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Bridge Deck Cracking Investigation and Repair

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BRIDGE DECK CRACKING INVESTIGATION AND REPAIR

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Dedication

I would like to thank all of those that helped me along the way with the research process and the completion of this thesis. First of all, thanks to my professors for the guidance, understanding, encouragement and wisdom imparted on me. I also extend my appreciation to Mr. Ivan Lasa and Mr. Mario Paredes from the Florida Department of Transportation for their cooperation. Thanks to all the undergraduate students that helped in the lab. To my fellow graduate students thank you for your support, patience, and assistance.

Finally, but not least, to my wife Nancy V. Vargas, thank you for pushing me to pursue my Master's when I was down-and-out with no prospects of a career. Thanks for all the long hours spent waiting for my last test in the lab and thanks for being my best friend when I needed it the most.

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ABSTRACT

The focus of this study is to investigate the cracking of concrete bridge decks and the sealants used in repairing transverse cracks. Cracking could occur in both hardened mature concrete and early age concrete. Several factors affect concrete cracking, such as age-dependent material properties, thermal- and moisture-related stresses and strains, material viscoelastic behavior, restraints, concrete expansion and contraction, casting sequence, formwork, material characteristics, and environmental exposure. The causes of early age cracking are primarily attributed to effects such as plastic shrinkage, temperature effects, autogenous shrinkage, and drying shrinkage. This deck cracking could greatly reduce durability, lead to a loss of functionality, loss of stiffness, and ultimately the loss of structural safety.

The study investigates the deck cracking in general and also the transverse cracks developed in hardened concrete at early ages before service loads application. Both experimental and analytical investigations were performed. The experimental study included testing of 9 reinforced concrete slab specimens (18”x 48”x 5.5”). Cracks were induced in the slabs with different crack widths and lengths, sealed with 4 different materials of sealants, and tested under static loading. The study also included tensile testing of dry hardened samples of sealants. In addition, field application was performed

on a bridge, where transverse deck cracks were sealed using 4 different sealant materials; cores were taken and tested according to ASTM-C496. The results of the testing showed that the 3-part HMWM was the best performing sealer for cracks between 0.01 and 0.019 inches of width with the epoxy sealer performing the best for cracks wider than 0.02 inches.

1. INTRODUCTION

1.1 Research Focus

The focus of this investigation is to study one of the biggest problems affecting bridge and transportation engineering community which is the deterioration of concrete bridge decks. The causes of early age cracking are varied but are primarily attributed to effects such as plastic shrinkage, temperature effects, autogenous shrinkage, and drying shrinkage. The cracking of bridge decks not only creates unsightly aesthetic condition but also greatly reduces durability, leads to a loss of functionality, loss of stiffness, and ultimately the loss of structural safety, resulting in aesthetic conditions that require the premature need for rehabilitation or replacement[1-6].

The basic problem of bridge deck cracking lies in the heating, hydrating, and expanding of young concrete next to older concrete and or fixed members that are cooling and shrinking at different rates which results in cracks in the young concrete. The cracks can be influenced by: material characteristics, casting sequence, formwork, climate conditions, and geometry; all of which are time dependent[7].

Due to the continued presence of bridge deck cracking in new structures there was a need to study the problem of early bridge deck cracking. This paper represents the

investigation of the deck cracking problem, concentrating on the literature review of early age transverse cracking of bridge decks focusing on the influencing factors, and the efforts to mitigate transverse deck cracking. The objectives of this paper are to evaluate the use of crack sealers to repair bridge deck cracking, and prepare a finite element model that can show the mechanisms that contribute to bridge deck cracking. The investigation also evaluates nine (9) slab samples with pre-formed cracks to study the properties of several crack sealers and give the transportation agencies a list of sealers with the properties required for the specific deck cracking problem. The study also included field investigation of several bridges.

1.2 Thesis Organization

The introductory chapter of this report presents the focus of this research. It includes a brief statement on the background of the research and its objectives, gives an outline of the report's organization and describes the bridges investigated and the mechanisms of bridge deck cracking.

Chapter 2, *Research Review,* documents the investigation into previous research of bridge deck cracking.

Chapter 3, *Crack Sealers,* presents an overview of the properties of the crack sealers used during this investigation.

Chapter 4, *Experimental Testing and Field Investigation,* presents the methodology of the experiments performed for the sealers both in the field and in the laboratory.

Chapter 5, *Recommendations and Conclusions,* this chapter presents the recommendations and conclusions reached from the experimental and analytical investigations performed during this study.

1.3 Bridges Investigated

During this research an assessment of several bridges was made for transverse, longitudinal, and map cracking those were:

- Fort Lauderdale bridges, 860524, 860526, 860527.
- Jacksonville bridges, 720701, 720702, 720704, 720705, 720706, 720707, 780121, 780122.
- Pensacola Bridge 580167.

Below are some of the bridges with their crack pattern and a summary of their condition.

Figure 1-1 Bridge 860524 Deck Cracking

Signs of steel corrosion

Corrosion of stay in place forms inside the steel box

Inspecting the inside of the steel box

Figure 1-2 Corrosion Inside Box Beams Bridge 860524

Bridge # 860524

Fort Lauderdale, Fl

As for the 860524 bridge in Fort Lauderdale a total of 19 cracks were found on the top-side of the deck as well as 44 was found on the exterior bottom side of the deck. The average crack widths for the cracks documented on this bridge ranged between 0.254mm and .5mm (.01 and .0197 in). The lengths of the aforementioned transverse cracks ranged from 3 feet to 36 feet and getting smaller in sections close to the parapet walls.

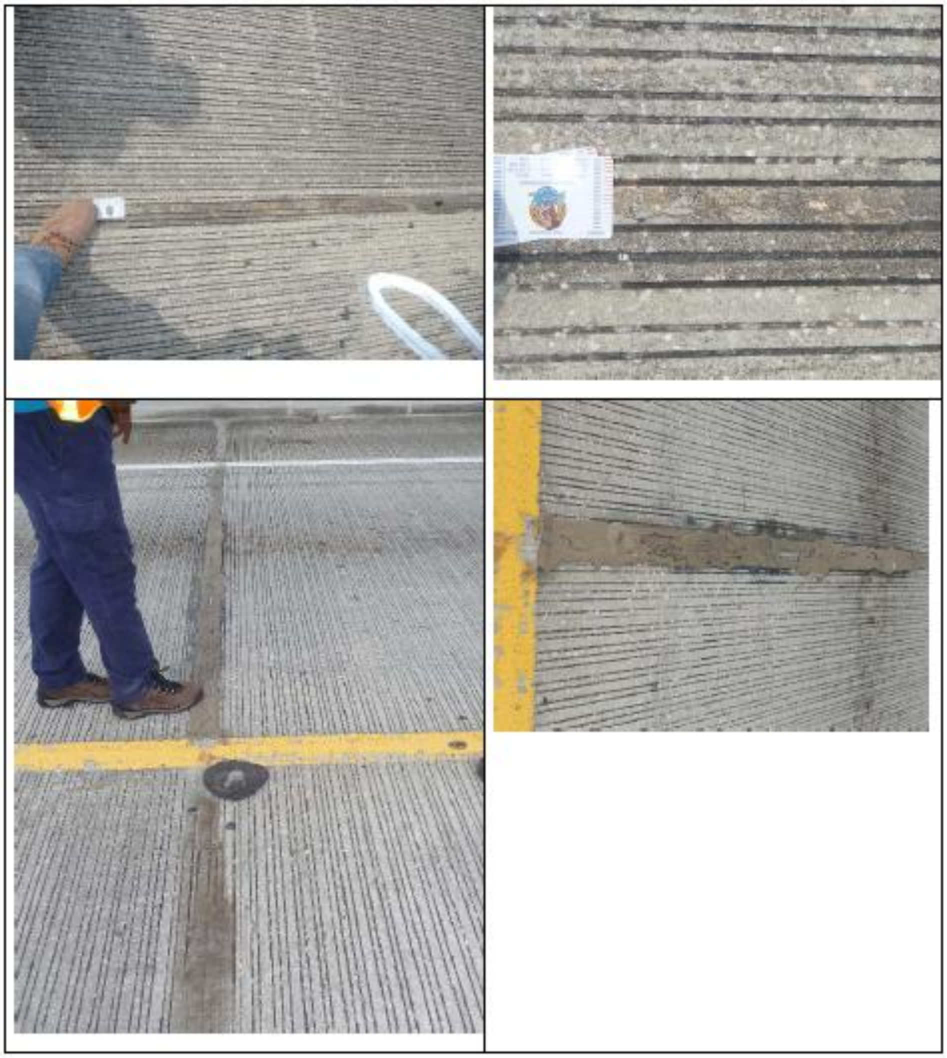


Figure 1-3 Bridge Deck Cracking on Bridge 860526

Bridge # 860526

Fort Lauderdale, Fl

Similar crack patterns to bridge 524 were found to exist on the 526 bridge. Cracks evident on the deck of this bridge had already been repaired therefore some of the documentation was impaired by the presence of epoxy and methacrylate sealants. The average visible crack widths still able to be measured were .304 mm (.012 in).

Bridge # 720701

SR 202 / SR 9A, JTB Interchange - Jacksonville, Fl

At this point only a preliminary analysis of the deck cracking for bridge 701 has been assessed. A total of 134 cracks were found visible on the top side of the deck across the length of the section. The average crack widths were in the range between 0.003 in and 0.025 in. The fore mentioned cracks were recorded having lengths between 0.4 ft and 33 ft.

Bridge # 720702

SR 202 / SR 9A, JTB Interchange- Jacksonville, Fl

The characteristics of the cracking that were found on bridge 702 included only 35 cracks with an average crack width range from 0.001 in. to 0.017in. The lengths of these cracks were determined to range from 3 to 33 feet.

Bridge # 720704

SR 202 / SR 9A, JTB Interchange-Jacksonville, Fl

Bridge 704 has a total of 261 documented cracks across the length of the sections. The cracks depicted in the drawing have a range of average width from 0.001 in. to 0.015 in. As can be seen from the schematics most of the cracking occurs around mid-span between piers and not over the piers. The lengths of these cracks ranged between .8 and 33 feet. Detailed drawings of bridge 704 are also shown below.

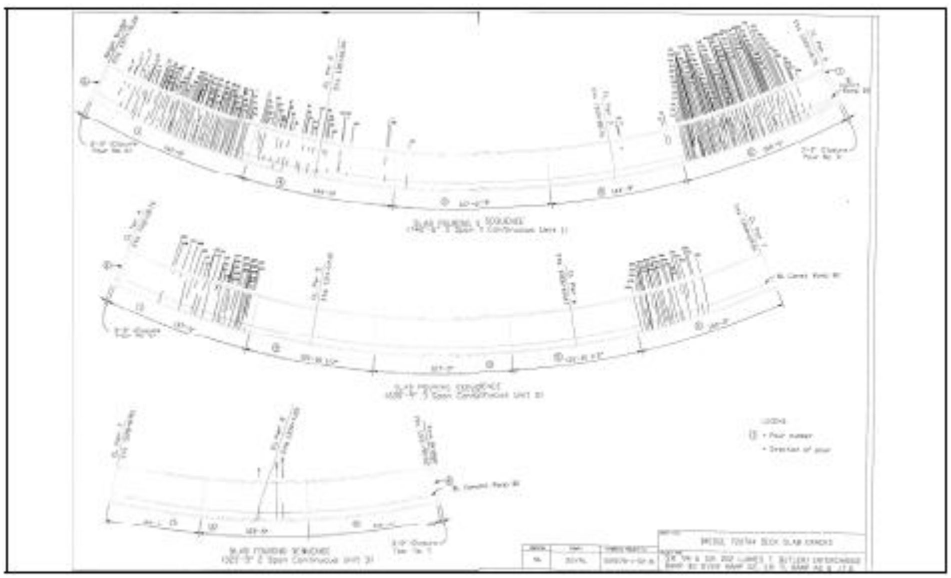


Figure 1-4 Full Sections of Bridge 704 and its Cracking Pattern

Bridge # 720705

SR 202 / SR 9A, JTB Interchange- Jacksonville, Fl

The 705 bridge cracks have been documented with an average crack width between .001 and .006 inches. The cracks throughout this bridge are far less severe than the others recorded. Similarly the lengths of the cracks only propagated between 1.6

feet and 33 feet. There were a total of 44 of these cracks found throughout the length of the 705 bridge section.

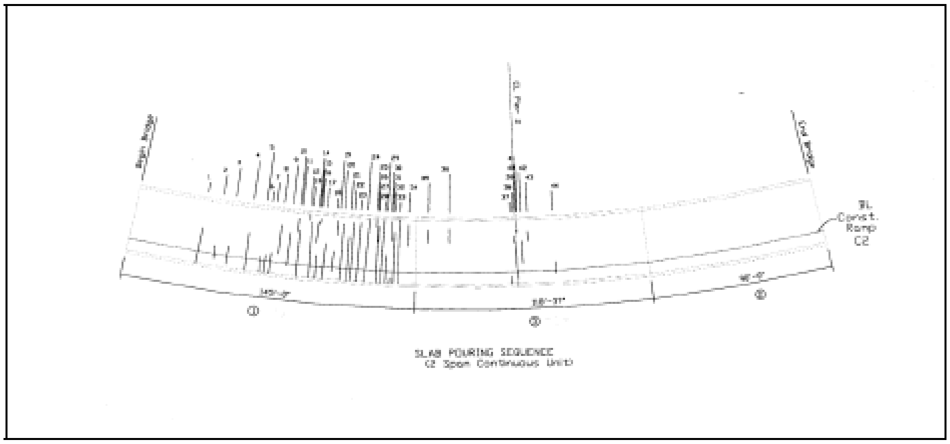


Figure 1-5 Full Sections of Bridge 705 and its Cracking Pattern

Bridge # 720706

SR 202 / SR 9A, JTB Interchange- Jacksonville, Fl

On bridge 706 only 29 visible cracks were found and documented. The cracks documented for bridge 706 are depicted visually in Figure 1-6. It was found that the cracks present on the top side of the deck on bridge 706 had widths ranging from 0.001 in to .005 in. The lengths of the cracks ranged from 1.4 feet to 32.3 feet.

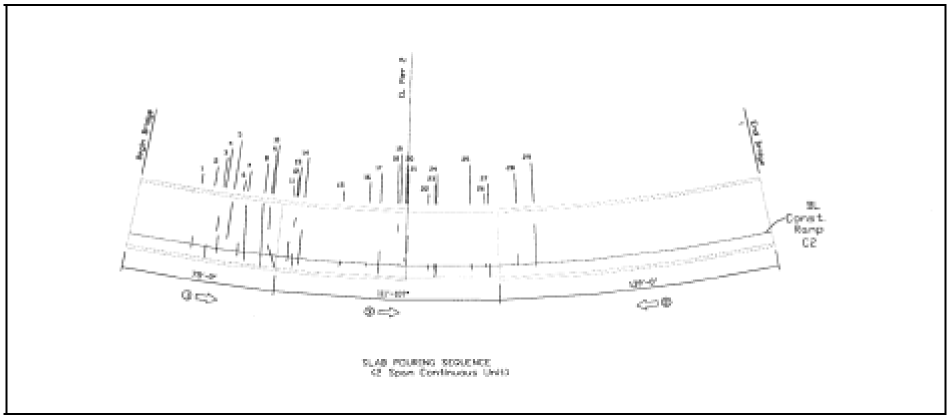


Figure 1-6 Full Sections of Bridge 706 and its Cracking Pattern

Bridge # 720707

SR 202 / SR 9A, JTB Interchange-Jacksonville, Fl

The total number of transverse cracks counted for the sections in the 707 bridge added up to 153 visual cracks. The ranges of the average widths of these cracks were from .001 in to 0.008 in; with lengths recorded from 1.1 feet to 45.7 feet. Detailed views of these cracks can be found in figure 1-7.

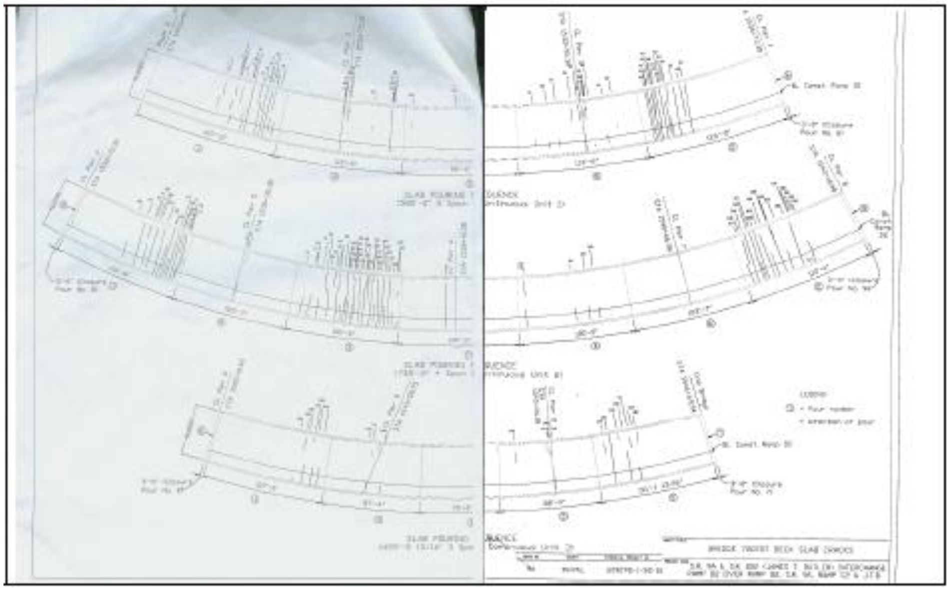


Figure 1-7 Full Sections of Bridge 707 and its Cracking Pattern

Similar to the other examples in this document, the majority of the cracking visible on the top side of the deck was found around the mid-span of the sections between piers.

Crack pattern in the bridges inspected in Fort Lauderdale is consistent with that in Jacksonville inspected bridges. Many transverse cracks have developed at mid span and few near the piers (negative moment region). There is evidence of some steel corrosion due to water leakage inside the steel box of bridge 860524 of Fort Lauderdale. Its location is associated with the deck cracks identified at the top of the deck surface.

Bridge # 780121

US-1, Jacksonville, Fl

We were fortunate to be able to track this bridge from the beginning, from reinforcement application to concrete placement. The procedures for concrete placement and curing were followed as advised by many researchers with finishing of the concrete within 20 minutes of placement and a wet curing time of 10 days however even with proper procedures the bridge deck began to show signs of transverse cracking within 30 days of concrete placement with 14 cracks appearing on day 26.

Figure 1-8 Reinforcement on Bridge 780121

Figure 1-9 SIP Forms for Bridge 780121

Figure 1-10 Concrete Placement and Curing for 780121

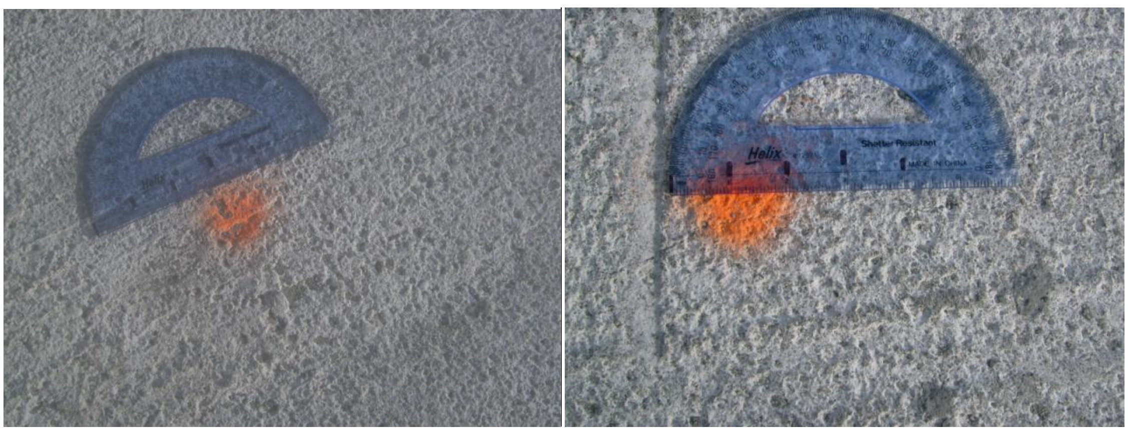


Figure 1-11 Transverse Cracking Before Deck Grooves

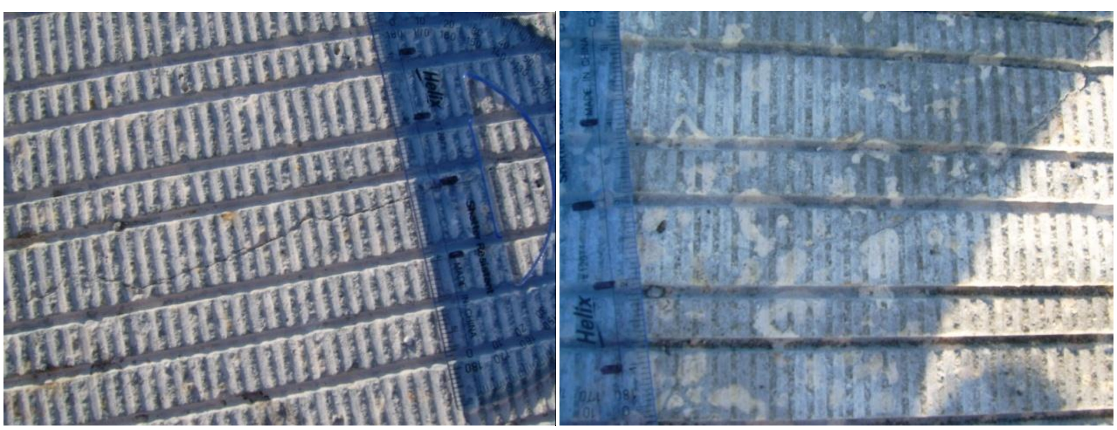


Figure 1-12 Cracking after Deck Grooves

Bridge # 780122

US 1, Jacksonville, Fl

This is a prestressed girder bridge with 14 spans that showed no signs of transverse cracking throughout its construction, however during the placement of the last span the concrete provider had to replaced and the new concrete that was placed began cracking 36 days after placement. The compression test of the concrete was 7477 psi which again verifies the conclusion of many researchers that high compression concrete has a higher propensity for transverse cracking.

Figure 1-13 View of Bridge 780122

Figure 1-14 Deck Cracking and Sealing of Bridge 780122

Bridge # 580167

I-10 over Blackwater River, Pensacola, Fl

This bridge is a 16 span steel girder bridge that developed severe transverse deck cracking and is the bridge used for the field test of the sealers. Most of the cracks on the bridge developed around the transverse rebar and ran all the way across to the parapet wall.

Figure 1-15 Transverse Cracking of Bridge 580167

Figure 1-16 Cracking on Parapet Wall Bridge 580167

Figure 1-17 Core Sampling of Bridge 580167

From this investigation, a survey was sent to the Florida Department of Transportation (FDOT) District offices asking if the cracking problem was widespread. The responses are summarized as follows;

- D1 & D7: "Deck cracking and leakage into D1 & D7 steel box girder bridges has not been an issue as far as we know. D1 and D7 report no such occurrences, but we do recommend that we always find ways to monitor this issue closely and improve design and construction whenever appropriate to ensure we do not have a maintenance problem."
- District 4: "We currently have 12 steel box girders in Broward County that is in the work program for deck sealing as the decks have many cracks over the steel box girders and can be seen going thru the deck at the overhangs. We have some rusting of the galvanized stay-in-place forms.

We also have one area that may be leaking into the box. The project is for year 2014".

- District 3: "I am not aware of any widespread issue of water infiltrating steel box girder bridges, but by copy of this email I am asking the District Structures Maintenance Engineers to respond directly to you with any occurrences. About a year ago Steve Plotkin in the Construction Office conducted a survey on cracking of concrete decks of steel bridges, to determine if there was a need to update the construction specifications regarding the curing of concrete decks on steel bridges".
- District 2: Construction Structures Engineer, FDOT, Jacksonville indicated: "My investigation concluded that the current curing procedures are very effective and that the problem is design related since the coefficient of expansion for steel beams is significantly larger than it is for the concrete deck, the deck is put into tension during times of maximum expansion of the beams and this causes the deck cracking. This is a nationwide issue and has been for a very long time but is generally considered benign in Florida since we do not use deicing salts and the cracking is typically minor. The State Structures Design Office looked into a solution to this problem and concluded, based on the lack of deck deterioration problems reported by Maintenance, that the cost of adding enough crack control rebars to eliminate or dramatically reduce the cracking would not be worth the cost. In other words, there is a good cost benefit to allowing minor deck cracks to form since performance or durability of the decks is not reduced significantly during their service life".

1.4 Mechanism of Cracking in Hardened Concrete

The initial review of early age transverse deck cracking is a study of hardened concrete as compared to cracking of concrete while still in its plastic state. There are several mechanisms contributing to cracking of hardened concrete. This paper focuses on three of these mechanisms: drying shrinkage, autogenous shrinkage, and thermal stresses.

Restrained drying shrinkage occurs due to the volume change induced by a loss of moisture in the cement paste. The concrete would not crack if this shrinkage could

occur without the restraint from structural elements, the subgrade, or the moist interior of the concrete itself. This volume change coupled with restraint cause tensile stresses in the concrete that can lead to cracking[4]. These tensile stresses are influenced by the amount and rate of shrinkage, the degree of restraint, the modulus of elasticity, and the amount of creep. The amount of drying shrinkage is a function of the amount and type of aggregate and the cement paste content of the concrete. Methods to reduce shrinkage cracking include using contraction joints, careful detailing of reinforcement, shrinkage-compensating admixtures, and reducing the sub slab restraint.

Autogenous shrinkage is a special type of drying shrinkage, resulting from self-desiccation or internal drying, occurring in concretes with water-cementitious (w/cm) materials below 0.42. This type of shrinkage differs from typical drying shrinkage in that there is no loss of moisture from the bulk concrete. Autogenous shrinkage strain is typically about 40 to 100 microstrain, but has been measured as high as 2300 microstrain in concrete with a w/cm ratio of 0.2. Autogenous shrinkage has been found to increase with increasing temperature, cement content, and cement fineness.

Thermal cracking is also discussed. Temperature differences in a concrete structure result in volume changes causing tensile stresses. The dissipation of the heat of hydration of cement and changes in ambient temperature can create temperature differentials that cause tensile stresses in concrete structures. These tensile stresses are proportional to the temperature differential, the coefficient of thermal expansion, the effective modulus of elasticity, and the degree of restraint. Methods of reducing

thermal cracking include reducing maximum internal core temperature, delaying the onset of surface cooling, controlling the rate at which the concrete cools, and increasing the early age tensile strength of the concrete[5].

1.5 Transverse Cracks in Bridge Decks

Transverse cracks have been a common problem in highway bridge decks in the past and continue to cause maintenance headaches today. Transverse cracks in bridge decks develop during the hardened concrete phase at early ages before service loads are applied. They are full-depth cracks and are typically spaced at 3 to 10 feet apart[6]. Transverse cracks are the most frequently observed cracks in concrete bridge decks. Below in Fig. 1-8 is a depiction of the cause of transverse cracking.

Graphic redacted. Paper copy available upon request to home institution.

Figure 1-18 Cause of Transverse Cracking on Bridge Decks (courtesy of http://www.whrp.org/research-areas/structures)

There are a number of problems associated with transverse cracking of bridge decks. Transverse cracks can reduce the service life of structures and increase

maintenance costs. Structural problems include accelerated corrosion of reinforcing steel, deterioration of deck concrete, and possible damage to underlying components. Transverse deck cracking can also be detrimental to the overall bridge aesthetic. Transverse deck cracking also increase carbonation and chloride penetration leading to accelerated corrosion and deterioration.

1.6 Causes of Transverse Cracking

From observation, bridges designed by the American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) specifications have an optional deflection limit. The AASHTO standard specification limits live-load deflections to L/800 for ordinary bridges and L/1000 for bridges in urban areas that are subjected to pedestrian use. This deflection limit is also incorporated in AASHTO LRFD specifications in the form of optional serviceability criteria.

This limit has not been a controlling factor in most past bridge designs. Previous research has shown that justification for the current AASHTO live-load deflection limits is not clearly defined, and the best available information indicates that these limits were developed to control undesirable bridge vibration and to ensure comfort. We have also observed that due to increased deflections on steel girders, the frequency of transverse deck cracks are greater than those on decks, of the same thickness, with concrete girders.

2. Research Review

The preliminary step in the research of bridge deck cracking was a thorough review of available research papers to gain a better understanding of the problem and to see if any of them would be relevant to our research, below is a summary of the most relevant research papers a more thorough review can be found in appendix A.

The earliest noted study conducted by the Portland Cement Association, the Bureau of Public Roads, and ten state highway departments, and was released in 1970. The purpose of the study was to determine concrete bridge deck durability problems, causes of the types of deterioration, methods to improve durability, and methods to inhibit existing deterioration.

In this study transverse cracking was observed to be the most common type of cracking. Older decks and longer spans showed more transverse cracking, and continuous span bridges and steel girders appeared to exacerbate transverse cracking[7].

In a study conducted for the Pennsylvania Department of Transportation, researchers surveyed four year old bridge decks in Pennsylvania to investigate the extent and causes of concrete bridge deck deterioration[8]. The researchers found transverse cracks in 60% of all spans and 71% of all bridges.

In another study assessed bridges in Pennsylvania through 99 field surveys and 12 in-depth surveys to determine the causes of transverse cracking. These surveys included crack mapping, crack width measurements, rebar location and depth surveys, concrete coring, and construction records[9].

An important finding made by the researchers was that the transverse cracks intersected coarse aggregate particles; this indicates that transverse cracking occurs in hardened concrete rather than plastic concrete.

Schmitt and Darwin conducted a study on the effects of different variables on bridge deck cracking, dividing the variables into five categories: material properties, site conditions, construction procedures, design specifications, and traffic and age[10].

The material properties considered included admixtures, slump, percent volume of water and cement, water content, cement content, water-cement ratio, air content, and compressive strength.

Site condition factors considered in the study were average air temperature, low air temperature, high air temperature, daily temperature range, relative humidity, average wind velocity, and evaporation.

Construction procedure factors considered in the study were placing sequence, length of placement, and curing. There were no observed relationships between length of placement or type of curing materials and cracking. No correlation between cracking and placing sequence could be determined due to lack of information[10].

Design factors considered in the study included structure type, deck type, deck thickness, top cover, transverse reinforcing bar size, transverse reinforcing bar spacing, girder end conditions, span length, bridge length, span type, and skew.

Regarding traffic and age, the researchers found that cracking increased with traffic volume and that bridges constructed prior to 1988 exhibited less cracking than bridges constructed after 1988. The increase in cracking in newer bridges was attributed to changes in construction, material properties, and design specifications[10].

Krauss and Rogalla conducted what is likely the most comprehensive study to date. They surveyed 52 transportation agencies in the United States and Canada to evaluate early age transverse cracking. Over 100,000 bridges were found to have developed early transverse cracks. Analytical studies were also performed using both theoretical and finite element analysis to evaluate the influence of several different parameters on transverse cracking[11].

The researchers determined that span type, concrete strength, and girder type were the most important design factors influencing transverse cracking. Material properties such as cement content, cement composition, early-age elastic modulus, creep, aggregate type, heat of hydration, and drying shrinkage also influenced deck cracking[11].

Researchers conducted a field investigation of 72 bridge decks in Minnesota. The researchers determined that design factors most related to transverse cracking were

longitudinal restraint, deck thickness, and top transverse bar size[12]. Material factors most affecting transverse cracking were cement content, aggregate type and quantity, and air content.

Researchers in Minnesota performed a parametric study considering bridges with steel and prestressed concrete girders[12]. Among variables considered for steel girder bridges were: end conditions, girder stiffness, locations of cross frames, girder splices, supplemental reinforcing bars, shrinkage properties; concrete modulus of elasticity; and temperature differential due to heat of hydration. Variables considered for prestressed girder bridges were the times casting relative to the times of both strand release and deck casting, and shrinkage properties of the deck and girders.

From a research sponsored by the Indiana Department of Transportation, researchers conducted a field study and constructed laboratory specimens to investigate the behavior of transverse cracks[13]. Using these specimens, the researchers could evaluate the effects of differing bridge deck designs on the control of overall shrinkage and the contribution of Stay-in-Place (SIP) steel forms to the formation of transverse cracking.

3. Crack Sealers

The most commonly marketed sealers include; epoxies, reactive methyl methacrylates (MMA), methacrylates, high-molecular weight methacrylates (HMWM), and polyurethanes. All these products have distinct characteristics that make them favorable for some uses and unfavorable for others. Properties include volatility, viscosity, initial shrinkage, tensile strength, and tensile elongation. Some surveys of 40 states 60% indicated that they did not have a crack sealing program, 24% use epoxies and methacrylates, none were asked about HMWMs, MMAs, or polyurethane resins[14, 15]. Another survey stated that epoxy was the predominant sealer[15]. Only four of sixteen states that had a crack sealing program claimed to use HMWM sealers.

This investigation will concentrate on epoxies and methacrylates, both HMWM and MMA, as they possess the properties closest to the requirements in the Qualified Products List (QPL) of the Florida Department of Transportation.

3.1 Sealer Products

In addition to MMA, HMWM, and Polyurethane; epoxy was also investigated. Epoxies are typically developed by a reaction between biphenol A and epichorohydin. They are made from cyclic ethers that harden during a polymerization process.

Epoxies have high tensile strengths sometimes four times higher than HMWMs. Epoxies are typically more expensive than other types of crack sealers.

HMWMs are made from methacrylate monomers, and while in the curing process an initiator is added to create an oxidation / reduction reaction, the monomer then develops into a high molecular weight polymer[16-21]. Care should be used when mixing the three component system as it has the potential of becoming violent. HMWM sealers are known for their low viscosity and high penetration depths. MMAs are two-component sealers that have some of the same characteristics as HMWMs but are much safer to use. MMA is formed from reactive methyl methacrylate catalyzed by a 50% dibenzoyl peroxide powder.

3.2 Material Characteristics and Performance Measures

NCHRP indicated that crack sealers are measured in four primary ways: depth of penetration, bond strength, chloride content / resistance to corrosion, and seepage rate. There is a lack of standardized tests to investigate the performance of crack sealers making it more challenging to compare results. Elongation is being considered as another characteristic to be investigated.

Depth of Penetration: The test for depth of penetration for crack sealers is completely different to that of a concrete sealant. Sealers are used to cover or fill an already formed crack. It is presumed that the larger the depth a sealer can penetrate the better the seal it will create, but due to the variability of crack widths it may be

more useful to measure the percentage of penetration versus the actual penetration depth [22, 23, 24]. The method of testing for depth of penetration involves removing a core from the concrete deck and looking at a cross section of the crack with a microscope. If the resin has faded or is not readily visible, a florescent dye can be applied to the crack and viewed under an ultraviolet light. Another method involves cracking the core sample and placing drops of water until the water stops beading then obtaining the average depth from all the cores. The important part in the depth of penetration of a sealer is the proper cleaning of the crack prior to the sealer application as this affects the performance of the sealer the most [25, 26].

Bond Strength: The ability of a resin to repair the structural problem in a cracked deck is measured by its bond strength. There is no standard method to test for bond strength, as a consequence of this engineers use a few different tests to determine bond strength. The most common test is the tensile splitting test from the American Society for Testing and Materials (ASTM) C496. This test involves placing a core sample on its side in a compression machine. The repair crack is placed perpendicular with the compressive load, which causes a tensile load to develop in the crack. The compressive load required for the repaired crack to fail is then compared to compressive load used to fail the uncracked core sample. A ratio is obtained by dividing the cracked sample capacity by the uncracked sample capacity. This is the percentage of the strength retained by the sealer. Another method is the three-point bending flexural test ASTM C293. This test is normally performed with beams cast in the laboratory. Again a ratio is

developed to obtain the percent of strength retained by the sealer. Once the test is chosen and conducted, the failure surface is observed and documented. From these data three different types of failure planes can be produced, these are concrete, bond, and sealer failure.

Seepage: The indication of how well the repaired pavement will prevent chloride ion ingress is called seepage. Seepage is measured by the volume of water that passes through the cracked concrete. It is suggested that the least amount of water that passes through the crack the better the rebar of the deck is protected. Several tests are used to check for seepage. One test involves forming a barrier around the top of the concrete core sample, after the sides are waterproofed; water is poured into the barrier on top of the core sample. The water height is kept constant and the rate in which water passes through the core is recorded. The number of leaks before the cracks were sealed is compared to the number of leaks after the cracks were sealed. This test is mainly used in the field to give an indication of the success of the repair.

Chloride ingress and corrosion: Chloride ions can infiltrate the concrete and corrode the reinforcement if there exists any cracking on the bridge deck. Crack sealers act as a barrier to slow down this ingress of chloride ions into the concrete. This problem occurs mainly in the northern states where there is tendency of having freeze-thaw cycles and the use of road salt for deicing.

Elongation: There is a big variation for elongation of different sealers that range between 3% and 60%. It is worth investigating.

We will only concentrate on three of the performance measures for crack sealers; depth of penetration, bond strength, and elongation.

4. Experimental Testing and Field Investigation

In this study, an investigation of a database on bridge information was made that included, crack location, concrete mix ingredients and properties, construction method, superstructure type, possible causes for cracking, and other relevant data for selected bridges in Florida. The study aimed at gaining a better more up-to-date understanding of early concrete cracking of bridge decks and overlays, identifying the key factors which cause early concrete cracking in bridge deck, investigating whether live-load deflection limits or vibration control are important factors in bridge deck cracking, identifying suitable materials for crack sealing with the most suitable materials presented with their ability to span cracks of various widths. The benefits and limitations of each material will also be presented.

Laboratory tests were performed on crack sealants with the following performance criteria; penetration depth, bond strength plus elongation with factors of temperature, type of sealant, debris and elongation.

Field tests of sealant were also performed on bridges. Tests were also performed on slab and beam samples. Tests also included chloride sampling cores or drill dust samples or water flooding of the treated deck areas to check water leak, core tests to determine the depth of sealant penetration, use dissection/stereo microscopes to

determine resin depth and fluorescent and long-wave UV lighting. Observe for new cracks to form near newly repaired cracks. All preliminary testing was performed at the FDOT testing facilities.

4.1 Lab Procedures for Materials and Testing Results

From the information gathered we established a testing schedule for the crack sealers. First we researched available crack sealers and their properties to establish a list of candidates that most closely matched the FDOT Qualified Products List (QPL) 413.

Table 4-1 QPL 413 Material Properties (copied from FDOT Products Manual)

413-3.2.1 Properties: Use a methacrylate material that meets the following physical and performance requirements:

Table 2: Physical Properties of Methacrylate Resin	
Viscosity (Brookfield RVT)	14-20 cps at 50 rpm
Density (ASTM D1481)	8.5 - 9.0 lb/gl at 77° F
Flash Point (ASTM D93)	> 200 °F (Pensky Martens CC)
Odor	Low
Bulk Cure Speed	3 Hours @ 73° F (max.)
Surface Cure	8 Hours @ 73° F (max.)
Gel Time (ASTM 2471)	60 minutes (max.)
Tack Free Time	5 Hours (max.) (at 72 °F and 50% Relative Humidity
Compressive Strength (AASHTO T106)	6,500 psi (min)
Tensile Strength (ASTM C307)	1,300 psi (min)
Shear Bond Adhesion (ASTM C882)	600 psi (min)
Wax Content	0

From the research, five manufacturers were identified with a total of ten products tested for compatibility to the QPL. The products were further researched and a final list

of five products from four manufacturers was thoroughly tested, both in the field and on the laboratory.

The manufacturers were contacted and asked to provide both wet and dry samples for testing. Preliminary tests were performed on the sealing materials according to the specifications to obtain a baseline. Figure 4-1 shows samples of prepared sealing materials. Figure 4-2 also shows testing the sealant material that bonded concrete specimens in accordance with ASTM C882, this test is performed as follows; the bond strength is determined by using the epoxy system to bond together two equal sections of a 3 by 6-in. [75 by 150-mm] Portland-cement mortar cylinder, each section of which has a diagonally cast bonding area at a 30° angle from vertical. After suitable curing of the bonding agent, the test is performed by determining the compressive strength of the composite cylinder. Other tests were conducted on scaled-down deck panels sealed with the sealing materials that had the best performance. The sealing materials were also tested on a Florida Bridge as shown in Figure 4-3.

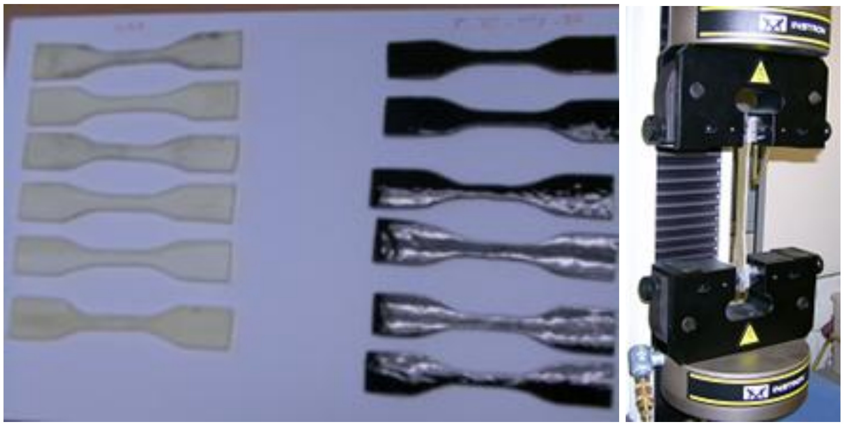


Figure 4-1 Preparing the Specimens for Testing

Figure 4-2 Applying the Sealant to Concrete Specimens and Testing

From the list of sealers shown in Table 4-2, five sealers were selected that more closely matched the Qualified Products List (QPL) of the Florida Department of Transportation. The chosen five were further tested in the lab to verify that the properties in the manufacturer's data sheet was accurate. To avoid any questionable or nulled results we requested that the sealer samples be prepared by the manufacturer under their lab conditions. Test results are shown in Table 4-3.

Table 4-2 Properties of Sealant Materials Investigated in the Study

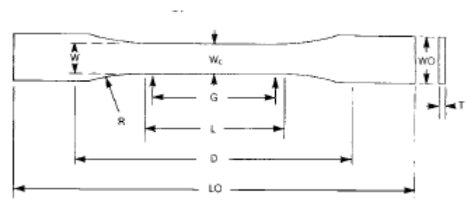
Sealant	Description
1	Methacrylate. Bond Strength 615 psi, Tensile Elongation 3-5%. Viscosity <20 cps, Flash Point >210F. Pot Life 70F: 25-40 min, Tack Free 70F: 4-7 hrs
2	Methacrylate. Bond Strength 615 psi. Tensile Elongation 30%. Viscosity <25 cps, Flash Point >200F. Pot Life 70F: 40-60 min, Tack Free 70F: 5-8 hrs
3	Is a 2-Component, 100% solids, Moisture-tolerant, epoxy crack healer/ Penetrating sealer. Bond Strength 14 days – 2,500psi. Tensile Strength 7, 100 psi, Elongation 10%. Viscosity 105 cps, Flash Point N/A. Pot Life 20 min, Tack Free 73F: 6hrs, 90F: 2.5 hrs
4	Methacrylate. Is a 3 component, low viscosity, solvent free, high molecular weight methacrylate penetrating sealer and crack healer. Tensile Strength 2,800 psi, Elongation 40-50%. Viscosity 5-20 cps, Flash Point >200F. Pot Life 45 min, Tack Free up to 6 hrs
5	Epoxy Sealer. Is a two- component, ultra low viscosity, gravity feed or pressure injected. Bond Strength 14 days – 3,450 psi. Tensile Strength 7,100 psi, Elongation 2.9%. Viscosity 95 cps, Flash Point >200F. Pot Life 45 min, Tack Free 70F: 12 hrs, 80F: 6 hrs
6	Methyl Methacrylate (MMA). Is a solvent free, 2-component, 100% reactive-resin. Tensile Strength 1,200 psi, Elongation 220-300%. Viscosity 95 cps, Flash Point 48F. Pot Life 25 min, Tack Free 1hr
7	Methacrylate. Is a low viscosity, low surface tension, solvent free, penetrating sealer and crack healer. Tensile Strength 8,100 psi, Elongation 5.5%. Viscosity 5-15 cps, Flash Point 48F. Pot Life 15-20 min, Tack Free 1hr
8	Epoxy. Is a rapid-curing, skid-resistant epoxy concrete overlay system. Tensile Strength 2,500 psi, Bond Strength 2,500 psi. Elongation 30%, Viscosity 1000-2500 cps. Flash Point 200F, Pot Life 15-25 min, Tack Free 2hrs
9	Methacrylate. Is a 3 component, reactive resin used as a wearing coarse. Tensile Strength 1,290-1,380 psi (Body coat), 2,150 psi (Top coat). Elongation 13% (Body coat), 35% (Top coat). Viscosity N/A, Flash Point 48F, Pot Life N/A. Tack Free 1hr
10	Cementitious Material. Is a 2 component screedable, shrinkage-compensated pre-extended cementitious repair material. Tensile Strength 500 psi Bond Strength >2,000 psi (28 days).

Table 4-3 Selected Sealer Tests and Results

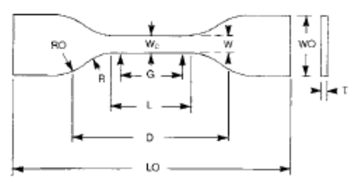
	Manufacturer's Data				Lab Test Data (7/29/10)	
Actual Product	Product	Tensile Strength (MPa)	Viscosity (cps)	Elongation (%)	Tensile Strength (MPa)	Elongation (%)
7	1-A	56.4	5-15	5.5	22.6	2.1
4	2-B	19.3	5-20	40-50	0.9	99.6
6	3-C	41.0-48.0	30	3-7	23.4	10.5
8	4-D	6.9	80	60	3.36	213
2	5-E	8.2	10-25	30	No test	No test

*Actual product number is from table 4-2.

Table 4-4 Specimen Preparation Procedures from ASTM D638



TYPES I, II, III & V



TYPE IV

Specimen Dimensions for Thickness, *T*, mm (in.)[A]

Dimensions (see drawings)	7 (0.28) or under		Over 7 to 14 (0.28 to 0.55), incl	4 (0.16) or under		Tolerances
	Type I	Type II	Type III	Type IV[B]	Type V[C,D]	
W—Width of narrow section[E,F]	13 (0.50)	6 (0.25)	19 (0.75)	6 (0.25)	3.18 (0.125)	±0.5 (±0.02)[B,C]
L—Length of narrow section	57 (2.25)	57 (2.25)	57 (2.25)	33 (1.30)	9.53 (0.375)	±0.5 (±0.02)[C]
WO—Width overall, min[G]	19 (0.75)	19 (0.75)	29 (1.13)	19 (0.75)	...	+ 6.4 (+ 0.25)
WO—Width overall, min[G]	...	...	...	...	9.53 (0.375)	+ 3.18 (+ 0.125)
LO—Length overall, min[H]	165 (6.5)	183 (7.2)	246 (9.7)	115 (4.5)	63.5 (2.5)	no max (no max)
G—Gage length[I]	50 (2.00)	50 (2.00)	50 (2.00)	...	7.62 (0.300)	±0.25 (±0.010)[C]
G—Gage length[I]	...	...	...	25 (1.00)	...	±0.13 (±0.005)
D—Distance between grips	115 (4.5)	135 (5.3)	115 (4.5)	65 (2.5)[J]	25.4 (1.0)	±5 (±0.2)
R—Radius of fillet	76 (3.00)	76 (3.00)	76 (3.00)	14 (0.56)	12.7 (0.5)	±1 (±0.04)[C]
RO—Outer radius (Type IV)	...	...	...	25 (1.00)	...	±1 (±0.04)

Further sealant testing was performed on this set of sealers as discussed later in the field testing results.

4.2 Field Material Testing and Results

After compiling the baseline material properties we coordinated a field test of the products with the FDOT. The manufacturers were contacted to provide the materials and a crew to mix and apply the products to avoid any nulled results. We performed the field test on bridge # 580167 at I-10 over the Blackwater River in August 2010. The east-bound right hand lane and shoulder was divided into six sections of twenty feet by fifty feet and had the manufacturers provide their own crews to apply the products to avoid any improper mixing and application procedures. Below is a schematic of the bridge area divided into the six test section.

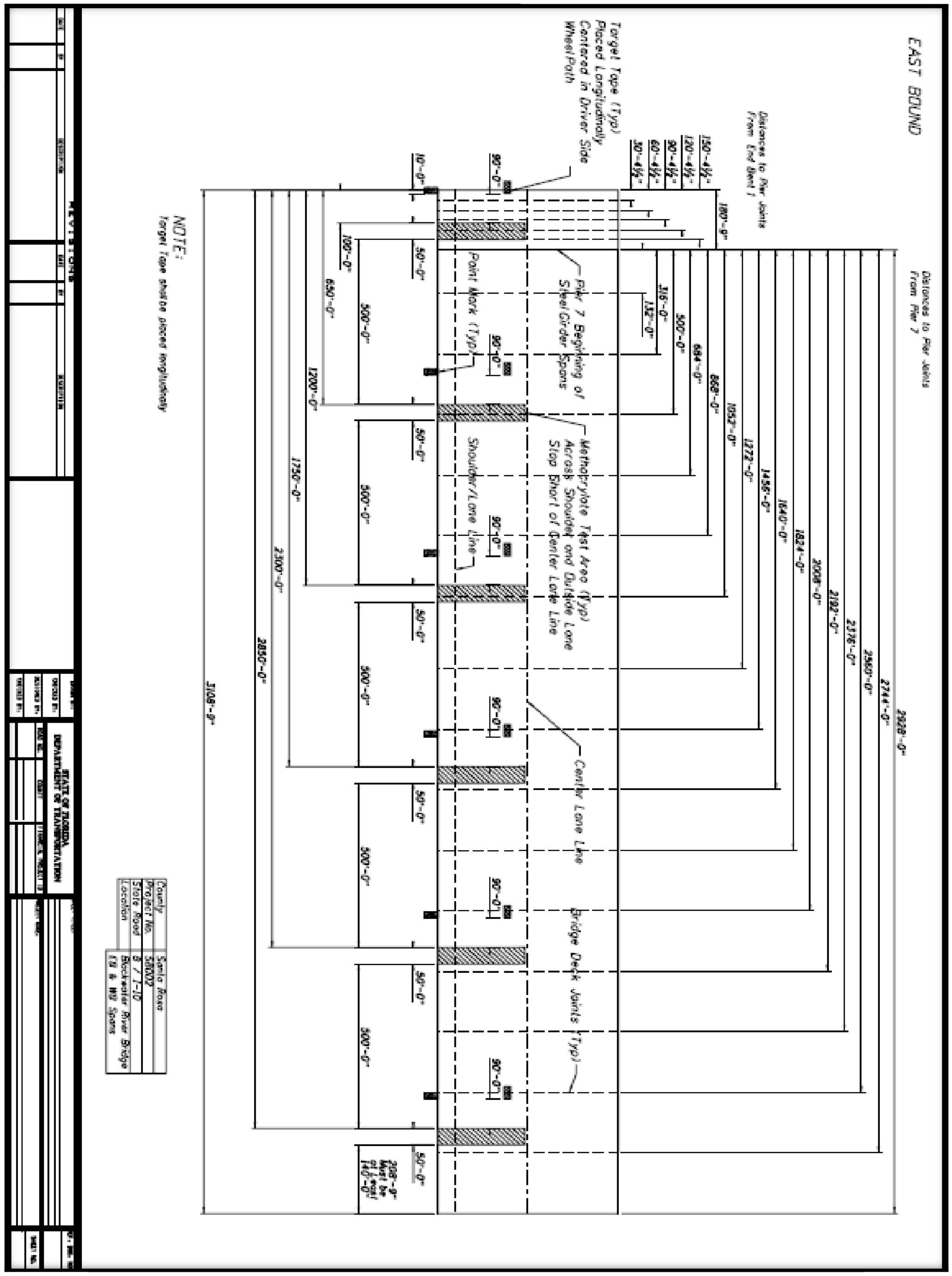


Figure 4-3 East-bound Section Divided into Six Test Areas

Figure 4-4 Clean-up of Bridge 580167

Figure 4-5 Sealer Application Bridge 580167

Figure 4-6 Sand Application for Skid Resistance Bridge 580167

The surface was prepared, the cracks were cleaned properly by the FDOT, the sealant materials were applied according to the specified procedure in the manufacturer's data sheet, and sand was sprinkled to provide skid resistance. Table 4-5

shows the field test results for the five sealants applied to parts of the same bridge. The deck surface was allowed to dry for the indicated time of each product. Core samples were taken, after curing of the sealant, at random locations of each test area and tests were performed in accordance to ASTM C-496 at the FDOT laboratory facilities. The cores and testing procedure is shown in Figure 4-7.

Figure 4-7 Bridge 580167 Cores and Testing Procedure

Table 4-5 Field Test Results Bridge 580167(Provided by FDOT)

Sample ID	Material Type	Location	Peak Load (lbf)	Ave (lbf)	Penetration (in.)	Ave (in.)
Core 1-1	Concrete 1.75" core	Test section 1	1391	1591	0.70	0.475
Core 1-2			1131		--	
Core 1-3			2250		0.25	
Core 1-4			2540		N/A	
Core 2-1		Test section 2	1809	2363	0.70	0.767
Core 2-2			2230		0.70	
Core 2-3			3050		0.90	
Core 3-2		Test section 3	1954	1277	0.90	
Core 3-3			599		0.80	0.850
Core 5-1		Test section 5	2090	1731	N/A	
Core 5-2			2320		0.50	
Core 5-3			3000		N/A	
Core 5-5			1035		0.45	
Core 5-6			1838		0.50	0.483
Core 6A-		Test section 6 – first part	1973	2237	1.00	
Core 6A-			2770		N/A	
Core 6A-			2340		N/A	
Core 6A-			2500		--	1.000
Core 6B-		Test section 6 – second part	1641	2241	0.60	
Core 6B-			2840		0.60	0.600
Core 6B-			1799		N/A	
Core 6B-			1921		N/A	
Core 6B-			1860	2290	N/A	

Table 4-6 below also shows the core test results and the field skid test.

Table 4-6 Sealant Field Test Results for Bridge 580167 (Provided by FDOT)

Test Site #	Product	Components	Viscosity (cps)	Elongation (%)	Curing time (Hr)	Skid Ave.	Tensile strength Ave. (lbf.)
TS -1	1	2 part Epoxy	30	5.5	2-3	62	1591
TS -2	2	3 part Methacrylate	5-20	40-50	4-6	37	2363
TS -3	3	2 comp Methacrylate	5-15	5.5	1	22	1276
NA*	4	2 comp Methacrylate	1100-1500	220-300		---	---
TS -5	5	3 part Methacrylate	14-15	20	4-6	37	1731
NA*	6	3 part Methacrylate	14-15	30	6	---	---
TS -6	7	Epoxy	80	60	2	50/74	2236/2240
TS-4	Con-trol	No sealer applied				48	2290

*Not applied

The average penetration of the material in the cracks ranged from 0.5 to 1 inch. However, because of the different crack widths, the information should not be used for direct comparison of the materials. All materials appear to have acceptable penetration. Below you will see pictures of the penetration test that was performed on some of the core samples using the water drop/bead test procedure.

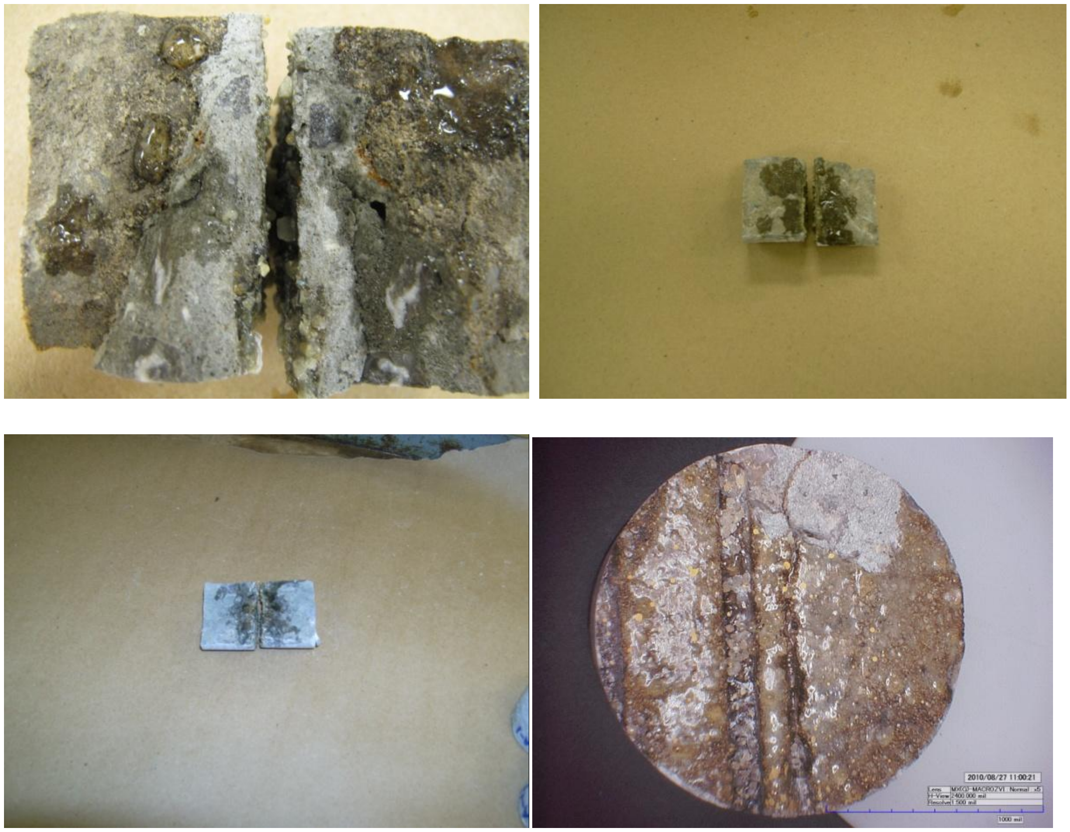


Figure 4-8 Penetration Tests of Core Samples

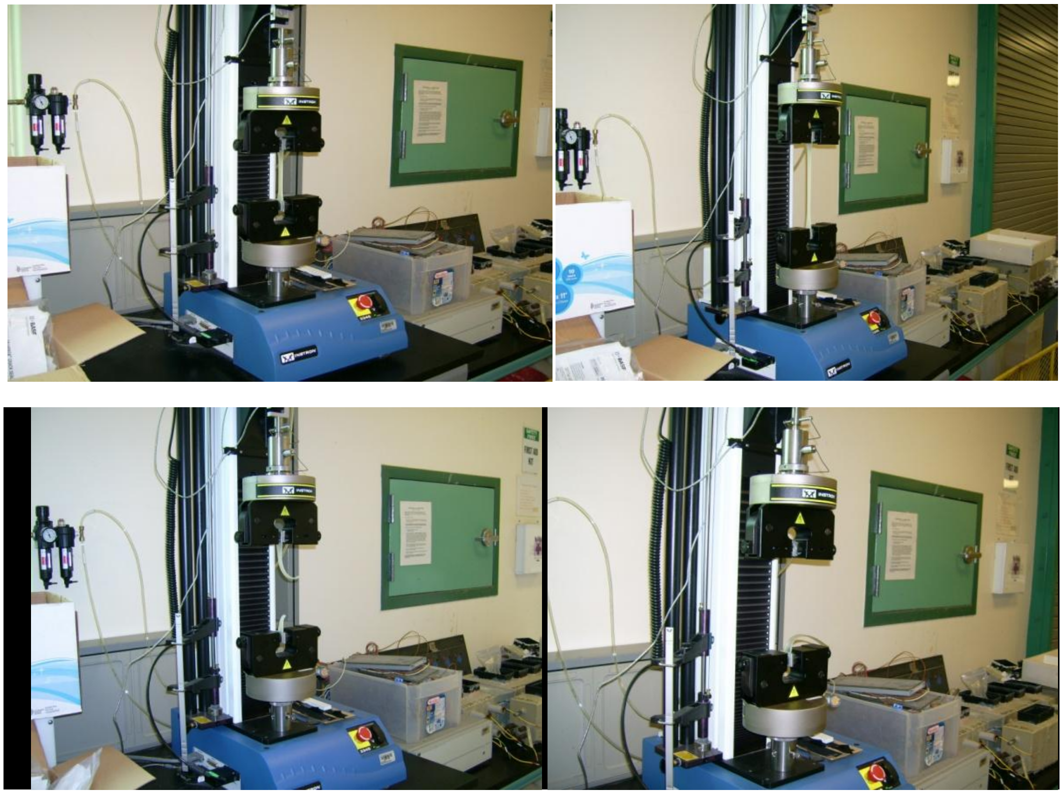


Figure 4-9 Tensile Testing of Sealants

The results were shared with the manufacturers whose main complaint was the width of the cracks was not equal for all and from their comments it was decided to perform a secondary lab test on the crack sealer products, as shown in figure 4-9, with the following results shown in Table 4-7.

Table 4-7 Tensile Testing of Deck Sealers

Manufacturer's Data				Lab Test Data (7/29/10)		Lab Test Data (12/8/10)	
Product	Tensile Strength (MPa)	Viscosity (cps)	Elongation (%)	Tensile Strength (MPa)	Elongation (%)	Tensile Strength (MPa)	Elongation (%)
1-A	56.4	5-15	5.5	22.6	2.1	25.3	2.6
2-B	19.3	5-20	40-50	0.9	99.6	7.3	17.2
3-C	41.0-48.0	30	3-7	23.4	10.5	27.2	N/A
4-D	6.9	80	60	3.36	213	5.5	102
5-E	8.2	10-25	30	No test	No test	5.7	1.3

As can be seen the results of the second test is very close to the initial test but still far away from the manufacturers data. With the manufacturers input we devised a plan to build nine scaled down slabs for further testing of the sealers. The slabs were poured using the recommended Type II Portland Cement with a designed compressive strength (f'_c) of 5000 psi and a final compressive strength of 7966 psi. The slabs measured 18 inches wide by 4 feet long and 5.5 inches thick as shown in figure 4-10, 4-11. Lab testing was conducted, on slab model construction having blade placement to create "ideal" cracks of 0.01 to 0.02 inch of width and a spacing of 4 inches from center. The main complaint from manufacturers was that the crack width in the bridge decks was not equal in the field test. Therefore we created a set of blades (figure 4-12) with two different widths and lengths to plunge into the concrete, while still in the plastic stage, and remove before final set. That resulted in inducing 2 different crack widths (0.01", 0.02") and 2 different crack lengths (9", 18"). (All previous testing performed by researchers dealt with cracks created by stressing concrete or by load application but

that would not have created equal cracks for all the slabs as requested by manufacturers).

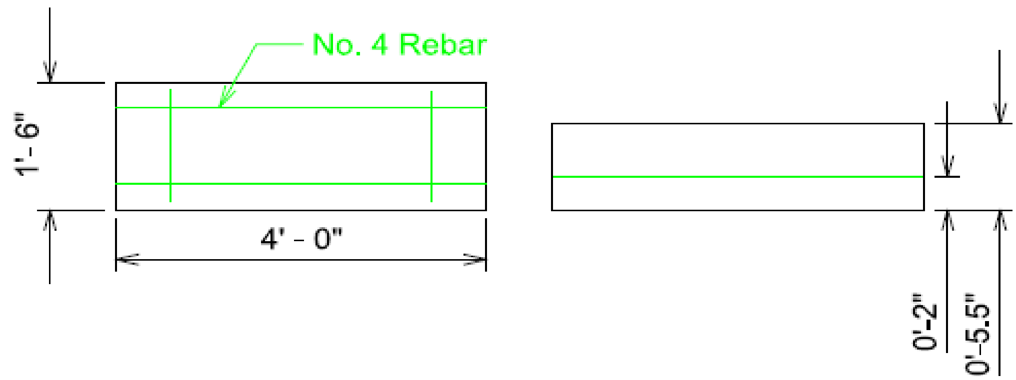


Figure 4-10 Sketch of Slab for Construction

Figure 4-11 Construction of Test Slabs and Placement of Blades for Crack Width

Figure 4-12 Blades Constructed for Ideal Cracks

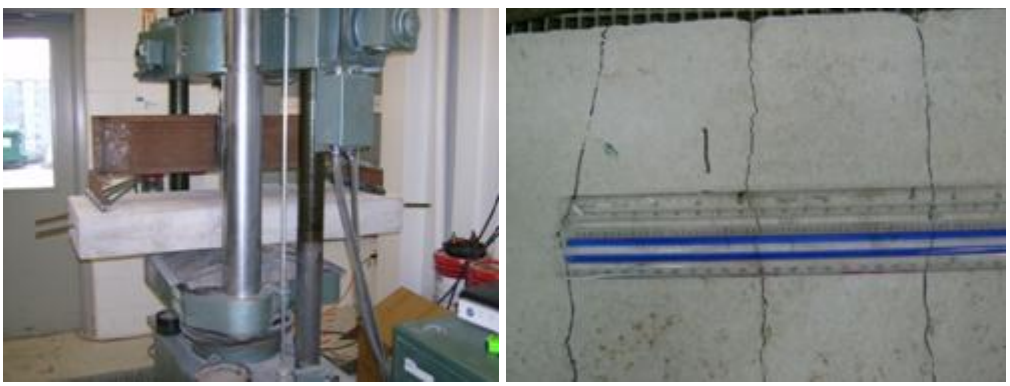


Figure 4-13 Testing of Control Slab Specimen and Crack Development

The final set time of the concrete was calculated using Schindler's research on "Prediction of Concrete Setting" so that we could remove the blades without the concrete closing the crack created, we also used a one inch piece of blade to be applied at slab corner to check the theoretical time obtained from the equations.

From previous research and observation on the tested bridge, we tested a control slab (with no cracks) to a load of 7200 lb. That load caused 3 cracks spaced at

approximately 4 inches beginning at the center line of the slab as shown in Figure 4-13, this confirmed the placement of the blades on the slabs with induced cracks as being correctly placed, with the load test results shown in Figure 4-14, maximum deflection occurred at 7000 lbf and was 0.01914 in with the first crack occuring at 2250 lbf the second at 4000 lbf and the third at 6250 lbf.

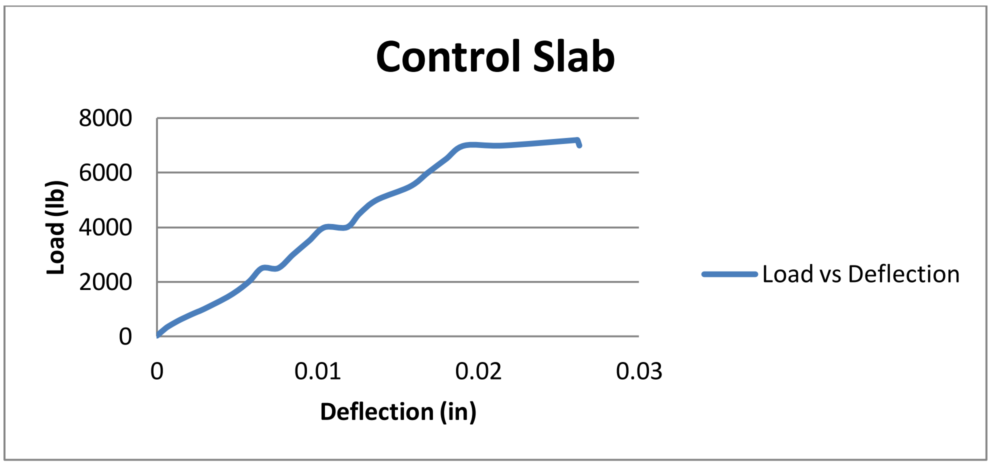


Figure 4-14 Control Slab Load Test Results

The cracks on the rest of the slabs were sealed with the appropriate sealer, as shown in Figure 4-15 and allowed to cure for several weeks. During that time we devised a way to test the slabs upside down (cracks facing up) so as to simulate the cracking on a continuous bridge where the cracks appear on the top surface of the opposite span from where the load is applied. Another reason was to observe the crack behavior under the load during the test. To accomplish both of these goals we built a spreader bar and two different point load bars with the single point load shown in figure 4-16.

Figure 4-15 Sealed Slabs with LDVT's and Dial Gage

Figure 4-16 Spreader Bar and Single Point Load Bar

The slabs were then subjected to the single point load to observe the behavior of the sealer under load.

The results of the load testing of the slabs are shown in Figures 4-17 and 4-18.

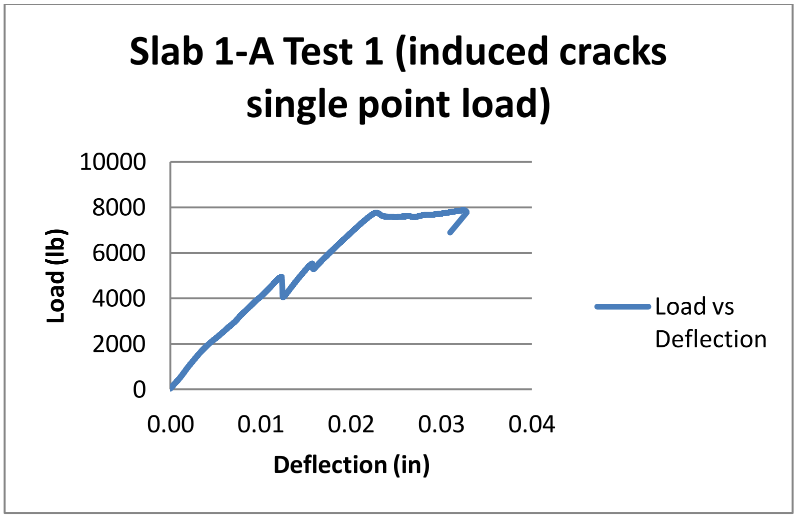


Figure 4-17 Slab 1-A Single Point Load Test

Maximum deflection at 7000 lbf was 0.0203 inches.

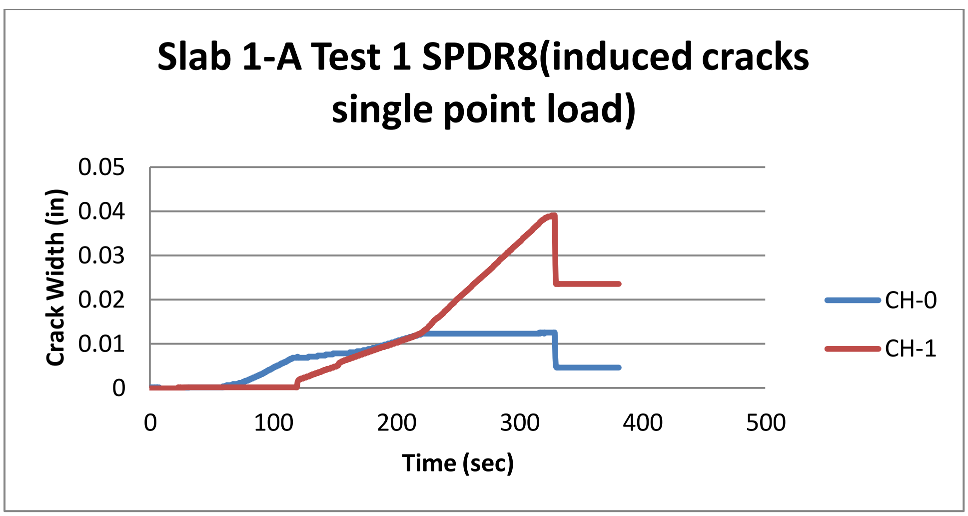


Figure 4-18 Slab 1-A Single Point Load Crack Width

Maximum crack width for channel 0 was 0.01255 inches and for channel 1 was 0.03912 inches. Slab 1-A was tested with a single line load. Channel Zero (0) is the induced center crack (0.01 in) and channel 1 is right side induced crack (0.02 in). Center crack initial cracking occurred at 5000 lbs but other cracks did not appear. From the

moment diagram of a single point load the maximum stress occurs at the center with variable stress to the sides. Therefore, a second test with two point loads was performed, this makes the moment diagram take the shape of a flat top pyramid, whereas the outside crack propagated but no visible sealant debonding as shown in Figure 4-19 which also shows the two point load bar constructed for the test. The test results are shown in Figures 4-20 and 4-21.

Figure 4-19 Slab 1-A Crack Propagation and Double Point Load Plate

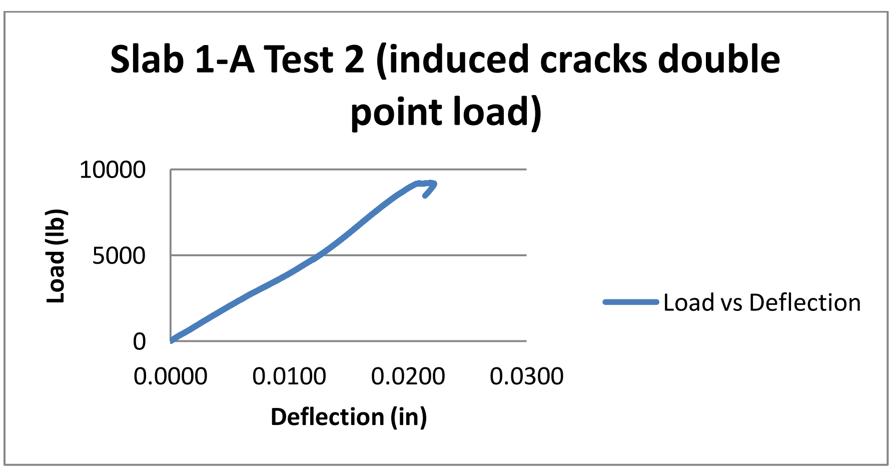


Figure 4-20 Slab 1-A Test 2 (Double Point Load)

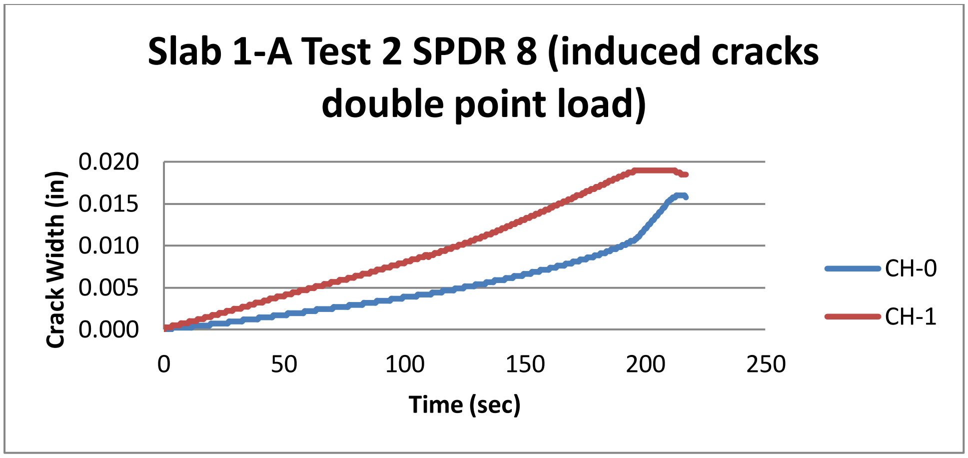


Figure 4-21 Slab 1-A Crack Width (Double Point Load)

Test 2 for Slab 1-A yielded a maximum deflection at 7000 lbf of 0.0163 inches with a maximum crack width of 0.0159 in at channel 0 and 0.0189 in at channel 1.

The remainder of the slabs where tested in the same manner as the the previous slabs making sure not to test past the yielding of the steel as the best performing sealers would be re-tested on their corresponding slabs to check for the resealing bond strength. Below are the corresponding charts for each of the tested slabs with the results of the testing.

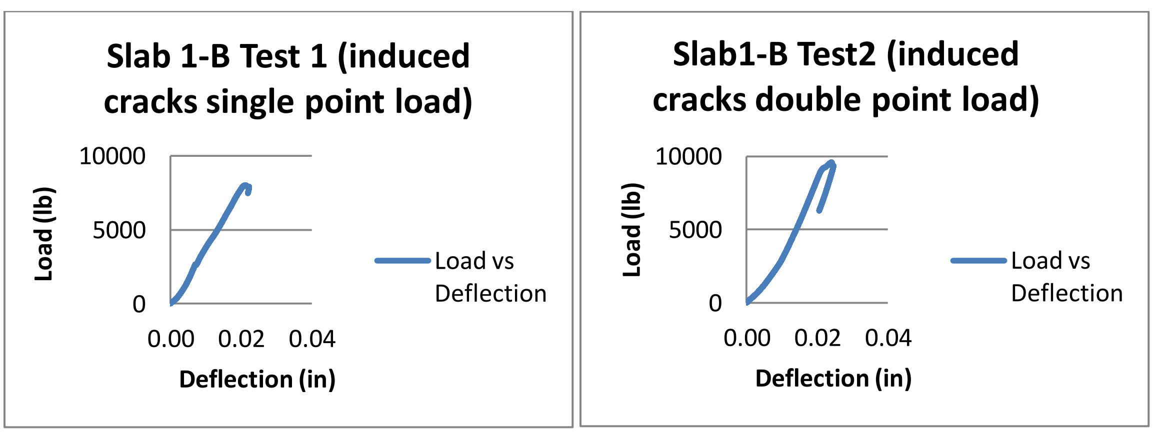


Figure 4-22 Slab 1-B Test Results

Maximum deflection at 7000 lbf was 0.0180 in for test 1 and 0.0176 in for test 2.

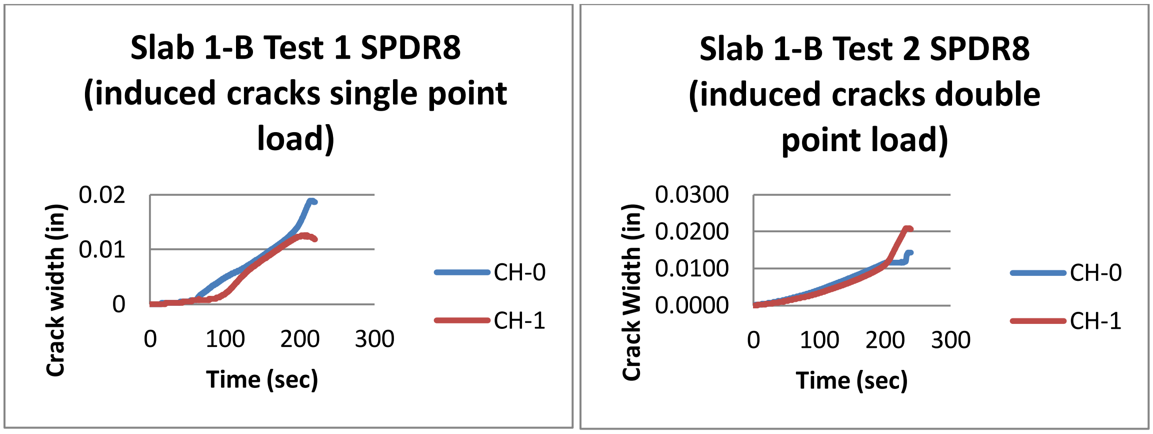


Figure 4-23 Slab 1-B Crack Width Test Results

Maximum crack width for test 1 was 0.0189 in for CH-0 and 0.0125 for CH-1.

Maximum crack width for test 2 was 0.0143 in for CH-0 and 0.0209 in for CH-1.

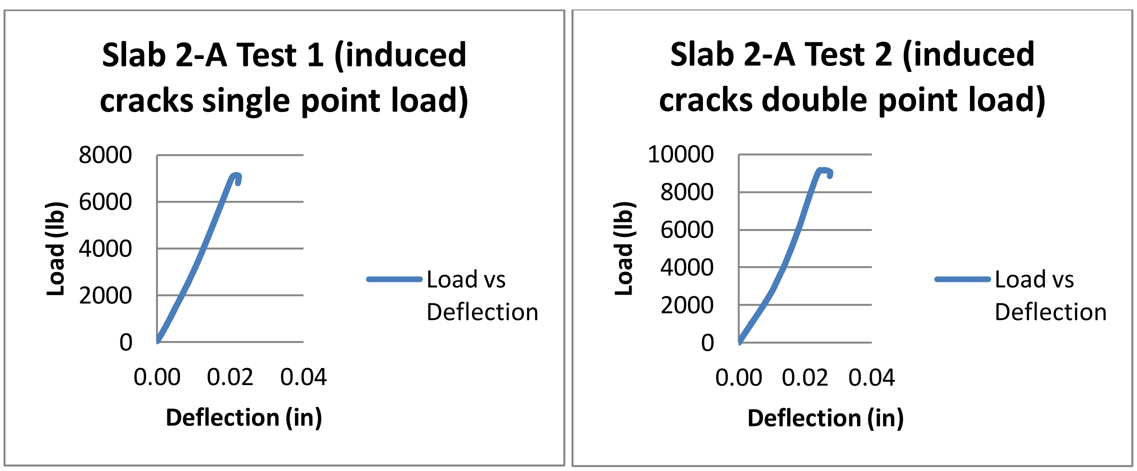


Figure 4-24 Slab 2-A Load Test Results

Maximum deflection at 7000 lbf for test 1 was 0.0203 in and 0.0197 in for test 2.

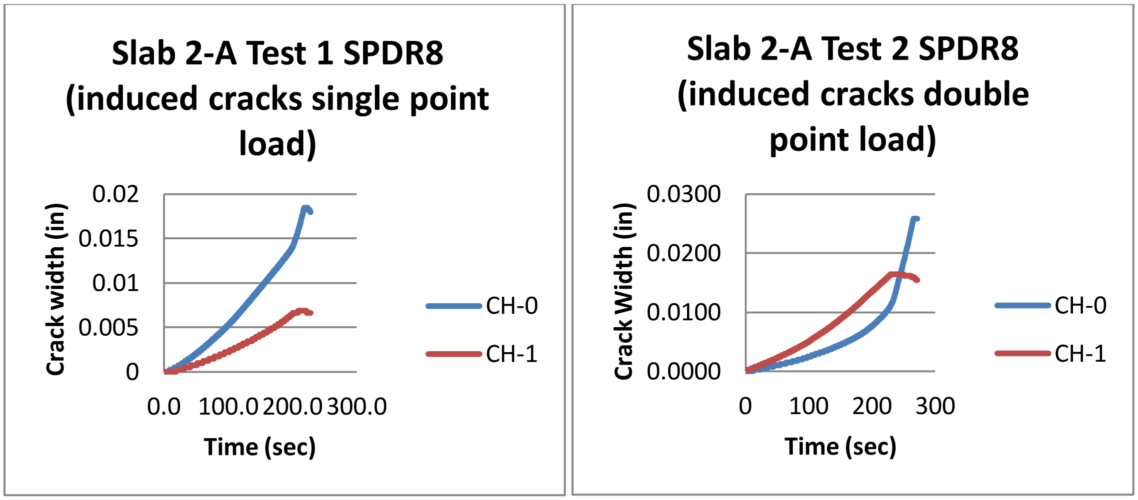


Figure 4-25 Slab 2-A Crack Width Test Results

Maximum crack width for test 1 was 0.0185 in for CH-0 and 0.0069 in for CH-1, for test 2 the maximum crack width was 0.0258 in for CH-0 and 0.0165 in for CH-1.

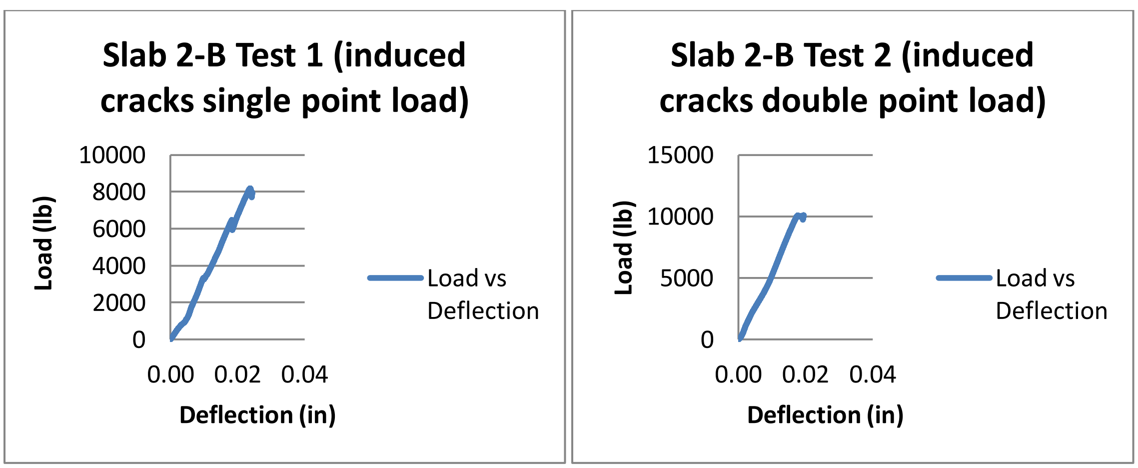


Figure 4-26 Slab 2-B Load Test Results

Maximum deflection at 7000 lbf was 0.0207 in for test 1 and 0.0125 in for test 2.

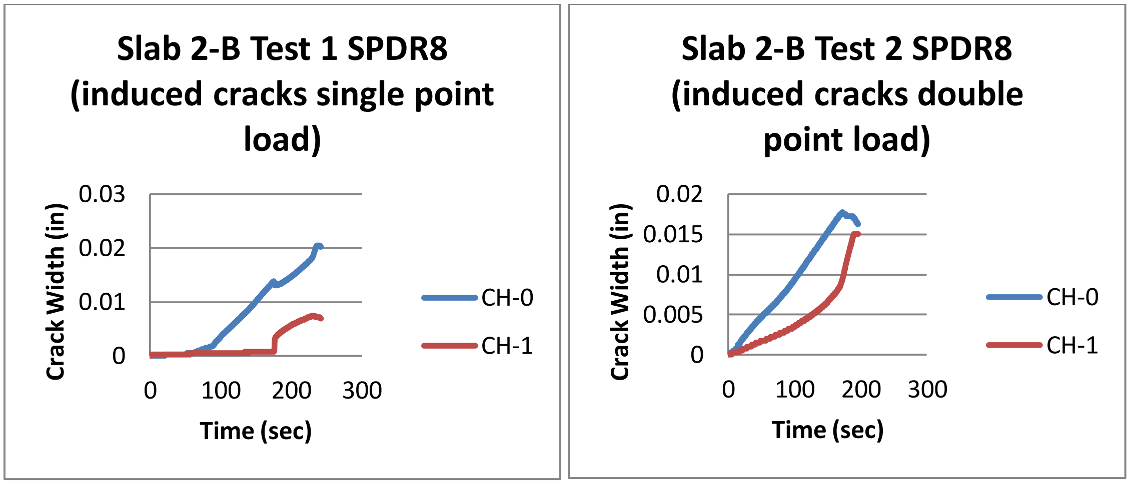


Figure 4-27 Slab 2-B Crack Width Test Results

Maximum crack width for test 1 was 0.0204 in for CH-0 and 0.0074 in for CH-1 for test 2 the maximum crack width was 0.0177 in for CH-0 and 0.0150 for CH-1.

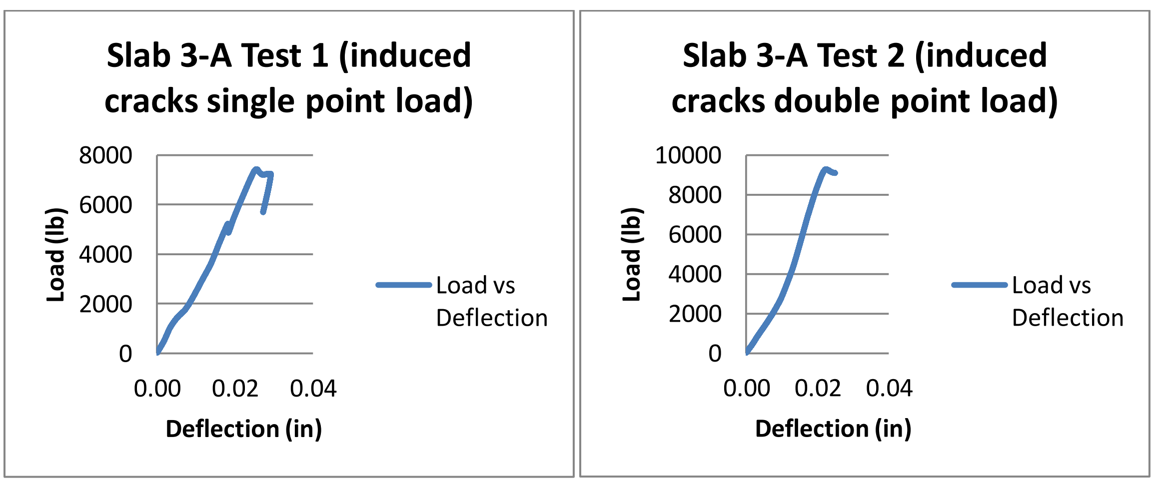


Figure 4-28 Slab 3-A Load Test Results

The maximum deflection at 7000 lbf was 0.0239 in for test 1 and 0.0175 in for test 2.

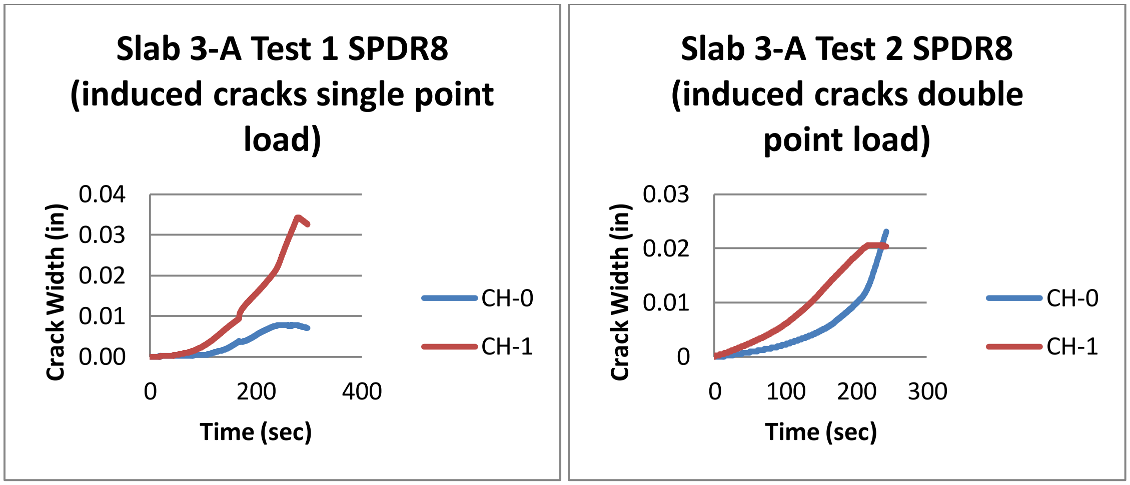


Figure 4-29 Slab 3-A Crack Width Test Results

Maximum crack width for test 1 was 0.0079 in for CH-0 and 0.0342 in for CH-1, for test 2 the maximum crack width was 0.0231 in for CH-0 and 0.0205 in for CH-1.

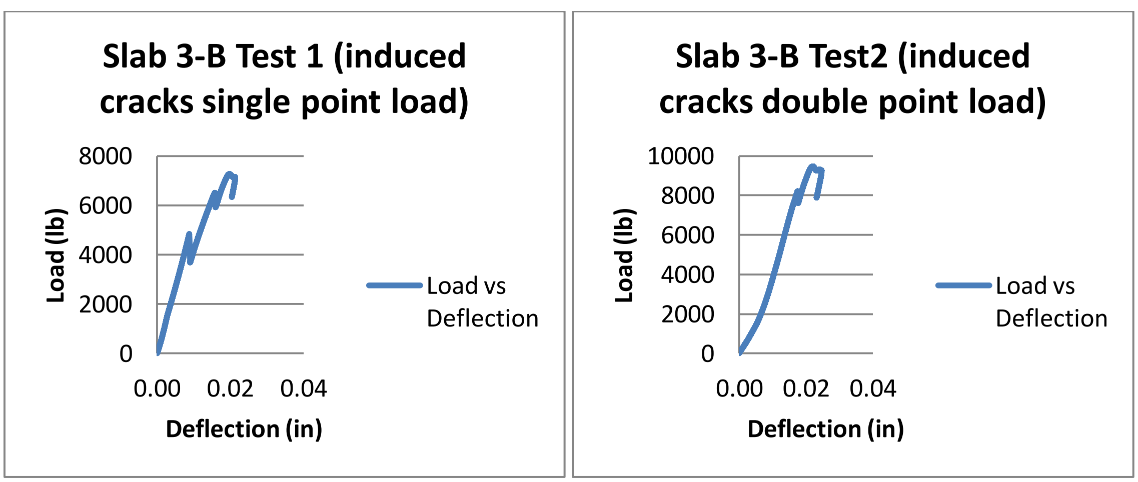


Figure 4-30 Slab 3-B Load Test Results

Maximum deflection at 7000 lbf was 0.0185 in for test 1 and 0.0151 in for test 2.

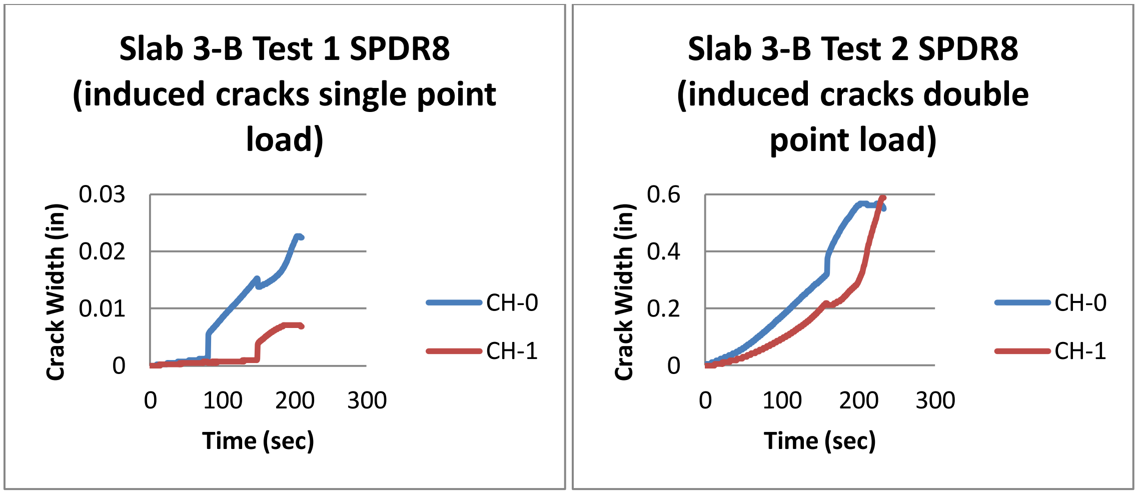


Figure 4-31 Slab 3-B Crack Width Test Results

Maximum crack width for test 1 was 0.0226 in for CH-0 and 0.0071 for CH-1, for test 2 the maximum crack width was 0.0224 in for CH-0 and 0.0231 in for CH-1.

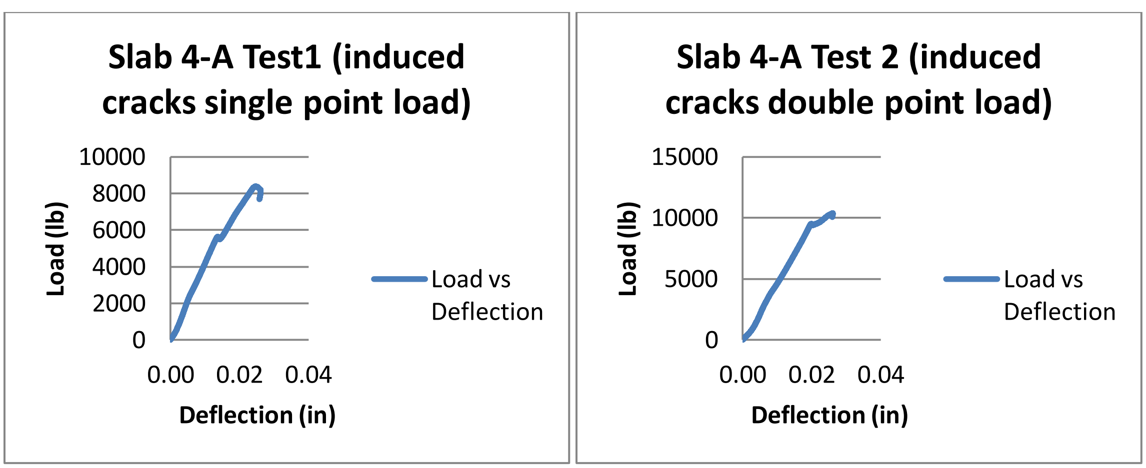


Figure 4-32 Slab 4-A Load Test Results

Maximum deflection at 7000 lbf was 0.0192 in for test 1 and 0.0149 in for test 2.

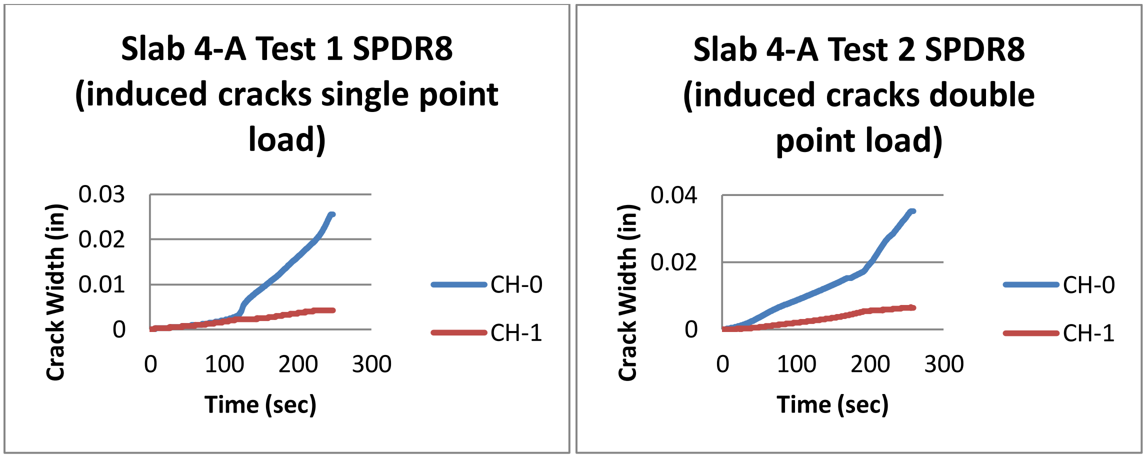


Figure 4-33 Slab 4-A Crack Width Test Results

Maximum crack width for test 1 was 0.0256 in for CH-0 and 0.0042 in for CH-1, for test 2 the maximum crack width was 0.0352 in for CH-0 and 0.0066 in for CH-1.

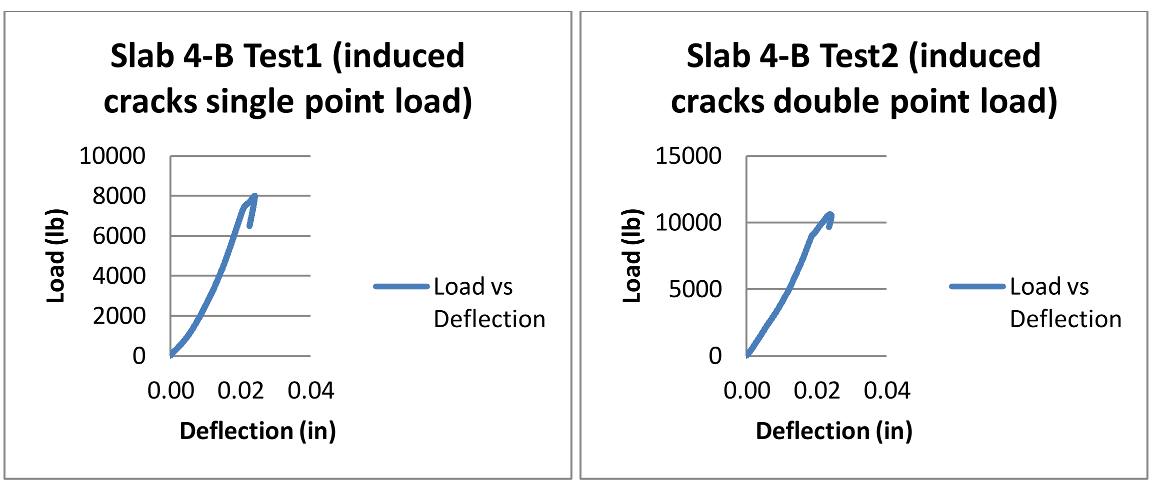


Figure 4-34 Slab 4-B Load Test Results

Maximum deflection at 7000 lbf was 0.0200 in for test 1 and 0.0156 in for test 2.

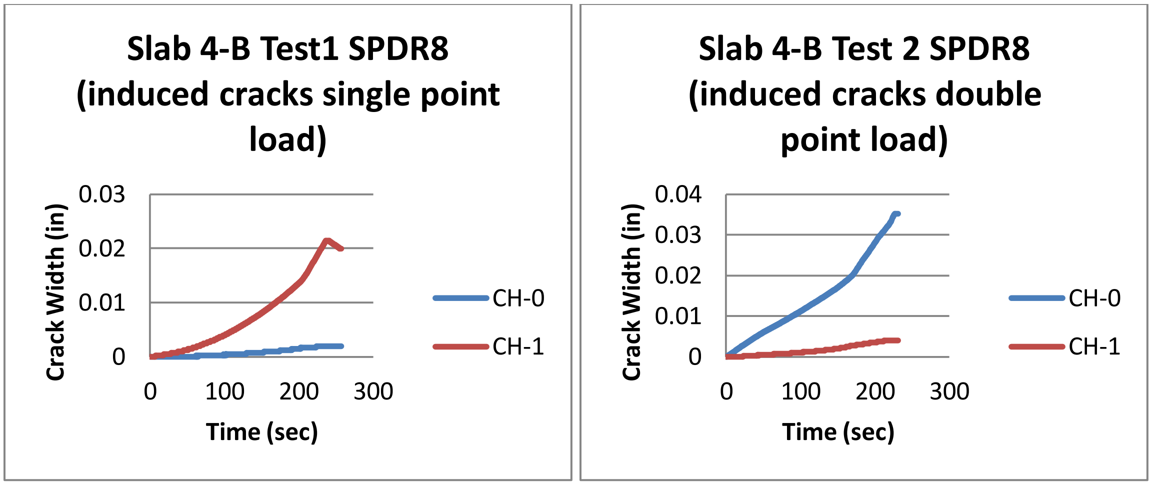


Figure 4-35 Slab 4-B Crack Width Test Results

Maximum crack width for test 1 was 0.0019 in for CH-0 and 0.0214 in for CH-1, for test 2 the maximum crack width was 0.0352 in for CH-0 and 0.0039 in for CH-1. The load test results and the crack/bond condition results are shown in appendix C. In Table 4-8 below are the results of the bond strength of the sealers for each of the induced cracks.

Table 4-8 Slab Test Table

SAMPLE Slab #	Load to Crack Full cracks (Lbf)		Load to crack Half L Crack (Lbf)
	0.01 in	0.02 in	0.01 in
1-A	5525	4946	7704
1-B	6521	2671	7997
2-A	5911	4477	7872
2-B	6483	3347	8100
3-A	5918	2238	7415
3-B	6465	4842	7203
4-A	7300	5641	8380
4-B	7615	5917	8007

From the results, the top three sealers were re-applied to their proper slabs with the top performing sealer being applied to the control slab to simulate actual deck cracking

repair as the control slab had its cracks created by the load test and not induced by the blades, below are the charts for the top three sealers.

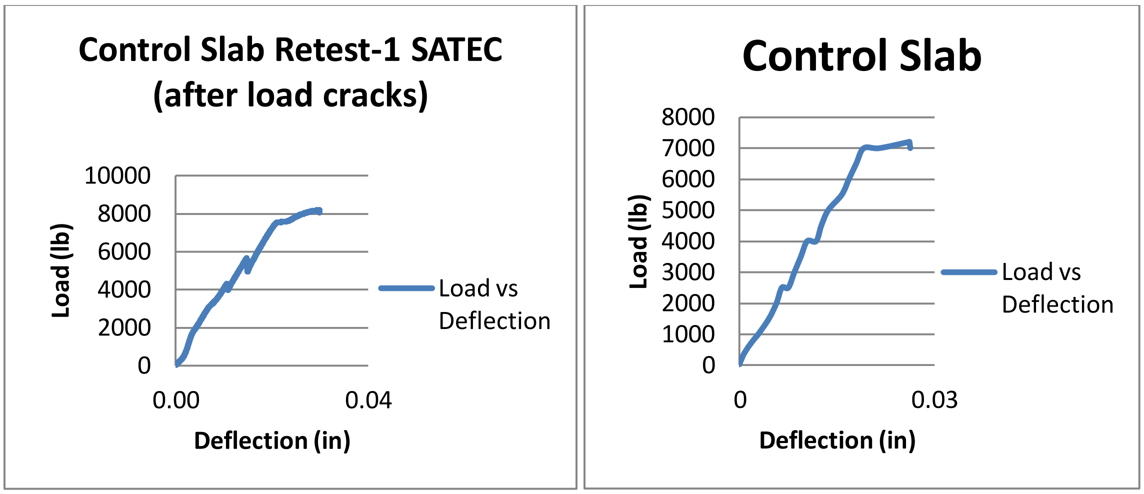


Figure 4-36 Control Slab Retest Result Comparison

The retest chart shows the result of the control slab (no cracks induced) with the three-part HMWM sealer applied after load testing with the chart on the right being the original test of the control slab with no sealer applied.

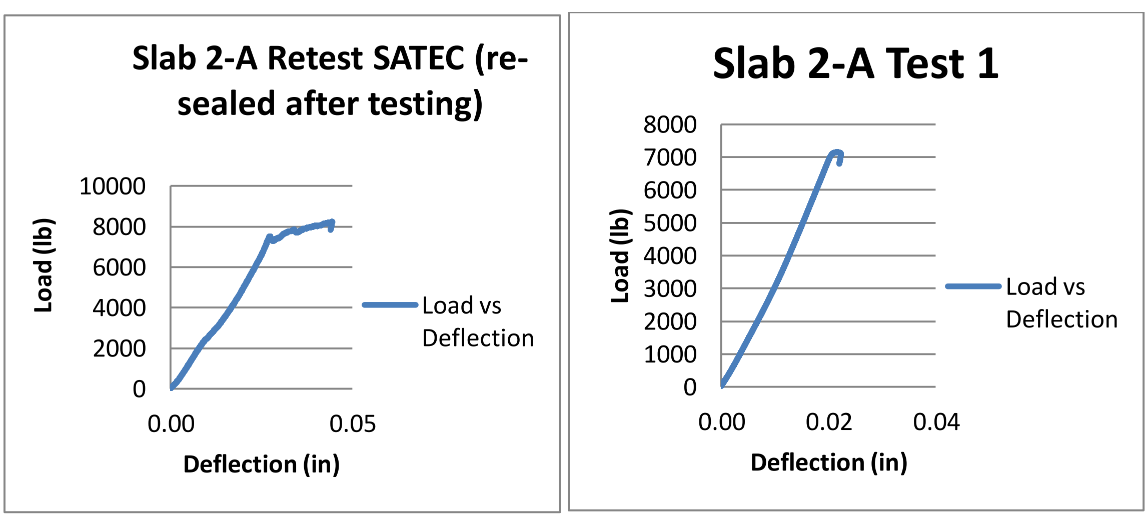


Figure 4-37 Slab 2-A Retest Result Comparison

This is the second best performing sealer an MMA re-applied to slab 2-A, with the reapplication of the sealer on the left chart and the original application on the right chart.

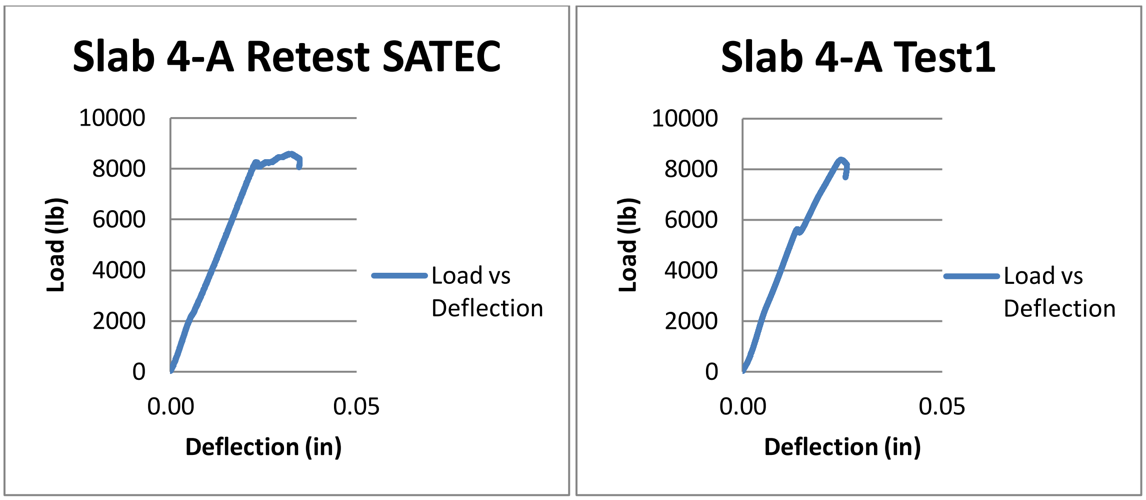


Figure 4-38 Slab 4-A Retest Result Comparison

This is the third best performing sealer an epoxy with the reapplication results in the left chart and the original application on the right chart.

Below are the charts of the second set of slabs cast for the purpose of verifying the finite element analysis that was performed.

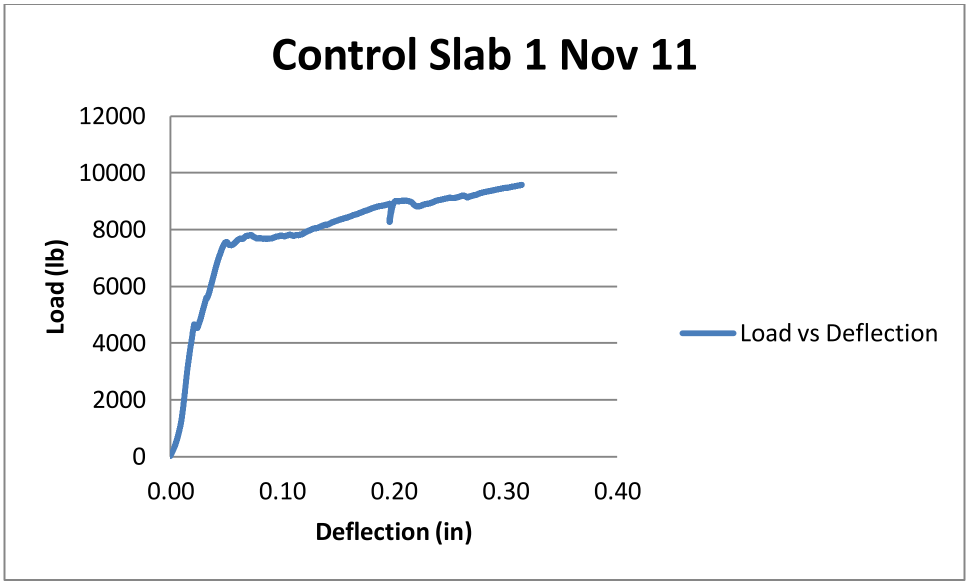


Figure 4-39 Control Slab 1 Load vs Deflection Results

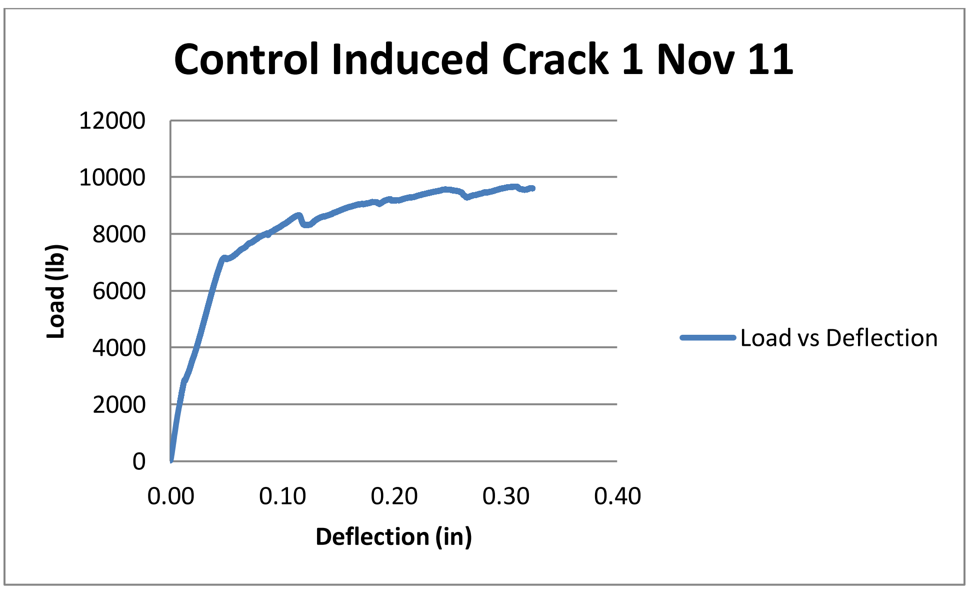


Figure 4-40 CIC 1 Slab Load vs Deflection Results

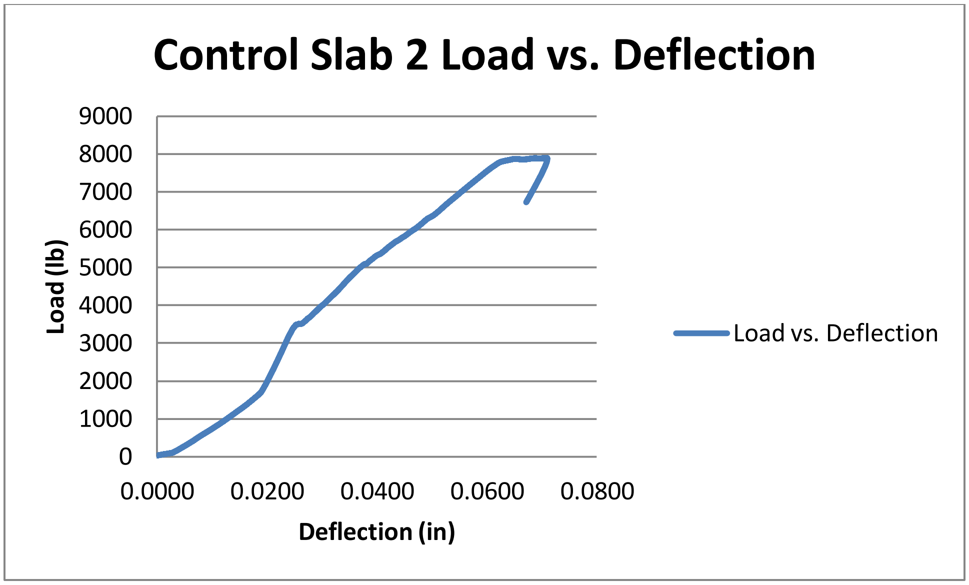


Figure 4-41 Control Slab 2 Load vs Deflection Results

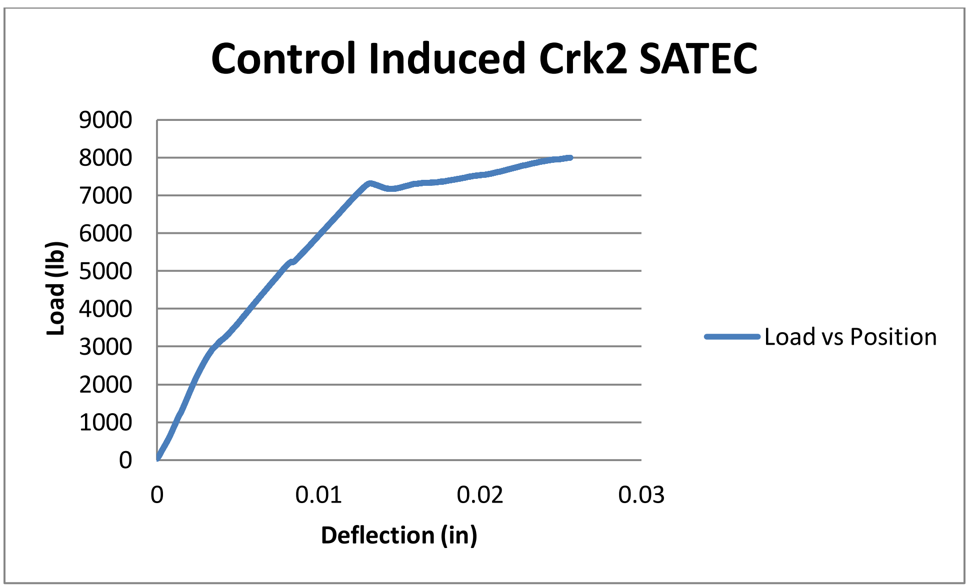


Figure 4-42 CIC 2 Slab Load vs Deflection Results

A Finite Element study was also performed on the slab models as verification to the findings from the lab testing, as shown in Figures 4-43 and 4-44. Table 4-9 and figure 4-45 show the model results.

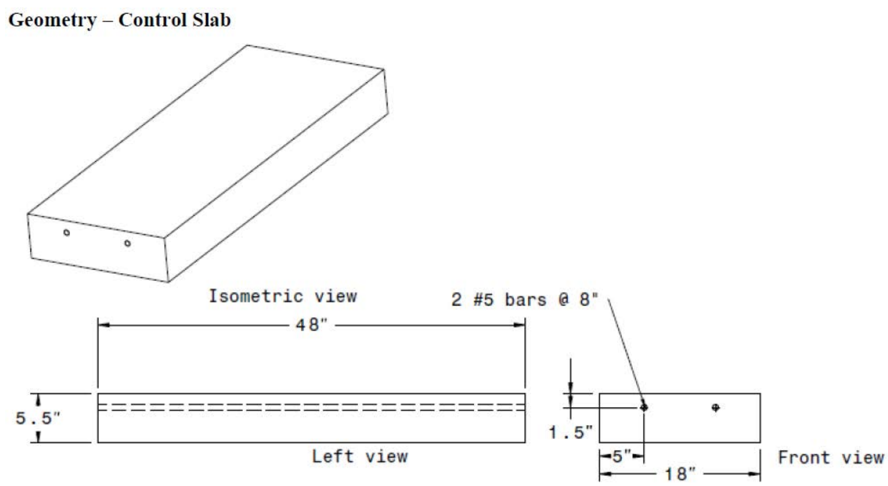
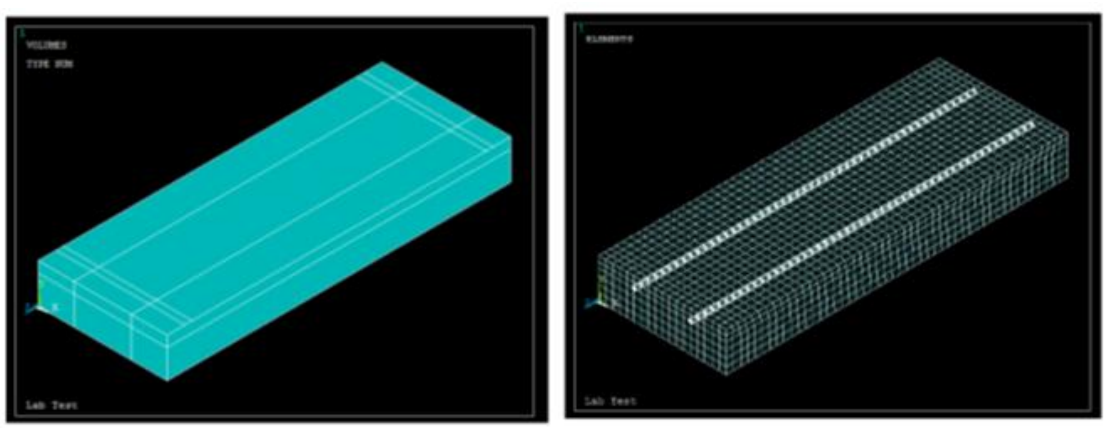


Figure 4-43 Slab Details



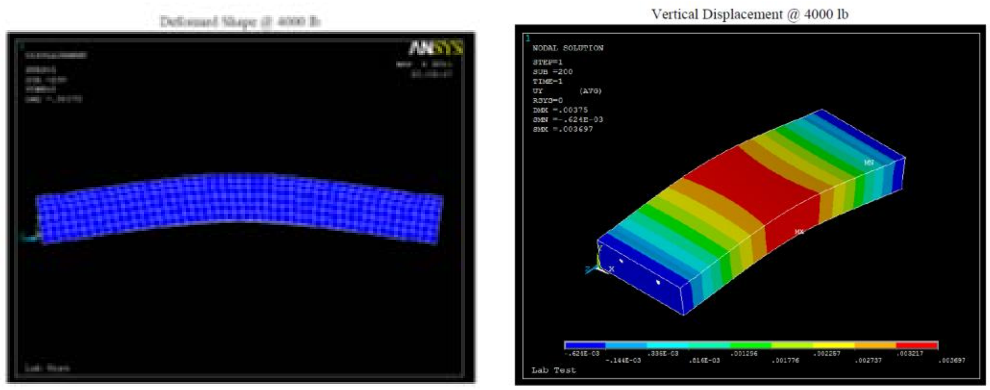
(a) ANSYS model (b) Mesh

Figure 4-44 Finite Element Model of Slab

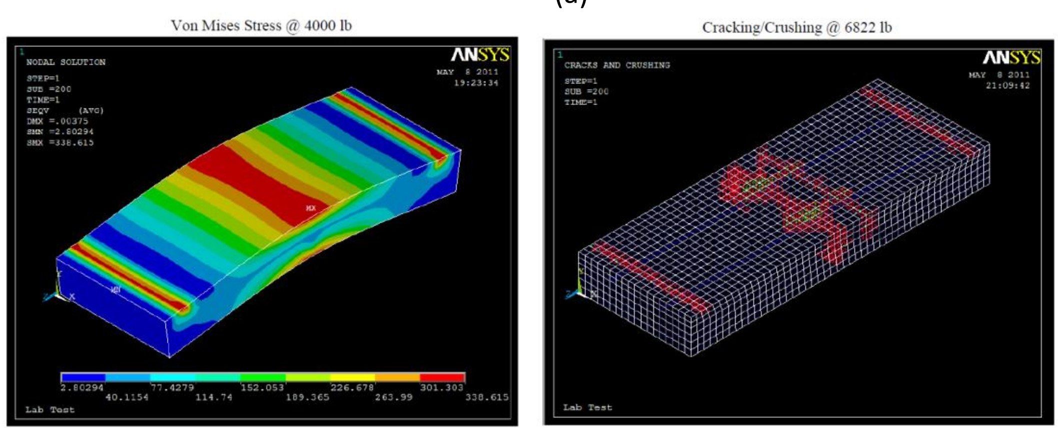
Table 4-9 Model Test Results

Results

Simulation Results					
	Load		Max Vertical Deflection	Max Equivalent Von Mises Stress	Cracking
	lb/in^2	*lb*	*in*	*lb/in*^{2}	
1	55.56	1000	0.00092429	84.661	N
2	111.11	2000	0.0018484	169.31	N
3	222.22	4000	0.0036968	338.62	N
4	333.33	6000	0.0067324	587.89	N
5	379.00	6822	0.033627	1912.2	Y

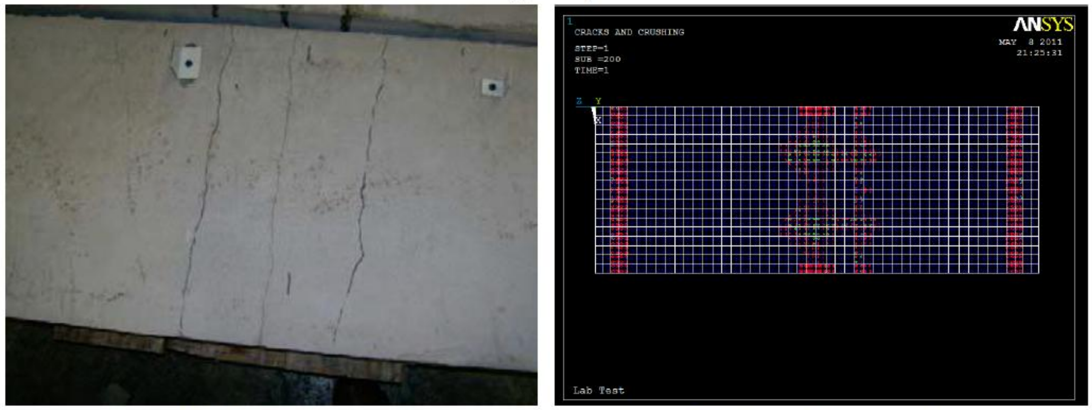


(a)



(b)

Cracking Comparison



Control Slab Cracking ANSYS Simulation Cracking

(c)

Figure 4-45 FE Model Results

Cracking is shown with a circle outline in the plane of the crack, and crushing is shown with an octahedron outline. If the crack has opened and then closed, the circle outline will have an X through it. Each integration point can crack in up to three different planes. The first crack at an integration point is shown with a red circle outline, the second crack with a green outline, and the third crack with a blue outline.

5. SUMMARY AND CONCLUSIONS

5.1 Summary

The objective of this project was to investigate the different kinds of deck sealers available on the market and test such sealers for compatibility with the FDOT's QPL 413. Evidence from various DOT surveys, and field observations was presented to illustrate that bridge decks are in need of repair and that there is a need for a larger group of qualified products to repair those bridge decks. The major forms of deterioration associated with bridge decks, including cracking, surface spalling, surface scaling, and delamination, were described. Of which, a literature review indicates that transverse cracking, due to the longitudinal tensile stresses in the deck exceeding its tensile capacity, is the most prominent form of bridge deck deterioration.

Included in the study were several Florida bridges showing various amount of deterioration and cracking of the bridge deck. There was a field test performed to test the five products selected from the review of properties and those products were then tested again in the laboratory. The results of the testing showed that the three-part HMWM performed the best for cracks less than 0.019 inches with the Epoxy product performing the best for cracks equal to and larger than 0.02 inches.

5.2 Conclusions and Recommendations

A literature review has shown that there has been a lot of research on bridge deck cracking but research on the sealers used to repair them has been lacking with only a few manufacturers getting rigorous testing. We endeavored to test the capabilities of a larger group of sealers in the Florida climate and that would meet the requirements of the FDOT.

1. All of the tested sealers performed well. The sealed slabs performed very close to the control slab obtaining an average of 7845 pounds of force before steel yielding.
2. The three-part HMWM performed best for cracks <0.020 inches wide and the Epoxy was the best performer for cracks ≥ 0.020 inches wide.
3. Sealer debonding was minimal on the single point load.
4. Performance of sealers complied with the Qualifying Products List parameters and the ones that were outside were within the recommendations of other researchers.
5. The concrete compressive strength has an important contribution to cracking of bridge decks. Different conclusions can get drawn depending on the type of applied load. A moderate compressive strength concrete for bridge decks should be considered on steel girder bridge designs. The use of a compressive strength of 5000-6000 psi is recommended as transverse cracking develops exponentially at higher compressive strengths.

6. The stiffness of the bridge deck is also important and affects the behavior. The thicker the deck, the lower the stress is and more chance for fewer cracks. Based on this study a deck thickness of more than 7 inches is recommended.
7. The deflection limit given by AASHTO affects the likelihood of cracking and should be revised to account for higher numbers found during the research.
8. There is very little research on re-cracking of sealers and performance after reapplication which prompts further investigation and second phase of research.
9. More research is needed in the area of crack behavior during the loading phase and the reaction of the sealer to crack opening and closing.

APPENDIX A

Literature Review

The earliest noted study conducted by the Portland Cement Association, the Bureau of Public Roads, and ten state highway departments, and was released in 1970. The purpose of the study was to determine concrete bridge deck durability problems, causes of the types of deterioration, methods to improve durability, and methods to inhibit existing deterioration.

In this study transverse cracking was observed to be the most common type of cracking. Older decks and longer spans showed more transverse cracking, and continuous span bridges and steel girders appeared to exacerbate transverse cracking.

The three important factors cited by the study include:

1. Restraint from the girders on the early and long term shrinkage of the deck
2. Influence of top slab reinforcement as a source of internal restraint
3. Internal restraint of the concrete due to differential drying shrinkage

The researchers concluded that vibration characteristics were not a factor in the deterioration of bridge decks. Restraint to thermal variations was also believed to contribute to cracking. Recommendations by the Portland Cement Association include:

1. Limit slump to 2 in. ± 0.5 in.

2. Maintain the water/cement ratio less than 0.48.

3. Use large sized aggregates.

4. Reduce bleeding by having a smooth grading curve and test mixes for bleeding.

5. Select aggregates with low shrinkage.

6. Avoid placement temperatures over 80° F and consider nighttime deck placement.

7. Provide 1.5 in. minimum concrete cover for top mat reinforcement.

8. Consider increasing the amount of longitudinal reinforcement.

In a study conducted for the Pennsylvania Department of Transportation, Cady et al. surveyed four year old bridge decks in Pennsylvania to investigate the extent and causes of concrete bridge deck deterioration. The researchers found transverse cracks in 60% of all spans and 71% of all bridges. They concluded that:

1. Decks constructed with stay-in-place forms exhibited much less cracking than those built with removable forms.
2. The transverse crack intensity (total length of cracks per 100 ft^2) increased as the span length increased.
3. Superstructure type had a significant effect on the amount of cracking observed. Steel bridges had more cracking than prestressed concrete bridges
4. Cracking is more prevalent on continuous spans than simple spans.

5. Construction practices were the single most influential variable in the extent of cracking observed in bridge decks
6. The use of retarder is not an important factor.

Purvis et al. assessed bridges in Pennsylvania through 99 field surveys and 12 in-depth surveys to determine the causes of transverse cracking. These surveys included crack mapping, crack width measurements, rebar location and depth surveys, concrete coring, and construction records. An important finding made by the researchers was that the transverse cracks intersected coarse aggregate particles; this indicates that transverse cracking occurs in hardened concrete rather than plastic concrete. Recommendations and/or conclusions from this study include:

1. Use the largest possible aggregate size to reduce deck cracking.
2. Deck cracking increases with increased water/cement ratio.
3. Use of Type II cement reduces cracking.
4. Limit the cement content to 725 lb/yd^3.
5. Girder temperature should be maintained at 55 – 75°F in cold weather.
6. Temperature difference between deck and girder should be limited to 22°F for at least 24 hours.

Schmitt and Darwin conducted a study on the effects of different variables on bridge deck cracking, dividing the variables into five categories: material properties, site conditions, construction procedures, design specifications, and traffic and age.

The material properties considered included admixtures, slump, percent volume of water and cement, water content, cement content, water-cement ratio, air content, and compressive strength. Conclusions regarding material properties were:

1. Deck cracking increased with increasing slump, water content, cement content, and water-cement ratio.
2. Cracking increased as the water and cement volumes grew above 27.5%
3. Cracking increased as compressive strength increased corresponding to increasing cement content.
4. Cracking decreased as air content increased, particularly above 6%
5. Use of silica fume may significantly increase cracking if precautions are not taken to prevent plastic cracking.
6. No correlation between deck cracking and the type of admixture was determined.

Site condition factors considered in the study were average air temperature, low air temperature, high air temperature, daily temperature range, relative humidity, average wind velocity, and evaporation. Conclusions regarding site conditions included:

1. There were no discernible correlations concerning cracking and average or low air temperature, relative humidity, average wind velocity, or evaporation rate.
2. Cracking increased significantly as the maximum daily air temperature increased.

3. Cracking increased when the daily temperature range increased.

Construction procedure factors considered in the study were placing sequence, length of placement, and curing. There were no observed relationships between length of placement or type of curing materials and cracking. No correlation between cracking and placing sequence could be determined due to lack of information.

Design factors considered in the study included structure type, deck type, deck thickness, top cover, transverse reinforcing bar size, transverse reinforcing bar spacing, girder end conditions, span length, bridge length, span type, and skew. Conclusions from the study were:

1. Girder end condition appeared to affect deck cracking with fixed girders having more cracks than pinned girders.
2. Cracking increased as transverse reinforcement spacing increased
3. Cracking increased as bar size increased.
4. There were no discernible correlations concerning cracking and top cover, span length, span type, or skew.

Regarding traffic and age, the researchers found that cracking increased with traffic volume and that bridges constructed prior to 1988 exhibited less cracking than bridges constructed after 1988. The increase in cracking in newer bridges was attributed to changes in construction, material properties, and design specifications.

Krauss and Rogalla conducted what is likely the most comprehensive study to date. They surveyed 52 transportation agencies in the United States and Canada to evaluate early age transverse cracking. Over 100,000 bridges were found to have developed early transverse cracks. Analytical studies were also performed using both theoretical and finite element analysis to evaluate the influence of several different parameters on transverse cracking.

The researchers determined that span type, concrete strength, and girder type were the most important design factors influencing transverse cracking. Material properties such as cement content, cement composition, early-age elastic modulus, creep, aggregate type, heat of hydration, and drying shrinkage also influenced deck cracking. Conclusions and/or recommendations include:

1. Recommended clear cover is between 1.5 and 3 in.
2. Recommended thickness of the deck is between 8 and 9 in.
3. Use the largest possible size aggregate and use low shrinkage aggregate.
4. Type II cement reduces cracking in bridge decks.
5. Increasing cement content increases the amount of deck cracking due to higher drying shrinkage, higher temperature rise during hydration, and higher early modulus of elasticity.
6. Increase in deck cracking since the 1970s may coincide with AASHTO's 1973 increase of minimum strength from 3000 psi to 4500 psi; consequently, use of concrete with low early strength is recommended.

7. There is not relationship between slump and cracking tendency.
8. Use of retarders may reduce the rate of early temperature rise and early gain of modulus of elasticity.
9. Silica fume may significantly increase cracking if precautions are not taken to prevent plastic cracking.
10. Concrete placement temperature should be no greater than 80°F and should be 10-20°F cooler than ambient temperature.
11. Special consideration should be taken when evaporation rates are more than 0.2 $lb/ft^2/hr$ for normal concrete and 0.1 $lb/ft^2/hr$ for low w/c ratio concrete.
12. The following procedure is recommended for curing:
    a. Use of fog nozzle water spray in hot weather to cool concrete and to cool the steel and forms immediately ahead of placement—ponding of water on the forms or plastic concrete should not be allowed.
    b. Use of wind breaks and enclosures when the evaporation rates exceed 0.2 $lb/ft^2/hr$ for normal concrete and 0.1 $lb/ft^2/hr$ for low w/c ratio concretes susceptible to plastic cracking.
    c. Application of water mist of monomolecular film immediately after strike-off or early finishing.
    d. Application of white-pigmented curing compound as soon as bleed water diminishes.

e. Application of pre-wetted burlap as soon as concrete resist indentation—the burlap must be kept wet by continuous sprinkling or by covering the burlap with plastic sheeting and periodic sprinkling.
f. Continuation of wet curing for a minimum of 7 days, preferably, 14 days—curing should be extended in cold weather until the concrete has gained adequate strength.

13. Early finishing reduces cracking.
14. SIP forms sometimes increase deck cracking.
15. Decks on steel girders tend to crack more when compared to decks on concrete girders and cracking is more prevalent on continuous spans than on simple spans.
16. Girder restraint and studs cause significant cracking.
17. Increasing deck thickness reduces deck cracking.
18. Increasing the amount of longitudinal reinforcement is recommended (#4 bars at 6 in. spacing).
19. Reducing deck stiffness reduces deck cracking.

Eppers et al. (1998) conducted a field investigation of 72 bridge decks in Minnesota. The researchers determined that design factors most related to transverse cracking were longitudinal restraint, deck thickness, and top transverse bar size. Material factors most affecting transverse cracking were cement content, aggregate type and quantity, and air content. Conclusions and/or recommendations from the study include:

1. Decks constructed on simply supported prestressed girder bridges were in good condition relative to those on continuous steel girder bridges.
2. Diaphragms caused stress concentrations and staggered diaphragms with close spacing resulted in more closely spaced, more narrow cracks.
3. Restraint should be reduced using bridge expansion joints, simply supported spans, increasing girder spacing, and providing fewer shear connectors.
4. Use #5 bars for top transverse reinforcement in concrete bridge decks on steel girders.
5. Reduce the paste volume of the mix designs used.
6. Use lower water-cement ratios
7. Select minimum air content between 5.5% and 6.0%.
8. Maximize coarse and fine aggregate content.
9. Improve curing in the field.

Le, French, and Hajjar Le et al. (1998) performed a parametric study considering bridges with steel and prestressed concrete girders. Among variables considered for steel girder bridges were: end conditions; girder stiffness; locations of cross frames, girder splices, and supplemental reinforcing bars; shrinkage properties; concrete modulus of elasticity; and temperature differential due to heat of hydration. Variables considered for prestressed girder bridges were the times casting relative to the times of both strand release and deck casting, and shrinkage properties of the deck and girders. The researchers came up with the following conclusions and/or recommendations.

1. Prestressed girder bridges with typical construction timelines did not exhibit transverse cracking due to lack of restraint at the end supports and the ability of concrete girders to shrink with the deck over time.
2. Prestressed girder bridges where strand release was delayed resulted in higher tensile stresses in the deck.
3. Decks placed on aged, prestressed girders developed high tensile stresses as a result of differential shrinkage between the girder and the deck.
4. Steel girder bridges exhibited cracking in both the positive and negative moment regions of the bridge deck.
5. Differential shrinkage between the deck and the girders was the main cause of cracking.
6. Ultimate shrinkage did not significantly affect the tensile stresses in the deck due to mitigation of stress through creep of the concrete.
7. End conditions significantly affected the amount of transverse cracking. Cracking was most extensive in the fixed-fixed case and not observed in the simply supported case.
8. Girder stiffness, cross frames, and splices dictated crack locations.
9. Longitudinal restraint should be reduced by using expansion joints on continuous girders, increasing girder spacing, and minimizing shear connector restraint by using fewer rows of smaller-diameter studs.

Frosch et al. (2003), sponsored by the Indiana Department of Transportation, conducted a field study and constructed laboratory specimens to investigate the behavior of transverse cracks. Using these specimens, the researchers could evaluate the effects of differing bridge deck designs on the control of overall shrinkage and the contribution of Stay-in-Place (SIP) steel forms to the formation of transverse cracking. The researchers concluded from the field investigation and laboratory study that:

1. Bridges cast monolithically with a concrete superstructure had the fewest cracks.
2. The restraint of the concrete deck on steel superstructure bridges, through the use of composite action and/or stay-in-place steel forms, induced more transverse cracking than those not incorporating composite action and/or stay-in-place steel forms.
3. Transverse cracks were observed on more bridges with a steel girder superstructure than bridges with a concrete superstructure than bridges with a concrete superstructure. Precast, prestressed concrete superstructure bridges likely behave similar to the monolithic concrete bridges and shrink with the deck instead of restraining the shrinkage when the concrete girders and deck are close in age.
4. Transverse cracking was not influenced by live loads or vibrations caused by live loads.
5. The stiffness of SIP deck pans in restraining shrinkage is not significant

6. Sealing the bottom surface of a bridge deck was found to significantly influence deck shrinkage.
7. Increasing deck thickness decreased the total magnitude of shrinkage
8. As epoxy coating of rebar increased, average and maximum crack widths increased.
9. Wet curing should last at least 7 days.
10. Less shrinkage should be achieved through mix designs.
11. Concrete strength should be minimized.
12. Temperature and shrinkage reinforcement spacing should be limited to 6 inches.
13. SIP forms produce curling that can exacerbate cracking on the top surface.

Xi et al. (2003), sponsored by the Colorado Department of Transportation, reviewed CDOT practices and compared them with the practices of other DOT's for the construction of bridges. A database analysis was conducted on field inspection results in 72 bridges built by CDOT between 1993 and 2002. The database analysis was confirmed with field inspections conducted on nine newly constructed bridge decks that show excessive cracking. Recommendations made by the researchers include:

1. Use Type I or Type II Portland cement for bridge deck construction.
2. Limit cement content to about 470 lb/yd^3 or lower if possible.
3. Use a water/cement ratio of around 0.40.
4. Limit silica fume to 5% by weight of cement to reduce permeability.

5. Use large sized and well-graded aggregate.
6. Use smaller bars for transverse reinforcement.
7. Concrete girders should be preferred for equivalent coefficients of thermal expansion.
8. Consider a minimum deck thickness of 8.5”.
9. Do not cast decks when ambient temperature is below 45° F or over 80° F.
10. Avoid concrete placement when the evaporation rate is above 0.20 $lb/ft^2/hr$ for normal concrete and 0.10 $lb/ft^2/hr$ for low water/cement ratios.

Saad et al. (2003), sponsored by the New Jersey Department of Transportation surveyed 24 bridges in New Jersey built after 1994. Based on the surveys and design and construction documents a database was developed. Statistical analysis of the database was conducted to identify major factors causing transverse deck cracking. A narrow list of factors was also investigated using finite element analysis. Recommendations made by the researchers include:

1. Specify an upper limit on concrete strength and use low early strength concrete when possible.
2. Minimize the ratio of girder/deck stiffness through changes in deck thickness, girder spacing, and girder moment of inertia.
3. Increase the deflection limits to employ a more flexible superstructure.

4. Uniform reinforcement meshes on top and bottom are recommended to control cracking. Increasing the volume of reinforcement above code requirement does not have an effect on cracking.
5. Reduce cement content to 650-660 lb/yd^3 and consider using fly ash.
6. Use Type II cement for bridge deck construction.
7. Limit the water/cement ratio to 0.4-0.45.
8. Maximize the aggregate content and use the largest possible aggregate size.
9. Employ the following pouring sequence:
    a. Pour complete deck at one time whenever feasible within the limitation of the maximum placement length based on drying shrinkage consideration.
    b. If multiple placements must be made and the bridge is composed of simple spans, then place each span in one placement.
    c. If bridge is simple span but cannot be placed in a single placement, divide the deck longitudinally and make two placements.
    d. If the bridge is simple span and single placement cannot be made over the full span length, then place the center of span segment first and make this placement as large as possible.
    e. If multiple placements must be made and the bridge is continuous span, then place concrete in the center of positive moment region first and observe a 72 hour delay between placements.

    f. When deck construction joints are created, require priming existing interfaced surfaces with a primer/bonding agent prior to placement of new concrete.

10. Wet cure for at least 7 days; consider 14 day wet cure when possible.

APPENDIX B

Equations and Details of Theory

Concrete Initial and Final Set

This is the equation used to determine the initial and final set time of the slabs for the placement of the blades to induce the cracks for the test of sealers.

$$t_e\,(T_r\,) = \sum_0^t \exp(\tfrac{\mathrm{E}}{\mathrm{R}}(\tfrac{1}{273+T_r} - \tfrac{1}{273+T_c})) * \Delta t \qquad (1)$$

Where, $t_e\,(T_r\,)$ = equivalent age at reference curing temperature (hours),

Δt = chronological time interval (hours),

T_c = average concrete temperature during the time interval, Δt (°C),

T_r = constant reference temperature (°C),

E = activation energy (J/mol), and

R = universal gas constant (8.3144 J/mol/K).

$$\propto (t_e) = \propto_u * \exp\,(-[\tfrac{\tau}{t_e}]^{\beta}) \qquad (2)$$

where, $\propto (t_e)$ = the degree of hydration at equivalent age, t_e,

τ = hydration time parameter (hours),

β = hydration shape parameter, and

$\propto_u$ = ultimate degree of hydration.

ASTM C 403 Initial set: α_i = 0.15× (w/cm) (3)

ASTM C 403 Final set: α_f = 0.26× (w/cm) (4)

where, α_i = degree of hydration at initial set

α_f = degree of hydration at final set, and

w/cm = water-cementitious material ratio

Early-Age Cracking

This section presents some of the theory used to develop the spreadsheet used to obtain the early-age cracking charts. Temperature development in concrete due to hydration and ambient temperature conditions can be determined from the general differential equation for heat transfer.

$$k * \frac{(d^2T)}{dx^2} + Q_h\ (t_tT) = \rho c_p\ \frac{dT}{dt}$$

Where,

k = thermal conductivity (W/m.C),

ρ = density (kg/cu. m),

c_p = specific heat (J/kg .C),

Q_h = generated heat from the hydration process and external sources (W/m^3),

T = temperature (C), and

t = time (s).

In concrete placed under field conditions, heat will be transferred to and from the surroundings, and the temperature development in the concrete structure is

determined by the balance between heat generation in the concrete and heat exchange with the environment.

Predicting the Thermal Coefficient of Expansion (TCE)

One of the keys to characterizing the effects of thermal properties on a young concrete pavement structure is the accounting of thermal movements in relation to the TCE. Accurate values of the TCE are needed to predict potential thermally induced movements of the concrete pavement. The TCE of early-aged concrete is a function of both the concrete age and relative humidity.

Contrary to the TCE of the cement paste, the TCE for the aggregate is independent of the concrete age. Therefore, the TCE of the cement paste typically governs the overall expansion of the concrete mix during the hardening process. It is also noted that the TCE of concrete in a hardened state is typically larger than the aggregate TCE. Past research has indicated that the TCE of hardened concrete can be estimated from the volumes of coarse aggregate and mortar.

Shrinkage Prediction

The two accepted models of prediction drying shrinkage are the Bazant and Panula, with modifications by the RILEM B_3 model. The model for autogenous shrinkage was developed by Jonasson and Hedlund. Although both models account for drying and autogenous shrinkage, the model by Bazant and Panula has been calibrated for concretes with water-to-cement ratios above 0.40 while the model by Jonasson and

Hedlund has been specifically developed to predict autogenous shrinkage in concrete with water-to-cement ratios below 0.40.

Autogenous shrinkage is defined by the Japanese committee on autogenous shrinkage as, “The macroscopic volume reduction of cementitious materials when cement hydrates after initial setting. Autogenous shrinkage does not include the volume change due to loss or ingress of substances, temperature variation, the application of an external force and restraint”. The magnitude of autogenous shrinkage has been found to depend on the water to cementitious materials ratio (w/cm) in the concrete. As the w/cm ratio gets lower the greater the importance of autogenous shrinkage, as compared to drying shrinkage.

Temperature and moisture in young concrete are a function of the heat and moisture transport characteristics of concrete, curing conditions, and properties of adjacent materials such as forms and support. The hydration characteristics for a given concrete mix depend primarily on the amount and properties of the cement and admixtures used (e.g. a cement type III will generate a higher heat of hydration and at a higher rate than a cement type I or a cement type I + Fly Ash). The primary factors that influence hydration are the chemistry of the cement, the cement grind, and the presence of admixtures.

Prediction of Portland Cement Concrete (PCC) temperature can be used to predict development of mechanical properties taking into account the temperature-maturity properties of concrete. One of the mechanical properties required for the determination

of stress is the concrete TCE. The concrete TCE is a measure of the amount of free contraction or expansion in the concrete for a given change in temperature. Other properties of interest include the modulus of elasticity and tensile strength of the concrete[3]. In addition, it is known that while keeping the concrete under a constant level of stress, the deformation in the concrete tends to increase with time due to the creep- relaxation characteristics of the concrete. Due to this situation, the elastic modulus has to be adjusted to take into account this behavior to properly characterize the stresses in the concrete.

APPENDIX C

Industry Participants and Result Charts

Table C-1 Industry Participants

Sealant	Company	Description
Sealate T-70	Pilgrim Permocoat, Inc.	Methacrylate Bond Strength 615 psi, Tensile Elongation 3-5% Viscosity <20 cps, Flash Point >210F Pot Life 70F: 25-40 min, Tack Free 70F: 4-7 hrs
Sealate T-70 MX-30	Pilgrim Permocoat, Inc.	Methacrylate Bond Strength 615 psi, Tensile Elongation 30% Viscosity <25 cps, Flash Point >200F Pot Life 70F: 40-60 min, Tack Free 70F: 5-8 hrs
Sikadur 55 SLV	Sika Costal Construction	Is a 2-Component, 100% solids, Moisture-tolerant, epoxy crack healer/ Penetrating sealer Bond Strength 14 days – 2,500psi Tensile Strength 7, 100 psi, Elongation 10% Viscosity 105 cps, Flash Point N/A Pot Life 20 min, Tack Free 73F: 6hrs, 90F: 2.5 hrs
Duraguard 401	ChemMasters, Inc 300 Edwards Street Madison, Ohio 44057 (440)428-2105	Methacrylate Is a 3 component, low viscosity, solvent free, high molecular weight methacrylate penetrating sealer and crack healer Tensile Strength 2,800 psi, Elongation 40-50% Viscosity 5-20 cps, Flash Point >200F Pot Life 45 min, Tack Free up to 6 hrs
Epoxeal GS – Structural	BASF Construction Chemicals, LLC 889 Valley Park Dr. Shakopee, MN. 55379	Epoxy Sealer Is a two- component, ultra low viscosity, gravity feed or pressure injected Bond Strength 14 days – 3,450 psi Tensile Strength 7,100 psi, Elongation 2.9% Viscosity 95 cps, Flash Point >200F Pot Life 45 min, Tack Free 70F: 12 hrs, 80F: 6 hrs
Degadur 332	BASF Construction Chemicals, LLC 889 Valley Park Dr. Shakopee, MN. 55379	Methyl Methacrylate (MMA) Is a solvent free, 2 component, 100% reactive resin Tensile Strength 1,200 psi, Elongation 220-300% Viscosity 95 cps, Flash Point 48F Pot Life 25 min, Tack Free 1hr
Degadeck Crack Sealer Plus	BASF Construction Chemicals, LLC 889 Valley Park Dr. Shakopee, MN. 55379	Methacrylate Is a low viscosity, low surface tension, solvent free, penetrating sealer and crack healer Tensile Strength 8,100 psi, Elongation 5.5% Viscosity 5-15 cps, Flash Point 48F Pot Life 15-20 min, Tack Free 1hr
Traffic Guard	BASF Construction	Epoxy Is a rapid-curing, skid-resistant epoxy concrete

EP-35	Chemicals, LLC 889 Valley Park Dr. Shakopee, MN. 55379	overlay system Tensile Strength 2,500 psi, Bond Strength 2,500 psi Elongation 30%, Viscosity 1000-2500 cps Flash Point 200F, Pot Life 15-25 min, Tack Free 2hrs
Degadeck Deck overlay System	BASF Construction Chemicals, LLC 889 Valley Park Dr. Shakopee, MN. 55379	Methacrylate Is a 3 component, reactive resin used as a wearing coarse Tensile Strength 1,290-1,380 psi(Body coat), 2,150 psi (Top coat) Elongation 13%(Body coat), 35% (Top coat) Viscosity N/A, Flash Point 48F, Pot Life N/A Tack Free 1hr
Pro-Poxy LV LM Epoxy	UNITEX - Chemicals, 3101 Gardner Avenue Kansas City, MO. 64120	100% solids, low modulus, highly penetrating epoxy polymer Tensile Strength 1000 psi, Elongation 60%, Viscosity 80 cps, Flash Point>200F, Pot Life 15 min, Tack Free 2hrs

Table C-2 Slab Test Table

SAMPLE	LOAD (kips)	FULL CRACK(0.01")		FULL CRACK(0.02")		HALF CRACK(0.01") Test 1		HALF CRACK(0.01") Test 2		CRACK Cond.	PEAK LOAD (lbf)	
		Width	Bond	Width	Bond	Width	Bond	Width	Bond		Test 1	Test 2
1-A	0		1		1	0	1	0	1	1		
1-A	1		1		1	0	1	0	1	1		
1-A	2		1		1	0.02	1	0.02005	1	1		
1-A	3		1		1	0.03	1	0.03003	1	1		
1-A	4	See Note	2	See Note	1	0.04	1	0.04006	2	2		
1-A	5		2		2	0.05	1	0.06003	3	3		
1-A	6		2		2	0.07	1	0.09002	3	3		
1-A	7		2		3,4	0.08	1	0.1	3	3	7875	
1-A	8		2		4	0.1	1	0.106	3	3		
1-A	9		3		4					3		9219
1-B	0		1		1	0	1	0	1	1		
1-B	1		1		1	0	1	0	1	1		
1-B	2		1		1	0.008	1	0.01	1	1		
1-B	3		2		2	0.009	1	0.02004	2	1		
1-B	4	See Note	2	See Note	2	0.01	2	0.03001	2	1		
1-B	5		2		2	0.01	2	0.03008	2	2		
1-B	6		2		3	0.01002	2	0.04005	2	3		
1-B	7		3		3	0.01002	3	0.05001	3	4		
1-B	8		3		3,4	0.01003	3	0.05008	3	4	8002	
1-B	9		3		4					4		9604
2-A	0		1		1	0	1	0	1	1		
2-A	1		1		1	0.01	2	0.01	1	1		
2-A	2		2		1	0.01006	4	0.01009	1	1		
2-A	3		2		2	0.02003	4	0.02006	1	1		
2-A	4	See Note	2	See Note	2	0.04001	4	0.03003	2	2		
2-A	5		2		3	0.06002	4	0.05	2,3	3		
2-A	6		3		3,4	0.06008	3,4	0.08006	3	3		
2-A	7		3		4	0.09002	3	0.09005	4	4	7153	
2-A	8		3		4	0.101004	3,4	0.101007	4	4		
2-A	9		3		4			0.103005	4	4		9180
2-B	0		1		1	0	1	0	1	1		
2-B	1		1		1	0.01003	1	0.001	1	1		
2-B	2		1		1	0.01007	1	0.002	1	1		
2-B	3		1		2	0.02004	1	0.01	1	1		
2-B	4	See Note	2	See Note	2	0.03006	1	0.03	1	1		
2-B	5		2		2	0.04002	1	0.05002	1	1		
2-B	6		2		2	0.05002	2	0.08001	2	2		
2-B	7		2		2,3	0.07004	2	0.103004	2	2		
2-B	8		3		3	0.103004	2	0.104001	2	2	8196	
2-B	9		3		4	0.106002	2	0.105007	2	2		10093

SAMPLE	LOAD (kips)	CRACK Cond.
CONTROL	0	N/A
CONTROL	1	N/A
CONTROL	2	N/A
CONTROL	3	N/A
CONTROL	4	Micro
CONTROL	5	Micro
CONTROL	6	Center
CONTROL	7	right/left
CONTROL	8	
CONTROL	9	

Bond	Condition
1	normal
2	crack
3	propagation
4	debond

3-A	0		1		1	0	1	0	1	1		
3-A	1		1		1	0.008	1	0.002	1	1		
3-A	2		1		2	0.01002	1	0.004	1	1		
3-A	3		2		2	0.02005	1	0.005	1	1		
3-A	4	See Note	2	See Note	2	0.03002	1	0.01004	1	1		
3-A	5		2		2	0.04003	1	0.02009	1	1		
3-A	6		3		3	0.05005	2	0.04003	2	2		
3-A	7		3		3	0.07003	2	0.05007	2	2	7416	
3-A	8		3		3	0.102004	2	0.07003	2	2		
3-A	9		3		3			0.1006		2		9279
3-B	0		1		1	0	1	0	1	1		
3-B	1		1		1	0.002	1	0.002	1	1		
3-B	2		1		1	0.003	1	0.004	1	1		
3-B	3		1		1,2	0.004	1	0.005	1	1		
3-B	4	See Note	2	See Note	2	0.004	1	0.007	1	1		
3-B	5		2		2	0.006	2	0.009	1	1		
3-B	6		2		2,3	0.01007	2	0.01009	1	1		
3-B	7		3		3	0.02008	3	0.02008	2	2	7265	
3-B	8		3		3,4	0.04004	3	0.03004	3	3		
3-B	9		3		4			0.04008	3	3		9484
4-A	0		1		1	0	1	0	1	1		
4-A	1		1		1	0.002	1	0.003	1	1		
4-A	2		1		1	0.002	1	0.005	1	1		
4-A	3		1		2	0.003	1	0.01003	1	1		
4-A	4	See Note	2	See Note	2	0.003	2	0.01009	1	1		
4-A	5		2		2	0.007	2	0.02001	2	2		
4-A	6		2		3	0.01008	2	0.03002	2	2		
4-A	7		2		3	0.02	2	0.04003	2	2,3		
4-A	8		3		4	0.03	3	0.07003	3	4	8388	
4-A	9		3		4	0.04	3	0.08007	4	4		10357
4-B	0		1		1	0	1	0	1	1		
4-B	1		1		1	0.005	1	0.009	1	1		
4-B	2		1		1	0.006	1	0.01004	1	1		
4-B	3		1		1	0.01002	1	0.02004	1	1		
4-B	4	See Note	1	See Note	1	0.02003	1	0.03003	1	1		
4-B	5		1		2	0.04006	2	0.04005	1	1		
4-B	6		2		2	0.05001	2	0.05003	2	2		
4-B	7		2		2	0.06008	4	0.06008	2	2,3		
4-B	8		3		2	0.07004	4	0.101003	2	3	8009	
4-B	9		3		3			0.102006	3	3		10645

Full cracks are the entire width of the slab or 18 inches long the width of the cracks are 0.01 in.(3mm) and 0.02 in.(6mm)	
Half cracks are 9 inches long and 0.01 in.(3mm) wide	
1-A,B	BASF
2-A,B	C.M
3-A,B	Pilgrim
4-A,B	Unitex
* NOTE: See crack width chart for data	

Table C-3 Slab Crack Width

Test	Slab Num	CH-0 Max Width(mm)	CH-0 Max Width (in)	CH-1 Max Width (mm)	CH-1 Max Width (in)
1	1-A	0.03188	0.01255	0.09938	0.03912
2	1-A	0.40625	0.01599	0.48125	0.01895
1	1-B	0.48125	0.01895	0.31875	0.01255
2	1-B	0.36250	0.01427	0.53125	0.02092
1	2-A	0.46875	0.01846	0.17500	0.00689
2	2-A	0.65625	0.02584	0.41875	0.01649
1	2-B	0.51875	0.02042	0.18750	0.00738
2	2-B	0.45000	0.01772	0.38125	0.01501
1	3-A	0.02000	0.00787	0.86875	0.03420
2	3-A	0.58750	0.02313	0.52500	0.02067
1	3-B	0.57500	0.02264	0.18125	0.00714
2	3-B	0.56875	0.02239	0.58750	0.02313
1	4-A	0.06500	0.02559	0.10625	0.00418
2	4-A	0.89375	0.03519	0.16875	0.00664
1	4-B	0.05000	0.00197	0.54375	0.02141
2	4-B	0.89375	0.03519	0.10000	0.00394

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VITA

Vidal V. Vargas was born . He retired from the U.S Navy in January 31, 2001 with twenty years of honorable service, attaining the rank of Aviation Electrician's Mate First Class. He was the shift supervisor for a crew of twenty-five technicians charged with maintaining eleven aircraft. From 1999 to 2001 managed Command Services office for a command of 1100 personnel, maintaining a budget of $275,000 annually and achieving two fiscal years with zero errors and zero debts.

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