PERFORMANCE OF GROUTED CONNECTIONS FOR PREFABRICATED BRIDGE ELEMENTS PART II: Component-Level Investigation on Bond and Cracking

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ABSTRACT

This paper discusses the behavior of different grout connection materials in series of large-scale deck-level connection subassembly tests. Non-shrink cementitious, magnesium phosphate, epoxy, and ultra-high performance concrete (UHPC) grouts were investigated. Deck panel specimens were subjected to low-level cyclic loading, fatigue loading, and monotonic ultimate loading. One major focus was the bond strength between precast concrete and the closure grout. The different grout materials were cast against precast concrete with different levels of surface roughness. Results indicate that the bond strength between the connection grout and precast concrete is heavily influenced by both the grout type and precast concrete surface preparation. A second focus of the study was the resistance of the grouted connection region to cracking during applied loading. It was found that cracking within the grouted connection region was also highly dependent on the type of grout material. The research presented in this paper is the second part of a larger study on connection grout materials. The first part of the study is presented in a separate paper, and discusses material-level testing of the aforementioned grout materials for shrinkage and bond properties. Discussion will correlate results from both studies, and design recommendations will be provided.

Keywords: Deck-level Connection, Fatigue, Cyclic loading, Interface, Shrinkage, Bond Strength

INTRODUCTION

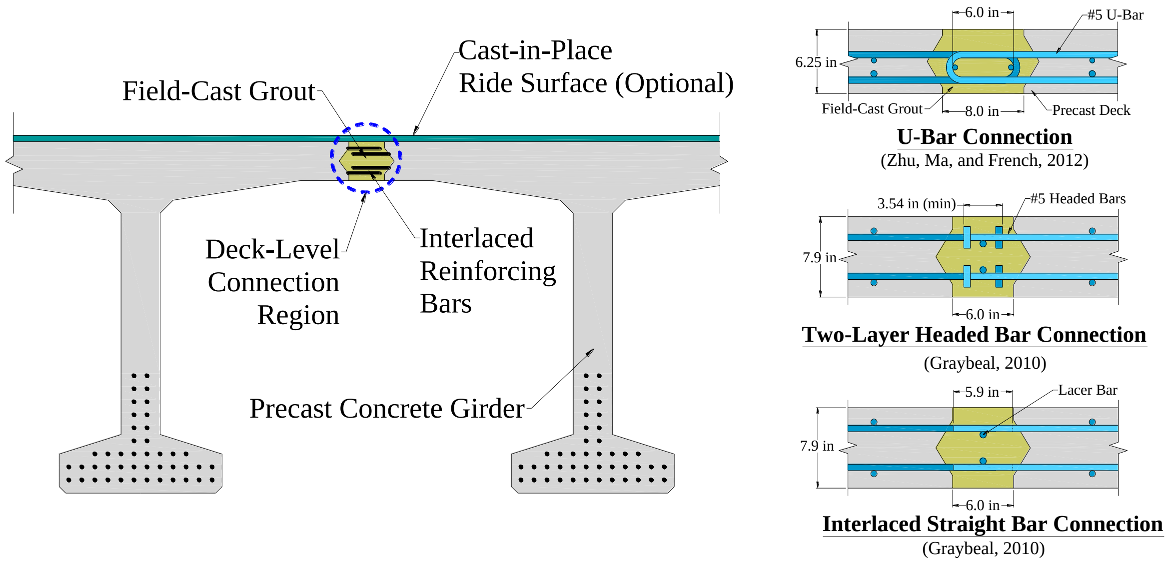
BACKGROUND

Accelerated bridge construction (ABC) has become increasingly popular for new bridges and for replacement/rehabilitation projects given its numerous advantages, which include reduced traffic disruption, expedited project delivery, and increased work zone safety. In order to realize some of these advantages, prefabricated bridge elements and systems (PBES) are typically employed. Prefabricated reinforced concrete elements can be manufactured off-site, in parallel with on-site construction tasks, and can have superior quality compared with cast-in-place concrete elements. Once delivered to site, prefabricated elements can be rapidly assembled to form the bridge structure.

The performance of prefabricated bridge systems is highly dependent on the design and detailing of connections between elements. Typically, elements are joined using field-cast grout (or cast-in-place concrete) cast over interlaced reinforcing bars, or into ducts used to embed bars into adjacent elements. The selection of the connection grout and the detailing of the connection can have a significant impact on both the structural performance and long-term durability of the connection and the system. Furthermore, if the reinforcement or geometric details of the prefabricated element or its connection are too complex, element construction and field installation can become difficult. This can result in project delay and can compromise the integrity of the prefabricated system and its connection. Thus, constructible, robust connection details are desirable.

The research presented in this paper is the second part of a larger study on connection grout materials for prefabricated bridge elements (PBE). The first part of the study is presented in a separate paper (De la Varga, Haber, and Graybeal, 2016), and discusses material-level testing of different grout materials for shrinkage and bond properties. Four different pre-bagged grouts were investigated, namely, non-shrink cementitious grout, magnesium phosphate rapid-set grout, epoxy grout, and ultra-high performance concrete (UHPC) grout. The primary objective of the material-level (Part I) study was to assess the bond strength of these grout-type materials when bonded to precast concrete. Bond strength was assessed using a flexural beam test based on ASTM C78. Three precast concrete surface preparation methods were investigated: pressure washing (PW), sand blasting (SB), and an exposed aggregate (EA) surface preparation. Additionally, a number of practical strategies to improve the bond strength between connection grouts and precast concrete were proposed, and were validated by bond testing according to ASTM C1583. Along with improving bond to precast concrete, some of the proposed strategies also reduced both autogenous and drying shrinkage of the grout material. Thus, it was hypothesized that these two behaviors, bond strength and dimensional stability, are intra-related.

This paper focuses on how some pre-bagged grout materials tested by De la Varga et al. (2016) behave in series of prefabricated bridge deck connection tests. The connection type tested is referred to as a "deck-level" connection, which refers to those connections that do not contain shear connectors within the grouted connection region. Figure 1a shows an example of a deck-level connection between two adjacent precast, prestressed decked bulb-tee bridge girders. Deck-level connections employing straight, hooked, or headed non-contact lap-spliced bars have become popular given that they are easy to fabricate at the precasting yard and can be rapidly assembled with relatively minimal on-site detailing; examples of laboratory-tested deck-level connections are shown in Figure 1b.



(a) Decked bulb-tee girder with deck-level connection (b) Connection detail examples

Figure 1. Deck level connections with field-cast grouted connections

OBJECTIVE AND SCOPE

There are a number of design considerations that can have a significant impact on the long-term performance of PBE connections. Some of these design considerations include selection of the reinforcement details between adjacent prefabricated deck panels, selection of the field-cast grout that is used to fill the connection seam, and the geometry and detailing of precast concrete panel shear keys. Although a number of studies have been conducted to investigate how these different design considerations affect performance (Badie and Tadros, 2008; Graybeal, 2010; Li, Ma, Griffey, and Oesterle, 2010; and Zhu, Ma, and French, 2012), most studies have considered only a limited number of variable combinations or have focused on only certain aspects of deck-level connection performance. Given the current demand for PBES technology, further investigation of these types of connections is warranted.

The objective of the research presented in this paper was to advance the understanding of deck-level connections for prefabricated bridge decks. Deck-level connections employing interlaced reinforcing bars with different grout materials and different precast panel details were investigated for potential use in accelerated bridge construction projects. The work presented here builds upon and complements the research presented in the Part I paper presented by De la Varga et al. (2016). This study presented herein is broken down into two phases. In phase I, a series of large-scale deck panel connection tests were conducted using the same connection grout materials tested in the Part I paper. Along with using different connection grouts, a number of other parameters frequently considered during the design of these connections were investigated including lap splice length, reinforcement type, precast surface preparation, and shear key shape. The second phase of this research, which is currently on-going, focuses on a component-level investigation of methods to improve the performance of pre-bagged non-shrink cementitious grouts, and investigates a series of different UHPC-class materials for PBE connections.

EXPERIMENTAL PROGRAM

The first phase of the experimental program, which has been completed, consisted of 72 large-scale precast deck panel connection tests. Deck panel specimens were intended to have details representative of prefabricated bridge deck systems currently being employed in the field. Figure 2 shows an illustration summarizing the test variables, specimen details, and the specimen nomenclature. Test variables are shown in bold italics, and a list of the different variables is provided underneath along with the variable nomenclature, which is shown in parentheses. Specimens were identified by combining the test variable nomenclature into a single string. An example is shown in the lower left-hand side of Figure 2 identifying a specimen (denoted C-12-B-EA) with non-shrink cementitious grout (C), a 12-in lap splice, black rebars (B), and exposed aggregate (EA) precast concrete surface preparation. As noted previously, four different field cast grouts were investigated: non-shrink cementitious grout (denoted “C”), magnesium phosphate rapid-set grout (denoted “M”), epoxy grout (denoted “E”), and an ultra-high performance concrete (UHPC) grout (denoted “U”). Grout “C” met ASTM C1107 standards while the other three grouts tested did not have ASTM standard specifications. Lap splice lengths varied for each grout type. The majority of specimens had either 12-in (for specimens with grouts “C”, “M”, or “E”) or 5.5-in (specimens with grout “U”) splice lengths. It should be noted that not all possible combinations of test variables were investigated. For example, specimens that employed ultra-high performance concrete grout were only studied using 5.5-in lap splices.

Individual deck panel halves were reinforced with No. 5, grade 60 reinforcing bars spaced at 6 in. Specimens were constructed by placing two individual precast deck panels together such that the protruding rebar dowels interlaced between panels as shown in Figure 3a; the pocket shown is referred to as the “connection region”. The connection region was blocked off and sealed prior to grouting. Each grout material was mixed and placed according to manufacturer specifications, and was allowed to cure for at least 24 hours at controlled laboratory temperatures prior to removal of forms. Figure 3b depicts formwork and grout casting for a specimen using grout “U”.

Deck panels were tested in four-point bending using a servo-controlled hydraulic actuator as shown in Figure 4. The tension face of the panel was positioned upward to facilitate inspection during loading. The connection region was located within the constant moment region between the center supports. Specimens were instrumented with strain gages and displacement transducers over the precast panel-to-grout interface. Specimens were subjected to three different loading protocols applied in succession, which are discussed within individual results sections.

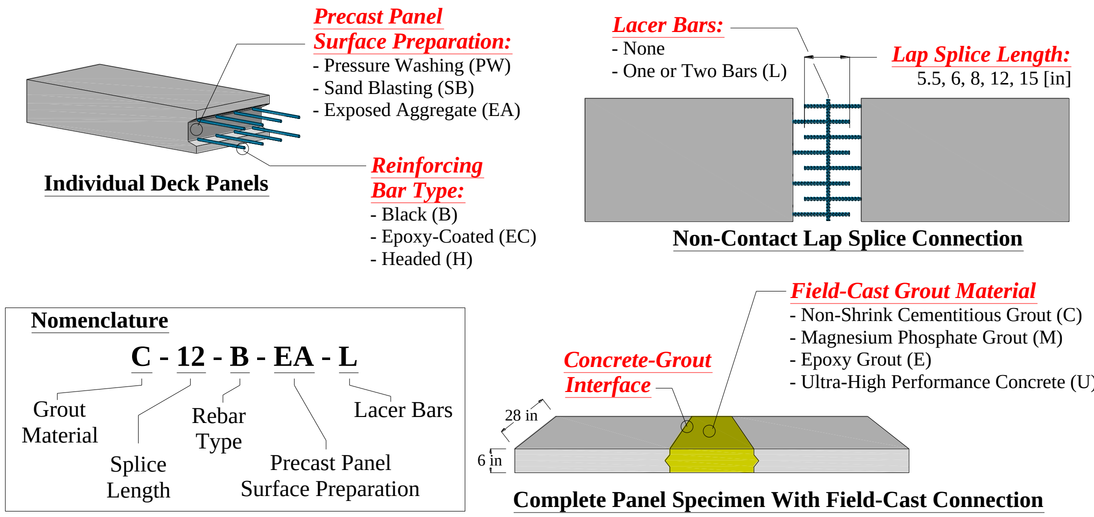


Figure 2. Test variables, specimen details, and specimen nomenclature

(a) Interlaced reinforcement

(b) Grout casting

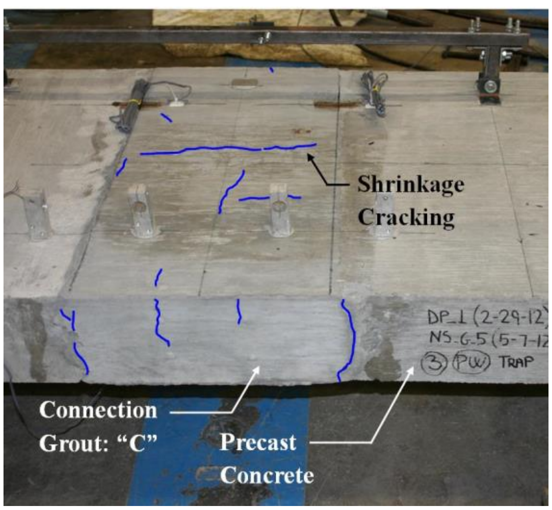
Figure 3. Specimen construction

Figure 4. Test set-up

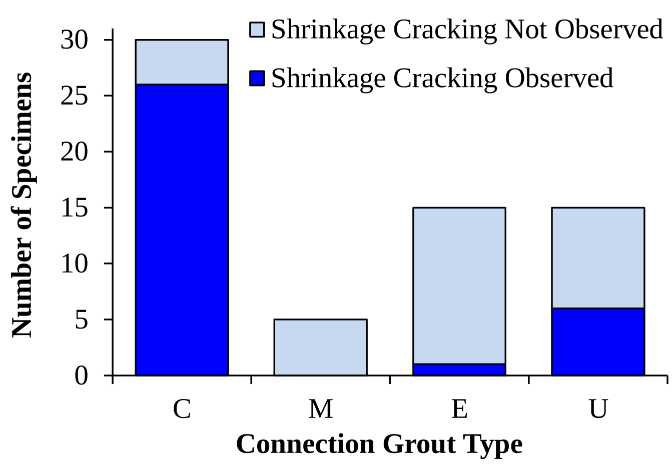
RESULTS FROM PHASE I STUDIES

PRE-TEST OBSERVATIONS

Shrinkage cracking in prefabricated deck panel connections can lead to durability-related problems. Furthermore, cracks initiated by shrinkage can propagate during mechanical and/or thermal loading resulting in damage and stiffness loss. Thus, prior to applying any mechanical loads, deck panel specimens were inspected for shrinkage cracks and other cracks resulting from handling. Particular attention was given to the grouted connection region. Shrinkage cracks found in this region were marked and photos were taken. Generally speaking, the majority of specimens employing the non-shrink cementitious grout "C" exhibited considerable shrinkage cracking within the grouted connection regions. Figure 5a shows a representative photo of the shrinkage cracking that was observed in specimens with grout "C". In many cases, cracks were found in both transverse and longitudinal directions. In some cases, cracks were found at the interface between the connection grout and precast concrete. Figure 5b quantifies the number of specimens per grout type that exhibited shrinkage cracking within the grouted connection region. Specimens grouted with the magnesium phosphate grout "M" grout did not exhibit shrinkage in the connection region. There was only a single specimen with epoxy grout "E" that exhibited slight shrinkage cracking. Specimens with UHPC grout "U" exhibited shrinkage cracking in the connection in 6 out of 15 panels. However, the shrinkage cracks found in the UHPC grout material were fine and mostly occurred near free edges and adjacent to the concrete-grout interface, and none were observed at the interface between the UHPC grout and precast concrete. These results are not unexpected. The dimensional stability of these four materials were previously evaluated by De la Varga and Graybeal (2014). In drying shrinkage tests conducted according to ASTM C1698 and ASTM C157, grouts similar to that of grout "C" exhibited as much as six times the shrinkage strain as grouts "M", "E", and "U" after 28 days.



(a) Observed shrinkage cracking



(b) Quantification of shrinkage cracking by connection grout type

Figure 5. Shrinkage cracking of specimens

CYCLIC CRACK LOADING

Deck panel specimens were first subjected to cyclic crack loading. This loading protocol had two primary objectives. The first was to evaluate grout-to-precast concrete interface behavior of the connection. That is, the resistance of each precast concrete surface preparation and connection grout combination to interface cracking under low-level cyclic loading; this could otherwise be referred to as the cyclic loading bond strength. The second was to evaluate each grout material's resistance to crack formation and propagation.

Due to field casting, a cold-joint forms between the precast deck panel and the grout material. As noted by De la Varga, et al. (2016) tensile bond strength of this interface, in some cases, is weaker than the tensile strength of precast concrete or the grout material. Premature cracking at this interface could lead to durability issues as a result of water or chloride intrusion through the crack opening. Thus, cracks occurring at the panel-grout interface during loading were of particular interest.

The loading protocol for this portion of the test program is shown in Figure 6. The calculated cracking moment, M_{cr}, for deck panel specimens was 6.98 kip-ft, which corresponded to an actuator force of 6.2 kips. The cracking moment was calculated according to provisions specified in section 5.7.3.3.2 of the AASHTO LRFD Bridge Design Specifications (AASHTO, 2010). Load was applied at a frequency of 5 Hz, and was cycled between 10% of M_{cr} and an upper load target which varied with the number of applied cycles. Five thousand cycles were applied for each upper load target which ranged from 30% to 120% of M_{cr}. An additional 50,000 cycles were applied at the 120% of M_{cr} target.

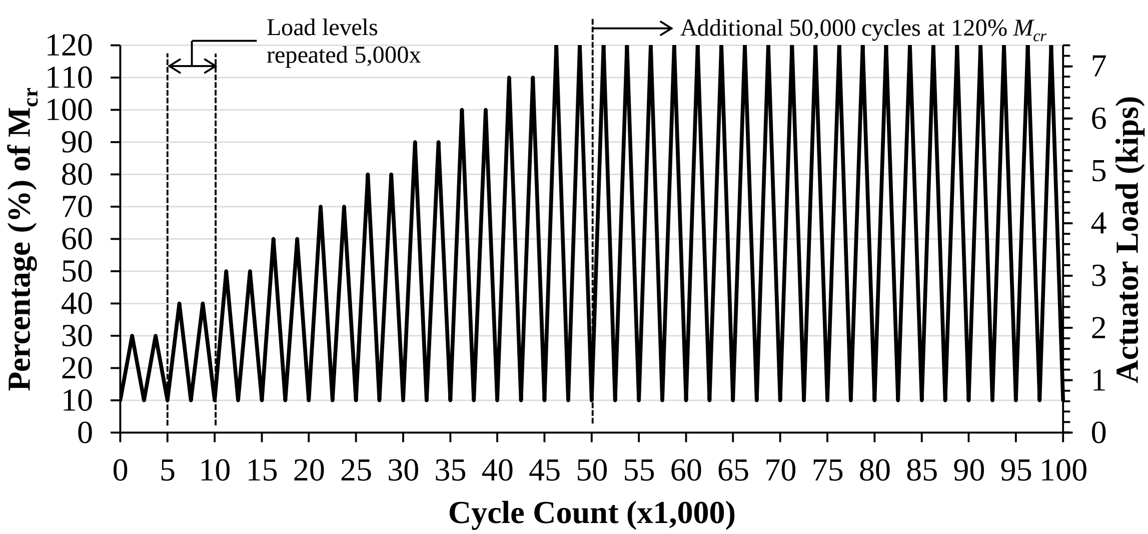


Figure 6. Cyclic crack loading protocol

The grout-to-precast concrete interface behavior was evaluated by two different methods. The first method was visual inspection. Visual inspections were conducted after the 80% (30,000 cycles), 100% (40,000 cycles), and 120% (50,000 cycles) of M_{cr} load levels, and after completion of the cyclic crack loading protocol (100,000 cycles). Quantitative results from visual inspections are presented in Figure 7. This figure depicts the number of specimens for each grout type that exhibited interface cracking after different load levels. It should be noted that precast concrete surface preparation is not specified here. This figure is to give a general sense of when interface cracking occurred for each grout type. As previously stated, the majority of specimens employing grout "C" were observed to have interface cracking caused by shrinkage regardless of the precast concrete surface preparation. In general, grout C and M both tended to exhibited interface cracking prior to 80% of M_{cr}. Specimens cast with grouts E and U tended to exhibit interface cracking at different load levels, which will be shown to be primarily a function of surface preparation.

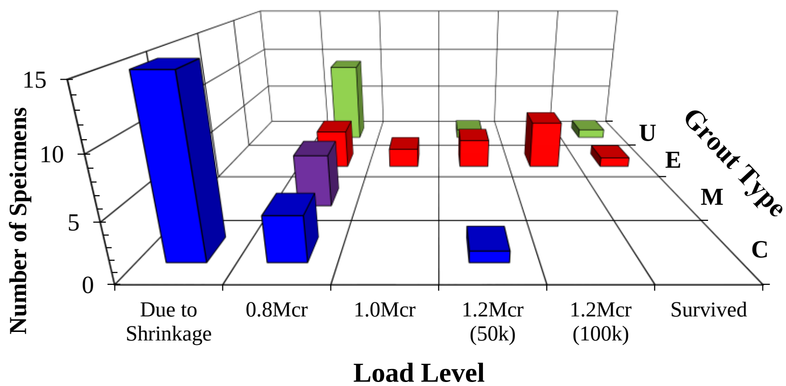


Figure 7. Interface cracking observed by visual inspection

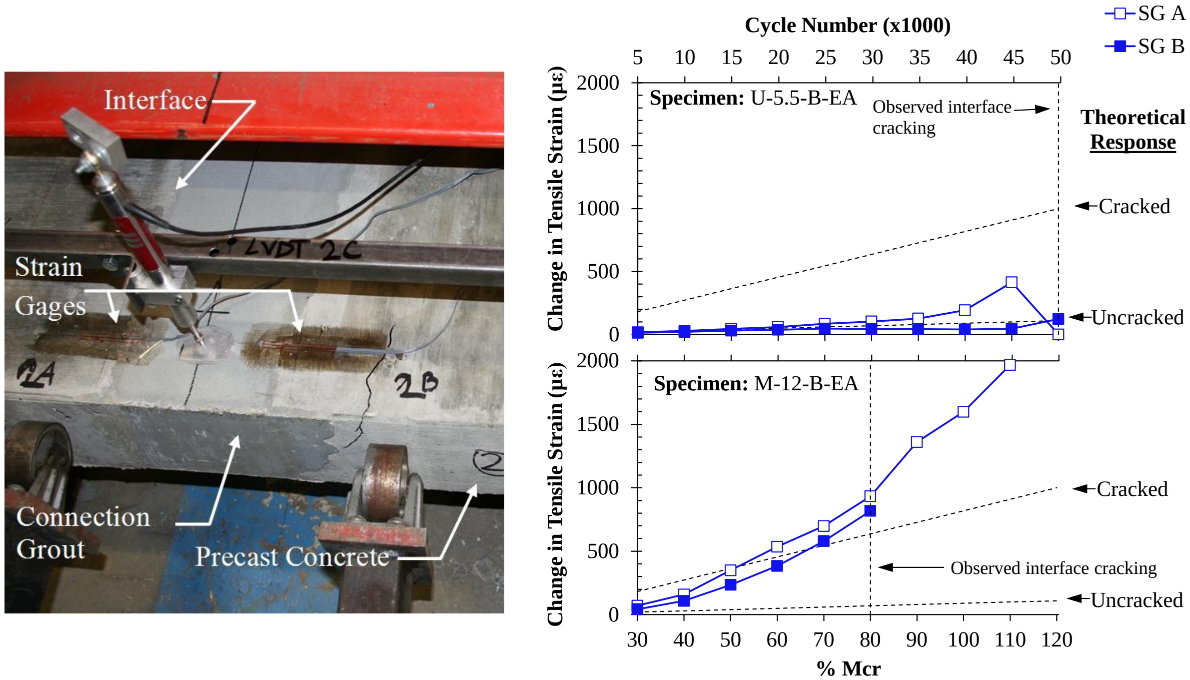
The second method used to evaluate the interface behavior was to compare the measured strains over the interface with the calculated theoretical response. After casting and curing of the connection grout, foil-backed resistive strain gages (SG) were installed such that the gage length spanned the interface of the two materials. This configuration is illustrated in Figure 8a. Data was recorded in bursts throughout each 5,000-cycle load level period. The measured results presented in this section reflect the last burst of data recorded for a given load level; this methodology captures the cumulative effect of the previous cycles at the same load level. The measured data and the calculated response are presented in the form of change in tensile strain, $\Delta\varepsilon$, for a given load level. The change in strain, $\Delta\varepsilon$, was defined as the difference of the maximum and minimum tensile strain for a given load level. Theoretically, when the deck panel section is uncracked, $\Delta\varepsilon$ should be relatively small. Once the section cracks the $\Delta\varepsilon$ for a given load level should increase significantly. In either case, the behavior should be linear-elastic, although there exists a difference in stiffness, and thus the relationship between $\Delta\varepsilon$ and the applied load should be linear before and after cracking.

Figure 8b depicts the relationship between $\Delta\varepsilon$ and the percentage of M_{cr}. Each plot shows a set of dashed lines that represent the calculated response for cracked and uncracked sections. Furthermore, each plot represents a deck panel specimen with different connection details. Both specimens employed black reinforcing bar and exposed aggregate (EA) surface preparation on precast concrete. The primary difference between the two specimens was the type of connection grout used; the upper and lower plots represent specimens that used grouts "U" and "M", respectively. Lastly, each plot notes the load level where visual inspection identified interface cracking.

Comparison of measured data to the calculated theoretical response indicated that the majority of specimens followed one of two distinct trends. These two trends can be observed in Figure 8b. The first trend, which is referred to as “late cycle bond failure”, can be seen in the upper plot for specimen “U-5.5-B-EA”. In this case, the measured response is similar to theoretical response for an uncracked section. The measured change in strain begins to increase more rapidly after 100% of M_{cr} has been reached. Thus, it is likely that the interface for this specimen cracked during the 110% of M_{cr} cycle set, which was not verified with visual inspection until after the 120% of M_{cr} cycle set.

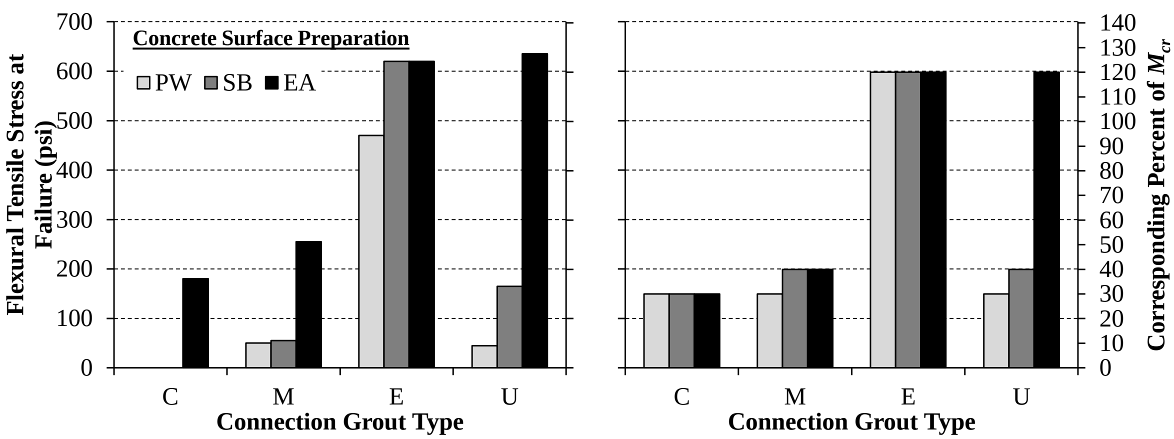
The second trend is referred to as “early-cycle bond failure”, and can be observed in the lower plot shown in Figure 8b, which represents specimen “M-12-B-EA”. In this case, the measured $\Delta\varepsilon$ begins to increase rapidly during the first few load levels. After cycling at 40% of M_{cr}, the measured strains have already well exceeded the mark for the theoretical uncracked section response, and after cycling at 50% of M_{cr} the measured $\Delta\varepsilon$ has exceed the theoretical response for a cracked section. In the case of specimen “M-12-B-EA” interface cracking was not visually observed until after the 80% of M_{cr} cycle set, but strain gage data suggests cracking occurred very early in the cyclic loading protocol.

This type of analysis was completed for each specimen. In general, evaluation of strain measured over the grout-to-concrete interface revealed that specimens employing grout “C” and “M”, and the majority of specimens employing grout “U”, cracked well before the first visual observation was made after the 80% of M_{cr} cycle set. In most cases, these specimens exhibited interface cracking prior to 60% of M_{cr} cycle set. Figure 9 compares results from the material-level bond tests discussed in Part I of this paper (Figure 9a) with results from interface strain measurements taken from deck panels in this study (Figure 9b). Results are presented as functions of precast concrete surface preparation and grout type. Furthermore, both plots have the same scale on the vertical axis, which depict bond failure of the grout-to-concrete interface. The two data sets exhibit very similar trends. That is, with the exception of specimens with grout “E”, pressure washed (PW) and sand blasted (SB) surface preparations did not promote good bonding between precast concrete and the grout material. The exposed aggregate (EA) surface preparation tended to increase bond strength between the grout materials and precast concrete. However, even when this preparation method was used in conjunction with grouts “C” and “M”, the bond strength was still low compared with grouts “E” and “U”.



(a) Interface instrumentation (b) Examples of interface behaviors

Figure 8. Determination of interface behavior and cracking

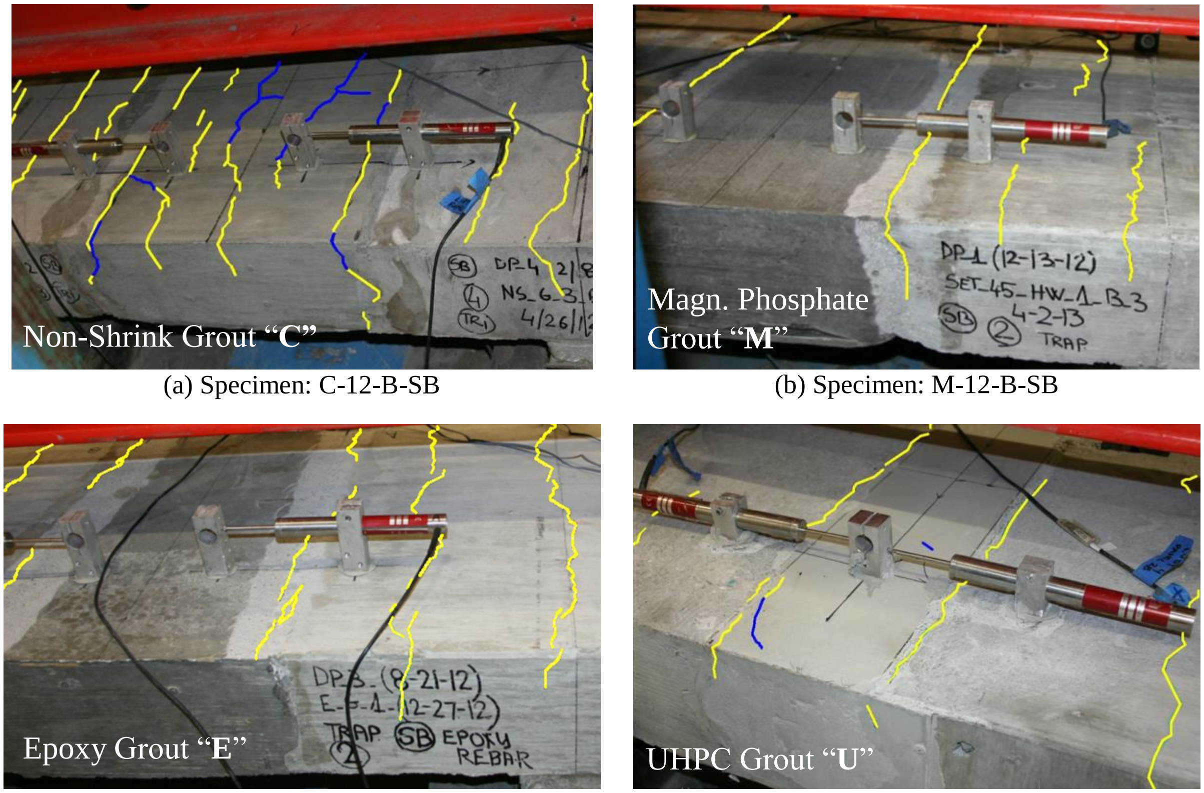


(a) Average 7-day tensile bond strength measured from small-scale beam tests (De la Varga et al., 2016)

(b) Interface cracking levels of select deck panel specimens as determined by strain measurements

Figure 9. Bond Behavior of Connection Grouts

Figure 10 depicts the apparent damage after completion of the cyclic crack loading protocol. A representative photo was selected for each grout type used. All specimens exhibited flexural cracks in the precast concrete deck panels. Specimens employing grout "C" exhibited interface cracks and significant cracking within the grout material. In many cases, cracking was apparent along the entire depth of the concrete-grout interface. Furthermore, it was apparent that pre-existing shrinkage cracking tended to propagate during loading. Specimens employing the remaining three grout types ("M", "E", and "U") sustained cracks along the concrete-grout interface, but had minimal if any visible cracks within the grouted connection region. Lastly, it can be observed in Figure 10d that even though specimen "U-5.5-B-SB", which employed UHPC grout, exhibited minor shrinkage cracking, these cracks did not grow substantially during cyclic crack loading.



(a) Specimen: C-12-B-SB (b) Specimen: M-12-B-SB

(c) Specimen: E-12-Ep-SB (d) Specimen: U-5.5-B-SB

Figure 10. Observed cracks after completion of cyclic crack loading: "Blue" lines correspond to cracks caused by shrinkage; "Yellow" lines correspond to cracks from mechanical loading

FATIGUE LOADING

The fatigue loading protocol was subsequently applied to each specimen after completion of the cyclic crack loading protocol. The primary objective of the fatigue loading protocol was to subject specimens to high stress amplitudes without causing fatigue-rupture of the internal reinforcing bars. The fatigue loading protocol is shown in Figure 11. Prior to initiating the true fatigue cycles, four ramp-up load steps were applied, and each step consisting of 5,000 cycles of increasing load. After the ramp-up cycles, the panels were subjected to 1,000 cycles of "overloading", which was imposed to increase the demand. During overloading cycles, specimens were subjected to repeated loading at 60% of the yield moment, M_y. After overloading, 99,000 load cycles were applied with a maximum target load that was 37% of M_y. The set of overloading and subsequent fatigue cycles was repeated 20 times or until failure occurred.

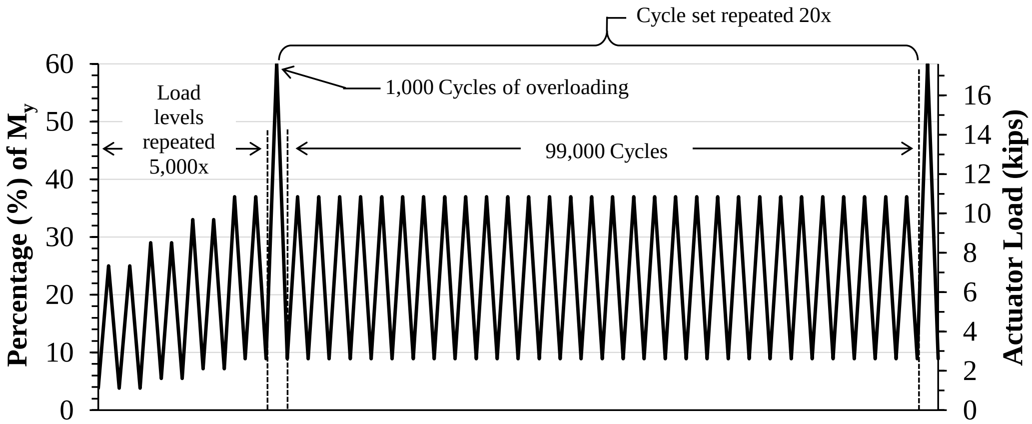


Figure 11. Fatigue loading protocol

The performance of the deck panels under fatigue loading was primarily controlled by the grout material and the reinforcement detailing within the connection. Forty six percent (14 out of 30) of specimens employing grout "C" and all of the specimens employing grout "M" failed during fatigue loading. All specimens with grouts "E" and "U" survived fatigue loading without signs of distress or significant cracking in the connection region. In most cases, specimens that failed during fatigue loading failed during overload cycles. This is not unexpected given that the demand on the specimens was nearly doubled during this portion of the loading protocol. The majority of specimens with grouts "C" and "M" failed during the first round of overloading as a result of reinforcing bar bond failure. A representative photo of this type of failure mode is shown in Figure 12.

Figure 13 depicts the fatigue performance of specimens with connection grouts "C" and "M". The x-axis depicts the number of overload cycles sustained by a given specimen in log-scale, and the y-axis depicts what is referred to herein as the "bond index". The expression for this index is shown within the plot area of Figure 13 where L_{em} is the measured lap splice length for a given specimen, and f'_g is the compressive strength the connection grout at the beginning of fatigue cycling in psi. In the equation for *BI*, the quantity in the denominator represents that maximum value of the product $L_{em}\sqrt{f'_g}$ for the set of results shown in Figure 13. This index is representative measure of the bond strength for a given connection; it is generally accepted that bond strength is a function of embedded length and square root of the compressive strength of concrete (ACI, 2012). It can be observed that specimens that exhibited shrinkage cracking, which is denoted by "-", typically did not survive the fatigue loading protocol. In some cases the addition of lacer bars, denoted by marker outlined in **bold**, tended to improve the fatigue life of specimens. Three specimens employed headed reinforcing bars. Despite exhibiting shrinkage cracks, these specimens all survived the fatigue loading protocol. It has been shown that lacer bars, which are known to provide transverse confinement to a lap splice (ACI, 2012), and headed bars, which do not require long lap lengths (Thompson et al., 2006), can improve the bond behavior of lap spliced bars.

As shown in Figure 13 a few specimens employing grout "C" did survived the fatigue loading procedure, which are denoted "runout". By the end of fatigue loading, these specimens exhibited significant cracking and distress in the connection grout. Figure 14 shows a set of photos taken at different points throughout the fatigue loading protocol for specimens with grouts “C”, “E”, and “U”. These photos are representative of other specimens, for a given grout type, that survived the fatigue loading procedure. The crack propagation and cumulative damage can be observed. As previously stated, specimens with grout “C” exhibited the most distress in the connection regions. These specimens exhibited wide cracks at the precast panel-to-grout interface prior to yielding of tensile reinforcement, which indicates initiation of bond failure between the grout and the embedded reinforcing bars. Longitudinal splitting cracks were also observed, which are also a typical precursor to bond failure. Although specimens employing grouts “E” and “U” exhibited fully-cracked interfaces, which is to be expected, little to no apparent damage was observed within the connection grout.

Figure 12. Example of fatigue failure in specimens with grouts “C” and “M”

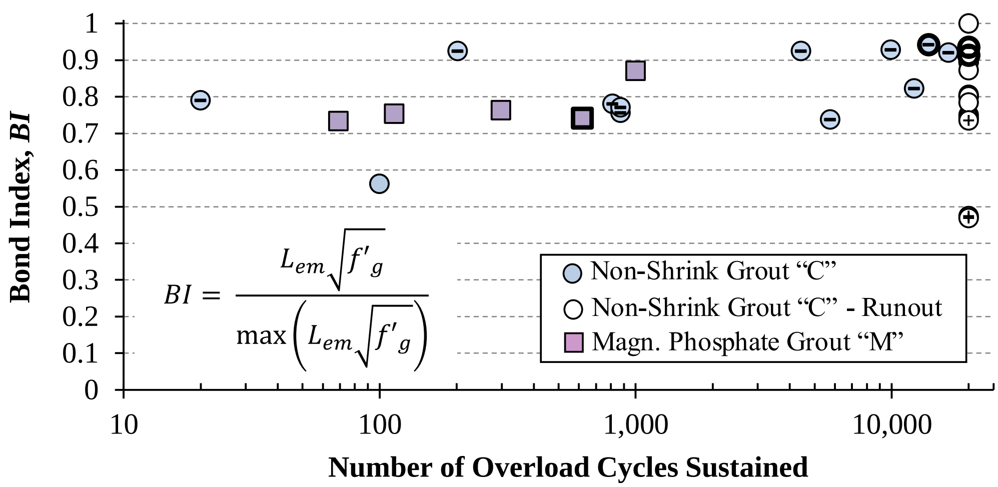


Figure 13. Fatigue performance of specimens with connection grouts “C” and “M”: “-” indicates specimens that exhibited shrinkage cracking similar or worse than that shown in Figure 5; “+” indicate specimens that employed headed bars and had shrinkage cracking; and marker outlined in “**bold**” indicate specimens that employed lacer bars in the connection.

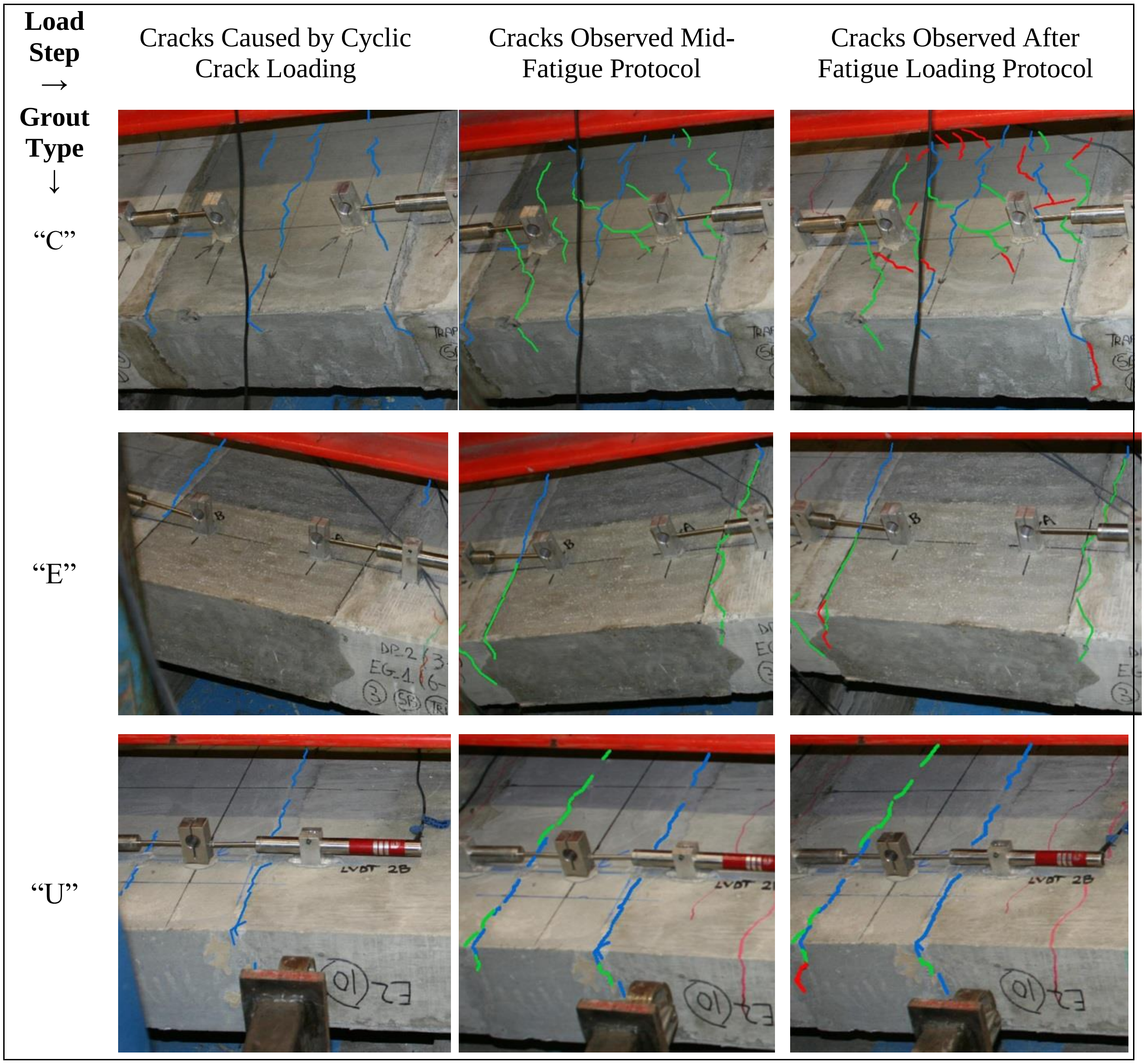


Figure 14. Progression of damage during fatigue loading

ULTIMATE LOADING

Although the focus of this paper is bond and cracking behavior, it is important and beneficial to note some of the general observations from ultimate loading of deck panel specimens. Specimens that survived the fatigue loading protocol where subsequently subjected to monotonic loading until failure. Similar to observations from fatigue loading, the design parameters that primarily controlled the ultimate behavior of specimens were the grout material and the reinforcement detailing within the connection. The ultimate loading behavior of the precast panels was compared to panels cast monolithically with continuous bars. Specimens employing grouts “E” and “U” exhibited force-displacement behavior similar to the monolithic panel with continuous bars. In all cases, these specimens failed by concrete crushing and also exhibited good flexural ductility. Specimens employing grout “C” did not demonstrate flexural ductility, and in most cases failed prior to yielding of steel. The primary failure mode for these specimens was bond failure between the embedded reinforcing bars and the connection grout similar to that shown in Figure 12. The flexural behavior of specimens with grout "C" was slightly improved when lacer bars or headed reinforcement were employed. Further information on flexural behavior of specimens tested in Phase I can be found in (Haber and Graybeal, 2015).

RESEARCH PLAN FOR PHASE II STUDIES

The second phase of this component-level study has two main foci. The first is to further investigate, at the component level, methods to improve the dimensional stability and mechanical properties of pre-bagged non-shrink cementitious grouts for PBE connections. In the first part of this two-part paper a series of tests were conducted that focused on methods to improve the bond behavior of pre-bagged non-shrink cementitious grout to precast concrete (De la Varga, Haber, and Graybeal, 2016). Furthermore, it was noted that some of these methods can also improve the dimensional stability of some grout systems. Lastly, it was hypothesized that there is a correlation between bond strength and dimensional stability. Research results from the first phase of this study indicated that poor dimensional stability, i.e. excessive shrinkage, may result in increased connection cracking (Figure 10), and may reduce the bond strength of reinforcement embedded in the connection grout material, and lead to premature failure under repeated loading (Figure 13). Given that non-shrink cementitious grouts are an economical option, and many bridge construction contractors and bridge designers are familiar with this class of materials, further investigation of improving their performance in PBE connections is warranted.

As many as six new deck-level connections tests will be performed. Some of the grout modification techniques that will be tested may include addition of internal curing (IC) by light-weight aggregate (LWA) and addition of fiber-reinforcement. Deck panel specimens will be similar to those tested in this study in terms of geometry. However, the reinforcement details within the connection region will be slightly different. Instead of using straight lap spliced bars, a U-bar detail will be used which is similar to that tested by Zhu et al. (2012) (Figure 1b). Lastly, all specimens will employ the exposed aggregate (EA) surface preparation given that it exhibited the best results in both material- and component-level tests. This finish is achieved using an in-form, paint-on retarder. After concrete has cured for 24 hours, the retarded surface is pressure washed to expose the underlying aggregate. Figure 15 shows some of the detailing within the connection region of a phase II test specimen. The primary goal of this research is to provide recommendations and design guidelines on how to improve PBE connections using conventional grout materials.

The second focus of the phase II investigation will be to evaluate a series of different UHPC-class materials for use in deck-level PBE connections. As UHPC becomes more popular for PBE connections, the commercial market landscape for these materials may change in the US. That is, there may be new UHPC products available in the US market. Currently, work is being conducted at the FHWA Turner-Fairbank Highway Research Center to evaluate the reinforcing bar bond strength in commercial-available UHPC-class materials from the US and Europe (Yuan and Graybeal, 2016). As many as five different UHPC-class materials will be tested in deck-level connections similar to those discussed previously in this paper. Furthermore, specimens will be subjected to the same loading protocols such that the current data set for this type of connection can be substantially expanded. Lastly, a series of companion tests will be conducted to evaluate tensile bond strength of the new UHPC materials to precast concrete; companion tests will be similar to those discussed in Part I paper by De la Varga, et al. (2016). One of the primary goals of this work is to develop a more generalized set of design guidelines for UHPC connections.

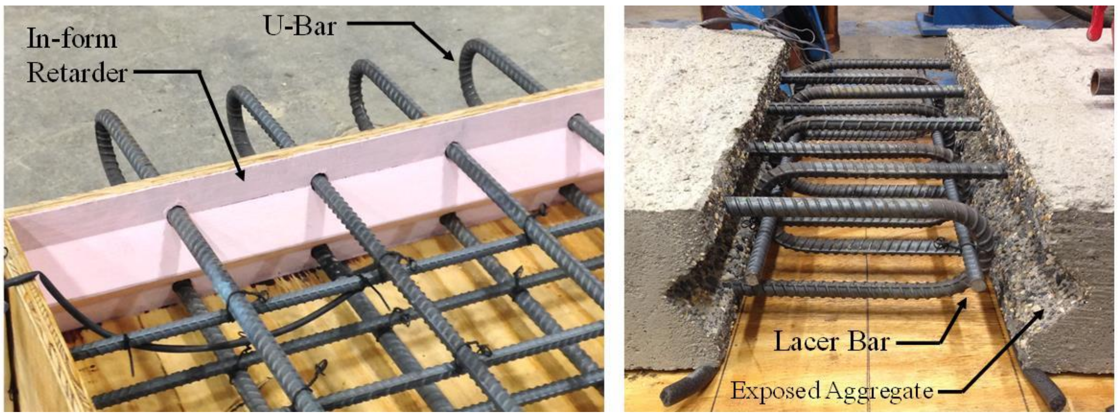


Figure 15. Deck panel specimen details for connection to be filled with cementitious grouts

CONCLUSIONS

The research presented in this paper was the second part of a larger study on connection grout materials for prefabricated bridge elements (PBE). In the component-level investigation a number of different parameters frequently considered during the design of these connections were investigated under different loading protocol. Seventy two large-scale deck-level connection assemblies were subjected to cyclic crack loading, fatigue loading, and monotonic loading until failure. Load levels and protocols were selected such that different parameter combinations could be evaluated under realistic performance demands. This paper presented a small portion of the results from this study. The following conclusions can be made based on the findings from experimental testing:

1. Test variables had varying influence on the behavior of deck-level connections. The precast concrete surface preparation and lap splice length affected only certain aspects of deck-level connection performance such as cracking behavior and fatigue/ultimate behavior, respectively. The type of grout material used had significant impact on all aspects of deck-level connection performance. Thus, the selection of field-cast grout materials is one of the most critical design considerations for deck-level PBES connections.

2. Some pre-bagged cementitious grouts are susceptible to premature failure under repeated loading as a result of formation and propagation of cracks. Without confinement and/or fiber reinforcement, cracks cannot be arrested and will continue to grow with the number of load cycles. Both micro- and macro-scale cracks weaken the bond between the grout and load carrying reinforcing bars which may lead to premature performance degradation. Lastly it was shown that dimensional stability of the connection grout may also influence the performance of a connection under mechanical loading.

3. Although epoxy “E” and UHPC “U” grout systems have higher initial cost, they may provide better value when constructability, long-term performance, and required maintenance are considered.

4. Depending on the grout material selected for a deck-level connection, surface preparation of precast concrete deck panels can have a significant impact on tensile bond resistance of the concrete-grout interface. The exposed aggregate surface preparation promotes good bonding conditions between precast concrete components and cementitious grout connection materials.

In summary, careful thought must be given to the selection of field-cast grout materials for connecting PBE elements. Similarly, design details can play an important role in the behavior of connections. In order to maximize performance and minimize potential issues related to durability, exposed aggregate surface preparation should be provided on precast concrete deck panels surfaces that will be in contact with a cementitious field-cast grout within a connection region. Without compromising development of bars and connection strength, deck-level connections should be designed and detailed to minimize the distance between adjacent elements such that effects of connection grout shrinkage / expansion can be minimized. Reinforcement detailing and lap splice lengths need to be considered in conjunction with the strength and materials characteristics of the intended field-cast grout systems to be used for connections.

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REFERENCES

1. American Concrete Institute, "Bond and Development of Straight Reinforcing Bars in Tension," *Report by ACI Committee 408,* 2012, 49 pp.

2. Badie, S. S., and Tadros, M. K., "Full-Depth Precast Concrete Bridge Deck Panel Systems," *NHCRP Report 584*, 2008, 110 pp.

3. De la Varga, I., Haber, Z. B., and Graybeal, B. A., "Performance of Grouted Connections for Prefabricated Bridge Elements – Part I: Material-Level Investigation on Shrinkage and Bond," *Proceedings of the 2016 PCI National Bridge Conference*, March 2016, 12pp.

4. De la Varga, I. and Graybeal, B. A., "Dimensional Stability of Grout-Type Materials Used as Connections Between Prefabricated Concrete Elements," *Journal of Materials in Civil Engineering*, DOI: 10.1061/(ASCE)MT.1943-5533.0001212. 2014.

5. Graybeal, B. A., "Behavior of Field-Cast Ultra-High Performance Concrete Bridge Deck Connections Under Cyclic and Static Structural Loading," *Report No. FHWA-HRT-11-023*, 2010, 106 pp.

6. Haber, Z. B. and Graybeal, B. A., "Experimental Evaluation of Prefabricated Deck Panel Connections," *Proceeding of the TRB 2015 Annual Meeting*, January 2015, 10 pp.

7. Li, L., Ma, Z. J, Griffey, M. E., and Oesterle, R. G., "Improved Longitudinal Joint Details in Decked Bulb Tees for Accelerated Bridge Concrete: Concept Development," *ASCE Journal of Bridge Engineering*. Vol. 15, No. 3, 2010, pp. 327-336.

8. Swenty, M. K, and B. A. Graybeal, "Material Characterization of Field-Cast Connection Grouts," Report No. PB2013-130231, National Technical Information Service, Springfield, VA., 2013, XX pp.

9. Thompson, M. K., Ledesma, A., Jirsa, J. O., and Breen, J. E., "Lap Splices Anchoraged by Headed Bars," *ACI Structural Journal*¸ Vol. 103, No. 2, March-April 2006, pp.271-279.

10. Yuan. J. and Graybeal, B. A., "UHPC Connections for Prefabricated Bridge Elements: Embedment and Splice of Reinforcement," *Proceedings of the 2016 PCI National Bridge Conference,* March 2016, 21 pp.

11. Zhu, P., Ma, Z. J., French, C. E., "Fatigue Evaluation of Longitudinal U-Bar Joint Details for Accelerated Bridge Construction," *ASCE Journal of Bridge Engineering,* Vol. 17, No. 2, 2012, pp. 201-210.