the split cylinder tests are provided in the Appendix, Tables A-1 through A-4

4.1 Breakout Results

During the first stage of testing all failures occurred in concrete breakout until the concrete age reached 3 days. For the 28 day block all failures occurred in the steel stud. Using Equation 2-9 for comparison, the average test-to-predicted ratios for the concrete breakout failures ranged from 1.3 to 1.8, Tables 4-1 and 4-2. Therefore the ACI capacity equations under predicted the strength in all cases. The data and a comparison with Equation 2-9 and 2-12 are presented in Figure 4-1 and Figure 4-2 and the raw test data is summarized in the Appendix: Table A-5 and Table A-6 for the Series 1 and Series 2 tests respectively.

Table 4-1 requires additional discussion. For the 8 and 28 day tests, the predicted failure mode changes from concrete breakout to steel yield. Thus, only the data less than 8 days is appropriate for analysis of concrete breakout capacity. Secondly, the material specifications for the study provide only the minimum yield and ultimate tensile stress. Loads exceeding these minimum values are possible. Over-strength conditions occurred in numerous test, evidenced in Table 4-1 where both breakout loads and steel yield loads exceed the lower bound of 45.4 kN (10.2 kips) yield and 57.8 kN (13.0 kips) ultimate. This is discussed in more detail in section 4.4. All test were terminated at about 89 kN (20 kips) to protect the load cell.
Table 4-1 Comparison of Predicted Breakout Strength and Test Results for Test Series 1

<table>
<thead>
<tr>
<th>Test</th>
<th>$P_u$ (lbs)</th>
<th>$P_u/N_{cb}$ (kN)</th>
<th>$P_{u/1}$ (lbs)</th>
<th>$P_{u/1}/N_{cb}$ (kN)</th>
<th>$P_{u/2}$ (lbs)</th>
<th>$P_{u/2}/N_{cb}$ (kN)</th>
<th>$P_{u/3}$ (lbs)</th>
<th>$P_{u/3}/N_{cb}$ (kN)</th>
<th>$P_{u/4}$ (lbs)</th>
<th>$P_{u/4}/N_{cb}$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>8067</td>
<td>35.9</td>
<td>10105</td>
<td>45.0</td>
<td>14176</td>
<td>63.1</td>
<td>11498</td>
<td>51.2</td>
<td>7868</td>
<td>35.0</td>
</tr>
<tr>
<td>2</td>
<td>8878</td>
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<td>12077</td>
<td>53.7</td>
<td>13961</td>
<td>62.1</td>
<td>11416</td>
<td>50.8</td>
<td>16206</td>
<td>72.1</td>
</tr>
<tr>
<td>3</td>
<td>8629</td>
<td>38.4</td>
<td>9566</td>
<td>42.6</td>
<td>15962</td>
<td>71.0</td>
<td>14001</td>
<td>62.3</td>
<td>17597</td>
<td>78.3</td>
</tr>
<tr>
<td>4</td>
<td>7800</td>
<td>34.7</td>
<td>10804</td>
<td>48.1</td>
<td>11695</td>
<td>52.0</td>
<td>13039</td>
<td>59.7</td>
<td>14875</td>
<td>66.2</td>
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<tr>
<td>5</td>
<td>9667</td>
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<td>9736</td>
<td>43.3</td>
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<td>59.2</td>
<td>15583</td>
<td>69.3</td>
<td>10931</td>
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<td>47.3</td>
<td>9379</td>
<td>41.7</td>
<td>12783</td>
<td>56.9</td>
<td>14099</td>
<td>62.7</td>
<td>12243</td>
<td>54.5</td>
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<td>7</td>
<td>11696</td>
<td>52.0</td>
<td>10504</td>
<td>46.7</td>
<td>15061</td>
<td>66.8</td>
<td>17161</td>
<td>76.4</td>
<td>11875</td>
<td>52.8</td>
</tr>
<tr>
<td>8</td>
<td>10981</td>
<td>48.9</td>
<td>10618</td>
<td>47.3</td>
<td>12510</td>
<td>55.7</td>
<td>13823</td>
<td>61.5</td>
<td>13289</td>
<td>59.1</td>
</tr>
<tr>
<td>9</td>
<td>9110</td>
<td>40.5</td>
<td>10123</td>
<td>45.0</td>
<td>11923</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td>1.73</td>
<td>1.45</td>
<td>1.82</td>
<td>1.41</td>
<td>1.25</td>
<td>1.44</td>
<td></td>
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</tr>
</tbody>
</table>

Notes: 1) Values in bold (red) represent tensile failures of insert assembly. 2) $h_{ef} = 71.5$ mm (2.813 in.) for all predictions

Table 4-2 Comparison of Predicted and Test Results for Test Series 2

<table>
<thead>
<tr>
<th>Test</th>
<th>$P_u$ (lbs)</th>
<th>$P_u/N_{cb}$ (kN)</th>
<th>$P_{u/1}$ (lbs)</th>
<th>$P_{u/1}/N_{cb}$ (kN)</th>
<th>$P_{u/2}$ (lbs)</th>
<th>$P_{u/2}/N_{cb}$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>9933</td>
<td>44.2</td>
<td>14267</td>
<td>63.5</td>
<td>19878</td>
<td>88.5</td>
</tr>
<tr>
<td>2</td>
<td>8399</td>
<td>37.4</td>
<td>13418</td>
<td>59.7</td>
<td>20732</td>
<td>92.3</td>
</tr>
<tr>
<td>3</td>
<td>8295</td>
<td>36.9</td>
<td>14012</td>
<td>62.4</td>
<td>20923</td>
<td>93.1</td>
</tr>
<tr>
<td>4</td>
<td>9561</td>
<td>42.5</td>
<td>11007</td>
<td>49.0</td>
<td>20297</td>
<td>90.3</td>
</tr>
<tr>
<td>5</td>
<td>14942</td>
<td>22.0</td>
<td>12483</td>
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<td>20182</td>
<td>92.8</td>
</tr>
<tr>
<td>6</td>
<td>7026</td>
<td>31.3</td>
<td>11690</td>
<td>52.0</td>
<td>&gt;20000</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>7075</td>
<td>31.5</td>
<td>10128</td>
<td>45.1</td>
<td>&gt;20000</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>6239</td>
<td>27.8</td>
<td>20852</td>
<td>92.8</td>
<td>&gt;20000</td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td>1.28</td>
<td>1.66</td>
<td>2.45</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Conclusions or recommendations in this report are the opinions of the authors. PCI assumes no responsibility for the interpretation or application of the information contained herein.
The second series of tests was conducted to determine if the compressive field caused by the loading frame may have caused the test-to-predicted ratio to be higher than 1.0. Once again the average test-to-predicted ratios were all greater than 1.0. At 16 and 20 hours the test-to-predicted ratios are higher than series 1 tests suggesting the tripod frame did not affect the results. At 20 hours three of the studs exceed the 89 KN (20 kips) capacity of the load cell and the test was stopped to avoid damaging the data acquisition setup. These test results were not included in the test-to-predicted calculations.
As an additional means of comparison to the current design equations the 5% fractile was calculated for each of the different curing times from each series using Equation 2-1 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.

4.2 Cylinder Results

The compressive cylinder results show a 28-day average compressive strength of 36.7 MPa (5320 psi) for the first series of testing and 35.3 MPa (5120 psi) for the second series of testing. The cylinder’s compressive and splitting tensile strength increased with age, as was to be expected. The strengths were then compared to several of the equations presented in chapter 2, Figure 4-3.
Average tensile strengths are consistently higher than the expected tensile strengths based on the equations presented in the literature review comparing the concrete compressive strength to the splitting tensile strength. Using the recommendations put forth by Teychenné (1954) for beam tensile strength and Equation 2-19 by Oluokun for split cylinder tensile strength, the tensile strength is increased on average by 25% when crushed coarse aggregate is used. Applying this increase to the equations more closely matches the data collected as can be seen in Figure 4-4. A power fit is also plotted as a thin line for comparison and also has a power coefficient greater than the 0.5 in. the ACI equation.
4.3 Statistical Analysis

4.3.1 Regression Fitting

Several variables were chosen for a possible model for early age breakout strength. The data was limited to the breakouts from the tests with a concrete age of 20 hours and less. This was done so that only breakout failures would be 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 analysis agrees with a model selection analysis using both Cp and adjusted R squared. Cp is a calculation in the statistical reduction program used to prevent under-fitting the data, assures there are enough variables to adequately describe the data, and to reduce bias. The best fit line using the square root of compressive strength, compressive strength, and an intercept is plotted in Figure 4-5, as the R best fit line.

This particular statistical model has several problems. Over this range of compressive strengths there is an issue with multicollinearity between the square root of compressive strength and compressive strength. This makes it difficult to determine how the variables relate to one another and their meaning in the regression equation. There is an additional issue with the intercept and the slope of the line below about 6.9 MPa (1000 psi) 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 empirical 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. Using the same variables that were used for the best fit equation produced the R 0-Intercept line shown in Figure 4-5. This gives an R squared value of nearly 0.97. This is almost identical to the 0.98 listed by Meinheit, Anderson and Tureyen (2007). However, this high value is due to the model used for comparison and should not be used as a means of determining how well the equation fits the data. A power fit using only compressive strength was also used in an attempt to develop an equation similar to the current equations. The exponent that produces the best fit of the data is equal to 0.853. This value is larger than the exponents of about 0.5 used in the current equations.

Figure 4-5 additionally separates the test 1 and test 2 data. Both tests use the same commercial concrete mixtures and concrete strength is reported at the time of testing, for example 20 hours. The strength gain of the test 2 data is slightly less than test 1. In consequence, the statistical data examines the tests 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.
4.3.2 Statistical Significance Tests

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

Nine 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.

4.4 Steel Stud Tests

Several concrete breakout strength failures are above the theoretical strength of the studs. Furthermore, many of the steel failures exceeded the expected failure load based on the mechanical properties listed in PCI Design Handbook 6th Ed. The single stud shaft tests also exceeded the
expected yield and ultimate stresses. The yield stress from the steel stud shank test was approximately 455 MPa (66 ksi) and the ultimate stress was 565 MPa (82 ksi). This leads to a stud failure value of 71 kN (16 kips) on a 13 mm (0.5 in.) 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 3 tests on full insert assemblies was approximately 375 MPa (54 ksi) and the average ultimate tensile stress of the 3 tests was 531 MPa (77 ksi), table 4-3. All failures were cup and cone tensile failures away from the weld, Figure 4-6.

<table>
<thead>
<tr>
<th>Stud</th>
<th>Failure Load – MPa (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>535 (77.7)</td>
</tr>
<tr>
<td>2</td>
<td>522 (75.8)</td>
</tr>
<tr>
<td>3</td>
<td>535 (77.7)</td>
</tr>
</tbody>
</table>

Table 4-3 Stud Assembly Failure Loads

Figure 4-6: Headed studs after testing
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 725 MPa (105 ksi). The higher load is attributed to the eccentricity in some of the inserts. This leads to a higher load to straighten the insert before achieving tensile failure. Because the inserts failed in tension rather than concrete breakout, the tests were consistent with other inserts failing in tension.

4.5 Impact hammer results

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. The impact hammer results can be found in Table A-6 through Table A-11. For the 12 hour strength concrete the hammer often reported a factor less than 10, meaning the readings were not reliable. The overall observation is that the hammer was unable to identify significant differences in the test block strength arising before and after the testing was complete. The hammer readings do give some indication of strength gain in the four hours between tests, but the resolution of the strength gain would require a more complete sampling data set than obtained on these blocks.

4.6 Load deflection behavior

All breakout tests recorded the load-deflection data as a function of time. The loading was applied with a positive displacement pump, thus load and time are effectively interchangeable. A typical test required between 90 and 120 seconds to complete when the load is applied at the rate specified in ASTM E 488. Figure 4.7 is representative of a typical failure. The breakout begins at about 90 seconds and is complete by 110 seconds. The corresponding load versus time curve shows the nearly linear application of load and the sudden loss of capacity, Figure 4-8. A substantial number of the inserts failed suddenly, Figure 4-9. In this instance the breakout failure occurred in approximately 10 seconds from the initiation of deflection due to fracture until the total failure.
Conclusions or recommendations in this report are the opinions of the authors. PCI assumes no responsibility for the interpretation or application of the information contained herein.

Figure 4-7: Insert deflection as a function of time

Figure 4-8: Load application as a function of time
Conclusions or recommendations in this report are the opinions of the authors. PCI assumes no responsibility for the interpretation or application of the information contained herein.

**Figure 4-9:** Insert deflection as a function of time

**Figure 4-10:** Insert load as a function of time
5.0 Design and Production Considerations

5.1 Breakout Sensitivity

Figures 4-7 through 4-10 indicate that insert breakout is very brittle. The compressive strength of a member when the load is applied is an important variable. At lower strengths a greater embedment depth is needed to develop the stud tensile capacity. Using this information, charts can be generated relating the embedment depth required at the specified concrete stripping strength and the necessary embedment depth at different stripping strengths. An example plot for a single stud is provided below in Figure 5-1.

![Chart](image)

Figure 5-1: Comparison of Minimum of Embedment Depth at Various Compressive Strengths

For example, if a precast panel is to have load applied at 3000 psi, but the strength at the time of stripping is 2000 psi, the embedment length must be approximately 15 percent greater to avoid a breakout failure. Therefore, embedment depth for early stripping must be based on the lowest specified strength at the time of load application and that this strength is validated.

Development of this type of chart, based on the breakout strength of the insert, may complement plant experience on striping and handling precast elements. The chart addresses capacity but does not address the load applied to the member. In addition to the self-weight of the
member, suction generated when striping a piece from the form or dynamic loads during handling can increase the demand on the study assembly.

5.2 Insert strength considerations

Greater than specified steel strength can lead to other design considerations. In the case of an overload most designs are preferably ductile failure in the steel. If the yield or ultimate steel stress is greater than specified, there is an increased possibility of a brittle breakout failure. The difference in embedment depth needed for steel yielding using the current ACI equation can be seen in Figure 5-2. This graph shows the embedment depth necessary for equal steel yielding and concrete breakout load with a single stud diameter of 13 mm (0.5 in.) and a specified steel yield strength of 352 MPa (51 ksi) versus an upper bound actual yield of 455 MPa (66 ksi).

![Embedment Depth to Equal Concrete Breakout and Steel Failure Loads](image)

**Figure 5-2: Sensitivity to Steel Yield Stress**

The increase in embedment depth required is even greater with studs groups and larger diameter studs. These concerns can be addressed by adding as little as 25 mm (1 in.) to the stud length or establishing a maximum yield stress to assure ductile failure.

5.3 Other considerations of concrete tensile stress

Forms 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 or the mix design may have a lower than average tensile to compressive strength ratio or be a function of the insert geometry. If a plant is using a mix design with lower than average tensile to compressive strength, i.e., $f_t < 6\sqrt{f'c}$, it may be prudent to either conduct in-plant tests to determine the true breakout strength of the anchors used in their construction or to adjust the specified stripping strength to account for the lower tensile capacity.
6.0 Conclusions

The average breakout values in Tables 4-1 and 4-2 exceeded the expected breakout values based on Equation 2-9 and Equation 2-12 by the amount suggested in Figure 2-1. Based on this test program and the theoretical tensile strength gain, the capacity predictions in the PCI Design Handbook 6th Ed are sufficient for the design of inserts or lifting inserts for concrete compressive strengths as low as 5 MPa (1000 psi) in uncracked concrete. This is consistent with the findings in the literature review that tensile strength gains faster than compressive strength at early age.

Even though the age of the concrete does not need to be corrected for the compressive strength, the strength at release or striping remains an important factor. Low strength concrete is much more sensitive to breakout, thus the strength specified for striping and handling must be verified.

Lastly, to reiterate important initial assumptions, headed studs were used as a proxy for all lifting inserts. Specialty inserts are projected to behave similar 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 having unusual geometry or displaying erratic behavior is highly recommended. No shear or side breakout strength tests were performed to validate this assumption.

7.0 Acknowledgements

This research was conducted with the support of a Daniel P. Jenny research fellowship. The authors’ extend their thanks to PCI for the support and to the advisory committee for their thorough review. Special acknowledgment goes to Ned Cleland, Don Meinheit, Don Logan and Tom D’Arcy for comments on the testing conducted in this work. The opinions and conclusions in this paper are those of the authors.
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Appendix A - Raw Data

Table A-1: Compressive Cylinder Strengths for First Series Tests

<table>
<thead>
<tr>
<th>Age, hours</th>
<th>12</th>
<th>16</th>
<th>20</th>
<th>79</th>
<th>205</th>
<th>672</th>
</tr>
</thead>
<tbody>
<tr>
<td>Age, days</td>
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<td>0.67</td>
<td>0.83</td>
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<td>8.54</td>
<td>28</td>
</tr>
<tr>
<td>Cylinder 1</td>
<td>790</td>
<td>1230</td>
<td>1710</td>
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<td>3030</td>
<td>5220</td>
</tr>
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<td>Cylinder 2</td>
<td>740</td>
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<td>1790</td>
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<td>Cylinder 3</td>
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<td>1940</td>
<td>2820</td>
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<tr>
<td>Cylinder 4</td>
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<td>1340</td>
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<td>Cylinder 5</td>
<td>860</td>
<td>1560</td>
<td>1520</td>
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<td>4190</td>
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Table A-2: Splitting Tensile Cylinder Strengths for First Series of Tests

<table>
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<tr>
<th>Age, hours</th>
<th>12</th>
<th>16</th>
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<tbody>
<tr>
<td>Age, days</td>
<td>0.50</td>
<td>0.67</td>
<td>0.83</td>
</tr>
<tr>
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<td>800</td>
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<td>1890</td>
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<td>740</td>
<td>1580</td>
<td>1990</td>
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<td>1030</td>
<td>1640</td>
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<tr>
<td>Cylinder 4</td>
<td>1180</td>
<td>1390</td>
<td>1980</td>
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Table A-3: Compressive Cylinder Strengths for Second Series Tests

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<th>20</th>
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<th>672</th>
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<tr>
<td>Age, days</td>
<td>0.50</td>
<td>0.67</td>
<td>0.83</td>
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<td>28</td>
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<tr>
<td>Cylinder 1</td>
<td>85</td>
<td>220</td>
<td>385</td>
<td>370</td>
<td>585</td>
<td>830</td>
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<tr>
<td>Cylinder 2</td>
<td>105</td>
<td>230</td>
<td>405</td>
<td>360</td>
<td>560</td>
<td>840</td>
</tr>
<tr>
<td>Cylinder 3</td>
<td>165</td>
<td>270</td>
<td>355</td>
<td>550</td>
<td>550</td>
<td>775</td>
</tr>
<tr>
<td>Cylinder 4</td>
<td>270</td>
<td>285</td>
<td>385</td>
<td>660</td>
<td>545</td>
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<tr>
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Table A-4: Splitting Tensile Cylinder Strengths for Second Series of Tests

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<td>Age, days</td>
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<td>0.83</td>
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<td>285</td>
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<tr>
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Table A-5: Ultimate Failure Loads for First Series Tests

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<th>Block 5</th>
<th>Block 6</th>
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</thead>
<tbody>
<tr>
<td>Test 1</td>
<td>8067</td>
<td>10105</td>
<td>14176</td>
<td>11498</td>
<td>7868</td>
<td>12292</td>
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<tr>
<td>Test 2</td>
<td>8887</td>
<td>12077</td>
<td>13961</td>
<td>11416</td>
<td>16206</td>
<td>17222</td>
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<tr>
<td>Test 3</td>
<td>8629</td>
<td>9566</td>
<td>15962</td>
<td>14001</td>
<td>17597</td>
<td>15141</td>
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<tr>
<td>Test 4</td>
<td>7800</td>
<td>10804</td>
<td>11695</td>
<td>12105</td>
<td>14875</td>
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</tr>
<tr>
<td>Test 5</td>
<td>9667</td>
<td>9736</td>
<td>13314</td>
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<td>9379</td>
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<td>14099</td>
<td>12243</td>
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<td>14807</td>
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<tr>
<td>Test 9</td>
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<td>10123</td>
<td>11923</td>
<td>12978</td>
<td>12507</td>
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<tr>
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</table>

Note: A small number of inserts failed at loads higher than the yield strength of the materials. The possible rationale for this behavior is discussed in Section 4.4.
Table A-6: Ultimate Failure Loads for Second Series Tests

<table>
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<tr>
<th>Age, hours</th>
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<th>Block 5</th>
<th>Block 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Age, days</td>
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<td>0.67</td>
<td>0.67</td>
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<td>Ultimate Load, lbs</td>
<td>Failure Mode</td>
<td>Ultimate Load, lbs</td>
<td>Failure Mode</td>
</tr>
<tr>
<td>Test 1</td>
<td>9933 Breakout</td>
<td>4942 Breakout</td>
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<td>12483 Breakout</td>
<td>19878 Breakout</td>
<td>20923 Breakout</td>
</tr>
<tr>
<td>Test 2</td>
<td>8399 Breakout</td>
<td>7026 Breakout</td>
<td>13418 Breakout</td>
<td>11690 Breakout</td>
<td>20732 Breakout</td>
<td>&gt;20000</td>
</tr>
<tr>
<td>Test 3</td>
<td>8295 Breakout</td>
<td>7075 Breakout</td>
<td>14012 Breakout</td>
<td>10128 Breakout</td>
<td>&gt;20000</td>
<td>&gt;20000</td>
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<tr>
<td>Test 4</td>
<td>9561 Breakout</td>
<td>6239 Breakout</td>
<td>11007 Breakout</td>
<td>20297 Breakout</td>
<td>20852 Breakout</td>
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</tr>
</tbody>
</table>

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Table A-7: Impact Hammer Results for Series 1 12 Hour Tests

<table>
<thead>
<tr>
<th>Cylinder 1</th>
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<th>Cylinder 3</th>
<th>Block Side</th>
<th>Block Top</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;10</td>
<td>10</td>
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<td>18</td>
<td>16</td>
</tr>
<tr>
<td>12</td>
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</tr>
<tr>
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<td>20</td>
<td>17</td>
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<tr>
<td>&lt;10</td>
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<td>&lt;10</td>
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<td>12</td>
<td>16</td>
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</tbody>
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Table A-8: Impact Hammer Results for Series 1 16 Hour Tests

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<tr>
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Table A-9: Impact Hammer Results for Series 1 20 Hour Tests

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Table A-10: Impact Hammer Results for Series 1 3 Day Tests

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</thead>
<tbody>
<tr>
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</table>

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Table A-11: Impact Hammer Results for Series 1 3 Day Tests

<table>
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<th>Series 1 8 Day</th>
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Table A-12: Impact Hammer Results for Series 1 28 Day Tests

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</thead>
<tbody>
<tr>
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Table A-13: Impact Hammer Results for Series 2 12 Hour Tests

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Table A-14: Impact Hammer Results for Series 2 16 Hour Tests

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<th>Block 2 Side</th>
<th>Block 2 Top</th>
</tr>
</thead>
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Table A-15: Impact Hammer Results for Series 2 20 Hour Tests

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<th>Block 2 Top</th>
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</thead>
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Table A-16: Impact Hammer Results for Series 2 28 Day Cylinder Tests

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</tr>
<tr>
<td>17</td>
<td>18</td>
<td>16</td>
</tr>
</tbody>
</table>
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Appendix B - Test Photographs after breakout

Figure B-1: First Series 12 Hour Block

Figure B-2: First Series 16 Hour Block
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Figure B-3: First Series 20 Hour Block

Figure B-4: First Series 3 Day Block
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Figure B-5: First Series 8 Day Block

Figure B-6: First Series 28 Day Block
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Figure B-9: Second Series 16 Hour Block 1

Figure B-10: Second Series 16 Hour Block 2
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Figure B-11: Second Series 20 Hour Block 1

Figure B-12: Second Series 20 Hour Block 2