Influence of Differential Deflection on Staged Construction Deck-Level Connections

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FOREWORD

Construction-induced traffic congestion is a growing problem in the United States. The general state of repair of the highway infrastructure and the need to reconstruct structures without severing transportation links has resulted in the demand for advances in the construction field. Staged construction, wherein portions of a bridge are progressively reconstructed while the remainder of the bridge continues to carry traffic, is being engaged more frequently as a means to maintain traffic. The Federal Highway Administration is engaged in research to advance the state-of-the-practice for field-cast connections between structural components in highway bridges. Staged construction presents special challenges with regard to field-cast connections, as the loadings a bridge might experience during connection completion could force a concentration of differential deflections within the connection and thus reduce the service capacity, ultimate capacity, and durability of the connection. The research presented herein focused on the effects of differential deflections across a connection on the bond strength between reinforcing bars and field-cast connections in bridge deck-level connections. This research represents a move toward furthering the understanding of the performance requirements which must be met in order to successfully complete staged construction activities.

This report corresponds to the TechBrief titled "Influence of Differential Deflection on Staged Construction Deck-Level Connections" (FHWA-HRT-12-055). This report is being distributed through the National Technical Information Service for informational purposes. The content in this report is being distributed "as is" and may contain editorial or grammatical errors.

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16. Abstract

Many rapid construction methods have been investigated and implemented using modular bridge deck components; however, the loadings a bridge might experience during staged construction could force a concentration of differential deflections within the connection and thus reduce the connection's service capacity, ultimate capacity, and durability. The research presented herein focused on the effects of differential deflections across a connection on the bond strength between reinforcing bars and field-cast connections in bridge deck-level connections. Six inch cube pull-out type specimens with number four bars were cast. Differential deflections were imparted from casting until final set that ranged from 0.1 in. (0.254 cm) to 0.005 in. (0.0127 cm) of linear bar movement perpendicular to the bar axis. A range of different grout materials which might be deployed in these field-cast connections were engaged in this research program, including standard conventional grouts, deck concretes, ultra-high performance concretes (UHPC), epoxy grout, magnesium phosphate grout, and cable grout.

The results of this research provide an initial assessment of whether differential deflections across field-cast connections may be of concern. From the perspective of the pure bond strength, deflecting the rebar prior to final set of the embedment material can have a detrimental effect on the bond. When the rebar deflected 0.05 in. (1.27 mm) or more, reduced bond capacity was observed. When the rebar deflected 0.01 in. (0.25 mm) or less, the results indicate only minor variations in bond strength.

This report corresponds to the TechBrief titled "Influence of Differential Deflection on Staged Construction Deck-Level Connections" (FHWA-HRT-12-055).

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.
(Revised March 2003)

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CHAPTER 1. INTRODUCTION

INTRODUCTION

Rapid construction methods help prevent traffic delays and minimize the inconveniences to the traveling public. Many new methods have been investigated and implemented using precast subassemblies on bridges. These methods have shown great promise because precast components can be produced with great quality control in precast plants, resulting in superior products that allow for expedited construction schedules. States continue to investigate and advance their respective bridge programs through the use of precast products such as precast bulb tees, full depth precast bridge decks, and box beams.

The most critical field construction process for precast subassemblies is completing the connections. Long term performance problems have arisen in connections on past projects. These performance problems have been attributed to a variety of causes, including construction techniques, materials, and poor designs. Much research attention has been placed on making better connections between the components.

One area of concern is the process of completing connections while portions of the bridge remain under traffic loads. This is frequently referred to as staged construction. The traffic loading causes deflections on portions of the bridge, potentially resulting in significant differential deflections occurring across the connections which join construction stages. These connections between stages may exhibit poor performance due to the differential deflections causing the field-cast connection grout to exhibit poor bond with the discrete connector elements within the connection.

This research effort aimed to begin to study the impact of differential deflections across staged construction connections on the short and long-term performance of said connections. The size, frequency, and the duration of the deflections are all factors of concern. Many different types of grout materials have been proposed for use in these field-cast connections, and differential deflections may affect each one differently. The goal is to understand how the bond between rebar and the grout material changes due to movements which occur during the curing process.

OBJECTIVE

The objective of this research project was to determine the impact of differential movement across a staged construction connection. Specifically, the research investigated the bond strength of reinforcing bars embedded within freshly cast connection grouts as impacted by differential movement of the rebar with respect to the grout.

SUMMARY OF APPROACH

This research focused on the bond strength between reinforcing bars and grout materials that were subjected to differential movement during curing. Six inch cube pull-out specimens with number four bars were cast based on the ASTM C234-91a standard[21]. Differential deflections were imparted that ranged from 0.1 in. (0.254 cm) to 0.005 in. (0.0127 cm) of linear bar movement perpendicular to the bar axis. The deflections were applied to the cubic molds while the rebar was fixed in place. The deflections were imparted periodically at 30 second intervals and continued from casting until final set of the grout.

A range of different grout materials which might be deployed in these field-cast connections were engaged in this research program. These included standard conventional grouts, deck concretes, ultra-high performance concretes (UHPC), epoxy grout, magnesium phosphate grout, and cable grout. The materials were cast while the forms were deflecting at set intervals. After final set, the cubes were allowed to cure until approximately 24 hours after casting at which point they were tested. A series of control cubes were also cast and tested. These remained stationary during curing.

Pullout tests were performed on the cube specimens. The load resisted and the deflection observed for the reinforcing bar relative to the grout material was recorded for each specimen. Comparisons were then made between the ultimate load held by the control and the deflected specimens for each series of tests. A statistical analysis was performed to determine if particular conditions affected the bond with the given materials and deflection criteria. The results helped indicate whether relative movement between grout materials and rebar impacts the bond strength. This research provides an initial assessment of whether differential deflections across field-cast connections may be of concern.

OUTLINE OF REPORT

The report is divided into five chapters. Chapter 1 provides an introduction to the study. The second chapter gives background information on why the study was performed. Chapter 3 presents the physical properties of the specimens and materials used in the study along with information on the test setup. The fourth chapter provides the results and analysis. The last chapter, Chapter 5, provides conclusions based on these tests and suggestions for future work.

CHAPTER 2. BACKGROUND

INTRODUCTION

Differential deflections between different stages of construction have had limited study. The body of research on this topic has not demonstrated whether the bond created between reinforcing bars and grout are negatively affected by different forms and frequencies of traffic load-induced differential deflections. A number of test methods exist that have been used in past studies on measuring bond strength between reinforcing bars and embedment materials. What is needed is a method to test the bond strength after the bond has undergone typical dynamic conditions experienced on a bridge during construction. The tests could provide evidence facilitating a better understanding of the bond performance when different materials are applied with varying types of traffic loads.

DIFFERENTIAL DEFLECTIONS IN BRIDGE DECKS

Precast construction on bridge superstructures has gained popularity in recent years. Many studies have been done on the implementation on precast decks, girders, box beams, and other assemblages. In many cases these techniques are used for bridge reconstruction projects in addition to new construction projects. When rebuilding a bridge that is in service, staged construction techniques are commonly used. Staged construction keeps a route open and allows traffic to continue traveling on the structure with reduced width and/or shifted lanes. This is a high priority for many bridge owners.

One of the final steps in staged construction is to connect the different staged sections of a bridge. Joints are left open and then cast-in-place concrete or grout is placed between the different pieces. While this construction activity is being completed, traffic frequently remains on sections of the bridge thus creating differential deflections between the staged sections as shown in Figure 1. These differential deflections are heavily dependent on the structural configuration of the bridge and on the traffic loads applied. The deflections may affect the long-term performance of these field-cast connections.

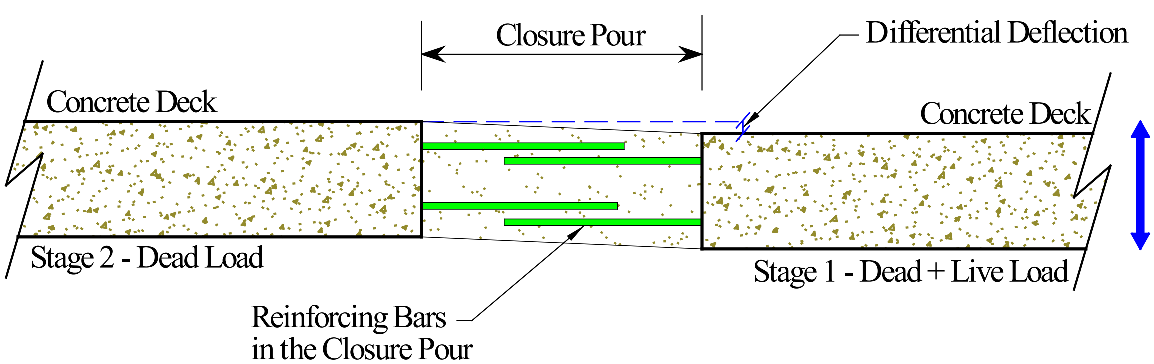

Figure 1. Illustration. Differential deflection at a closure pour.

The Manual of Concrete Practice[1] recommends differential deflections not exceed ¼ inch (6 mm). In cases when they do, the recommendation is to remove or reroute traffic on the bridge to ensure the deflections fall below the limit. The lack of research on this topic raises questions as to the appropriateness of these recommendations.

BOND STRENGTH BETWEEN REINFORCING BARS AND CONCRETE

A bond is formed between a reinforcing bar (rebar) and the material that is cast around the bar. The bond between reinforcing bars consists of three main components: chemical adhesion, surface friction, and bearing on the deformations on the bar. Typical bridge construction uses deformed rebar which greatly increases the bond strength as compared to smooth rebar. The chemical adhesion and friction are greatly reduced after a small tension force is applied. The primary loading carrying mechanism for deformed rebar is bearing on the deformations [2,3].

The concrete bearing blocks next to the reinforcing bar ribs must provide an equal and opposite force to transfer the load. The concrete bearing block is formed when the concrete is poured around the reinforcing bar and then cured. The mechanical properties of this concrete and the amount of concrete around the reinforcing bar will directly relate to the strength of the bond[3]. Any disruption to the concrete during casting and curing could affect the formation of the concrete block and hence the strength of the bond. The same process and concerns are still apparent when different types of materials are used in place of concrete such as grouts, epoxy grouts, or high performance mix designs.

STUDIES ON BONDS AFFECTED BY DIFFERENTIAL DEFLECTIONS

There has been a limited amount of research on the performance of joints that were built with differential deflections. Much of the literature is based on field inspections that indicate there may or may not be problems with the bond formation depending on the size of the differential deflections and the type of concrete used. Manning[4] did a large investigation using literature, state surveys, and past experience. His conclusion was there is "insufficient evidence" that differential deflections cause problems with bonds. It appeared that there may not be a problem on bridge decks, however there was enough evidence from past projects to show that cracking did occur in many cases. The state of Michigan did a study on approximately 100 bridges, many under differential deflections during construction, by inspecting them nine years after construction. The results showed that shoring during construction did not help the performance and that concrete mixes with excess water (high slumps) did not perform as well as stiffer mixes [5]. Another researcher also noted that high slump concretes and reinforcing bars with bends tend to be more affected by deflections[6]. A study in the state of California focused on inspecting bridges that were widened while traffic remained in place. The inspections revealed very few had negative impacts based on cracking observed during standard bridge inspections[7]. A study in Maryland on effects of differential deflections on bond strength was inconclusive[8].

A study[9] on the structural integrity of slabs cast between old and new concrete bridge decks indicates this structural design may result in performance issues. A typical 92 ft (28 m) span bridge with a maximum differential deflection of 0.5 in. (12.3 mm) was used for the analysis. The results revealed that differential deflections may affect the strength development and structural integrity of the concrete in the slab. The bond strength was not investigated. The conclusion indicates a reduction in differential deflection is needed by adding supports or removing traffic from the bridge decks during the casting procedure. A separate study in Ohio on one bridge revealed that vibrations from traffic did not appear to cause early cracking. However, testing was recommended on a wider range of variables including frequency and angular rotation at the connection[10].

Silfwerbrand[11] performed a study on the effects of traffic induced deflections on a freshly placed overlay on a concrete surface. Deflections were hypothesized to reduce interface bonding strength in this type of construction. The study involved a series of pull off tests of the overlay on a deck surface. The conclusion was that the vibrations in their tests may have been applied at the wrong time after placing the overlay (3-10 hours). Further tests were recommended for overlays and bonds between concrete and reinforcing bars.

A similar study[12] was performed by the Georgia Department of Transportation to determine if widened bridges with closure pours had been detrimentally affected. Twenty-three bridges were inspected: Eighteen had been widened with closure pours and five had no closure pours. Of the bridges widened with closure pours, two had regular cracks at intervals less than 2 feet (61 cm) but no severe cracking. Seven had minor cracks at intervals greater than 3 feet (92 cm) while nine had insignificant cracks. The five control bridges had no significant cracks.

Following the in-service study done by the Georgia Department of Transportation, a field test was performed[12]. Two pullout specimens were formed across a closure pour during two bridge expansion projects. The specimens were cast on site during construction. The blocks were then removed from the bridge and tested as pullout specimens. There was no report on the type, frequency, or amount of traffic that was allowed on the bridge during the concrete placement. The deflections of the bridge were recorded after the specimens were poured. Due to the limitations of the study the results are inconclusive; however, the testing method did simulate a disturbed bond across a closure pour. A more controlled environment with more testing variables would provide a basis for further assessment of the connection performance.

A visual inspection was performed on 30 bridges in Texas after the bridges' decks were widened[13]. There was no visual distress immediately after construction. Core samples were taken and inspected around the rebar. Visually, the bond did not look distressed in the majority of the cores but the imprints made by the reinforcing bars did indicate the differential movement had an effect. One bridge was instrumented and the closure pour reinforcing bars, plastic concrete, and formwork all vibrated together under traffic loads. The exact traffic loads were not reported. The effect of the vibrations on the instrumented bridge's closure pour was assumed minimal.

A follow up test was performed on 10.6 ft (3.2 m) by 7 in. (18 cm) by 12 in.(31 cm) beams(13). Standard bridge deck concrete for Texas was used in the specimens. Concrete slumps varied between 3 in. (7.6 cm) and 6 in. (15.2 cm). The beams consisted of a simple span and a cantilever that was loaded with a point load. The beams were loaded at 5 minute intervals with a single 1 Hz cycle with deflections of 0.15 in. (0.4 cm) and 0.25 in. (0.6 cm). The load was applied at the end of the beam and deflected the plastic concrete, formwork, and embedded dowel bar. Transverse cracking did occur in the negative moment region over the support and blurred imprints from the reinforcing bars were noted in the core samples taken after testing. However, the curvature measured in the test was three times that measured in the field study. The study concluded that these differential deflections have minimal effect on the strength of the bond for straight bars in a closure pour that is at least 20 bar diameters long.

An in-depth study was performed at the University of Kansas concerning bridge deck deflections on bond strength gain (14). However, the bond strength was based on both the reinforcing bars and concrete vibrating together. There was no differential deflection. The study does recommend further study on differential deflections and the results suggest that high slump concretes (greater than 4 in. (10.2 cm)) may result in lower bond strengths.

The most direct study on the effects of differential deflections on bond strength was done in the early 1950's (15). The study looked at the effects of vibration by comparing pullout specimens. The vibrations were all induced with a hand-operated surface vibrator, typically used for consolidating plastic concrete, operating at 100 Hz. The amplitude of the vibrations in the study was not reported. Different sets of specimens were re-vibrated for one 2 minute period after the concrete was cast. The interval for each set of specimens varied from 15 minutes to 6 hours after placing the concrete. Two sets of vibrations were induced on different specimens: One to the concrete and rebar as a unit and one to the rebar while holding the concrete in place. The vibration of the unit caused bond reductions up to 28% if induced 30 minutes or more after placement. The vibration of the rebar alone showed no significant bond reduction.

While the study is very informative it does not directly correlate to the conditions of differential deflections occurring during bridge construction. In a bridge the vibration would be for shorter intervals over a longer period of time. The consolidation vibrator caused very intense high frequency vibrations which lasted for 2 minutes. Vehicles would tend to cause a quick vibration that might last for approximately a second and be followed by hold times of varying lengths with little movement. A study similar to that reported by Larnach with differing vibration types and intervals, but a similar test setup, would be more relevant to the conditions on a bridge deck.

Much of the past experience looking at differential deflections effects on concrete bonds is based on in service bridge inspections, incomplete studies, or studies that do not correlate well to bridge conditions. The evidence is inconclusive in most cases or does not compare well to the conditions on bridge reconstruction projects. Based on the literature, there is a need to study what happens to bond strengths when differential deflections exist on bridges under staged construction. A variety of conditions need to be considered that simulate the conditions on a bridge.

TESTING BOND STRENGTHS

Bond tests between reinforcing bars and concrete may be grouped into two broad categories: eccentric and concentric tests. The concentric tests have been used for a much longer period of time and have been conducted in a more standardized fashion, but also tend to relate higher bond strengths. The eccentric methods simulate bending situations and give more realistic bond strengths, but the methods are not as uniform or widely adopted.

Eccentric Tests

Various tests have been devised for assessing bond strength by using eccentric loads. Eccentric tests are performed to try to simulate conditions in a beam subjected to flexural forces. The clear cover around the bar is normally a couple inches allowing the bar to fail the bond by splitting. The eccentric tests tend to be specific to an application and not uniform among different testing programs. Two typical layouts are shown in Figure 2.

One set of tests shown in the figure consisted of an eccentric pullout layout designed to eliminate compressive forces in the pullout block(16). This method provided limited clear cover to the specimens resulting in splitting failures. Results from this test were close to results from bond strengths in beam tests with small amounts of clear cover. The focus of this test was on beams in flexure with small amounts of clear cover that fail in splitting.

Similar types of eccentric pullout tests have been performed on larger scales. Hamad and Sabbah performed a series of eccentric pullout tests on concrete specimens with different sizes of rebar. Using the test setup shown in Figure 2, various thicknesses of clear cover were used on the top side of the specimen. Splitting failures resulted from these tests; however, the clear covers were varied to simulate a beam type failure. (17) Lutz performed a similar eccentric pullout test to assess the fundamental characteristics of reinforcing bars in concrete. These were done because they included effects from shear and diagonal tension seen in beams(18). Tholen and Darwin did a beam type pullout test. The test pulled on a reinforcing bar embedded on the top edge a concrete block in a similar manner as the test performed by Hamad and Sabbah. The block was held down to prevent rotation while the bar was pulled(19).

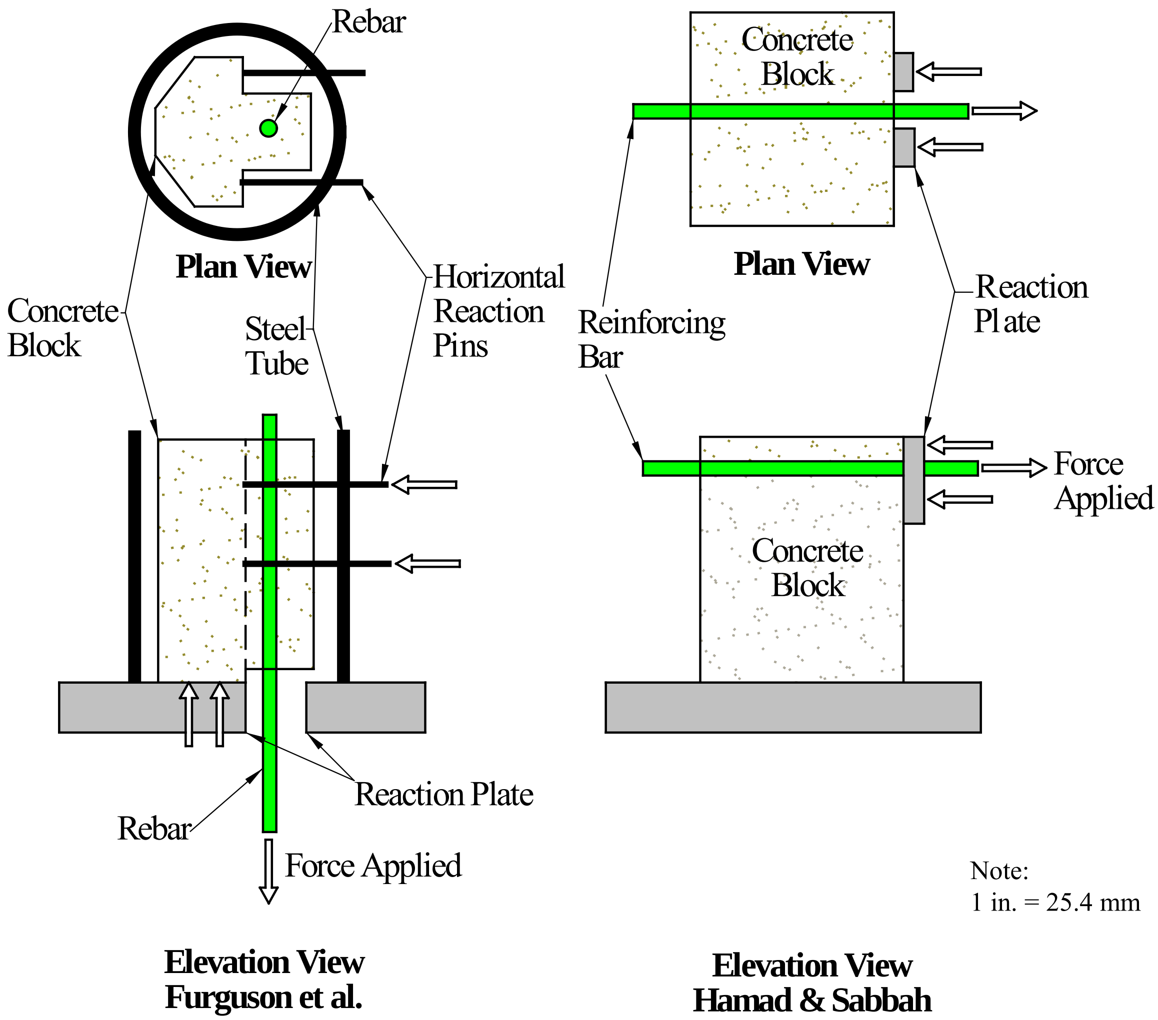

Figure 2. Illustration. Eccentric pullout tests.

Harsh and Darwin did similar types of tests on a full size bridge deck. The bars were cast in the deck and then pulled out using a modified cantilever method. The method ensured that the concrete and rebar were both in tension while the rebar was being pulled out of the deck concrete[(20)].

Concentric Tests

One of the most widely used tests for bond strength is ASTM C234 - Standard Test Method for Comparing Concretes on the Basis of the Bond Developed with Reinforcing Steel[(21)]. This test method was intended to compare the bond strength between a concrete material and a #6 (#19M) reinforcing bar as shown in Figure 3. The method was created to test variations in bond development from a standard reinforcing bar and different surface treatments or material types. The method was not created to test variations in cover around the bar, bar sizes, or bar types.

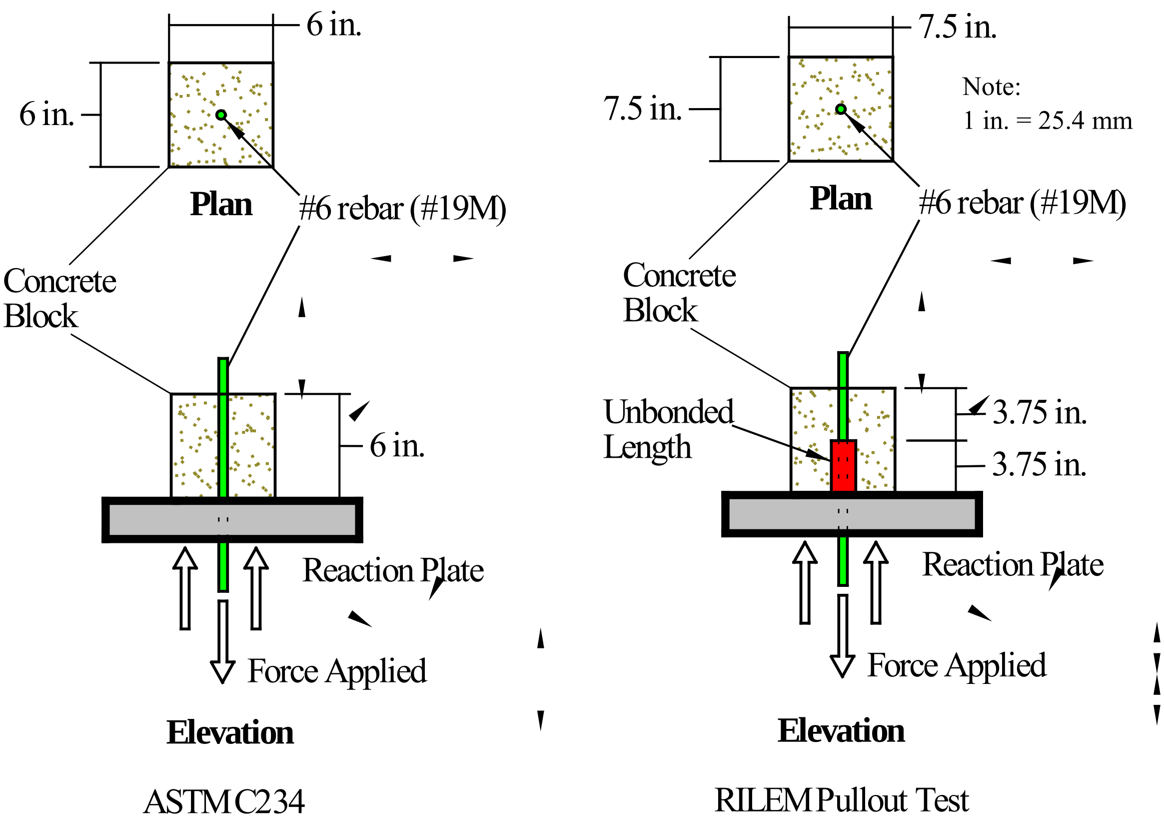

Figure 3. Illustration. Typical concentric pullout tests.

The ASTM C234 was not renewed in 2000. The method does not test the true bond strength commonly used with beams in flexure. The bond strength is measured in an uncracked 6 in. (152 mm) cube. The embedment is through the entire depth of the cube or 8 times the diameter of the bar (8db) for a #6 (#19M) bar. This method has been effectively used for plain reinforcing bars but has been questioned when used with deformed reinforcing bars [22]. When deformed bars are tested in a pullout setup, a compressive confinement force is induced on the bar near the bearing plate creating a higher bond stress with deformed bars. This compressive force will result in higher bond strengths. In addition, some concretes will result in a splitting failure instead of the pullout failure mechanism thereby resulting in lower bond strengths. The higher reported strengths and variation in failure mechanism raised questions regarding the test method.

The RILEM pullout test [23], although somewhat similar to the ASTM C234 test, has a variable sized specimen depending on the rebar size. This test setup is also shown in Figure 3. The sample size changes in dimension with the size of rebar tested allowing tests on multiple sizes of rebar. Another primary difference is the unbonded section. The RILEM test specimen is bonded for a length of 5 bar diameters and then unbounded for an additional 5 bar diameters. The unbounded section is next to the bearing plate (pulled end of the rebar) in order to reduce the effects from confinement forces. Another difference is the overall specimen size. The ASTM test method specifies an 8 bar diameter specimen size with an 8 bar diameter bonded length for a #6 (#19M) rebar. The RILEM test method specifies a 10 bar diameter specimen size with a 5 bar diameter bonded length for the rebar.

A study by Cairns and Abdullah set out to determine if the pullout tests provided data that could be correlated to more realistic splitting bond failures(24). They tested different deformed bars with three different bond tests: the British Standard Pullout Test(25), the RILEM pullout test, and a spliced rebar test.

The results confirmed that concentric pullout tests give higher bond results than design values. However, the RILEM pullout test was found to have benefits. For one, the short embedment length provided a very uniform stress along its length. In addition, it effectively reduces the high confinement forces that may affect the bond strength. Another benefit is the specimens are cast on their side providing a realistic casting position. The tests provided a "fundamental" measure of the bond strength that was good for comparison purposes(24).

Correlations were made between the RILEM pullout results and the British Design Code's bond guidelines. The slip measurement was recorded at the free end of the rebar in the block. The results at 0.0004 in. (0.01 mm) of slip at the free end corresponded to the serviceability stress in the British Design Code. The results at this slip were about 20% higher than the values from a true splice test. The authors felt this result was appropriate considering the splice tests had a smaller amount of concrete cover. The authors did feel that pullout test should not be abandoned but rather correlated to more realistic bond tests. The pullout test remains a good comparative testing method for bond strengths and remains easy to use and replicate(24).

A study on the bond strength of fiber reinforced polymer bars in concrete was done using a direct pullout method(26) . The bars were embedded in blocks and pulled out in a manner similar to ASTM C234. Much like other recent pullout tests, the specimens had an unbonded region in the specimen at the loaded end of the rebar. The RILEM pullout test was used as a guideline for designing the specimens(23). This method of designing pullout specimens helps prevent compressive stresses from interfering with the bond strength. It is still noted that the resulting bond stresses are higher than what occurs in a bending configuration. The main reason was because splitting is not a failure mode when testing with pullout failures unlike when testing with flexure specimens.

The pullout test has been used to compare bond strengths while adjusting other bond variables(27). Normal and lightweight self-consolidating concrete mixes were compared based on the bond strength developed in pullout tests. The ultimate strengths were measured and normalized based on the differing concrete strengths used in the specimens.

Lanarch tested the effects of intense vibrations from a concrete vibrator on pull-out specimens[(15)]. The specimens were 4 in. (101 mm) by 4 in. (101 mm) by 6 in. (152 mm). A #4 (#13M) bar was used throughout and embedded for 6 in. (203 mm) in the box (i.e., 12 bar diameters). The test was performed with the same methods used for ASTM C234. A direct comparison was made of control versus vibrated specimens.

A research group recently used a modified pullout test to measure the bond strengths between glass fiber reinforced polymer (GFRP) reinforcing bars and concrete [(28)]. The study investigated the rib spacing and height of the GFRP bar. The pullout tests all resulted in pullout failures under a displacement controlled load application. The bar diameters varied from 0.32 in. (8 mm) to 0.5 in. (12 mm) and the cubes were 6 in. (152 mm). The embedment was 4 bar diameters for all specimens. A debonded region was left along the end of the rebar next to the reaction face of the specimen (i.e., the pulled end of the rebar). The bonding length was short enough to assume the bond stress was uniform throughout.

The body of research delineated above reveals that concentric bond tests continue to be used to compare differences in bond strengths. The ASTM C234 method was taken out of print, however altered versions of this method continue to be used. The RILEM method in particular has proven to be a valuable guideline as it reduces the effects of confinement stresses and has a short embedment length. The RILEM method also tends to produce pullout failures versus the splitting type of failure more commonly observed in specimens with longer embedment lengths and smaller amounts of clear cover.

CHARACTERISTICS OF DIFFERENTIAL DEFLECTIONS ON BRIDGE DECKS

Some studies have focused on the characteristics of differential deflections on bridge decks. The typical amplitudes, vibration frequencies, and intervals between deflections are of interest.

A series of tests were designed by Harsh and Darwin to study the vibration effects caused by traffic on bridge repairs. An intermittent vibration was applied every 4 minutes at a frequency of 0.5 Hz and amplitude of 0.5 in. (12.7 mm). A continuous vibration was applied at a frequency of 4.0 Hz and amplitude of 0.04 in. (1 mm). These corresponded to a peak particle acceleration of 15.6 in/sec^2 (39.6 cm/sec^2) and a peak particle velocity of 1.44 in/sec (3.66 cm/sec^2). This interval started 10 minutes after floating the concrete and continued for 30 hours. The specimens were not moved until they reached 3 ksi (21 MPa)[(20)].

In Texas a sample of overall midspan deflections were measured on 30 bridges. The results fostered a decision to use maximum deflections of 0.25 in. (6 mm) and 0.15 in. (4 mm) within the experimental testing program. An attempt was made to measure differential deflection in one bridge but it was reported that the entire closure pour moved together[13].

Whiffin and Leonard report that most bridges are stiff and heavy which greatly reduces vibration problems. Ten bridges were analyzed for peak deflections and frequencies. The maximum mean deflections ranged from 0.01 in. to 0.10 in. (0.28 mm to 2.6 mm). The maximum frequencies ranged from 2.3 Hz to 5.6 Hz[29].

A study in Georgia focused on identifying the maximum deflections on two continuous steel rolled shape girder bridges with spans of 70 ft (21.3 m) and 80 ft (24.4 m) respectively.   The maximum deflection for the two bridges was approximately 0.01 in. (0.28 mm). No additional information was provided on the girder sizes, girder spacing, or dimensions of the bridge[12].

Studies have been done to identify the typical vibration characteristics that average bridges experience. Generally, continuous bridges vibrate with a frequency of 2 to 5 Hz [30,4]. A study in Texas used a 1 Hz vibration to apply a reoccurring load to a beam test. This was based on a field study[13].

The reoccurring interval to apply traffic would depend on the expected traffic count during construction. An interval of 5 minutes was used on the Texas research[13]. This would correspond with an Average Daily Traffic (ADT) count of 288 vehicles which generate the type of deflections which are of interest.

CHAPTER 3. SPECIMEN DESIGN, FABRICATION, AND MATERIAL PROPERTIES

INTRODUCTION

A series of bond tests were performed to determine the reduction in bond strength caused by the deflection of the reinforcing bar embedded in typical bridge deck materials. A concentric pullout test was chosen as the basic test method to compare bond strengths. The pullout test was adjusted from standard procedures to create a differential deflection between the reinforcing bar and the embedment material. Three control and three deflected specimens were made for each embedment material. Nine different materials that may be used for bridge deck construction were used as the embedment material. Standard #4 (#13M) deformed reinforcing bars (rebar) were used throughout the study. During the pullout portion of the test, a deflection-controlled load was applied to the specimens and the ultimate capacity was compared among control and vibrated specimens.

SPECIMEN DESIGN

The bond test specimens were based on the layout of the ASTM C234 test. The ASTM C234 test method is a traditional concentric pullout test with a #6 (#19M) deformed reinforcing bar embedded concentrically in a 6 in. (152 mm) cube specimen. The bar extends out one side of the cube approximately ½ in. (13 mm), bonds throughout the entire length of the cube (6 in. (152 mm)), and extends out the opposite side approximately 18 in. (46 cm). The specimens were made such that the reinforcing bar was perpendicular to the side of the cube it entered as shown in Figure 3.

The ASTM C234 test specimen design and test setup were initially engaged for the present study; however, pilot test results led to test procedure alterations being implemented prior to the completion of the final tests. As explained in the Preliminary Tests section of Chapter 4, the initial tests failed in a splitting mode of failure. After analyzing the initial results, the embedment length was reduced to 3 in. (76 mm) or 6 bar diameters and the rebar size was reduced to a #4 (#13M) bar. A bond breaker was applied between embedment material and reinforcing bar for the first 3 inches (76 mm) next to the reaction face of the cube (Figure 4). The material used for the bond breaker was 3/8 in. (10 mm) thick, 1.5 in. (38 mm) outside diameter foam pipe insulation.

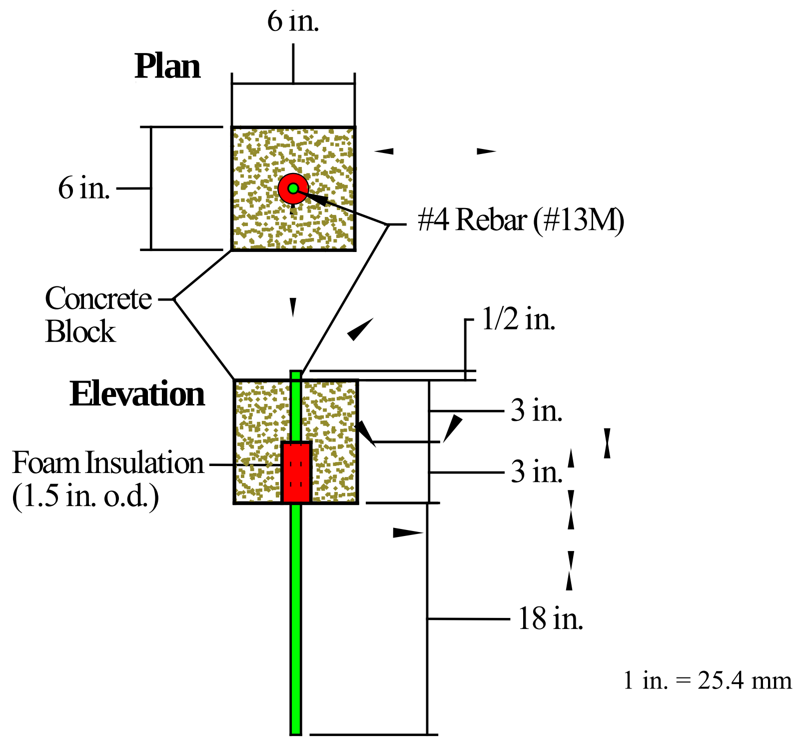

Figure 4. Illustration. Final pullout specimen geometry.

SPECIMEN FABRICATION

The specimens were cast and tested over a period of approximately six months in the spring and summer of 2011. A set of 6 specimens were cast during each material placement. The specimens were subjected to the differential deflection loading protocol, demolded, and tested at approximately 24 hours. This procedure was followed to ensure that testing occurred during the early age of each material. The tests were designed to focus on early age loss in bond strength.

Steel formwork was made for the six specimens, and illustrations of the formwork are provided in Figure 5. The forms were connected with bolts in a similar manner as shown in ASTM C234 and oiled with a form release agent prior to placement of the materials.

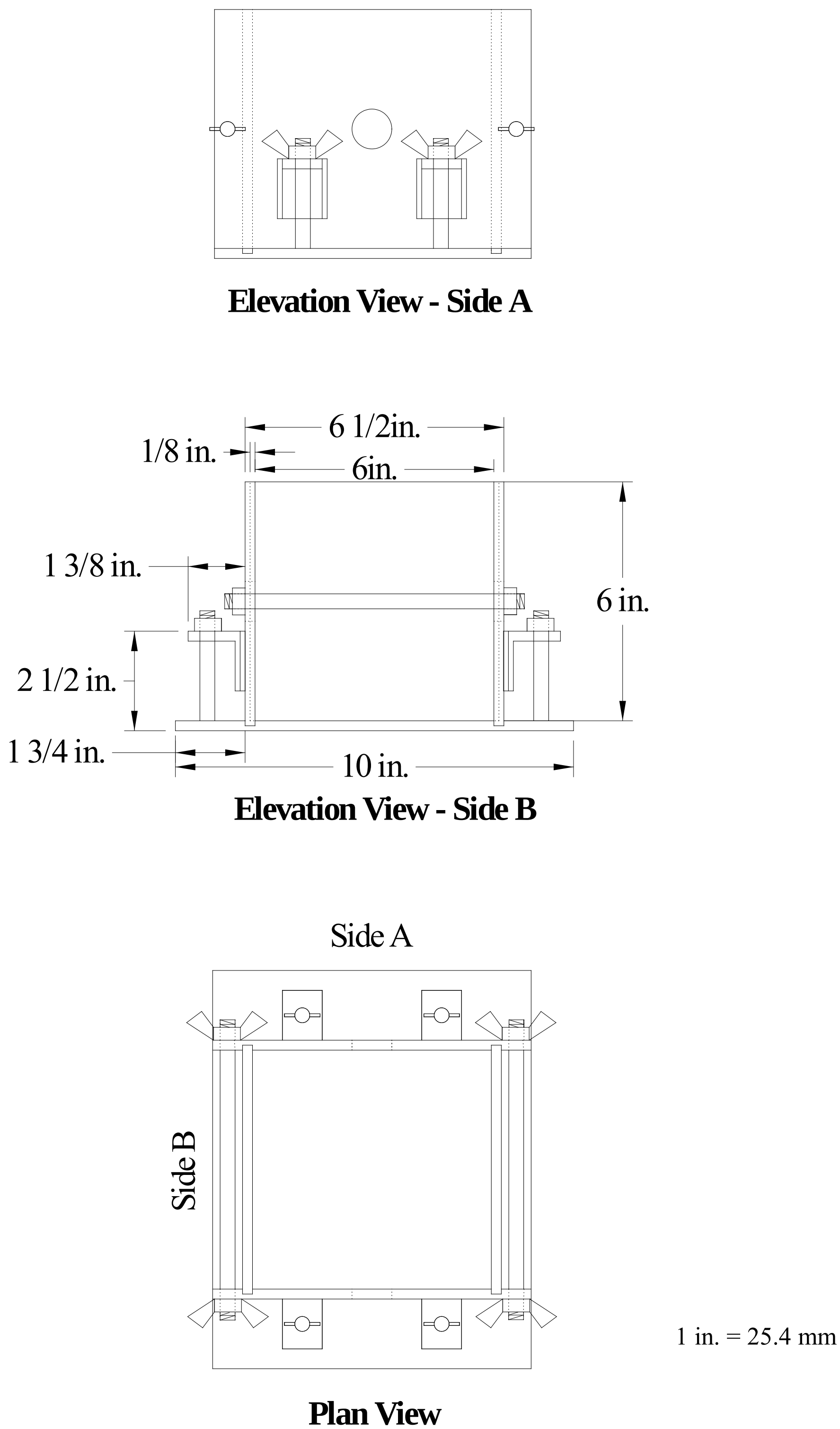

Figure 5. Illustration. Formwork for the pullout specimens.

Two groups of three specimens were made when each material was cast. For the first 3 specimens, referred to as the static specimens, a steel support was set approximately 1 ft (30 cm) from the adjacent face of the steel forms. The reinforcing bars were aligned inside the steel forms and then firmly attached to the steel support with screws. The reinforcing bar and embedment material were thus prevented from moving differentially during the placement and curing processes. This fixturing can be observed in Figure 6.

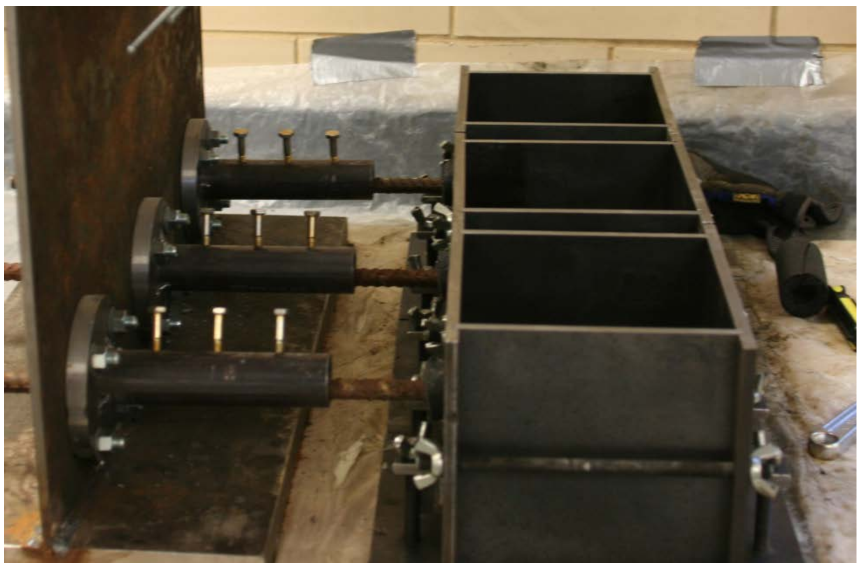

Figure 6. Photograph. Static specimen form setup.

The reinforcing bars in the second set of three specimens, referred to as the deflected specimens, were allowed to move differentially with respect to the steel cube formwork. A similar steel support was placed approximately 1 ft (30 cm) from these steel forms. The only difference between the supports was that four 4 $^{7}/_{8}$ in. (12.4 cm) by ¾ in. (1.9 cm) steel stiffeners were placed along the vertical member of this support. Figure 7 shows a prototype of the support prior to welding the four stiffeners perpendicular to the vertical member. This support was affixed to the platen of a MTS hydraulic testing machine. The steel cube forms were attached to the head of the hydraulic actuator. The reinforcing bars were fixed to the steel support and aligned to pass through the two outlets in the steel cube forms. The formwork was allowed to move with the hydraulic actuator while the reinforcing bars remained stationary as shown in Figure 7.

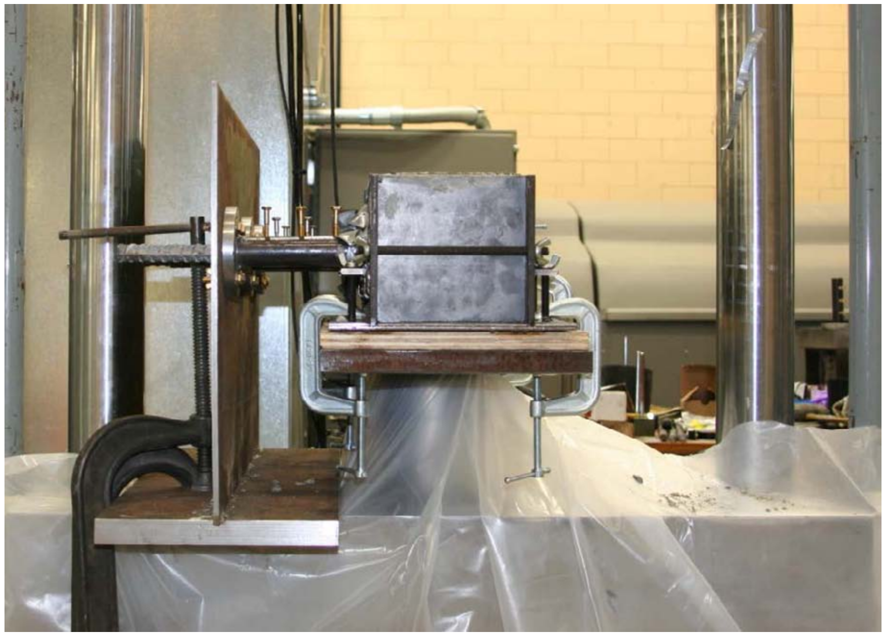

Figure 7. Photograph. Deflected specimen form setup with prototype rebar support.

The steel forms were prepared to remain water tight during the material placement. A layer of pliable caulking material (commonly called rope caulk) was applied between the reinforcing bar and the two outlets on the faces of the steel forms. The rope caulk was then coated in a silicon caulk to ensure no material leaked out of the forms. The steel formwork was coated with a layer of form oil.

Once the formwork was set up, preparations were made for placement of each field-cast grout material. The deflection induced by the hydraulic actuator was started prior to the placement of each material in the form. The actuator was programmed to induce a deflection based on total amplitude, frequency, and lag time between deflections.

The materials were mixed in the concrete materials laboratory at the Turner-Fairbank Highway Research Center (TFHRC). Immediately after mixing, the materials were placed into the forms in two layers. Each layer was rodded 25 times with a 5/8 in. (16 mm) diameter steel rod and the formwork was tapped with a rubber mallet approximately 5 times. Care was taken to not disturb the reinforcing bar during placement.

Approximately 30 minutes after placing the respective materials, plastic was placed over the specimens for the next 24 hours. Wet burlap was placed beneath the plastic when recommended by the manufacturer of the material. The forms were monitored to ensure material did not leak out of the forms or around the holes between the reinforcing bars and steel formwork.

Samples were taken to measure material properties. Nine specimens were made to measure compressive strength and three specimens were made for split cylinder strength. For grouts, 2 in. (5 cm) by 2 in. (5 cm) cubes were cast (ASTM C109), and for concretes, 4 in. (10 cm) by 8 in. (20 cm) cylinders were cast (ASTM C39). Split cylinder strengths were all measured with 4 in. (10 cm) by 8 in. (20 cm) cylinders (ASTM C496).

The set times were also measured for every material using ASTM C403 Standard Test Method for Time of Setting of Hydraulic Cement Paste. The initial and final set times were of interest. The purpose of the differential deflections was to disturb the bond up until the material reached final set. The set time measurements ensured that the material only experienced vibrations until final set and that the material was ready for testing by 24 hours.

Other material properties were also recorded for each material. This included temperature of the grout after mixing, unit weight, and the temperature of the laboratory. The initial rheology of each material was measured via ASTM C1437 both prior to dropping the table and then again after dropping the table 25 times.

The movement of the actuator was recorded to ensure that a continuous deflection was induced until final set of the materials. Both the amplitude and frequency of the deflection was recorded. In addition, the time between deflections was also recorded.

EMBEDMENT MATERIAL PROPERTIES

Nine different materials were used as embedment for the pullout specimens. This ranged from prebagged grouts, to UHPC, to typical bridge deck concretes. Each material was cast and tested independently but under the same basic procedures. The materials are listed in Table 1.

Table 1. Naming convention for the embedment materials.

Manufactured Name	Reference Name
Five Star Grout	G1
BASF Embeco 885 Grout	G2
BASF Set 45 Grout	M1
Five Star HP Epoxy Grout	E1
Euclid Euco Cable Grout PTX	T1
Lafarge Ductal JS1000	U1
Lafarge Ductal JS1000-RS	U2
Virginia A4 Concrete Mix Design – Normal Slump	C1
Virginia A4 Concrete Mix Design – High Slump	C2

Material G1

The first material, G1, was a high early strength, low shrinkage grout made by Five Star. This material was initially chosen as one of the standard grouts that have properties typical of grouts used on bridge projects.

Material G1 was used during the trial stages of the test setup and investigation of critical vibration values. Prior to recording the final test results for all of the other materials, numerous tests were made using material G1 to assist in identifying any problems with the test setup or mixing procedures. Once the setup was finalized, this grout was also used to test for critical differential deflection characteristics. Five batches were used for five different sets of tests to determine the critical vibration frequency and deflection amplitude.

The results from mixing five different batches of material G1 are listed in Table 2. The manufacturer's recommendations were followed for a fluid mix. The grout was placed in a pan mixer and then the mixer was started. Water was slowly added over the next 30 seconds. The mixing time was 5 minutes. The water was left at room temperature, 71 to 77°F (21.8 to 25.0°C), for approximately 24 hours prior to placing the grouts. The grout gained approximately 5°F (2.8°C) during the mix procedure. The mixes were all workable. The spread measurements were slightly less than 5 in. (12.6 cm) without dropping the table and surpassed the 10 in. (25.4 cm) limit after dropping the table 25 times. The unit weight was approximately 130 lb/ft^3 (2100 kg/m^3) for all mixes.

Table 2. Material G1 mixing conditions.

Placement Date	Grout, lbs (kg)	Water, lbs (kg)	Mix Time, Mins.	Lab Temp., °F (°C)	Grout Temp. After Mixing, °F (°C)	Spread -NO- Table Drops, in. (cm)	Spread -25- Table Drops, in. (cm)	Unit Weight, lb/ft3 (kg/m^3)
April 27, 2011	117 (53.0)	21.1 (9.6)	5	77.2 (25.1)	80.1 (26.7)	4.4 (11)	9.25 (24)	131 (2100)
May 5, 2011	117 (53.2)	21.1 (9.6)	5	75.2 (24.0)	80.8 (27.1)	4.4 (11)	10 (25)	1328 (2110)
June 7, 2011	129 (58.4)	23.2 (10.5)	5	72.9 (22.7)	77.9 (25.5)	4.7 (12)	10 (25)	131 (2100)
June 13, 2011	129 (58.4)	23.2 (10.5)	5	72.0 (22.2)	76.7 (24.8)	*	10 (25)	131 (2100)
June 16, 2011	129 (58.4)	23.2 (10.5)	5	71.4 (21.9)	76.7 (24.8)	*	10 (25)	132 (2110)

* Measurement not completed.

The material properties for G1 were obtained approximately one day after casting (Table 3). The compressive strengths ranged from 3250 to 5400 psi (22.4 to 37.2 MPa) at testing. The split cylinder strength varied between 360 and 460 psi (2.48 and 3.17 MPa). This grout exhibited usable strengths within 24 hours as expected for a high early strength grout.

Table 3. G1 material properties.

Placement Date	Age, Hours	Compressive Strength, psi (MPa)	Split Cylinder, psi (MPa)
April 27, 2011	24.0	3250 (22.4)	
	24.0		460 (3.17)
May 19, 2011	24.0	3770 (26.0)	
	26.3	3740 (25.8)	
	24.5		420 (2.90)
June 7, 2011	23.9	5040 (34.7)	
	28.2	5400 (37.2)	
	28.2		450 (3.10)
June 13, 2011	24.0	4120 (28.4)	
	28.0	4830 (33.3)	
	28.0		430 (2.96)
June 16, 2011	24.7	3820 (26.3)	
	26.4	4320 (29.8)	
	26.8		360 (2.48)

The initial and final sets were measured for each material placement. The plot of resistance versus time for material G1 poured on May 5, 2011 demonstrates a typical plot as shown in Figure 8. The resistance is plotted over time and a curve is fitted to the data. The curves are typically second or third order curves in the algebraic form $y = m_1x^2 + m_2x + b$ or $y = m_1x^3 + m_2x^2 + m_3x + b$, respectively. The third order curves were used with the fastest setting grouts because of the rapid increase in strength over a short period of time. The points where the curve crosses the 500 psi (3.45 MPa) and the 4000 psi (27.6 MPa) lines correspond to initial and final set, respectively.

Table 4 shows the initial and final set times for material G1 placements. The initial sets all occurred within a 33 minutes window between 4.7 and 5.25 hours after mixing. The final sets varied about one hour between 5.94 and 6.92 hours after mixing.

Table 4. ASTM C403 set times for material G1.

Placement Date	Initial Set, Hours	Final Set, Hours
April 27, 2011	4.95	6.92
May 19, 2011	4.70	5.98
June 7, 2011	5.25	6.70
June 13, 2011	5.22	6.70
June 16, 2011	5.18	6.43

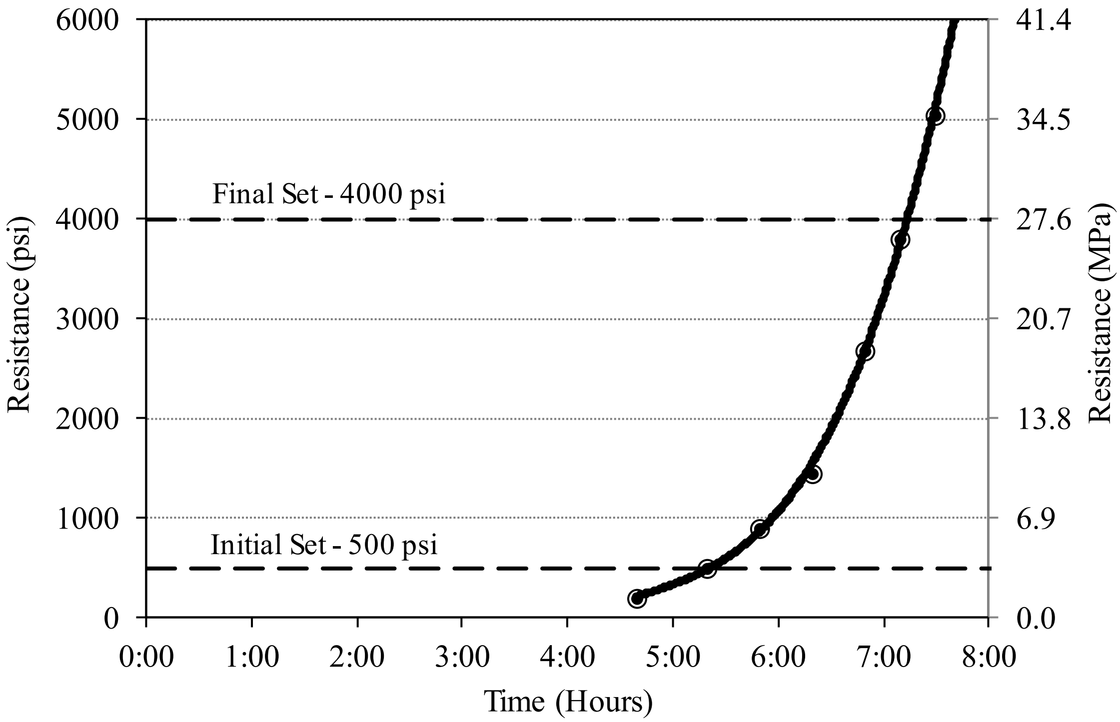

Figure 8. Graph. Typical ASTM C403 set time results.

Material G2

Material G2 is a prebagged high early strength rapid setting grout marketed by BASF under the label of Embeco 885. This grout, similar to material G1, exhibited typical properties needed for use on a bridge construction project. It was chosen as a representative product for its category with the realization that numerous other similar products could have also been chosen.

The results from mixing two batches of this grout are listed in Table 5. A pan mixer was used to mix the components. The grout was added to the mixer. The mixer was then started and the water was added over the next 30 seconds. The mix was then mixed for a total of 5 minutes. The water was left at room temperature, about 76°F (24°C), for approximately 24 hours prior to placing the concrete. The mixes were both workable and exhibited spreads of 10 in. (25 cm) without dropping the table. The unit weight averaged 149 lb/ft^3 (2390 kg/m^3) for both mixes.

Table 5. Material G2 mixing conditions.

Placement Date	Grout, lbs (kg)	Water, lbs (kg)	Mix Time, Mins.	Lab Temp., °F (°C)	Grout Temp. After Mixing, °F (°C)	Spread -NO- Table Drops, in. (cm)	Spread -25- Table Drops, in. (cm)	Unit Weight, lb/ft^3 (kg/m^3)
July 26, 2011	147 (66.9)	24.5 (11.2)	5	76.1 (24.5)	79.3 (26.3)	10 (25)	10 (25)	149 (2390)
July 28, 2011	147 (66.9)	24.7 (11.3)	5	75.0 (23.9)	75.3 (23.9)	10 (25)	10 (25)	149 (2390)

The properties for material G2 were obtained about 24 hours after casting (Table 6). The compressive strengths ranged from 5520 to 6190 psi (38.1 to 42.7 MPa) at testing. Split cylinder strengths were both approximately 470 psi (3.24 MPa). This grout exhibited rapid strength gain sufficient for creating a bond to the reinforcing bars within 24 hours.

Table 6. Material G2 properties.

Placement Date	Age, Hours	Compressive Strength, psi (MPa)	Split Cylinder, psi (MPa)
July 26, 2011	26.1	6190 (42.7)	
	25.6		469 (30.2)
July 28, 2011	25.6	5520 (38.1)	
	27.0	5790 (39.9)	
	27.2		471 (30.3)

Table 7 shows the initial and final set times for the G2 material placements. The initial set occurred approximately 9 hours after mix initiation. The final set occurred approximately 10 hours after mix initiation. The initial set took longer than other materials but the time to reach final set was very rapid after initial set. The first set of data on July 26, 2011 was only taken through 7.7 hours after casting. The grout had not reached initial set at this point in time, however the data that was available followed a similar pattern to the data from the second batch. The results from July 28, 2011 are based on a full set of data.

Table 7. ASTM C403 set times for material G2.

Placement Date	Initial Set, Hours	Final Set, Hours
July 26, 2011	*	*
July 28, 2011	8.82	10.02

*Incomplete set of data for this placement.

Material M1

Material M1 is a magnesium phosphate based grout produced by BASF under the name Set 45. This material was selected as a typical magnesium phosphate grout with representative properties.

On the first cast, the pouring of all relevant specimens was attempted. However, the extremely short working time afforded with the grout prevented successful casting of all specimens. In subsequent casts, three separate mixes were made for each overall casting. With each mix, one vibrated and one control pullout specimen was made along with three cubes. These specimens were all cast and poured with 30 minutes of one another. Only one complete set of tests (six pullout specimens) was performed.

The results from material M1 are listed in Table 8. The manufacturer’s recommendations were followed for a fluid mix. The grouts were placed in a 5 gallon plastic bucket, the water was added, and a drill powered paint mixer was used to mix the grout. The grout was mixed for approximately 2.5 minutes and then immediately poured into the specimens. Because the material set so rapidly, it could not be mixed in the pan mixer in the concrete materials lab. There was not sufficient time to remove the material, clean the tools, and place the material in the specimens. The room temperature in laboratory were the material was mixed was approximately 65.0°F (18.3°C). The grout temperature after mixing, spread measurements, and unit weight could not be obtained because of a lack of time. The mix was easy to pour for approximately 5 minutes after mixing.

Table 8. Material M1 mixing conditions.

Placement Date	Grout, lbs (kg)	Water, lbs (kg)	Mix Time, Mins.	Lab Temp., °F (°C)	Grout Temp. After Mixing, °F (°C)	Spread -NO- Table Drops, in. (cm)	Spread -25- Table Drops, in. (cm)	Unit Weight, lb/ft^3 (kg/m^3)
July 7, 2011	62.5 (28.3)	5.51 (2.5)	2.5	65.0 (18.3)	*	*	*	*

*These properties could not be measured because the material set too fast.

Only compressive strength was obtained from material M1. The split cylinder strength was not obtained because there was not enough work time to make cylinders. Properties were obtained approximately 24 hours after casting (Table 9). The average compressive strength was 5060 psi (34.9 MPa) at 22 hours. This grout gained strength very quickly even though it was not tested until 22 hours after pouring.

Table 9. Material M1 grout material properties.

Placement Date	Age, Hours	Compressive Strength, psi (MPa)	Split Cylinder, psi (MPa)
June 19, 2011	22.0	5060 (34.9)†	*

*Specimens were not produced or tested.
†This was the average of 3 separate mixes made within a 30 minute interval

Table 10 shows the initial and final set times for material M1 placements. The initial set occurred at about 7 minutes while the final set occurred less than a minute later. These values are estimates considering how fast the grout set. The time between obtaining the first penetrometer reading at 7 minutes and the second reading was about a minute, however the grout went from initial set to significantly past final set in this time frame. The penetrometer test is not effective at obtaining precise readings for materials that set this rapidly. The test did show that within ten minutes the grout was well past a final set condition.

Table 10. ASTM C403 set times for material M1.

Placement Date	Initial Set, Hours	Final Set, Hours
June 19, 2011	0.12	0.13

Material E1

Material E1 is an epoxy grout made by Five Star named HP Epoxy Grout. This material was selected as a typical epoxy grout with representative properties.

Material E1 had rapid strength gains within the first 24 hours. The strength was large enough to create a bond stronger than the tensile strength of the reinforcing bars in the pullout tests. These results are presented in Chapter 4. Because of the high strength gain, only one set of tests were performed.

The results from the batch of material E1 are listed in Table 11. The manufacturer's recommendations were followed for a fluid mix. The components for the two part epoxy were first mixed with a drill operated paint mixer in a 3 gallon (19 L) bucket. The grout aggregate was then placed in a pan mixer, the mixer was started, and the mixed epoxy from the bucket was added over the next 30 seconds. The total mixing time was 3.5 minutes. The room temperature was 72.3°F (22.4°C) and the grout temperature after mixing was 78.8°F (26.0°C). The mix was easy to pour however the spread was 5.6 in. (14 cm) without dropping the table and 6.4 in. (16 cm) after dropping the table 25 times. The unit weight was approximately 136 lb/ft^3 (2180 kg/m^3).

Table 11. Material E1 mixing conditions.

Placement Date	Grout, lbs (kg)	Water, lbs (kg)	Mix Time., Mins.	Lab Temp., °F (°C)	Grout Temp. After Mixing, °F (°C)	Spread -NO- Table Drops, in. (cm)	Spread -25- Table Drops, in. (cm)	Unit Weight, lb/ft^3 (kg/m^3)
June 19, 2011	*	*	3.5	72.3 (22.4)	78.8 (26.0)	5.6 (14)	6.4 (16)	136 (2180)

*Premeasured grout and epoxy components – No water added

Only compressive strength was obtained from material E1. The cube specimens were damaged at demolding because the grout adhered to the steel formwork. The three cylinders were used to measure strength instead of the cubes. The split cylinder strength was not obtained because there were no cylinders available. Properties were obtained approximately six hours and one day after casting (Table 12). The compressive strengths were 10,100 psi (69.7 MPa) at 6 hours and 12,100 psi (83.3 MPa) at 24 hours. This grout exhibited very high strengths within 6 hours.

Table 12. Material E1 material properties.

Placement Date	Age, Hours	Compressive Strength, psi (MPa)
June 19, 2011	6.0	10,100 (69.7)
	25.1	12,100 (83.3)

Table 13 shows the initial and final set times for material E1 placements. The initial set occurred at about 2 hours while the final set occurred about 15 minutes later. Once the material began to set, data had to be taken at 5 minute intervals for about 30 minutes to obtain a set curve. The penetrometer readings were off the scale within 30 minutes after initial set.

Table 13. ASTM C403 set times for material E1.

Placement Date	Initial Set, Hours	Final Set, Hours
June 19, 2011	2.08	2.33

Material T1

Material T1 is a prebagged grout mix made by the Euclid Chemical Company under the name Euco Cable Grout PTX. This material was selected as a typical cable grout with representative properties.

Two preliminary casts were attempted. Upon each trial, material T1 exhibited significant surface cracking on all of the specimens (Figure 9). When testing the pull out specimens, the samples split apart at the surface cracks and did not provide significant bond strength. The material also did not have sufficient strength for testing at 24 hours. It was determined that material T1 would not provide sufficient bond strength for this application. The material properties are presented below for the two preliminary tests, however this material was not included in the final test matrix. The set times and strengths were not recorded.

Figure 9. Photograph. End view of a specimen made with material T1 at demolding.

The results from two preliminary tests with material T1 are listed in Table 14. The manufacturer's recommendations were followed for a fluid mix. The grouts were placed in a pan mixer, the mixer was started, and then the water was added over a period of 30 seconds. The grout was mixed for a total of 5 minutes. The room temperature in the materials laboratory where the material was mixed was approximately 75.0°F (23.9°C). The material was very workable as exhibited by an average 9.5 in. (24 cm) spread measurement without dropping the table. The unit weight averaged 117 lb/ft^3 (1870 kg/m^3) making this the lightest material tested.

Table 14. Material T1 mixing conditions.

Placement Date	Grout, lbs (kg)	Water, lbs (kg)	Mix Time, Mins.	Lab Temp., °F (°C)	Grout Temp. After Mixing, °F (°C)	Spread -NO- Table Drops, in. (cm)	Spread -25- Table Drops, in. (cm)	Unit Weight, lb/ft^3 (kg/m^3)
April 5, 2011	125 (56.8)	30 (13.6)	5	75.2 (24.0)	73.9 (23.3)	9 (23)	10 (25)	117 (1870)
April 12, 2011	125 (56.8)	30 (13.6)	5	75.7 (24.3)	85.1 (29.5)	10 (25)	10 (25)	117 (1880)

Material U1

Material U1 is an ultra high performance concrete with standard set times made by Lafarge under the name Ductal JS1000. The results from mixing two different batches are listed in Table 15. The manufacturer's recommendations were followed throughout. A pan mixer was used to mix the components. Half of the superplasticizer and the water were poured in the premix during the first two minutes. At approximately 6.5 minutes the rest of the superplasticizer was added. The mix reached a paste at approximately 30 minutes and the fibers were added a minute later. The fibers were added over a course of 2 minutes at a uniform rate. The total mixing time was about 36 minutes for each batch. The water was left at room temperature, 72°F (21.7°C), for approximately 24 hours prior to placing material U1. The grout gained approximately 15°F (8.5°C) during the mix procedure. The mixes were both workable. The spread measurements were between 7.0 in. (18 cm) and 8.3 in. (21 cm) without dropping the table. Both batches reached the spread limit of 10 in. (25.4 cm) after dropping the table 25 times. The unit weight was 156.2 lb/ft^3 (2100 kg/m^3) for the second mix.

Table 15. Material U1 standard set mixing conditions.

Placement Date	Premix, lbs (kg)	Water / Super-Plasticizer, lbs (kg)	Fibers, lbs (kg)	Mix Time., Mins.	Lab Temp., °F (°C)	Temp. After Mixing, °F (°C)	Spread -NO- Table Drops, in. (cm)	Spread -25- Table Drops, in. (cm)
June 21, 2011	171 (77.7)	10.1 / 2.34 (4.58/1.06)	15.2 (6.90)	35	71.7 (22.1)	87.2 (30.7)	8.3 (21)	10 (25)
June 27, 2011	171 (77.7)	10.1 / 2.34 (4.58/1.06)	15.2 (6.90)	37	*	87.0 (30.6)	7.0 (18)	10 (25)

* Result not recorded.

Material properties could not be obtained for material U1 until about two days after casting (Table 16). Demolding was not possible prior to this time without damaging the specimens. The compressive strengths ranged from 5030 to 6070 psi (34.7 to 41.9 MPa) at testing. ASTM C496 split cylinder strengths were not obtained for material U1 because of its fiber matrix. This grout exhibited usable strengths somewhat slower than other materials but still in less than 48 hours.

Table 16. Material U1 standard set material properties.

Placement Date	Age, Hours	Compressive Strength, psi (MPa)	Split Cylinder, psi (MPa)
June 21, 2011	42.8	5030 (34.7)	*
	44.0	5270 (36.3)	
June 27, 2011	43.8	5600 (38.6)	*
	44.6	6070 (41.9)	

*ASTM C496 split cylinder strength result was not obtained for UHPC

Table 17 shows the initial and final set times for material U1 placements. The initial sets all occurred between 7 and 9 hours after mixing. The final sets varied between 15 and 19 hours after mixing. These set times are estimates. Because of the long set times, data values were not uniformly spaced up to the initial and final set times. The set times occurred in the middle of the night so the reported set times were extrapolated from the data obtained.

Table 17. ASTM C403 set times for material U1.

Placement Date	Initial Set, Hours	Final Set, Hours
June 21, 2011	8.77	18.32
June 27, 2011	7.85	15.52

Material U2

Material U2 is an ultra high performance concrete with rapid set times made by Lafarge under the name Ductal JS1000-RS. The results from mixing one batch are listed in Table 18. The manufacturer's recommendations were followed throughout. A pan mixer was used to mix the components. The admixtures and the water were poured into the premix during the first two minutes. The mix reached a paste at approximately 14.5 minutes and the fibers were added two minutes later. The fibers were added over a course of 1 minute at a uniform rate. The total mixing time was about 19.5 minutes. The water was left at room temperature, 75°F (24°C), for approximately 24 hours prior to placing material U2. The mix was very workable. The spread measurement reached the limit of 10 in. (25.4 cm) without dropping the table. The unit weight was 152 lb/ft^3 (2430 kg/m^3) for the mix.

Table 18. Material U2 mixing conditions.

Placement Date	Premix, lbs (kg)	Water / Super-Plasticizer, lbs (kg)	Fibers, lbs (kg)	Accele-rator, lbs (kg)	Mix Time, Mins.	Lab Temp., °F (°C)	Spread -NO-Table Drops, in. (cm)	Unit Weight, lb/ft^3 (kg/m^3)
August 9, 2011	137 (62.3)	9.36 / 1.12 (40.3 / 0.51)	8.96 (4.07)	0.75 (0.34)	19.5	75.3 (24.1)	10 (25)	152 (2430)

Material properties were obtained about one day after casting for material U2 (Table 19). The compressive strength was 13,800 psi (270 MPa) at testing. Traditional split cylinder strengths are not obtained for material U2 because of its fiber matrix. This grout exhibited usable strengths within 24 hours much like the other grouts.

Table 19. Material U2 material properties.

Placement Date	Age, Hours	Compressive Strength, psi (MPa)	Split Cylinder, psi (MPa)
August 9, 2011	27.25	13,800 (95.0)	*
	30.25	14,600 (101)	*

*ASTM C496 split cylinder strength was not obtained for UHPC

Table 20 shows the initial and final set time for material U2 placements. The initial set occurred at 1.18 hours after mix initiation. The final set occurred 4.98 hours after mix initiation. These values confirm that material U2 sets much faster and gains strength much quicker as compared to material U1 (the other UHPC material) as well as some other grouts. As compared to material U1, material U2 would seem more generally appropriate for accelerated bridge construction based on strength and set time.

Table 20. ASTM C403 set times for material U2.

Placement Date	Initial Set, Hours	Final Set, Hours
June 21, 2011	1.18	4.98

Material C1
Material C1 is an average slump concrete design based on the Virginia Department of Transportation's (VDOT) A4 concrete. The constituent materials for the deck concrete were obtained from local suppliers in the Washington, D.C. metropolitan region.

The results from mixing two batches of material C1 are listed in Table 21. A concrete drum mixer was used to mix the components. The aggregates were pre-soaked to a saturated surface dry condition. The aggregates and half of the water were poured into the mixer. The mixer was then started and the cement and the other half of the water were added over the next minute. The materials were then mixed for 3 minutes, rested for 2 minutes, and mixed for an additional 2 minutes. The water was left at room temperature, 76°F (24°C), for approximately 24 hours prior to placing the concrete. The mixes were both workable and had slumps of 4 in. (10 cm) and 4.5 in. (11 cm), respectively. The unit weight was approximately 160 lb/ft^3 (2560 kg/m^3) for both mixes.

Table 21. Material C1 mixing conditions.

Placement Date	Cement, lbs (kg)	Water, lbs (kg)	Course Agg., lbs (kg)	Fine Agg., lbs (kg)	Mix / Rest / Mix Time, Mins.	Lab Temp., °F (°C)	Slump, in. (cm)	Unit Weight, lb/ft^3 (kg/m^3)
July 12, 2011	41.2 (18.7)	18.5 (8.39)	131 (59.4)	64.4 (29.2)	3/2/2	75.5 (24.2)	4 (10)	160 (2560)
July 14, 2011	41.2 (18.7)	18.5 (8.39)	131 (59.4)	64.4 (29.2)	3/2/2	76.1 (24.5)	4.5 (11)	161 (2570)

Material properties were obtained approximately 24 hours after casting material C1 as shown in Table 22. The compressive strengths ranged from 2260 to 2630 psi (15.6 to 18.1 MPa) at testing. Split cylinder strengths ranged from 310 to 350 psi (2.1 to 2.4 MPa). This material exhibited a slower strength gain than the grouts but still sufficient strength to create a bond to the reinforcing bars by 24 hours after casting.

Table 22. Material C1 material properties.

Placement Date	Age, Hours	Compressive Strength, psi (MPa)	Split Cylinder, psi (MPa)
July 12, 2011	24.1	2460 (17.0)	
	26.6	2630 (18.1)	
	27.1		350 (2.4)
July 14, 2011	24.5	2260 (15.6)	
	27.1	2390 (16.5)	
	27.8		310 (2.1)

Table 23 shows the initial and final set times for the average slump concrete placements. The initial sets both occurred at 3.75 hours after mix initiation. The final sets varied between 5.83 and 6.08 hours after mix initiation.

Table 23. ASTM C403 set times for material C1.

Placement Date	Initial Set, Hours	Final Set, Hours
July 12, 2011	3.77	5.83
July 14, 2011	3.73	6.08

Material C2

Material C2 is a high slump concrete design based on the Virginia Department of Transportation's (VDOT) A4 concrete. The constituent materials for the deck concrete were obtained from local suppliers in the Washington, D.C. metropolitan region.

The results from mixing two batches are listed in Table 24. A concrete drum mixer was used to mix the components. The coarse aggregate was pre-soaked to a saturated surface dry condition. The aggregates and half of the water were poured into the mixer. The mixer was then started and the cement and other half of the water were added over the next minute. The mix was then mixed for 3 minutes, rested for 2 minutes, and mixed for an additional 2 minutes. If needed, additional water was added to reach the desired 8 in. (20 cm) slump. The water was left at room temperature, 76°F (24°C), for approximately 24 hours prior to placing the concrete. The mixes were both workable and had slumps of 9 in. (10 cm) and 8 in. (11 cm), respectively. The unit weight averaged 158 lb/ft^3 (2560 kg/m^3) for both mixes.

Table 24. Material C2 mixing conditions.

Placement Date	Cement, lbs (kg)	Water, lbs (kg)	Course Agg., lbs (kg)	Fine Agg., lbs (kg)	Mix / Rest / Mix Time, Mins.	Lab Temp., °F (°C)	Slump, in. (cm)	Unit Weight, lb/ft^3 (kg/m^3)
August 2, 2011	41.2 (18.7)	18.5 (8.39)	131 (59.4)	64.4 (29.2)	3/2/2	78 (25.6)	9 (23)	155 (2480)
August 4, 2011	41.2 (18.7)	19.9 (9.03)	131 (59.4)	64.4 (29.2)	3/2/2	76.3 (24.6)	8 (20)	162 (2530)

The material properties were obtained around 24 hours after casting material C2 (Table 25). The compressive strengths ranged from 1390 to 1910 psi (9.6 to 13.2 MPa) at testing. Split cylinder strengths ranged from 252 to 270 psi (1.7 to 1.9 MPa). Material C2 exhibited a slower strength gain than the grouts but still sufficient strength to create a bond to the reinforcing bars.

Table 25. Material C2 material properties.

Placement Date	Age, Hours	Compressive Strength, psi (MPa)	Split Cylinder, psi (MPa)
August 2, 2011	24.6	1390 (9.6)	
	26.5	1620 (11.1)	
	26.9		252 (1.7)
August 4, 2011	24.6	1670 (11.5)	
	26.8	1910 (13.2)	
	27.0		270 (1.9)

Table 26 shows the initial and final set times for material C2 placements. The initial sets both occurred between 4.22 and 4.95 hours after mix initiation. The final sets varied between 5.87 and 6.18 hours after mix initiation. Compared to material C1, the lower slump concrete deck mix, the initial set was delayed by over an hour but the final set times were approximately the same.

Table 26. ASTM C403 set times for material C2.

Placement Date	Initial Set, Hours	Final Set, Hours
August 2, 2011	4.87	6.18
August 4, 2011	4.22	5.87

Summary of Material Properties

Figure 10 shows the variations in flow measurements for the different grout materials. Note materials C1 and C2 were not measured because the test is not applicable to concrete. All materials except M1 and E1 had full spread measurements (10 in. (25.4 cm)) with the 25 table drops. Material M1 reached initial set so fast that the measurement could not be taken and material E1 expressed little flow with or without the table drops. The spread of material G1 more than doubled after the table drops. Material U1 had about a 25% increase in spread with the table drops. Materials G2 and U2 flowed to the maximum spread without dropping the table. Note that the individual batches are labeled in the figure by the material designation followed by the placement number.

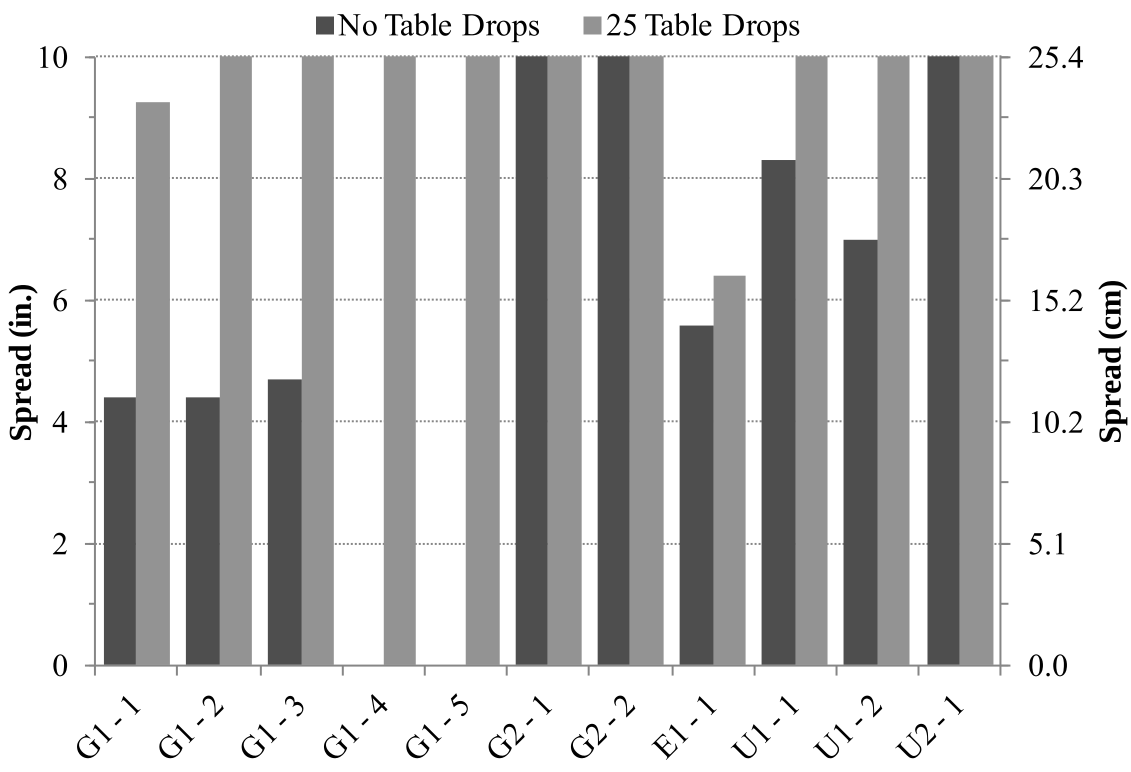

Figure 10. Graph. Spread measurements for the grout materials.

Figure 11 shows a comparison of the initial and final set times for each material. Materials U1 and G2 experienced the slowest initial set with average times of greater than 8 hours. Materials M1, U2, and E1 had initial sets of approximately 2 hours or less. The final set of most materials was between 4 and 7 hours. Materials U1 and G2 had the slowest final sets of 10 hours or more. The fastest final set was material M1 followed by material E1. Again, the individual batches within the figure are labeled based on the material designation followed by the placement number.

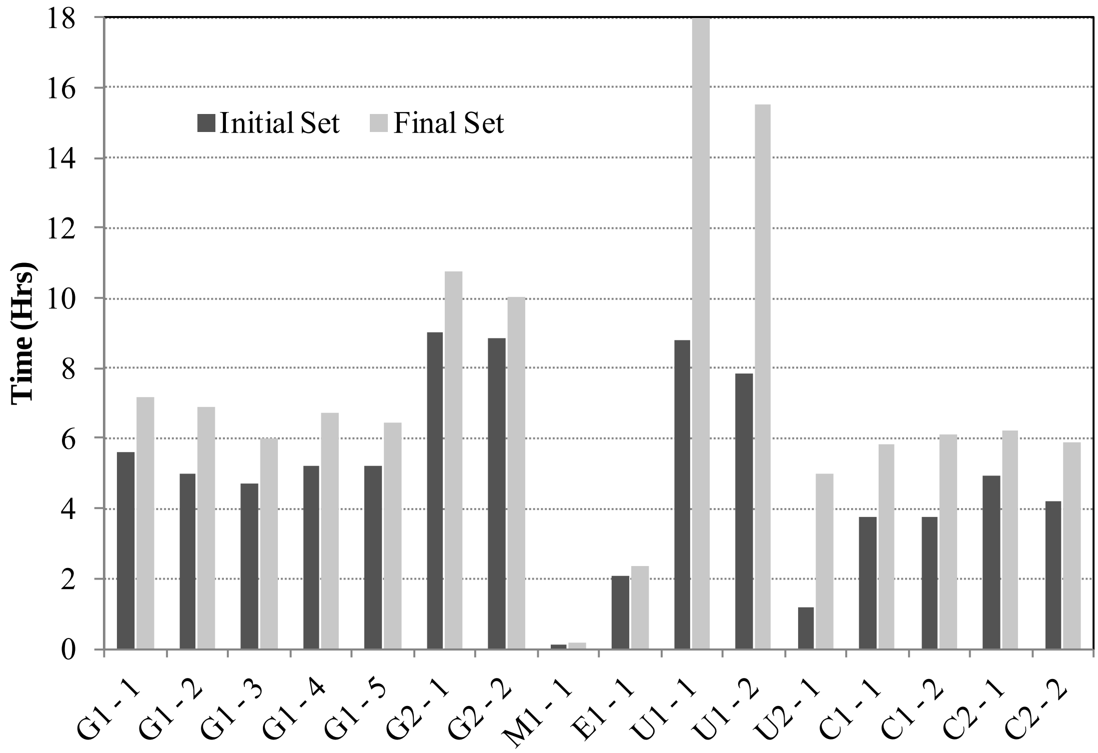

Figure 11. Graph. Initial and final set times for embedment materials.

Figure 12 shows the compressive strengths at testing ranged from about 1500 psi (12 MPa) to 14,000 psi (115 MPa). Testing occurred approximately 24 hours after material placement for most materials. The one exception was U1, wherein the testing occurred at approximately 44 hours after mix initiation because the specimens could not be demolded earlier. Materials C1 and C2 had the lowest compressive strength at testing throughout. U2 and E1 had the highest compressive strengths at testing. Most of the other materials ranged in strength from 3000 psi (22 MPa) to 6000 psi (46 MPa).

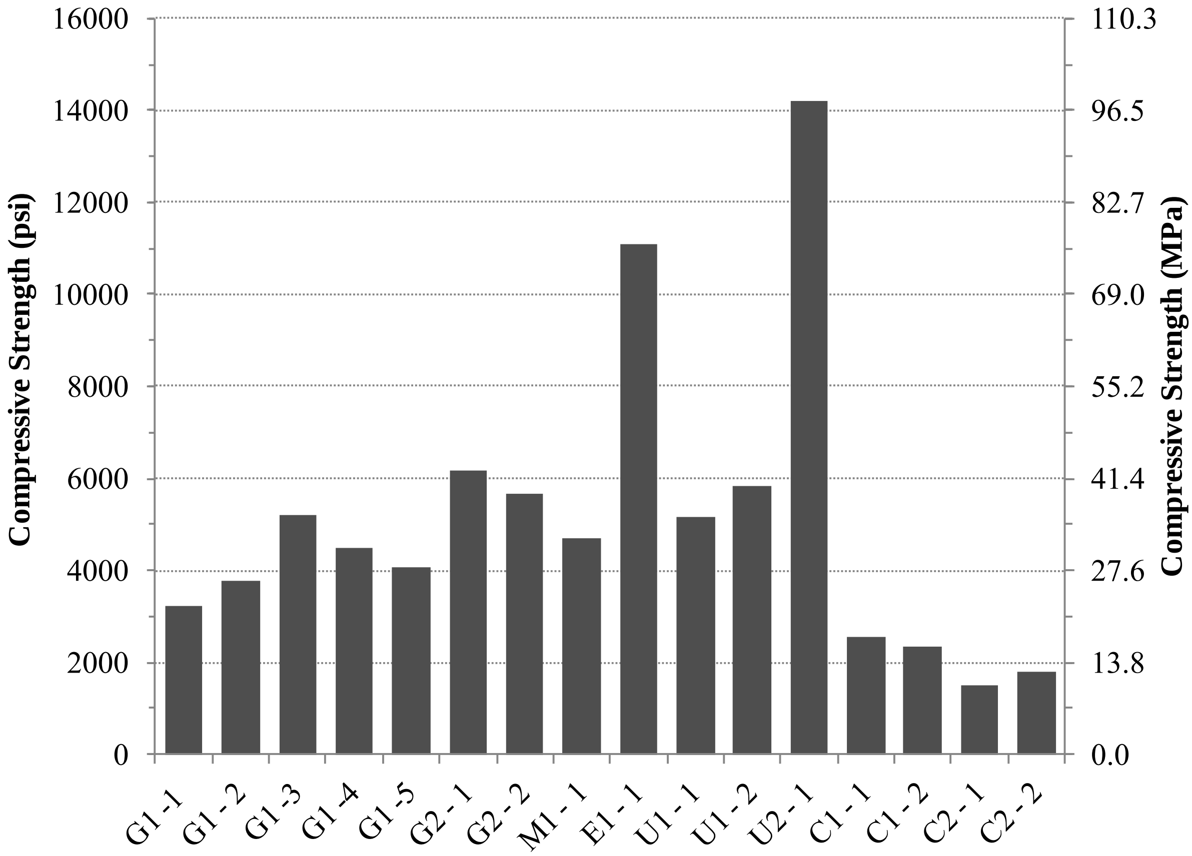

Figure 12. Graph. Compressive strengths of the embedment materials at time of pullout testing.

REINFORCING BAR PROPERTIES

The steel used for the reinforcing bars in the pullout specimens was designed as a constant within the testing program. The reinforcing bars were all #4 (#13M), Grade 60, with the same rib pattern. The material was purchased as a single delivery from an individual supplier. All of the reinforcing bars had a light coating of surface rust at the time of specimen casting and testing. As discussed later, note that some preliminary tests used #6 (#18M) bars, but this bar size was deemed to be too large based on the embedment material splitting responses observed.

The diameters were measured at six locations in a sample of reinforcing bars to confirm the height of the ribs. The diameter of the core of the bar and the peak of the ribs were measured. The height of the rib was also calculated. These results are provided in Table 27.

Table 27. Reinforcing bar dimensions.

Core Diameters, in. (cm)	Rib Diameters, in. (cm)	Rib Heights, in. (cm)	Average Core Diameter, in. (cm)	Average Rib Diameter, in. (cm)	Average Rib Height, in. (cm)
0.4655 (1.182)	0.5255 (1.335)	0.0600 (0.1524)	0.4651 (1.181)	0.5292 (1.344)	0.0641 (0.163)
0.4640 (1.179)	0.5265 (1.337)	0.0625 (0.1588)			
0.4665 (1.185)	0.5310 (1.349)	0.0645 (0.1638)			
0.4660 (1.184)	0.5300 (1.346)	0.0640 (0.1626)			
0.4650 (1.181)	0.5310 (1.349)	0.0660 (0.1676)			
0.4635 (1.177)	0.5310 (1.349)	0.0675 (0.1715)			

Uniaxial tension tests were performed on three samples of the reinforcing bar. The stress versus strain response was captured and the yield and ultimate strengths were determined. These values are reported in Table 28.

Table 28. Reinforcing bar tensile strength results.

Yield Stress, psi (MPa)	Ultimate Stress, psi (MPa)	Average Yield Stress, psi (MPa)	Average Ultimate Stress, psi (MPa)
68,000 (469)	113,000 (779)	68,200 (470)	114,000 (784)
68,500 (472)	116,000 (796)		
68,000 (469)	114,000 (783)		

IMPARTED DIFFERENTIAL DEFLECTIONS

The purpose of the pullout tests was to compare a disturbed bond with a standard bond formed under static conditions. Differential deflections were imparted to half of the specimens via a computer controlled hydraulic actuator. The motions of each deflection were designed to follow a sine curve as shown in Figure 13. A delay was imparted between each individual sine curve to replicate intermittent traffic on a bridge. The amplitude and frequency of differential deflections were recorded periodically before and during the casting and setting of each specimen.

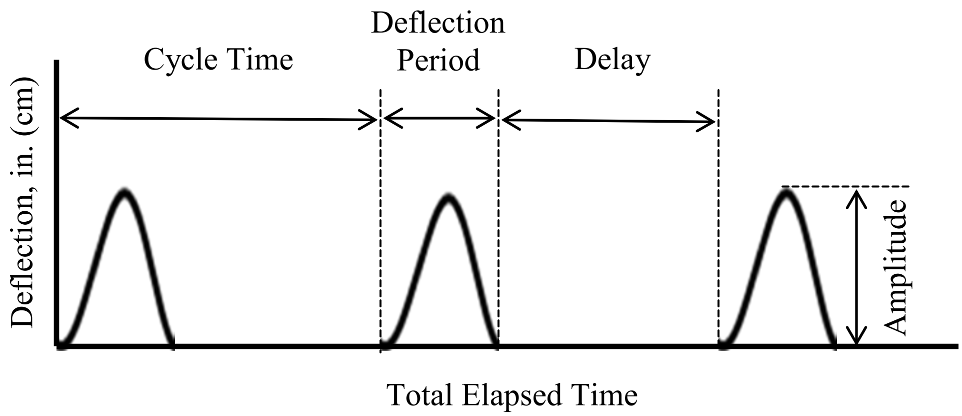

Figure 13. Illustration. Differential deflection loading program.

The deflections were imparted on the specimens up until the point each respective embedment material reached final set according to ASTM C 403. During the setting process the deflection motion remained constant but the force needed to impart the motion increased. The increase in force on the setting material was recorded by a load cell attached to the actuator. This load cell can be observed immediately under the formwork in Figure 7.

Table 29 contains the deflection information for each set of specimens. The variation in deflection amplitude and maximum force were captured during testing. The variation in amplitude demonstrates the range over which the deflections were observed to be applied. All tests were designed to have a 30 second cycle time which included the delay time and the deflection period.

The deflections imparted on the specimens were verified to have closely mimicked the design values. The delay between deflections was always less than one percent different from the design of 30 seconds. The average amplitude imparted for each respective specimen was within 4% of the design value. The variations in amplitude from the averages were determined to be minimal.

Table 29. Applied differential deflections.

Sample Type	Frequency, Hz	Design Amplitude, in. (cm)	Average Amplitude, in. (cm)	Variation in Amplitude, in. (cm)	Maximum / Minimum Force Applied, lb (N)	Cycle Time, sec.
G1	2	0.10 (0.254)	0.1014 (0.2576)	0.1011 - 0.1018 (0.2568 - 0.2586)	47.7 - 104.0 (212.3 – 462.8)	30.01
	2*	0.01 (0.0254)	0.0101 (0.0256)	0.0100 – 0.0102 (0.0254 – 0.0258)	27.0 – 48.0 (120.2 – 213.6)	30.01
	2	0.005 (.0127)	0.0048 (0.0123)	0.0046 – 0.0050 (0.0116 – 0.0128)	6.2 – 33.8 (27.6 – 150.4)	30.01
	5	0.01 (0.0254)	0.0101 (0.0256)	0.0098 – 0.0102 (0.0248 – 0.0260)	11.3 – 39.4 (50.3 – 175.3)	29.71
	5	0.005 (.0127)	0.0050 (0.0127)	0.0048 – 0.0053 (0.0122 – 0.0135)	5.6 – 28.2 (24.9 – 125.5)	30.01
G2	5	0.01 (0.0254)	0.0104 (0.0264)	0.0098 – 0.0105 (0.0248 – 0.0267)	8.4 – 62.9 (37.4 – 279.9)	30.01
	5	0.05 (0.127)	0.0502 (0.1274)	0.0484 – 0.504 (0.1229 – 0.1279)	13.8 – 44.9 (61.4 - 199.8)	30.01
M1	5	0.01 (0.0254)	0.0099 (0.0251)	0.0098 – 0.0104 (0.0250 – 0.0264)	6.1 – 51.8 (27.1 – 230.5)	30.01
E1	5	0.01 (0.0254)	0.0099 (0.0250)	0.0095 – 0.0104 (0.0242 – 0.0264)	8.7 – 25.9 (38.7 – 115.3)	30.01
U1	5	0.01 (0.0254)	0.0100 (0.0254)	0.0098 – 0.0105 (0.0250 – 0.0266)	8.6 – 37.8 (38.3 – 168.2)	30.01
	5	0.005 (.0127)	0.0051 (0.0129)	0.0049 – 0.0053 (0.0124 – 0.0135)	3.7 – 26.0 (16.5 – 115.7)	30.01
U2	5	0.01 (0.0254)	0.0099 (0.0250)	0.0096 – 0.0110 (0.0244 – 0.0280)	7.5 – 49.2 (33.4 – 218.9)	30.01
C1	5**	0.01 (0.0254)	0.0100 (0.0253)	0.0095 – 0.0102 (0.0241 – 0.0258)	6.6 – 48.8 (29.4 – 217.2)	30.01
	5	0.005 (.0127)	0.0500 (0.1269)	0.0494 – 0.0510 (0.1255 – 0.1295)	18.5 – 67.7 (82.3 – 301.3)	30.01
C2	5	0.01 (0.0254)	0.0103 (0.0262)	0.0101 – 0.0116 (0.0256 – 0.0294)	7.6 – 42.9 (33.8 – 190.9)	30.01
	5	0.05 (0.127)	0.0501 (0.1273)	0.0489 – 0.0519 (0.1243 – 0.1318)	14.0 – 90.1 (62.3 – 400.9)	30.01

*During this placement, the actuator did not impart any displacements for 1.43 hours immediately after grout placement. It was restarted prior to initial set.

**During this placement the actuator did not impart any displacements from 50 minutes to 2.17 hours after grout placement. It was restarted prior to initial set.

CHAPTER 4. TEST PROGRAM, RESULTS, AND ANALYSES

INTRODUCTION

A series of concentric pullout tests was performed. The tests were designed based on a 6 in. (153 mm) cube with a #4 (#13M) reinforcing bar embedded concentrically in one side. The reinforcing bar was pulled out of the cube under a deflection controlled load. A series of nine different materials were considered. In each series of tests, six specimens were constructed: three control specimens (i.e., static) and three differential deflection specimens (i.e., deflected).

The objective was to compare the results of the control and the deflected pullout tests. The load and actuator travel were recorded for each specimen. The peak load, deflection at peak load, and failure method were noted. The performance of the static and deflected specimens in each series was compared.

Prior to performing the tests on each material, a series of preliminary tests were executed. In these tests, the test setup and specimen dimensions were adjusted. Material T1 was tested during these preliminary tests, but was eliminated from further consideration based on the results.

BOND TESTING PROGRAM

Demolding and Curing

Each set of tests had six pullout specimens, three deflected and three static, and were cured in the same environment. All testing took place at the Turner-Fairbank Highway Research Center (TFHRC). Each specimen was covered in wet burlap and plastic for approximately the first 20 hours. The specimens were then demolded and left inside the laboratory until they were tested at approximately 24 hours. The only exceptions included M1, which was not covered in wet burlap as recommended by the manufacturer, and U1, which was tested at approximately 43 hours. Care was taken to not bend the reinforcing bar during demolding and handling. Any extending remnants of the bond-breaker foam insulation were removed from the specimen prior to placing it in the testing frame.

Test Setup

The test setup involved anchoring the pullout blocks and the free rebar. Figure 14 shows a diagram of the test setup. A 1 in. (2.5 cm) thick steel reaction plate was tied down with 4 – ¾ in. (1.9 cm) threaded rods. The plate had a 1 in. (2.5 cm) hole drilled through the center of the plate were the rebar was inserted. A ¼ in. (0.6 cm) thick neoprene pad was placed between the load bearing end of the block and the steel plate. The long end of the rebar was placed through the hole in the steel plate. This end of the rebar was then firmly attached to the crosshead of the testing machine with a wedge anchor. The anchor was designed for use with ½ in. (1.3 cm) post-tensioning strands but also worked for the rebar. The entire setup was leveled and then tightened to ensure the specimen was centered with the top and bottom of the MTS actuator.

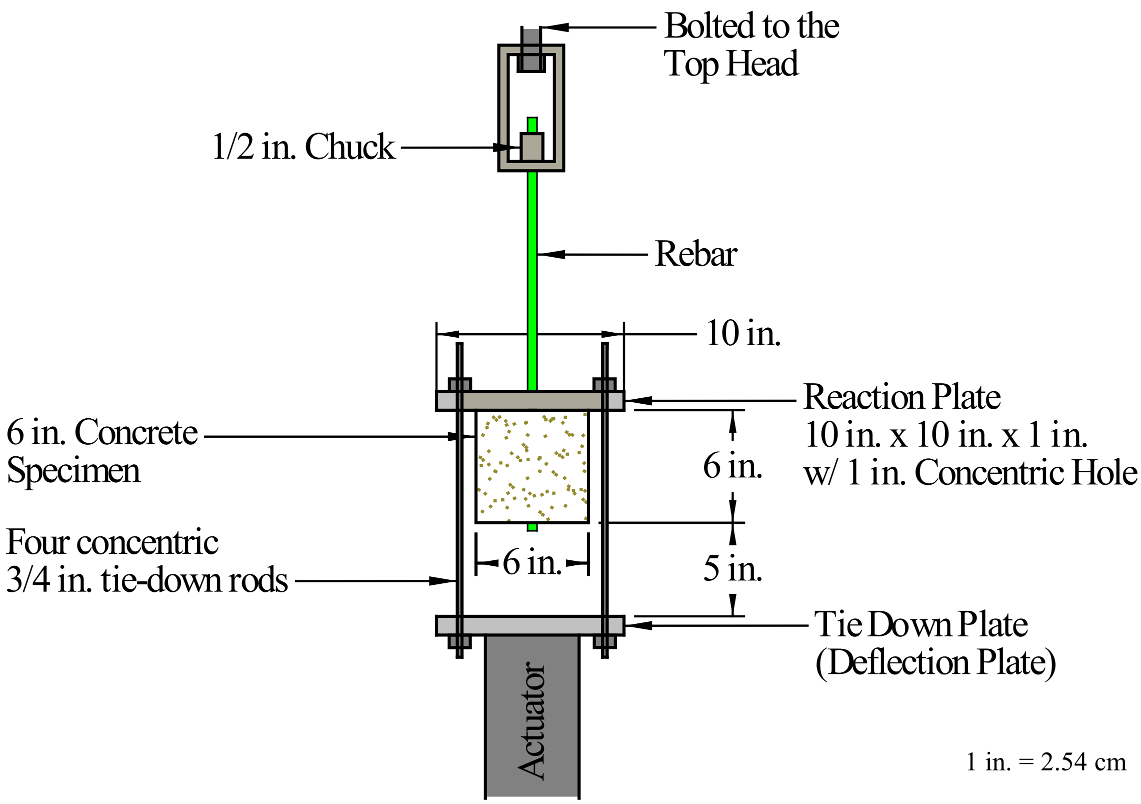

Figure 14. Illustration. Pullout test setup.

A small tensile load of approximately 200 lbs (90 kg) was initially applied to seat the specimen. Next, three LVDTs were attached to the specimen. LVDT #3 measured the differential deflection between the bottom face of the load bearing plate and the free end of the rebar. LVDTs #1 and #2 both measured the differential deflection of the long end of the rebar in relation to the outer edge of the reaction plate. They were placed approximately 180˚ apart on the rebar. The distance between the two attachments on the rebar and reaction plate were measured for each test. The LVDT locations are shown in Figure 15.

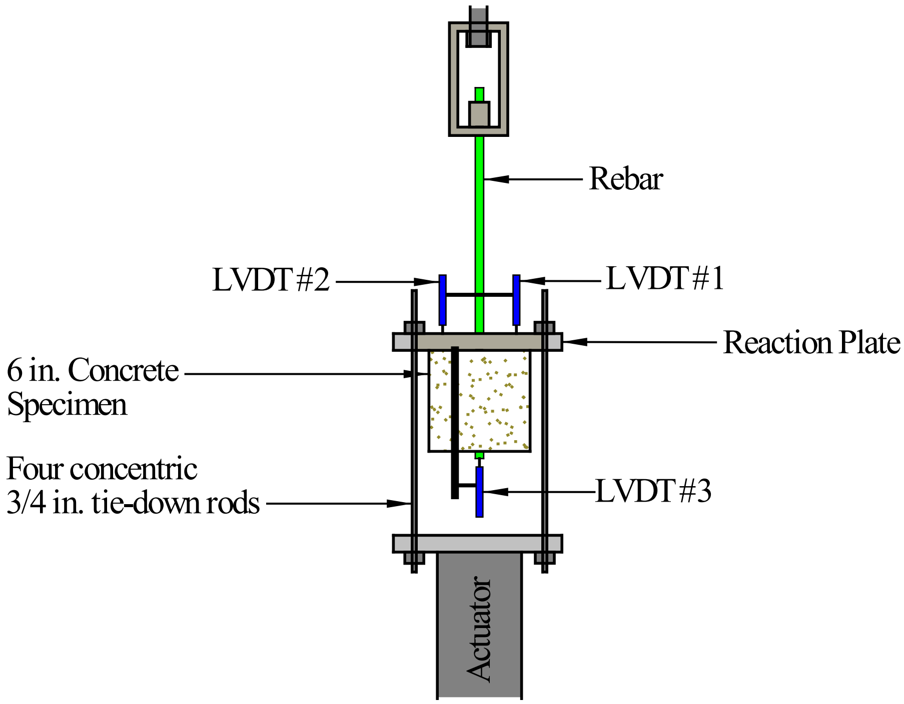

Figure 15. Illustration. LVDT placement on the pullout test.

Testing Procedures

The uniaxial tensile load was applied in a displacement control mode beginning immediately after the LVDT installation. Failure was determined when: 1) the rebar reached its ultimate capacity, 2) the block split, or 3) the load had decreased to less than ½ of the peak applied load. The load was applied at an actuator displacement rate of 0.05 in./min. (0.127 cm/min.) from test initiation through conclusion.

Data was taken throughout the testing process. This included movement of the rebar as measured by the three LVDTs. A load cell was placed on the actuator to measure the load applied. The displacement of the actuator was also recorded, however this displacement included deformations of the entire frame and testing setup. It was not generally used in the data analysis but did provide information on the behavior of the test setup.

Displacement versus load plots were created for each test. The critical point on the plot was the peak applied load. The deformations of both ends of the rebar at the peak load were also recorded. A typical load versus displacement curve for each type of failure mode is shown in Figure 16. For a pullout mode of failure the curve has a nearly linear ascent, a sharp peak, and then a gradual decline in load as the displacement rate increases. This was the mode of failure for the majority of the specimens. For the specimens that had stronger bonds than the rebar capacity, the plot reached the ultimate capacity of the rebar then dropped to no load when the rebar ruptured. The ascending shape was similar to the pullout failure mode. The splitting failure mode had a similar linear ascending region followed by a substantial loss in load which is coincident with an increase in displacement when splitting began. The samples held a very small amount of load at this point and the deflection increased rapidly.

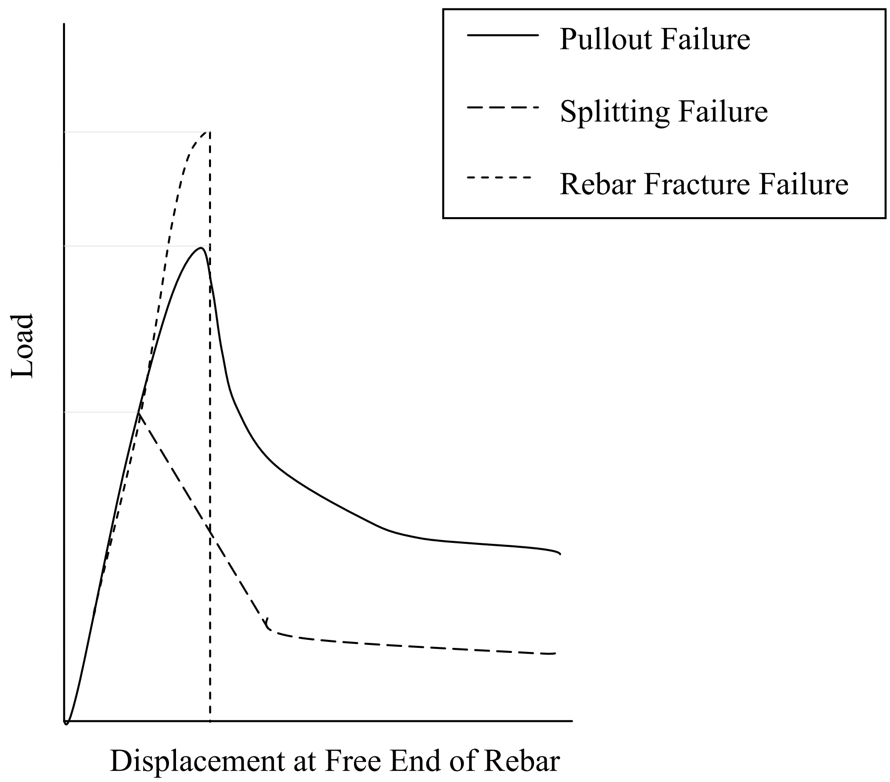

Figure 16. Illustration. Typical load versus displacement plots for pullout tests.

PRELIMINARY TESTS

Specimen and Reinforcing Bar Size

The initial test specimens were built following the recommendations from ASTM C234. The formwork was built with ¼ in. (7 mm) thick steel and ¼ in. (7 mm) diameter bolts. The reinforcing bar (rebar) was a #6 (#18M) bar. For the control specimens, a support stand was installed to firmly hold the free end of the rebar in place. The rebar was placed inside the formwork such that it was perpendicular to the face of the forms and exited the far side of the form by approximately ½ in. (13 mm). Figure 17 provides a photo of the formwork, rebar, and support stand.

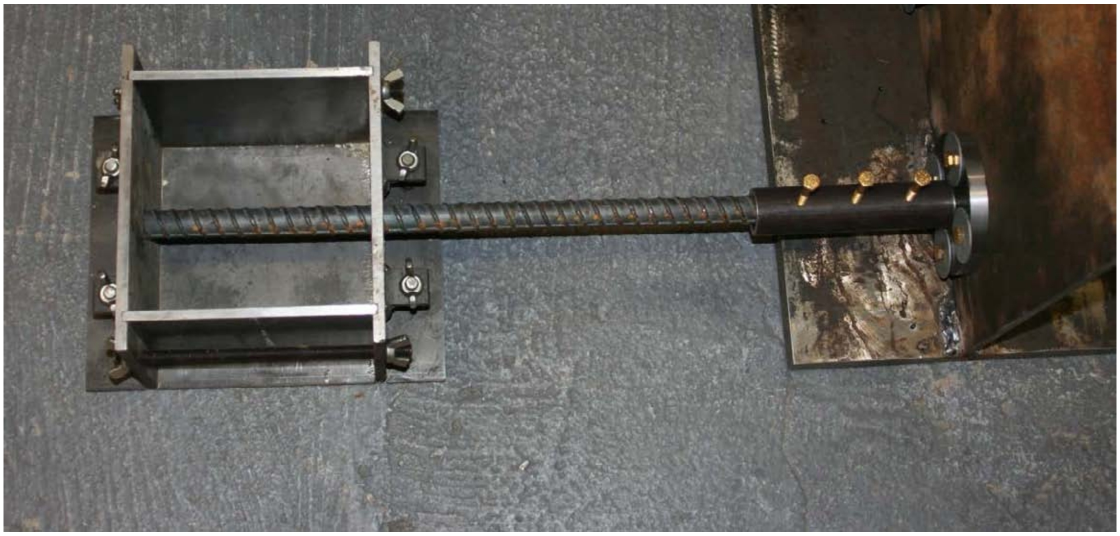

Figure 17. Photograph. Setup for casting pullout specimens.

A similar layout was used to make the initial vibrated test specimens. The difference was the foundation for the support stand and steel formwork. The steel formwork was affixed to the actuator head with C-clamps. The actuator was used to impart deflections on the steel formwork. The rebar was held in place by the support stand, thereby eliminating any movement of the rebar. The support stand was fixed to the load frame. Recall that Figure 7 provides a photograph of the setup for the casting of the deflected specimens.

Two sets of specimens made of grout G1 were created to test the setup for errors. A 2 Hz, 0.1 in. (0.25 cm) amplitude deflection with a 30 second interval between deflections was initially used. With #6 (#19M) rebar, all 12 trial specimens split in half during testing. The purpose was to test the bond between the rebar and material encompassing it with a pullout failure. Splitting failures tend to relate to the clear cover and tensile strength of the cast material, thus not providing useful information regarding the bond performance of the reinforcing bar.

The dimensions of the specimens are directly related to the types of bond failures. The specimens were 6 in. (15.2 cm) cubes. Thus, the clear cover around the rebar was 3.5 times the bar diameter and the embedment length of 6 in. (15.2 cm) was 8 times the bar diameter. This clear cover ratio is small and the embedment length ratio is large compared to other pullout tests methods as explained in the literature review discussion on concentric tests.

Adjustments were made to the test layout on a second set of trial tests. In this case the specimen and rebar size remained the same. However, the embedment length was reduced to

3 in. (7.1 cm) or 4 times the bar diameter (4db). This shorter length was designed to keep the bond stresses uniform while reducing the overall bond strength producing pullout failures. A $^3/_8$ in. (0.95 cm) thick by 1.5 in. (3.8 cm) outer diameter foam pipe insulation was attached to half of the rebar in the cube (Figure 18). By debonding the rebar next to the reaction face of the cube, the compressive forces on the rebar were also reduced during testing. These trial tests again resulted in splitting failures. It was determined that the #6 (#19M) rebar was too large for the standard size pullout cube.

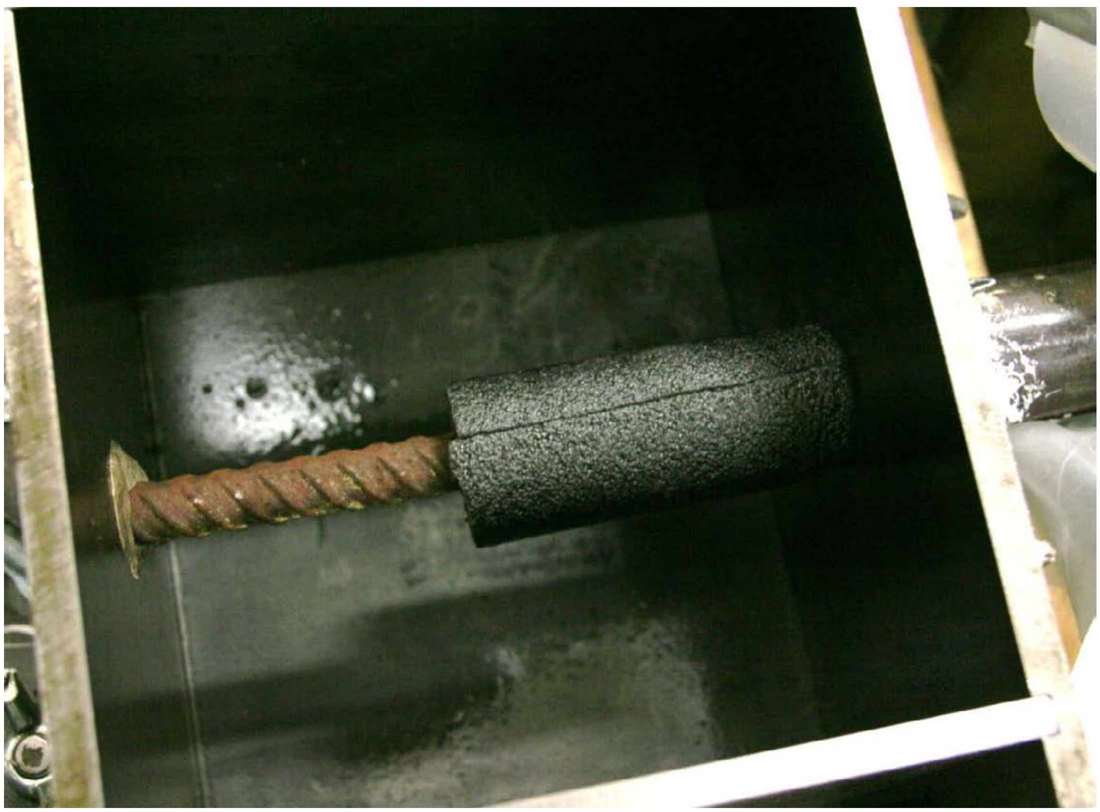

Figure 18. Photograph. Debonded section of the pullout specimen.

The last set of trial specimens were nearly the same as the previous tests with the exception of the rebar size. Number 4 (#13M) bars were used with the same 6 in. (15.2 cm) cube specimens. The embedment length was also 3 in. (7.1 cm). These specimens were tested with standard grout G1, and all tests resulted in pullout failures. The grout material around the rebar failed prior to the splitting the cubes.

The final dimensions were 6 in. (15.2 cm) cubes with concentric #4 (#13M) rebar. The rebar was debonded for 3 in. (7.1 cm) on the reaction face side of the cube with $^3/_8$ in. thick

(0.95 cm) foam pipe insulation. The embedment length was 6 times the diameter of the bar. The clear cover was 5.5 times the diameter of the bar.

Form Construction

Originally a 1 in. (2.5 cm) hole was drilled concentrically through the steel forms for the rebar. The forms were designed for #6 rebar that would remain static during the casting procedure. Given that the imposed differential deflection was as large as 0.1 in. (0.25 cm), the hole was enlarged to allow the rebar to pass through the form and deflect without impacting the formwork. The holes were enlarged approximately 0.25 in. (0.65 cm) in the direction of the differential deflection movement.

As shown in Figure 19, this form setup resulted in gaps between the forms and rebar. A flexible material had to be placed between the rebar and formwork in order to prevent the cast material from leaking through the gap. At the same time the flexible material could not interfere with the differential deflection. A layer of rope caulk encompassed by silicone caulk was chosen because they bonded well to the steel, maintained a tight seal, and did not impede the deflections.

Figure 19. Photograph. Flexible caulk between the formwork and reinforcing bar.

Material T1

Material T1, a standard cable grout, was used on two sets of initial specimens. Along with the specimens, cylinders and cubes were made for material testing. After 24 hours, the samples were demolded. It was observed that material T1 cracked significantly throughout all the specimens. The cracks traveled through the entire cube section including to the rebar in the center. The specimens did not hold a significant load under the pullout tests. The cracking revealed that the material was not well suited for this particular application. Recall that the previously reported material property results also indicated that exhibit a significant amount of early age shrinkage cracking. Because of these results the material was eliminated from further study. In the final results section, T1 is not reported.

Preliminary Test Summary

A series of initial tests were performed to check the pullout casting and testing setup. The final dimensions of the rebar, embedment length, and specimen size were calibrated to create pullout type failures and not splitting failures. The forms were built to ensure that the specimens could be cast without leaking material during the casting operation.

BOND TESTING RESULTS

Each of the eight materials was tested individually. The control and deflected samples were made with the same batch of material and then tested together. The following summarizes the results for all of the tests. Each batch of test specimens is named via the following nomenclature: name of the grout - batch number.

Material G1

Material G1 had properties of a typical low shrinkage high early strength grout, thus leading to its use on the initial series of five tests. The results from these tests helped in selecting the critical vibration amplitude and frequency for the future tests. Based on the results from material G1, an amplitude of 0.01 in. (0.25 mm) at 5 Hz was used in at least one set of tests for every type of material.

G1 - 1: 0.1 in. (2.5 mm) at 2 Hz

The first set of G1 tests was performed with a deflection amplitude of 0.1 in. (2.5 mm) at a frequency of 2 Hz (Table 30). A 30 second interval was placed between deflections and #4 (#13M) rebar was used throughout. The grout reached final set at 6.92 hours and the deflections were stopped at 7.0 hours after the initiation of mixing the grout materials. The average time of testing was 1.26 days after initiation of mixing for the static samples and 1.17 days for the deflection samples.

The pullout test load results are shown in Table 31. All six samples failed in a pullout mode. The average peak load withstood by the deflected samples was approximately 77% less than the average load of the static samples.

Table 30. G1-1 testing properties under 0.1 in. (2.5 mm) deflections at 2 Hz.

	Time After Mix Initiation				Design Values		
Sample	Time to Test, days	Avg. Time to Test, days	Time to Final Set, hrs	Defl. Start, hrs	Defl. Amplitude, in. (mm)	Defl. Freq., Hz	Interval Between Defl., sec.
Static1	1.24	1.26	6.92	7.00	0.1 (2.54)	2	30
Static2	1.26						
Static3	1.27						
Deflected1	1.15	1.17					
Deflected2	1.17						
Deflected3	1.18						

Table 31. G1-1 load results under 0.1 in. (2.5 mm) deflections at 2 Hz.

Sample	Failure Type	Maximum Load, lbs	Average Maximum Load, lbs	Standard Deviation, lbs
Static1	Pullout	8641		
Static2	Pullout	8973	8704	244
Static3	Pullout	8498		
Deflected1	Pullout	2020		
Deflected2	Pullout	2007	2043	52
Deflected3	Pullout	2103		

1 lb = 4.45 N

G1 – 2: 0.01 in. (0.25 mm) at 2 Hz

The second set of G1 tests was performed with a deflection amplitude of 0.01 in. (0.25 mm) at a frequency of 2 Hz (Table 32). A 30 second interval was placed between deflections and #4 (#13M) rebar was used throughout. The grout reached final set at 5.98 hours and the deflections were stopped at 6.17 hours after initiation of mixing the grout. The average time of testing was 1.05 days after initiation of mixing for the static samples and 1.03 days for the deflected samples.

The pullout test strength results are shown in Table 33. All six samples failed in a pullout mode. The average peak load withstood by the deflected samples was approximately 19% less than the average load of the static samples.

Table 32. G1-2 testing properties under 0.01 in. (0.25 mm) deflections at 2 Hz.

	Time After Mix Initiation				Design Values		
Sample	Time to Test, days	Avg. Time to Test, days	Time to Final Set, hrs	Defl. Start, hrs	Defl. Amplitude, in. (mm)	Defl. Freq., Hz	Interval Between Defl., sec.
Static1	1.02						
Static2	1.05	1.05					
Static3	1.06		5.98	6.17	0.01 (0.25)	2	30
Deflected1	0.99						
Deflected2	1.01	1.03					
Deflected3	1.08						

Table 33. G1-2 load results under 0.01 in. (0.25 mm) deflections at 2 Hz.

Sample	Failure Type	Maximum Load, lbs	Average Maximum Load, lbs	Standard Deviation, lbs
Static1	Pullout	6576		
Static2	Pullout	7457	7166	511
Static3	Pullout	7465		
Deflected1	Pullout	6118		
Deflected2	Pullout	5229	5772	476
Deflected3	Pullout	5969		

1 lb = 4.45 N

G1 – 3: 0.005 in. (0.127 mm) at 2 Hz

The third set of G1 tests was performed with a deflection amplitude of 0.005 in. (0.127 mm) at a frequency of 2 Hz (Table 34). A 30 second interval was placed between deflections and #4 (#13M) rebar was used throughout. The grout reached final set at 6.68 hours and the deflections were stopped at 7.08 hours after initiation of mixing the grout. The average time of testing was 1.08 days after initiation of mixing for the static samples and 1.10 days for the deflection samples.

The pullout test strength results are shown in Table 35. All six samples failed in a pullout mode. The average peak load withstood by the deflected samples was approximately 13% more than the average load of the static samples.

Table 34. G1-3 testing properties under 0.005 in. (0.127 mm) deflections at 2 Hz.

	Time After Mix Initiation				Design Values		
Sample	Time to Test, days	Avg. Time to Test, days	Time to Final Set, hrs	Defl. Start, hrs	Defl. Amplitude, in. (mm)	Defl. Freq., Hz	Interval Between Defl., sec.
Static1	1.03						
Static2	1.06						
Static3	1.14	1.08	6.68	7.08	0.005 (0.127)	2	30
Deflected1	1.05						
Deflected2	1.10						
Deflected3	1.15	1.10					

Table 35. G1-3 load results under 0.005 in. (0.127 mm) deflections at 2 Hz.

Sample	Failure Type	Maximum Load, lbs	Average Maximum Load, lbs	Standard Deviation, lbs
Static1	Pullout	8958		
Static2	Pullout	9245	8941	312
Static3	Pullout	8621		
Deflected1	Pullout	11028		
Deflected2	Pullout	10199	10,090	998
Deflected3	Pullout	9042		

1 lb = 4.45 N

G1 - 4: 0.01 in. (0.25 mm) at 5 Hz

The fourth set of G1 tests was performed with a deflection amplitude of 0.01 in. (0.25 mm) at a frequency of 5 Hz (Table 36). A 30 second interval was placed between deflections and #4 (#13M) rebar was used throughout. The grout reached final set at 6.70 hours and the deflections were stopped at 6.85 hours after initiation of mixing the grout. The average time of testing was 1.12 days after initiation of mixing for the static samples and 1.13 days for the deflection samples.

The pullout test strength results are shown in Table 37. All six samples failed in a pullout mode. The average peak load withstood by the deflected samples was approximately 19% less than the average load of the static samples.

Table 36. G1-4 testing properties under 0.01 in. (0.25 mm) at 5 Hz.

	Time After Mix Initiation				Design Values		
Sample	Time to Test, days	Avg. Time to Test, days	Time to Final Set, hrs	Defl. Start, hrs	Defl. Amplitude, in. (mm)	Defl. Freq., Hz	Interval Between Defl., sec.
Static1	1.08						
Static2	1.11	1.12					
Static3	1.16						
Deflected1	1.10		6.7	6.85	0.01 (0.254)	5	30
Deflected2	1.12	1.13					
Deflected3	1.17						

Table 37. G1-4 load results under 0.01 in. (0.25 mm) at 5 Hz.

Sample	Failure Type	Maximum Load, lbs	Average Maximum Load, lbs	Standard Deviation, lbs
Static1	Pullout	10596		
Static2	Pullout	9682	9798	747
Static3	Pullout	9115		
Deflected1	Pullout	6868		
Deflected2	Pullout	8743	7980	985
Deflected3	Pullout	8328		

1 lb = 4.45 N

G1 - 5: 0.005 in. (0.127 mm) at 5 Hz

The fifth set of G1 tests was performed with a deflection amplitude of 0.005 in. (0.127 mm) at a frequency of 5 Hz (Table 38). A 30 second interval was placed between deflections and #4 (#13M) rebar was used throughout. The grout reached final set at 6.43 hours and the deflections were stopped at 6.75 hours after initiation of mixing the grout. The average time of testing was 1.06 days after initiation of mixing for the static samples and 1.07 days for the deflection samples.

The pullout test strength results are shown in Table 39. All six samples failed in a pullout mode. The average peak load withstood by the deflected samples was approximately the same as the average load of the static samples.

Table 38. G1-5 testing properties under 0.005 in. (0.127 mm) at 5 Hz.

	Time After Mix Initiation				Design Values		
Sample	Time to Test, days	Avg. Time to Test, days	Time to Final Set, hrs	Defl. Start, hrs	Defl. Amplitude, in. (mm)	Defl. Freq., Hz	Interval Between Defl., sec.
Static1	1.03	1.06	6.43	6.75	0.005 (0.127)	5	30
Static2	1.06						
Static3	1.08						
Deflected1	1.04	1.07					
Deflected2	1.07						
Deflected3	1.09						

Table 39. G1-5 load results under 0.005 in. (0.127 mm) at 5 Hz.

Sample	Failure Type	Maximum Load, lbs	Average Maximum Load, lbs	Standard Deviation, lbs
Static1	Pullout	7225	6963	241
Static2	Pullout	6915		
Static3	Pullout	6750		
Deflected1	Pullout	6367	7016	1001
Deflected2	Pullout	6512		
Deflected3	Pullout	8168		

1 lb = 4.45 N

Material G2

G2 – 1: 0.01 in. (0.25 mm) at 5 Hz

The first set of G2 tests was performed with a deflection amplitude of 0.01 in. (0.25 mm) at a frequency of 5 Hz (Table 40). A 30 second interval was placed between deflections and #4 (#13M) rebar was used throughout. The grout reached final set at 10.75 hours and the deflections were stopped at 21.83 hours after initiation of mixing the grout. The large time difference was due to the grout setting at a rate much slower than anticipated. The final set occurred late in the evening and the actuator was not stopped until the following day. The average time of testing was 1.04 days after initiation of mixing for the static samples and 1.05 days for the deflection samples.

The pullout test strength and deflection results are shown in Table 41. All six samples failed in a pullout method. The average peak load withstood by the deflected samples was approximately 12% more than the average load of the static samples.

Table 40. G2-1 testing properties under 0.01 in. (0.25 mm) at 5 Hz.

	Time After Mix Initiation				Design Values		
Sample	Time to Test, days	Avg. Time to Test, days	Time to Final Set, hrs	Defl. Start, hrs	Defl. Amplitude, in. (mm)	Defl. Freq., Hz	Interval Between Defl., sec.
Static1	1.01	1.04	10.75	21.83	0.01 (0.25)	5	30
Static2	1.03						
Static3	1.06						
Deflected1	1.03	1.05					
Deflected2	1.05						
Deflected3	1.07						

Table 41. G2-1 load results under 0.01 in. (0.25 mm) at 5 Hz.

Sample	Failure Type	Maximum Load, lbs	Average Maximum Load, lbs	Standard Deviation, lbs
Static1	Pullout	9465	9987	558
Static2	Pullout	10575		
Static3	Pullout	9921		
Deflected1	Pullout	10234	11,206	874
Deflected2	Pullout	11928		
Deflected3	Pullout	11456		

1 lb = 4.45 N

G2 - 2: 0.05 in. (1.27 mm) at 5 Hz

The second set of G2 tests was performed with a deflection amplitude of 0.05 in. (1.27 mm) at a frequency of 5 Hz (Table 42). A 30 second interval was placed between deflections and #4 (#13M) rebar was used throughout. The grout reached final set at 10.03 hours and the deflections were stopped at 10.10 hours after initiation of mixing the grout. The average time of testing was 1.01 days after initiation of mixing for the static samples and 1.02 days for the deflection samples.

The pullout test strength results are shown in Table 43. All six samples failed in a pullout mode. The average peak load withstood by the deflected samples was approximately 63% less than the average load of the static samples.

Table 42. G2-2 testing properties under 0.05 in. (1.27 mm) at 5 Hz.

	Time After Mix Initiation				Design Values		
Sample	Time to Test, days	Avg. Time to Test, days	Time to Final Set, hrs	Defl. Start, hrs	Defl. Amplitude, in. (mm)	Defl. Freq., Hz	Interval Between Defl., sec.
Static1	0.98	1.01	10.03	10.10	0.05 (1.27)	5	30
Static2	1.01						
Static3	1.03						
Deflected1	1.00	1.02					
Deflected2	1.02						
Deflected3	1.04						

Table 43. G2-2 load results under 0.05 in. (1.27 mm) at 5 Hz.

Sample	Failure Type	Maximum Load, lbs	Average Maximum Load, lbs	Standard Deviation, lbs
Static1	Pullout	10955	10,682	418
Static2	Pullout	10889		
Static3	Pullout	10201		
Deflected1	Pullout	3965	3958	129
Deflected2	Pullout	4083		
Deflected3	Pullout	3825		

1 lb = 4.45 N

Material M1

M1 - 1: 0.01 in. (0.25 mm) at 5 Hz

The only set of M1 grout tests was performed with a deflection amplitude of 0.01 in. (0.25 mm) at a frequency of 5 Hz (Table 44). A 30 second interval was placed between deflections and #4 (#13M) rebar was used throughout. Three grout placements were used to make these specimens because of the short setting time. Each grout placement reached final set at 0.13 hours after initiation of mixing the grout. The deflections were stopped at an average of 0.80 hours after initiation of mixing the first grout. The average time of testing was 0.86 days after initiation of mixing for the static samples and 0.88 days for the deflection samples.

The pullout test strength results are shown in Table 45. All six samples failed in a splitting type failure mode. The failures were very dynamic as the grout split in half and the bond lost all load carrying capacity at once. The average peak load withstood by the deflected samples was approximately 4% less than the average load of the static samples.

Table 44. M1-1 testing properties under 0.01 in. (0.25 mm) deflections at 5 Hz.

	Time After Mix Initiation				Design Values		
Sample	Time to Test, days	Avg. Time to Test, days	Time to Final Set, hrs	Defl. Start, hrs	Defl. Amplitude, in. (mm)	Defl. Freq., Hz	Interval Between Defl., sec.
Static1	0.84	0.86	0.13	0.80	0.01 (0.25)	5	30
Static2	0.86						
Static3	0.89						
Deflected1	0.85	0.88					
Deflected2	0.88						
Deflected3	0.90						

Table 45. M1-1 grout load results under 0.01 in. (0.25 mm) deflections at 5 Hz.

Sample	Failure Type	Maximum Load, lbs	Average Maximum Load, lbs	Standard Deviation, lbs
Static1	Splitting	9276		
Static2	Splitting	9360	9790	819
Static3	Splitting	10734		
Deflected1	Splitting	9267		
Deflected2	Splitting	8243	9371	1183
Deflected3	Splitting	10603		

1 lb = 4.45 N

Material E1

E1 – 1: 0.01 in. (0.25 mm) at 5 Hz

The only test with E1 was performed with a deflection amplitude of 0.01 in. (0.25 mm) at a frequency of 5 Hz (Table 46). A 30 second interval was placed between deflections and #4 (#13M) rebar was used throughout. The grout reached final set at 2.33 hours and the deflections were stopped at 2.67 hours after initiation of mixing the grout. The average time of testing was 1.05 days after initiation of mixing for the deflection samples. The static samples were tested 0.26, 1.08, and 1.12 days after initiation of mixing the material. The static sample average time was lower than 1.0 days because one test was performed following the final set of the grout at 0.26 days. This was performed because it was estimated that E1 grout would gain strength too fast and result in failure of the rebar at 24 hours. This was proven correct as demonstrated by the results of the other five samples.

The pullout test load results are shown in Table 47. All five samples tested after 24 hours reached the ultimate tensile strength of the rebar; therefore, the rebar bond to the E1 matrix did not fail.

Table 46. E1-1 grout testing properties under 0.01 in. (0.25 mm) deflections at 5 Hz.

	Time After Mix Initiation				Design Values		
Sample	**Time to Test, days**	**Avg. Time to Test, days**	**Time to Final Set, hrs**	**Defl. Start, hrs**	**Defl. Amplitude, in. (mm)**	**Defl. Freq., Hz**	**Interval Between Defl., sec.**
Static1	0.26	0.82	2.33	2.67	0.01 (0.25)	5	30
Static2	1.08						
Static3	1.12						
Deflected1	1.01	1.05					
Deflected2	1.06						
Deflected3	1.10						

Table 47. E1-1 grout load results under 0.01 in. (0.25 mm) deflections at 5 Hz.

Sample	Failure Type	Maximum Load, lbs	Average Maximum Load, lbs	Standard Deviation, lbs
Static1	Rebar Fracture	18708	20,843	1862
Static2	Rebar Fracture	21693		
Static3	Rebar Fracture	22128		
Deflected1	Rebar Fracture	21773	21,604	325
Deflected2	Rebar Fracture	21809		
Deflected3	Rebar Fracture	21229		

1 lb = 4.45 N

Material U1

U1 - 1: 0.01 in. (0.25 mm) at 5 Hz

The first set of U1 tests was performed with a deflection amplitude of 0.01 in. (0.25 mm) at a frequency of 5 Hz (Table 48). A 30 second interval was placed between deflections and #4 (#13M) rebar was used throughout. The grout reached final set at 18.32 hours and the deflections were stopped at 18.67 hours after initiation of mixing the grout. The average time of testing was 1.79 days after initiation of mixing for the static samples and 1.81 days for the deflection samples. The samples were tested almost two days later due to the slower rate of strength gain of U1.

The pullout test strength results are shown in Table 49. All six samples failed in a pullout mode. The average peak load withstood by the deflected samples was approximately 20% less than the average load of the static samples.

Table 48. U1-1 testing properties under 0.01 in. (0.25 mm) deflections at 5 Hz.

	Time After Mix Initiation				Design Values		
Sample	**Time to Test, days**	**Avg. Time to Test, days**	**Time to Final Set, hrs**	**Defl. Start, hrs**	**Defl. Amplitude, in. (mm)**	**Defl. Freq., Hz**	**Interval Between Defl., sec.**
Static1	1.77						
Static2	1.80	1.79					
Static3	1.82		18.32	18.67	0.01 (0.25)	5	30
Deflected1	1.78						
Deflected2	1.81	1.81					
Deflected3	1.83						

Table 49. U1-1 load results under 0.01 in. (0.25 mm) deflections at 5 Hz.

Sample	Failure Type	Maximum Load, lbs	Average Maximum Load, lbs	Standard Deviation, lbs
Static1	Pullout	15072		
Static2	Pullout	16051	15,794	634
Static3	Pullout	16258		
Deflected1	Pullout	13571		
Deflected2	Pullout	11138	12,593	1285
Deflected3	Pullout	13069		

1 lb = 4.45 N

U1 - 2: Standard Set – 0.005 in. (0.125 mm) at 5 Hz

The second set of U1 tests was performed with a deflection amplitude of 0.005 in. (0.125 mm) at a frequency of 5 Hz (Table 50). A 30 second interval was placed between deflections and #4 (#13M) rebar was used throughout. The grout reached final set at 15.52 hours and the deflections were stopped at 16.33 hours after initiation of mixing the grout. The average time of testing was 1.83 days after initiation of material placement for the static samples and 1.84 days for the deflection samples. The samples were tested almost two days later due to the slower rate of strength gain of U1.

The pullout test strength results are shown in Table 51. All six samples failed in a pullout mode. The average peak load withstood by the deflected samples was approximately 3% more than the average load of the static samples.

Table 50. U1-2 testing properties under 0.005 in. (0.127 mm) at 5 Hz.

	Time After Mix Initiation				Design Values		
Sample	Time to Test, days	Avg. Time to Test, days	Time to Final Set, hrs	Defl. Start, hrs	Defl. Amplitude, in. (mm)	Defl. Freq., Hz	Interval Between Defl., sec.
Static1	1.81						
Static2	1.83	1.83					
Static3	1.86		15.52	16.33	0.005 (0.127)	5	30
Deflected1	1.82						
Deflected2	1.84	1.84					
Deflected3	1.87						

Table 51. U1-2 load results under 0.005 in. (0.127 mm) at 5 Hz.

Sample	Failure Type	Maximum Load, lbs	Average Maximum Load, lbs	Standard Deviation, lbs
Static1	Pullout	15983		
Static2	Pullout	17418	17,035	922
Static3	Pullout	17703		
Deflected1	Pullout	17752		
Deflected2	Pullout	17005	17,610	548
Deflected3	Pullout	18074		

1 lb = 4.45 N

Material U2

U2 – 1: 0.01 in. (0.25 mm) at 5 Hz

The only test with U2 was performed with a deflection amplitude of 0.01 in. (0.25 mm) at a frequency of 5 Hz (Table 52). A 30 second interval was placed between deflections and #4 (#13M) rebar was used throughout. The grout reached final set at 4.98 hours and the deflections were stopped at 5.50 hours after initiation of mixing the grout. The average time of testing was 1.20 days after initiation of mixing for the static samples and 1.22 days for the deflection samples.

The pullout test load results are shown in Table 53. All six samples reached the ultimate tensile strength of the rebar; therefore, the rebar bond to the U2 matrix did not fail.

Table 52. U2-1 testing properties under 0.01 in. (0.25 mm) deflections at 5 Hz.

	Time After Mix Initiation				Design Values		
Sample	Time to Test, days	Avg. Time to Test, days	Time to Final Set, hrs	Defl. Start, hrs	Defl. Amplitude, in. (mm)	Defl. Freq., Hz	Interval Between Defl., sec.
Static1	1.10	1.20	4.98	5.50	0.01 (0.25)	5	30
Static2	1.24						
Static3	1.26						
Deflected1	1.13	1.22					
Deflected2	1.25						
Deflected3	1.27						

Table 53. U2-1 load results under 0.01 in. (0.25 mm) deflections at 5 Hz.

Sample	Failure Type	Maximum Load, lbs	Average Maximum Load, lbs	Standard Deviation, lbs
Static1	Rebar Fracture	21515	21,512	4
Static2	Rebar Fracture	21513		
Static3	Rebar Fracture	21507		
Deflected1	Rebar Fracture	21499	21,199	538
Deflected2	Rebar Fracture	21520		
Deflected3	Rebar Fracture	20578		

1 lb = 4.45 N

Material C1

C1 – 1: 0.01 in. (0.25 mm) at 5 Hz

The first set of C1 tests was performed with a deflection amplitude of 0.01 in. (0.25 mm) at a frequency of 5 Hz (Table 54). A 30 second interval was placed between deflections and #4 (#13M) rebar was used throughout. The grout reached final set at 5.83 hours and the deflections were stopped at 6.00 hours after initiation of mixing the grout. The average time of testing was 1.06 days after initiation of mixing for the static samples and 1.07 days for the deflection samples.

The pullout test strength results are shown in Table 55. All six samples failed in a pullout mode. The average peak load withstood by the deflected samples was approximately 8% less than the average load of the static samples.

Table 54. C1-1 testing properties under 0.01 in. (0.25 mm) deflections at 5 Hz.

	Time After Mix Initiation				Design Values		
Sample	Time to Test, days	Avg. Time to Test, days	Time to Final Set, hrs	Defl. Start, hrs	Defl. Amplitude, in. (mm)	Defl. Freq., Hz	Interval Between Defl., sec.
Static1	1.04						
Static2	1.06	1.06					
Static3	1.07		5.83	6.00	0.01 (0.25)	5	30
Deflected1	1.05						
Deflected2	1.07	1.07					
Deflected3	1.08						

Table 55. C1-1 load results under 0.01 in. (0.25 mm) deflections at 5 Hz.

Sample	Failure Type	Maximum Load, lbs	Average Maximum Load, lbs	Standard Deviation
Static1	Pullout	6586		
Static2	Pullout	9189	7930	1304
Static3	Pullout	8014		
Deflected1	Pullout	7951		
Deflected2	Pullout	6802	7281	598
Deflected3	Pullout	7089		

1 lb = 4.45 N

C1 - 2: 0.05 in. (1.27 mm) at 5 Hz

The second set of C1 tests was performed with a deflection amplitude of 0.05 in. (1.27 mm) at a frequency of 5 Hz (Table 56). A 30 second interval was placed between deflections and #4 (#13M) rebar was used throughout. The grout reached final set at 6.08 hours and the deflections were stopped at 6.00 hours after initiation of mixing the grout . The average time of testing was 1.04 days after initiation of mixing for the static samples and 1.06 days for the deflection samples.

The pullout test strength results are shown in Table 57. All six samples failed in a pullout method. The average peak load withstood by the deflected samples was approximately 68% less than the average load of the static samples.

Table 56. C1-2 testing properties under 0.05 in. (1.27 mm) deflections at 5 Hz.

	Time After Mix Initiation				Design Values		
Sample	Time to Test, days	Avg. Time to Test, days	Time to Final Set, hrs	Defl. Start, hrs	Defl. Amplitude, in. (mm)	Defl. Freq., Hz	Interval Between Defl., sec.
Static1	1.00	1.04	6.08	6.00	0.05 (1.27)	5	30
Static2	1.04						
Static3	1.08						
Deflected1	1.01	1.06					
Deflected2	1.07						
Deflected3	1.09						

Table 57. C1-2 load results under 0.05 in. (1.27 mm) deflections at 5 Hz.

Sample	Failure Type	Maximum Load, lbs	Average Maximum Load, lbs	Standard Deviation, lbs
Static1	Pullout	5598	5887	354
Static2	Pullout	5780		
Static3	Pullout	6282		
Deflected1	Pullout	2076	1883	442
Deflected2	Pullout	1377		
Deflected3	Pullout	2195		

1 lb = 4.45 N

Material C2

C2 - 1: 0.01 in. (0.25 mm) at 5 Hz

The first set of C2 tests was performed with a deflection amplitude of 0.01 in. (0.25 mm) at a frequency of 5 Hz (Table 58). A 30 second interval was placed between deflections and #4 (#13M) rebar was used throughout. The concrete reached final set at 6.18 hours and the deflections were stopped at 6.13 hours after initiation of mixing the grout. The average time of testing was 1.06 days after initiation of mixing for the static samples and 1.07 days for the deflection samples.

The pullout test strength results are shown in Table 59. All six samples failed in a pullout mode. The average peak load withstood by the deflected samples was approximately the same as the average load of the static samples.

Table 58. C2-1 testing properties under 0.01 in. (0.25 mm) at 5 Hz.

	Time After Mix Initiation				Design Values		
Sample	Time to Test, days	Avg. Time to Test, days	Time to Final Set, hrs	Defl. Start, hrs	Defl. Amplitude, in. (mm)	Defl. Freq., Hz	Interval Between Defl., sec.
Static1	1.02						
Static2	1.07	1.06					
Static3	1.09		6.18	6.13	0.01 (0.25)	5	30
Deflected1	1.04						
Deflected2	1.08	1.07					
Deflected3	1.10						

Table 59. C2-1 load results under 0.01 in. (0.25 mm) at 5 Hz.

Sample	Failure Type	Maximum Load, lbs	Average Maximum Load, lbs	Standard Deviation, lbs
Static1	Pullout	5394		
Static2	Pullout	6067	5401	662.5
Static3	Pullout	4742		
Deflected1	Pullout	5379		
Deflected2	Pullout	4771	5422	673.5
Deflected3	Pullout	6116		

1 lb = 4.45 N

C2 – 2: 0.05 in. (1.27 mm) at 5 Hz

The second set of C2 tests was performed with a deflection amplitude of 0.05 in. (1.27 mm) at a frequency of 5 Hz (Table 60). A 30 second interval was placed between deflections and #4 (#13M) rebar was used throughout. The concrete reached final set at 5.87 hours and the deflections were stopped at 6.02 hours after initiation of mixing the grout. The average time of testing was 1.08 days after initiation of mixing for the static samples and 1.09 days for the deflection samples.

The pullout test strength results are shown in Table 61. All six samples failed in a pullout mode. The average peak load withstood by the deflected samples was approximately 72% less than the average load of the static samples.

Table 60. C2-2 testing properties under 0.05 in. (1.27 mm) at 5 Hz.

	Time After Mix Initiation				Design Values		
Sample	Time to Test, days	Avg. Time to Test, days	Time to Final Set, hrs	Defl. Start, hrs	Defl. Amplitude, in. (mm)	Defl. Freq., Hz	Interval Between Defl., sec.
Static1	1.07	1.08	5.87	6.02	0.05 (1.27)	5	30
Static2	1.08						
Static3	1.10						
Deflected1	1.07	1.09					
Deflected2	1.09						
Deflected3	1.10						

Table 61. C2-2 load results under 0.05 in. (1.27 mm) at 5 Hz.

Sample	Failure Type	Maximum Load, lbs	Average Maximum Load, lbs	Standard Deviation
Static1	Pullout	6665		
Static2	Pullout	5993	6073	557
Static3	Pullout	5560		
Deflected1	Pullout	2301		
Deflected2	Pullout	1378	1686	533
Deflected3	Pullout	1380		

1 lb = 4.45 N

ANALYSIS OF BOND TEST RESULTS

The average peak load resisted by each set of pullout tests was the critical result obtained from each series of tests. The short bond length of 3 in. (7.6 cm) ensured that the actual bond stress was approximately uniform throughout an individual specimen. The material properties, dimensions, and testing were consistent throughout all tests. First, the average peak loads were compared among all of the static specimens. Then, the average peak loads were compared between the static and deflected specimens of each cast material to determine whether a particular deflection was detrimental to the bond.

Comparisons were made among the average peak loads of each series of pullout tests using three different methods. First, a comparison was made by finding the percent difference between the static and deflected samples. Second, the standard deviation was computed for the average peak load of each set of static samples. A comparison was then made to determine if the average deflected peak load was within three standard deviations of the average peak load of the static samples. Third, a two sample T-test was used to compute a

90% and 95% confidence interval for the average peak loads of the static tests. The average peak loads for the deflected samples were then compared to these confidence intervals.

The average deflections at peak loading were not consistent among samples. The majority of tests did not return valid data due to problems with the measurement equipment. In the samples that returned good deflection data, only very general conclusions were made.

The average peak load resisted by each set of static samples was compared to the set times, spread measurements, and compressive strengths (Figure 10 - Figure 12) for the associated cast material. The spread and set times were not observed to directly correlate with the peak bond strength or percent differences. Within the materials tested, the average peak static loads increased as the compressive strength of the associated material increased.

Average Peak Loads of Static Specimens

Figure 20 shows the average peak loads of the static sample sets for each material. The horizontal axis lists the material types and the design amplitude of the deflection in inches for each test. These labels are in the format "material – placement number". Table 62 contains the peak loads and the peak load per area for the static samples in each grout placement. The area was calculated based on the inner diameter of the reinforcing bars assuming a value of 0.50 in. (1.27 cm) throughout and a 3 in. (7.6 cm) embedment length.

All materials except E1, U1, and U2 varied in average peak load from 5400 to 10,680 lbs (24,030 to 47,530 N). The concrete materials C1 and C2 were on the lower end of the average peak value range while the materials G1, G2, and M1 were near the top of the range.

Materials U1, U2, and E1 varied in average peak load from 15,790 to 21,510 lbs (70,270 to 95,720 N). The upper limit of this bond strength was governed by the ultimate capacity of the rebar for materials E1 and U2. Material U1 was at the lower end of this range.

The average load per area followed the same patterns as the average peak load. Since the exposed bond area of each specimen was the same, the average load per area was directly proportional to the average peak load.

Table 62. Average Peak Static Loads.

Grout - Placement	Load, lbs	Load per Area, psi
G1 – 1	8704	1847
G1 – 2	7166	1521
G1 – 3	8941	1897
G1 – 4	9798	2079
G1 – 5	6963	1478
G2 – 1	9987	2119
G2 – 2	10,682	2267
M1 – 1	9790	2078
E1 – 1	20,843	4423
U1 – 1	15,794	3352
U1 – 2	17,035	3615
U2 – 1	21,512	4565
C1 – 1	7930	1683
C1 – 2	5887	1249
C2 – 1	5401	1146
C2 – 2	6073	1289

1 lb = 4.45 N, 1 psi = 6.89 kPa

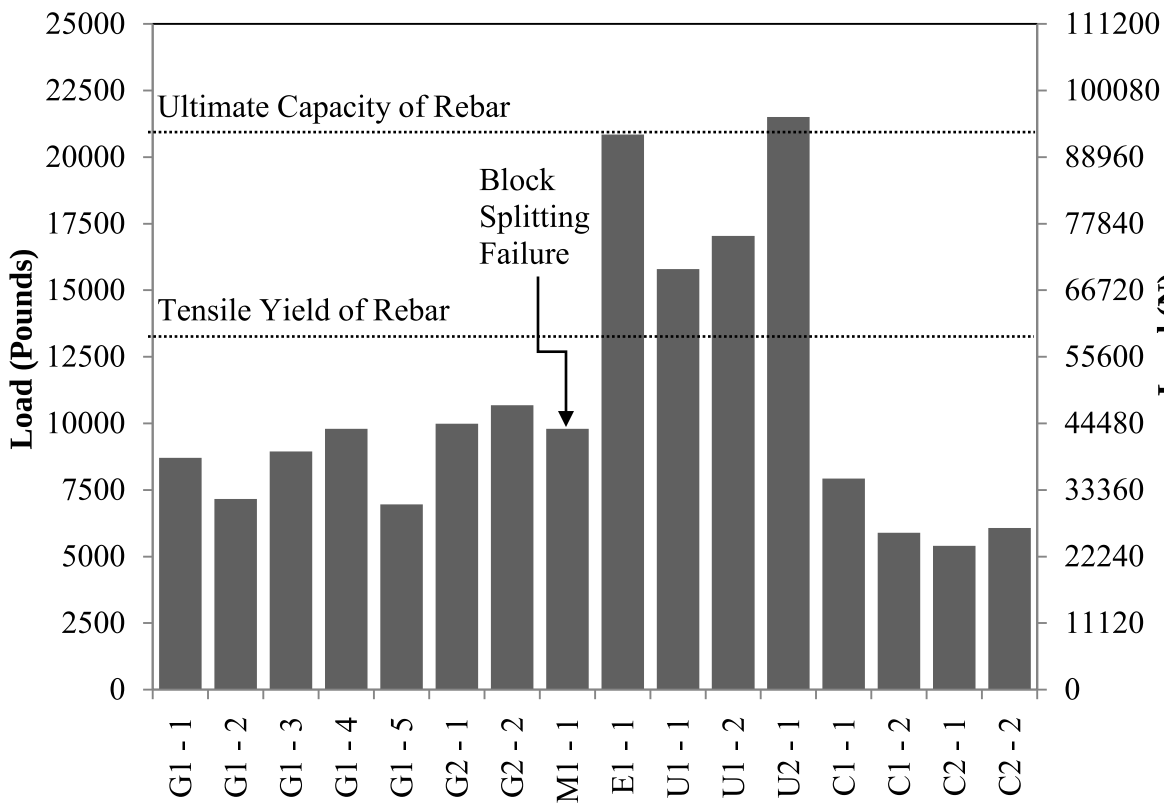

Figure 20. Graph. Average peak pullout loads for static specimens of each material. Label indicates "material - casting".

Average Peak Load Capacity Comparison

Figure 21 shows the average peak loads of the static and deflected sample sets for each material. The horizontal axis lists the material types and the design amplitude of the deflection in inches for each test. These labels are in the format “material – placement number : design amplitude of deflection in inches”.

The U2 and E1 materials developed bonds that caused the rebar to fracture at peak. The bond was not the limiting case since the deflected and static samples did not fail by rebar pullout. The M1 grout was the only set of samples that failed by splitting of the sample. All of the remaining samples failed the bond between the rebar and embedment material.

The test series with deflections greater than or equal to 0.05 in. (1.27 mm) showed significant reductions in peak load capacity. Materials C1, C2, G1, and G2 show this pattern of peak strength reduction.

The test series with deflections equal to 0.005 in. (0.127 mm) showed no reduction in peak load, with some even showing a slight increase in capacity. This was true for three sets of samples, namely two G1 samples and one U1 sample.

The test series with deflections of 0.01 in. (0.254 mm) had minor reductions in the peak load capacity. This deflection represents the point when the capacity reduction became apparent in many of the materials.

The first three samples on the left side of the chart were performed with deflections run at 2 Hz unlike the rest of the table that had deflections at 5 Hz. Deflection frequency was not observed to impart any appreciable difference in the results.

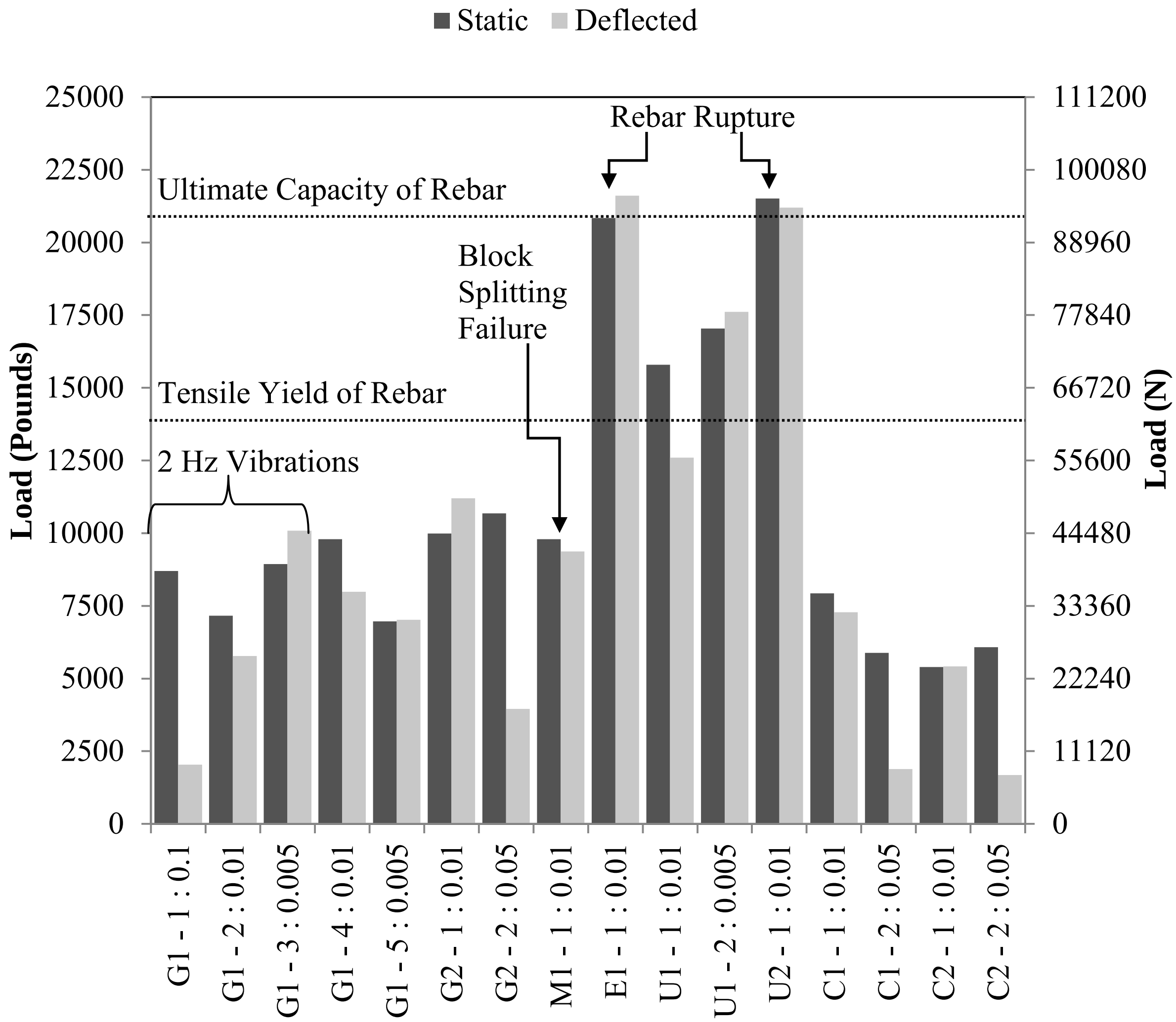

Figure 21. Graph. Average peak pullout loads for each material. Label indicates "material - casting : deflection amount (inches)".

Percent Difference of Average Peak Loads

The percent difference between the static and deflected samples was made for each of the 16 sets of tests. The results are presented in the order they were tested in Table 63. The static test results were treated as the control while the deflected sample results were used as the variable. The average peak loads for each set of 3 samples were used throughout.

The first five tests were done with the standard grout G1. In the first test the largest deflection, 0.1 in. (2.5 mm), was used. As seen in Table 63 the largest percent difference, 76.5%, resulted. It is also shown that the G1 samples with 0.01 in. (0.25 mm) of deflection reduced the strength by -18.6% and -19.5%. The deflection frequency made very little difference. When the G1 samples underwent a deflection of 0.005 in. (0.13 mm) the peak load increased by 0.8% and 12.8%. The lower frequency increased the strength 12% more than the higher frequency. Based on these tests it was determined that the frequency makes little difference, therefore the 5 Hz was chosen throughout the rest of testing. For the rest of the materials tested, a deflection of 0.01 in. (0.25 mm) was chosen as the initial deflection.

At a deflection of 0.01 in. (0.25 mm) the different materials reacted differently. The G1 and U1 materials reduced in average peak load capacity between 18.6% and 20.3%. The E1, G2, and C2 materials increased in peak load capacity between 0.4% and 12.2%. The U2, M1, and C1 materials had reductions between 1.5% and 8.2%. Based on this data, only the G1 and U1 materials experienced significant reductions in peak load with deflections of 0.01 in. (0.25 mm).

The G1 and U1 materials were tested at a deflection of 0.005 in. (0.13 mm). In all three tests the average peak load went up between 0.8% and 12.8%. These results indicate that a very small deflection will have no major impact or even a slight positive impact on the peak bond capacity.

The C1, G2, and C2 materials were tested with a deflection of 0.05 in. (1.27 mm). In all three cases significant reductions were demonstrated in the average peak pullout capacity. The reductions ranged from 62.9% to 72.2%.

Based on the percent difference comparisons, a lateral deflection of the bonded rebar smaller than 0.01 in. (0.25 mm) does not significantly affect the average peak load. A value of 0.05 in. (1.27 mm) or greater will significantly affect the average peak load. The deflection of 0.01 in. (0.25 mm) was observed to be the transition point for most materials.

Table 63. Percent Difference of Average Peak Loads.

Grout	Amplitude, in. (mm)	Deflection Frequency, Hz	Percent Difference* - Average Peak Loads - (Static versus Deflected)
G1 - 1	0.1 (2.5)	2	-76.5
G1 – 2	0.01 (0.25)	2	-19.5
G1 – 3	0.005 (0.13)	2	12.8
G1 – 4	0.01 (0.25)	5	-18.6
G1 – 5	0.005 (0.13)	5	0.8
G2 – 1	0.01 (0.25)	5	12.2
G2 – 2	0.05 (1.27)	5	-62.9
M1 – 1	0.01 (0.25)	5	-4.3
E1 – 1	0.01 (0.25)	5	3.6
U1 – 1	0.01 (0.25)	5	-20.3
U1 – 2	0.005 (0.13)	5	3.4
U2 – 1	0.01 (0.25)	5	-1.5
C1 – 1	0.01 (0.25)	5	-8.2
C1 – 2	0.05 (1.27)	5	-68.0
C2 – 1	0.01 (0.25)	5	0.4
C2 – 2	0.05 (1.27)	5	-72.2

* Negative values indicate that the deflected specimen exhibited a reduction in average peak load.

Standard Deviation Comparison of Average Peak Loads

The standard deviation comparison was performed on each set of 16 tests. The average and standard deviation for each set of peak static specimen loads was computed. The standard deviation was then multiplied by 3. This value was added to and subtracted from the average value to create a range of loads. This range was compared to the average peak load of the deflected samples. Based on this statistical comparison, if the value was within the range, then the sample was considered similar in strength, whereas if the value was outside the range, then it was considered significantly different.

The standard deviation test was not as sensitive to peak load deviations as were other analysis methods. As seen in Table 64, all the deflections with amplitudes of 0.05 in. (1.27 mm) or greater were significantly different. Save one U1 sample set and one G1 sample set, all the tests with amplitudes less than or equal to 0.01 in. (0.25 mm) were not significantly different. Specifically, the G1 sample set with an amplitude of 0.005 in. (0.13 mm) exhibited a greater capacity after being subjected to the deflection. The U1 sample set with an amplitude of 0.01 in. (0.25 mm) was close to the lower limit. Its pullout capacity was still above that exhibited by every other non-UHPC or non-epoxy grout specimen set.

Table 64. Three Standard Deviation Comparison of Average Peak Loads.

Grout	Amplitude, in. (mm)	Deflection Frequency, Hz	Average Peak Load – Deflected Samples, lbs	Significance Range for Peak Loads		Significantly Different, (Yes/No)
				Lower Bound, lbs	Upper Bound, lbs	
G1 – 1	0.1 (2.5)	2	2043	7973	9435	YES
G1 – 2	0.01 (0.25)	2	5772	5633	8699	NO
G1 – 3	0.005 (0.13)	2	10,090	8004	9878	YES
G1 – 4	0.01 (0.25)	5	7980	7556	12,039	NO
G1 – 5	0.005 (0.13)	5	7016	6240	7687	NO
G2 – 1	0.01 (0.25)	5	11,206	8313	11,661	NO
G2 – 2	0.05 (1.27)	5	3958	9429	11,934	YES
M1 - 1	0.01 (0.25)	5	9371	7334	12,246	NO
E1 – 1	0.01 (0.25)	5	21,604	*	*	NO
U1 – 1	0.01 (0.25)	5	12,593	13,893	17,694	YES
U1 – 2	0.005 (0.13)	5	17,610	14,269	19,800	NO
U2 - 1	0.01 (0.25)	5	21,199	*	*	NO
C1 – 1	0.01 (0.25)	5	7281	4019	11840	NO
C1 – 2	0.05 (1.27)	5	1883	4824	6949	YES
C2 - 1	0.01 (0.25)	5	5422	3413	7389	NO
C2 - 2	0.05 (1.27)	5	1686	4402	7743	YES

*Rebar rupture; no bond failure.

1 lb = 4.45 N

The U2 and E1 samples were not compared with this test. In both cases the rebar bond to the embedment material was stronger than the tensile capacity of the rebar. Therefore, the statistics could not be used to compare bond strengths for these particular tests because the bond strengths were not known.

In general, the deflections with amplitudes greater than 0.05 in. (1.27 mm) were significantly different. Deflections less than this did not cause significant reductions in the bond strength.

Statistical t-Test of Average Peak Loads

The statistical t-Test of the average peak loads was performed on each set of 16 tests. The two sample t-Test and corresponding confidence interval was used. The *pooled t procedure* was chosen. This was done for a number of reasons. The first was there were two unique samples with one difference, specifically, the average peak load. Second, the two sets of data would theoretical have the same distribution and standard deviation if the deflections had no effect on the outcome. In this case the values could be “pooled together” to estimate the variance. Third, both populations were assumed normal and the sample variances were assumed to be a good representation of the population variances [31].

The pooled population estimator was first computed and used in place of the standard deviation. This was based on using the values from both populations. Because the null hypothesis was that no significant change existed between the average peak loads, pooling or weighting the estimator was valid. The standard deviation and average should be the same if no change occurred. The equation is shown in Figure 22.

$$S_p^2 = \frac{(m - 1)*S_1^2 + (n - 1)*S_2^2}{m + n - 2}$$

S_p	=	Pooled Estimator
m	=	Sample size of static samples
S_1	=	Standard deviation of static samples
n	=	Sample size of deflected samples
S_2	=	Standard deviation of deflected samples

Figure 22. Equation. Pooled Population Estimator.

The test statistic value was computed next inserting the S_p value in for the standard deviation for the pooled tests as shown in Figure 23. The assumption in the Null hypothesis is that the population means are the same. Therefore, $\mu_1 - \mu_2$ would be equal to 0.

$$t = \frac{x - y - (\mu_1 - \mu_2)}{S_p\sqrt{\frac{1}{m} + \frac{1}{n}}}$$

x	=	Sample mean of static samples
y	=	Sample mean of deflected samples
μ_1	=	Population mean of static samples
μ_2	=	Population mean of deflected samples

Figure 23. Equation. Test Statistic Value.

Both 90% and 95% confidence intervals were determined for the static and deflected bond test results. A two tail test was chosen because the averages could be both higher or lower than one another. The critical values for a t distribution at 95% is 2.776 and at 90% is 2.132 [31]. The deflected bond test results were then compared to the static results in order to determine if the static and deflected results could be considered statistically equal. Table 65 contains the statistics for the t test and presents the results.

The t-tests had similar results as the percent difference and standard deviation tests. All samples with deflections of 0.05 in. (1.27 mm) or greater had a significant difference with 90% and 95% confidence intervals. All samples with deflections of 0.01 in. (0.25 mm) or less except for two were not significantly different. The only exceptions were the U1 and G1 materials deflected at 0.01 in. (0.25 mm). Both materials had significant differences under the 90% confidence interval and were very close to the 95% confidence interval.

The U2 and E1 materials were not compared with this test. In both cases the bonds were stronger than the rebar ultimate tensile capacity. Therefore, the statistics could not be used to compare bond strengths for these particular tests because the bond strengths were not known.

In general, the deflections with amplitudes greater than 0.05 in. (1.27 mm) were significantly different. Deflections less than this did not tend to cause significant changes in the bond strength.

Table 65. Statistical t-Test of average peak loads.

Grout	Amplitude, in. (mm)	Deflection Frequency, Hz	Pooled Estimator, S_p^2	Pooled T Test Value	Are Static and Deflected Bond Strengths Equal? 95 % Confidence Interval	90 % Confidence Interval
G1 – 1	0.1 (2.5)	2	31,048	46.3	NO	NO
G1 – 2	0.01 (0.25)	2	243,889	3.5	NO	NO
G1 – 3	0.005 (0.13)	2	546,283	1.9	YES	YES
G1 – 4	0.01 (0.25)	5	764,141	2.5	YES	NO
G1 – 5	0.005 (0.13)	5	529,659	0.1	YES	YES
G2 – 1	0.01 (0.25)	5	537,788	2.0	YES	YES
G2 – 2	0.05 (1.27)	5	95,525	26.6	NO	NO
M1 - 1	0.01 (0.25)	5	1,035,314	0.5	YES	YES
E1 – 1	0.01 (0.25)	5	*	*	*	*
U1 – 1	0.01 (0.25)	5	1,025,678	3.9	NO	NO
U1 – 2	0.005 (0.13)	5	575,275	0.9	YES	YES
U2 - 1	0.01 (0.25)	5	*	*	*	*
C1 – 1	0.01 (0.25)	5	1,028,419	0.8	YES	YES
C1 – 2	0.05 (1.27)	5	160,406	12.2	NO	NO
C2 - 1	0.01 (0.25)	5	446,293	0.0	YES	YES
C2 - 2	0.05 (1.27)	5	296,689	9.9	NO	NO

*Rebar rupture; no bond failure.

Average Deflections at Peak Loads

During the completion of the test program, it was recognized that the LVDTs frequently ceased to work correctly and/or provided insufficient or incorrect data. These instrumentation issues were resolved and appropriate data was collected for tests: G1-2, G2-1, G2-2, C2-1, and C2-2. Table 66 through Table 70 contain the data for each test, respectively. Although this limited amount of data is presented, solid conclusions are not formed.

The average deflections at the free end of the rebar in each pullout test are recorded in Table 71. A percent difference is computed of the two values. Note that positive values mean the deflected sample's average deflection at peak load was greater than the static sample. The deflections of the static and deflected sample sets were compared and in all cases they were within 7% of one another. In general, the deflections had very little variation based on this data.

Another comparison was made between the standard deviations of the samples. As shown in Table 72, the standard deviations as a percent of the average peak loads varied from 6.8% to 20.3%. Considering the inherent variability in this type of pullout test method, these results are considered close.

Based on the limited deflection data on the free end of the rebar for the G2 grout and C2 concrete, it was observed that the deflections may be recorded. There appears to be little difference in the deflection of the rebar at peak load, however this is not conclusive since the data is incomplete. In the future better instrumentation could provide better rebar deflection results.

Table 66. G1-2 deflection results under 0.01 in. (0.25 mm) deflections at 2 Hz.

	Loaded End of the Rebar			Free End of the Rebar		
Sample	**Rebar Deflection, in.**	**Average Rebar Deflection, in.**	**Standard Deviation, in.**	**Rebar Deflection, in.**	**Average Rebar Deflection, in.**	**Standard Deviation, in.**
Static1	0.0505			0.0403		
Static2	0.0575	0.0584	0.0084	0.0561	0.0441	0.0106
Static3	0.0673			0.0360		
Deflected1	0.0533			0.0441		
Deflected2	0.0634	0.0570	0.0056	0.0501	0.0503	0.0064
Deflected3	0.0542			0.0568		

Note: 1 in. = 2.54 cm

Table 67. G2-1 deflection results under 0.01 in. (0.25 mm) at 5 Hz.

Sample	Loaded End of the Rebar: Rebar Deflection, in.	Loaded End of the Rebar: Average Rebar Deflection, in.	Loaded End of the Rebar: Standard Deviation, in.	Free End of the Rebar: Rebar Deflection, in.	Free End of the Rebar: Average Rebar Deflection, in.	Free End of the Rebar: Standard Deviation, in.
Static1	0.0880			0.0520		
Static2	0.0574	0.0707	0.0157	0.0400	0.0433	0.0076
Static3	0.0668			0.0380		
Deflected1	0.0596			0.0500		
Deflected2	0.0657	0.0648	0.0048	0.0450	0.0483	0.0029
Deflected3	0.0690			0.0500		

Note: 1 in. = 2.54 cm

Table 68. G2-2 deflection results under 0.05 in. (1.27 mm) at 5 Hz.

Sample	Loaded End of the Rebar: Rebar Deflection, in.	Loaded End of the Rebar: Average Rebar Deflection, in.	Loaded End of the Rebar: Standard Deviation, in.	Free End of the Rebar: Rebar Deflection, in.	Free End of the Rebar: Average Rebar Deflection, in.	Free End of the Rebar: Standard Deviation, in.
Static1	0.0622			0.0410		
Static2	0.0653	0.0687	0.0087	0.0470	0.0517	0.0136
Static3	0.0786			0.0670		
Deflected1	0.0619			0.0530		
Deflected2	0.0748	0.0746	0.0125	0.0510	0.0563	0.0076
Deflected3	0.0870			0.0650		

Note: 1 in. = 2.54 cm

Table 69. C2-1 deflection results under 0.01 in. (0.25 mm) at 5 Hz.

Sample	Loaded End of the Rebar: Rebar Deflection, in.	Loaded End of the Rebar: Average Rebar Deflection, in.	Loaded End of the Rebar: Standard Deviation, in.	Free End of the Rebar: Rebar Deflection, in.	Free End of the Rebar: Average Rebar Deflection, in.	Free End of the Rebar: Standard Deviation, in.
Static1	0.0592			0.0540		
Static2	0.0408	0.0532	0.0107	0.0300	0.0457	0.0136
Static3	0.0595			0.0530		
Deflected1	0.0622			0.0510		
Deflected2	0.0485	0.0528	0.0081	0.0420	0.0443	0.0059
Deflected3	0.0478			0.0400		

Note: 1 in. = 2.54 cm

Table 70. C2-2 deflection results under 0.05 in. (1.27 mm) at 5 Hz.

	Loaded End of the Rebar			Free End of the Rebar		
Sample	Rebar Deflection, in.	Average Rebar Deflection, in.	Standard Deviation, in.	Rebar Deflection, in.	Average Rebar Deflection, in.	Standard Deviation, in.
Static1	0.0475			0.0360		
Static2	0.0418	0.0441	0.0030	0.0340	0.0343	0.0015
Static3	0.0431			0.0330		
Deflected1	0.0408			0.0420		
Deflected2	0.0573	0.0494	0.0083	0.0540	0.0483	0.0060
Deflected3	0.0503			0.0490		

Note: 1 in. = 2.54 cm

Table 71. Average deflections in the pullout tests of the rebar's free end at the peak load.

Grout	Amplitude, in.	Deflection Frequency, Hz	Average Deflection Static Samples, in.	Average Deflection Deflected Samples, in.	Percent Difference
G1 – 2	0.01	2	0.0622	0.0600	-3.56
G2 – 1	0.01	5	0.0760	0.0707	-7.02
G2 – 2	0.05	5	0.0743	0.0767	3.14
C2 - 1	0.01	5	0.0560	0.0557	-0.60
C2 - 2	0.05	5	0.0473	0.0503	6.34

Note: 1 in. = 2.54 cm

Table 72. Comparisons of the standard deviations to the average deflections of the rebar at the free end at the peak load.

Grout	Amplitude, in.	Deflection Frequency, Hz	Static Standard Deviation as a percent of the Average	Deflected Standard Deviation as a percent of the Average
G1 – 2	0.01	2	13.9	9.0
G2 – 1	0.01	5	20.3	7.3
G2 – 2	0.05	5	11.4	16.3
C2 - 1	0.01	5	18.6	14.5
C2 - 2	0.05	5	6.8	15.9

Note: 1 in. = 2.54 cm

CHAPTER 5. CONCLUSIONS

TEST SUMMARY

The bond strength between rebar and an embedment material is commonly formed at a time when differential deflections may be induced between the two materials and prior to the final set of the embedment material. A pullout test was devised to test the early age bond strength of the embedment system, with the control variable being the differential deflections between the rebar and embedment material. The test was for comparison of only this variable, the differential deflections, with the realization that actual bond strengths in structures may vary depending on the application. The deflections were induced from the time of casting until the material reached final set. The pullout test was then performed at 24 hours after casting or as soon thereafter as possible.

Eight materials were chosen because they each represented a typical material for their particular category. The different materials included: Five Star grout (G1), BASF Embeco 885 grout (G2), BASF Set 45 grout (M1), Five Star epoxy grout (E1), Euclid Cable Grout PTX (T1), Lafarge Ductal JS1000 (U1), Lafarge Ductal JS1000-RS (U2), normal slump Virginia A4 concrete (C1), and high slump Virginia A4 concrete (C2). The identifier used throughout this report is shown in parentheses.

The pullout test method employed was a modified version of ASTM C234. The modifications allowed for the application of the differential deflections and the use of high early strength materials. The forms were built with a malleable seal between the reinforcing bar and formwork to allow for the differential deflections. Number 4 (#13M) bars were used throughout with an embedment distance of 3 in. (7.6 cm) or six times the diameter of the bar. The short bond length in relation to the size of the rebar helped create a relatively constant bond stress throughout. A bond breaker was added to the other half of the rebar in the 6 in. (15.2 cm) forms in order to help alleviate compressive stresses near the bearing plate.

All of the materials reached final set within 24 hours. All materials except for U1 had achieved sufficient compressive strength to allow the pullout testing to be completed by approximately 24 hours after initiation of grout mixing. U1 specimens were tested at approximately 48 hours.

Each set of tests consisted of two sets of three specimens. The peak load values were averaged and then the averages were compared between tests. Three different statistical methods were used to assess the peak load applied to each pullout specimen. The first method was the percent difference between the ultimate load of the deflected and static samples. The second method included comparing the peak load capacity of the deflected samples with a range three standard deviations away from the average of the static sample. The last method included using 90% and 95% confidence intervals with a t-Test.

CONCLUSIONS

1. From the perspective of the pure bond strength, deflecting the rebar prior to final set of the embedment material can have a detrimental effect on the bond. When the rebar deflected 0.05 in. (1.27 mm) or more, reduced bond capacity was observed. Deflections of 0.01 in. (0.25 mm) or less created only small changes in the bond strength. The results indicate this range is near the transition point for the embedment materials tested.
    - All materials were tested with a differential deflection of 0.01 in. (0.25 mm), with the results indicating only minor variations in bond strength. The results ranged from a peak load capacity reduction of 20% to a peak load capacity increase of 12%. Materials G1, U1, U2, M1, and C1 experienced reductions in bond strength. Materials E1, G2, and C2 exhibited increases in bond strength. In general the variations were small and are considered minor deviations. Note that M1 specimens exhibited a splitting failure, while all of the other specimens exhibited a pullout failure.
    - Significant reductions in bond strength were observed in all of the samples deflected 0.05 in. (1.27 mm) or more. This included tests on materials G1, G2, C1, and C2.
    - In general, a differential deflection of 0.005 in. (0.13 mm) has little detrimental effect on the bond and it may have a small beneficial effect. Materials G1 and U1 were deflected 0.005 in. (0.13 mm) to determine the impact of this deflection, and the results indicated an increase in peak bond strength.
2. Deflection frequencies of 2 Hz and 5 Hz were not observed to affect the bond capacity results based on tests with embedment material G1. These deflection frequencies have previously been reported as typically occurring on bridge superstructures.
3. Materials E1 and U2 exhibited the highest static bond strengths. With the 3 inch (76 mm) embedment, #4 (#13M) rebar cast into E1 and U2 ruptured at their ultimate tensile capacity. The compressive strengths of these materials at the time of pullout testing were approximately 12 ksi and 14 ksi (83 and 96 MPa), respectively.
4. The test method engaged in the test program proved to be an appropriate mechanism through which to assess the impact of differential deflection during staged construction on the bond performance of a rebar to field-cast grout. The specimen geometry and loading protocols allowed the majority of test specimens to fail in a pullout mode, thus providing an indication of the pullout resistance of the bond. The lone exceptions were grouts M1, U2, and E1.
    - Grout M1 produced splitting failures. This type of failure is the result of a low tensile strength in relation to the local bond interaction between the bar and the grout. For materials of this type, further modification of the test may be necessary.

- Grouts U2 and E1 produced rebar tensile failures. For specimens producing this type of failure, a shorter bond length may be appropriate or earlier testing (less than 24 hours) at a lower compressive strength.

RECOMMENDATION FOR FUTURE RESEARCH

The research findings presented herein provide an initial indication of some of the behaviors which might be encountered with differentially deflected staged construction connections. Further research on this topic is warranted. Topics of investigation could include:

- additional testing of the investigated embedment materials with differential deflections between 0.01 and 0.05 inches (0.25 and 1.27 mm),
- testing of epoxy-coated reinforcing bars, and
- testing of alternative embedment materials.

The pure bond strength is not the only factor affecting the performance of reinforcing bars in staged construction connections. Future research may focus on additional testing variables. These variables could include longer bond lengths using flexure tests that simulate more realistic stress conditions in beams or bridge decks. The magnitudes of differential deflections may or may not affect longer straight or hooked rebar embedment configurations. Additionally, the embedment lengths may be affected by the stiffness of the rebar during the differential deflections, as well as the rebar's positive connections to other parallel or intersecting bars.

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