Technical Report Documentation Page

1. Report No. FHWA/TX-11/0-6590-2	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Materials Selection for Concrete Overlays: Final Report		5. Report Date October 2011, Published July 2012 6. Performing Organization Code
7. Author(s) Dong H. Kim, David W. Fowler, Raissa P. Ferron, Manuel M. Trevino, David P. Whitney		8. Performing Organization Report No. 0-6590-2
9. Performing Organization Name and Address Center for Transportation Research The University of Texas at Austin 1616 Guadalupe Street, Suite 4.202 Austin, TX 78701		10. Work Unit No. (TRAIS) 11. Contract or Grant No. 0-6590
12. Sponsoring Agency Name and Address Texas Department of Transportation Research and Technology Implementation Office P.O. Box 5080 Austin, TX 78763-5080		13. Type of Report and Period Covered Technical Report 09/2009 - 08/2011 14. Sponsoring Agency Code

15. Supplementary Notes

Project performed in cooperation with the Texas Department of Transportation and the Federal Highway Administration.

16. Abstract

Concrete overlays have been a rehabilitation method for many years. It has been extensively utilized and studied in other states, but Texas is still at an initial stage of fully implementing the method. The large volume of concrete highways in Texas makes bonded concrete overlays, unbonded concrete overlays, and whitetoppings very viable options. However, there is a lack of educational guidelines for pavement engineers for concrete overlay construction. This research presents the information gathered from literature review, condition survey, and evaluation of existing concrete overlays in Texas. Also, a laboratory research was performed for recommendations for materials selection and construction for concrete overlays. From these, guidelines for materials selection and construction method developed that will assist in future concrete overlays in Texas are presented.

17. Key Words Concrete, Overlay, Materials	18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161; www.ntis.gov.

19. Security Classif. (of report)	20. Security Classif. (of this page)	21. No. of pages	22. Price
Unclassified	Unclassified	210	

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

Materials Selection for Concrete Overlays: The Final Report

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CTR Technical Report:	0-6590-2
Report Date:	October 2011
Project:	0-6590
Project Title:	Materials Selection for Concrete Overlays
Sponsoring Agency:	Texas Department of Transportation
Performing Agency:	Center for Transportation Research at The University of Texas at Austin

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Acknowledgments

The researchers acknowledge the invaluable assistance provided by Clifford Halvorsen, TxDOT project director for Project 0-6590. Also appreciated is the guidance provided by German Claros and Ryan Barborak with TxDOT.

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Chapter 1. Introduction

1.1 Research Background

The infrastructure of the state of Texas is in poor condition, and in the 2011 Texas Infrastructure Report Card Update, the state's roads and bridges were give a grade of D. However, increases in the expected capacity of the state roads will put an enormous demand on the transportation infrastructure. Thus, it is important that roadway constructed repairs are done quickly so that traffic congestion in minimized. Portland cement concrete overlays (herein referred to as concrete overlays) constructed on top of existing concrete pavement or asphalt concrete pavement are cost-effective rehabilitation strategies. Concrete overlays are simply an additional layer of concrete placed on top of existing pavements to increase the load bearing and serviceability capacity. When properly constructed, overlays provide an optimum utilization of the qualities of the existing pavement, by increasing the structural capacity, improving the durability, and providing increased serviceability.

1.2 Research Objectives

The goal of this research was to investigate and recommend materials for concrete overlays and to provide construction guidelines that can produce a good performing concrete overlay. In order to achieve this goal, the following objectives were set:

- Conduct a comprehensive literature review to capture the existing knowledge on concrete overlay usage, materials selection, and performance.
- Perform condition surveys on existing concrete overlays in Texas to gather information regarding durability and conditions surveys and identify the role of material constituents, bonding agents, mixture design, and concrete placement factors that have major influence on the performance of concrete overlays.
- Select candidate materials for our laboratory study, based on our literature review and condition surveys.
- Develop a laboratory process to evaluate overlay mixture proportioning.
- Evaluate data with a performance prediction model.
- Develop guidelines for materials selection.
- Develop guidelines for construction procedures.

1.3 Scope of Study

Based on the literature review and the condition surveys and recommendations from TxDOT personnel, candidate materials were selected and mixture designs were developed. Mixture designs included statistically based variations in varying the cement content, fly ash content, and aggregate ratios. In addition, the effect of different types of fibers was also examined. Tests performed were compressive strength, flexural strength, modulus of elasticity, coefficient of thermal expansion, drying shrinkage, average residual strength for fiber in concrete, and bond strength.

Chapter 2. Literature Review

2.1 Introduction

This chapter presents a comprehensive literature review about the current state of knowledge on the material selection, mixture design/proportioning, and construction for concrete overlays. The task involved gathering information on concrete overlays applications by the Texas Department of Transportation (TxDOT) as well as other states, with special emphasis focused on gathering information about the constituent materials, mixture design/proportioning, and recommended construction methods. The research team also reviewed performance and characteristics of fibers used in concrete overlays.

2.2 Overview of Concrete Overlays

The main purpose of constructing concrete overlays is to optimize/extend the use of the remaining life of the existing pavement by placing an additional layer of concrete above it. By choosing the concrete overlay option, it is possible to expedite construction, reduce cost, increase structural integrity, improve riding quality, and protect structure against deleterious environmental effects.

Concrete overlays are categorized into two types: bonded concrete overlay and unbonded concrete overlay. In bonded concrete overlays, there are ultra-thin and thin whitetoppings and bonded concrete overlays (BCOs). These concrete overlays require bonding between the concrete overlay and the existing pavement. Unbonded concrete overlays include conventional whitetopping, and unbonded concrete overlay (UBCOs). Whitetoppings can be bonded or unbonded depending on their thickness. One other type, partially bonded concrete overlays, is not discussed in this report since it is not used for highway applications in Texas.

2.2.1 Whitetopping

The term "whitetopping" indicates a concrete overlay that is used to resurface an existing asphalt pavement. Whitetoppings are subcategorized by the thickness and the bond conditions.

Purposes and Uses

The purposes of whitetopping are to rehabilitate deteriorating asphalt pavements, to increase load capacity and improve ride quality. Since whitetoppings do not develop typical distresses that are found in asphalt pavements, it is a good alternative to placing an asphalt overlay. Whitetoppings are classified as the following:

- Ultra-thin whitetopping (UTW): This overlay typically consists of a 2-to 4-in.-thick concrete and is used when the existing pavement is considered to be in fair or better condition with minor surface distresses (shoving, rutting, alligator cracking, etc. The overlay relies on existing pavement to carry much of the load, and good bond will promote monolithic behavior. Monolithic behavior reduces flexural stresses in the overly, which can lead to early cracking and failure. UTWs are generally used in light traffic applications.
- Thin whitetopping (TWT): TWT is identical to UTW, but the overlay is thicker (typically around 5 to 8 in.) and is used when the existing pavement is considered to

be more deteriorated than for UTW requirements. The overlay relies on existing pavement to carry some of the load by monolithic behavior through a good bond between overlay and substrate. TWTs are generally used when moderate traffic is present.

- Conventional whitetopping (CWT): CWT is typically 9 in. or thicker and is used when the existing pavement is in a severely deteriorated condition. CWT design assumes an unbonded condition, so the existing pavement is only expected to serve as a subbase. The new overlay will carry the entire traffic load. CWTs are generally used when heavy traffic is anticipated.

Performance Factors

The following factors determine the performance of whitetoppings:

- Effectiveness of bond: For whitetoppings that rely on existing pavement to carry load through bonding, properly achieved bond will promote monolithic behavior. This behavior is crucial in ensuring that the stiffness of the rehabilitated pavement (overlay and existing pavement) will carry the traffic load as one structure.
- Existing pavement condition: Since UTWs and TWTs rely on the existing pavement to assist in carrying the traffic load, the condition of the existing pavement affects the performance of the rehabilitated pavement. Proper repairs or upgrades should be made to the substrate to provide adequate support as required by design.
- Proper joint spacing: If joints are required, proper joint spacing helps to reduce curling stresses and bending stresses due to traffic loads. This is especially true for UTW and TWT because of their thinness.

Common Modes of Failure

The following failure modes are commonly seen in whitetoppings:

- Loss of bond: The bond between the overlay and the existing pavement can be lost due to lack of quality control in surface preparation or placement.
- Rapid transition zone failure: Accelerated deterioration in the transition zones can occur at the interface between asphalt and the concrete overlay. Thicker concrete overlay sections are recommended in these areas [1].

2.2.2 Bonded Concrete Overlay

A bonded concrete overlay (abbreviated "BCO") is a relatively thin concrete layer that is used to resurface an existing concrete pavement. This type of overlay is typically 2 to 4 in. thick and its performance depends on good bond to the existing pavement.

Purposes and Uses

The purpose of the BCO is to rehabilitate deteriorating concrete pavements to increase load capacity and ride quality. A BCO is recommended when the existing pavement is considered to be in fair or better condition with minor surface distresses and has less than a few punchouts (e.g. three punchouts) per lane mile.

Performance Factors

The following factors determine the performance of BCOs:

- Effectiveness of bond: Proper bond will provide monolithic behavior, ensuring that the stiffness of the rehabilitated pavement (overlay and existing pavement) will carry the traffic load as one structure.
- Existing pavement condition: Since BCOs rely on the existing pavement to assist in carrying the traffic load; the condition of the existing pavement affects the performance of the rehabilitated pavement. Proper repairs or upgrades should be made to provide adequate support as required by design.
- Proper joint spacing: If joints are made, well designed joint spacing helps to reduce curling stresses and bending stresses due to traffic and environmental loads. It is crucial that the transverse joints in the BCOs match those in the existing pavement to promote monolithic behavior.

Common Modes of Failure

The following failure modes are commonly seen in BCOs:

- Loss of bond: The bond between the overlay and the existing pavement can be lost due to lack of quality control in surface preparation or placement during construction.
- Delamination due to difference in coefficient of thermal expansion (CTE): If the BCO has a CTE that is equal or greater than the CTE of the existing pavement, the overlay will expand or contract more than the existing pavement. This results in shear stresses forming at the interface, and these induced stresses can cause the overlay to crack and delaminate.
- Higher stresses at boundaries: Stresses in BCOs at the boundaries (edges of the overlay and along cracks) are higher than in the bonded areas on the interior. The effect is highest at the very edge and diminishes rapidly. This is due to curling and warping stresses in the top of the overlay because temperatures and moisture conditions change more rapidly at the top than in the rest of the slab depth.

2.2.3 Unbonded Concrete Overlay

The term “unbonded concrete overlay (UBCO)” is used to categorize relatively thick concrete overlay that are used to resurface the existing concrete pavement. This type of overlays is typically 5 to 11 in. and is designed to perform without bonding to the existing pavement [4].

Purposes and Uses

The purpose of UBCO is to rehabilitate deteriorating concrete pavements and to improve load capacity and ride quality. A UBCO is used when the existing pavement is severely deteriorated with major surface distresses. A separation layer (typically, a 1-in-thick asphalt layer) is used to maintain separation between concrete overlay and existing pavement.

Performance Factors

The following factors determine the performance of UBCOs:

- Effectiveness of the separation layer: An effective separation layer will act as a shear plane that will prevent cracks from reflecting up from the existing pavement into the overlay. In addition, the separation layer prevents bonding between the new and the old layer allowing them to move independently.
- Effective drainage: A well-constructed drainage system will prevent the building up of pore pressure from the traffic loads. The system serves to prolong the life of the overlay by reducing pumping, asphalt stripping of the separation layer, faulting, and cracking.

Common Modes of Failure

The following failure modes are commonly seen in UBCOs:

- Failure to consider at-grade and overhead structures: The elevation of the pavement after an UBCO placement will increase, in some cases, significantly therefore, at-grade and overhead structures should be raised, or the existing pavement should be removed and replaced near these structures [2].
- Inadequate separation layer: The separation layer prevents reflective cracks from occurring. If the new overlay is not structurally separated from the deteriorated existing pavement, the movement of two structures will be similar, which can induce heavy reflective cracking to the overlay from underneath.
- Poor drainage: The higher elevation of the pavement necessitates a change in the drainage grade lines. Additional right-of-way may be required to provide the proper slopes for the ditches [3].

2.3 Recommended Materials

This section discusses general recommended materials for concrete overlays. Materials discussed have been historically effective for concrete overlay construction; therefore provide an insight on which materials to select for the research.

2.3.1 Cement

The most commonly used cement types are Type I, Type I/II, and Type III. Type I is usually preferred over Type III because it develops less heat of hydration, avoiding many of the problems associated with high temperature development [4, 5].

When high early strength is desired, a Type III or more finely ground Type I cement is used. However, the use of these cements will result in an increased heat of hydration, and caution should be taken to reduce thermal cracking. Other characteristics to consider when selecting cement are long-term mechanical properties, toughness, volume stability, and long life in severe environments [4, 6].

Where local sulfate contamination of the roadway is an issue, Type II or V cements are desirable because they are resistant to sulfate attack and have lower heat of hydration than other cements. Strength gain and set time may be regulated with admixtures and mixture proportioning [7, 8].

To prevent alkali-silica reaction (ASR), low alkali cement (total alkalis Na_2O equivalent<0.6%) should be used for any type of cement coming in contact with ASR-prone aggregates [60]. ASR can occur when siliceous aggregates are used, and alkalis from the cement react to form expansive gel causing deleterious effects. Cement should have low alkali content and supplementary cementing materials (SCM) substitutions to prevent ASR from occurring.

2.3.2 Aggregates

To construct an efficient concrete overlay, the aggregate should be strong and physically chemically stable. The aggregates make up between 65 and 75% of the total concrete volume; therefore, their properties have a definite influence on those of the concrete.

Available aggregates should be evaluated carefully to determine whether an adequate strength will be achieved. Performance requirements may justify purchase of more expensive (high-strength, crushed) aggregates, or careful aggregate blending [10]. Aggregates that conform to Item 421 of TxDOT Standard Specifications should be used.

To prevent ASR, non-reactive aggregates should be selected. Many durability problems result from the reaction between the silica in the aggregates (e.g., siliceous river gravel) and alkalis contained in the cement [11]. If reactive aggregates are used, proper mitigation procedures must be used as required by TxDOT specification Item 421.

Unsaturated absorptive aggregates have a higher moisture demand and can contribute to debonding during curing. These aggregates will absorb available moisture, hindering the curing procedure and affecting shrinkage [4, 10, 12].

Coarse Aggregate (CA)

The maximum CA size is a function of the overlay thickness. It is recommended that the largest practical maximum CA size be used in order to minimize paste requirements, reduce shrinkage, minimize costs, and improve mechanical interlock properties at joints and cracks [4, 9]. Maximum CA sizes of 0.75 to 1 in. have been commonly used, but a reduction in size may be necessary for thinner overlays. For non-reinforced pavement structures, a maximum aggregate size of one-third of the slab thickness or less is recommended [5, 4, 11]. The lowest allowable maximum aggregate size specified should be 0.5 in.

For BCOs only, the compatibility of materials between the old concrete and the new concrete is fundamental for the success of the bond. The coefficient of thermal expansion (CTE) of concrete overlay should be less or at least similar to that of existing pavement [10, 18, 42]. This is because higher slab stresses and wider joint openings can occur when aggregates with higher CTE are used [8]. Since the CTE of the overlay is governed by the coarse aggregate properties, the CTE of the coarse aggregates used in the overlay should be less or equal to that of the existing pavement. Significant difference should be avoided in order to reduce the differential movement between overlay and substrate. In other words, it is recommended that the coarse aggregate in BCOs should have a thermal coefficient no higher than that of the coarse aggregate in the existing pavement. For this reason, it is advisable to utilize a limestone aggregate for BCO if existing concrete has siliceous river gravel as coarse aggregate, because of limestone's lower CTE, but the opposite arrangement will make up for a BCO prone to delamination [11]. If the existing pavement used limestone aggregate as CA, only limestone aggregate with equal or lower thermal coefficient should be used for new BCOs.

Also, the modulus of elasticity (MOE) for BCOs should be lower than for the existing pavement [11]. For the same strain, concrete with higher MOE will have higher stress.

Fine Aggregate (FA)

FA must be sound and nonreactive. It is necessary that FA be sufficiently resistant to tire wear (polishing) to prevent loss of skid resistance. The polish resistance may be improved by using durable and angular fine aggregates [4, 6, 10]. Calcium carbonate fines are known to polish excessively. TxDOT recommends a minimum acid insoluble residue of 60 % [63].

Gradation

Using uniformly and densely graded aggregates is recommended to reduce shrinkage because it reduces required paste. This is helpful in thin concrete overlays, because the risk of debonding due to shrinkage and curling potential is decreased [10]. Both the top size and gradation of the aggregate will also affect aggregate interlock at the joint, which is another important consideration, because thin concrete overlay joints are typically not dowelled [8].

2.3.3 Fly Ash

Cement may be partially replaced with fly ash, which can lead to higher ultimate concrete strengths and lower permeability [7]. Moreover, replacing cement with fly ash can reduce cost, increase workability, and increased protection against deleterious environment. Due to the lower specific gravity of fly ash, as compared to cement, replacement of cement with fly ash increases the volume of cementitious paste in the mixture. This increased volume of paste provides an improved coating of fibers and aggregate in the mixture, leading to improved workability and fiber distribution [4]. However, higher the fly ash replacement lowers the early strengths. This may lead to delay in construction and opening to traffic.

2.3.4 Slag

For concrete overlays, granulated, ground, blast furnace slab (GGBFS), blast furnace slag (BFS), or, simply, slag is typically used in replacement proportions of 25 to 35 %. It is normally substituted for cement by mass. The proportion of slag cement is usually dictated by requirements for strength, durability, time of set, and the resistance of the concrete to ASR. Mixtures should be optimized for strength and durability using trial batches and the appropriate test methods. It is not uncommon to find that total cementitious material can be reduced by using appropriate levels of slag cement to replace cement when strength is used as the evaluating criteria.

2.3.5 Silica Fume

Generally, addition of silica fume will increase the compressive strength. However, an unbalanced addition will attract agglomerated silica fume particles to provide fast crack propagation path within the matrix [13]. And higher compressive strengths usually mean higher modulus for concrete, and that may not be desirable in thin overlays on very low-modulus asphalt substrates. Also, typically, silica fume is relatively expensive, rarely available, and difficult to handle, so the use of silica fume is not recommended for overlays.

2.3.6 Admixtures

Typical admixtures used in concrete overlays include air entrainment, high range water reducers, and retarders. When combinations of these admixtures are used, their combined effects should be considered. Care must be taken to avoid any admixtures that cause unnecessary reduction in the rate of strength gain. Trial batches should be made to evaluate the interaction between the admixtures and other constituents. For BCO applications, preliminary bond tests should be conducted to see if chemical admixtures affect bond strengths obtained at early ages [5].

Air Entrainment

Air entrainment protects the hardened concrete from freeze-thaw damage and deicer scaling. Air entrainment also helps increase the workability of fresh concrete, significantly reducing segregation and bleeding [4].

High Range Water Reducers (HRWRs)

HRWRs can make concrete with a low water-to-cementitious materials ratio workable enough for placement [10]. This allows for a lowering of the w/cm, while maintaining a desired slump. This has the beneficial effect of reducing permeability. Although HRWR is not commonly used in concrete overlay constructions, whenever fibers are used in concrete overlays the use of HRWR is highly recommended.

2.3.7 Reinforcement

Since concrete is weak in tension, reinforcement can be added to increase the performance of concrete overlays. The installation of reinforcement can be time consuming; therefore, careful planning is needed to minimize time spent from using reinforcement.

Wiremesh

Based on an evaluation survey done on IH610 in Houston, welded wire mesh fabric provided more effective restraint on concrete volume change potential than steel fibers [18]. The increase in volume change restraint can achieve better bond between the overlay and the existing pavement. Wire mesh is relatively easier to install than reinforcement bars; however, it still takes careful placement and additional construction time compared to using fibers.

Reinforcement Bars [4]

When reinforcement bars are placed in concrete overlays, typically No. 5 and No. 6 bars are used for longitudinal and transverse reinforcement. Larger bar sizes are likely to cause segregation of the coarse aggregates and voids in the mixture near the bars.

Reinforcement Bars for BCOs

Steel bars can be placed directly over the surface of the existing pavement, rather than at mid-depth of the overlay. The performance of the steel has been demonstrated to be the same, but placing it on top of the existing pavement saves construction time and costs, because it is much easier and more economical to lay it or the surface than to place it on chairs at mid-depth [11].

An experiment was conducted at Center for Transportation (CTR) at the University of Texas at Austin to determine the effect of the steel position on its bonding to the concrete [43]. Two types of concrete slabs were cast in the laboratory. The first group consisted of 12 slabs, 12-in. by 12-in. by 3-in.-thick. Steel bars were laid on the 3-in.-thick base, after the surfaces were scarified and before placing an overlay. For the second set of slabs, 12 more specimens were cast, this time placing the steel at mid-depth. All slabs were cured under normal laboratory conditions. Schematics of both types of specimens are shown in Figure 2.1.

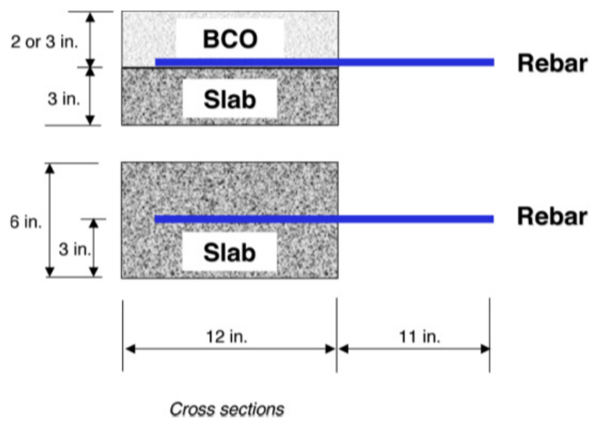

Figure 2.1: Experiment on reinforcement location

The test consisted of pulling the steel bars from the slabs. All bars failed in tension before they could be pulled out from the slab, showing that its bond strength is higher than the steel tensile strength, regardless of the position of the bars. From the test, it is inferred that the bars will not fail in anchorage, even when placed directly on the surface of an existing pavement. Therefore, the reinforcement steel can be placed directly on the surface, as is shown in Figure 2.2, saving construction time, labor, and money. Furthermore, the reinforcement placed directly on top of the substrate helps to restrain the movement of the new concrete slab due to environmental changes, which in turn, improves the bond between both pavement layers. The steel will restrain concrete volume changes at the interface most effectively, which will prevent or retard debonding.

Figure 2.2: Steel placed directly on top of existing pavement

Fibers

Fiber incorporation can provide improved flexural ductility and toughness, fatigue capacity, and abrasion and impact resistance [19]. The effect of fibers in concrete on compressive strength generally varies from a negligible increase or decrease to strength gains around 20% [20, 21]. Also, fibers can be beneficial to reduce crack development, to slow crack growth, and to delay debonding propagation while providing residual strength in pavements that have already cracked [22]. Fibers are usually used in thinner overlays because of their high cost.

Fibers can bridge cracks in concrete and restrain them from opening thus increasing the load ability of the concrete overlay [23]. Fiber-reinforced concrete pavements should have a longer service life and require less maintenance than non-reinforced concrete pavements [24].

However, some past experiences have shown that negative effects can be expected from fiber reinforced concrete overlays. The most prevalent effect is the cost. Addition of fibers will tremendously increase the project cost, and, sometimes it is difficult to calculate cost-to-benefits ratio of using fibers.

Another problem is the difficulty in handling fibers during construction. Fiber balling [23] is a phenomenon that occurs when lack of effort to disperse the fiber in concrete matrix, bunched fibers appear in the concrete overlay surface. Fiber balling not only reduces benefits from using the fibers, but also, creates weak spots in concrete overlays.

Proper handling of fibers is required to increase performance in concrete overlays. Increase in fiber dosage can lead to significant decrease in compressive and flexural strength. However, without proper dispersion of fibers, the crack bridging benefits cannot be expected.

Polypropylene microfibers are produced either as cylindrical monofilaments or fine fibrils with a rectangular cross section. Polypropylene microfibers can be in monofilament, multifilament, or fibrillated form (Figure 2.3).

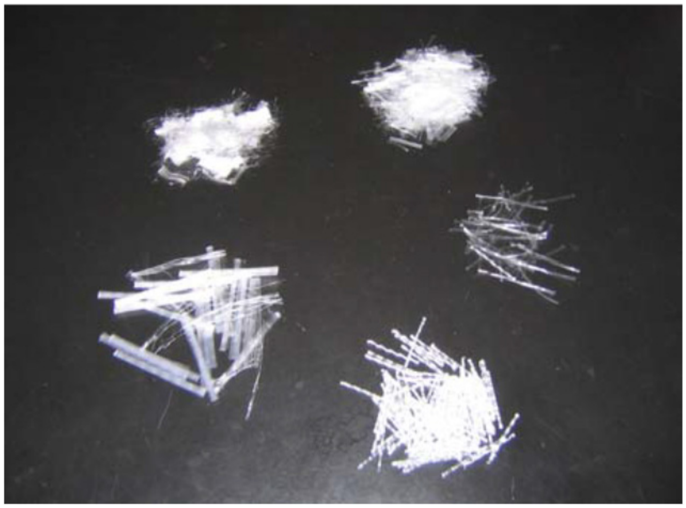

Figure 2.3: Different forms of polypropylene fibers

Microfibers are effective in controlling plastic shrinkage and settlement cracking. The fibrillation process greatly enhances the bonding between the concrete and the polypropylene fibers and can provide residual strength in pavement that has already cracked [6].

Polypropylene macrofibers are coarse fibers that allow greater surface area contact within the concrete, resulting in increased interfacial bonding and flexural toughness. Polypropylene macrofibers can be used as secondary reinforcement and can provide greater post-crack strength and concrete slab capacity. Additional benefits include improved impact, abrasion, and shatter resistance.

Polyester fibers are available only in monofilament form. They commonly have relatively low fiber content and are used to control plastic shrinkage-induced cracking. Synthetic fibers do not absorb water and therefore do not affect the mixing requirements.

Steel fibers are primarily made of carbon steel, although stainless steel fibers are also manufactured. Perhaps the biggest advantage of steel fibers is their high tensile strength and their ability to bridge joints and cracks to provide tighter aggregate interlock, resulting in increased load-carrying capacity. Steel fiber reinforced pavements exhibit excellent toughness and pre- and post-crack capacity [19].

The aspect ratio is an important parameter influencing the bond between the concrete and the fiber, with longer fibers providing greater bond strength and toughness, often at the expense of workability. Steel fibers may also have certain geometric features to enhance pullout or anchorage within the concrete mixture. These features may include crimped or hooked ends or surface deformations and irregularities (Figure 2.4).

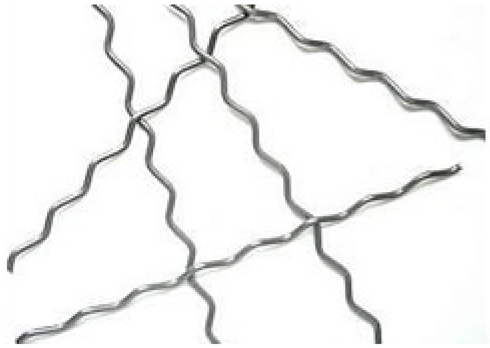

Figure 2.4: Deformed steel fiber

Blended fiber systems combine macrofibers with microfibers or steel fibers. The microfibers in these systems provide resistance to plastic shrinkage and settlement cracking, while the macrofibers or steel fibers provide long-term secondary reinforcement. Blended systems provide higher levels of fatigue resistance, greater flexural toughness, and improved durability. Additional benefits include improved impact, abrasion, and shatter resistance [62].

2.3.8 Bonding Agents [11]

Bonding agents, e.g., portland cement grouts, latex-modified portland cement grout, and epoxy resins, are sometimes used to improve bond. However, bonding agents cannot compensate for bad substrate surface preparation and may act as a bond breaker when used inappropriately, thus it is not recommended to use bonding agents unless under special circumstances. The use of bonding agents leads to two interfaces and thus to the creation of two possible planes of weakness instead of one. Besides, a grout often has a high water-cement ratio leading to a low strength and the risk of a cohesive failure within the bonding agent itself.

Under normal placement conditions, the performance of the BCOs and whitetoppings is better if no bonding agent is utilized [25], as long as the surface has adequate texture and has been cleaned and free of dust, white water, other debris, and a saturated surface dry (SSD) condition is achieved.

The shear strength at the bond interface should be at least 1.4 MPa (200 psi) [11, 26]. Bond strength can be improved by increased surface roughness, which exposes aggregates to lock the layers together [1].

Grout

If the surface happens to be wet, a concrete grout will assure better bond strength. If a grout is used, the overlay should be placed before the grout dries; otherwise, the bond strength of the overly may be significantly reduced, because dried grout increases the probability of delamination by acting as a bond breaker [11]. Past experiences have shown that grouting is not needed, especially when the existing surface has been milled and cleaned well. A cleaned and properly moistened surface is enough to ensure proper BCO bonding.

Figure 2.5 shows that immediately before paving, a grout can be uniformly broomed over the full width of the prepared surface. The prepared surface must be in SSD condition. Typically water-to-cement ratio of the grout is around 0.62 to 0.70 by weight, or approximately seven gallons of water per sack of cement [27].

Figure 2.5: Spraying grout immediately ahead of paver

There are reports that grout does little to improve bond strength [28]. However, if used properly with a clean, textured surface, good bond can be achieved [29]. Nevertheless, placing grout is an additional step that will slow the paving process and adds to the cost. The BCO placed in Houston, on the South Loop [36], showed that dry grout could act as a bond breaker between the existing pavement and the overlay: the experimental sections where the grout was used and allowed to dry prior to the paving of the overlay caused early delaminations. The overlay in those sections had to be removed shortly after construction. Because grout is not needed when the surface preparation and cleaning are adequate, its use is not recommended, as it could cause debonding.

Epoxy

Where the substrate has been treated by a less expensive surface cleaning procedure and, therefore, is not rough enough to ensure an adequate bond, liquid epoxy materials have been reported to provide extremely high bonding strengths in the laboratory (higher than 5000 psi) [30]. When epoxy is used, it is very important to apply the epoxy immediately ahead of paving. If not, the epoxy will harden and act as a bond breaker, which will lead to delamination.

Shear Connectors or "Jumbo Nails"

Use of shear connectors or "jumbo nails" can effectively control development of the overlay drying shrinkage cracks at early age [31]. These nails are installed along the pavement edges and longitudinal saw cuts – the areas more susceptible to debonding. Nails are installed on the original pavement prior to the overlay placement. Installation consists of a three-step process: drilling, drill-hole cleaning, and nail driving. The high-strength steel nails are driven into the predrilled holes in the existing pavement by an actuator that makes use of an explosive charge. The top part of the nail remains out of the existing pavement to be covered by the concrete overlay when the new concrete is cast.

Similar to shear connectors, curb-type reinforcement bars epoxied into the existing pavement surface have been used successfully to prevent edge curling and warping [29]. Usage of nails is at about 6-in. from the edge or joint, with spacing between nails of 15 to 30 in. Smaller nail spacing results in a higher number of cracks of smaller width.

2.3.9 Incidental Materials [4]

It is not practical to install dowel bars, tie bars, or keyway in thin concrete overlays because of the lack of concrete cover. Field evaluation has indicated that the load transfer provided by aggregate interlock is generally high because of the joint spacing and the support provided by the asphalt layer. [1, 23] However, dowel bars, tie bars, and key ways play an important role in improving load transfer efficiently in UBCO applications where aggregate interlock alone is not enough. Unlike whitetoppings or BCOs, UBCOs offer thicker layers of concrete so that these materials can stay safely embedded.

Dowel Bars

Typically, billet steel, grade 60 bars that conform to ASTM A615 – 09b "Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement" or AASHTO M31 are used. Sometimes the sizes are reduced to accommodate thinner concrete overlays. The recommended number and spacing of dowels is the same as those for new pavements. In general, a uniform 12-in. spacing is recommended, but non-uniform spacing has also been used successfully. In the non-uniform dowel spacing design, the dowels are concentrated in the wheel paths [4, 32].

Tie Bars

Typically, billet steel, grade 40 bars that meet ASTM A615 or AASHTO M31 specifications are used.

Joint Sealant Materials

If used, joint sealant materials are (1) hot-poured rubberized materials conforming to ASTM D660 – 93 "Standard Test Method for Evaluating Degree of Checking of Exterior Paints," or per normal design, (2) silicone materials conforming to a governing state specification, or (3) reformed compression seals conforming to ASTM D2628 – 91 "Standard Specification for Preformed Polychloroprene Elastomeric Joint Seals for Concrete Pavements," or a governing state specification. When small panel sizes are constructed, sealant is often not used.

2.3.10 Separation Layer Materials for UBCOs

A separation layer allows the existing pavement and the new concrete overlay to act independently. It also prevents distresses from reflecting into the concrete overlay. Typically, 1- to 2-in.-thick asphalt concrete has been widely used for the purpose and has been proven effective. Materials such as polyethylene, roofing paper, and curing compound do not work. Most failures in unbonded concrete overlays are due to the use of inadequate separation layers or insufficient overlay thickness.

Thin separation layers (such as sheathings) must be avoided because they are more likely to permit reflective cracking from the existing pavement. Thicker separation layers can prevent reflective cracks from occurring [44]. Figure 2.6 shows how a smooth slip plane can prevent reflective cracks from occurring.

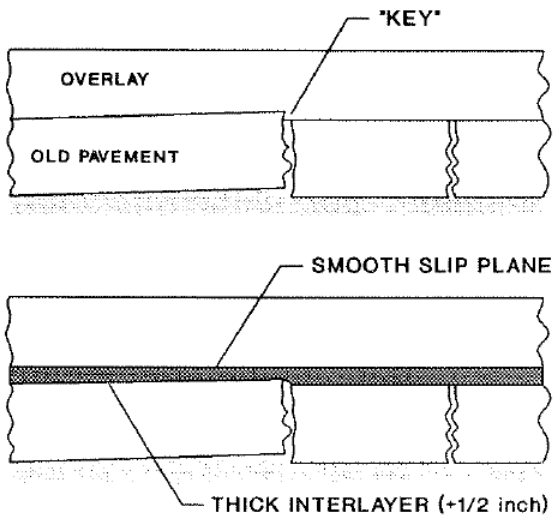

Figure 2.6: Purpose of interlayer

2.4 Recommended Mixture Design/Proportioning

Once, the potential materials are selected, proper design/proportioning of those materials is very important to ensure desired concrete overlay performance. In this section, brief discussions for each design/proportioning criteria and recommendations are provided.

2.4.1 Cementitious Materials Content

A concrete overlay must have enough cementitious paste to coat the aggregates, and the interface layer [6, 7]. Insufficient cementitious material content can lead to low early strength. However, if more than enough cementitious paste is used; it will increase the chance of durability issues such as shrinkage and alkali-silica reaction (ASR) and issues caused by high heat of hydration. Based on findings from the literature, depending on surface treatments, CA shape and texture, lower cement content is recommended [10, 23, 29].

2.4.2 Water-to-Cementitious Materials Ratio (w/c)

Lower w/c ratios are often used for concrete overlays to minimize drying shrinkage. However, lack of water can lead to less than ideal amounts of paste, hindering the complete coating of aggregates that may lead to reduced workability [6, 7]. The low w/c of the El Paso BCO concrete, coupled with a very dry surface, is blamed for causing overlay debonding. However, too much water increases shrinkage and evaporation rate [12, 1]. Higher water contents provide greater potential for shrinkage as the water evaporates [4, 18]. For normal placement, 0.40 to 0.45 w/c is recommended and maximum of 0.35 w/c is recommended for expedited placement.

2.4.3 Fly Ash Content

A study [15] showed that addition of Class C fly-ash resulted in increased cracking when using cement replacement range of 0 to 15%. However, beyond this replacement rate, one can expect beneficial effects of fly ash addition. In other words, there appears to be a threshold cement replacement rate of around 20% at or higher which beneficial effects of fly-ash addition on shrinkage cracking occurs. A 30% replacement of cement with fly ash improved workability, reduced heat of hydration, noticeably increased long-term strength, and enhanced resistance to environmental attack. Too much fly ash will reduce short term strength gain. A replacement of at least 25% of fly ash is recommended.

2.4.4 Aggregate Ratio

Since cement paste shrinks and expands more than aggregates, using as much aggregate as possible while using as little cement possible is beneficial (while meeting other requirements. An increase in the aggregate/cement (a/c) ratio was highly effective in reducing shrinkage cracking [15].

2.4.5 Fiber Content

Fiber addition impacts water demand and workability. Changes to water content and water-reducing admixture dosage will be required for adequate workability. The extent of these changes will be dependent on fiber type and fiber dosage rate [19]. A main factor that will determine the content or even the usage of fibers at all will be the economic feasibility. A cost–benefit analysis will need to be performed to determine if or which fibers can reduce long-term cost (obviously, initial cost is going to be much higher).

Each type of fiber has a recommended dosage from the manufacturer. Typical amounts of synthetic fibers used are at least 0.1% by volume or around 3 lb/yd^3. Normal steel fiber contents range widely from 0.25% to 2% by volume or 33 to 265 lb/yd^3.

Numerous studies have shown indefinite trends of compressive and/or tensile strength change due to the addition of fibers [19, 21, 23, 33]. There were instances where the strengths increased, decreased, or stayed the same. For each project, the amount of fiber desired should be calculated in terms of cost and required strength. TxDOT has developed a minimum average residual strength (ARS) requirement based on ASTM C1399.

2.4.6 Air Entrainment

Enough air must be entrained for durability as required for environmental reasons and chloride exposure. Typically, in the northern–most parts of Texas 4 to 7% of air is entrained depending on project specifications [63]. The interaction between air entrainment and other admixtures should be considered.

2.4.7 Admixture Dosage

Admixture dosage can be different for each batch. The dosage should be adjusted through trial batches, and the interaction between admixtures should be considered. Too much HRWR can cause the mixture to get sticky and make finishing more difficult.

2.5 Recommendations for Construction

Construction is a crucial phase that can determine the level of performance. Many failures in concrete overlays are caused by poor construction procedures [34]. Good selection and mixture/proportioning of the materials need to be supplemented with good construction practices to produce desired performance in concrete overlays.

2.5.1 Environmental Limitations

Weather conditions prevailing during concrete overlay construction can be critical to the performance; environmental variables that play a key role in the behavior of the concrete overlay are temperature, moisture surrounding the concrete. Hot and dry climates pose the most problematic conditions for concrete overlay placement, because these conditions favor the loss of moisture from fresh concrete. Excessive water evaporation from the concrete can cause plastic shrinkage cracking, which reduces the integrity of the concrete surface and may lead to reduced durability.

A combination of high wind velocity, high air temperature, low relative humidity, and high concrete temperature is the most harmful for paving conditions, because it results in high water evaporation. Placing during low temperature months, i.e., December and January, can minimize climatic stresses and cracking [35].

Caution must be taken in hot, dry, and/or windy climates to avoid excessive evaporation of water from concrete and produce plastic shrinkage cracking. Weather stations should be used to monitor the weather conditions and the ACI 503R nomograph should be used to determine the evaporation rate. The following adverse conditions must be monitored in construction [4, 36]:

- Surface of the existing pavement should not exceed 125 °F immediately before placement.
- Temperature differential in the 24 hours after the placement must be less than 25 °F.
- Evaporation rates should not exceed 0.2 $lb/ft.^2/hr$ based on the ACI 503R nomograph and may need to be lower depending on mixture constituents.

If any of the adverse conditions mentioned above occurs during the placement of concrete, the placement should be avoided unless the following conditions can be achieved [4, 36]:

- Cooling the aggregate or concrete.
- Special curing methods (See section 2.5.7).
- Use of fly ash as cement replacement to lower the heat of hydration.

2.5.2 Surface Preparation

Surface preparation encompasses the operations conducted on the existing pavement surface to enhance it in such a way that the new concrete layer can behave as designers intended. The level of surface preparation will determine to a significant extent the longevity of a concrete overlay. Surface preparation is crucial for any type of concrete overlay [28].

For BCOs

It is not an overstatement to say that the longevity of a BCO is mainly determined by the effectiveness of bond at the interface. This statement is truer the thinner BCOs because thin BCOs rely on the existing pavement to carry the traffic load; thus good bond is the most important factor.

If the existing pavement is overlaid with AC layers, these layers must be removed by milling prior to surface preparation and repair of distresses. Remnants of AC will hinder the bond between concrete layers and are likely to trigger delaminations, because AC works as a bond-breaking layer between concrete layers. Complete milling of these layers will ensure that all surface contaminants such as oil, carbonates, and acids are removed.

Three typical means of surface preparation for BCOs are shotblasting, milling, and sand blasting. The most efficient method is by means of shotblasting equipment, such as the Skidabrader™ machine. Unlike cold milling, shotblasting can achieve adequate depth without causing microcracking. It can remove concrete matrix leaving the CA intact undamaged.

Sandblasting is suitable for small and hard to reach areas. It is not recommended for large areas because of its uneven removal of surface material. Surface preparation procedures are listed in Table 2.1.

Table 2.1: Surface preparation procedure [11]

Removal method	**Principle behavior**	**Depth action (mm)**	**Important advantages**	**Important disadvantages**
Shotblasting	Blasting with steel shot.	No (12)	No microcracking, dust.	Not selective.
Milling (scari-fying)	Longitudinal tracks are introduced by rotating metal lamellas.	Yes (75)	Suitable for large volume work, good bond if followed by water flushing.	Microcracking is likely, reinforcement may be damaged; dust development, noisy; not selective. Newer machines can cause less microcracking.
Sandblasting	Blasting with sands.	No	No microcracking.	Not selective; leaves considerable sand.
Scabbling	Pneumatically driven bits for impacting the surface.	No (6)	No microcracking, no dust.	Not selective.
Grinding (planning)	Grinding with rotating lamella.	No (12)	Removes uneven parts.	Dust development, not selective.
Flame-cleaning	Thermal lance	No	Effective against pollution and painting, useful in industrial and nuclear facilities.	The reinforcement may be damaged; smoke and gas development; safety considerations limit use; not selective.
Pneumatic (jack) hammers (chip-ping), hand-held or boom-mounted	Compressed-air-operated chipping	Yes	Simple and flexible use, large ones are effective.	Microcracking, damages reinforcement; poor working environment; slow production rate; not selective.
Explosive blasting	Controlled blasting using small, densely spaced blasting charges.	Yes	Effective for large removal volumes.	Difficult to limit to solely damaged concrete; safety and environmental regulations limit use; not selective.

Removal method	Principle behavior	Depth action (mm)	Important advantages	Important disadvantages
Water-jetting (hydro demolition)	High pressure water jet from a unit with a movable nozzle	Yes	Effective (especially on horizontal surfaces), selective, does not damage reinforcement or concrete, improved working environment.	Water handling, removal in frost degrees; costs for establishment.

Depth of scarification and type of aggregate of the existing concrete may dictate the type of surface preparation to use. Cost is also a factor to take into consideration. Typically shotblasting is the most inexpensive option and produces better prepared surfaces [29].

The scarification depth and texture should be specified for each project, depending on economic considerations as well as the materials properties, both of the existing pavement and the new overlay. For instance, if the existing pavement grout paste is relatively soft and the coarse aggregate is especially hard, a light shotblasting will be sufficient to remove the paste to reach the specified depth, leaving the aggregate intact, resulting in a good surface texture.

Typically, the depth of surface removal is about 0.25 in. into the coarse aggregate [42]. It can also be specified in terms of some standardized texture test method, such as the sand patch test (ASTM E 965) or circular track meter (CT Meter). Typical texture readings from this test are between 0.050 in. and 0.099 in.

For Whitetopping/Unbonded Concrete Overlay (UBCO)

If surface distortions on the existing asphalt pavement are excessive (greater than 2 in.), either milling or a leveling course may be necessary to provide proper grading. The milling process should be controlled by a string line to prevent concrete quantity overruns [9]. Typically, milling is used to scarify the existing pavement to roughen up the surface.

For UBCO placements, the existing pavement acts as a base, and a separation layer is placed on top to separate the UBCO and the existing pavement to prevent cracks from reflecting through. Since the existing pavement serves only as a base, no special preparation is needed. Usually, a thin layer of asphalt is used to act as a separation layer, so if there are any asphalt patches on the existing pavement there is no need to remove them.

White pigmented curing can be used to cool the existing pavement prior to pouring. This curing compound reflects heat and prevents heat build-up in the dark surface, reducing shrinkage cracking in the concrete and potential paving problems due to a soft surface [8, 24]. Water fogging is another method that can reduce the asphalt temperature. Figure 2.7 shows a way to cool down the prepared surface by spraying with water. It is good practice to water fog if the asphalt surface heat makes it uncomfortable to touch with an open palm [37]. It was found that mix water in the fresh concrete overlay was absorbed into the dry substrate, reducing the amount of water available to fully hydrate the cement paste at the bonding interface [38].

Figure 2.7: Cooling down the prepared surface before placement

2.5.3 Surface Cleaning

Surface cleaning refers to the removal of dust and debris after the surface preparation is complete and prior to the placement of the concrete overlays, to ensure that no foreign elements interfere with the achievement of bonding between both layers. Any loose/foreign materials present at the interface will act as a bond breaker and can cause delamination.

After the surface preparation operations are finalized and the reinforcing steel is in place, the last cleaning of the surface is done by air blasting just before concrete placement. It should be noted that air blasting and water blasting should be used only as supplementary cleaning procedures for loose material and debris elimination from the surface after milling, shotblasting, or sandblasting, because these methods are not capable of removing paint stripes, tire marks, or grout matrix. Air blasting is to be used just before overlaying to thoroughly remove debris from milling or shotblasting operations. It is important not to leave a large time lag between the final surface cleaning and paving in order to prevent the contaminants from resettling. Figure 2.8 shows hydro cleaning as a way to clean the prepared surface.

Figure 2.8: Cleaning the surface with hydro cleaning equipment

If trucks or equipment need to drive on top of prepared the surface, tarps should be placed to prevent any foreign materials falling on the surface [28]. The ultimate goal in surface preparation is to achieve a well textured and clean surface to receive the concrete overlay.

2.5.4 Fiber Incorporation

For mixing purpose, both steel and synthetic fibers can be packaged in water-soluble bags designed to break down in the mixer and allow the fibers to be evenly dispersed in the mixture [24]. If not handled properly, remnants of the paper wrapper used to bundle the polyolefin fibers may appear in the mixture. To avoid producing fiber balls and uncoated fibers, fibers can be introduced into the mixture sooner; however, the mixing time should be slightly increased and the batch size should be reduced. These procedures may help, but do not completely eliminate the problem [24]. Figure 2.9 shows that blowing fiber into the mixing process to disperse fiber evenly in the mixture.

Figure 2.9: Blowing the fiber into the mixing truck

2.5.5 Placement

The following are general considerations for placement of concrete overlays:

- To prevent water loss in concrete due to absorption by the existing pavement, the prepared surface ahead of the paving machine should be dampened with water to achieve SSD condition [11, 18, 39].
- Tracking of dirt or debris ahead of paving machine should be prevented.
- Bonding agents should not be used unless under special circumstances. With a properly prepared surface in SSD condition, a bonding agent, such as epoxy, is not required. If bonding agents are used improperly, they may act as bond breakers. (See section 3.8).
- For BCOs, reinforcements can be directly placed on top of existing pavement. Laboratory studies have shown that reinforcement placed at the interface develops the same bond as reinforcement placed in the middle of the overlay. Placement of the reinforcement at the interface also eliminates the risk of concrete honey combing and poor consolidation beneath the steel [4].
- The grading machine must be adjusted to achieve the required thickness of the concrete overlay.
- The steel fibers at the surface of the pavement can become entangled with burlap and can be pulled out along with other fibers and coarse aggregates. An un-weighted carpet drag can be a substitution to provide a satisfactory interim surface finish on the pavement [23, 24].
- Finishing of the new concrete overlay surface should follow the same practices used to finish any concrete pavement [4].

2.5.6 Jointing

To reduce the edge and corner stresses, longitudinal joints should not be placed in the wheel path. Heavy loads concentrated near the edge of the thin panels should not exceed their load capacity [40, 41]. The following are recommendations for jointing:

- The timing of joint sawing is critical. Sawing too early can cause excess raveling, and sawing too late can result in shrinkage stress causing uncontrolled random cracking.
- ACPA recommends that joint spacing be about 12 to 15 times the slab thickness.
- Joint spacing has a significant effect on the rate of corner cracking. Short joint spacing, common on thin concrete overlays, reduces load-related stresses, because the slabs are not long enough to develop as much bending moment [8]. The joint location is also important to avoid concentrated loads. For example, 4-ft. by 4-ft. panels on a 12-ft.-wide lane would put truck tires on the edge of the panels, and significant distress would occur if the thin concrete overlays became de-bonded from the existing pavement [9]. Figure 2.10 is a good example of failed joints in wheel paths.

Figure 2.10: Failed joints in wheel path

2.5.7 Curing [11]

The importance of proper curing can never be understated. Proper curing procedures are essential in preventing excessive moisture loss at early ages that can result in plastic shrinkage and loss in tensile strength capacity at the surface. Curing should begin as soon after placement and finishing as possible to minimize loss of bleed water. Figure 2.11 shows the curing crew following the paving machine closely. For concrete overlays, It is recommended that a double application of the curing compound be used [5, 32].

Figure 2.11: Prompt curing following the paver

Types of curing procedure include the following:

- Curing compound: For textured or tined surface the spray application should be applied from two directions to ensure that the entire surface is coated.
- Membrane curing: Various liquid sealing compounds, e.g. bituminous and paraffinic emulsions, coal tar cut backs, pigmented and non-pigmented resin suspensions, or suspensions of wax or non-liquid protective coating such as sheet plastics or water proof paper, are used to restrict evaporation of water.
- Curing blankets: A cover of sacks, mats, cotton bats, burlap, straw, or other suitable paper is placed over the surface to reduce evaporation and to reduce the temperature reduction at the surface. When used to reduce evaporation the blankets are generally wetted.
- Monomolecular film (MMF): MMFs are compounds that form a thin monomolecular film to reduce moisture loss from the concrete surface prior to curing. Another curing method should be used after the evaporation retardant is sprayed on. Research has shown, however, that the use of MMF followed by application of curing compound does not consistently provide less evaporation than curing compound alone.

Chapter 3. Condition Surveys of Existing Concrete Overlays in Texas

Forensic studies conducted on bonded concrete overlays (BCO) that had construction problems in Texas were reviewed. Also, a number of sites were visited throughout Texas to perform condition/evaluation surveys, as part of CTR Project 0-6590 Research Task 2, Condition Survey and Evaluation of Existing Concrete Overlays in Texas. The research objectives in this task entail the review of reports of construction and performance histories and making site visits to existing concrete overlays in the state to evaluate their performance to date. Two researchers performed the surveys and wrote the findings in this chapter.

3.1 Forensic Study of BCO on IH-10 El Paso [45]

This bonded concrete overlay experienced delaminations in several areas soon after the construction. This report provides the causes of failure and recommendations for future expedited concrete overlays. Delamination of the overlay was due mainly to inadequate paste adhesion. The factors that led to delaminations include the following:

- Very low initial water content of the concrete mixture.
- Loss of available water to substrate absorption.
- High rate of evaporation.
- Withholding water from the mixture at batching.

The following recommendations were made based on the forensics report:

- Surface preparation: Shotblasted and/or hydro cleaned surface should be in SSD at placement of the overlay and should be air blasted within an hour before placement.
- Evaporation rate: Placement only under acceptable environmental conditions (typically evaporation rates of less than 0.2 lb/ft.^{2}/hr).
- Opening to traffic: Must be at least 12 hours old and has attained a splitting tensile strength of at least 500 psi.
- Timing of curing: Apply curing as soon as possible to prevent water loss due to evaporation.

3.2 Forensic Study of BCO on IH-10 Fort Worth [46]

This project also experienced early delaminations after construction. After extensive forensics work, the cause of the problem was found to be due to negligence in surface preparation in construction and was not a design failure. Debris was found in core samples that were extracted where the delaminations occurred. The conclusions regarding the cause of the problems are listed below:

- Surface preparation and cleaning was done in a hurried way.

- The debris and contamination was generated by the slab sawing operations in which contaminants were carried to the prepared surface by surface water prior to concrete placement.
- The debris accumulated just ahead of the paving machine.

The following recommendations were made in the forensics report:

- Monitor for severe weather conditions contributing to excessive evaporation rate or temperature limits.
- Follow specifications for proper surface preparation and cleaning.
- Ensure the prompt application of curing compound.
- Be prepared for special curing, if necessary, when adverse weather conditions warrant it.

3.3 Evaluation Study of First BCO in Texas

The first BCO project in Texas was implemented in 1983, in Houston on IH-610 (south loop). The project was an experimental BCO on a 1000-ft. continuously reinforced concrete pavement (CRCP) segment, built in July and August of 1983. It consisted of five 200-ft. test segments, with several combinations of reinforcement (no reinforcement, welded wire fabric, and steel fibers) and BCO thicknesses (2 and 3 in.), all constructed on the four eastbound lanes, between Cullen Blvd. and Calais St. The surface was prepared by cold milling and sandblasting; portland cement grout was used as a bonding agent for the vast majority of the sections. The following lists the findings from the construction report [47]:

- Adding an overlay to an aged pavement improves the structural quality of the pavement as measured by the reduction in (falling weight) deflection both at cracks and at mid-span positions.
- Overlaying on dry surface results in better bond strength at the interface than overlaying on a wet surface. Specifically, overlaying when the surface is wet resulted in the weakest interface bond strength. Under this condition, it is advisable to apply a grouting agent or to dry the surface before overlaying. If the surface is dry, there is no need for using a grouting agent.
- Roughening the surface by milling or scarifying helps produce a better bond.
- The effect of positioning overlay reinforcements at different heights versus the interface bond strength is insignificant. Hence reinforcing bars can be placed on the original surface in the interest of cost saving.
- (Steel) fiber reinforced overlays are a good alternative to plain or bar-reinforced concrete overlays.

The following lists the findings from an evaluation report after two years of service [4]:

- In BCO construction, a mixture of water, cement, and plasticizer is an adequate bonding agent.

- Two to three years after BCO placement, no significant debonding seemed to have occurred.
- The (steel) fiber reinforced sections proved to be far superior in their ability to control longitudinal and transverse cracking.
- After almost two years of BCO service virtually none of the cracks showed evidence of spalling.
- The five test sections on South Lop 610 have been monitored for approximately two years at the time of the report. During this period, satisfactory performance has been noted on most of the pavement response variables. However, the long-term performance still needs to be established.

A sounding survey conducted in 1990 on this section revealed some minimal delamination of the overlay [5]. Condition surveys conducted in 1996 showed few distresses on the section and no major performance problems [48]. The success of this first experience led TxDOT to implement a second BCO project.

3.4 Evaluation Study of Second BCO in Texas

The second BCO was implemented on the north side of IH-610 in Houston. The section of interest consisted of a 3.5-mile stretch on the northwest part of the loop, between East T.C. Jester Blvd. and IH-45. Originally built in the late 1950s, the 8-in. slab of CRCP on a 6-in.-thick cement stabilized subbase was overlaid with a 4-in.-thick BCO in 1986 [30]. This overlay project experimented with several variables, including reinforcement, coarse aggregates, bonding agents and existing pavement conditions (various levels of distress).

Within the project limits, ten test subsections were identified, with lengths ranging from 400 to 600 ft., each including different combinations of the aforementioned variables. During and after construction, some delamination took place between the BCO and the original pavement. It was found that most the delaminations occurred within the first 24 hours after placement, due to the presence of adverse environmental conditions during overlay placement, i.e., high evaporation rates and high daily temperature differentials, and the delaminations always occurred at a joint, crack or edge. The following lists the findings from the evaluation report [47]:

- Bonded concrete overlays significantly reduce the pavement deflection. The deflection reduction magnitudes indicate the slab performed monolithically. The section of CRC with siliceous river gravel reduced deflection the most as expected due to its higher modulus of elasticity.
- The existing pavement conditions did not affect the overlay pavement performance as long as most of the existing distresses were repaired before the overlay was placed.
- Overall, there was a significant decrease in the amount of all types of distress. The section of CRCP with limestone had the least number of transverse cracks, and the siliceous river gravel and fiber reinforced sections were second and third, respectively. Spalling and punch outs did not exist on any of the test sections.

Even though in some segments the delamination was extensive, it did not continue to deteriorate over time and did not appear to significantly affect performance [49]. Relatively recent condition surveys conducted on this segment in November 2000 and June 2006 revealed that, in spite of the heavy traffic, after 20 years of service, the BCO was still in excellent shape, presenting a minimal number of distresses [18, 50].

3.5 Evaluation Study of Third BCO in Texas

The third BCO rehabilitation in Texas was also implemented on the IH-610 Loop in Houston. In this case, the rehabilitated section was located on the southeast quadrant of the urban interstate loop. Important lessons learned in the IH-610 North project were applied in the construction of this rehabilitation project. TxDOT implemented new construction controls, such as monitoring evaporation rates in real time with a portable electronic weather station and limiting paving to periods when the evaporation rate during construction was less than 0.2 lb/ft.^{2}/hr, and concrete placement was allowed only when the predicted temperature differential during the 24 hours following placement was less than 25°F. They were motivated to implement these concepts, since the adverse environmental conditions surpassing these limits were identified as the primary triggers of the IH-610 North BCO delaminations.

The 8-in.-thick CRCP section is about 4 miles long, and it included the aforementioned BCO built in 1983. The approximate rehabilitation project limits are from just east of SH 288, to just west of Telephone Rd. [30]. This project started in 1989 and was completed in 1990. It consisted of a 4-in.-thick BCO placed on 112 lane miles. The reinforcement was wire mesh and the coarse aggregate was limestone. Portland cement grout was used as the bonding agent [5, 30].

The BCO included ten experimental sections, each 400-ft. long and four lanes wide, in which several combinations of bonding agents, reinforcement types, and surface treatments were implemented for performance comparisons [36]. Substantial early delaminations occurred in the sections where a latex modified portland cement grout was used as a bonding agent, and this prompted the removal of the overlay shortly after construction. The reason for the delamination was that the grout was being sprayed too far ahead of the paving machine, allowing much of the grout to dry. Before the concrete overlay was placed the contractor applied fresh grout over the dried grout, in which the solid latex at the interface behaved as a bond-breaking layer. The BCO was replaced within 30 days, after the newly prepared (debonded overlay material removed) sections received the same treatment as the control sections (cold milling and PC grout). Aside from dismissing the use of latex as a bonding agent, another important lesson learned is that the bond failures were induced at relatively low stresses (under 50 psi), while the overlay was still in its early age. The experimental results also emphasized the importance of good surface preparation.

3.6 Houston – SH 146 and SH 225

On June 8, 2010, a field trip was made to visit two bonded concrete overlay (BCO) projects in the Houston area. The purpose of the trip was to perform visual condition surveys on the BCOs on SH 146 and SH 225.

Unfortunately, only one of the overlay surveys could be done as planned; just at the time of setting up for the survey, it was found that the 4.25 mi long BCO segment on SH 225, from the IH-610 Loop to Redbluff has been resurfaced with asphalt, making it impossible to perform the survey on this section.

The survey on SH 146 was conducted. The BCO is located in Harris County in Baytown, and its limits are from the Chambers County line to North Main St., for an approximate length of 4.5 mi. The overlay consists of a 3-in.-thick BCO on top of the existing 11-in.-thick CRCP. This BCO was designed in 1998, and it was constructed in the early 2000s.

3.6.1 SH 146 Survey

The survey consisted of a visual inspection of the outside lane of the BCO, observing the cracks and distress, and photographing some of the more interesting. Because the road was open for traffic, it was not possible to conduct this survey in more detail. The survey was conducted while driving at a very low speed in the outside traffic lane, while a cushion truck and a shadow vehicle, provided by the district, moved along behind the survey vehicle for protection. During part of the survey, there was rain, which made it hard to see the pavement at times. The rain was particularly intense during the survey of the southbound lanes; therefore, the level of detail of this part of the survey was not optimal. The survey started with the northbound sections, from North Main St. to the Chambers Co. line, and then the southbound sections followed.

The distresses consisted mainly of punchouts and minor spalls. There was a number of punchouts at the south end of the section in the southbound lanes, where the pavement showed a limited number of cracks, but some distresses had been repaired and were failing again. There were also some sections in very good condition without many cracks or distresses. Table 3.1 shows the results of the northbound sections, and Table 3.2 presents the southbound sections.

As mentioned before, some segments on the southbound side could not be surveyed thoroughly because of the rain, but the overall assessment of those is shown in the comments column in the tables. Some of these segments did not show any distress or damaged cracks.

In summary, with the exception of a few segments that are badly deteriorated, some showing severe failures, the overlay is in good condition. Photographs from the survey are presented in Appendix A-1.

Table 3.1: SH 146 survey – Northbound direction

Mileage	Distance ft	Transverse Cracks	Minor Spalls	Severe Spalls	Punchouts	Patches	Average Crack Spacing (ft)	Comments
0.0 to 0.1	528	111	1				4.8	
0.1 to 0.2	528	100	2		1	1	5.3	
0.2 to 0.3	528	84	2				6.3	
0.3 to 1.4	5808	**	**				**	2 Bridges
1.4 to 1.5	528	21					25.1	
1.5 to 2.0	2640	**	**				**	Bridge over Alexander Dr.
2.0 to 2.1	528	65					8.1	
2.1 to 2.2	528	160	1				3.3	Longitudinal joint damage
2.2 to 2.3	528	195	1	1			2.7	
2.3 to 2.4	528	154	3				3.4	
2.4 to 2.5	528	167	2	1	1		3.2	
2.5 to 2.6	528	75			1		7.0	
2.6 to End		**			1		**	
Totals		1132	12	2	4	1		

** Section where cracks and/or distresses could not be counted

Table 3.2: SH146 survey – Southbound direction

Mileage	Distance ft	Transverse Cracks	Minor Spalls	Severe Spalls	Punchouts	Patches	Average Crack Spacing (ft)	Comments
0.0 to 0.1	528	12					44.0	
0.1 to 0.2	528	**	2		1		**	Includes a bridge
0.2 to 0.3	528	**				1	**	AC patch
0.3 to 0.4	528	**				1	**	AC patch
0.4 to 0.5	528	**			2	1	**	AC patch
0.5 to 0.6	528	**					**	Good condition
0.6 to 0.7	528	**					**	Good condition
0.7 to 0.8	528	**					**	Good condition
0.8 to 0.9	528	**	1				**	Good condition
0.9 to 1.0	528	87			1		6.1	
1.0 to 1.1	528	141					3.7	
1.1 to 1.2	528	125	1				4.2	
1.2 to 1.3	528	76	3				6.9	
1.3 to 1.4	528	**					**	Joint damage
1.4 to 2.1	3696	**					**	Entrance ramp
2.1 to 2.2	528	**			4	2		AC patches. Not many cracks
2.2 to 2.3	528	**			4			
2.3 to end		**					**	Bridge
Totals		441	7	0	12	5		

** Section where cracks and/or distresses could not be counted

3.7 Houston – Beltway 8

On May 19, 2010, a field trip was conducted to visit the BCO project on Beltway 8 in Houston. The pavement of interest is a 2-in.-thick BCO with steel fibers constructed in 1996, on Beltway 8, the urban outer loop that surrounds IH-610 in Houston. The BCO project section, approximately 5.3 miles long, is located between Greenspoint Drive, just east of IH-45, and Aldine Westfield, near Houston's George Bush Intercontinental Airport. The original 13-in.-thick CRCP structure, built in 1984, experienced a severe spalling problem just a few years after construction. By 1995, when the overlay rehabilitation project was undertaken, the CRCP section was in poor condition. A CTR investigation on that pavement concluded that the reasons for the spalling were high evaporation rates and high daily temperature differentials that occurred during the construction. Falling Weight Deflection tests were performed, and core samples were extracted at the time to evaluate the structural integrity of the pavement. The tests showed that the spalling problem was only superficial, and it did not significantly affect the load-carrying capacity of the pavement, making it a good candidate for BCO rehabilitation. As a result of that study, a 2-in.-thick BCO reinforced with steel fibers was designed and constructed in 1996.

3.7.1 Condition Survey

The survey consisted of a visual inspection of the outside lane of the BCO, observing the cracks and distress, and taking photographs. Because the road was open for traffic, it was not possible to walk the section to conduct this survey in more detail. The survey was performed while driving at a very low speed on the outside traffic lane, while a cushion truck and a shadow vehicle, provided by the district, moved along behind the survey vehicle for protection. The survey started with the westbound section, from Aldine Westfield, and proceeded to Greenspoint Dr., followed by the eastbound section.

The transverse cracks appeared to be in good shape. They were not wide and the spacing seemed to be adequate. The few distress symptoms found consisted mainly of minor spalls. There was a very long longitudinal crack that extended for a few tenths of a mile on the westbound outside lane. However, this crack was narrow and despite its length, it did not seem to be associated with any additional distress. The eastbound section also showed some asphalt patches, but mainly the patches were concrete. There was a particularly long PCC patch that extended for about 45 ft. on the eastbound inside lane. Table 3.3 summarizes the results in the westbound section, and Table 3.4 presents the eastbound results.

In summary, the overlay was in excellent condition. The few spalls that were present, mainly in the westbound section, were all minor. Even though there are a handful of distress symptoms, none of them appeared to be severe. No punchouts were observed. Photographs from the survey are presented in Appendix A-2.

Table 3.3: Beltway 8 survey – Westbound direction

Mileage	Distance ft	Transverse Cracks	Minor Spalls	Average Crack Spacing (ft)	Comments
0.0 to 0.1	528	42	3	12.6	
0.1 to 0.2	528	40	3	13.2	
0.2 to 0.3	528	59	6	8.9	Longitudinal crack
0.3 to 0.4	528	34	1	15.5	Longitudinal crack
0.4 to 0.5	528	38	3	13.9	Longitudinal crack
0.5 to 0.6	528	39	1	13.5	Longitudinal crack
0.6 to 1.4	***	15		***	Hardy Bridge
1.4 to 2.1	***			***	Bridge
2.1 to 2.2	528	29	0	18.2	
2.2--	***			***	Bridge

*** Section where cracks and/or distresses could not be counted

Table 3.4: Beltway 8 survey – Eastbound direction

Mileage	Distance ft	Transverse Cracks	Minor Spalls	Average Crack Spacing (ft)	Comments
0.0 to 0.1	***		1	***	
0.1 to 0.8	***			***	Bridge
0.8 to 0.9	528	3		176.0	
0.9 to 1.0	528	33		16.0	
1.0 to 1.1	528	39		13.5	Patch, Longitudinal crack
1.1 to 1.2	528	41		12.9	
1.2 to 1.3	528	60	1	8.8	Patch
1.3 to 1.4	528	39		13.5	
1.4 to 1.5	528	33		16.0	
1.5 to 1.6	528	33		16.0	Patch (next to loops)
1.6 to 1.7	***			***	Bridge

*** Section where cracks and/or distresses could not be counted

3.8 Houston – IH 610 North

On October 28, 2009, a visual condition survey was performed on the BCO sections on IH-610 North, in Houston.

One of the earliest BCO experiences in the state, from the 1980s, was the subject of this survey; the overlay project is located on IH-610 North, between East T.C. Jester Blvd. and IH-45 (from station 207+78.37 to station 400+00). In this section, the main roadway is an eight-lane freeway, with four 12-ft. lanes in each direction, a 20-ft. median with a concrete traffic barrier and 10-ft. outside shoulders. The original pavement structure, constructed in the late 1950s, consists of an 8-in.-thick slab of CRCP on top of a 6-in.-thick cement stabilized subgrade. The BCO is a nominal 4-in.-thick CRCP constructed in 1986. At the time this survey was conducted, construction had begun already for the placement of a new pavement, so part of the overlay was already removed, and only some lanes could be observed. The surveyors took advantage of the construction closures to be able to walk the section and perform the survey within a barricaded area closed to traffic. The project staff considered it a valuable opportunity to visit this overlay to observe its performance before it was completely removed.

One of the most interesting features of this BCO was that it included ten experimental sections, each with different combinations of reinforcement types, coarse aggregates, bonding agents and existing pavement conditions (various levels of distress). The characteristics of each of those experimental sections are presented in Table 3.5. During and after the overlay construction, back in 1986, delamination occurred between the BCO and the original pavement. A study [30] of those sections at that time found most of the delaminations occurred within the first 24 hours of age and happened on segments constructed when the difference between the maximum and minimum daily temperature was greater than 25 F and the evaporation rate was greater than 0.2 $lb/ft.^2/hr$. However, those delaminations did not appear to have any effect on the overlay performance. The layout of the experimental sections is shown in Figure 3.1.

Table 3.5: Experimental section factorial (Section number)

Reinforcement		Welded Wire Fabric			Fibers		
Pavement Original Condition		No Distress	Moderate Distress	Severe Distress	No Distress	Moderate Distress	Severe Distress
BCO Coarse Aggregate	SRG	2 10	1 3	6 9			4 5
	LS	8		7			

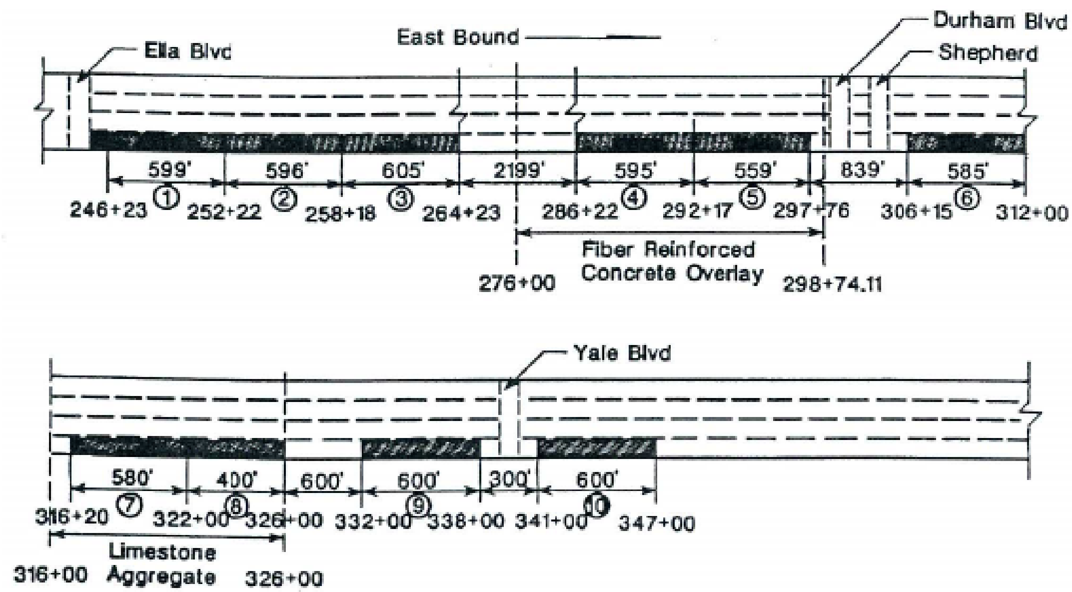

Figure 3.1: Layout of IH-610 North experimental sections

In November 2000, a condition survey was conducted as part of another TxDOT research project. That survey [50] demonstrated that the BCO performance had been excellent after 15 years of traffic. In spite of the fact that the sounding tests performed back in 1987 showed some delaminations soon after the overlay was placed, the 2000 survey showed that those delaminations had not turned into punchouts, indicating the good performance of the BCO up to that point. The number of distress issues was minimal, most of which were minor spalls.

3.8.2 Condition Survey

The 2009 survey consisted of a detailed visual inspection of the experimental sections. Transverse cracks were counted, as well as distresses and patches. The results are summarized, by section, in Table 3.6, and photographs of the survey are included in Appendix A-3.

Table 3.6: Summary of condition survey on IH-610 North

Test Section	From	To	Length (ft)	Transverse Cracks	Minor Spalls	Severe Spalls	Punchouts	Patches	Average Crack Spacing
1	246+23	252+22	599	296	4	0	0	0	2.0
2	252+22	258+18	596	220	2	0	0	0	2.7
3	258+18	264+23	605	96	2	0	0	0	6.3
4	286+22	292+17	595	32	1	5	0	21	18.6
5	292+17	297+76	559	59	3	5	0	9	9.5
6	306+15	312+00	585	83	1	0	0	0	7.0
7	316+20	322+00	580	45	0	0	0	0	12.9
8	322+00	326+00	400	20	0	0	0	0	20.0
9	332+00	338+00	600	135	0	0	0	0	4.4
10	341+00	347+00	600	63	0	0	0	0	9.5

The results indicate that eight of the ten experimental sections are still in very good condition, while the remaining two, namely sections 4 and 5, look much more deteriorated.

These two sections were constructed with siliceous river gravel aggregate, and the original PCC had a severe level of distress prior to the BCO construction. Also, those two sections were the only ones that featured fibers for reinforcement. The number and aspect of the PCC patches observed in these two sections are an indication that TxDOT had repaired these sections many times in the past. These are the only sections that feature the combination of those three characteristics: SRG, fiber reinforcement and severe PCCP distress prior to BCO placement.

It is interesting to note that there were other sections that had suffered severe distress before the BCO was constructed (Sections 6, 7 and 9), and despite such condition, the survey results shows that their performance was still excellent (only one spall in section 6, and no other distress). The difference between these successful BCOs and the BCO sections in poor condition is that the better performing ones did not use fibers as reinforcement. Survey photos are given in Appendix A-3.

3.9 Houston – IH 610 South

On July 22, 2010, a field trip was conducted to inspect two more BCO sections in Houston. Both of these BCO projects visited are on Interstate Highway 610, the urban section known as the South Loop, which is a major freeway encircling downtown Houston. These projects were deemed as some of the most important of the existing concrete overlays in the state, because they correspond to some of the oldest rehabilitations of this type that are still in service and performing well, and both of them involve experimental sections. One of them corresponds to the first BCO project in Texas, which was constructed in 1983.

The project was an experimental BCO on a 1000-ft. CRCP segment. Built in July and August of 1983, the BCO has delivered excellent performance over time, as previous condition surveys have demonstrated. It consists of five 200-ft. test segments, with several combinations of reinforcement (no reinforcement, welded wire fabric, and steel fibers) and BCO thicknesses (2 and 3 in.), all constructed on the four eastbound lanes, between Cullen Blvd. and Calais St. For the BCO construction, the surface was prepared by cold milling and sandblasting. Portland cement grout was used as a bonding agent for the vast majority of the section. The original existing pavement, constructed in 1969, consisted of 8-in.-thick CRCP on top of a 6-in.-thick cement treated subbase. Table 3.7 shows the experimental factorial for thickness and reinforcement, the variables investigated in the experimental sections [43, 47].

Table 3.7: South Loop factorial for 1983 BCO experiment

		Reinforcement Type		
		None	Steel Mat	Steel Fibers
Overlay Thickness	2 in.	□	□	□
	3 in.		□	□

A sounding survey conducted in 1990 on this section revealed some minimal delamination of the overlay [5]. Condition surveys conducted in 1996 showed little distress on the section and no major performance problems [48].

The other BCO project studied in this field trip is also on the southeast part of the IH-610 Loop. The approximate project limits are from just east of SH 288, to just west of Telephone Rd., extending for about 4 miles; this overlay is placed on either side of the 1983 experimental sections. This project started in 1989 and was completed in 1990. It consisted of a 4-in.-thick

BCO. The reinforcement was wire mesh and the coarse aggregate was limestone. Portland cement grout was used as the bonding agent. The original pavement was 8-in.-thick CRCP [30].

This BCO project included ten experimental sections, each 400-ft. long and four lanes wide, in which several combinations of bonding agents, reinforcements and surface treatments were implemented. The experimental sections were placed in the eastbound direction only. Table 3.8 shows the combinations implemented in each test section [3].

The survey for this BCO project consisted of a visual inspection of the experimental sections and of the rest of the overlay, as well, An emphasis, however, was placed on walking the experimental sections, both the 1983 and 1989-1990 BCOs in the eastbound lanes, while the remaining part of the eastbound and westbound BCO was surveyed from a vehicle traveling at low speed.

The survey started at approximately 9:30 am, on the eastbound lanes, at Calais St., where the 1989-1990 BCO experimental sections begin, and proceeded eastward. The researchers walked this segment on the inside lane while recording observations, taking photographs, and also performing sounding tests. Some representative photographs from the survey are included in Appendix A-4.

Table 3.8: South Loop IH-610 experimental factors for 1989-1990 BCO

Test Section Identifier	Date of Paving	Surface Preparation	Bonding Agent	Reinforcement
A	1/2/90	Cold Milling	PC grout	Welded Wire Fabric
1	1/2/90	Cold Milling	None	Welded Wire Fabric
2	1/2/90	Cold Milling	PC grout	Steel Fibers
3	1/2/90	Cold Milling	PC grout	Welded Wire Fabric
4	7/10/89	Light Shotblasting	Epoxy	Welded Wire Fabric
5	7/10-11/89	Light Shotblasting	Latex Modified PC grout	Welded Wire Fabric
6	7/11/89	Heavy Shotblasting	Latex Modified PC grout	Welded Wire Fabric
7	7/11/89	Heavy Shotblasting	PC grout	Welded Wire Fabric
8	7/11/89	Heavy Shotblasting	None	Welded Wire Fabric
B	7/11/89	Cold Milling	PC grout	Welded Wire Fabric

The segment between Calais St. and Martin Luther King Blvd. includes the experimental sections designated as A, 1, 2, and 3. One punchout and one patch were found in this stretch, but the overall condition was good. No delaminations were found. The other experimental sections from 1989, sections 4, 5, 6, 7, 8 and B (see Table 3.8), east of MLK Blvd., did not present any signs of distress, and their condition was excellent.

Following the survey of this group of experimental sections, the remaining part of the eastbound overlay until its limit near Telephone Rd., was surveyed from the vehicle. The overall condition was good, but there were a few distress symptoms, including several patches, some covering the full width of the lane and some half of it. There were also some longitudinal cracks and a few spalls. The westbound overlay was surveyed in a similar manner, from the vehicle traveling on the outside lane at very low speed, while being protected by the cushion truck in the back. The condition of the westbound overlay was good as well, and the presence of distress is comparable to the eastbound lanes, both in frequency and severity.

Finally, the eastbound experimental sections from 1983, which start after the Cullen Blvd. Bridge, were surveyed in detail, walking on the outside lane and conducting sounding tests. This part of the survey started around 11 am. These sections were in excellent condition, even better than the 1989-1990 experimental sections. Only one longitudinal crack, and a segment about 20-ft. long that had some delamination, were noteworthy, besides the otherwise outstanding condition of the overlay. That delaminated area was next to an exit ramp, separated from it by a wide longitudinal joint, which probably triggered the delamination.

In summary, both the experimental sections and the remaining part of the BCO were in very good condition. The 1983 experimental sections, even though they are very short, seemed to be in the best condition among the sections surveyed in this trip. The good performance of the overlays was remarkable because they have been in service for a long time, while carrying some of the heaviest and most intense traffic in the state.

3.10 Sherman – US 75 (Pre-construction)

A condition survey was conducted on a jointed concrete pavement in Sherman. Even though this is not an existing overlay, because a new bonded concrete overlay (BCO) was going to be constructed on top of the existing pavement on a section of US 75, in Sherman, it was considered a valuable project to study, as there are not many opportunities of visiting the site of a future overlay and following up through its construction. On February 10, 2010, a survey was performed on this pavement prior to the construction the BCO, to observe the distresses that existed on the original surface.

The construction site is half a mile long, just north of US 82, and it includes only the southbound direction. The overlay consists of a 7-in.-thick BCO on top of the existing 9-in.-thick jointed concrete, which is approximately 28 years old. According to the TxDOT engineer who helped CTR during this visit, Mr. Ali Esmaili-Doki, this section of pavement required patching every year, so TxDOT decided to place a BCO to solve this recurring problem.

3.10.1 Existing Pavement

A typical cross-section of the existing jointed pavement is shown in Figure 3.2, showing two main lanes with shoulders on either side.

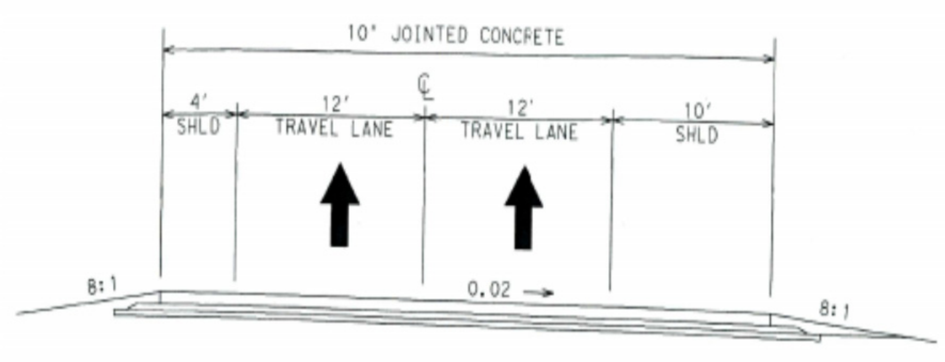

Figure 3.2: Typical section of US 75 in Sherman

3.10.2 Proposed Overlay

A typical cross-section of the BCO pavement is presented in Figure 3.3.

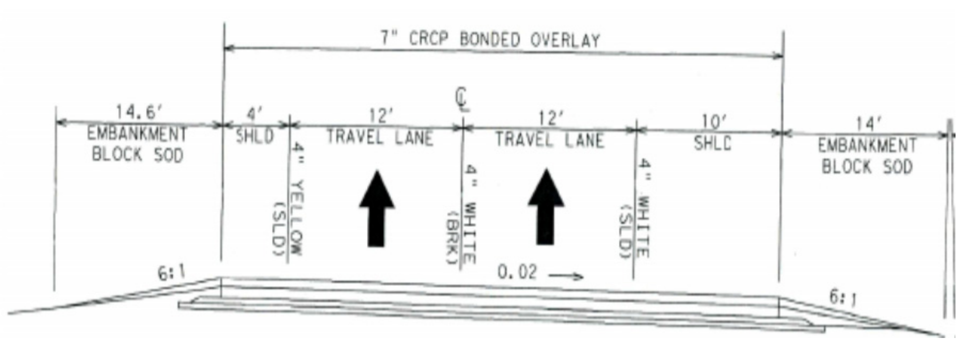

Figure 3.3: Typical section of the proposed BCO

It is not common to place a (continuously reinforced) BCO on top of a jointed pavement, so this is considered an experimental overlay. Some of its outstanding design features are as follows:

- Direct reinforcement placement: Research has shown that normal deformed reinforcing bars placed on the substrate surface will achieve the same pull-out bond strength as reinforcing placed at mid-depth of a 6-in.-thick slab. The decision to place the reinforcement directly on the substrate saved construction time and labor.
- Surface preparation: The surface was adequately cleaned by shot blasting to achieve a moderately rough texture. The surface was kept clean prior to placement of concrete. No bonding agent was used.

3.10.3 US 75 Survey

The pre-construction survey was a visual inspection of the outside lane of the half-mile-long segment from the shoulder, observing the cracks and other signs of distress, and making photographic records. Because this heavily traveled road was still open for traffic, it was not possible to conduct this survey with more detail.

The distress observed included punchouts, corner breaks, pumping stains along longitudinal joints, and open cracks. There were a few open joints, both longitudinal and transverse. It was obvious that several patches had been repaired and had failed again. There were also some slabs in good condition without cracks or distresses. More than 100 areas of distress needed to be repaired prior to BCO placement, and those areas were already marked with spray paint at the time of this survey, as can be seen in the photographs, shown in Appendix A-5.

3.11 Sherman – US 75 (Construction)

On June 12 2010, a survey was performed on the new bonded concrete overlay (BCO) on the SB half-mile-long section of US 75, in Sherman. The overlay is a 7-in.-thick BCO on top of the existing 9-in.-thick jointed concrete pavement, which was approximately 28 years old BCO was selected to solve the recurring repair problems on this section of highway.

The survey consisted of visually monitoring the construction of the outside lane of the half a mile-long BCO section and recording details with photographs. The prepared surface was checked to be sure there was no debris that could cause debonding, and to ensure that the surface ahead of the paver was kept in saturated surface dry (SSD) condition by spraying water and blowing off any standing water just before the paver covered the substrate with fresh BCO concrete. Also, quality control was performed by monitoring the construction, finishing, and curing process.

3.11.1 Surface Preparation and Placement of Reinforcements

The surface was roughened up and cleaned before the BCO placement. However, the survey team noticed that the yellow pavement stripe was still on the cleaned substrate. This raised concern that the probability of the stripe acting as a debonder at the interface, especially since it was adjacent to the longitudinal joint. Therefore, the contractor had to remove the yellow stripe and thoroughly clean the surface again before the construction began. Figure 3.4 shows the pavement before the yellow strip was removed and Figure 3.5 shows it after removal.

Figure 3.4: Prepared surface prior to the yellow stripe removal

Since the BCO was relatively thick to provide more than enough concrete cover for reinforcement bars, the bars were placed on chairs resting on top of the prepared surface of the existing pavement. Figure 3.6 shows the chairs used to elevate the reinforcement bars. The workers were asked to wet the surface before the construction began.

Figure 3.5: After the removal of the yellow stripe

Figure 3.6: Reinforcement detail

Transition area reinforcement details are illustrated in Figures 3.7 and 3.8. Transition areas typically have the highest concentration of dynamic traffic loads; so proper reinforcement is very important. If bond fails at a transition area, the failure can propagate through the concrete overlay due to repeated dynamic wheel loads. As the figures show, there is a row of bent reinforcement extruding from the prepared surface. In cases of poor or uncertain bond or in areas with specific demands (e.g., high shear or tensile stresses are serious contributors in failures), it might be necessary to strengthen the shear and tension capacity by installing shear reinforcement crossing through the interface.

Figure 3.7: Transition reinforcement detail

Figure 3.8: Transition reinforcement detail close-up

3.11.2 Placement

The slip form paving machine was used for placement of the BCO. A worker was in charge of spraying water immediately ahead of the paving machine, so that the surface stayed wet or in SSD condition. Figure 3.9 shows the worker spraying water ahead of the paver.

There were instances when concrete was placed too far ahead of the paving machine, creating risks of the concrete setting even before the paving machine reached those areas. Also, the substrate surface around the prematurely dumped pile of fresh concrete dried while waiting for the paving machine. Figure 3.10 shows the deposited concrete waiting for the paving machine to catch up, and Figure 3.11 shows the dried surface around the concrete. The contractor was asked to slow the depositing of concrete ahead of the paving machine.

Figure 3.9: A worker spraying water on the surface ahead of the paver

Figure 3.10: Concrete too far ahead of the paving machine

Figure 3.11: Dried surface around concrete

3.11.3 Finishing and Curing

Finishing and curing phases went very smoothly. The finishing machine was capable of carpet dragging, tining, and spraying curing compound, which were all done in a timely manner. To finish, wet burlap was laid on top of the finished concrete overlay to provide extra protection from excessive evaporation. Figures 3.12 to 3.13 show the finishing and curing process.

Figure 3.12: Carpet dragging, tining, and spraying curing compound

Figure 3.13: Laying down burlaps

3.11.4 Conclusion

The overall paving operation was very satisfactory. The contractor was asked to fix and adjust minor things, and he responded promptly. Surface preparation was thorough, the paving machine provided enough concrete cover, and finishing and curing was done in a timely manner. Additional construction photographs are provided in Appendix A-6.

3.12 Wichita Falls – US 281

On September 28, 2010, a field trip was made to visit the BCO section on US 281 in Wichita Falls. The pavement of interest is a 4-in.-thick BCO constructed in 2002, on US 281. The project section, approximately 3.3 miles long, is located between the Archer and Wichita County line, and the Holliday Creek Bridge. The original 8-in.-thick CRCP structure was constructed in 1969. In 2001, CTR conducted a study for the rehabilitation of that pavement, which included surveys, FWD and RDD deflection tests, as well as core sampling. The study concluded that the pavement was a good candidate for BCO rehabilitation, and the overlay was designed as a 4-in.-thick layer, with limestone coarse aggregates, steel mat reinforcement, and shotblasting as the surface preparation procedure. The overlay was constructed during June and July of 2002.

3.12.1 Condition Survey

The survey consisted of a visual inspection of the outside lane of the BCO, observing the cracks and distress, and taking photographs to document findings. The survey was performed while walking on the outside traffic lane, while a cushion truck and protection vehicles, provided by the district, moved along behind and ahead the surveyors. Besides recording the signs of distress, because of the level of protection provided by the traffic control crew, the survey team had the opportunity to perform sounding along the entire outside lane to detect any delaminated areas of the overlay. Delaminations are indicated by the characteristic hollow sound produced when a steel bar is dropped onto the pavement surface. Commonly, delaminations start at the edges of the pavement, most likely, along the longitudinal edge of the lanes, and propagate inwards from the presence of some form of distress. The survey started with the southbound lanes, from the Holliday Creek Bridge, and proceeded towards the Wichita-Archer County line, followed by the northbound lanes. The survey results are summarized in Table 3.9 (southbound), and Table 3.10 (northbound), and are illustrated in the photographs presented in Appendix A-7.

The vast majority of transverse cracks observed on the overlay appeared to be in good shape, were mostly narrow and their spacing seemed to be adequate. Some of the cracks, however, were spalled. The spalls were all minor. It was found in some cases that around minor spalls, some degree of delamination has developed along the edge of the overlay. Those instances of delamination detected through the sounding technique, were marked with spray paint with an arrow on the shoulder, as can be seen in the pictures in Appendix A-7. The presence of those delaminations did not seem to be causing any further problems in the performance of the overlay, as there were no loose pieces of concrete in any of them. There were also a few longitudinal cracks. That did not appear to be causing trouble at the time. There were a couple of areas, however, that could develop into punchouts in the future, but should not be a reason for any serious concern.

Table 3.9: US 281 survey – Southbound direction

Mileage	Cumulative Distance (ft)	Minor Spalls	Delaminations	Comments
0.0 to 0.1	528	1		
0.1 to 0.2	1056	3	1	Small delamination at construction joint (966)
0.2 to 0.3	1584	1		Diamond grinding starts at 1160
0.3 to 0.4	2112			Diamond grinding ends at 1914
0.4 to 0.5	2640	2		
0.5 to 0.6	3168			
0.6 to 0.7	3696			Bridge starts at 3620
0.7 to 0.8	4224			
0.8 to 0.9	4752			
0.9 to 1.0	5280			
1.0 to 1.1	5808	3		Bridge ends at 5420
1.1 to 1.2	6336			
1.2 to 1.3	6864		2	Small delamination at 6450 Small delamination at 6666 Bridge starts at 6819 RM 194 at 7148
1.3 to 1.4	7392			
1.4 to 1.5	7920			
1.5 to 1.6	8448			
1.6 to 1.7	8976	2		Bridge ends at 8629
1.7 to 1.8	9504	5		
1.8 to 1.9	10032	1		
1.9 to 2.0	10560	2		
2.0 to 2.1	11088	1		
2.1 to 2.2	11616	1		Punchout at joint (inside lane)
2.2 to 2.3	12144			Bridge starts at 11792
2.3 to 2.4	12672			
2.4 to 2.5	13200			
2.5 to 2.6	13728			Bridge ends at 13664
2.6 to 2.7	14256	1		
2.7 to 2.8	14784			
2.8 to 2.9	15312		1	Small delamination at 15188
2.9 to 3.0	15840			Bridge starts at 15582
3.0 to 3.1	16368			
3.1 to 3.2	16896			
3.2 to 3.3	17420			RM 196 at 17341 End of BCO at 17420

Table 3.10: US 281 survey – Northbound direction

Mileage	Cumulative Distance (ft)	Minor Spalls	Delaminations	Comments
0.0 to 0.1	528			RM 196 at 81 Bridge
0.1 to 0.2	1056			
0.2 to 0.3	1584			
0.3 to 0.4	2112	3		Bridge ends at 1812
0.4 to 0.5	2640	1	1	Small delamination at 2485
0.5 to 0.6	3168	4	1	Large delaminated area at 2876
0.6 to 0.7	3696			Bridge starts at 3478
0.7 to 0.8	4224			
0.8 to 0.9	4752			
0.9 to 1.0	5280			
1.0 to 1.1	5808			
1.1 to 1.2	6336			Bridge ends at 6235
1.2 to 1.3	6864			
1.3 to 1.4	7392			
1.4 to 1.5	7920	1		
1.5 to 1.6	8448	1	1	Small delamination at 8746
1.6 to 1.7	8976			Bridge starts at 8780
1.7 to 1.8	9504			
1.8 to 1.9	10032			
1.9 to 2.0	10560			RM 194 at 10244
2.0 to 2.1	11088			Bridge ends at 10564
2.1 to 2.2	11616			
2.2 to 2.3	12144			Bridge starts at 11956
2.3 to 2.4	12672			
2.4 to 2.5	13200			
2.5 to 2.6	13728			
2.6 to 2.7	14256			Bridge ends at 13749
2.7 to 2.8	14784			
2.8 to 2.9	15312	1	1	Small delamination at 14877
2.9 to 3.0	15840	1		
3.0 to 3.1	16368			
3.1 to 3.2	16896	1		
3.2 to 3.3	17424			
	17631			End of BCO at 17631

Throughout the entire BCO section there was one serious problem that stood out. It was a large punchout that developed in the southbound direction at a transverse joint on the inside lane only between 2.1 and 2.2 miles from the start of the overlay section. The survey, as mentioned above, was conducted on the outside lanes only, but the presence of this distress was very noticeable. The punchout extends for the entire width of the lane and should be repaired before it causes more extensive delamination.

3.13 Laredo – IH 35

On December 15, 2009, a survey was performed on the Laredo Whitetopping Project on IH-35. The segment in question is located north of Laredo, near Artesia Wells in LaSalle County, between mileposts 51 and 52. The IH-35 whitetopping section, placed on the northbound lanes only, was constructed in December of 2001 and January of 2002, and consists of a 9-in.-thick layer of continuously reinforced concrete pavement (CRCP) on top of the existing ACP, from which approximately 3 in. were milled off [35].

This project has been monitored by CTR since its construction. The fact that two other surveys have been conducted before the current one allows for an evaluation of its performance over time.

The survey this time consisted of a visual inspection of the outside lane of the mile-long segment from the shoulder, recording the location of every crack and sign of distress, while making photographic records of findings. In the next section, some of the details of the overlay construction are presented.

3.13.1 IH 35 Whitetopping Construction

Being an experimental project, several features related to the design and construction of the overlay were carefully selected to insure its success over time. The first was the time of placement for the concrete pavement. Over a number of years of observing PCC pavement performance using a rigid pavement database system and other studies, it has been found that pavements constructed under extremely hot conditions have a higher probability of developing failures, because of the increased climatic stresses, which in turn result in cracking. The adverse climatic conditions subject the freshly placed concrete to higher evaporation of water from the pavement surface, which may cause increased plastic shrinkage cracking. Furthermore, placing warmer PCC on a hot AC surface can lead to excessive thermal restraint stresses resulting from the large gradient between the PCC at hardening and overnight low temperatures [12]. Therefore, any construction during the summer months was ruled out for this project, and it was recommended to schedule the construction for the winter months.

In order to provide a structure able to carry the heavy traffic that traverses IH-35, PCC shoulders were used in lieu of asphalt shoulders. Tied PCC shoulders provide support to the edge of the slab, where the stress concentration is critical, thus, reducing the stresses and deflections in the main slab, and decreasing fatigue and damage. Also, the tied PCC shoulders are better able to carry main lane traffic during construction and maintenance operations.

Excellent performance has been experienced over the state where pavements have been constructed using coarse aggregates with low coefficient of thermal expansion and lower modulus of elasticity [54]. Generally, these characteristics are provided by coarse aggregates from limestone. Hence, considering the availability of aggregates in the area, the specification required that a limestone source be used, establishing a limit design value of 5.5×10^{-6} in./in./°F for the coefficient of thermal expansion.

Another critical design feature was the requirement for a carpet drag finish in lieu of tining. Tining has evolved as the standard finishing for PCC pavements, and has generally been required by the FHWA since the 1970s. The need for tining originated from efforts to eliminate hydroplaning on concrete pavements in wet weather conditions. Then it was promoted to help with skid resistance where concrete pavements had polished to a glass-like finish [55]. In Texas, at the same time that some early pavements were tined, limestone fines had been eliminated from use in PCC paving mixtures by the use of an acid insoluble test.

In parallel to this development, several extensive studies in Texas have found a major loss of pavement performance due to spalling that occurs as a consequence of the delay that the tining operations cause in the placement of the curing compound [54, 56]. When the pavement is tined, the curing compound is not applied immediately, because the concrete is allowed to take its initial set so that the surface can be properly tined. During this period there may be a considerable loss of moisture in the pavement, and consequently, an increase in the incidence of plastic shrinkage cracking, which in turn, may result in spalling. Studies comparing pavements constructed with tining and normal carpet drag finish have found that the accident rates are similar for both surface finishes when the limestone fines had been eliminated. Thus, considering the unique characteristics of the Laredo District, where rainfall is limited, carpet drag was specified as the desired finish for the whitetopping overlay.

3.13.2 Condition Survey Results

The existing transverse crack locations on the outside lane were recorded as seen and measured from the shoulder. Figure 3.14 shows the crack spacing distribution.

The average crack spacing was 7.8 ft., with a standard deviation of 1.6 ft., and a coefficient of variation of 21.2%. These numbers indicate that, regarding crack spacing, the section is behaving as designed.

To assess the crack spacing pattern throughout the section, the one-mile stretch of road was divided into 100-ft.-long stations, and the average crack spacing for each of these stations was computed. Figure 3.15 illustrates the average crack spacing by station. For the purposes of the survey, Station number 1 starts at MP 51, at the southernmost end of the whitetopping section; station 53 ends at MP 52, at the northernmost end of the section.

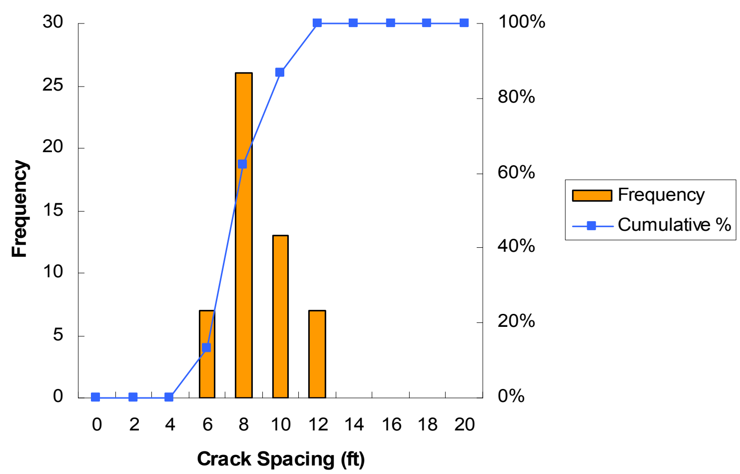

Figure 3.14: Cracking spacing distribution

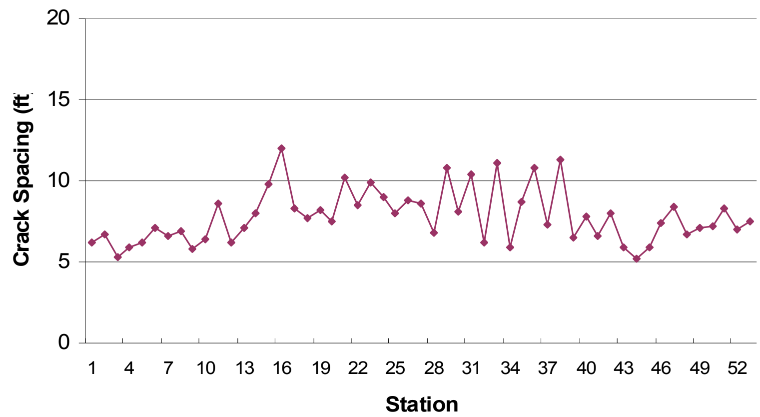

Figure 3.15: Average crack spacing by station

The plot shows that almost all of the average crack spacings are between 5 and 10 ft. This is a normal pattern for CRCP sections. In regards to distress, five punchouts and two spalls were found. Besides the punchouts, there were several areas that had the potential for developing into punchouts, given the pattern and proximity of the cracks. On the other hand, there were many cracks that appear in very good shape. The pictures in Appendix A-8 show all of the punchouts along with some areas with potential for developing punchouts, as well as some stretches in excellent condition.

There is a segment of the project that has been textured with diamond grinding. This segment is about 500 ft. long, and within it there is an area where a few traffic-counting loops have been installed. Notice the exposed limestone aggregate in the diamond-ground segment, particularly in Pictures 11 and 12. All these features are also illustrated in the Appendix A-8.

3.13.3 Previous Condition Surveys

Two previous surveys were conducted on this pavement in its initial life stages [35]. These surveys, along with the current one, present the opportunity to observe the deterioration of the pavement over time, by analyzing the crack patterns, and the appearance of distresses.

Shortly after the whitetopping section was opened to traffic, on February 8, 2002 a condition survey was conducted on the outside lane of the section. On that occasion, the average crack spacing was found to be 12.2 ft. Most of the crack spacings were close to the mean, i.e., between 10 and 14 ft., thus, there was not much variability, as confirmed by the standard deviation of 1.3 ft. The low coefficient of variation (11.4%) also characterizes the crack spacing distribution closeness to the mean.

In May 2003, a new condition survey was conducted. As expected, a few more cracks appeared after the previous survey. Nonetheless, the results continued to be excellent; the mean crack spacing was 9.2 ft., the standard deviation was 1.3 ft., and the coefficient of variation was 14.6%. Figure 3.16 illustrates the cumulative frequency distributions obtained in the three surveys. No distress was found during the previous two surveys.

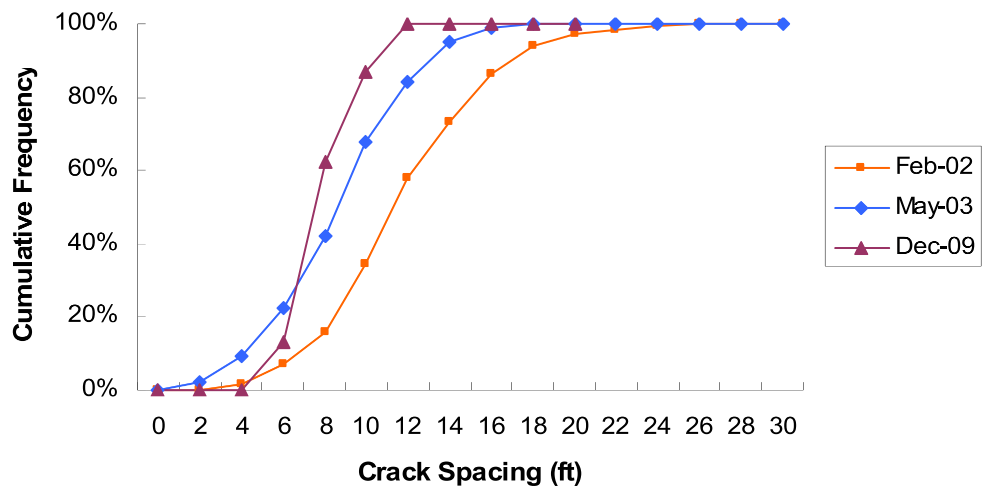

Figure 3.16: Crack spacing distribution

Table 3.11 summarizes the crack spacing results for the three surveys. It can be seen that the crack spacings are getting smaller and that the variability has increased over time. With the two previous surveys occurring in the early stages of the pavement's life, and the current survey being performed more than six years after the last one, it can be concluded that the crack spacing has stabilized.

Table 3.11: Summary of crack spacing results

	Feb-02	May-03	Dec-09
Crack Spacing (ft)	12.2	9.2	7.8
Standard Deviation (ft)	1.3	1.3	1.6
C. of Variation (%)	11.4	14.6	21.2

A comparison of the changes of crack spacing with time can be seen in Figure 3.17, in which the average crack spacings by station are plotted for the three surveys.

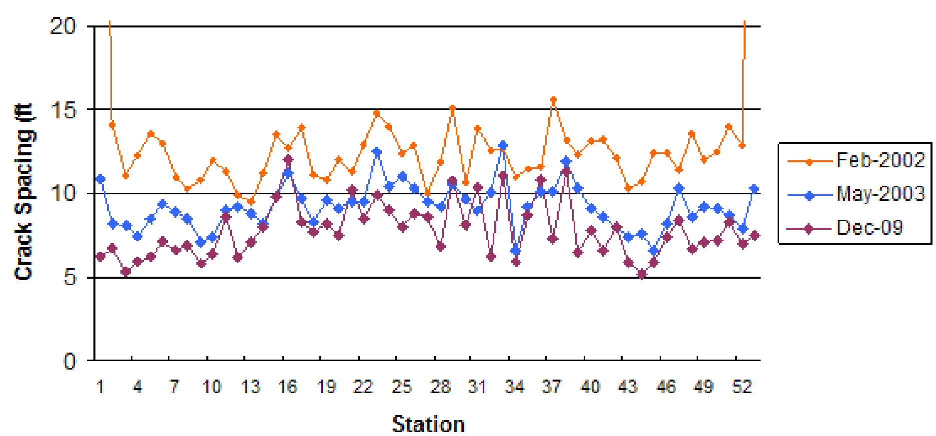

Figure 3.17: Average crack spacing with time by station

During the two previous surveys the condition of the section was excellent. The cracks were narrow, none of them was spalled, and the section did not show any early age distresses. There were no punchouts. Another indication of the good condition of the pavement at the time of those surveys was the crack pattern: all the cracks were transverse, none of them were meandering. On this occasion, a few meandering cracks were found, and the observance of distress shows some signs of deterioration, but the condition is still very good, and those types of distress, even though some of them are severe, are normal, given the type of traffic that the section carries.

The traffic forecasts obtained from the District at the time the rehabilitation was designed indicated that the highway will carry approximately 50 million ESALs in the 30-year design period following the placement of the whitetopping overlay.

3.13.4 Discussion and Recommendations

The previous two surveys indicated that the pavement was in excellent condition. No early-life distress symptoms were found, and this could be attributed to important design and construction factors at the time the overlay was constructed. The following are the most important factors that are believed to have contributed to the successful performance of the whitetopping:

- The decision to use a low coefficient of thermal expansion coarse aggregate (limestone) resulted in a low coefficient of thermal expansion concrete, which in turn reduced climatic distresses and cracking.
- The placement occurred during low temperature months, i.e., December and January, also minimizing climatic stresses and cracking.
- The quick application of curing compound (which could be accomplished because of the elimination of tining).

The distress found in the current survey can be considered normal, and the overall condition is still very good. It seems that the crack pattern has stabilized, considering that the pavement is eight years old. The existing distress has most likely occurred as a result of the heavy traffic that the pavement carries. The few punchouts need to be repaired in the near future to prevent further deterioration. Those few locations will need full-depth repairs. Further evaluation to assess the extent of the need for full-depth repairs at those locations will include sounding, which could indicate the possibility of delaminated areas in the vicinity of the punchouts. An even more comprehensive evaluation could be done by the measurement of deflections. Regarding the spalls, it seems that the few existing spalls are of low severity, and will not affect the ride quality, so those may not require repair for the time being, or perhaps some proper patching could be enough for those repairs in the future.

3.14 El Paso – IH 10

On October 20 and 21, 2010, CTR made a field trip to the site of the BCO section on IH-10 in El Paso. The overlay is significantly thicker (6.5 in.) than other BCOs in Texas studied under this project. The project was intended as an expedited BCO when it was constructed. Between Franklin St. Bridge and Missouri St. Bridge in downtown El Paso lies a segment of IH-10 known as the “depressed section” because it goes from four lanes in each direction to three lanes without a decrease in traffic [57]. It has heavy traffic and the District asked the researchers to conduct the survey after 10 pm, so that the traffic would be less intense when lane closures were made.

The original pavement section consisted of 8-in.-thick CRCP built in 1965; a feasibility study recommended that this section could be rehabilitated with a BCO in 1993 [57]. In June and July of 1996, a segment about three-quarters of a mile long in each direction was overlaid with a BCO.

The overlay was planned as an expedited BCO [58, 59], which means that mixture proportioning for paving methods were designed to reduce the normal time between placement and opening the lanes to traffic. With this, the overall cost of the project would have been reduced and the burden to the public originated by lane closures and detours would have been diminished.

However, in spite of the planning and research invested in the project, problems with the concrete mixture resulted in the delamination of most of the eastbound and some of the westbound BCO. Shortly after construction, some delaminations were identified during the extraction of core samples from the pavement. Coring and seismic tests confirmed the severity and extension of the delaminations. The comprehensive investigation that followed these events identified the high amount of water lost by the concrete before the curing compound was applied as a major cause of the debonding problem. A number of factors contributed to these unusual moisture losses from the concrete: the delay in applying the curing compound in conjunction with very high evaporation rates and inadequate surface preparation (not wetting the substrate to SSD). . Additionally the mixture had low water content ($w/c < 0.30$) to begin with, because of the higher early strength requirement of an expedited BCO, and this resulted in a stiff mixture. Then, the surface of the existing pavement slab was not dampened before placing the overlay, which caused moisture losses through the bottom of the slab. (To prevent water loss into the substrate, the substrate surface should have been prepared by spraying water on it before pouring the concrete [45]. Additionally, the cement properties changed from the time when the mixture design was originally approved to the time of placing the overlay. All these issues affecting critical water content combined to create a very stiff, very strong overlay with very little adhesion to its substrate.

The BCO had to be repaired by means of injecting epoxy into spaced holes in order to bond or re-bond the delaminated overlay to the substrate. The repair work took three weeks to complete, and Falling Weight Deflectometer (FWD) tests confirmed the success of the remedy. However, subsequent pullout tests indicated that some areas if the interface were not completely filled with epoxy, and some locations had deflections higher than expected, so it was considered at the time that those areas were still debonded [45]. These observations from shortly after the epoxy repairs were completed support the results observed in this survey.

3.14.1 Condition Survey

For the visual inspection of the overlay in both traffic directions transverse cracks were counted, as were distress locations and types and patches. Lane closures allowed the researchers to walk on the outside lanes in each direction, while conducting sounding tests to detect delaminations. The survey of the westbound direction started at 10:30 pm, and the eastbound direction started at 12:45 am. The results are summarized in Tables 3.12 and 3.13, and some representative photographs of the survey are included in Appendix A-9.

Table 3.12: Summary of westbound condition survey on IH-10

Mileage	Distance ft	Transverse Cracks	Minor Spalls	Patches	Average Crack Spacing (ft)	Comments
0.0 to 0.1	528	84			6.3	
0.1 to 0.2	528	63		1	8.4	Small PCC patch with small delamination adjacent
0.2 to 0.3	528	83			6.4	Longitudinal crack. Two small delaminated areas
0.3 to 0.4	528	93			5.7	Longitudinal crack. Three small delaminated areas
0.4 to 0.5	528	63			8.4	Two small delaminated areas
0.5 to 0.6	528	73			7.2	One small delaminated area
0.6 to 0.7	528	82	1		6.4	Three small delaminations and one large delamination
0.7 to 0.73	160	12			13.3	Two small delaminated areas

Table 3.13: Summary of eastbound condition survey on IH-10

Mileage	Distance ft	Transverse Cracks	Minor Spalls	Patches	Average Crack Spacing (ft)	Comments
0.0 to 0.1	528	67	4		7.9	
0.1 to 0.2	528	53	4	1	10.0	PCC Patch, one small delamination
0.2 to 0.3	528	86			6.1	One small delaminated area
0.3 to 0.4	528	85	5		6.2	
0.4 to 0.5	528	103	2		5.1	
0.5 to 0.6	528	99			5.3	
0.6 to 0.7	528	71	1		7.4	
0.7 to 0.75	264	23		3	11.5	PCC Patches at the joint at the end of BCO, some delamination there

The overlay was in very good condition. This may be due to the fact that the cohesive strength and stiffness of the overlay were designed for very high early strength requirements, and this often results in very high ultimate strengths at full maturity. There were only a few patches, and several spalls, which were all very minor. The tables indicate the presence of several delaminated areas. However, those delaminations do not seem to be causing performance problems to the BCO, since they were not related to any distresses on the surface. None of the delaminations spanned the entire width of the lane. Usually, they start at the longitudinal edge of the slab and extend toward the center. These might be the same delaminated areas found after the repairs were completed in 1996. It seems that no subsequent deterioration has occurred, and regardless of their presence, the overlay has performed well during its service life thus far. A similar case occurred with the BCO on IH-610 North, in Houston, which showed the presence of early-age delaminations, but in spite of them, it continued to perform well without any signs of further deterioration [30].

The survey results also indicate that there were no punchouts, and the crack spacing was adequate. The cracks were all narrow. The delaminations present at the end of the BCO in the eastbound direction, an area in which there are several small PCC patches too, seemed to be related to the joint and the adjacent bridge, rather than being an overlay problem. From the results of the survey, it seemed that the BCO still has years of service life ahead.

Chapter 4. Laboratory Investigation

This chapter reports on the activities and results of the laboratory investigation of relationships between portland cement concrete overlays (herein referred to as concrete overlays) and the critical physical properties of typical overlay mixture constituents. The investigation methods were based on the information and data gathered from literature reviews and from forensic reports and condition surveys on existing Texas overlays. The main objective of this investigation was to determine from a selected group of constituent concrete overlay materials and in what proportions yielded the best performance. It is anticipated that this information will then be used to produce longer lasting concrete overlays for Texas highways.

4.1 Introduction

From the information gathered from literature reviews and condition surveys, candidate materials for concrete overlays were identified and characterized. A factorial matrix was constructed to study the interaction between the materials. A number of tests was performed and analyzed to determine performance limits that can be used to specify which materials are to be used in concrete overlays and at what levels.

4.2 Selection of Candidate Materials

Potential materials that may be suitable for concrete overlay constructions were identified from the literature review and conditions surveys.

4.2.1 Cement

A typical Type I/II portland cement that meets ASTM C150 -11 "Standard Specification for Portland Cement" was obtained from Texas Industries, Inc. (TXI) in Bridgeport, Texas. This cement had a Blaine fineness of 375 m^2/kg. The Na_2O_{eq} and SO_3 content are 0.41 and 2.7%, respectively. The cement had the following Bogue composition: C_3S = 63.2%, C_2S = 10.7, C_3A = 6.6% and C_4AF = 9.4%. The single drop test yielded a w/f = 0.580 and a packing density of 0.634. Unless concrete overlay is expedited or prone to sulfate attack, Type I/II is typically adequate for concrete overlay purpose because this type of cement develops less heat of hydration avoiding many problems associated with high temperature development during hydration.

4.2.2 Coarse Aggregate (CA)

A locally available dolomitic crushed limestone that conformed to the requirements found in Item 421 and 360 in TxDOT specification was selected [63]. The maximum nominal size of the CA was 1 in. which is a commonly used size for 3 in. or thicker concrete overlays.

4.2.3 Fine Aggregate (FA)

Since there was no specific requirement for the selection of fine aggregate found in the literature review, locally available Colorado River (siliceous) sand that meets TxDOT Standard Specifications Items 421 and 360 was selected.

4.2.4 Fly Ash

Class F fly ash "Legs" was selected to determine the effects of its integration in concrete mixtures. Compared to Class C fly ash, Class F fly ash typically generates less heat of hydration. Use of fly ash can improve workability, fiber distribution, and reduce shrinkage. The properties of "Legs" are provided in section 4.3.1.

4.2.5 Reinforcements

Due to their recent increased usage in concrete overlays applications around the country, several fibers suggested by TxDOT were selected for use in this study. Types of fibers included four macro synthetic types, one micro synthetic type, and one steel fiber type. Fibers were also blended to determine the combined effects in shrinkage control.

In addition to fibers, No. 4 reinforcement bars and No. 6 wire mesh were used for a customized restrained shrinkage experiment. Usage of reinforcement bars is the traditional way of controlling shrinkage cracks in concrete pavement construction. However, the installation of reinforcement bars is a time consuming and labor intensive process. Wire mesh has frequently been used as an alternative method to control shrinkage cracks in the field.

4.2.6 Admixtures

Admixtures vary for every project. Since the focus of the research was not to study admixtures and their effects, only a water reducing agent, WRDA 82 (manufactured by Grace), was used.

4.2.7 Bonding Agents

In bonded concrete overlays or whitetoppings (under normal placement conditions), the performance is shown to as good or better, if no bonding agent is utilized, as long as the surface has adequate texture and is clean and dry (but brought to SSD just before the overlay placement) and free of dust, white (contaminated) water, and other debris. Therefore, no bonding agents were utilized in this research.

4.3 Characterization of Candidate Materials

In order to establish the properties and practicality of using the candidate materials, a series of laboratory tests has been performed. Candidate materials have been characterized, passed through a screening process, and used in making concrete specimens, which are tested for performance limits.

4.3.1 Materials Properties

The following are properties of materials selected for the research:

- Cement: Type I/II from TXI in Bridgeport.
    - Specific Gravity = 3.15.
- Fly ash: Class F "Legs (name of the plant)."
    - Chemical and physical properties given in Table 4.1.

Table 4.1: Chemical and physical properties of "Legs" fly ash

Sample ID	Legs
Lab Sample #	4697-01
CHEMICAL TESTS	Results
Silicon Dioxide (SiO_2), %	56.18
Aluminum Oxide (Al_2O_3), %	20.37
Iron Oxide (Fe_2O_3), %	6.77
Sum of SiO_2, Al_2O_3, Fe_2O_3, %	83.32
Calcium Oxide (CaO), %	9.95
Magnesium Oxide (MgO), %	2.55
Sulfur Trioxide (SO_3), %	0.53
Sodium Oxide (Na_2O), %	0.47
Potassium Oxide (K_2O), %	1.08
Total Alkalies (as Na_2O), %	1.18
PHYSICAL TESTS	Results
Loss on Ignition, %	0.19

- CA: Dolomitic crushed limestone.
    - Specific gravity = 2.79.
    - Absorption = 0.56%.
    - TxDOT grade No. 4. (Grading is provided in Appendix B-1)
    - CTE = 3.35x10^6 in/in/ºF
- FA: Colorado siliceous River sand.
    - Specific gravity = 2.59.
    - Absorption = 0.70%.
    - TxDOT grade No. 1. (Grading is provided in Appendix B-1)
    - Fineness modulus = 2.62.
- Water reducer: WRDA 82 (ASTM C494 – 10a "Standard Specification for Chemical Admixtures for Concrete": Type A).
- Fibers – Macro Synthetic.
    - Tuf-Strand SF (TSSF) – Polypropylene.
    - MasterFiber MAC470 (MAC470) – Polypropylene.
    - Performax Fiber (PF) – Polyolefin fibrillated.
    - Fibermesh 300 (F300) – Polyolefin fibrillated.
- Fibers – Micro Synthetic.
    - Grace MicroFiber (GMF) – Polypropylene.
- Fibers – Steel.

- Novocon XR (NXR).

In Table 4.2, the dosage rate (unless stated otherwise) and property for each fiber are provided. The dosage rate was recommended by Ryan Barborak, TxDOT CST, based on his experience with the fibers. TxDOT determined the dosage rate to achieve adequate workability. In Figure 4.1, pictures of all the fibers are provided.

Table 4.2: Fiber dosage rate and properties

	TSSF	GMF	MAC470	PF	F300	NXR
Dosage, lb/CY:	4	1.5	10	4	3.5	50
Specific Gravity:	0.92	0.91	0.92	0.91	0.91	N/A
Aspect Ratio:	74	N/A	37	N/A	Graded	34
Tensile Strength, ksi:	87 - 94	N/A	80	N/A	N/A	140 - 180

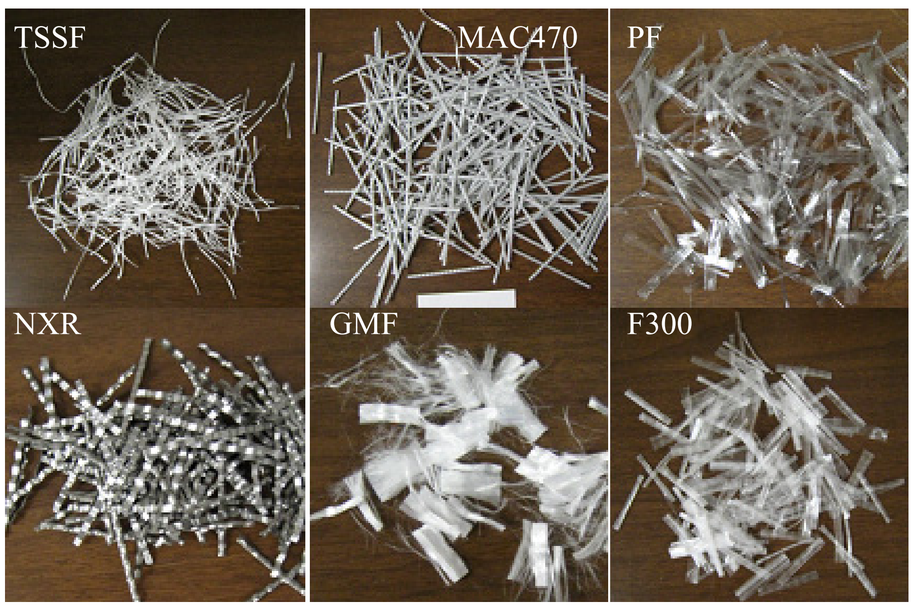

Figure 4.1: Fibers used in the research

4.3.2 Flow Table for Screening Process

The flow table experiment was performed in accordance with ASTM C1437 – 07 “Standard Test Method for Flow of Hydraulic Cement Mortar.” The goal was to explore how the different materials used in the research affect workability and so help researchers eliminate the mixture proportions that performed poorly (i.e. bad workability and would be difficult/if not impossible to mix, cast, and/or place). Since the research involved the addition of fibers, understanding the interaction between fibers and different aggregate ratios and cement contents was one of the objectives of the experiments. One liter of mortar per design mixture was made in accordance with ASTM C109 “Standard Test Method for Compressive Strength of Hydraulic

Cement Mortars” with modifications to accommodate a water reducing agent and fibers. The following are the test variables for each fiber:

- Cement: 658, 611, 564, and 517 lb (equivalent to7, 6.5, 6, and 5.5-sack cement factors, respectively).
- FA/(FA+CA): 0.45, 0.40, and 0.35 (by weight).
- Maximum manufacturer recommended dosage of WRDA 82.

The basic approach was to determine the lowest cement content that can be used before reducing the workability to a predetermined level based on the flow table. As seen in Figures 4.2 through 4.4, when the FA content was increased or the cement content (paste content for a fixed w/c) was decreased, the workability was reduced. At the highest aggregate ratio and at the lowest cement content, the workability was below the acceptable limit (the red dotted line). The acceptable limit was decided based on the experience that mortars that produced a flow table number of 110 and below gave poor slump values for pavement mixtures.

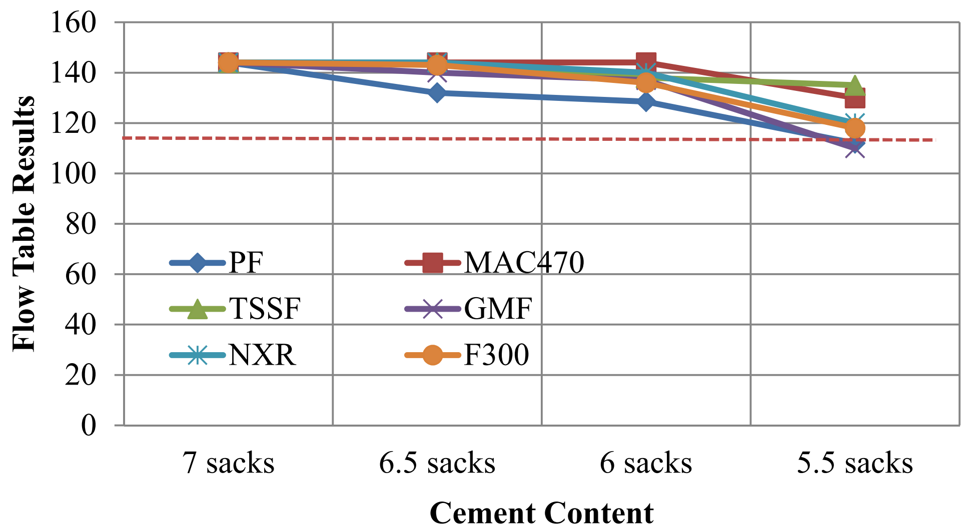

Figure 4.2: Workability comparison at FA/(FA+CA) = 0.35

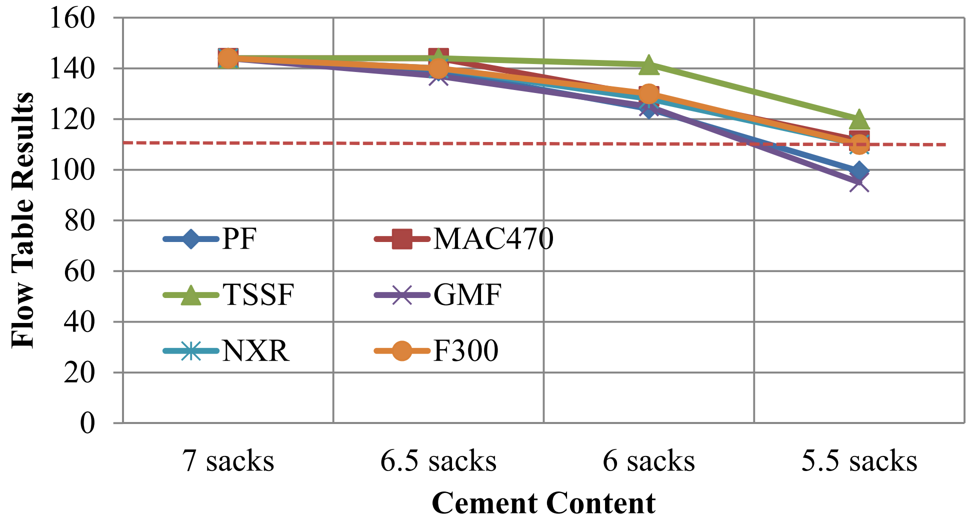

Figure 4.3: Workability comparison at FA/ (FA+CA) = 0.40

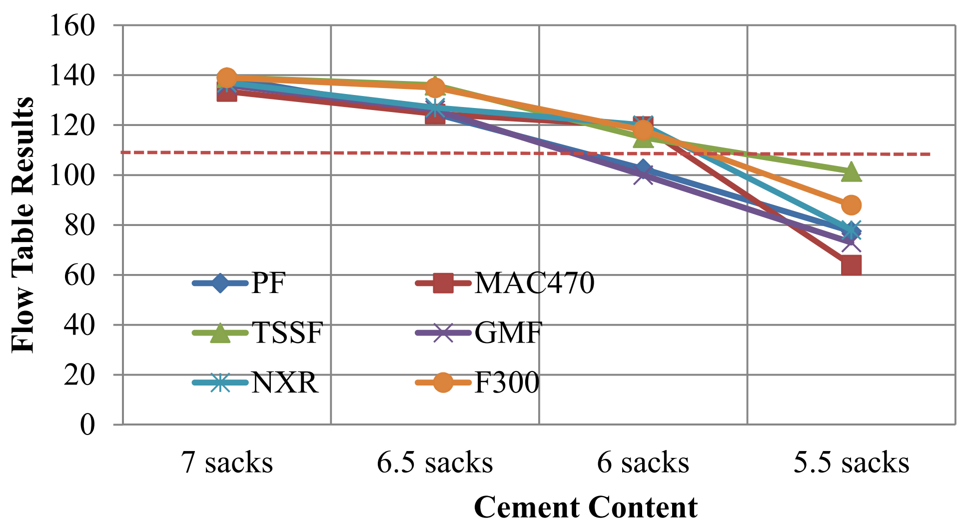

Figure 4.4: Workability comparison at FA/ (FA+CA) = 0.45

Results indicated that the equivalent 517 lbs (5.5 sacks) of cement content performed poorly, even with the maximum manufacturer dosage of WRDA 82. With the same mixture, double and triple dosages of WRDA 82 were added to see the effects. HRWA was not used because HRWA is not typically used in concrete overlay construction. As shown in Figure 4.5, the flow did not increase. Based on this fiber screening experiment, it was decided that 517 lb (5.5 sacks) of cement content alone will result poor workability; therefore, the test variable was removed.

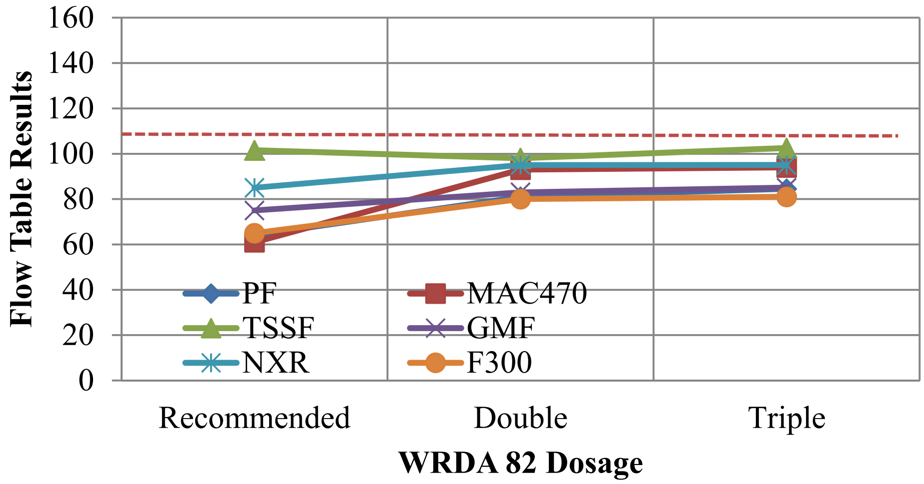

Figure 4.5: 5.5 sack mixture with different WRDA 82 dosage

The comparison of workability between the fibers shows that PF produced the least workable mixtures. This is likely attributed to the high surface area for a given weight compared to the other two fibers. PF is designed to shear into tiny strands of fibers once it is introduced into the mixer causing its surface area to increase.

Another interesting discovery during this part of the research was that fibers tend not to blend into the concrete mixture until water reducer was added. As Figure 4.6 shows, white strands of fibers (TSSF) were acting as separate inclusions in the mixture. However, once the water reducer was added, the fibers blended into the mixture, as shown in Figure 4.7, with increased workability. It seemed as if the water reducer acted as a wetting agent to help coat fiber strands with the mortar.

Figure 4.6: Mortar with fiber before adding WRDA 82

Figure 4.7: Mortar with fiber after adding WRDA 82

4.3.3 Factorial Matrix

After the screening process, a matrix was formulated. Instead of formulating a full matrix system, a design of experiment approach using a face-centered cubic (FCC) design was used to develop the mixture proportions.

Three factors were used for the FCC design and the design was augmented with axial points. This type of approach reduces the number of tests without sacrificing the accuracy of data. A FCC design is good to use when the critical factors which affect a response are known, and the region of interest of where the factors influence the response have been identified.

The results from analyzing a FCC design can be used to (1) determine the factor levels that will simultaneously satisfy a set of desired specifications, (2) determine the optimum combination of factors that yield a desired response, (3) achieve a quantitative understanding of the system's behavior over the region tested and/or (4) determine how a specific response is affected by changes in the level of the factors over the specified levels of interest. The advantage of using this approach is that it is a quadratic model and will provide information about the main effects, factor interactions, as well as the domes and basins in a model.

Three factors—SCM content, fine aggregate-to-total aggregate ratio, and cement content—were examined at three levels each. The dosage level for each material was selected in following manner.

- Class F fly ash.
  - 0%.
  - 25%.
  - 50% - to explore the possibility of high volume fly ash in concrete overlays
- FA/ (FA+CA).
  - 0.35 – increased coarse aggregate content.
  - 0.40 – typical aggregate ratio.

- 0.45 – increased fine aggregate content.

- Cement content.
    - 564 lb (6 sacks).
    - 611 lb (6.5 sacks).
    - 658 lb (7 sacks).

The high level and low level for each factor in un coded units is given in Table 4.3 and in coded units in Table 4.4. Thus +1 represents the high level, -1 represents the low level, and 0 represents a level that is at the center of the high and low levels. The axial point values for a factor were selected to correspond with the center point level. For example, for fly ash replacement, the high level was 50%, low level was 0%, and the center point level (and axial point level) was 25%.(Detailed concrete mixture proportions for the mixtures are provided in Appendix B-1.)

Table 4.3: Mixture proportions in uncoded units

Run	Fly Ash (%)	FA/(FA+CA)	Cement Cont. (lb)
1	0	0.35	564
2	50	0.35	564
3	0	0.45	564
4	50	0.45	564
5	0	0.35	658
6	50	0.35	658
7	0	0.45	658
8	50	0.45	658
9-center	25	0.40	611
10-center	25	0.40	611
11-center	25	0.40	611
12-axial	0	0.40	611
13-axial	50	0.40	611
14-axial	25	0.35	611
15-axial	25	0.45	611
16-axial	25	0.40	564
17-axial	25	0.40	658

Table 4.4: Mixture proportion in coded units

Run	Fly Ash	FA/(FA+CA)	Cement Cont.
1	-1	-1	-1
2	+1	-1	-1
3	-1	+1	-1
4	+1	+1	-1
5	-1	-1	+1
6	+1	-1	+1
7	-1	+1	+1
8	+1	+1	+1
9-center	0	0	0
10-center	0	0	0
11-center	0	0	0
12-axial	-1	0	0
13-axial	+1	0	0
14-axial	0	-1	0
15-axial	0	+1	0
16-axial	0	0	-1
17-axial	0	0	+1

4.4 Tests

In order to establish the properties and practicality of using the candidate materials, a series of laboratory tests was performed. Candidate materials were previously characterized, passed through a screening process, and tested for performance limits. For each test, a factorial matrix was performed. In addition, effects of using fibers in the mixture were experimentally determined. Using a constant mixture design, a control specimen and specimens with different fibers were made for each test. The mixture design is provided in Appendix B-2.

4.4.1 Compressive Strength

The compressive strength of concrete is a widely utilized parameter that can help quickly determine whether a given concrete is capable of withstanding typical traffic loads. The compressive strengths were determined in accordance with ASTM C39 – 10 "Stand Test Method for Compressive Strength of Cylindrical Concrete Specimens." For each mixture, six 4-in. by 8-in. cylinders were made. Seven-day and 28-day strengths were determined.

The cylinders were cured for 24 hours in the mixing room, which is kept constantly at 72ºF. For the first hour, they were cured uncovered. After an hour, they were covered with wet burlap to prevent any further evaporation of water. After 24 hours, they were removed from the plastic molds, labeled, and transferred to the moist room until the time of testing, which is kept at 72ºF and at 100% humidity.

The compressive strength tests were performed using a Forney universal load machine. The cylinders were loaded at an average rate of 30,000 lbf/min. Each cylinder was properly centered in the testing machine and neoprene pads were replaced at regular required intervals.

4.4.2 Flexural Strength

The flexural strengths were determined in accordance with ASTM C78 – 10 "Standard Test Method for Flexural Strength of Concrete." For each mixture, six 4-in. by 4-in. by 14-in. beams were made. 7-day and 28-day strengths were measured.

The beams were cured for 24 hours in the mixing room. For the first hour, they were cured uncovered. After an hour, they were covered with wet burlap to prevent any further evaporation of water. After 24 hours, they were removed from the steel molds, labeled, and transferred to the moist room until the time of testing. The flexural strength tests were performed using a third-point set-up in a Forney machine. The beams were loaded at an average rate of 6,000 lbf/min.

4.4.3 Length Change

Length change, due to drying shrinkage, was measured in accordance with ASTM C157 – 08 "Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete." The specimens were 3-in. by 3-in. by 11.25-in. beams with Humboldt pins on each end face.

The sides of the molds were oiled to facilitate removal. Oil was applied before putting the pins at the end faces to ensure that the concrete would bond to the pins. When placing the pins, care was exercised to prevent the pins from getting coated with oil. The equipment for measuring the changes in length was manufactured by Humboldt and had a resolution of 0.0001 in.

Measurements of drying shrinkage were taken at the following stages:

- Casting: soon after mixing, the concrete was poured into specimen beams. The beams were cured for 24 hours in the fog room.
- Demolding: after 24 hours, the specimens were carefully demolded and transferred from the fog room to limewater. Measurements were taken after the specimens had been submerged for 30 minutes (1st reading). The lime water in which the specimens were submerged to prevent leaching of cement paste constituents was made of approximately three cups of lime powder for each 5-gallon bucket of water or until supersaturated.
- Transferring: the specimens were left submerged in the lime water for six days. One week after the specimens were cast, they were transferred to the environmental chamber with a relative humidity of 50% and a temperature at around 75°F (2nd reading).
- Measurements: Length change measurements were taken at 4, 7, 14, 28, 56, and 112 days after the specimens were taken outside of the lime water.

4.4.4 Modulus of Elasticity

The modulus of elasticity was determined in accordance with ASTM C469. – 10 "Standard Test Method for Static Modulus of Elasticity." The modulus of elasticity was measured at seven-day and 28-day using specimens later tested for compressive strength. One of the three 4-in. by 8-in. concrete cylinders was loaded to 40% of its ultimate load. The other two cylinders were strain gauged and tested up to failure.

4.4.5 Average Residual Strength (ARS)

One of the benefits of incorporating fibers into concrete is their ability to bridge and carry load after the concrete cracks. The distributed fibers act as tensile-load carrying elements and their quantifiable after-crack strength is called residual strength.

The average residual strengths (ARS) were determined in accordance with ASTM C1399 – 10 "Standard Test Method for Obtaining Average Residual-Strength of Fiber-Reinforced Concrete" with a modification to the apparatus and procedure using four-point bending. For each type of fibers tested, five 4-in. by 4-in. by 14-in. specimens were made using cast-in-place method. It was made sure that the specimens were in 100% humidity until the specimens were ready for testing at 7 days. The test was performed at TxDOT Materials Lab using their modified test setup, as shown in Figure 4.8.

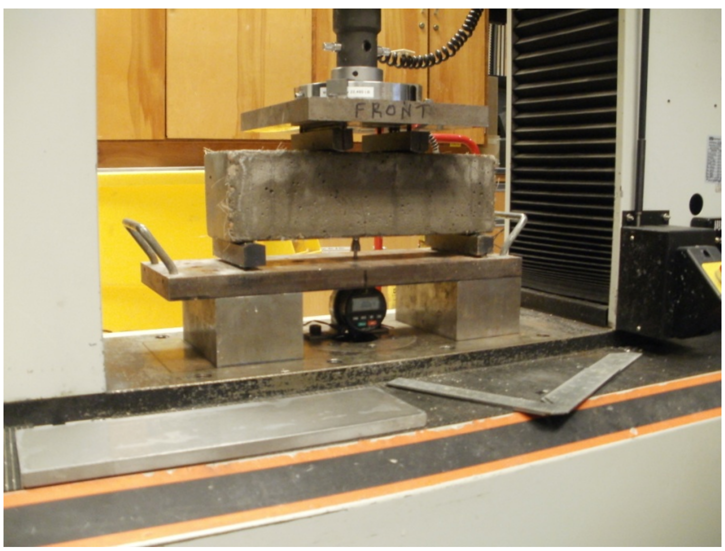

Figure 4.8: TxDOT ARS setup

The specimen was loaded until a TxDOT-specified 0.020-in. deflection was reached instead of 0.008 in. as specified according to ASTM C1399. The reason for the modification in procedure was that a majority of the specimens did not crack when a 0.008-in. deflection was reached. Based on previous experience, the first crack occurred before reaching 0.020-in. deflection on a majority of the specimens tested.

After the first crack, the specimen was unloaded, and the steel plate was removed. The specimen was then reloaded and measured for ARS at 0.020, 0.030, 0.040, 0.050-in. deflection increments. TxDOT developed a computer program that captured the data. After each test, the dimensions of the specimen were measured and input into the program to get the final result. A typical concrete pavement mixture design for ARS specimens was used and is provided in Appendix B-2.

Calculation for ARS can be referred to ASTM C1399. The specification instructs to calculate the average residual strength (ARS) for each beam to the nearest 2 psi using the loads determined at reloading curve deflections of 0.020, 0.030, 0.040, and 0.050 in. as follows:

$$ARS = ((P_A + P_B + P_C + P_D) /4) * k \tag{4.1}$$

where:

$k = L/bd^2$, in^{-2}
ARS = average residual strength, psi,
$P_A+P_B+P_C+P_D$ = sum of recorded loads at specified deflections, lbf,
L = span length, in,
b = average width of beam, in., and
d = average depth of beam, in.

Finally, the mean ARS for each set of beams is calculated to the nearest 0.05 MPa [5 psi].

4.4.6 Bond Strength

Bond strength is one of the most important performance criteria for bonded overlays, such as BCOs and ultra-thin or thin whitetoppings. For unbonded type overlays, bond strength is not a performance criterion.

Seventeen 3-ft. by 3-ft. by 3-in. concrete overlays were placed in a period of three days. A series of days with similar high and low temperatures, wind speed, and humidity was selected in the effort to reduce any relative performance differences due to changes in the environmental conditions. Also, each day, slabs were placed within a period of one and one-half hours to avoid any large temperature shifts.

Bond strength of the 3-in. concrete overlays was measured in accordance with ASTM C1583, and 2-in. diameter cores were tested in tension at 3, 7, and 28 days. A Dyna pull-off tester from Humboldt was used to measure the bond strength. It was made certain that each core was drilled at least 3.5 in. deep, well past the 3-in. depth of the interface for these concrete overlays. After the tops of the cored surfaces were cleaned off and dried, a thin layer of 3000-psi epoxy was applied, and the aluminum caps were then glued to the tops of the cores. After sufficient time had passed for the epoxy to cure, the pull-off test was performed to measure bond strength.

Figure 4.9: Pull-off testing

4.4.7 Customized Restrained Shrinkage

Initial plan of the research was to perform restrained shrinkage tests according to ASTM C1581 – 09a "Standard Test Method for Determining Age at Cracking under Restrained Shrinkage" as a part of the laboratory testing phase. However, based on the past experiences with the unreliability of the test procedures, equipment, and the results at CMRG, a customized test to measure restrained shrinkage cracking was developed. To more closely simulate the dimensional constraints on typical roadways, long and thin bonded concrete overlay slabs (9-in. by 3-in. by 8-ft.) were placed on cured concrete substrates outside as seen in Figure 4.10. This geometry induces transverse cracks in the specimens by making the specimens long and thin and restrained to the existing slab.

Figure 4.10: Customized Restrained Shrinkage Slabs

Fifteen slabs were made using one mixture design with different types of reinforcement (the layout is provided in Appendix B-3). The mixture design purposely utilized a w/c of 0.45 in a 7-sack mix to promote crack development while staying within a reasonable range of typical concrete pavement mixture designs. When high water to cementitious material ratio is used, the water evaporation from concrete will increase and often causes cracking. The mixture design is provided in Appendix B-3.

4.4.8 Surface Preparation

Two different methods were tried to prepare the surface of the existing slab for the customized restrained shrinkage and bond tests: sandblasting (Figure 4.11) and shotblasting (Figure 4.12). Sandblasting proved to be a more difficult process for two reasons. First, it was very difficult to produce an even texture due to the fact that the discharge of sand particles was excessively strong through the small nozzle opening. If the discharge nozzle was too close to the surface, it left narrow and deep streaks. If the discharge nozzle was too far from the surface, it left wide and shallow streaks. Thus, overall texture was very uneven. Second, the sandblasting not only eroded the cement matrix but also the coarse aggregate due to its high power. Exposed coarse aggregates are a key to proper bond between the concrete overlays and the existing slabs.

Figure 4.11: Sandblasting in progress

Figure 4.12: Shotblasting in progress

Shotblasting was much easier to operate and provided an even wear on surface without eroding the coarse aggregates. The only problem was that in using the unit with so much confinement by the narrow slab, the steel shot kept spilling out from the recirculating stream, requiring constant refilling of the equipment. This also may have been due to the deterioration of the equipment seals caused by age.

After shotblasting was finished, a Circular Track Meter (CT Meter) was used to measure the depth and consistency of the surface texture profile as shown in Figure 4.13. This test procedure is presented in ASTM E2157 – 09 “Standard Test Method for Measuring Pavement Macrotexture Properties Using the Circular Track Meter.” The CT Meter uses a laser to measure the profile of a circle 11.2 in. in diameter or 35 in. in circumference. The profile is divided into eight segments of 4.4 in. The average mean profile depth (MPD) is determined for each segment of the circle. The reported MPD is the average of all eight segment depths. The test was run at different locations in order to find an average MPD of all the areas tested. The data collected was then put through computer software called “C.T.Meter” that produces a plot and MPD. MPD found using a CT Meter is highly correlated to mean texture depth (MTD), which is a measurement of the depth of the prepared surface texture. Figure 4.14 shows a plot of test results that were performed at five different locations (shown by five different colored graphs). The letters on the top represents the eight segments of the circle, and the numbers on the left represents the profile depth. As shown, the graphs are consistent to each other with an exception to a peak at Segment F. This means that the surface texture is mainly consistent except in a small area.

Figure 4.13: Circular Track Meter (CT Meter)

The average MPD of all five segments was 0.80. From MPD obtained from the test, MTD can be found using Equation 4.1:

$$MTD = 0.947MPD + 0.069 \qquad (4\text{-}2)$$

The resulting MTD is 0.83 mm. Converting the value to inches gives 0.033 in. Using the CT Meter proved to be a fast method to measure the prepared surface MTD. The collected data is provided in Appendix B-4.

After determining that the MTD was satisfactory, the surface was cleaned and dampened to reach SSD condition just before the placement of concrete as shown in Figure 4.15.

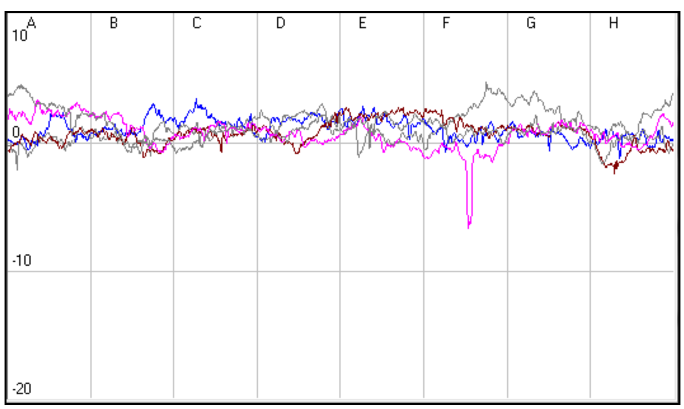

Figure 4.14: A plot of CT Meter results

Figure 4.15: Exposed CA shown in the prepared surface in SSD condition

4.5 Results

This section contains the results of the tests performed. Results for each test were analyzed and interpreted.

4.5.1 Analysis of Tests for Factorial Matrix

The results from tests in the factorial matrix were analyzed using a statistical software program distributed by Minitab Incorporated. By default, Minitab analyzes the designs using coded units, thus the regression equations will be shown using the coefficients for the coded units. The advantage of using coded units for analysis is that any erroneous statistical results due to different measurement scales for the factors (for example, inches vs. centimeters) is eliminated. Furthermore, using uncoded units tends to inflate the variability in the coefficient estimates and makes them difficult to interpret. Analyzing the design in the coded units does not change the variables in the model which are considered significant. However, it would influence the p-value for the constant term and the coefficients and the standard deviation of the resulting regression model.

All effects were assumed to be normally distributed with zero mean and the same variance. All results were first checked for normality and analyzed using the full quadratic model. Based on the results of the analysis, the most significant variables were determined, and then alternate models were examined until the best fit (as determined by p-values, R2 values, standard deviation, and lack of fit test) were selected. Unless otherwise stated, an alpha-value of 0.05 was used (95% confidence interval). Variables that have a p-value less than 0.05 were considered significant. However, for the lack of fit test, a p-value less than the alpha value of 0.05 indicated that the regression model did not adequately fit the data. Thus, a p-value > 0.05 indicated that the regression model selected was good.

4.5.2 Compressive Strength

The results for the compressive strength are shown in Table 4.5.

Table 4.5: Compressive Strength Results

	7 days (psi)	28 days (psi)
Mixture No.	**Average**	**Average**
1	5538	7590
2	3526	5798
3	5302	6238
4	3554	5721
5	6632	7909
6	3744	6128
7	6871	8017
8	3285	5837
9	5286	9149
10	5263	7196
11	4568	6294
12	5983	7501
13	3738	6058
14	5153	6929
15	5254	7359
16	5065	6960
17	6116	7888

Based on the statistical analysis, it was determined that the 7-day compressive strength was most affected by the following variables: SCM content, sacks of cement, SCM content*SCM content, and SCM content*sacks of cement. SCM content*SCM content and SCM content*sacks of cements are second order effects. If a second order effects is significant, then the regression model for the compressive strength will display curvature (as shown Figure 5.6). (Note, the second order effect of SCM content*SCM content can also be written as (SCM Content)^2. This yielded the following regression model (in coded units):

7-day compressive strength = 5243 -1248 SCM content + 366 sacks of cement – 426 (SCM content)^2 -339 SCM content*sacks of cement and R^2 = 95%. (4-3)

From the regression equation, it can be seen that 7-day compressive strength is most influenced by the SCM content. A unit change in the SCM content affected the 7-day compressive strength more than a unit change in the sacks of cement. Increasing the SCM content significantly reduced the compressive strength, whereas the compressive strength increased when the sacks of cement increased. In general, increasing the fine aggregate ratio decreased the compressive strength, but in the range used, the change in the compressive strength was not found to be significantly influenced.

With regards to the 28-day strength, the value for Mixture 9 appeared to be an outlier. Thus this value was ignored in the analysis. It was determined that the 28-day compressive strength was most affected by the following variables: SCM content and sacks of cement. This yielded the following regression model (in coded units):

28-day compressive strength = 6839 – 771 SCM content + 347 sacks of cement, and R^2 = 70%. (4-4)

A unit change in the SCM content had about twice the influence that a unit change in the sacks of cement had. Overall, the 28-day compressive strength was less affected by changes in the SCM content than the 7-day compressive strength.

With a constant mixture design, compressive strength change due to fiber addition was also explored. Fiber addition slightly improved 7-day (or early age) strength in some specimens. However, fiber addition slightly decreased 28-day strength in all specimens. The results are provided in Figure 4.16.

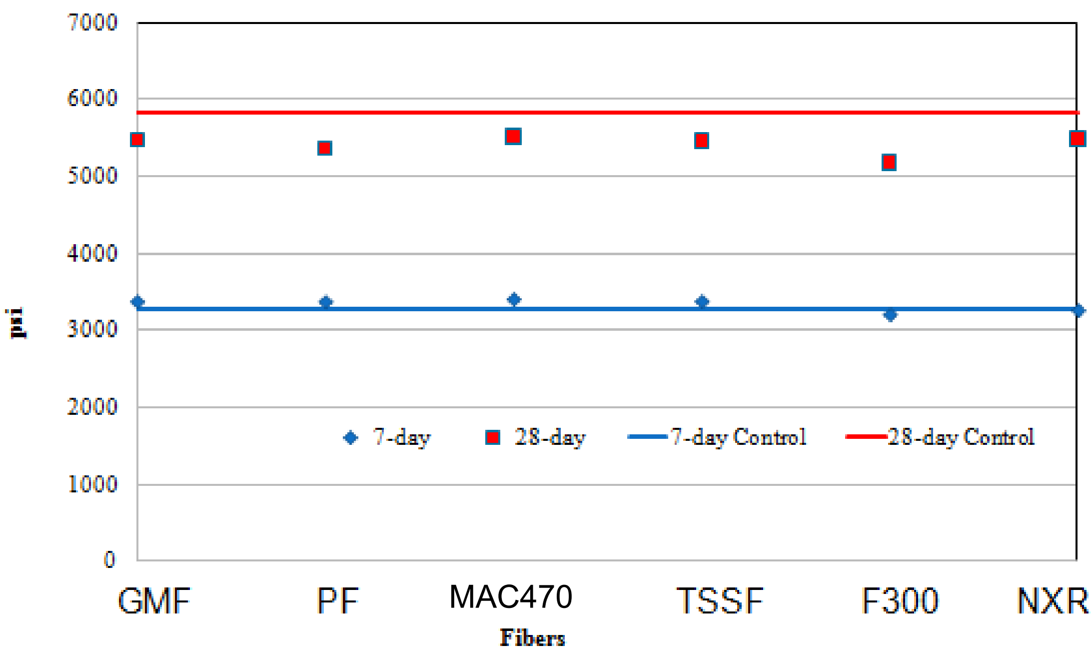

Figure 4.16: Compressive strength with fiber addition

4.5.3 Flexural Strength

The results for the flexural strength are shown in Table 4.6.

Table 4.6: Flexural strength results

	7 days (psi)	28 days (psi)
Mixture No.	**Average**	**Average**
1	786	799
2	525	693
3	753	777
4	555	692
5	816	884
6	640	830

	7 days (psi)	28 days (psi)
Mixture No.	**Average**	**Average**
7	909	867
8	511	776
9	691	938
10	687	845
11	675	820
12	805	836
13	542	801
14	692	809
15	736	850
16	715	817
17	740	871

Based on the statistical analysis, it was determined that the 7-day flexural strength was most affected by the following variables: SCM content and sacks of cement. The 7-day flexural strength could be accurately modeled by the following regression model:

7 day flexural strength = 692 – 130 SCM content + 28 sacks of cement, and R^2 = 89%. (4-5)

Thus, the 7-day flexural strength is mainly controlled by first-order effects and a unit change in the SCM content have about four times the influence than a unit change in the sacks of cement.

With regards to the 28-day flexural strength, the value for Mixture 9 appeared to be an outlier. Thus this value was ignored in the analysis. The following variables were found to be most significant: SCM content, sacks of cement, and SCM content*SCM content. This yielded the following regression model (in coded units):

28-day flexural strength = 835 – 37 SCM content + 45 sacks of cement – 40 $(\text{SCM content})^2$, and R^2 = 86%. (4-6)

Thus the 28-day flexural strength is more sensitive to changes in the SCM content than the 7-day flexural strength. The fine aggregate content was not found to be a significant variable (but in general increasing the fine aggregate ratio slightly decreased the flexural strength).

With a constant mixture design, flexural strength change due to fiber addition was also explored. Fiber addition slightly improved 7-day (or early age) strength in some specimens. However, fiber addition slightly decreased 28-day strength in all specimens. The results are provided in Figure 4.17.

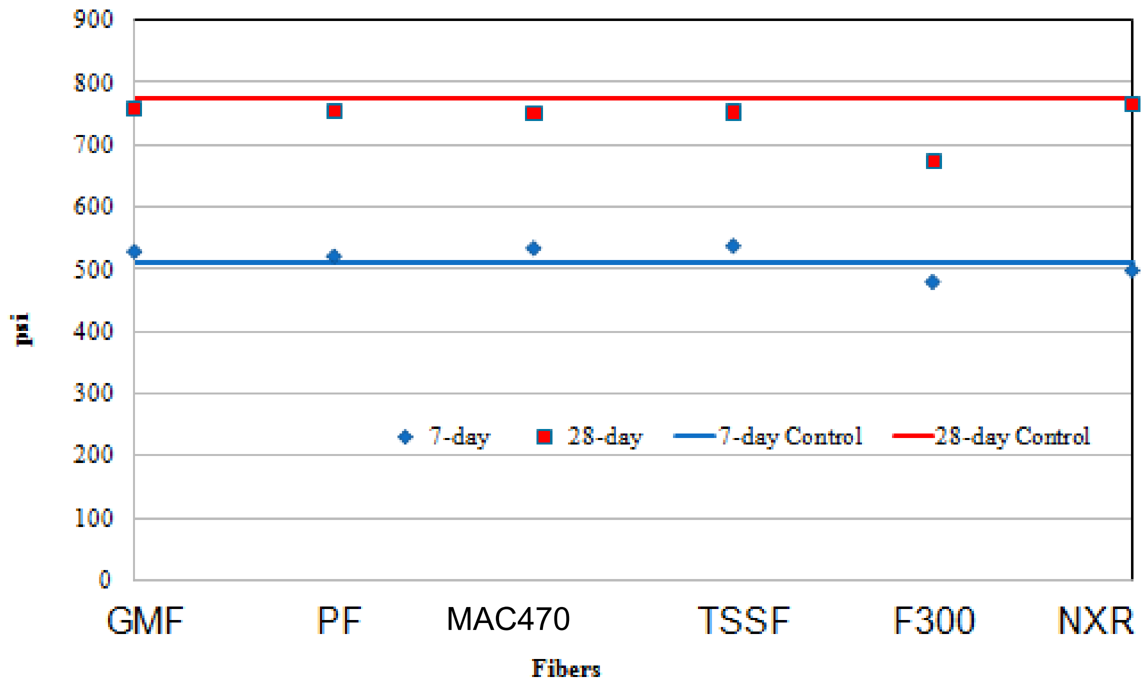

Figure 4.17: Flexural strength with fiber addition

4.5.4 Length Change

The results for the flexural strength are shown in Table 4.7. Typically, the range of long-term concrete shrinkage is 200 to 800 microstrains [60].

Table 4.7: Flexural strength results

Mixture No.	Microstrains
1	297
2	287
3	467
4	373
5	393
6	283
7	343
8	283
9	320
10	320
11	317

Mixture No.	Microstrains
12	407
13	290
14	347
15	310
16	363
17	323

The length change was found to be most significantly related to the SCM content (the fine aggregate ratio is the second most influential factor and its significance is marginal). The results were best modeled by a regression equation that incorporated all the variables:

Length Change = 0.0328– 0.0039 SCM Content + 0.0017 Fine Agg Ratio - 0.0016 Sack of Cement + 0.00131 (SCM Content)^2 - 0.0006 (Fine Agg Ratio)^2 + 0.0008 (Sack of Cement)^2 - 0.0004 (SCM Content *Fine Agg Ratio) – 0.0008(SCM Content* Sack of Cement) + 0.0038 (Fine Agg Ratio * Sack of Cement), and R^2 = 83.2%. (4-7)

With a constant mixture design, length change due to fiber addition was also explored. As Figure 4.18 shows, fiber addition did not have an effect on length change.

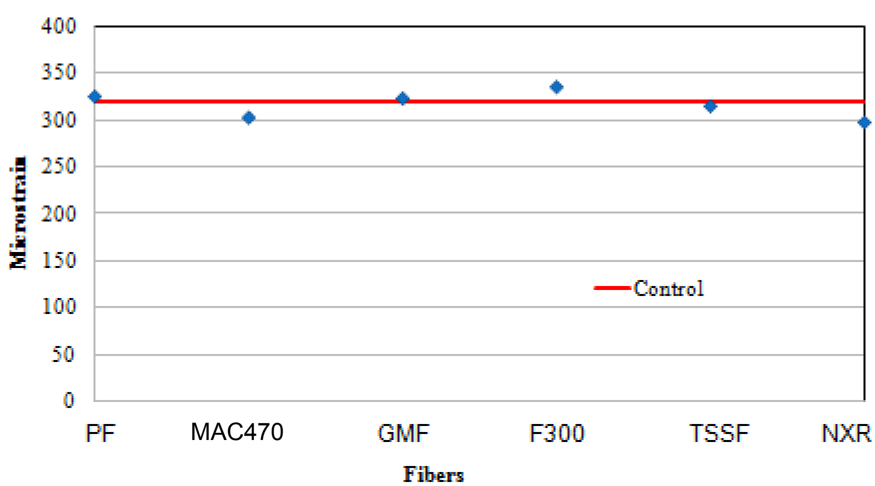

Figure 4.18: Length change results with fibers

4.5.5 Modulus of Elasticity

The results for the elastic modulus are shown in Table 4.8.

Table 4.8: Results of elastic modulus

Mixture No.	7 days (GPa)	28 days (GPa)
1	39	43
2	35	43
3	38	42
4	33	39
5	44	48
6	38	44
7	44	47
8	36	44
9	42	46
10	41	45
11	39	45
12	43	45
13	36	40
14	40	44
15	40	47
16	39	46
17	43	48

Based on the statistical analysis, it was determined that the 7-day elastic modulus was significantly influenced by the: SCM content, sacks of cement, and SCM content* SCM content. However, the regression model was found to be improved when the fine aggregate ratio was included in the model versus when it was ignored. Including the contribution from the fine aggregate ratio yielded the following regression model:

7-day elastic modulus = 41 – 3 SCM content – 0.6 Fine Aggregate Ratio + 2 sacks of cement - 2 (SCM content)^2, and R^2 = 91%. (4-8)

Thus it can be seen that the 7-day elastic modulus was most influenced by the SCM content. Increasing the SCM content resulted in a significant decrease in the 7-day elastic modulus.

With regards to the 28-day elastic modulus, the most significant Variables were the following: SCM content, sacks of cement, SCM content*SCM content. This yielded the following regression equation:

28-day elastic modulus = 45 – 1.5 SCM content + 1.8 sacks of cement - 2.5 (SCM content)^2, and R^2 = 75%. (4-9)

Similar to the 7-day elastic modulus, the 28-day elastic modulus was most influenced by the SCM content.

With a constant mixture design, MOE change due to fiber addition was also explored. Fiber addition slightly decreased both 7-day and 28-day MOE in all specimens. The results are provided in Figure 4.19.

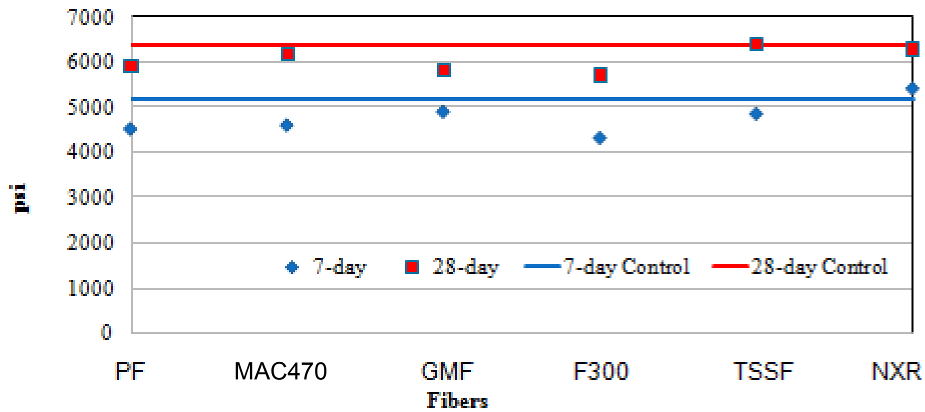

Figure 4.19: MOE with fiber addition

4.5.6 Average Residual Strength (ARS)

Two ways to quantify the effects caused by using fibers are ARS and workability. Increase in fiber dosage will increase ARS and decrease workability and vice versa. Workability also depends on concrete mixture design, and almost always there is a workability requirement. A relationship plot with ARS and workability with incremental fiber dosage should be established. This plot will provide the optimal fiber dosage range that permits maximizing ARS while meeting minimum workability requirement.

The following steps were followed for each fiber:

- Step 1: The slump requirement was determined based on TxDOT specification. Current minimums for paving concrete are 1.5 in. (slip-formed) or 4 in. (formed). Current maximums for paving concrete are 3 in. (slip-formed) or 6.5 in. (formed). For the research, it was assumed that the concrete was going be slip-formed.

- Step 2: A plot was developed with a set incremental fiber dosage in the X-axis and slump and ARS in the Y-axis. Two horizontal lines were drawn starting at the minimum and maximum required slump range and titled “slump requirement limit”.

- Step 3: A constant concrete mixture design was established. For the research, three incremental dosages were considered for each fiber: Low, medium, and high. Low and high dosages were determined by either manufactures’ or TxDOT’s recommended dosages. The medium dosage is at the middle of the two points. As dosage increased, slump decreased and eventually reached and fell below the minimum slump range. A best fit curve was used to connect the points, and the curve can be used to estimate approximate ARS for any given fiber dosage.

- Step 4: The fiber dosages between the two vertical lines represent the permissible range based slump only.

- Step 5: ARS tests were performed on the specimens made in Step 3. As dosage increased, ARS increased. Then, the values were superimposed on the plot, and a best fit curve was created.
- Step 6: Slump curve intersects the ARS curve represents the minimum dosage based on slump and ARS. The maximum dosage will always be based on minimum slump.

Using the above method, the optimal fiber dosage range for any concrete mixture design can be developed. Figure 4.20 shows an example plot with each step labeled. Figure 4.21 through 4.26 show the plots developed for each fiber. The results are shown in Table 4.8.

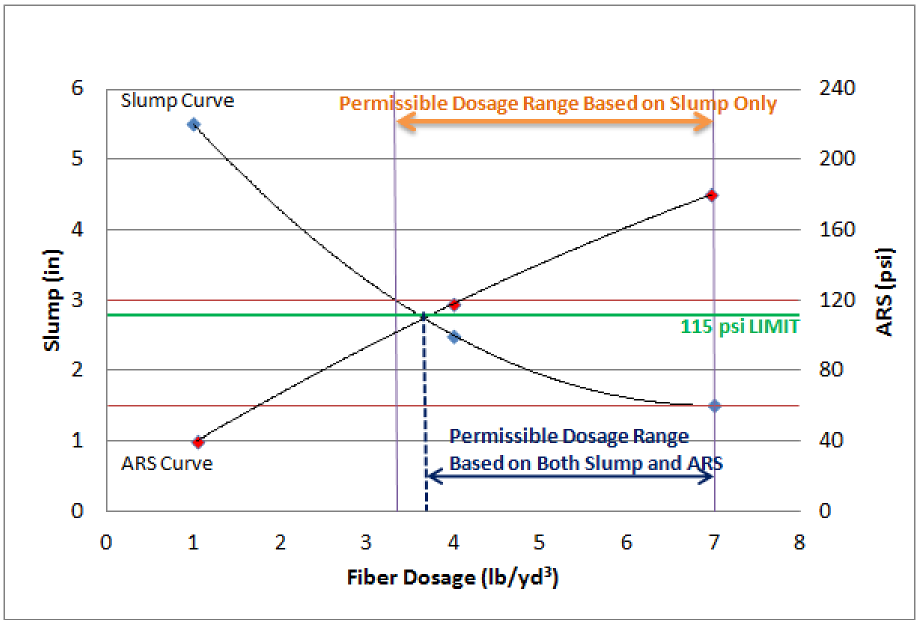

Figure 4.20: An example of ARS and workability relationship plot

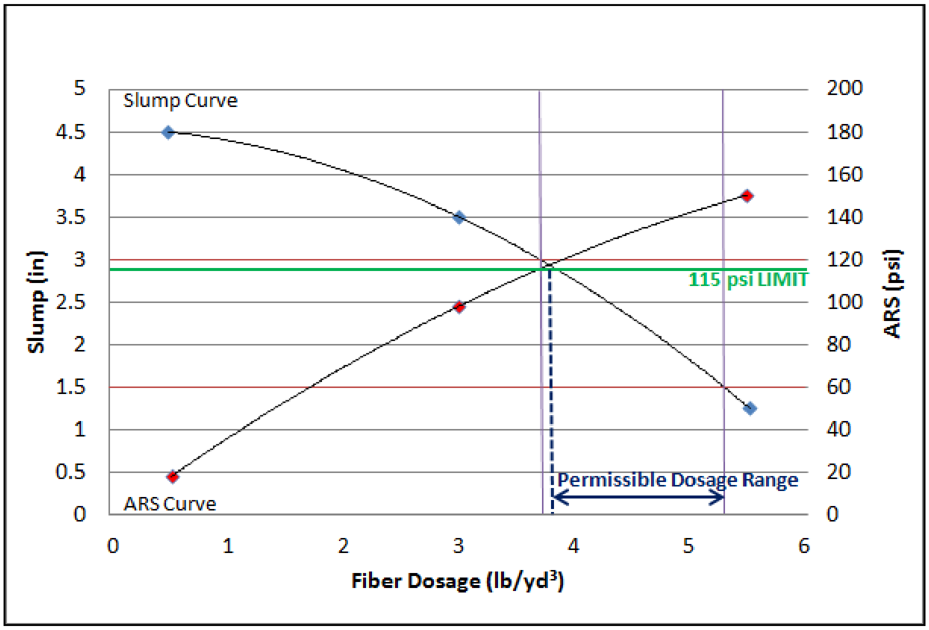

Figure 4.21: ARS and workability relationship plot for F300

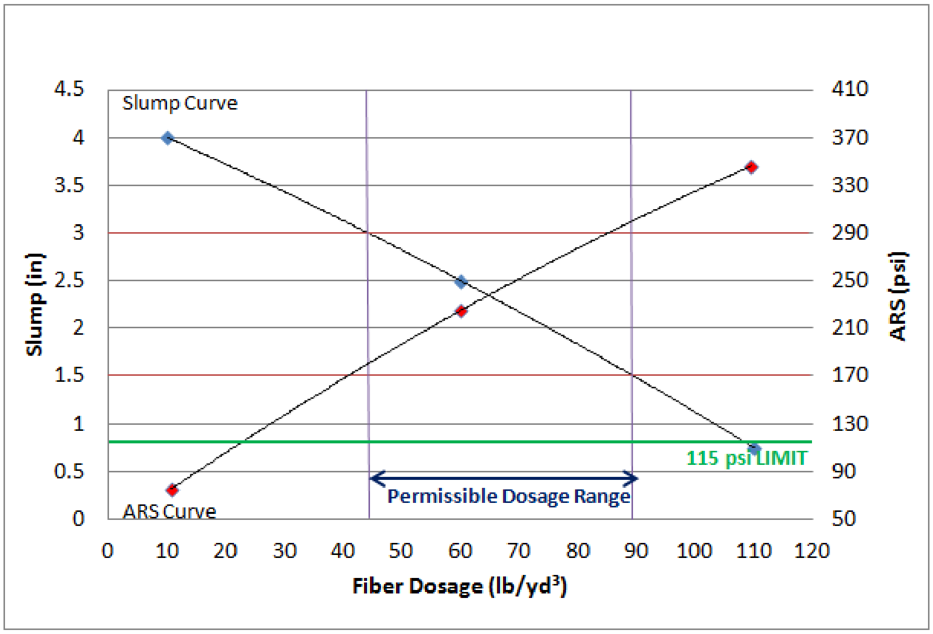

Figure 4.22: ARS and workability relationship plot for NXR

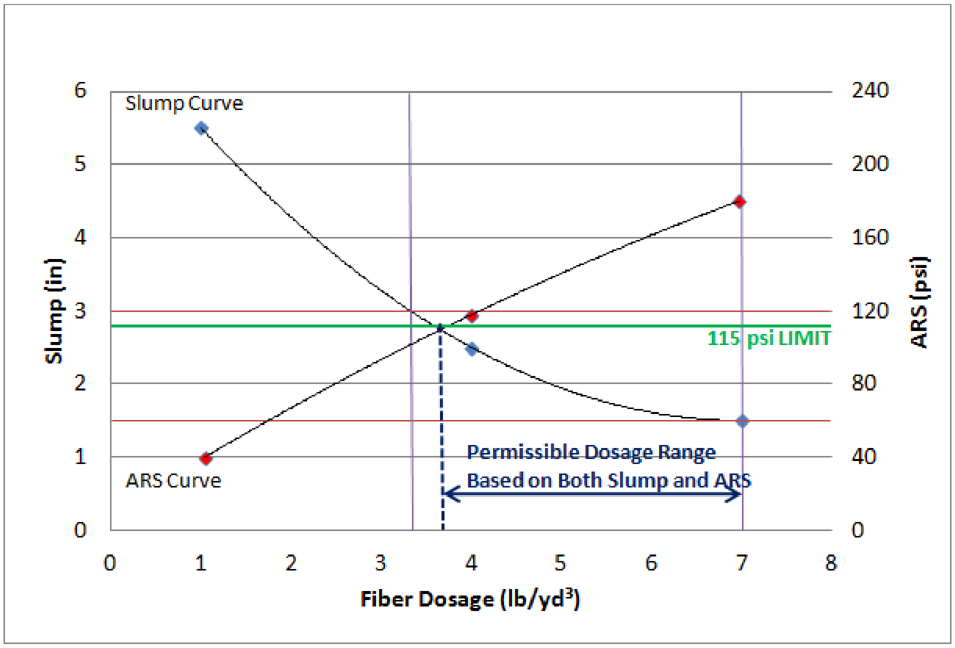

Figure 4.23: ARS and workability relationship plot for PF

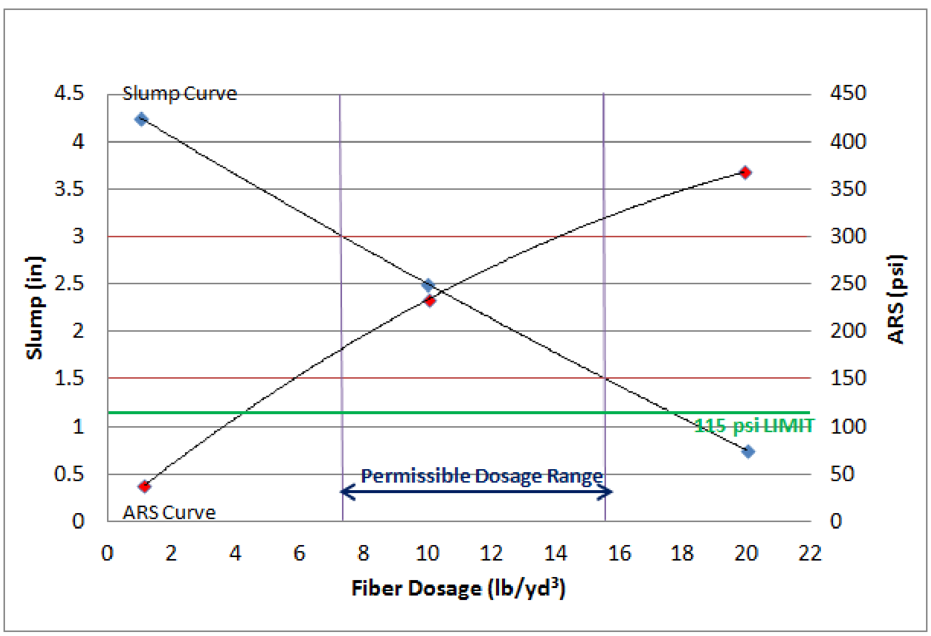

Figure 4.24: ARS and workability relationship plot for TSSF

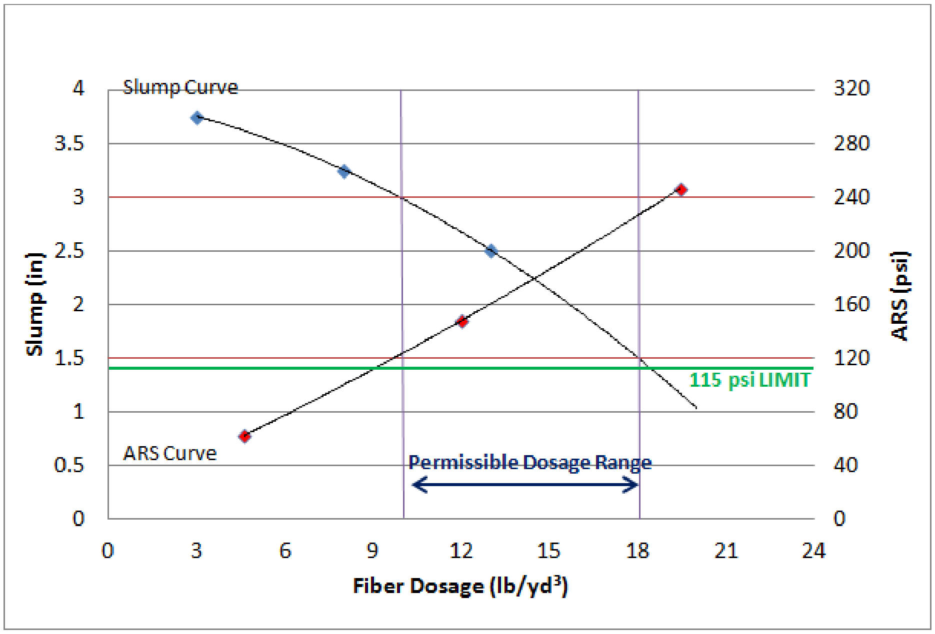

Figure 4.25: ARS and workability relationship plot for MAC470

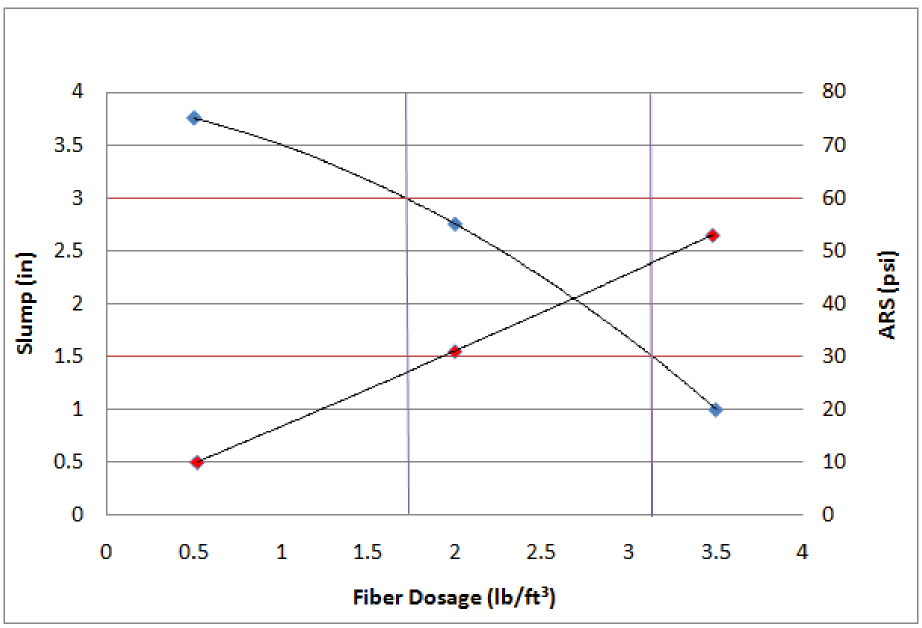

Figure 4.26: ARS and workability relationship plot for GMF (Note: Minimum ARS cannot be met)

From these ARS and workability relationship plots, one can estimate, for a given mixture design, the minimum and maximum fiber dosage within a required slump range and the corresponding ARS produced. For example, from Figure 4.21, 3.8 to 5.3 lb/yd³ of F300 dosage range is ideal for slip-form slump requirement and the approximated corresponding ARS range is 115 to 150 psi. ARS results are provided in Table 4.9.

Table 4.9: ARS Results

Fiber Dosage (lb/yd³)	Slump (in)	ARS (psi)	Fiber Dosage (lb/yd³)	Slump (in)	ARS (psi)
F300			**MAC470**		
0.5	4.5	18	3	3.75	62
3	3.5	98	8	3.25	148
5.5	1.25	150	13	2.5	246
TSSF			**GMF**		
1	4.25	38	0.5	3.75	10
10	2.5	234	2	2.75	31
20	0.75	368	3.5	1	53
NXR			**PF**		
10	4	75	1	5.5	40
60	2.5	225	4	2.5	118
110	0.75	346	7	1.5	180

Three main factors that determined the ARS of the fibers tested were fiber type, shape, and dosage/dispersion. Steel and rigid synthetic (sometimes referred to as "structural synthetic") fiber types (NXR, MAC470, and TSSF) gave significantly higher ARS when compared to other ordinary synthetic fibers (F300, PF, GMF). Unless high dosage is used, which is unlikely because it will significantly reduce workability, ordinary fibers do not produce comparable ARS. When ARS is one of the design criteria for concrete overlay construction, steel or rigid synthetic fibers are recommended.

Another fact to point out is that NXR, MAC470, and TSSF had deformed shapes as shown in Figure 3.2. When the load was applied to ARS beams, the failure happened either by fiber breakage or fiber pull-out. Fiber breakage is determined by the tensile strength and the pull-out is determined by shape. The deformed shape added extra ARS to the fibers. Once the concrete beam cracks, it relies solely on fibers at the crack interface to carry the load.

Specimens that had larger amounts of fibers present at the interface gave higher ARS compared to the specimens with lower amount of fibers at the interface. Figure 4.27 shows significantly more fibers present at the interface while Figure 4.28 shows significantly less. Consequently, the specimen in Figure 4.27 had the highest ARS, and the specimen in Figure 4.28 had the lowest ARS.

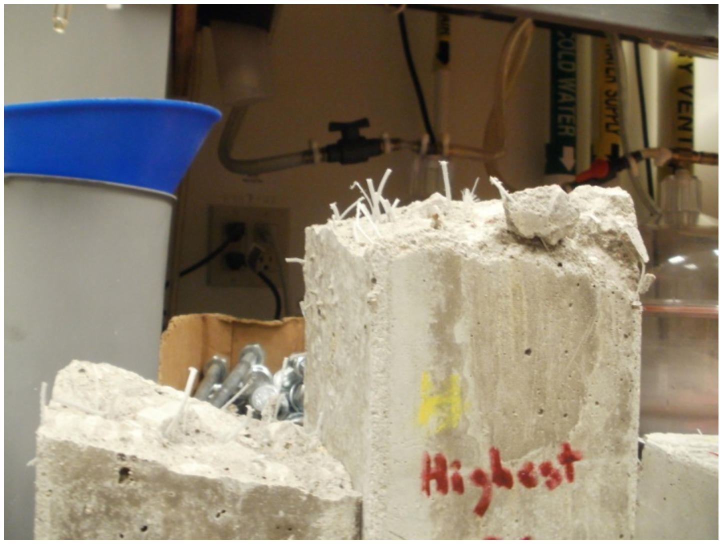

Figure 4.27: Failed interface of the specimen with highest ARS

Figure 4.28: Failed interface of the specimen with lowest ARS

The primary factor that determines the fiber presence at the interface is the dosage/dispersion. Within specimens containing a given fiber, variation in ARS was due to the amount of fiber present and the dispersion of the fiber. This is ultimately true in concrete overlay constructions where fiber balling is often found. Extra care is needed to promote fiber dispersion during fiber insertion during construction to maximize the benefits provided by fibers.

4.5.7 Bond Strength

Based on literature reviews, minimum bond strength of 200 psi based on a pull-off test similar to ASTM C1583 is the typical recommended value for concrete overlay construction. Table 4.10 shows the bond strengths achieved at 3, 7, and 28 days. The bond strength achieved at 3 days is typically adequate for construction loads to be allowed on the newly placed concrete overlay. By this time, 60 to 70% of the strength is achieved. Mixture 5, 7, and 17 gave bond strengths higher than 200 psi. All these mixtures had 7-sack cement content, and Mixture 17 gave a slightly lower value because it had 25% fly ash replacement.

The bond strength achieved at 7 days is typically adequate for public traffic loads and by then, 90 to 95% bond strength is achieved. Most of the mixtures were either above or at 200 psi bond strength with the exception of Mixture 2, 4, and 9. Based on the results, the main factor that determined the bond strength is the SCM content.

Table 4.10: Bond strength results at 3, 7, and 28 days

	Strength (psi)		
Mixture No.	**3 days**	**7 days**	**28 days**
1	152	239	254
2	87	174	181
3	145	239	261
4	80	152	189
5	239	348	355
6	116	196	196
7	232	334	348
8	123	196	210
9	123	174	196
10	126	181	190
11	125	184	194
12	181	297	297
13	102	175	194
14	131	196	210
15	131	181	210
16	116	203	218
17	203	297	312

Based on the statistical analysis, 3-day Bond Strength test results were most affected by the following variables: SCM Content and Sack of Cement. A slightly better model was achieved when the second order affects contributed from the cement content was included. This yields the following regression model (in coded units):

3-day Bond Strength = 131 – 44 SCM +33 Sack of Cement + 18.23 (Sack of Cement)^2 and R^2 = 76.13%. (4-10)

However, the regression model did not pass the "lack of fit test". Thus, this indicates that this model may not accurately fit the data. This may be due to several reasons, including but not limited to, an unmeasured variable causing some correlation in the measurements or pure error resulting from bias in the data collection procedures/results.

With regards to the 7-day bond strength, the data were best modeled by a regression equation that incorporated all the variables:

7-day bond strength = 197 – 56 SCM Content – 5 Fine Agg Ratio + 36 Sack of Cement + 25 (SCM Content)^2 - 23 (Fine Agg Ratio)^2 + 39 (Sack of Cement)^2 - 0.906 (SCM Content *Fine Agg Ratio) – 17 (SCM Content* Sack of Cement) + 0.906 (Fine Agg Ratio * Sack of Cement) and R^2 = 95.3%. (4-11)

Even though all the variables are used in the model, the variables which were most significant to the 7-day were the SCM Content, Sack of Cement, (Sack of Cement)^2, and (SCM Content* Sack of Cement. From the regression model, it can be seen that the SCM Content played a larger role (more than twice the influence) on the 7-day bond strength than the Sack of Cement. This model is borderline for the lack of fit test (p-value of the regression model is 0.49 and alpha value is 0.5).

The analysis for the 28-day bond strength also showed that the most significant variables were SCM Content, Sack of Cement, and the second order effects corresponding to (Sack of Cement)^2 and (SCM Content * Sack of Cement). However, similar for the results of the 7-day bond strength, the data for the 28 day bond strength were best modeled by a regression equation that incorporated all the variables:

28-day bond strength = 212 – 55 SCM Content +2 Fine Agg Ratio + 32 Sack of Cement + 19 (SCM Content)^2 - 17 (Fine Agg Ratio)^2 + 38(Sack of Cement)^2 +3 (SCM Content *Fine Agg Ratio) – 19 (SCM Content* Sack of Cement) -1 (Fine Agg Ratio * Sack of Cement). (4-12)

While this was the best model that was obtained, the model a poor lack of fit and thus would likely do a poor job predicting any future trends.

There were few specimens that failed at the interface even before any significant pull-off load was applied. Upon inspection, it was discovered that the interfaces had smoother surfaces than the other specimens that developed full pull-off strength. Lack of quality control led to inadequate surface preparation, which caused pull-off specimens to fail prematurely.

4.5.8 Customized Restrained Shrinkage

The degree of benefit gained by using different reinforcements was measured by performing pull-off tests. The reinforcements reduce stress at the interface when concrete shrinks and expands and, therefore, higher the bond strength equates to better the reinforcement at reducing stress at the interface. This leads to less cracks forming.

The results are shown in Figure 4.29. Fibers increased bond strength, and blending microfibers with either macro or steel fibers increased bonded strength even more. The specimens with river gravel as CA produced very poor bond strength due to its high CTE and induced high stress at the interface. No. 4 reinforcement bars produced very high bond strength because they effectively controlled the strain at the interface and then reduced in stress. Using reinforcement bars are still less expensive than using fibers.

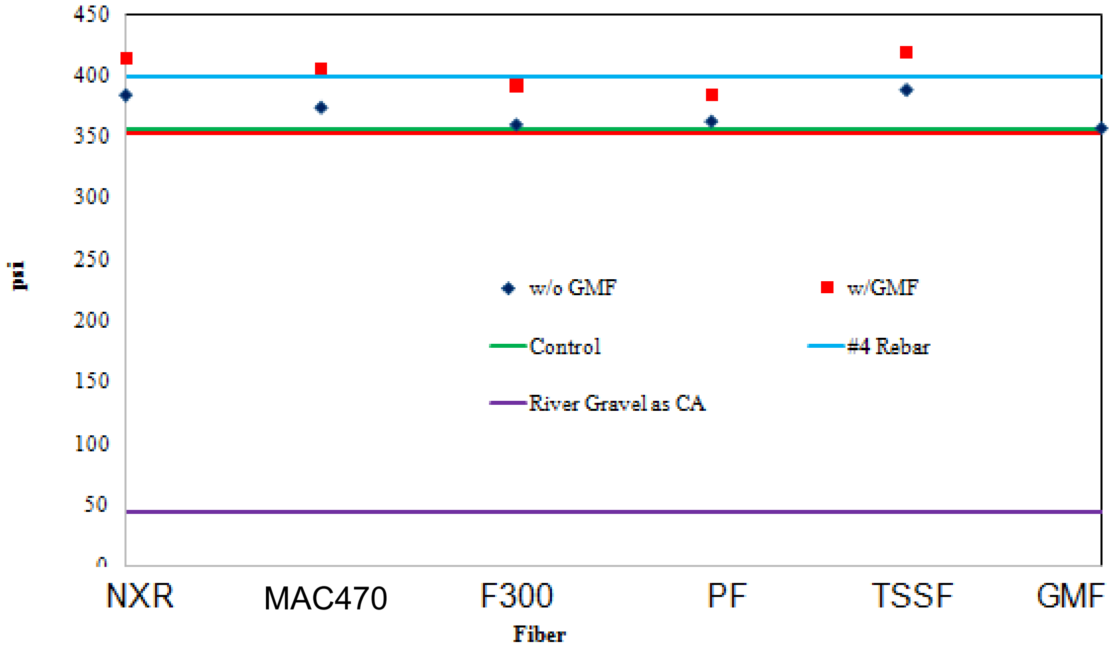

Figure 4.29: Bond strength with fibers

Chapter 5. Performance Prediction Modeling

HIPERBOND stands for "High Performance Bonded Concrete Overlays" performance prediction software. It is recommended for predicting bond strength and stress for design concrete mixtures for bonded concrete overlays (BCOs). The finite element modeling software was developed under a Federal Highway Administration contract [61] and has been used to evaluate early age bond performance in BCOs.

5.1 Review of HIPERBOND

HIPERBOND requires inputs for BCO design, materials and mixture design, construction procedure, and environmental conditions. Figure 5.1 shows an example of the software interface.

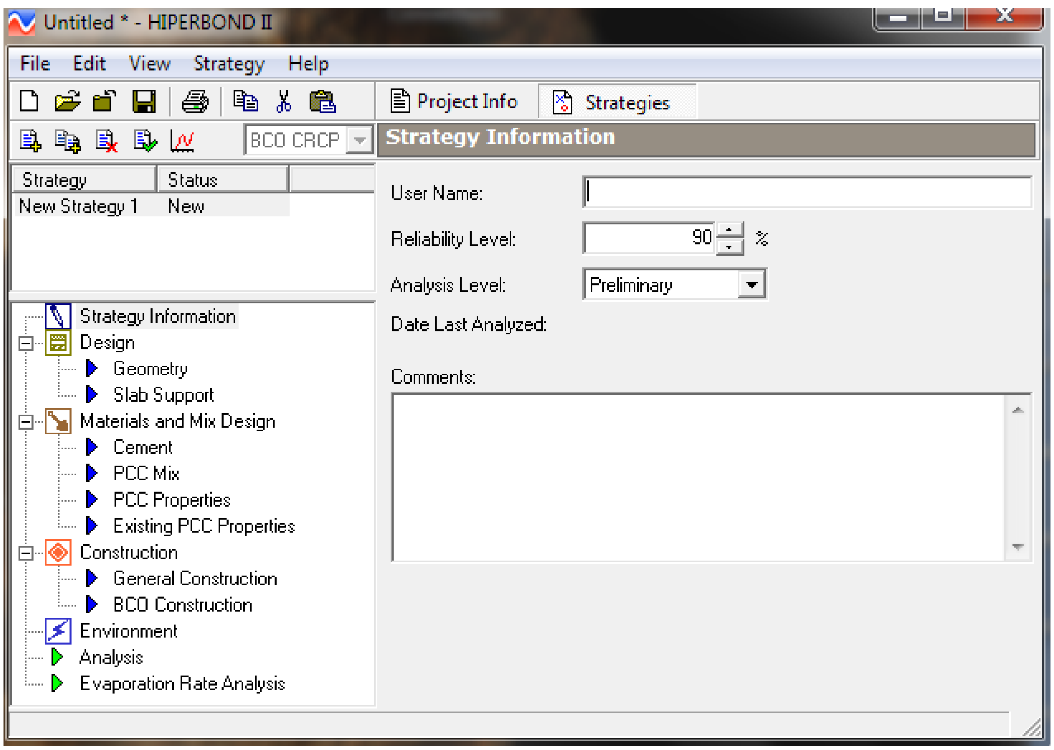

Figure 5.1: HIPERBOND interface

After all the required information is inputted, the analysis tab is selected. Once the analysis starts, tensile and shear stresses are calculated and compared to strength as a function of time since construction. If at any time either stress exceeds the available strength, the overlay is predicted to fail. Figure 5.2 shows an example analysis where tensile stress exceeded tensile stress, which predicts a failure. Figure 5.3 shows an example analysis where both tensile and shear stress fell below the strength, which predicts a satisfactory BCO design.

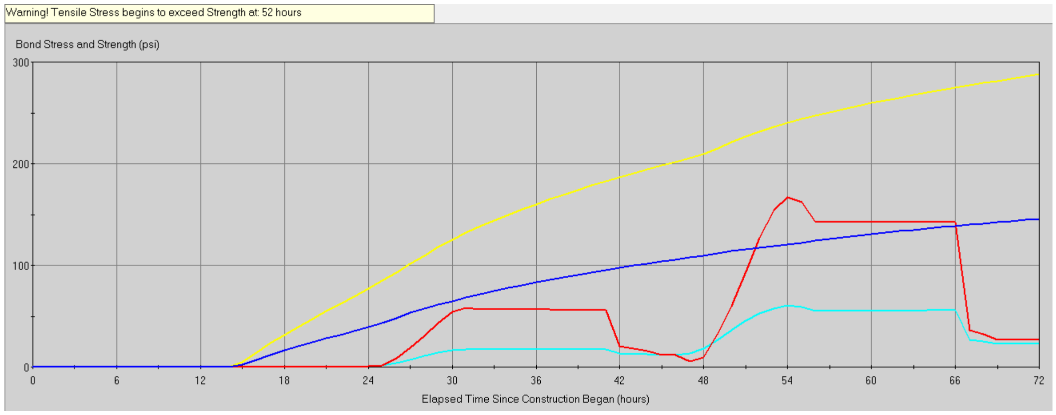

Figure 5.2: An example analysis predicting failure

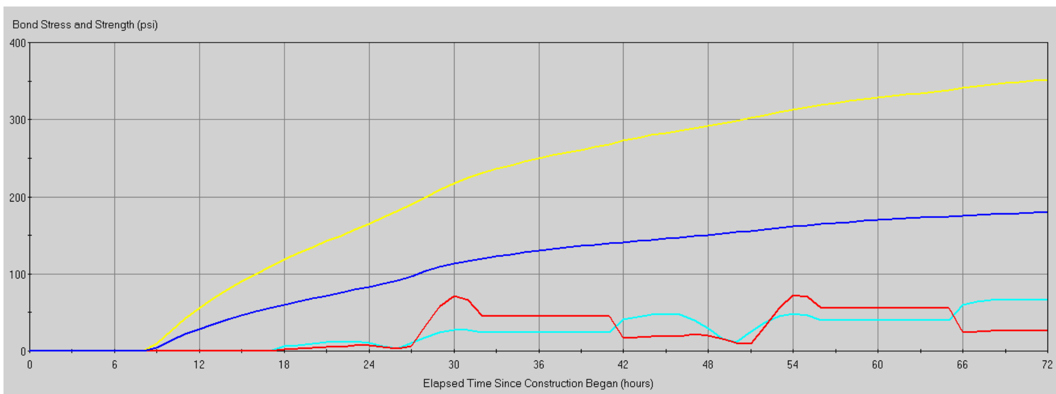

Figure 5.3: An example analysis predicting a satisfactory BCO design

5.2 Analysis of Factorial Matrix Designs

All 17 factorial matrix mixture designs, Table 4.2, were used for bond strength tests in section 4.5.7. The 17 designs were evaluated using HIPERBOND and the surface preparation and environmental conditions were inputted as they were when the bond strength specimens were placed. The variables used in HIPERBOND are provided in Table 5.1. There was not enough information for some of the inputs, and for these, instead, typical values were used that were predetermined by HIPERBOND.

Table 5.1: Analysis Variables and Inputs

Variables	**Input**
Design	PCC Overlay Thickness = 3 in. Existing slab thickness = 8 in.
Materials and Mixture Design	Cement = Type I Aggregate Type = Limestone Fly Ash Class = Class F (CaO >7%)

Variables	Input
	Batch Proportions and properties = 17 factorial matrix mixture and properties (section) PCC Properties = Average compressive strength for each factorial matrix Existing PCC Properties = Siliceous river gravel
Construction	Curing Method = Double Coat Liquid Curing Compound Surface Preparation Method = Light shotblasting Bonding Agent = None
Environment	Temperature of Mid-November 2010 High 78 and low 58 Construction time = 1 pm

5.3 Analysis Results

Results for the 17 analysis are shown in Table 5.2. As shown, Mixture 2, 4, 6, 8, 13 were unsatisfactory because the tensile stress exceeded allowable tensile strength after 28 to 30 hours since the placement. The rest of mixtures were satisfactory because tensile and shear stress fell below the strength capacity. The unsatisfactory mixtures achieved bond strength between 88 to 96 psi. The predicted low bond strengths corresponded to the low bond strength achieved in the results discussed in section 4.5.7.

The five unsatisfactory mixtures were reanalyzed assuming that they were placed in the morning at 8 am. As Table 5.2 shows, they were all satisfactory because both tensile stress and shear stress fell below the corresponding limits. This shows how much the time of day of the placement is also very important.

Table 5.2: Analysis results

Mixture	Under Original Condition	8 am placement
1	OK	OK
2	Not OK	OK
3	OK	OK
4	Not OK	OK
5	OK	OK
6	Not OK	OK
7	OK	OK
8	Not OK	OK
9	OK	OK

Mixture	Under Original Condition	8 am placement
10	OK	OK
11	OK	OK
12	OK	OK
13	Not OK	OK
14	OK	OK
15	OK	OK
16	OK	OK
17	OK	OK

Keeping all variables constant and only changing surface preparation method showed that slightly (yet significant) lower bond strength was achieved going from light shotblasting method to heavy shotblasting. When the milling method was used, the strength was even lower. Depending on the method of obtaining surface texture, the amount of cement content must be adjusted accordingly.

Moreover, keeping all variables constant and only adding bonding agents provided notable results. PCC grout actually increased tensile stress but the strength stayed the same. Epoxy did not alter the stress or the strength. Therefore, this model supports the conclusion that the use of bonding agents is not recommended. All of the analysis performed has been provided in Appendix C.

Chapter 6. Guidelines for Materials Selection

The following set of guidelines was developed to assist readers through the recommended process for selecting proper materials for proportioning durable overlays.

6.1 Introduction

It is important to keep in mind that materials selected should satisfy both the performance-based and prescriptive-based acceptance criteria. A set of flowcharts for materials selection has been developed to help guide the reader through the selection process.

6.2 Performance Based Acceptance Criteria

Candidate materials selected for concrete overlays must meet certain recommended performance limits. Criteria requirements for each constituent material are derived from both the recommendations from literature review (Chapters 2 and 3) and the results from the research (Chapter 4).

6.2.1 Compressive Strength (ASTM C39) and Modulus of Elasticity (ASTM C469)

Overlay concrete mixtures must meet minimum average compressive strengths of 3500 psi at 7 days or 4400 psi at 28 days. These limits are found in ITEM 360 in TxDOT Standard Specifications 2004.

For BCOs only, the maximum average must be controlled, so that the overlay's resulting modulus of elasticity (MOE), which is directly proportional to its compressive strength, is less than the MOE of the existing pavement. The MOE in the concrete overlay should always be lower than the MOE in the existing pavement, because differences in moduli between layers have significant influence on the both the traffic induced and the thermally induced stress at the interface. And the highest thermal stresses and strains will be introduced at the top one or two inches of depth from surface of the overlay. This is so, because only the top surface is directly exposed to the elements and concrete cannot efficiently conduct heat through the rest of the slab. When the overlay has a lower modulus than the substrate, it can change stress/strain responses easier than the substrate, minimizing any debonding shear forces at the interface.

To increase compressive strength in the overlay, the following actions can be taken:

- Decrease the w/c of the concrete mixture.
- Decrease fly ash replacement.
- Use coarse aggregate with higher strength.

The following may lower the compressive strength:

- Raising the w/c of the concrete mixture.
- Increased fly ash replacement.
- Fiber addition (only minimal decrease in the strength).

6.2.2 Flexural Strength (ASTM C78)

A minimum average flexural strength of 570 psi at 7 days or 680 psi at 28 days is recommended. These limits are found in ITEM 360 in TxDOT Standard Specifications 2004.

The flexural strength for a given mix design using the same aggregates is somewhat proportional to the compressive strength, so the same factors that raise or lower the compressive strength (Section 6.2.1 above) simultaneously raise or lower the flexural strength, too, but not in the same rate.

6.2.3 Coefficient of Thermal Expansion and Modulus of Elasticity

For BCOs only, designers must consider the coefficient of thermal expansion (CTE), ASTM E228, of both the overlay concrete and the substrate concrete. Changes in the COTE – 11 "Standard Test Method for Linear Thermal Expansion of Solid Materials with a Push-Rod Dilatometer," of concrete is attributed mostly to the coarse aggregate (CA). Candidate CAs must make concrete having a CTE that is equal to or lower than the CTE of the existing pavement. For example, it is advisable to utilize a limestone aggregate for the BCO concrete if the existing concrete has siliceous river gravel as CA, because of the lower CTE in limestone, but the opposite arrangement will make up for an overlay prone to delamination.

Also, remember that the modulus of elasticity (MOE), ASTM C469, of the concrete overlay must be equal to or lower than the MOE of the existing pavement. The basic premise for achieving equal or lower CTE and MOE is to lower stresses at the interface, because the increased stress at the interface will increase the possibility of debonding.

In order to lower MOE, the following actions can be taken:

- Increase fly ash replacement.
- Increase the w/c (lowers strength and MOE, too)
- Decrease portland cement content.
- Add fibers.

The following may heighten MOE:

- Decreased fly ash replacement.
- Increased portland cement content, lower w/c.

6.2.4 Tensile Bond Strength

The minimum tensile bond strength (ASTM C1583) should be greater than 200 psi. This limit comes from AASHTO that, based on a numerous past studies, bond strength greater than 200 psi resulted in satisfactory concrete overlay performance. Maximizing bond strength will ensure longevity of the new concrete overlay.

In order to raise bond strength, the following practice is recommended:

- Prepare the surface properly (See section 2.5.2 and 2.5.3).
- Use of CA with low CTE and MOE.
- Increase cement content, decrease w/c
- Decrease fly ash replacement.

- Add fibers – blending different types of fibers may be helpful.

The following may lower bond strength:

- Lack of proper surface preparation.
- Use of CA with high CTE and MOE.
- Decrease cement content, increase w/c.
- Increase fly ash replacement.

After the concrete overlay placement, the following practice will ensure that the minimum bond strength is achieved:

- Until minimum specified strength is achieved, vehicles (including construction vehicles) should not be allowed on the concrete overlay.
- Longer curing time is required when fly ash is used, because it slows the early strength gain.
- Pull-off tests (ASTM C1583) must be performed in order to make sure that the minimum strength is achieved.

6.2.5 Average Residual Strength (ASTM C1399)

The average residual strength (ARS) test should be used to evaluate any change in toughness due to fiber incorporations. Fibers act to bridge the cracks providing toughening mechanism to concrete.

To accurately and precisely perform ARS tests, fibers should be well dispersed to increase consistency and the average ARS maximum. Better fiber dispersion can be achieved by introducing the fibers gradually and early in the mixer.

In order to raise ARS, the following actions can be taken:

- Use steel or structural synthetic fibers.
- Increase fiber dosage (until at least the minimum workability is achieved).
- Increase fiber dispersion.
- Use fibers with deformed shape.

The following may lower ARS:

- Using normal synthetic fibers.
- Minimal fiber dosage.
- Lack of proper fiber dispersion.

6.2.6 (ASTM C157)

The goal in placing and curing concrete is to always minimize shrinkage as much as possible, because it reduces crack formation. Typically, the range of long-term concrete shrinkage is 200 to 800 microstrains. If new design mixtures are being tried, it is recommended to make specimens from control mixtures, too, to compare the shrinkage potentials. Since the primary reason for developing new design mixtures using new materials or different dosages is to increase performance of the concrete, the performance of the new mixture should typically be equal or better than the control mixture.

In order to lower shrinkage, the following practice is recommended:

- Increase fly ash replacement.
- Decrease cement content.
- Decrease w/c ratio.

The following may heighten shrinkage:

- Increased w/c ratio.
- Increased cement content.
- Decreased fly ash replacement.

6.2.7 Workability (ASTM C143)

Criteria for workability as measured by slump, in concrete overlays depend on the project and whether the "slip-form" or "formed" method is used for the construction. Currently, TxDOT specifies the following:

- Minimum: 1.5 in. (slip-formed) or 4 in. (formed).
- Maximum: 3 in. (slip-formed) or 6.5 in. (formed).

Trial batches should be made to achieve project specific workability limits. Workability can be reduced by following materials or adjustments: Addition of fibers, high fine aggregate content, using angular aggregates, decreasing paste content and/or decreasing water content (i.e. lowering water to cementitious ratio). Workability can be increased by following materials or adjustments: Addition of fly ash, low fine aggregate content, using water reducing agents, increasing paste content and/or increasing water content.

6.3 Prescriptive Based Acceptance Criteria

Candidate materials selected for concrete overlays must meet certain recommended design guidelines, which were derived from both the recommendations from literature review (Chapters 2 and 3) and from the research results (Chapter 4).

6.3.1 Cement Type

Type I/II is adequate for normal concrete overlays and has produced satisfactory results in the research. Literature suggests that Type I can be considered for normal concrete overlays. For expedited concrete overlays, Type III or more finely ground Type I is recommended. They are ideal for expedited construction because higher early strength can be achieved in less time. However, heat of hydration is increased and can cause thermal cracking. For possible sulfate contamination, Type II or V is recommended because they are chemically resistant.

6.3.2 Cement Content

A range of 6 to 7 sacks of cement was tested and produced acceptable results for the research. Cement content for bonded overlays should be increased when heavy shotblasting or milling is used for surface preparation. As the depth of texture in the substrate surface increases, the surface area at the interface increases. The increased surface area requires more cement paste available to adequately coat the irregular surface and achieve satisfactory bond at the interface. Also, if fiber is added to the mixture, increasing the cement content is recommended to coat the fibers and improve workability.

From the literature, up to 7.5 sacks of cement have been recommended for BCOs. Although this much is not normally recommended, increasing typical paving mixtures' cement contents will help ensure that the available paste is sufficient in quality and quantity to achieve adequate bond at the interface. A little extra paste also eliminates the need for a bonding agent, adequately coats all aggregates, and increases workability. However, too much cement content should be avoided to reduce shrinkage and potential for alkali-silica reaction (ASR) if reactive aggregates are used. Goals of reducing paste demand may be achieved by using well-graded aggregates.

Increasing the cement content alone decreases the w/c, which can lead to the following:

- Higher compressive strength at early age.
- Higher flexural strength at early age.
- Higher bond strength.

Decreasing the cement content alone increases he w/c, which can lead to the following:

- Lower drying shrinkage.
- Lower MOE.

6.3.3 Fly Ash Replacement

Fly ash can be used in a concrete mixture to improve workability, finishing, and durability. Fly ash also reduces amount of water required and the heat of hydration, which means less shrinkage and cracking. Moreover, using fly ash lowers the cost of concrete.

However, the main drawback is that as larger amounts of cement are replaced by fly ash, initial strength gain is significantly retarded. This means that the heavily substituted concrete overlay pavements need a longer time to cure until traffic loads can be allowed, and in colder weather the delay may not be practical. So, use of fly ash could potentially delay the construction and the opening to traffic.

Fly ash should replace portland cement in proportions that are calculated to ensure that enough is used to maximize its benefits, while minimizing additional time required to gain adequate strength. To accomplish this, environmental conditions must be taken into account. When the temperature is relatively low, fly ash will be slower to react and, therefore, slow down the initial strength gain. For this reason it is recommended that the amount of fly ash replacement should be adjusted to accommodate the seasons of the year and the time of the day (morning is cooler than afternoon).

Based on the research, around 25% fly ash replacement provided adequate strength (compressive, flexural, and bond strength). As the replacement amount got closer to 50%, the strengths were much lower. However, if a longer curing period is allowed, higher replacement rates can be utilized. Currently, Item 421 in TxDOT Standard Specifications 2004 allows 20% - 35% replacement. Within the range, depending on the environmental conditions, lower or higher replacement rates can be used.

Increasing the fly ash replacement can lead to the following:

- Higher workability.
- Lower MOE.
- Lower drying shrinkage.

Decreasing the fly ash replacement can lead to the following:

- Higher compressive strength.
- Higher flexural strength.
- Higher bond strength.

6.3.4 Water to Cementitious Ratio

For the research, a water-to-cementitious ratio (w/c) of 0.40 was used assuming normal placement conditions. This ratio provided enough workability without sacrificing other performance limits. Literature recommends w/c of 0.40 to 0.45 for normal placement conditions, and maximum of 0.35 for expedited placement conditions.

The idea of a minimum limit is that lowering w/c can lead to forming a less than ideal amount of paste that can hinder coating of the aggregates and of the interface to develop adequate bond strength, as well as reducing workability. Also, low w/c is generally associated with higher modulus of elasticity. Modulus of elasticity of the new concrete overlay should be either equal or lower than the modulus of the existing pavement. The maximum limit is set, because too much water can increase the chance of shrinkage due to evaporation rate, and it can reduce the strength of the overlay matrix and its bond to the substrate.

6.3.5 Aggregates

Aggregates that conform to Item 421 in the TxDOT Standard Specifications should be used. Aggregates that conform to Item 421 of TxDOT Standard Specifications Should be used, but extensive laboratory testing on trial mixtures or demonstrated field performance is required to ensure selection of suitable aggregates.

Coarse Aggregate

Since the CTE of concrete depends mostly upon the CTE of the CA, the candidate CA in the overlay should have a CTE that is equal or smaller than the CTE of the CA in the existing pavement but need not be less than 5.5 x 10^-6. Also, the MOE of the concrete overlay should be equal or lower than the MOE of the existing pavement. The basic premise for achieving equal or lower CTE and MOE is to lower stresses at the interface because the increased stress at the interface will increase the possibility of debonding.

Example: It is advisable to utilize a limestone CA for the BCO concrete, if the existing concrete has siliceous river gravel as its CA, because of the limestone lower thermal coefficient, but the opposite arrangement would make for an overlay prone to delamination. If the existing pavement used limestone aggregate as CA, only limestone aggregate with equal or lower thermal coefficient should be used for new BCOs.

Moreover, the maximum nominal size of CA should be no more than one-third the thickness of the concrete overlay. The minimum allowable maximum nominal size should be 0.5 in. Finally, a candidate CA must meet Item 421.1 in the TxDOT Standard Specifications.

Fine Aggregate

A candidate fine aggregate (FA) must meet Item 421.2 in the TxDOT Standard Specifications. Literature recommends selecting FA that is from natural siliceous deposits, is non-reactive, has low absorption and an acid soluble index (AI) (ASTM C1152-97 "Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete") greater than 25 . Some blended sands may be suitable, and TxDOT currently allows for any sands to be used for concrete pavements to have an AI from their test method of 60. But the use of blended sands was not addressed in this project. TxDOT Project 0-6255 "Manufactured Sand in Pavements" is investigating this issue. Calcium carbonate fines are known to polish excessively. TxDOT recommends a minimum acid insoluble residue of 60 % [63].

6.3.6 Aggregate Ratios

Research results from this project investigated showed inconclusive performance for varying the weight ratio of fine aggregate to overall aggregate (FA/(FA+CA)) between 0.35 and 0.45. Similarly, inconclusive results were found for performance trends when varying the weight ratio of aggregate to cement (a/c) in typical usage ranges from 4.7 to 6.0. Generally, increasing the a/c ratio will effectively reduce the paste content and lower workability, while decreasing the ratio will increase the paste content, thereby improving the workability. Also, maximizing a/c can help to reduce shrinkage. Within the explored ratio ranges, all of the concrete mixtures were satisfactory for use as overlays, so this aggregates ratio study never really approached any limit of acceptability for performance. Based on literature, it is preferred to maximize coarse aggregate and minimize fine aggregate to reduce shrinkage, increase workability, and reduce amount of cement paste required.

6.3.7 Admixture Selection and Dosage

Admixture dosage may need to be different for each batch. The dosage for HRWR and MRWR (mid-range water reducer) varies with the amount of cement and the type of aggregate. (This is especially true for fine aggregate, if manufactured sand blending is required. Varying sand types, however, was not investigated in this research project.) The dosage should be adjusted through trial batches, and the interaction between admixtures should be considered. For example, too much HRWR can cause the mixture to get sticky and will make finishing more difficult. A MRWR might be a better choice for this case, since it requires more MRWR to accomplish the same results, but this increases the paste volume in the stiffer paving mixture, which may improve the workability with less stickiness.

6.3.8 Fibers

Two ways to quantify the effects caused by using fibers are ARS and workability. An increase in fiber dosage will increase ARS and decrease workability and vice versa. Workability also depends on concrete mixture design, and almost always there is a workability requirement. For those who use fibers a relationship plot comparing ARS and workability with incremental fiber dosage for allowed fibers should be established. This plot will provide optimal fiber dosage that will maximize ARS while meeting the minimum workability requirement. Steps to creating this relationship plot are provided in 4.5.6.

Fiber Type

The most used synthetic fibers are typically made of polypropylene. Although polyester fibers are being produced, they are not as widely used. Synthetic fibers can be either "normal" or "structural. Normal synthetic fibers are much weaker than structural synthetic fibers and can only assist in shrinkage reduction. Structural synthetic fibers are typically rigid and relatively much stronger in tension. Normal synthetic fibers are quite useful to reduce plastic shrinkage in concrete in early age. Structural synthetic fibers are useful for both shrinkage reduction and toughening because of their strength.

Steel fibers are primarily made of carbon steel, although stainless steel fibers are also manufactured. Perhaps the biggest advantage of steel fibers is their high tensile strength and their ability to bridge joints and cracks to provide tighter aggregate interlock, resulting in increased load-carrying capacity. Steel fiber reinforced pavements exhibit excellent toughness.

Fiber Geometry and Shape

The aspect ratio is an important parameter influencing the bond between the concrete and the fiber, with longer fibers providing greater bond strength and toughness, often at the expense of workability. Steel fibers and structural synthetic fibers may also have certain geometric features to enhance pullout resistance, or anchorage, within the concrete mixture. These features may include crimped or hooked ends or surface deformations and irregularities.

Fiber Dosage

Addition of fibers impacts ARS and workability. The severity of these changes will be dependent on fiber dosage.

Increasing the fiber dosage can lead to the following:

- Lower workability.
- Slightly lower compressive strength.
- Slightly lower flexural strength.
- Higher ARS.

Decreasing the fiber dosage can lead to the following:

- Higher workability.
- Lower ARS.

6.3.9 Reinforcement Bars

Although using reinforcement bars (typically, No. 4 or 5) is a time consuming process in concrete overlay construction, it is one of the best methods to reduce shrinkage at the interface and, thus, increase bond strength. The reinforcement bars provide abundant tensile strength to the concrete and, consequently, reduce interface shear stresses that can lead to reduction in bond strength. It provides more reliable interface strengthening than fibers or wire mesh.

6.4 Flowchart

The following flowchart (Figures 6.1 and 6.2) summarizes in a simplified way the methodology proposed for the project selection stage. It is intended to assist in the materials selection process for concrete overlay constructions through a step-by-step procedure. The steps should be used as a checklist, and the flowchart users must refer to the guideline for in-depth understanding the appropriate materials choices.

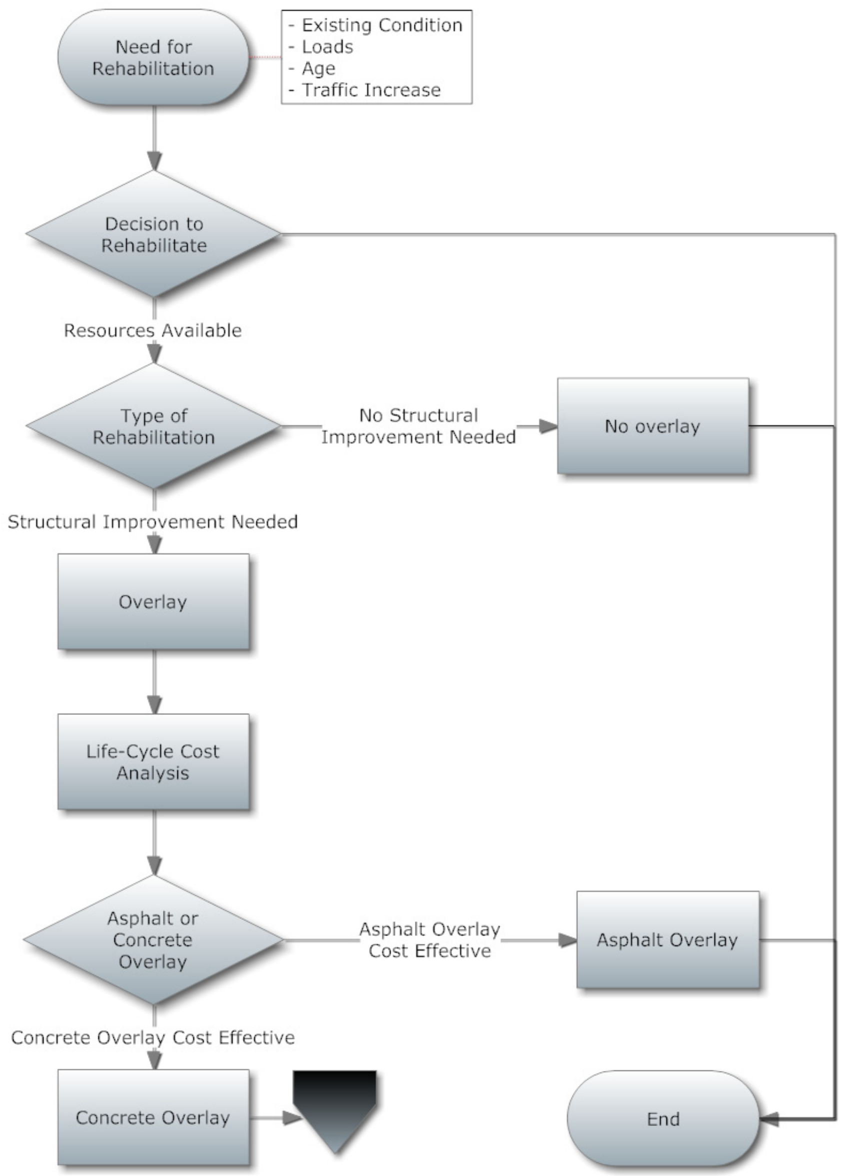

Figure 6.1: A conceptual flowchart of the project selection stage

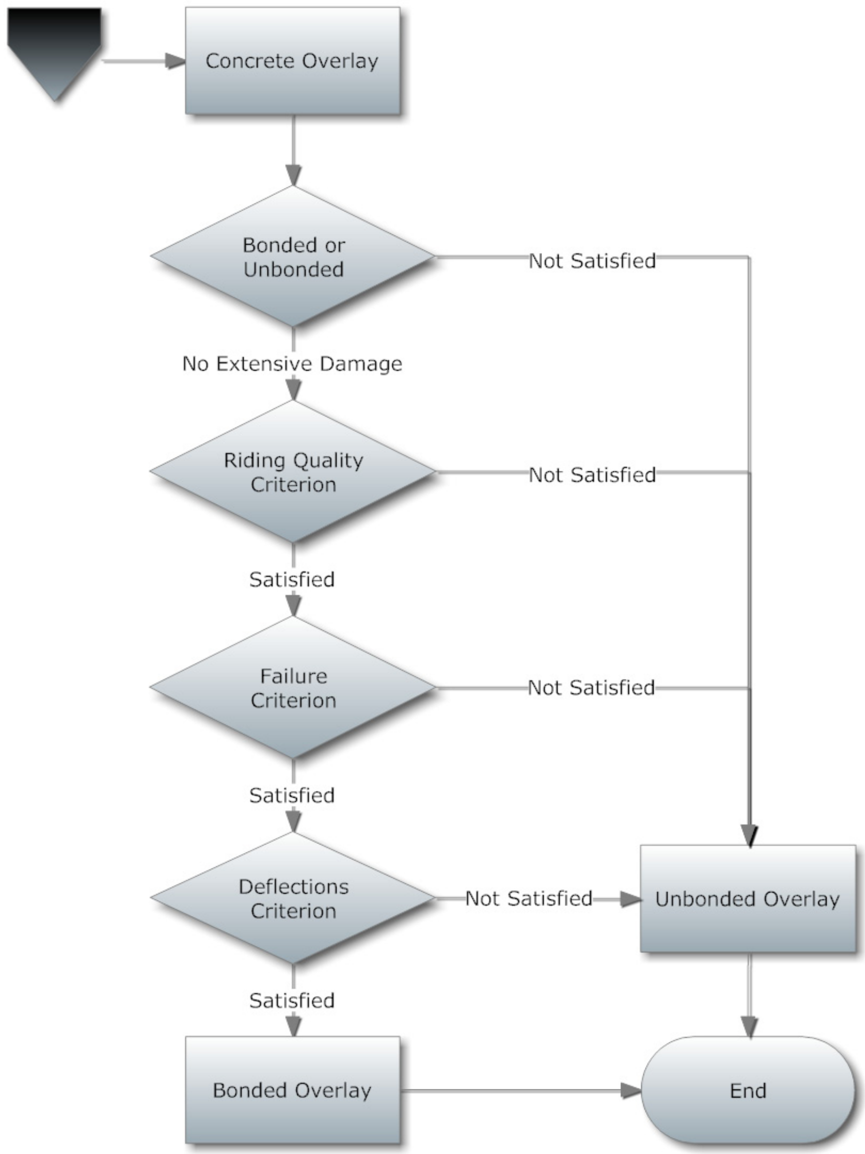

Figure 6.2: Continued conceptual flowchart of the project selection stage

6.4.2 Materials Selection Flowchart

The following flowcharts (Figures 6.3 - 6) have been developed by combining laboratory research results and the literature recommendations. It is intended to assist in the project selection and materials selection processes for concrete overlay constructions through a step-by-step procedure. The steps should be used as a checklist, and the flowchart users must refer to the guideline (Chapter 3) for in-depth understanding the appropriate materials choices.

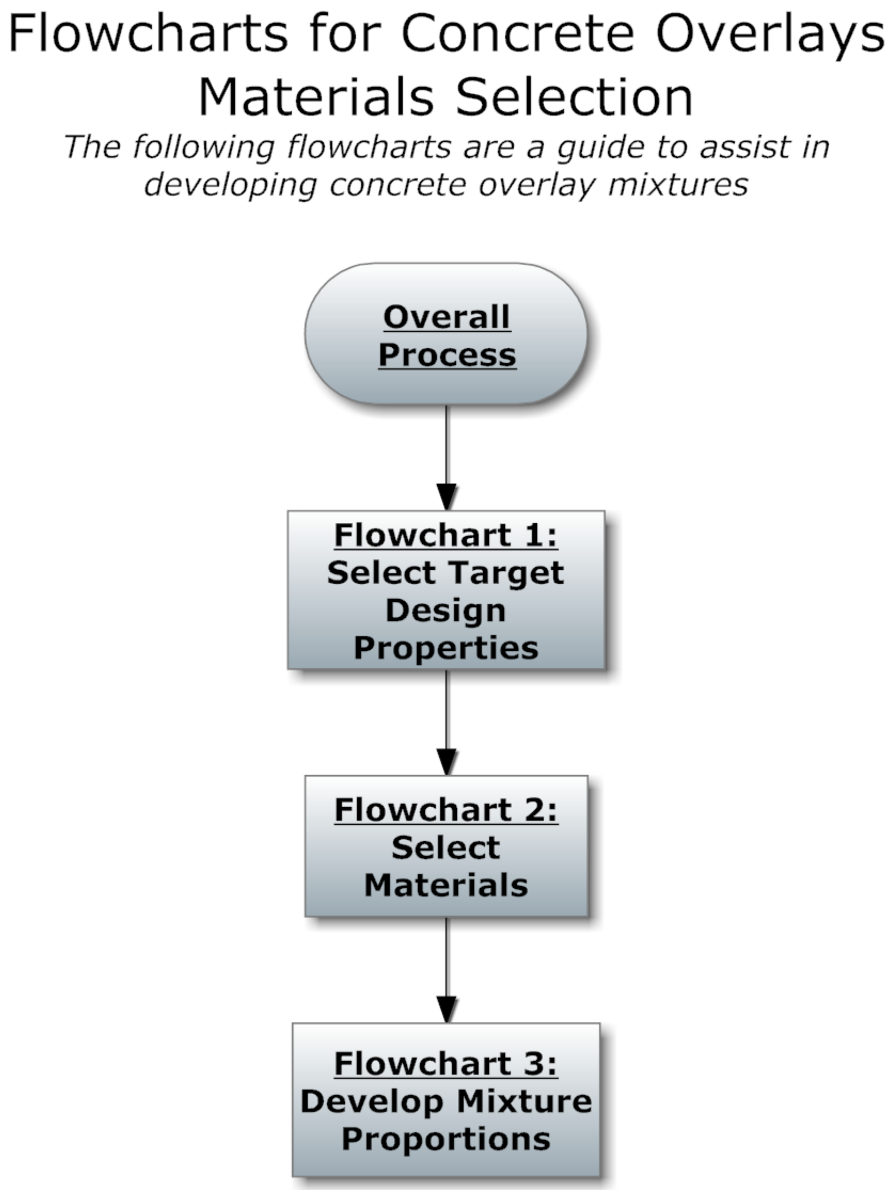

Figure 6.3: Concrete overlays materials selection flowchart

Flowchart 1: Target Design Properties

This flowchart provides a checklist for recommended target design properties for concrete overlay application (see corresponding sections in the 0-6590 training manual for further information)

Compressive and Flexural Strength (Sec. 3.2.1, 3.2.2)

-Min. comp.: 3500 psi (7-day) or 4400 psi (28-day)
-Min. flex.: 570 psi (7-day) or 680 psi (28-day)

To Increase Strength:
-Decrease w/c
-Decrease fly ash
-Use CA with higher strength

To Decrease Strength:
-Increase w/c
-Increase fly ash replacement
-Fiber addition

Drying Shrinkage (Sec. 3.2.6)

Minimize shrinkage as much as possible

To Reduce Shrinkage:
-Increase fly ash
-Decrease w/c
-Decrease w/c ratio

To Increase Shrinkage:
-Increased w/c ratio
-Increase w/c
-Decreased fly ash

Workability (Sec. 3.2.7)

Criteria depends on the project and construction method

To Increase Workability:
-Increase fly ash
-Low FA content
-Admixtures
-Increase w/c ratio

To Decrease Workability:
-Addition of fibers
-High FA content
-Using angular aggregates
-Decreased w/c ratio

For BCOs, also check for...

Bond Strength (Sec. 3.2.4)

-Minimum bond strength: **Greater than 200 psi**

To Increase Strength:
-Prep. surface properly
-Increase cement
-Decrease fly ash
-Add fibers

To Decrease Strength:
-Lack of surface prep.
-Decrease cement
-Increase fly ash
-Use of CA with high CTE

Coefficient of Thermal Expansion (CTE) (Sec. 3.2.3)

-CTE of CA ≤ CTE of the existing pavement

Example: it is advisable to utilize a limestone aggregate for the BCO concrete if the existing concrete has siliceous river gravel as CA, because of the lower CTE in limestone; use of high CTE overlay will increase likelihood of delamination. If the existing concrete has limestone aggregate, only limestone aggregate as CA should be used for the concrete overlay

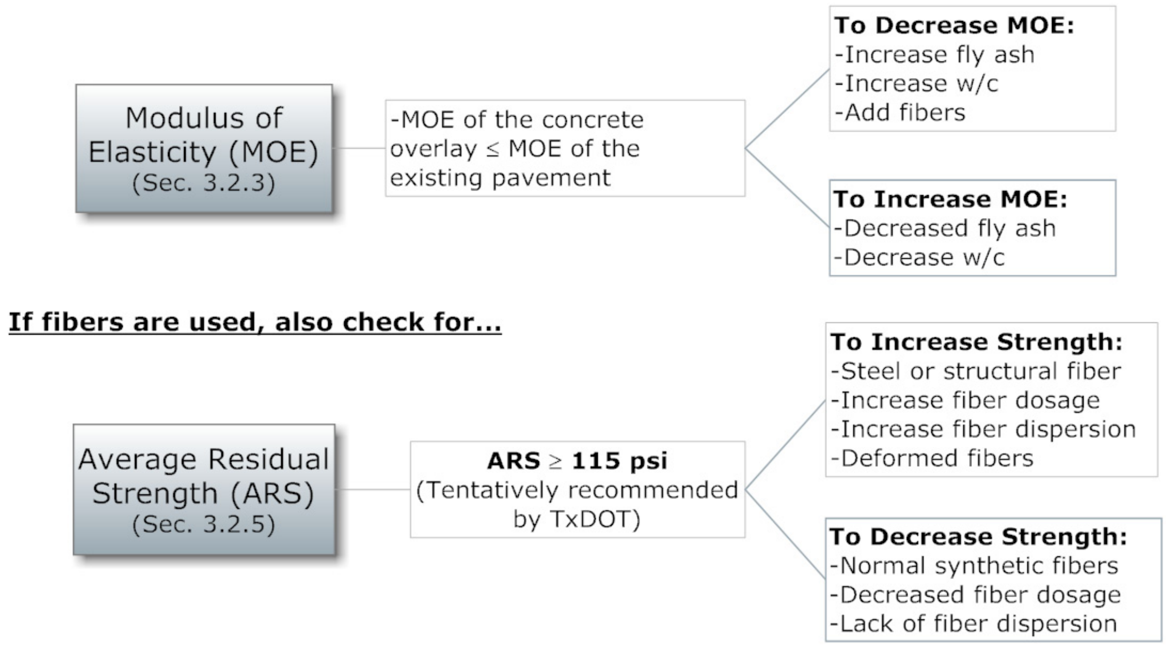

Figure 6.4: Flowchart 1

Flowchart 2: Materials Selection

This flowchart provides a checklist for selecting recommended materials for concrete overlay application (see corresponding section in the 0-6590 training manual for further information)

Cement
(Sec. 3.3.1)

Normal Placement:
-Type I/II

Expedited Placement:
-Type III or more finely ground Type I
-Will result in higher heat of hydration, which will encourage thermal cracking

Fly ash
(Sec. 3.3.3)

Consider using a Class F or C fly ash if the need is:
-Improve workability, finishing, and durability
-Lower heat of hydration
-Reduce cost

NOTE: Initial strength gain may be reduced, especially with a Class F fly ash

Aggregates
(Sec. 3.3.5)

Coarse Aggregate (CA):
-CTE and MOE of CA $\leq$ existing pavement
-Min. CA size = 0.5 inch
-Max. CA size = 1/3 the overlay thickness < 1 to 1.5 inches

Fine Aggregate (FA):
-Natural FAs are preferred due to concerns with skid resistance performance of concretes w/ manufactured FA
-non reactive and low absorption
-Acid insoluble residue > 60%

Admixtures
(Sec. 3.3.7)

Water Reducing Agent:
-Improves workability
-Improves distribution of fibers in mixture

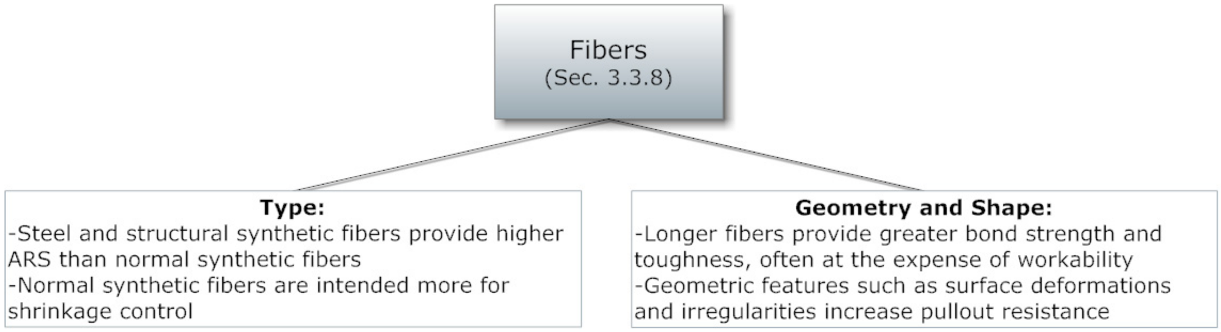

Figure 6.5: Flowchart 2

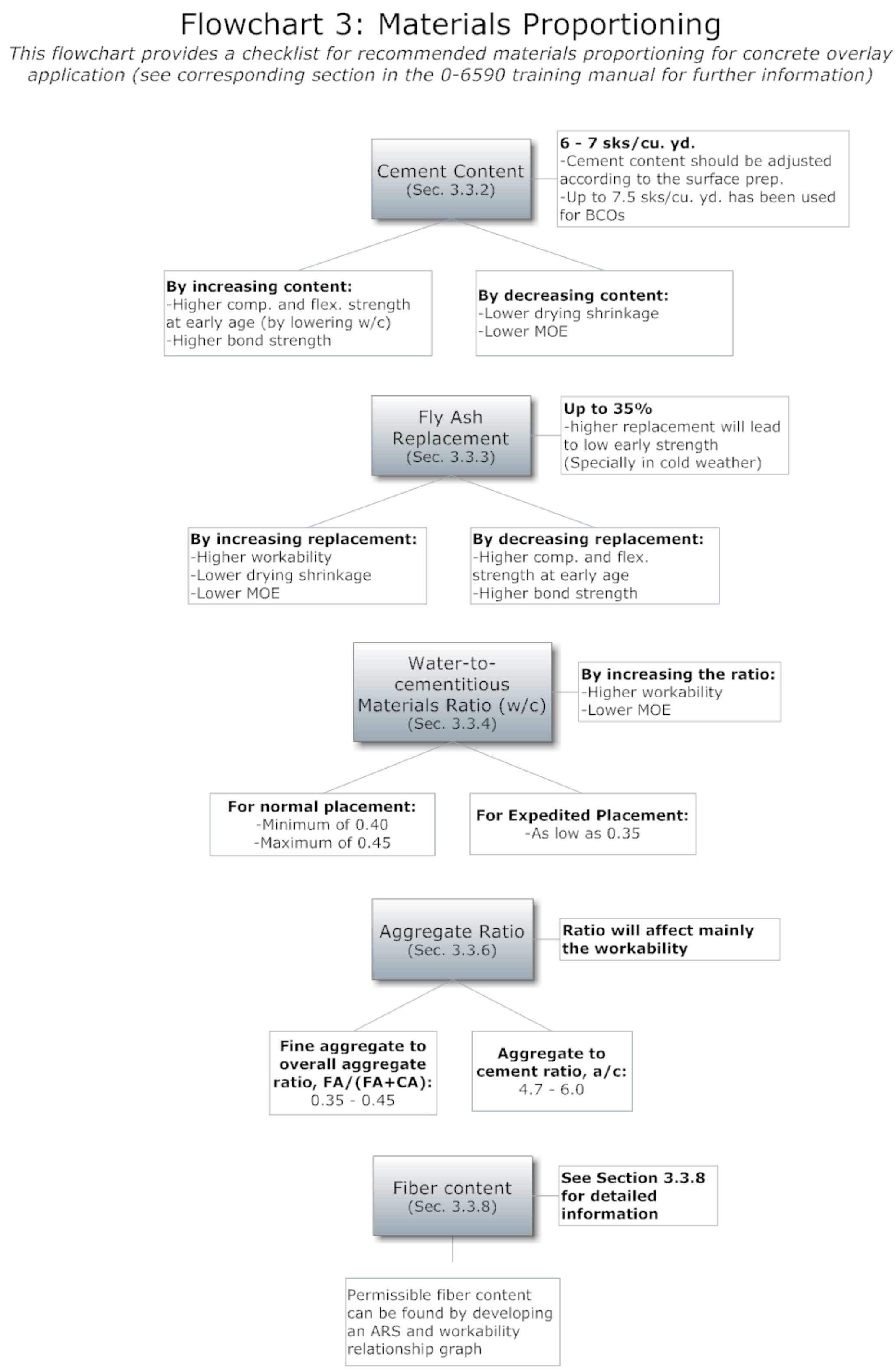

Figure 6.6: Flowchart 3

Chapter 7. Guidelines for Construction Procedures

In many aspects, it is not an overstatement to say that construction quality determines the quality of the concrete overlay. Selecting the right materials is important, but in many ways exercising proper construction methods is more important.

7.1 Introduction

This guideline gives recommended methods for each step of concrete overlay construction. The information in Chapter 7 is summarized from the literature review found in Chapter 2. The following items are discussed:

- Surface Preparation.
- Concrete Overlay Placement.
- Finishing and Curing
- Quality Assurance/Quality Control (QA/QC)

7.2 Surface Preparation

The following steps to achieve a well prepared surface are crucial in promoting successful interface bond between the concrete overlay and existing pavement. Overlays are classified as either "bonded" or "unbonded" and their success depends upon different surface preparation methods.

7.2.1 Bonded Type Overlays

Bonded overlays are concrete overlays that form a lasting bond with the existing pavement. Included in this category are bonded concrete overlays, ultrathin whitetoppings, and thin whitetoppings. The main goal of surface preparation for bonded overlays is to provide a good bonding surface because the bond will result in monolithic behavior from the concrete overlay with the existing pavement. The concrete overlay relies on a clean, rough and sound surface to achieve maximum bond strength between the existing pavement the overlay. Once the overlay has been placed, the new slab thickness is adequate to support the current and future design traffic loadings. There are five sequential steps for surface preparation:

- Surface repair.
- Bituminous and foreign material removal.
- Surface texturing.
- Surface cleaning.
- Wetting the surface before placement.

Surface Repair

Typically, bonded overlays are chosen for existing pavement with minor deterioration that require minimum repair. Bonded overlays are usually relatively thin (2 to 4 in.) and rely on the existing pavement to carry most of the traffic load. Therefore, the existing pavement should be in decent condition with transverse cracking, but without requiring extensive repair for spalls and punchouts.

It is recommended to perform spot-repairs to any severely deteriorated areas. Working cracks must be repaired or sawed out, removed or replaced, since they will reflect through the new concrete overlay. Severe edge failures and any longitudinal cracks must be patched. Localized areas of weakness can be strengthened through patching or can be removed. Figures 7.1 and 7.2 show typical distresses found in existing pavements and possible repairs for candidate bonded type concrete overlays.

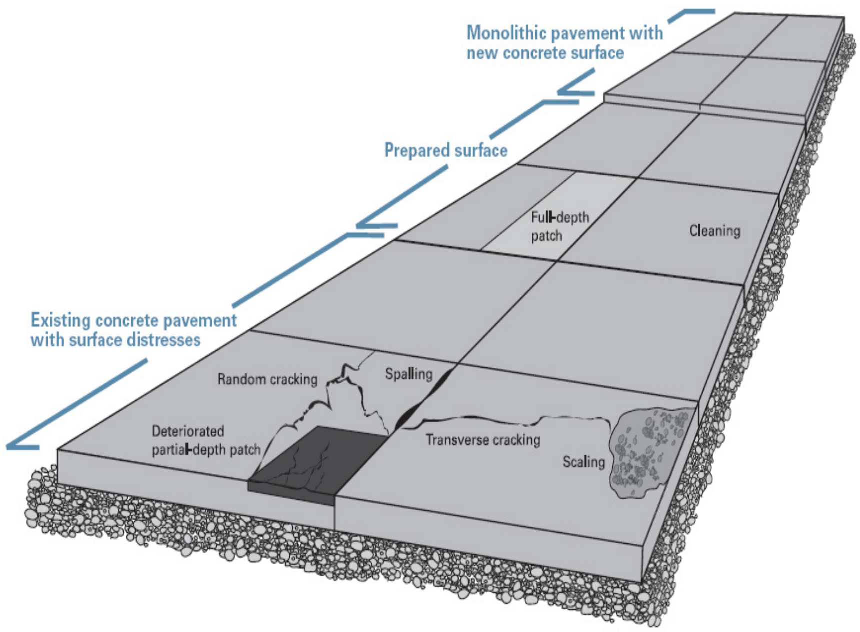

Figure 7.1: Notable distress on existing pavement and possible repairs for BCOs [4]

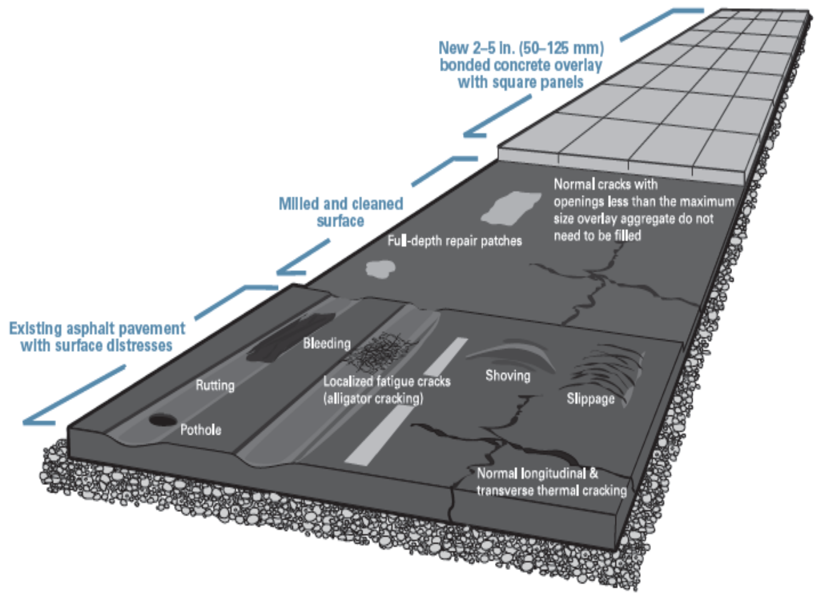

Figure 7.2: Notable distress on existing pavement and possible repairs for bonded whitetoppings [4]

Bituminous and Foreign Material Removal

Since the main goal is to promote good bond, any undesirable materials that are present on the surface and that may hinder the bond must be removed. If the existing pavement is a concrete pavement, any bituminous patching material found on the pavement surface must be removed prior to the overlay placement. Cold milling can be used for large areas, while jackhammers can be used to remove smaller areas filled with incompatible patching materials. Sandblasting can be used in areas where the shotblasting equipment cannot operate, including areas of distress and the pavement edges. Also, paint stripes and joint sealing compounds should be removed because, although these areas are small, they can act as origination sites for delamination.

Surface Texturing

Well and consistently textured surface is critical in encouraging aggregate interlock that promotes monolithic bonding. The idea here is to remove enough of the surface of the existing pavement to expose some of the coarse aggregate profile. Minimum texture depth should be 0.25 in. after coarse aggregate is exposed. The circular track meter (CT Meter) can be used to accurate by measure mean profile depth (MPD) and can be correlated to mean texture depth (MTD) to calculate the average textured depth of the prepared surface and, also, consistency of the texture can also be found. There are three most often used texturing operations: Milling, shotblasting, and sandblasting.

- Milling: Milling is ideal when the existing pavement is asphalt or asphalt overlaid. However, milling is not recommended for removing the top surface of the original concrete in pavements, because it tends to damage the coarse aggregate instead of exposing them, and it causes microcracking in the concrete substrate's surface. However, some of the new milling machines are less likely to produce microcracking. A stringent grade must be maintained during milling operations in surface preparation. Figure 7.3 shows a milled surface of an existing asphalt pavement.

- Shotblasting: It is most often used for cleaning and surface texturing concrete pavements. Shotblasting is intended to cleanly expose the coarse aggregate and provide a roughened surface texture to increase the bond of the overlay. Figure 7.4 shows a shotblasted surface of an existing concrete pavement.
- Sandblasting: Sandblasting is ideal for small and hard to reach areas.

Figure 7.3: Milled existing asphalt pavement

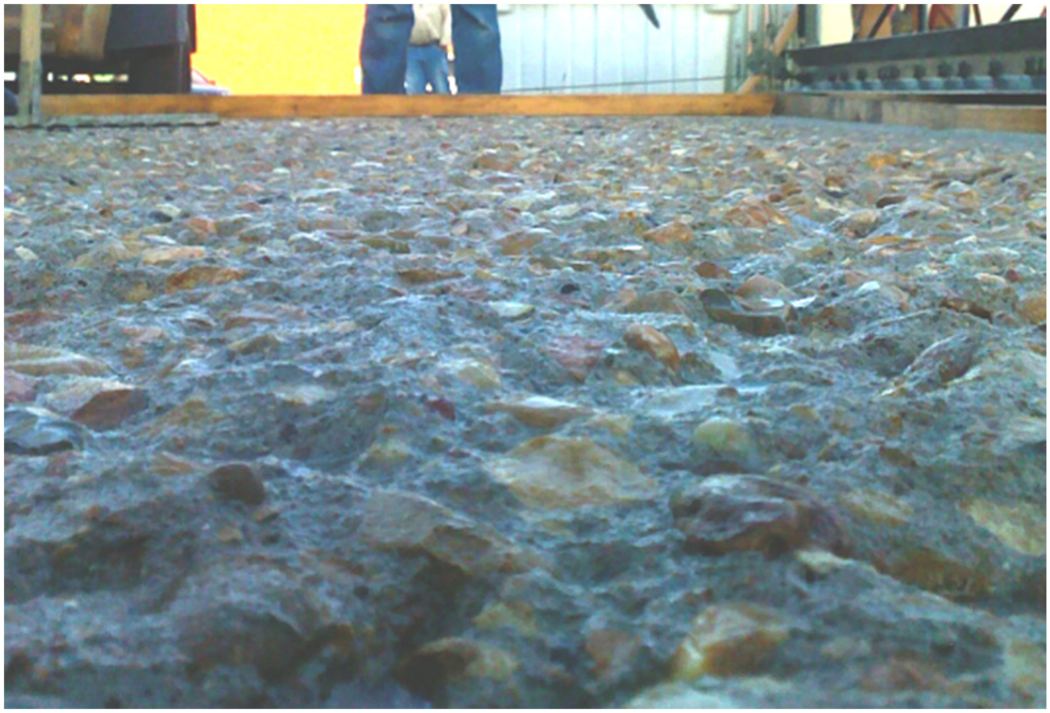

Figure 7.4: A shot blasted existing concrete pavement

Shotblasted or sandblasted fines should not be piled along the side of the road because they can be blown by wind or tracked back onto the surface contaminating it. Shotblasting/sandblasting should be performed near the paving operation, typically on the same day as paving. The cold milling and shot blasting combination proved to be a better method of preparing the existing pavement than either milling or shot blasting alone.

Surface Cleaning

Immediately prior to placing concrete, the prepared and textured surface should be thoroughly cleaned of all dust and loose particles by vacuum, air blowing, and/or by hydro blasting. During cleaning the surface should be dry, and thoroughly cleaned of all vegetation, dirt, mud, and other contaminants. Figure 7.5 shows a worker using an air hose to clean the surface and spread pooled water just ahead of the paver.

Figure 7.5: Use of an air hose to clean the surface and spread pooled water just ahead of the paver

Tarps should be required if trucks or other equipment will be driving on the cleaned surface. Trucks used for transporting concrete will be permitted to drive on the pavement being overlaid and deposit concrete directly in front of the concrete spreader, provided no loose foreign material oil or dirt is dripped or tracked onto the surface.

Wetting the Surface before Placement

The goal is to achieve saturated surface dry (SSD) condition on the cleaned surface in order to lower the surface temperature and prevent moisture loss from the fresh overlay to the existing pavement. The surface should be adequately wetted (but free of puddled water) immediately before the placement. Pooling of water must be avoided since the excessive water may introduce weakened planes at the interface and hinder bond strength. Figure 7.6 shows a surface that has been wetted but beginning to dry.

Figure 7.6: Wetted surface and dried surface

If the existing pavement is asphalt, spraying water on the asphalt surface ahead of the paver keeps the surface cooler, but should be kept to a minimum and allowed to dry to a SSD before the concrete overlay is placed in order to promote a good bond. The surface temperature should be less than 100 F (38 C) at the time of placement. This may require night placement, water fogging or other approved means of obtaining a cooler surface, however there should be no pooled water or other contamination to prevent bonding to the asphalt surface.

7.2.2 Unbonded Type Overlays

Unbonded overlays are concrete overlays that are intentionally not bonded to the existing pavement, such as unbonded concrete overlays and thick whitetoppings. The main goal of surface preparation for unbonded overlays is to isolate the concrete overlay from the existing pavement. In this strategy the existing pavement only serves as a subbase. Once the overlay has been placed, only the new concrete overlay supports the current and future design traffic loadings, as well as the environmental stresses. There are three sequential steps for surface preparation prior to unbonded type overlays:

- Surface Repair.
- Separation Layer Placement (for UBCOs).
- Wetting the Surface Before Placement.

Surface Repair

Typically, unbonded overlays are chosen for existing pavement with major deterioration. The condition of the existing pavement is typically poor enough that design only relies on it for providing a uniform strength platform for the new concrete overlay. However, some effort should be made to ensure that there is no reflective cracking from severely deteriorated areas. If there are questionable areas, they can be repaired, altered, and/or removed and replaced. Full-depth repairs are required only where structural integrity is lost in isolated locations. Unbonded type overlays are usually relatively much thicker (5 to 11 in.) and carry the entire traffic load. Figures 7.7 and 7.8 show typical distresses found in existing pavements and possible repairs for candidate bonded type concrete overlays.

Separation layer placement (for UBCOs)

A separation layer is essential to isolate the unbonded concrete overlay from the existing concrete pavement, to minimize reflective cracking, and to provide a smooth and level surface. Typically, an. asphalt layer is placed about 1-in thick. Some states have used a 2-in. asphalt layer. For whitetoppings that are intended to be unbonded, no separation layer is required. However, the substrate surface should be level and provide a uniform strength platform. Figure 7.9 shows construction of an UBCO.

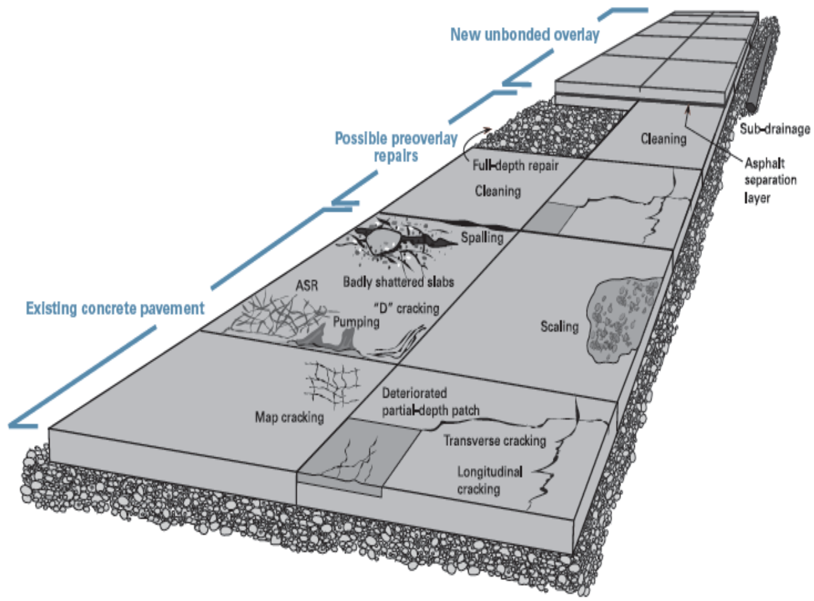

Figure 7.7: Notable distress on existing pavement and possible repairs for UBCOs [4]

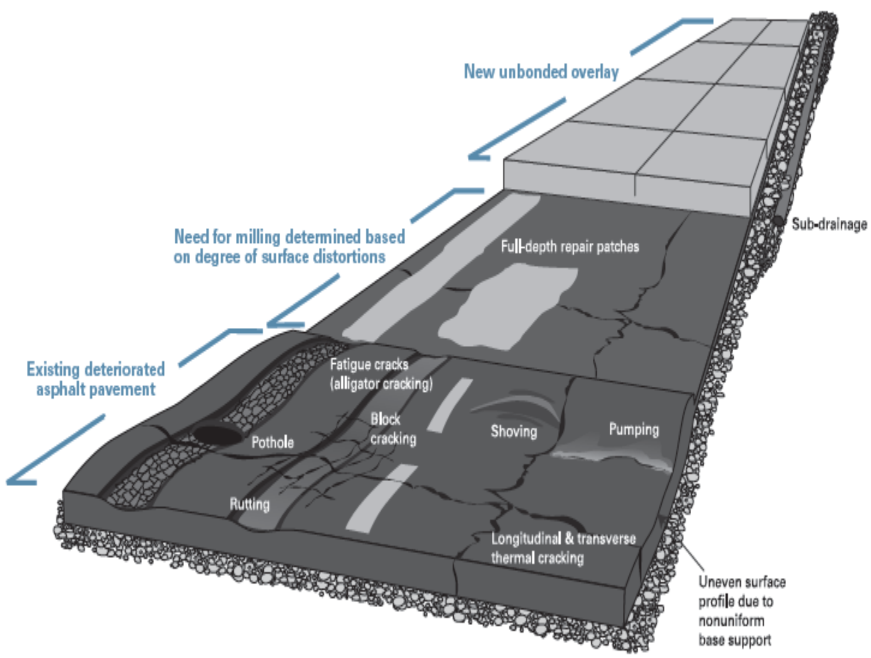

Figure 7.8: Notable distress on existing pavement and possible repairs for thick whitetoppings [4]

Figure 7.9: Existing pavement, 2-in. thick asphalt separation layer, and concrete overlay placement

White pigmented curing compound may be applied to the asphalt interlayer to further deter bonding and reflect heat so that large temperature differences do not develop between the bond-breaking interlayer and the concrete overlay.

Wetting the Surface before Placement

The goal is to lower the surface temperature of the substrate or bond breaking interlayer. The surface should be adequately wetted (but kept at minimum) immediately before the placement. If the existing pavement is asphalt, spraying water on the asphalt surface ahead of the paver keeps the surface cooler, but should be kept to a minimum and be sprayed off with clean (oil-free), dry compressed air or allowed to dry to a saturated surface condition (SSD) just before the concrete overlay is placed.

7.3 Concrete Overlay Placement

The following procedures are recommended for proper concrete overlay placement:

- Plan for and monitor environmental considerations.
- Bonding agent application (if required).
- Consistent concrete placement.
- Reinforcement placements.
- Dowel placements.

7.3.1 Environmental Considerations

Weather conditions prevailing during concrete overlay construction can be critical to the overlay performance. Environmental variables that play a key role in the behavior of the concrete overlay are ambient temperatures and moisture or humidity and wind speed at the specific placement site. Rising winds can almost immediately turn acceptable evaporation rates into unacceptable ones. Hot and dry climates, particularly under windy conditions, pose the most problematic setting for concrete overlay placement, because these conditions exacerbate the loss of moisture from the fresh concrete. Excessive water evaporation from the concrete causes plastic shrinkage cracking, which reduces the integrity of the concrete surface and reduces its durability.

A combination of high wind velocity, high air temperature, low relative humidity, and high concrete temperature is the most harmful for paving conditions, because it results in a high rate of water evaporation. Placing in minimal winds, during low temperature months, i.e., December and January, can minimize climatic stresses and cracking.

Caution must be taken in hot, dry, and/or windy climates that can cause excessive evaporation of water from concrete and produce plastic shrinkage cracking. Weather stations should be used to monitor the weather condition, and the ACI 503Rnomograph should be used to decide the severity of the environment condition. The following adverse conditions must be monitored in construction.

- Surface of the existing pavement should not exceed 125 °F immediately before placement.
- The predicted temperature differential within 24 hours after the placement must be less than 25 °F.
- A condition where water evaporation rates exceeding 0.2 lb/ft.^{2}/hr based on the ACI 503R monograph.

Contractors and inspectors must be prepared for an established plan of action when any of the adverse conditions mentioned above occurs during the placement of concrete. The placement should be avoided unless the following conditions can be achieved:

- Cooling the aggregate with cool water or the freshly batched concrete (with ice or liquid nitrogen).
- Cooling the prepared surface of the existing pavement with water
- Special curing methods (See section 2.5.7).
- Use of fly ash as cement replacement to lower the heat of hydration.

7.3.2 Bonding Agent Application

Bonding agents, e.g., portland cement grouts, latex modified portland cement grout, and epoxy resins, are sometimes used to improve bond. However, bonding agents cannot compensate for bad substrate surface preparation and may act as a bond breaker when used inappropriately; therefore it is not recommended to use bonding agents, unless under special circumstances. The use of bonding agents leads to two interfaces and thus to the creation of two possible planes of weakness instead of one. In addition, grout often has a high water-cement ratio leading to low strength and the risk of a cohesive failure within the bonding agent itself. Bond strength of the concrete overlay does not vary significantly whether with or without a bonding agent [25]. Also, placing bond agent is cumbersome, and it slows down the paving process.

If the surface happens to be wet, a concrete grout will assure better bond strength. Nevertheless, it would be much safer, if the construction can wait until the surface is dry and SSD condition is achieved just before the placement. Typically water-to-cement ratio of the grout is around 0.62 to 0.70 by weight [27], or approximately seven gallons of water per sack of cement. Grout slurry should be applied as a thin, even coat onto the cleaned SSD concrete surface just ahead of the paver. Figure 2.3 shows that immediately before paving, a grout can be uniformly broomed over the full width of the prepared surface.

When the substrate has been treated only by a less expensive surface cleaning procedure that develops minimal substrate surface texture insufficient to guarantee an adequate bond, epoxy resin bonding systems have been reported to provide extremely high bonding strengths in the laboratory (higher than 5000 psi)[30].

7.3.3 Consistent Concrete Placement

Air content and slump should be measured at regular intervals for consistency. Also, consistency in the thickness of the overlay must be checked against the designed thickness regularly. The minimum overlay thickness requirement must be met for successful performance. Transition zones from in-service existing pavement to the newly placed overlay should have thicker layers because higher stress has been reported at the transition zones.

7.3.4 Fiber Reinforcement Additions

In one unbonded overlay project [23] steel fibers were added in with the aggregates via a conveyor belt. Since fiber balling is one of the biggest problems, several attempts to reduce or eliminate the fiber balls and uncoated fibers were made at the mixing plant. Introducing the fibers into the mixture sooner, increasing the mixing time slightly, and reducing the batch size were all approaches taken to eliminate this problem. While some slight improvement was noticed following these alterations, some fiber balls and uncoated fibers were still observed throughout the overlay. The moral of the story is that all fibers must be added to the concrete mixture and mixed according to their respective manufacturer's recommendations.

7.3.5 Dowel Placement

Sufficient coverage of concrete over the dowel bars is needed to prevent spalling over the dowel bars. Thin overlays do not provide enough cover over dowel bars and can cause early spalling. Dowel bars should not be placed in thin overlays.

7.4 Finishing and Curing

Unlike the methods used for typical pavement constructions, finishing and curing methods for concrete overlays require more attention to the specified details. Thinner applications of fresh concrete dry out to critical levels in much less time, so plastic shrinkage is much more likely in drying conditions.

7.4.1 Unweighted Carpet Dragging

The use of a burlap drag is not recommended for finishing fiber-reinforced concrete overlays, because it can result in a poor finish due to the fibers becoming entangled in the burlap, leading to other fibers and coarse aggregate being pulled from the surface of the pavement. An unweighted carpet drag is an alternative option to provide an acceptable finish. Figure 7.10 shows a finished surface with carpet dragged surface texture.

Figure 7.10: Carpet dragged surface texture

7.4.2 Tining

After the completion of the hand finishing, transverse tining should be performed in a smooth and timely manner. The curing compound should be sprayed on the concrete surface immediately following the tining procedure.

7.4.3 Curing

Curing prevents moisture loss and thereby reduces early age shrinkage, leading to higher tensile strength at the onset of normal drying shrinkage. Simultaneously, other advantages are gained: reduced risk of cracking, higher strength, improved durability, and better wear resistance [11]. Curing compound should be applied promptly following paving operation. Figure 7.11 shows curing compound being promptly applied following paving operation.

Figure 7.11: Curing compound being promptly applied following paving operation

Typically, curing compound is applied at a much higher rate for overlays than the curing compound application rate for standard pavement placement. Thinner overlays require higher rates of application and more rapid application after finishing the overlay surface. Curing compound should be applied at 1.5 to 2 times the normal application rate [5, 32]. If blankets are used for fast tracking, they should be light in color and should not take the place of a curing compound. The temperature under the blanket must not exceed 160° F. Blankets should not be removed until the temperature under the blanket is within 40° F of the ambient temperature.

7.4.4 Jointing

Curing prevents moisture loss and thereby reduces early age shrinkage, leading to higher tensile strengths. The timing of joint sawing is critical. Sawing too early can cause excess raveling, and sawing too late can result in shrinkage stresses, causing uncontrolled random cracking. Early entry saws should be used before internal concrete stresses could build and cause cracking. No cracking should be observed prior to or following the sawing operation. Narrow joints, 1/8-in. wide by 1-in. deep, produced from these saws should minimize incompressible materials entering the joints; therefore no joint sealant is necessary. A maturity curve can be used to predict the right timing for the sawing operation. When the concrete is predicted to have attained adequate strength, according to the maturity curve, the early entry sawing can be allowed to begin. Figure 7.12 shows an early entry sawing at work.

Figure 7.12: Early entry sawing

For BCOs, joints in the overlay must match the joint orientations in the existing concrete pavement. Transverse expansion joints and longitudinal lane joints should be cut or placed to match the underlying joint configuration. In order to properly locate the saw cuts in the concrete overlay, the location of all transverse expansion joints and longitudinal lane joints in the existing pavement should be identified by a reliable method. The contractor must receive approval from the engineer for the procedure to be used to mark and relocate existing joints.

Joint spacing has a significant effect on the rate of corner cracking. Short joint spacing, common on thin concrete overlays, reduces load-related stresses, because the slabs are not long enough to develop as much bending moment. ACPA recommends that joint spacing be about 12 to 15 times the slab thickness.

To reduce the edge and corner stresses, longitudinal joints should not be placed in the wheel path. Heavy loads concentrated near the edge of the thin panels should not exceed their load capacity. For example, 4-ft. by 4-ft. panels on a 12-ft.-wide lane would put truck tires on the edge of the panels, and significant distress would occur, if the thin concrete overlays became de-bonded from the existing pavement. The joints are not to be sealed but should be cleaned of all deleterious material after sawing. Figure 2.8 shows a good example of failed joints in wheel paths.

7.5 Quality Assurance/Quality Control (QA/QC)

A comprehensive QA/QC program is a last step in concrete overlay construction. It is required to ensure that owners have a cost-effective and serviceable concrete overlay pavement with a uniform, durable, safe, and low maintenance riding surface for the public. The program should include some or all of the following items:

- Strength evaluations of overlay concrete such as compressive (ASTM C39 - 10), flexural (ASTM C78 - 10), and/or maturity testing (ASTM C1074 - 11): Compressive and flexural tests must be performed on field specimens to see if required strengths are met. The intent of maturity testing is to predict in-situ concrete strength in lieu of many cylinders, because cylinders are not always considered representative of actual in-place strength. In-place strength is felt to be higher because of the increased heat generated by a larger volume of concrete. Increased heat means a higher rate of hydration and hence, higher strengths. When time is of the essence, maturity testing can save time and money, while minimizing the inconvenience to the traveling public. It is important to note that a few proving cylinders must still be made and tested to prove that predictions of strength have actually been achieved before strength-critical operations are allowed.
- Bond strength testing: Pull-off tests (ASTM C 1583) should be performed to see if adequate bond strength has been achieved.
- Condition survey by visual monitoring of signs of distress: Visual distress surveys should be performed yearly for at least 5 years to determine development of cracking and any spalling. If the overlay is still performing well after 5 years, the surveys should be continued at whatever intervals deemed appropriate.
- Condition survey by locating area of delamination: By performing sounding tests (such as, chain dragging or bar dropping) locations and area limits of delaminations can be found. A solid sound indicates a non-delaminated area, while a hollow sound indicates a delaminated area.
- FWD testing: Nondestructive testing of pavements using FWD is excellent for in-service pavements. The data from these tests can be analyzed to obtain the effective stiffness of each pavement layer. This information is used to determine where pavement layers are weak, and hence evaluate the likely causes of pavement distresses. The FWD data can also be incorporated into an analytical (mechanistic) pavement design approach for assessment of remedial measures.

Chapter 8. Summary and Conclusions

8.1 Summary

Although concrete overlays have been used as a standard rehabilitation method on PCCP for many years in other states, implementation of the best concrete overlay methods may still be considered to be in the initial stages in Texas. The large volume of concrete highways in Texas makes bonded concrete overlays, unbonded concrete overlays, and whitetoppings very viable options. However, there is a lack of educational guidelines for pavement engineers for concrete overlay construction, and this makes further implementation difficult.

The goal of this research was to create guidelines for materials selection and construction methods for pavement engineers, so that they are better prepared for specifying and constructing successful concrete overlays. In order to achieve the goal, appropriate material constituents were tested and construction methods were studied, utilized and learned from literature. The individual task objectives of this research study were:

- Conduct a comprehensive literature review to capture the existing knowledge on concrete overlay usage, materials selection, construction methods and performance histories of concrete overlaid roadways.
- Perform condition surveys on existing concrete overlays in Texas to gather useful materials, traffic and performance information.
- Select candidate materials based on literature review and condition surveys, identify the role and effectiveness of material constituents, bonding agents, mixture design, and concrete placement factors that influence performance of concrete overlays, and to develop a laboratory process to evaluate overlay mixture proportioning.
- Evaluate data with a performance prediction modeling.
- Develop guidelines for materials selection.
- Develop guidelines for construction procedures.

8.2 Conclusions

Constructing concrete overlays differs from constructing a typical pavement. Concrete overlays require special attention in selecting appropriate materials that will promote compatibility between the new concrete overlay and existing pavement. In addition, condition and surface preparation of the existing pavement determines the effectiveness of the concrete overlay. Basically, a new concrete overlay must be designed and constructed, so that it is either bonded to the substrate to behave as one monolithic slab, or it is completely separated from the supporting substrate with a bond-breaking layer.

To guide readers in the design and construction of effective concrete overlays, guidelines have been developed for materials selection (Chapter 6) and construction procedures (Chapter 7). The guidelines are provided to assist TxDOT and their contractors in the selection of appropriate materials with their recommended usage levels and to assist in specifying and exercising proper construction practices to produce successful concrete overlays. These guidelines combine the best information from what was learned in the literature review, site surveys, and laboratory experiments.

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Appendix A

Appendix A-1

Start of SH 146 section Northbound

General view (NB)

Transverse cracks

Closely spaced transverse cracks

Minor spall

AC patch with failure

SRG aggregate

Alexander Dr. Bridge

Patch with Failure


Good condition


Longitudinal joint damage


Punchouts


Punchout
Rain during survey


Southbound direction


Good condition


Spalls


Wide transverse crack

Appendix A-2

Start of BCO (WB)
at Aldine Westfield

Excellent condition

Typical transverse crack

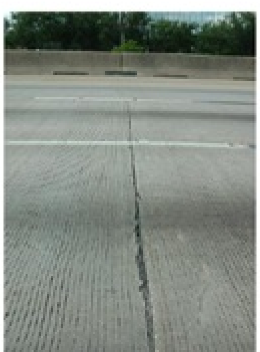
Minor spall


Spall


Fibers seen in spall


Spall


Transverse and longitudinal cracks


Joint damage

Longitudinal crack

WB Survey from the vehicle

AC Patch

Start of EB BCO at Greenspoint Dr.

Excellent condition


Longitudinal crack


Corner damaged repair


PCC Patch

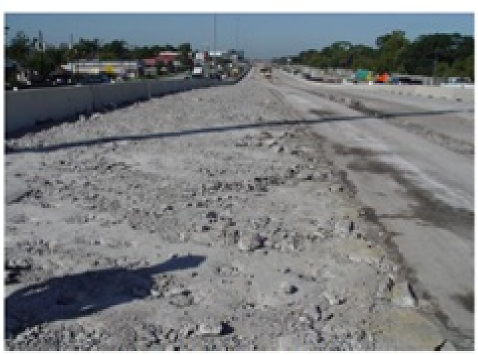

Patches


Open joints

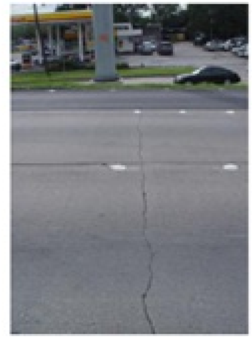
Appendix A-3


PCC Patch


Good condition


Overlay removed


Limestone coarse aggregate




Spall


Patch with spall


Punchout


Steel fibers

Appendix A-4

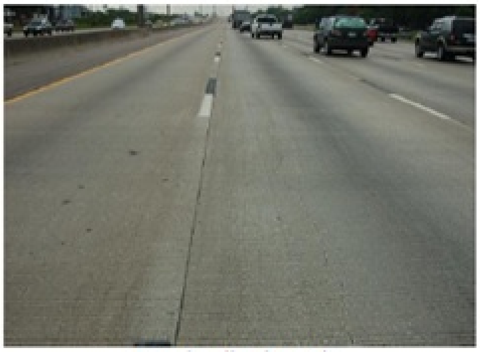
Beginning of survey, east of Calais St.
1989-1990 Experimental sections (EB)

Excellent condition
1989-1990 Experimental sections (EB)

Sounding test
1989-1990 Experimental sections (EB)

Small AC patch
1989-1990 Experimental sections (EB)

Punchout
1989-1990 Experimental sections (EB)

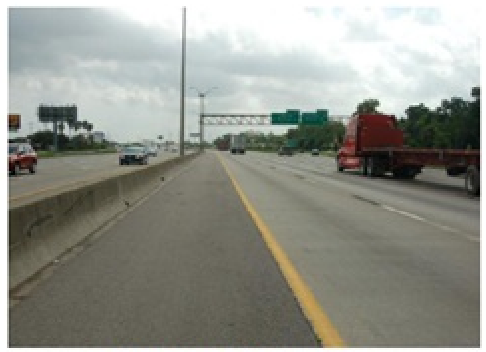


Longitudinal crack
1989-1990 Experimental sections (EB)

Typical transverse crack
1989-1990 Experimental sections (EB)

Small spalls
1989-1990 Experimental sections (EB)

PCC Patches
1989-1990 Experimental sections (EB)



View east of MLK Blvd.
1989-1990 Experimental sections (EB)

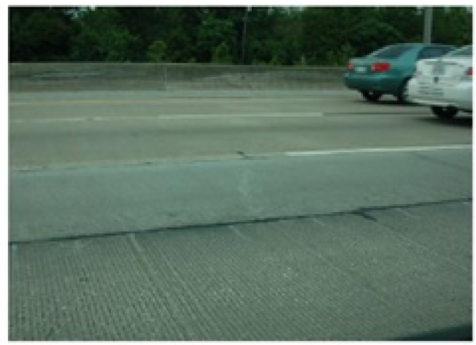
Survey from the vehicle
Eastbound outside lane. 1990 BCO

Transverse cracks


Large spall and patch


AC patch, longitudinal and transverse cracks

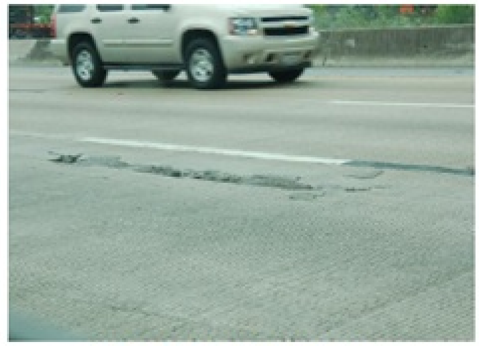

Punchout


Large PCC patch


Longitudinal cracks
Westbound


AC patch

Punchout



AC Patch, longitudinal cracks

PCC patch with spall

Beginning of survey- 1983 BCO
EB outside lane, after Cullen Blvd. overpass, exit 36A

Longitudinal crack
1983 Experimental sections

Excellent condition
1983 Experimental sections

Delaminated area, next to exit ramp
1983 Experimental sections

End of 1983 BCO, at Calais St.

End of 1983 BCO, at Calais St.

Appendix A-5


Future start of BCO


JCP slab in good condition


PCC patch


Shoulder joint damage


Open joint

Areas to be repaired prior to BCO placement marked with spray paint

Wide crack and punchout

Good condition

Damaged PCC patch


ACP patch


End of section

Appendix A-6

Ahead of the Paving Machine

Before the Pour (North)

Before the Pour (South)

Close-Up of Reinforcement at Transition

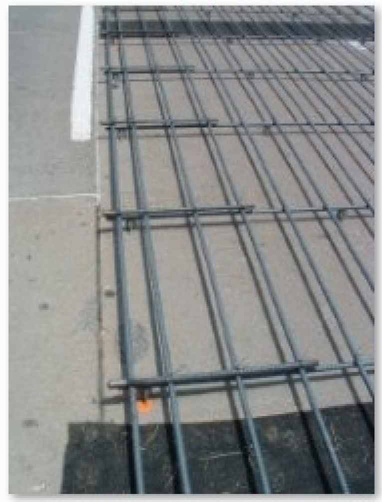
Feeding Concrete

Finishing Machine

Finishing the Surface

Following the Paver

Reinforcement at Transition



Reinforcement Detail

Reinforcements

Slip Machine

Spraying Curing Compound

Start of Construction

Tined Surface

Wet Surface

Appendix A-7

Start of BCO (SB)
at Holliday Creek

Excellent condition

Typical transverse crack

Diamond-ground texture

Narrow longitudinal crack

Delaminated area (edge of lane)


Minor spall


Excellent condition


Transverse and longitudinal cracks
(area of potential punchout)

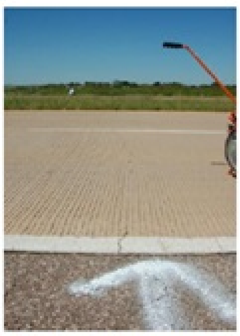

Punchout at joint (SB inside lane)


Delaminated area


Start of BCO (NB)
(Close to RM 196 and County line)

Joint at NB start of BCO

Spall



Delaminated area

Good condition

End of NB BCO at Holliday Creek bridge

Appendix A-8

Picture 1

Beginning of the Laredo Whitetopping section, RM 51

Picture 2

RM 51. Joint between ACP and whitetopping overlay

Picture 3

Transverse cracks

Picture 4

Area of potential punchout (RM 51 + 214 ft)

Picture 5

General view

Picture 6

Punchout (RM 51 + 413)

Appendix A-9

Beginning of westbound BCO

At Missouri St. Bridge, just past RM 20

Traffic loops

Typical transverse cracks

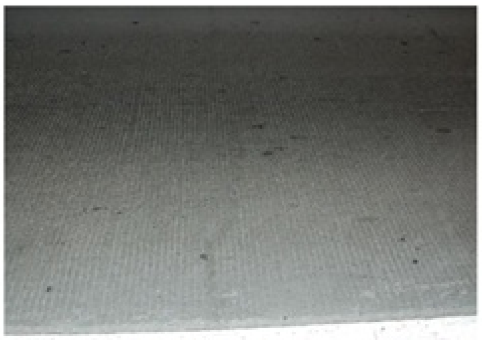
Small PCC patch

Good condition

Sounding of delaminated area with no superficial distresses

Narrow longitudinal crack

Sounding approaching N. Oregon St. Bridge
(Westbound direction)



Excellent condition

Transverse joint at end of BCO
(Westbound direction)

Beginning of BCO, Eastbound direction
at exit 19

Small spall

Spall at joint

Small PCC patch

Excellent condition

Small spalls

Patches at eastbound end of BCO

EB end of BCO


End of EB BCO looking west

Appendix B

Appendix B-1

Table B.1: Coarse Aggregate Grading

Sieve	Mass of Sieve (g)	Sieve + Agg (g)	Mass of Agg Retained (g)	% Retained	Cum. Mass Retained (g)	Cum. % Retained	Cum. % Passed
1	7126.8	7386.3	259.5	2.59	259.5	2.59	97.41
3/4	7830.6	9825.0	1994.4	19.92	2253.9	22.52	77.48
1/2	7093.8	10803.1	3709.3	37.05	5963.2	59.57	40.43
3/8	7308.1	9604.3	2296.2	22.94	8259.4	82.51	17.49
#4	6966.2	8555.2	1589.0	15.87	9848.4	98.38	1.62
#8	5700.5	5849.2	148.7	1.49	9997.1	99.87	0.13

Table B.2: Fine Aggregate Grading

Sieve	Mass of Sieve (g)	Sieve + Agg (g)	Mass of Agg Retained (g)	% Retained	Cum. Mass Retained (g)	Cum. % Retained
4	6967.6	6974.2	6.6	1.32	6.6	1.32
8	6533.5	6594.7	61.2	12.24	67.8	13.56
16	6976.0	7052.1	76.1	15.22	143.9	28.78
30	6838.1	6930.7	92.6	18.52	236.5	47.30
50	7560.7	7699.2	138.5	27.70	375.0	75.00
100	6112.6	6219.5	106.9	21.38	481.9	96.38
200	7368.7	7381.9	13.2	2.64	495.1	99.02
Pan	6084.4	6087.5	3.1	0.62	498.2	99.64

Appendix B-2

Table B.3: Compressive and Flexural Strength at 7-day

	7-day					
	Compressive			Flexure		
Mix No.	lbf	psi	Average	lbf	psi	Average
1	70021	5572	**5538**	4224	792	**786**
	69800	5555		4353	816	
	68971	5489		4005	751	
2	45050	3585	**3526**	2946	552	**525**
	44577	3547		2793	524	

	43307	3446		2667	500	
3	63483	5052	**5302**	3964	743	**753**
	67264	5353		4112	771	
	69125	5501		3979	746	
4	44695	3557	**3554**	2787	523	**555**
	44725	3559		3162	593	
	44577	3547		2924	548	
5	84250	6704	**6632**	4271	801	**816**
	82448	6561		4238	795	
	83335	6632		4542	852	
6	47472	3778	**3744**	3397	637	**640**
	48299	3844		3359	630	
	45379	3611		3489	654	
7	86259	6864	**6871**	4356	817	**815**
	86141	6855		4421	829	
	86643	6895		4256	798	
8	39348	3131	**3285**	2537	476	**511**
	43927	3496		2512	471	
	40561	3228		3134	588	
9	66387	5283	**5286**	3654	685	**691**
	66998	5332		3721	698	
	65899	5244		3678	690	
10	63365	5042	**5263**	3497	656	**687**
	65758	5233		3588	673	
	69273	5513		3904	732	
11	56334	4483	**4568**	3579	671	**675**
	57959	4612		3969	744	
	57900	4608		3256	611	
12	n/a	n/a	**5983**	3931	737	**805**
	75388	5999		4457	836	
	74975	5966		4484	841	
13	47177	3754	**3738**	2789	523	**542**
	47324	3766		2852	535	
	46408	3693		3024	567	
14	65403	5205	**5153**	3313	621	**692**
	65374	5202		3989	748	

	63483	5052		3774	708	
15	66378	5282	**5254**	4092	767	**736**
	65728	5230		3673	689	
	65965	5249		4013	752	
16	64044	5096	**5065**	3959	742	**715**
	63260	5034		3661	686	
	n/a	n/a		3812	715	
17	79583	6333	**6116**	3912	734	**740**
	74679	5943		4302	807	
	76290	6071		3628	680	
GMF	41298	3286	**3376**	2637	494	**529**
	42598	3390		2592	486	
	43366	3451		3234	606	
PF	41771	3324	**3373**	2635	494	**521**
	43011	3423		2587	485	
	42391	3373		3118	585	
MAC470	42775	3404	**3408**	2922	548	**534**
	42391	3373		2637	494	
	43307	3446		2987	560	
TSSF	41239	3282	**3378**	2879	540	**538**
	42745	3402		2727	511	
	43366	3451		3003	563	
F300	40914	3256	**3201**	2662	499	**480**
	40264	3204		2579	484	
	39496	3143		2436	457	
NXR	42066	3348	**3257**	2689	504	**498**
	41357	3291		2579	484	
	39378	3134		2702	507	

Table B.4: Compressive and Flexural Strength at 28-day

	28-day					
	Compressive			**Flexure**		
Mix No.	**lbf**	**psi**	**Average**	**lbf**	**psi**	**Average**
1	95731	7618	**7590**	4498	843	**799**
	95650	7612		4027	755	
	94770	7542		n/a	n/a	

2	74029	5891	**5798**	3779	709	**693**
	74797	5952		3921	735	
	69746	5550		3390	636	
3	74738	5947	**6238**	4340	814	**777**
	77920	6201		4249	797	
	82508	6566		3843	721	
4	71371	5680	**5721**	3702	694	**692**
	72670	5783		3900	731	
	71636	5701		3470	651	
5	98903	7870	**7909**	4690	879	**884**
	100580	8004		4699	881	
	98667	7852		4758	892	
6	80026	6368	**6128**	4457	836	**830**
	74738	5947		4485	841	
	76274	6070		4338	813	
7	100460	7994	**8017**	4048	759	**867**
	103030	8199		4779	896	
	98755	7859		5040	945	
8	73868	5878	**5837**	4029	755	**776**
	72906	5802		4191	786	
	73261	5830		4188	785	
9	115170	9165	**9149**	4894	918	**938**
	114770	9133		4855	910	
	n/a	n/a		5262	987	
10	88829	7069	**7196**	4297	806	**845**
	92020	7323		4456	836	
	n/a	n/a		4769	894	
11	80617	6415	**6294**	4380	821	**820**
	77574	6173		4330	812	
	n/a	n/a		4406	826	
12	96037	7642	**7501**	4420	829	**836**
	94058	7485		4398	825	
	92670	7374		4550	853	
13	75979	6046	**6058**	4410	827	**801**
	77161	6140		4806	901	
	75240	5987		3606	676	

14	85403	6796	**6929**	4580	859	**809**
	87382	6954		4268	800	
	88445	7038		4100	769	
15	92492	7360	**7359**	4576	858	**850**
	92552	7365		4779	896	
	92374	7351		4239	795	
16	86673	6897	**6960**	4436	832	**817**
	85964	6841		4276	802	
	89745	7142		4357	817	
17	96835	7706	**7888**	4650	872	**871**
	100157	7970		4690	879	
	100370	7987		4590	861	
GMF	67855	5400	**5487**	3988	748	**758**
	69687	5546		4125	773	
	69303	5515		4022	754	
PF	69007	5491	**5368**	3896	731	**755**
	68387	5442		3967	744	
	64960	5169		4222	792	
MAC470	64783	5155	**5518**	3985	747	**749**
	71961	5726		3796	712	
	71282	5672		4210	789	
TSSF	68653	5463	**5459**	3879	727	**752**
	68239	5430		3998	750	
	68919	5484		4156	779	
F300	63631	5064	**5180**	3765	706	**674**
	63926	5087		3555	667	
	67708	5388		3465	650	
NXR	68328	5437	**5491**	3879	727	**764**
	66142	5263		4215	790	
	72552	5774		4130	774	

Table B.5: Modulus of Elasticity at 7-day

Modulus of Elasticity at 7 day						
Mix No.	**Cyl No.**					**EM**
1	1	0.0006	4317	0.0035	22095	5183272

	2	0.0006	4347	0.0033	23510	6000933
2	1	0.0006	3751	0.0029	18079	5267164
	2	0.0006	3692	0.0031	17990	4835644
3	1	0.0006	3486	0.00365	24312	5773313
	2	0.0006	5465	0.00365	24164	5183673
4	1	0.0006	4077	0.0028	16128	4631477
	2	0.0006	3870	0.0028	16395	4813646
5	1	0.0006	3840	0.00435	32288	6414156
	2	0.0006	3988	0.00445	32258	6208464
6	1	0.0006	4224	0.0029	19113	5473395
	2	0.0006	4224	0.0028	18994	5676451
7	1	0.0006	4815	0.00455	34503	6354815
	2	0.0006	4992	0.00455	34503	6316928
8	1	0.0006	4372	0.00245	15774	5211088
	2	0.0006	4431	0.00245	15686	5143904
9	1	0.0006	5040	0.00405	30267	6182521
	2	0.0006	5197	0.0041	29883	5963507
10	1	0.0006	4460	0.00345	25257	6169854
	2	0.0006	5051	0.00355	25198	5774408
11	1	0.0006	3745	0.00327	22357	5893874
	2	0.0006	3889	0.00355	22451	5320125
12	1	0.0006	3693	0.00405	29275	6269523
	2	0.0006	4047	0.0041	29304	6101446
13	1	0.0006	4165	0.0029	18226	5169011
	2	0.0006	3752	0.0029	18256	5331864
14	1	0.0006	4136	0.00375	25257	5669216
	2	0.0006	4579	0.0036	25287	5836278
15	1	0.0006	4194	0.0038	26380	5862031
	2	0.0006	4519	0.00385	26409	5694839
16	1	0.0006	3950	0.0038	25520	5699270
	2	0.0006	3810	0.00385	25500	5642808
17	1	0.0006	4963	0.0043	31727	6116013

	2	0.0006	4910	0.00415	31290	6282978
PF	1	0.0006	3427	0.0032	17311	4515027
	2	0.0006	3693	0.00315	17281	4505411
MAC470	1	0.0006	3722	0.0035	19083	4478582
	2	0.0006	3486	0.00335	18788	4704729
GMF	1	0.0006	3368	0.0032	18404	4889653
	2	0.0006	3250	0.0032	18285	4889328
F300	1	0.0006	3132	0.00345	18166	4460143
	2	0.0006	3014	0.00365	18047	4167397
TSSF	1	0.0006	2896	0.00325	17928	4796119
	2	0.0006	2778	0.0032	17809	4888027
NXR	1	0.0006	3660	0.00285	17690	5272229
	2	0.0006	3542	0.00275	17571	5517055

Table B.6: Modulus of Elasticity at 28-day

Modulus of Elasticity at 28 day						
Mix No.	**Cyl No.**					**EM**
1	1	0.0006	4570	0.00405	29760	6173453
	2	0.0006	N/A			
2	1	0.0006	4667	0.00395	29659	6307762
	2	0.0006	3899	0.0041	29865	6272723
3	1	0.0006	4992	0.00405	29806	6081305
	2	0.0006	N/A			
4	1	0.0006	4195	0.0042	28500	5708371
	2	0.0006	4490	0.0042	28536	5647541
5	1	0.0006	5258	0.00465	39437	7135483
	2	0.0006	5760	0.00475	39496	6873288
6	1	0.0006	3722	0.00435	32052	6387551
	2	0.0006	5465	0.0042	32052	6244331
7	1	0.0006	5908	0.0049	40146	6732231
	2	0.0006	5110	0.0049	39969	6854338
8	1	0.0006	5406	0.0034	28359	6931073

	2	0.0006	3574	0.00425	28300	5727698
9	1	0.0006	5230	0.00495	39193	6601397
	2	0.0006	N/A			
10	1	0.0006	4549	0.0046	35685	6581455
	2	0.0006	N/A			
11	1	0.0006	4815	0.0042	32540	6511606
	2	0.0006	N/A			
12	1	0.0006	4926	0.005	38491	6449901
	2	0.0006	4726	0.005	38580	6505436
13	1	0.0006	4786	0.0043	30101	5784892
	2	0.0006	5081	0.0042	30427	5952865
14	1	0.0006	5496	0.0045	34562	6301439
	2	0.0006	4845	0.00445	34444	6500330
15	1	0.0006	4372	0.0047	36689	6664480
	2	0.0006	4844	0.0045	36778	6923215
16	1	0.0006	4992	0.0045	34858	6474877
	2	0.0006	5317	0.00425	34769	6822460
17	1	0.0006	N/A			
	2	0.0006	4678	0.0047	38757	7027843
PF	1	0.0006	5200	0.0038	28020	6029548
	2	0.0006	3600	0.0042	28236	5786111
Mac470	1	0.0006	3960	0.0037	28130	6592256
	2	0.0006	4100	0.0041	27960	5763967
GMF	1	0.0006	4103	0.0043	28100	5483708
	2	0.0006	3988	0.0039	28020	6157367
F300	1	0.0006	3873	0.00415	27940	5732086
	2	0.0006	3958	0.00415	27860	5692787
TSSF	1	0.0006	4043	0.00385	27780	6175350
	2	0.0006	4128	0.0036	27700	6643459
NXR	1	0.0006	4113	0.0036	27620	6625140
	2	0.0006	3998	0.00395	27540	5941794

Table B.7: Length Change (Shrinkage)

Mix 1	Day =	**1**	7	11	14	21	35	63	**119**
	1-1	**0.0693**	0.0692	0.0678	0.0676	0.067	0.0653	0.0664	**0.066**
	1-2	**0.0469**	0.0474	0.0459	0.0456	0.0451	0.0436	0.0447	**0.0445**
	1-3	**0.0632**	30632	0.0616	0.0613	0.0607	0.0595	0.0603	**0.06**
Mix 2		**1**	7	11	14	21	35	63	**119**
	2-1	**0.0369**	0.0357	0.0362	0.0349	0.0351	0.0347	0.0342	**0.0341**
	2-2	**0.0513**	0.0504	0.0508	0.0496	0.0499	0.0494	0.049	**0.048**
	2-3	**0.1109**	0.1101	0.1105	0.1095	0.1095	0.1093	0.1088	**0.1084**
Mix 3		**1**	7	11	14	21	35	63	**119**
	3-1	**0.0598**	0.059	0.0575	0.0575	0.057	0.0559	0.558	**0.0552**
	3-2	**0.033**	0.0324	0.0308	0.0308	0.0304	0.0291	0.0289	**0.0282**
	3-3	**0.0552**	0.0545	0.053	0.0531	0.0527	0.0515	0.0513	**0.0506**
Mix 4		**1**	7	11	14	21	35	63	**119**
	4-1	**0.0522**	0.0527	0.0512	0.0512	0.0503	0.0494	0.94	**0.0487**
	4-2	**0.0658**	0.0654	0.0646	0.0645	0.0636	0.0627	0.0626	**0.0621**
	4-3	**0.0805**	0.0796	0.0786	0.0787	0.078	0.0772	0.0771	**0.0765**
Mix 5		**1**	7	11	14	21	35	63	**119**
	5-1	**0.0848**	0.0846	0.0834	0.0833	0.083	0.0821	0.0819	**0.0811**
	5-2	**0.0435**	0.0432	0.0422	0.0423	0.0417	0.0402	0.0401	**0.039**
	5-3	**0.0644**	0.0637	0.0628	0.0629	0.0625	0.0616	0.0616	**0.0608**
Mix 6		**1**	7	11	14	21	35	63	**119**
	6-1	**0.0482**	0.0479	0.0473	0.0467	0.0464	0.0455	0.0457	**0.0454**
	6-2	**0.0483**	0.0483	0.0477	0.0465	0.0465	0.0457	0.0459	**0.0455**
	6-3	**0.0714**	0.0715	0.0708	0.0701	0.0695	0.0686	0.0688	**0.0685**
Mix 7		**1**	7	11	14	21	35	63	**119**
	7-1	**0.0574**	0.0572	0.0564	0.0556	0.0549	0.0539	0.0543	**0.0537**
	7-2	**0.0999**	0.0997	0.0991	0.0982	0.0977	0.0968	0.0971	**0.0966**
	7-3	**0.0395**	0.0398	0.0389	0.0382	0.0376	0.0367	0.0369	**0.0362**
Mix 8		**1**	7	11	14	21	35	63	**119**
	8-1	**0.0562**	0.0566	0.0558	0.0551	0.0547	0.0538	0.0539	**0.0532**
	8-2	**0.11**	0.1102	0.1095	0.109	0.1086	0.1077	0.1079	**0.1074**

	8-3	**0.0653**	0.0654	0.0647	0.0641	0.0636	0.0628	0.063	**0.0624**
Mix 9		**1**	7	11	14	21	35	63	**119**
	9-1	**0.0647**	0.0648	0.0634	0.0629	0.0624	0.0611	0.0613	**0.0609**
	9-2	**0.0615**	0.0616	0.0605	0.0597	0.0593	0.0585	0.0588	**0.0584**
	9-3	**0.1004**	0.1009	0.0994	0.0987	0.0984	0.0975	0.0979	**0.0977**
Mix 10		**1**	7	11	14	21	35	63	**119**
	10-1	**0.0497**	0.0495	0.0492	0.048	0.048	0.0475	0.0469	**0.0467**
	10-2	**0.0831**	0.0823	0.0822	0.0812	0.0811	0.0805	0.0798	**0.0794**
	10-3	**0.0726**	0.0726	0.0722	0.0712	0.0712	0.0707	0.0702	**0.0697**
Mix 11		**1**	7	11	14	21	35	63	**119**
	11-1	**0.0455**	0.0451	0.0451	0.0439	0.0439	0.0434	0.0426	**0.0424**
	11-2	**0.0476**	0.0475	0.0473	0.0462	0.0462	0.0457	0.0447	**0.0445**
	11-3	**0.0752**	0.0751	0.0748	0.0736	0.0738	0.0734	0.0723	**0.0719**
Mix 12		**1**	7	11	14	21	35	63	**119**
	12-1	**0.0537**	0.0536	0.0523	0.0521	0.0505	0.0512	0.0506	**0.0499**
	12-2	**0.0506**	0.0504	0.049	0.0485	0.047	0.0478	0.0472	**0.0464**
	12-3	**0.0698**	0.0697	0.0681	0.0678	0.0662	0.0669	0.0663	**0.0656**
Mix 13		**1**	7	11	14	21	35	63	**119**
	13-1	**0.0737**	0.0737	0.0727	0.0724	0.0712	0.0717	0.0712	**0.0708**
	13-2	**0.0674**	0.0675	0.0667	0.0662	0.0649	0.0654	0.0651	**0.0645**
	13-3	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Mix 14		**1**	7	11	14	21	35	63	**119**
	14-1	**0.0477**	0.0473	0.0464	0.046	0.0443	0.0447	0.0441	**0.0435**
	14-2	**0.1276**	0.1279	0.1269	0.1266	0.1251	0.1258	0.1252	**0.1246**
	14-3	**0.1014**	0.1015	0.1004	0.1002	0.0986	0.0994	0.0989	**0.0982**
Mix 15		**1**	7	11	14	21	35	63	**119**
	15-1	**0.0487**	0.0493	0.0486	0.0486	0.0474	0.0469	0.0463	**0.0457**
	15-2	**0.041**	0.0414	0.0404	0.0402	0.0394	0.039	0.0383	**0.0378**
	15-3	**0.0644**	0.0648	0.0641	0.0641	0.0631	0.0626	0.0613	**0.0613**
Mix 16		**1**	7	11	14	21	35	63	**119**
	16-1	**0.0458**	0.0462	0.0449	0.0448	0.044	0.0436	0.0431	**0.0426**
	16-2	**0.0297**	0.0295	0.0284	0.0283	0.0274	0.027	0.0265	**0.0258**

	16-3	**0.0584**	0.0583	0.0571	0.0572	0.0561	0.0557	0.0552	**0.0546**
Mix 17		**1**	7	11	14	21	35	63	**119**
	17-1	**0.021**	0.0214	0.0195	0.0193	0.0188	0.0182	0.0173	**0.0169**
	17-2	**0.0591**	0.0595	0.0586	0.0585	0.0578	0.0575	0.0569	**0.0562**
	17-3	**0.0878**	0.0883	0.0871	0.0871	0.0863	0.086	0.0856	**0.0851**
GMF		**1**	7	11	14	21	35	63	**119**
	1 of 3	**0.0598**	0.0598	0.0589	0.058	0.0577	0.0572	0.0567	**0.0566**
	2 of 3	**0.0387**	0.0388	0.038	0.0372	0.0366	0.0361	0.0358	**0.0356**
	3 of 3	**0.0454**	0.0451	0.0445	0.0437	0.0432	0.0427	0.0423	**0.042**
MAC470		**1**	7	11	14	21	35	63	**119**
	1 of 3	**0.0518**	0.052	0.0513	0.0488	0.0502	0.0497	0.0494	**0.0493**
	2 of 3	**0.0576**	0.0576	0.057	0.0561	0.0559	0.0555	0.055	**0.0548**
	3 of 3	**0.0699**	0.0708	0.0701	0.0691	0.0689	0.0685	0.0681	**0.0679**
PF		**1**	7	11	14	21	35	63	**119**
	1 of 3	**0.0855**	0.0853	0.0849	0.0838	0.0835	0.0834	0.083	**0.0827**
	2 of 3	**0.0465**	0.0464	0.0457	0.0442	0.0439	0.0437	0.0431	**0.0428**
	3 of 3	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
F300		**1**	7	11	14	21	35	63	**119**
	1 of 3	**0.0498**	0.0598	0.0589	0.058	0.0577	0.0572	0.0567	**0.0467**
	2 of 3	**0.0417**	0.0388	0.038	0.0372	0.0366	0.0361	0.0358	**0.0381**
	3 of 3	**0.0544**	0.0451	0.0445	0.0437	0.0432	0.0427	0.0423	**0.051**
TSSF		**1**	7	11	14	21	35	63	**119**
	1 of 3	**0.0541**	0.0598	0.0589	0.058	0.0577	0.0572	0.0567	**0.0511**
	2 of 3	**0.0399**	0.0388	0.038	0.0372	0.0366	0.0361	0.0358	**0.0366**
	3 of 3	**0.0467**	0.0451	0.0445	0.0437	0.0432	0.0427	0.0423	**0.0435**
NXR		**1**	7	11	14	21	35	63	**119**
	1 of 3	**0.0434**	0.0598	0.0589	0.058	0.0577	0.0572	0.0567	**0.0406**
	2 of 3	**0.0677**	0.0388	0.038	0.0372	0.0366	0.0361	0.0358	**0.0646**
	3 of 3	**0.0735**	0.0451	0.0445	0.0437	0.0432	0.0427	0.0423	**0.0705**
Control		**1**	7	11	14	21	35	63	**119**
	1 of 3	**0.0421**	0.0598	0.0589	0.058	0.0577	0.0572	0.0567	**0.0388**
	2 of 3	**0.0554**	0.0388	0.038	0.0372	0.0366	0.0361	0.0358	**0.0526**

	3 of 3	**0.0457**	0.0451	0.0445	0.0437	0.0432	0.0427	0.0423	**0.0422**

Table B.8: Bond Strength

Factorial Matrix Bond Test Results			
Mixture No.	Strength (psi)		
	3-day	7-day	28-day
1	152	239	254
2	87	174	181
3	145	239	261
4	80	152	189
5	239	348	355
6	116	196	196
7	232	334	348
8	123	196	210
9	123	174	196
10	126	181	190
11	125	184	194
12	181	297	297
13	102	175	194
14	131	196	210
15	131	181	210
16	116	203	218
17	203	297	312

Bond Test Results with Different Reinforcements	
Reinforcement	**Strength (psi)**
NXR	384
MFM470	374
F300	360
PF	363
TSSF	389
GMF	357
NXR +GMF	413
MFM470 +GMF	406
F300 +GMF	392
PF +GMF	384
TSSF +GMF	419
Wiremesh	352
#4 Rebar	399
River Gravel as CA	44
Control	355

Table B.9: Average Residual Strength (ARS)

NXR			
	Dosage (lb/yd³)		
Beam No.	**110**	**60**	**10**
1	322	216	66
2	356	188	80
3	402	234	82
4	320	252	76
5	330	236	70
Average:	**346**	**225**	**75**

PF			
	Dosage (lb/yd³)		
Beam No.	**7**	**4**	**1**
1	172	104	26
2	198	140	40
3	220	100	44
4	128	112	48
5	178	132	38
Average:	**179**	**118**	**39**

TSSF			
	Dosage (lb/yd³)		
Beam No.	**20**	**10**	**1**
1	338	280	40
2	322	188	32
3	378	258	46
4	390	254	32
5	412	190	42
Average:	**368**	**234**	**38**

MAC470			
	Dosage (lb/yd³)		
Beam No.	**13**	**8**	**3**
1	232	74	64
2	248	262	74
3	226	82	48
4	290	140	62
5	234	184	64
Average:	**246**	**148**	**62**

F300			
	Dosage (lb/yd³)		
Beam No.	**20**	**3**	**0.5**
1	168	84	16
2	144	78	18
3	112	92	14
4	158	112	20
5	166	120	22
Average:	**150**	**97**	**18**

GMF			
	Dosage (lb/yd³)		
Beam No.	**20**	**3**	**0.5**
1	50	30	12
2	62	32	10
3	44	28	12
4	48	24	10
5	60	40	8
Average:	**53**	**31**	**10**

Appendix B-3

Table B.10: Concrete Mixture Designs

Material Properties

Coarse (CDL[1])		Fine (CRS[2])		Cement (TXI)	Fly Ash	WRA
BSG[3] (SSD)	AC[4] (OD)	BSG (SSD)	AC (OD)	BSG (SSD)	Type	Type
2.79	0.56	2.6	0.7	3.15	Legs	WRDA82

[1]CDL = Crushed Dolomitic Limestone
[2]CRS = Colorado Riversand
[3]BSG = Bulk Specific Gravity
[4]AC = Absorption Capacity

Factorial Matrix Mixture Proportions

Mix No.	Cement lb/yd³	Fly Ash lb/yd³	Water lb/yd³	Coarse lb/yd³	Fine lb/yd³	WRA mL
1	564	0	225.6	2221.9	1118.9	92
2	282	282	225.6	2221.9	1118.9	0
3	564	0	225.6	1880.1	1438.6	106
4	282	282	225.6	1880.1	1438.6	10
5	658	0	263.2	2100.0	1057.6	75
6	329	329	263.2	2100.0	1057.6	0
7	658	0	263.2	1777.0	1359.7	69
8	329	329	263.2	1777.0	1359.7	0
9	458.25	152.75	244.4	1994.7	1243.7	73
10	458.25	152.75	244.4	1994.7	1243.7	65
11	458.25	152.75	244.4	1994.7	1243.7	60
12	611	0	244.4	1994.7	1243.7	80
13	305.5	305.5	244.4	1994.7	1243.7	5
14	458.25	152.75	244.4	2161.0	1088.3	78
15	458.25	152.75	244.4	1828.5	1399.2	50
16	423	141	225.6	2051.0	1278.8	73
17	493.5	164.5	263.2	1938.5	1208.7	30

Customized Restrained Shrinkage Slabs Mixture Proportions					
Cement lb/yd³	**Fly Ash lb/yd³**	**Water lb/yd³**	**Coarse lb/yd³**	**Fine lb/yd³**	**WRA mL**
705	0	317.3	1671.5	1279.0	Adjusted

15 slabs were made with the following reinforcements:

Fibers	**Dosage* lb/yd³**	**Fibers + GMF**	**Dosage* lb/yd³**	**Others**
MAC470	10	MAC470	10+1.5	#4 rebar
PF	4	PF	4+1.5	Wiremesh
F300	3.5	F300	3.5+1.5	River gravel as CA
NXR	50	NXR	50+1.5	Control
TSSF	4	TSSF	4+1.5	
GMF	1.5			

*TxDOT recommended dosage rate was used

Average Residual Strength Mixture Proportions					
Cement lb/yd³	**Fly Ash lb/yd³**	**Water lb/yd³**	**Coarse lb/yd³**	**Fine lb/yd³**	**WRA mL**
493.5	164.5	263.2	1938.5	1208.7	Max

Appendix B-4

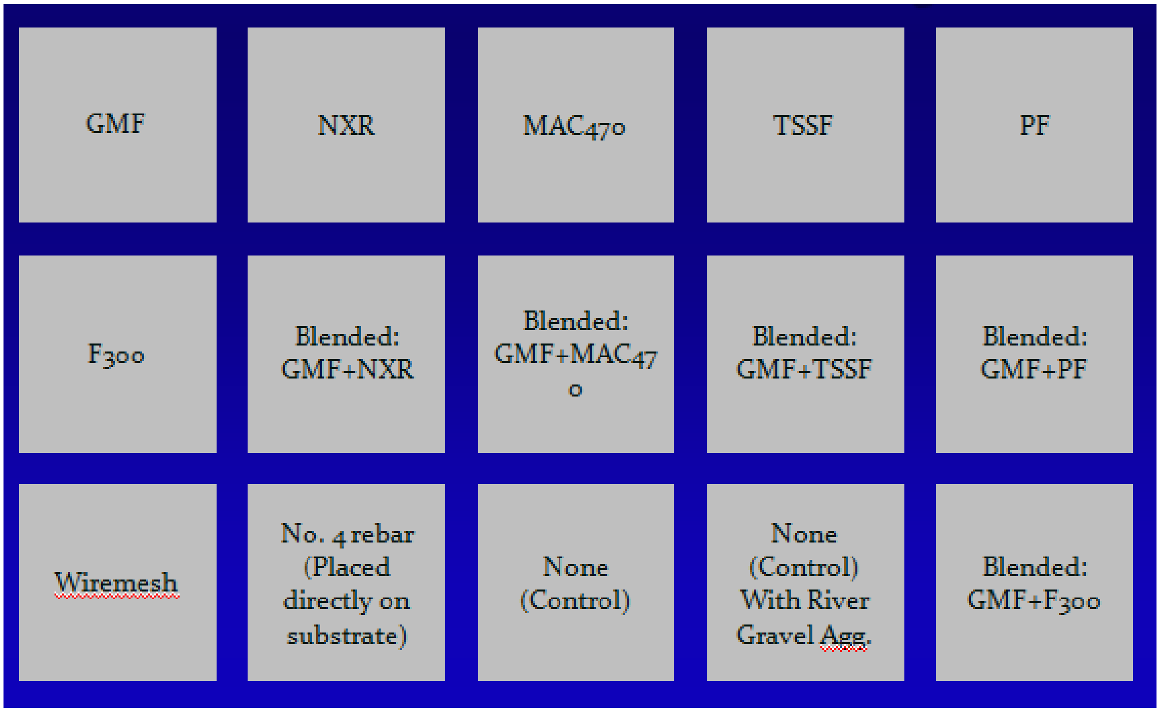


Figure B.1: Customized Restrained Shrinkage Specimens Layout

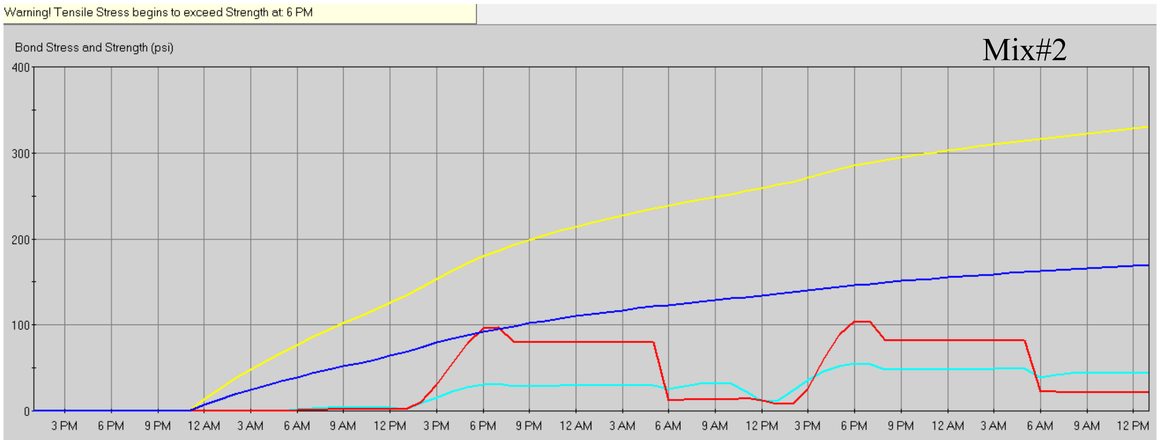
Appendix B-5

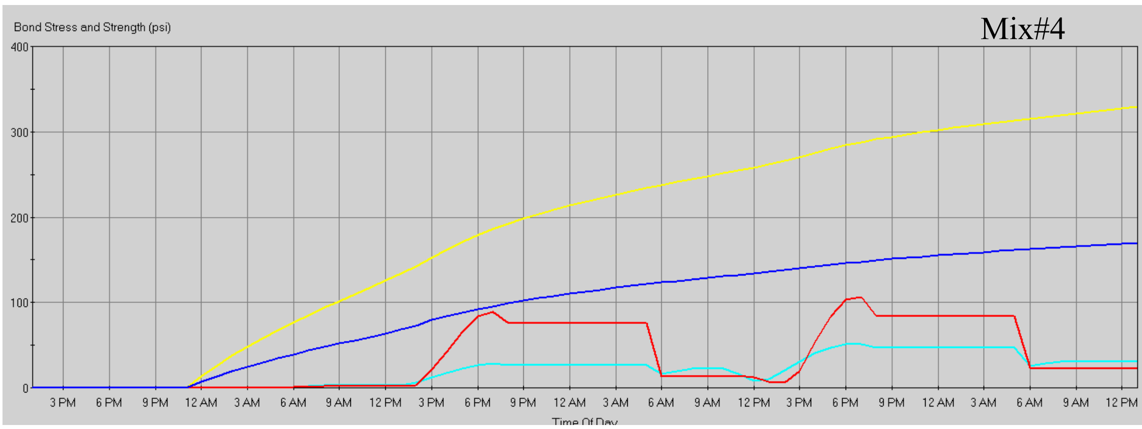
```
Data Analysis(1024)/Data Collection(1024)
[1]
 A: 1.01( 0%) [0.69] B: 0.84( 0%) [0.48] C: 1.14( 0%) [0.50] D: 0.63( 0%) [0.47]
 E: 0.78( 0%) [0.42] F: 1.04( 0%) [0.54] G: 0.54( 0%) [0.33] H: 0.67( 0%) [0.42]
 Ave: 0.83( 0%) [0.48]
[2]
 A: 0.68( 0%) [0.45] B: 0.97( 0%) [0.52] C: 0.61( 0%) [0.39] D: 0.86( 1%) [0.60]
 E: 0.64( 0%) [0.51] F: 0.81( 0%) [0.37] G: 0.82( 0%) [0.41] H: 0.93( 0%) [0.43]
 Ave: 0.79( 0%) [0.46]
[3]
 A: 0.60( 0%) [0.33] B: 1.08( 0%) [0.46] C: 0.60( 0%) [0.34] D: 1.12( 0%) [0.71]
 E: 0.54( 0%) [0.23] F: 0.58( 0%) [0.31] G: 0.43( 0%) [0.25] H: 1.48( 0%) [0.61]
 Ave: 0.80( 0%) [0.41]
[4]
 A: 0.83( 1%) [0.35] B: 0.93( 1%) [0.45] C: 0.52( 0%) [0.32] D: 0.62( 0%) [0.31]
 E: 0.73( 0%) [0.42] F: 1.13( 0%) [1.27] G: 0.57( 0%) [0.30] H: 1.20( 1%) [0.69]
 Ave: 0.82( 0%) [0.51]
[5]
 A: 0.63( 0%) [0.43] B: 0.86( 0%) [0.63] C: 0.56( 0%) [0.34] D: 0.56( 0%) [0.30]
 E: 0.72( 0%) [0.44] F: 0.64( 0%) [0.44] G: 0.87( 0%) [0.34] H: 1.06( 0%) [0.49]
 Ave: 0.74( 0%) [0.43]
[Ave]
 A: 0.75( 0%) [0.45] B: 0.94( 0%) [0.51] C: 0.69( 0%) [0.38] D: 0.76( 0%) [0.48]
 E: 0.68( 0%) [0.40] F: 0.84( 0%) [0.59] G: 0.65( 0%) [0.33] H: 1.07( 0%) [0.53]
 Ave: 0.80( 0%) [0.46]
```

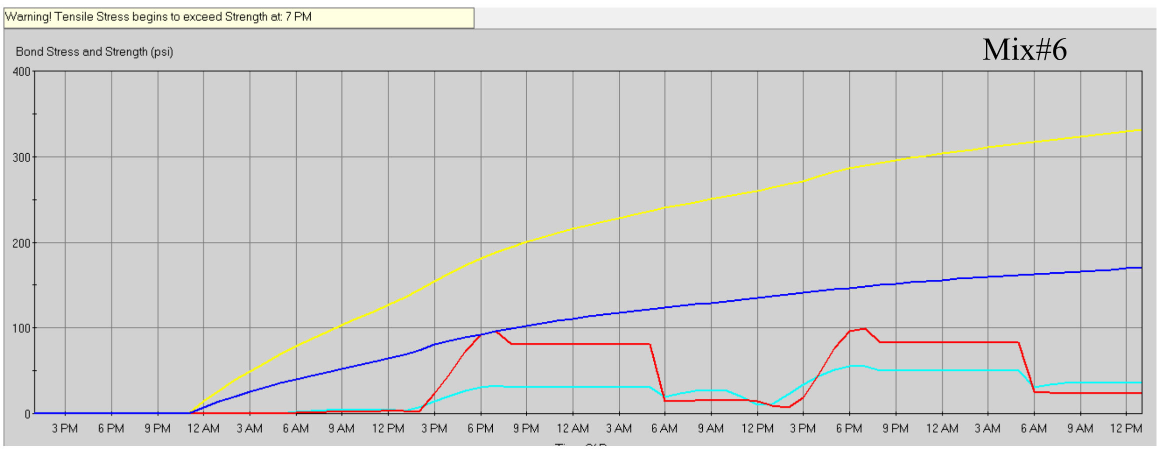
Figure B.2: Circular Track Meter (CT Meter) Results

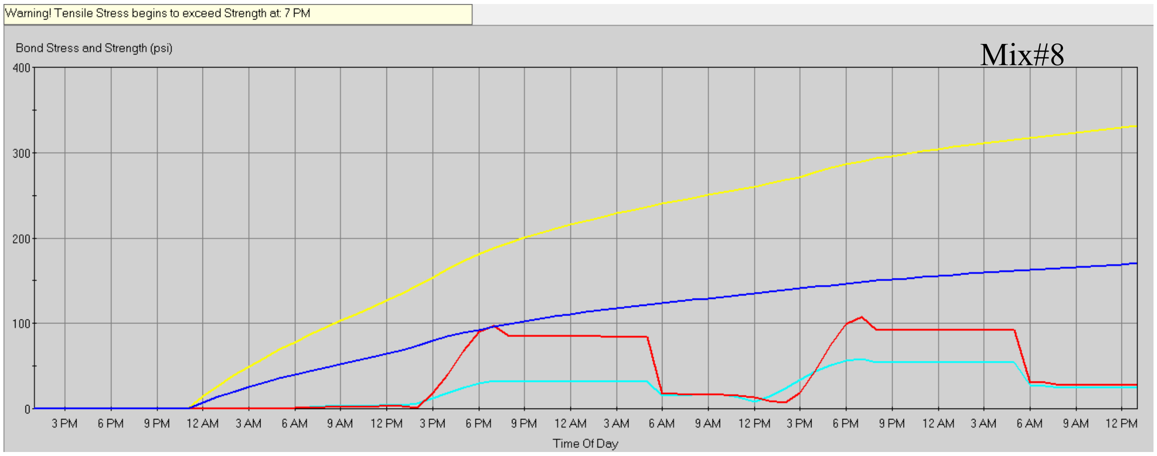
Appendix C

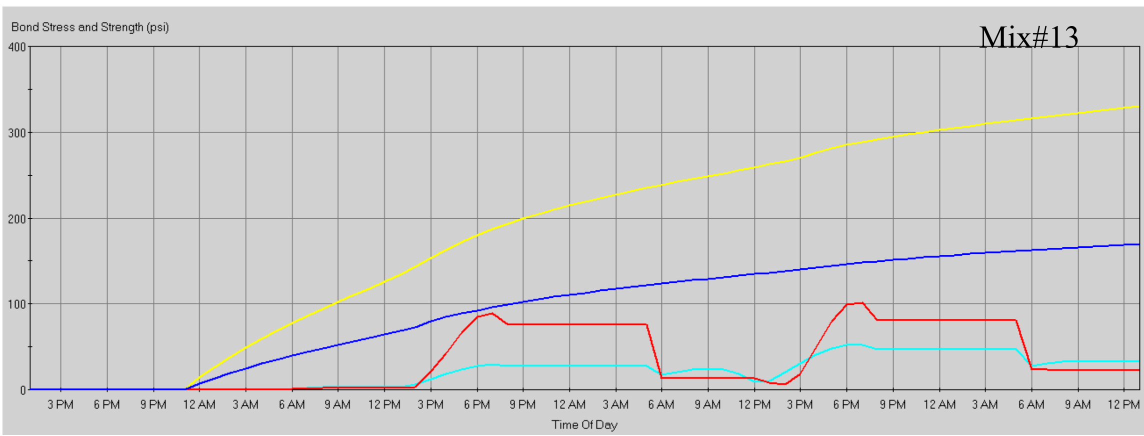
Appendix C-1: Failed HIPERBOND Evaluations



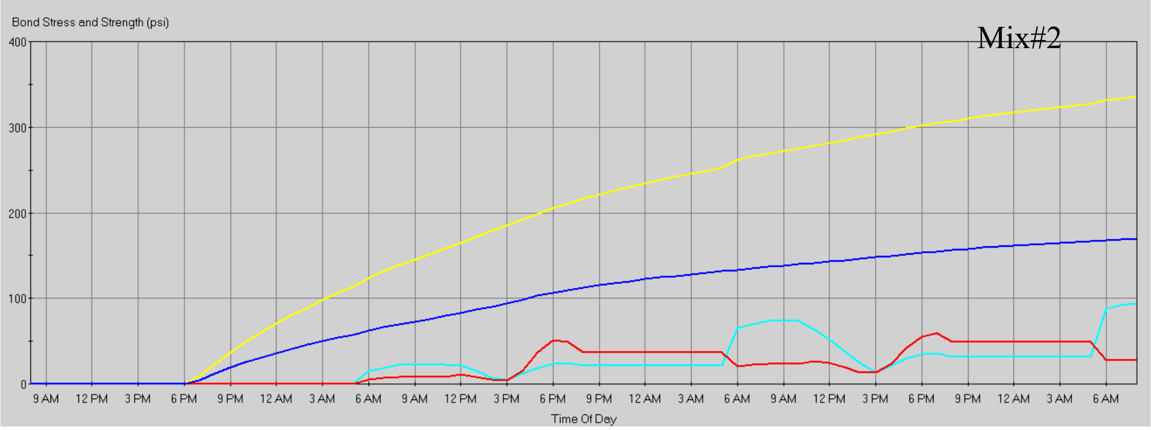


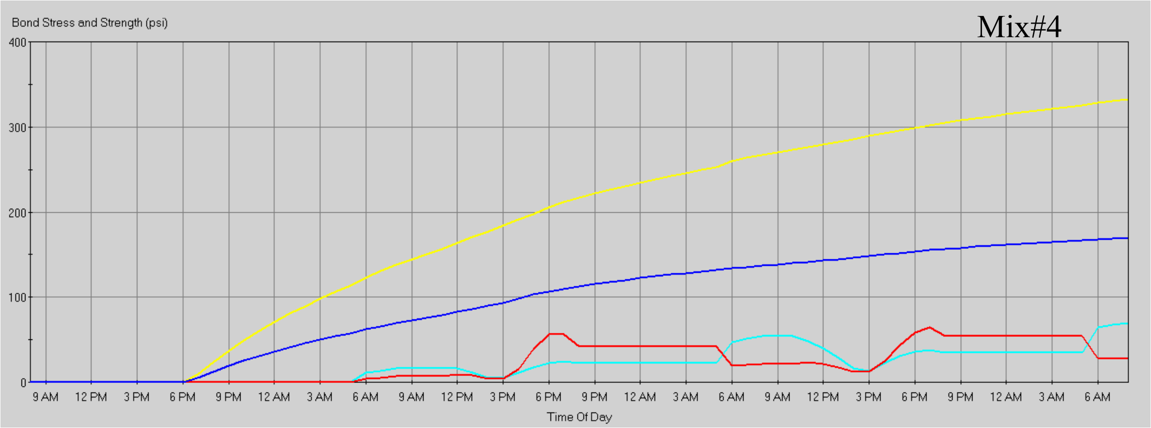


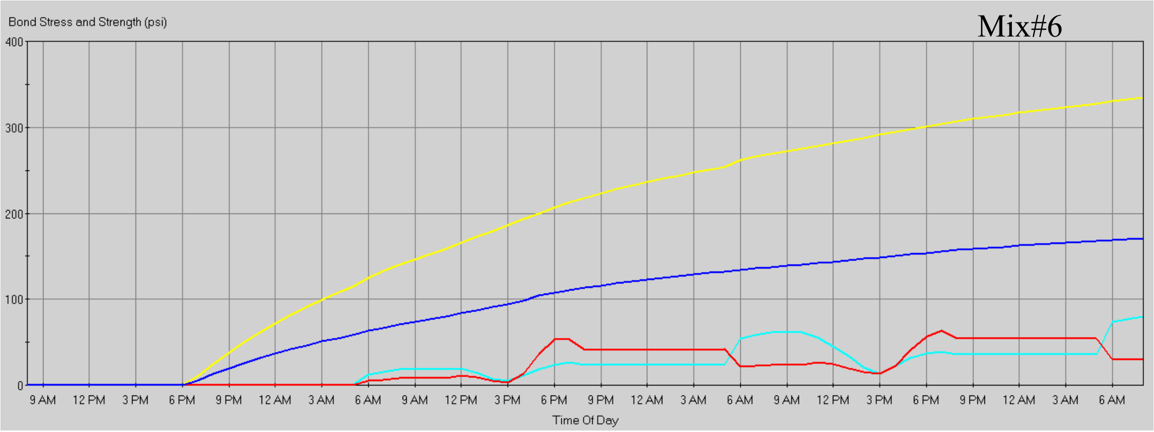

Warning! Tensile Stress begins to exceed Strength at: 7 PM
Bond Stress and Strength (psi)
Mix#8
400
300
200
100
0
3 PM
6 PM
9 PM
12 AM
3 AM
6 AM
9 AM
12 PM
Time Of Day

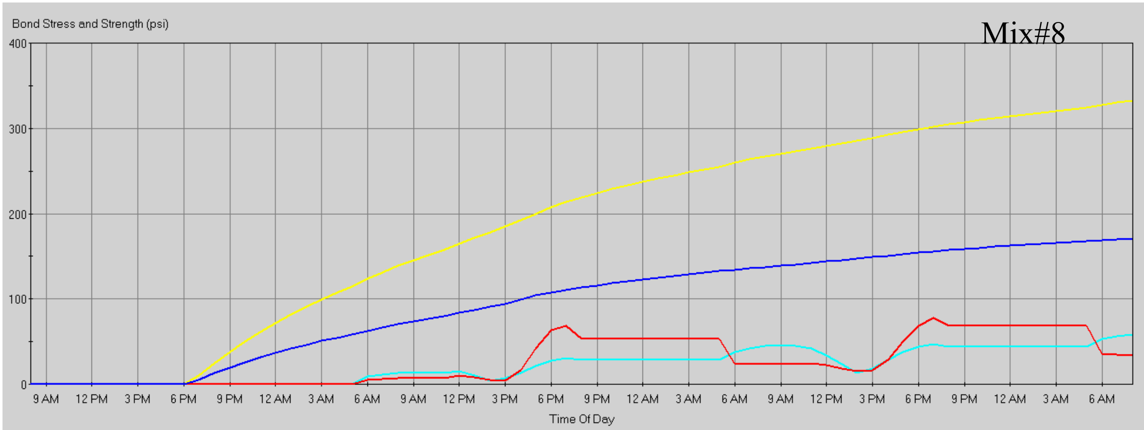

Bond Stress and Strength (psi)
Mix#13
400
300
200
100
0
3 PM
6 PM
9 PM
12 AM
3 AM
6 AM
9 AM
12 PM
Time Of Day

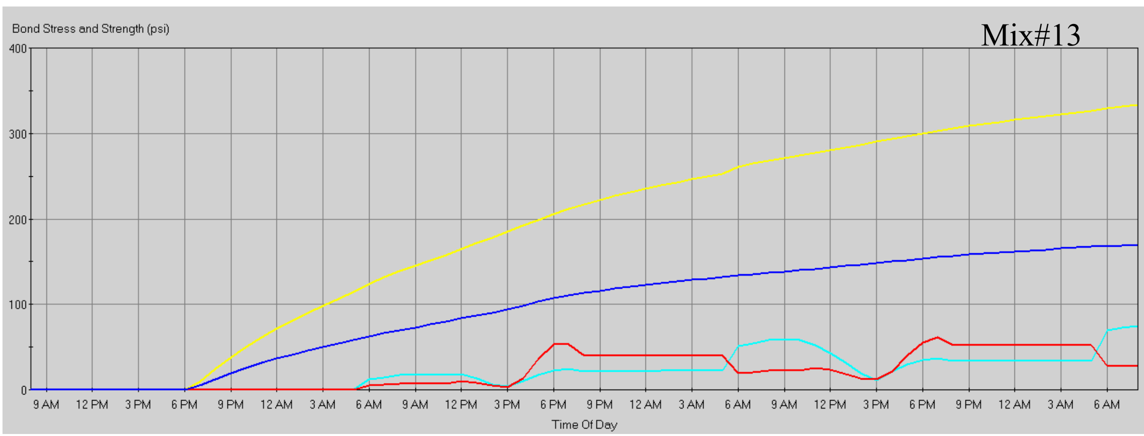
Appendix C-2: Failed HIPERBOND Revaluations at 8 AM



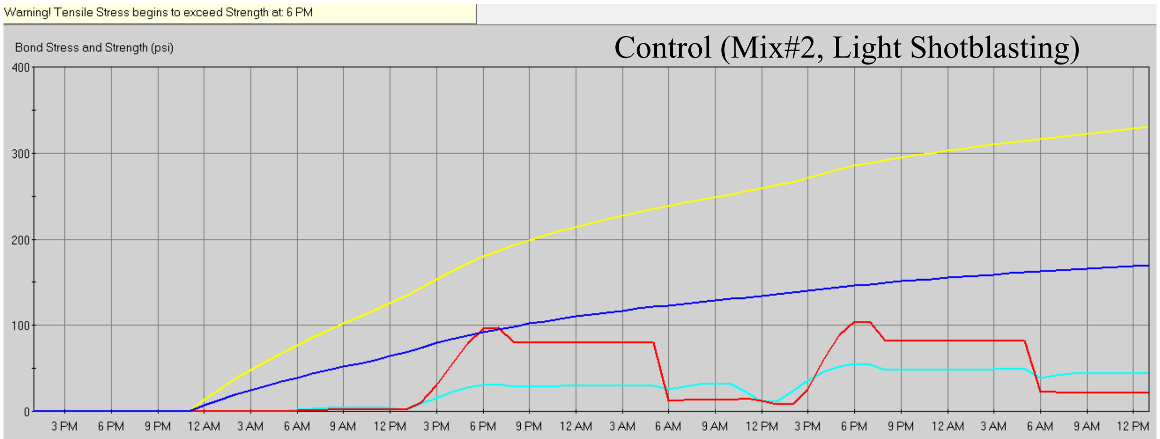


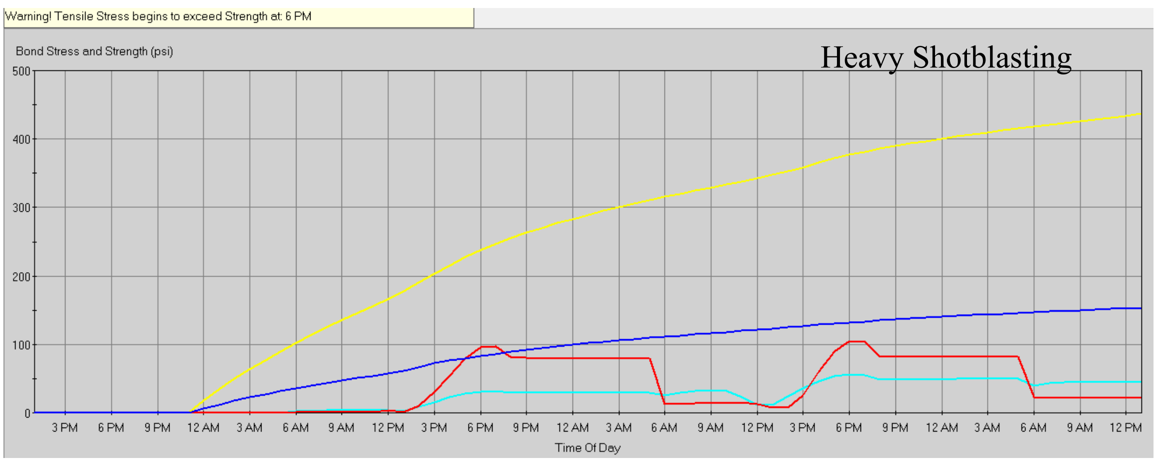


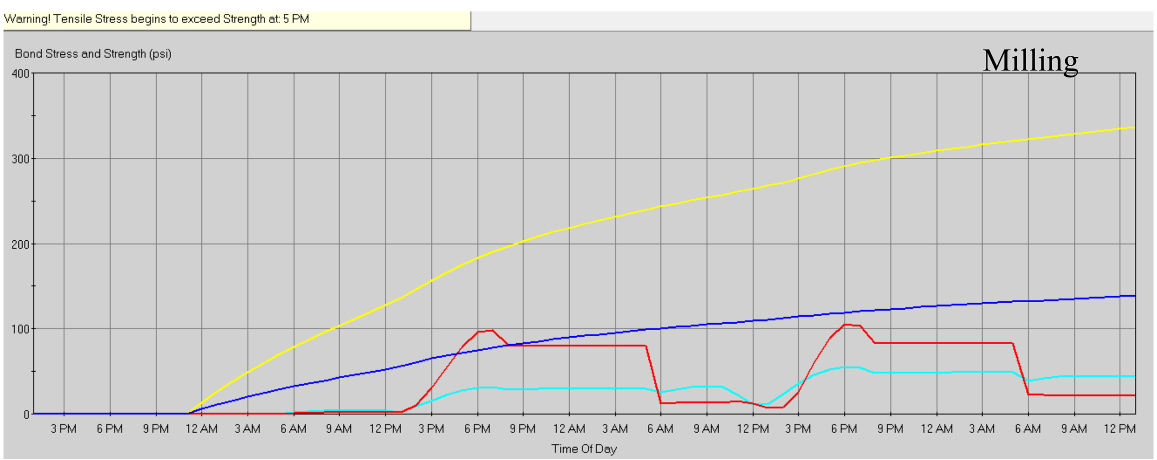

Bond Stress and Strength (psi)
Mix#8
400
300
200
100
0
9 AM
12 PM
3 PM
6 PM
9 PM
12 AM
3 AM
6 AM
9 AM
12 PM
3 PM
6 PM
9 PM
12 AM
3 AM
6 AM
9 AM
12 PM
3 PM
6 PM
9 PM
12 AM
3 AM
6 AM
Time Of Day

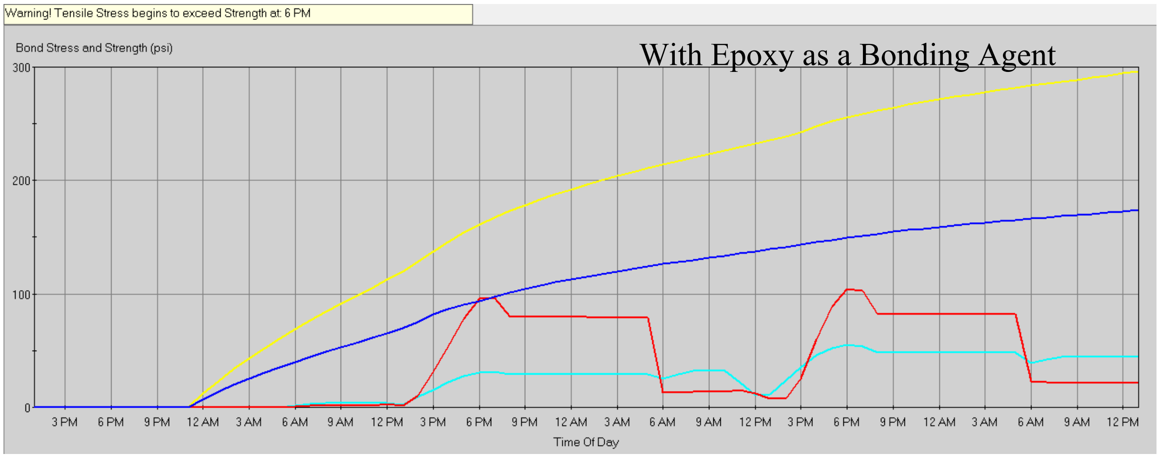

Bond Stress and Strength (psi)
Mix#13
400
300
200
100
0
9 AM
12 PM
3 PM
6 PM
9 PM
12 AM
3 AM
6 AM
9 AM
12 PM
3 PM
6 PM
9 PM
12 AM
3 AM
6 AM
9 AM
12 PM
3 PM
6 PM
9 PM
12 AM
3 AM
6 AM
Time Of Day

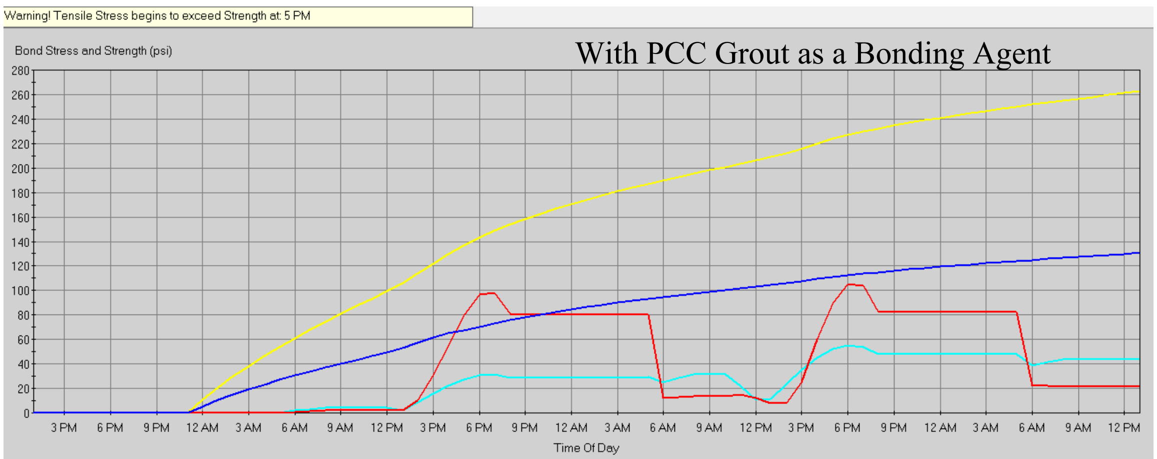
Appendix C-3: HIPERBOND Revaluations with Different Surface Preparation Methods and with Bond Agent








Warning! Tensile Stress begins to exceed Strength at: 6 PM
Bond Stress and Strength (psi)
With Epoxy as a Bonding Agent
300
200
100
0
3 PM
6 PM
9 PM
12 AM
3 AM
6 AM
9 AM
12 PM
3 PM
6 PM
9 PM
12 AM
3 AM
6 AM
9 AM
12 PM
3 PM
6 PM
9 PM
12 AM
3 AM
6 AM
9 AM
12 PM
Time Of Day


Warning! Tensile Stress begins to exceed Strength at: 5 PM
Bond Stress and Strength (psi)
With PCC Grout as a Bonding Agent
280
260
240
220
200
180
160
140
120
100
80
60
40
20
0
3 PM
6 PM
9 PM
12 AM
3 AM
6 AM
9 AM
12 PM
3 PM
6 PM
9 PM
12 AM
3 AM
6 AM
9 AM
12 PM
3 PM
6 PM
9 PM
12 AM
3 AM
6 AM
9 AM
12 PM
Time Of Day