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FIELD TEST REPORT

Tensile Bond Strength of a High Performance Concrete Bridge Deck Overlay

I-90, STURGIS, SOUTH DAKOTA
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EXECUTIVE SUMMARY

This paper presents results from an evaluation of a 2-year old fiber reinforced high performance concrete (HPC) overlay bonded to a badly deteriorated concrete bridge deck. The subject evaluation was focused on determining how well the overlay concrete was bonding to the underlying deck. To ensure long service of the rehabilitated deck, it is imperative that the overlay is well bonded to the underlying concrete. The evaluation consisted of employing a field tensile bond test (pull-off test) at 13 locations along the bridge decks and approaches, as well as subsequent laboratory tensile tests on seven companion cores for comparison testing. Results indicate that the non-metallic fiber reinforced HPC overlay is bonded sufficiently to the underlying concrete. However, all tensile failures occurred in the substrate material within 8mm of the bond interface, indicating that the existing bridge-deck concrete is the weakest portion of the system. It is suggested that the low tensile strength in the top portions of the bridge-deck concrete may be a result of existing delaminations or damage from milling and partial depth concrete removal during rehabilitation.

INTRODUCTION

Project Background

Due to tightening budget constraints and reprogramming of construction funds, two badly deteriorated bridges at Exit 32 on I-90 in South Dakota could not be reconstructed during 1997, and will probably not be reconstructed for another five to seven years. The bridges were constructed in 1963, and have been in continuous service since. Both bridges consist of three steel girder spans, roughly 19 m (60 feet) each. The concrete deck is approximately 165mm (6.5 in) thick with 38 mm (1.5 in) clear cover over black reinforcing steel. The bridges are located in an area of severe temperature swings, and experiences heavy de-icer use. Due to the extensive deterioration of the bridges, some form of rehabilitation was clearly necessary. Two alternate types of rehabilitation were considered, including deck replacements and deck overlays. In light of the fact that the bridges are to be totally reconstructed in less than 10 years, a deck overlay was selected as the most economical solution. In an effort to enhance the performance of their concrete overlay system in light of the poor condition of the decks, South Dakota Department of Transportation (SDDOT) decided to employ the use of a low slump dense non-metallic fiber-reinforced HPC.[1]

The bridges were rehabilitated during May and June of 1997. Pre-construction surveys conducted after the decks had been milled and prepared, revealed extensive map-cracking on the bottom surface of the deck, and delaminations across roughly 90% of the milled deck surfaces. There was also evidence of several partial and full-depth repair patches. All reinforcing steel exposed during milling was sand blasted to remove corrosion. No bonding agent was used for this project. Placement of the high performance concrete overlays (NMFRC) were successful, and periodic inspections of the bridge decks since rehabilitation have revealed no significant cracking, spalling, delamination or other deterioration. [1]

In a continued effort to evaluate the performance of the HPC bridge overlay system, SDSM&T contacted the Federal Highway Administration (FHWA) in spring of 1999 to aid in the assessment of the overlays' bond to the existing bridge decks. The core issue in question was: How well is the high performance concrete overlay bonding to the damaged concrete bridge deck? For the overlay rehabilitation to perform properly it is critical that the bond between the two materials is developed sufficiently and remains intact throughout its service life. This bond is a function of surface preparation and the physical and chemical characteristics of the repair material and the substrate concrete. In July of 1999, FHWA deployed the Mobile Concrete Laboratory to conduct a series of in-place tensile bond tests on the bridge decks in an attempt to get a relative measure of how well the two-year old non-metallic fiber reinforced HPC overlay is bonded to the existing bridge decks.

Bond Testing

As a part of an effort to demonstrate state of the art concrete technology in both laboratory and field testing through the use of innovative and nondestructive testing techniques, the Federal Highway Administration (FHWA) employed the use of an in-place direct tensile test for determining tensile bond strengths. A detailed description of the test method is presented in the next section. The in-place direct tensile test (pull-off test) was chosen over laboratory tests for several reasons: 1) The in-place tensile test is relatively simple to perform and gives immediate results in the field, 2) The in-place test does not require careful specimen handling during transport to a laboratory, and is thus less susceptible to specimen handling and storage issues, 3) Retrieving laboratory specimens in the field can sometimes prove difficult. As the laboratory specimens (cores) must include the bond zone along the length of the core, and sometimes during coring the core "breaks" at the bond instead of some distance below, the specimens are often rendered useless for bond testing. This in turn results in frequent re-coring and increased on-site time.

A number of different in-place direct tensile tests have been proposed in the last 20 years. A brief review of the most common tensile bond tests as well as an evaluation of three particular types of in-place direct tensile testing equipment was performed by Vaysburd and Mc Donald in 1999. [2] In their report, they recommend the tensile pull-off test as the best available method for monitoring bond strengths in the field. One of the devices evaluated in their study includes the device selected for use for this project (Proceq DYNA Z15).

Although tensile pull-off tests are becoming increasingly popular for both forensic studies and on-site QC/QA testing, little standardization has yet occurred. The American Society for Testing and Materials (ASTM) has not yet adopted a test method for in-situ pull-off testing. The American Concrete Institute (ACI) however, has presented a test method suitable for field evaluation of the tensile bond strength of patched or overlaid concrete in ACI 503R-93.[3] There have been some European efforts to standardize an in-place direct tensile test. The British have developed BS 1881: Part 207 (1992) that provides guidelines for the standardization of in-place direct tensile tests. The Dutch have developed a Standard that deals specifically with the pull-off test. CEN TC 104 is in the process of drafting a European Standard.[2]

All these tests methods and guidelines are essentially the same in that they involve applying a direct tensile load to a partial core advanced through the overlay material and into the underlying concrete, until failure occurs. What is apparent from the studies that have been performed to date is that there is a definite need for standardization of the pull-off test as well as a need for subsequent pull-off tensile strength data and overlay performance data. Without this data, no meaningful interpretation of absolute tensile failure strengths can be made.

THE PULL-OFF TEST

As mentioned in the previous paragraph, the pull-off test involves applying a direct tensile load to a partial core advanced through the overlay material and into the underlying concrete until failure occurs. The tensile load is applied to the partial core through the use of a metal disk with a pull pin, bonded to the overlay with an epoxy. A loading device with a reaction frame applies the load to the pull pin. The load is applied at a constant rate, and the ultimate load is recorded. Figure 1 illustrates the principle of the pull-off test.

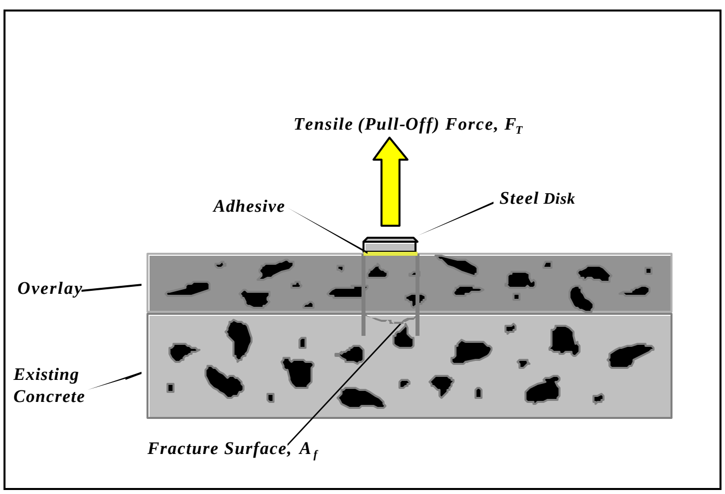

Figure 1. Schematic of Pull-Off Test Principle

The pull-off strength (S_{PO}) is defined as the tensile (pull-off) force (F_T) divided by the area of the fracture surface (A_f):

$$S_{PO} = F_T / A_f \quad (1)$$

There are essentially four different modes of failure when applying load in this manner. These different failure modes provide valuable information about the overlay system. The magnitude and location of the fracture surface determines what conclusions may be drawn from the test. First, if the failure occurs at the bond surface, the pull-off strength is in fact the tensile bond strength. In this case, the ultimate load is a direct measure of the adhesion between the overlay and the substrate concrete. Second, when the failure occurs between the disk and the overlay surface, there is an adhesive failure. In this case, the tensile strength of the overlay system is greater than the failure load, and a stronger adhesive is needed. Third, if the failure occurs in the overlay material, the repair material (overlay) is the weakest portion of the system, and we know the bond strength exceeds the ultimate stress applied. This is also referred to in the literature as cohesive failure of the overlay. [2,3,4] Finally, if the failure occurs in the substrate, or underlying concrete, the overlay (repair) concrete and the bond are stronger than the existing concrete, and the repair can be considered successful. This is again often referred to as cohesive failure of the substrate. The illustration in Figure 1 is an example of such a failure mode. In this case, the failure stress is the tensile stress of the substrate concrete. When this occurs, the failed specimens can be taken to the laboratory for further testing (direct shear, laboratory direct tension, etc.) if the bond strength value is desired. In some cases the failure occurs partially along the bond surface and partially in either the overlay or substrate concrete, and the failure mode is a combination of two or more of the failures discussed above.

The general procedure for performing a pull-off test can be summarized as follows:

1. Abraid the surface of the concrete in the test-area with a carbide stone or wire brush to remove any laitance and deposits. This aids in achieving sufficient bond between the steel disk and the overlay surface.
2. Advance a partial core (typically 50mm diameter) through the overlay, and a minimum of 25mm (or ½ core diameter) into the substrate concrete. Care should be taken to ensure that the core is advanced perpendicular to the overlay surface to minimize eccentricities during loading.
3. After the top of the partial core has been cleaned and dried (pressurized air is helpful), bond a metal disk (typically 50mm diameter) to the surface of the partial core with a fast-setting epoxy. Avoid applying too much epoxy, as excess will run down the sides of the core and possibly bond the core to the sides of the core-hole. Again, care should be taken to ensure that the disk is bonded to the middle of the partial core to minimize the potential for loading eccentricities.
4. After the epoxy has cured properly, attach the loading device to the metal disk. The loading device with its reaction frame should be adjusted to ensure that the load is applied parallel to the axis of the core. Some reaction frames have adjustable legs for this purpose.
5. Apply the tensile load to the core at approximately 0.1kN per second until the specimen fails. Record the failure load, as well as the failure mode and fracture location.

Figure 2 shows the tensile bond test device used for this evaluation (Proceq DYNA Z15).

Figure 2. Commercially Available Tensile Bond Strength Test Device

RESULTS

Pull-Off Test Results

Tables 1 and 2 show the pull-off test results from the eastbound and westbound passing lanes respectively. From this data it is apparent that the tensile strengths are relatively low. One would typically expect the tensile bond strength of a repair material to be at least 1.0 MPa. In all cases, the failures occurred in the substrate (underlying bridge deck) concrete (the fourth failure mode in the preceding section). Furthermore, all failures (with the exception of core E6) occurred very near the bond surface (within 8 mm), indicating that the top portion of the underlying bridge deck concrete is the weakest portion of the system. In this situation, the repair overlay can be considered successful, as the strength of the bond and the overlay are greater than the strength of the underlying bridge deck concrete. Core E6 failed directly above a steel reinforcing bar located approximately 27 mm below the bond surface. Upon examination of the core, a large void was evident immediately above the reinforcing bar, indicating poorly consolidated concrete. This void significantly reduced the cross-section of the core and was the probable reason the failure occurred at that depth. Consequently, core E6 has been excluded from subsequent data analyses.

Table 1. Pull-Off Results from Eastbound Passing Lane

Core #	Overlay Depth (mm)	Tensile Stress (kPa)	Location of Fracture/Comments
E1	70	1089	3 mm below interface in substrate concrete
E2	59	524	6 mm below interface in substrate concrete
E3	49	683	5 mm below interface in substrate concrete
E4	57	483	6 mm below interface in substrate concrete
E5	67	283	3 mm below interface in substrate concrete
E6	75	290	25 mm below interface at void above steel bar

Notes: 1. Depth of overlay is the average of three readings
2. Cores E1 and E6 are from the east and west bridge approaches respectively

Table 2. Pull-Off Results from Westbound Passing Lane

Core #	Overlay Depth (mm)	Tensile Stress (kPa)	Location of Fracture/Comments
W1	70	607	5 mm below interface in substrate concrete
W2	73	393	3 mm below interface in substrate concrete
W3	71	407	8 mm below interface in substrate concrete
W4	51	510	5 mm below interface in substrate concrete
W6	80	814	3 mm below interface in substrate concrete
W7	56	910	3 mm below interface in substrate concrete
W8	160	814	5 mm below interface in substrate concrete

Notes: 1. Depth of overlay is the average of three readings
2. Cores W7 and W8 are from the east and west bridge approaches respectively

Further study of the data indicates that the tensile strength of the concrete on the approaches to the bridges is approximately 80% greater than that for the bridge deck concrete. The average failure tensile stress for cores taken on the bridge deck is 523 kPa, while for the approaches it is 938 kPa (excluding core E6).

Visual examination of the fracture surface of the cores indicates that the failures are a combination of aggregate-paste bond failure, coarse aggregate failure and paste failure. Approximately 65% of the fractures appear to be due to failure in the bond between the aggregate and the paste fraction. The remaining 25% are due to a combination of coarse aggregate fractures and paste failures. Also evident from visual examination of the cores is a significant amount of entrapped air voids. This may be an indication that the low slump fiber reinforced HPC was not properly consolidated. Figure 3 gives a fracture surface view of pull-off cores E2 and E3.

Figure 3. View of Fracture Surface of Tensile Cores

Laboratory Tensile Test Results

South Dakota School of Mines and Technology (SDSM&T) also retrieved conventional 100 mm diameter companion cores from the bridge decks at the time of field testing. These cores were retrieved for subsequent laboratory tensile testing to correlate with the field pull-off data. The laboratory tensile cores were advanced in the same general location as the companion field pull-off test. The laboratory tensile test performed by SDS&MT is very similar to the field pull-off test. The tensile load is applied to the core through the use of metal disks with pull-pins bonded to the top (overlay side) and bottom (substrate side) of the core. Each core is sawed flat on the bottom prior to adhering the metal disk to it. In this case a Tinius Olsen load frame was used to apply the load to the pull pins until failure. Details of the laboratory tensile test are described in SDSM&T's report to SDDOT. [5] Table 3 includes the data from the laboratory tensile testing, and shows a comparison of tensile strengths from the laboratory test versus the pull-off test.

Table 3. Comparison of Field Pull-Off and Laboratory Tensile Testing

Location	Westbound Passing Lane				Eastbound Passing Lane			
	Lab Tensile Test		Field Pull-Off Test		Lab Tensile Test		Field Pull-Off Test	
	Core #	T. Strength (kPa)	Core #	T. Strength (kPa)	Core #	T. Strength (kPa)	Core #	T. Strength (kPa)
Bridge Deck			W1	607			E2	524
			W2	393	EP3	303	E3	683
			W3	407	EP4	793	E4	483
	WP3	296	W4	510			E5	283
	WP5	600	W6	814				
	Avg.	**448**	Avg.	**546**	Avg.	**548**	Avg.	**493**
Approach			W7	910	EP1	752	E1	1089
	WP6	910	W8	814	EP6	683		
		910	Avg.	**862**	Avg.	**718**		**1089**

As with the pull-off test, laboratory tensile tests of cores from the bridge approaches exhibit greater average tensile strengths than cores from the bridge deck. In all cases, the failures occurred in the substrate concrete just below the bond surface. Visual examination of the fracture surface of the laboratory cores indicates that the cores fractured in a similar manner to the field pull-off cores. Failures were a combination of aggregate-paste bond failure, coarse aggregate failure and paste failure, with approximately 50% of the fractures due to failure in the bond between the aggregate and the paste fraction.

From a practical standpoint, the average tensile strengths measured with the two methods are not substantially different. Although the difference appears large relative to the magnitude of the strength, the overall tensile strengths are so low that the results are much more sensitive to variations in such things as test alignment (load eccentricities) and load rate. In an effort to quantify the relative difference in consistency of the two test methods, analysis of the coefficient of variance (COV) was employed. COV is more appropriate for this purpose than standard deviation due to the significant variation in averages. The coefficient of variance (COV) for all field pull-off tests conducted on the bridge decks is 31%. The COV for all laboratory tensile tests conducted on the bridge deck cores is 49%. These COV's are relatively high and suggest that the test data is quite variable. Although, it should be noted that the magnitude if the COV is not as important in this particular case as the relative difference in COV between the two tests. The magnitude of the COV is not only a function of the precision of the test method, but also of the variability of the tensile strength in the decks. Consequently, if the test methods are reasonably similar in precision, their COV should be similar as well. In this case, considering the magnitude of the COV, their relative difference is acceptable, and the two test method's results may be considered comparable.

Summary of Results

The measured tensile strengths are variable to very variable, and are lower than expected. All failures occurred in the substrate material, close to the bond interface, suggesting that the bond and the repair material are stronger than the underlying bridge deck concrete. From visual examinations of the fracture surfaces, the fractures were primarily in the interface between the coarse aggregate and the paste fraction. There was also evidence of fractures through aggregate particles and cracks through the paste. The presence of significant entrapped air suggests that the HPC overlay may not have been properly consolidated. The tensile strengths are on average significantly greater on the approaches to the bridge than on the bridge decks themselves. There are no significant differences in the results from the field pull-off test and the laboratory tensile test.

CONCLUSIONS

Based on results gathered during this evaluation, the following conclusions may be drawn:

1. The low slump dense non-metallic fiber-reinforced HPC overlay is bonding well to the substrate concrete, and the overlay's tensile strength exceeds that of the substrate concrete. Therefore, the bridge-deck rehabilitation overlay can be considered successful. Some evidence of excessive entrapped voids was evident in all the cores retrieved, suggesting that increased attention should be focused on consolidating the low slump dense HPC mixture.
2. The average failure tensile stresses are lower than expected, but no meaningful interpretation of these absolute values can be made without additional correlating performance data. This clearly points to the need for standardization of the field pull-off test, so that a particular tensile value may be associated with an expected level of performance.
3. The low tensile strengths in the top portions of the substrate material are most likely a result of a combination of 1) existing delaminations in the bridge deck prior to rehabilitation and 2) damage from milling and partial depth concrete removal during rehabilitation. The pull-off tensile test can be useful in assessing the most effective (least damaging) surface preparation technique for bridge deck overlays
4. The pull-off tensile test results are quite variable. This indicates either a high test variability or a high variability in tensile strengths within the bridge decks. Most likely, it is a combination of both. Other research has found that although the pull-off tensile test is the best available test method for evaluating tensile strengths in the field, the results of the test do not necessarily indicate precise tensile bond values. [2] The test does however provide a good relative measure of

in-situ tensile strength. There are no significant differences in the results from the pull-off tensile test and the laboratory tensile test

5. The tensile strengths are on average significantly greater on the approaches to the bridge than on the bridge decks themselves. This is consistent with the conclusion that the low tensile strengths in the top portions of the substrate material are a result of a combination of existing delaminations in the bridge deck prior to rehabilitation and damage from surface preparation during rehabilitation. A bridge deck is more susceptible to damage incurred as a result of deflections and impacts during dynamic loading from milling as well as normal service, than a fully supported approach slab.
6. The pull-off tensile test can also be useful for estimating expected service life of bridge deck overlays, by measuring degradation of tensile strength with time.

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