Technical Report Documentation Page

1. Report No. FHWA/TX-05/0-4398-2	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle TECHNIQUES AND PROCEDURES FOR BONDED CONCRETE OVERLAYS		5. Report Date April 2003 Revised February 2004 6. Performing Organization Code
7. Author(s) Manuel Trevino, B. Frank McCullough, and David W. Fowler		8. Performing Organization Report No. 0-4398-2
9. Performing Organization Name and Address Center for Transportation Research The University of Texas at Austin 3208 Red River, Suite 200 Austin, TX 78705-2650		10. Work Unit No. (TRAIS) 11. Contract or Grant No. 0-4398
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 Research Report, 09/01/99-08/31/02 14. Sponsoring Agency Code
15. Supplementary Notes Project conducted in cooperation with the U.S. Department of Transportation, Federal Highway Administration, and the Texas Department of Transportation.		
16. Abstract There has been a significant increase in the use of BCOs as a way to extend the life of a pavement structure. This study establishes the appropriateness of a BCO as a pavement rehabilitation strategy proposing a methodology for project selection.		
17. Key Words bonded concrete overlays (BCOs), PCC pavement, pavement structure, rehabilitation		18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161.

19. Security Classif. (of report)	20. Security Classif. (of this page)	21. No. of pages	22. Price
Unclassified	Unclassified	226	

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

Center for Transportation Research
The University of Texas at Austin

Techniques and Procedures for Bonded Concrete Overlays

Manuel Trevino
B. Frank McCullough
David W. Fowler

CTR Research Report:	0-4398-2
Report Date:	April 2003 Revised: February 2004
Research Project:	0-4398
Research Project Title:	*Develop Guidelines for Designing and Constructing Thin Asphalt Pavement (ACP)*

This research was conducted for the Texas Department of Transportation in cooperation with the U.S. Department of Transportation, Federal Highway Administration by the Center for Transportation Research, Bureau of Engineering Research, The University of Texas at Austin.

Center for Transportation Research
The University of Texas at Austin
3208 Red River
Austin, TX 78705

www.utexas.edu/research/ctr

Disclaimers

Engineering Disclaimer

Acknowledgments

The researchers acknowledge the invaluable assistance provided by Richard Schiller, TxDOT project director for Project 9-572. Also appreciated is the guidance provided by Andrew Wimsatt, Research Engineer, TxDOT.

Table of Contents

List of Figures

List of Tables

1. Introduction

In this chapter, the concept of a bonded concrete overlay is introduced. The advantages and disadvantages of this pavement rehabilitation method are discussed. The objectives and the scope of this report are presented as well as the outlined contents of each chapter.

1.1 Background

Roads play an important role in the nation's economy, serving as a means to transport products, services, and people. The magnitude of a country's road network is often gauged as the strength of the national economy. Thus, the preservation of the integrity of the highway system becomes an issue of capital importance. Of all the components of a highway system, the item with the largest total investment is the pavement structure. All pavements undergo damage as a consequence of normal use, excessive loads, cumulative traffic, and environmental effects. When a pavement structure approaches the end of its intended service life or experiences an unacceptable level of deterioration, rarely is the solution to this problem tearing it apart and building a new facility. Rehabilitation of the pavement is generally the most sensible choice. The rehabilitation of a concrete pavement becomes necessary when the existing pavement has reached a condition of failure, which may be either a structural or a functional failure. The road has to be repaired with minimal disruption to traffic flow, preserving the functionality of the network and, therefore, causing minimal harm to the economy. However, the solution as to how to approach the rehabilitation is not singular. There is a wide spectrum of options from which to choose, one of which is the subject of this report.

A bonded concrete overlay (BCO) is a rehabilitation procedure for concrete pavements. With the placement of a BCO, the existing pavement will not only be restored from the faulty conditions, but will also gain additional service life. Therefore, the BCO serves not only as a rehabilitation system, but also as a means of protecting and enhancing the infrastructure for the future; thus, it is an investment. The usefulness of this rehabilitation system will be discussed in detail in this chapter, as well as it advantageous characteristics and limitations.

1.1.1 Definition

To better understand the concept of a BCO and how it works, it is necessary to provide a formal definition. A BCO is a layer of concrete bonded to the top of an existing pavement to rehabilitate it, to restore and increase its structural and functional capabilities, and forming a monolithic structure with the existing pavement. This is illustrated in Figures 1.1 and 1.2, in which a cross section of an existing continuously reinforced concrete pavement (CRCP) before a BCO placement, and a cross section of a BCO on top of the CRCP are shown, respectively. The BCO layer is generally between 2 and 6 in. thick, which is relatively thin, as compared to the existing pavement. For the BCO to perform as intended, the BCO and the existing pavement should behave as a single layer, meaning that the bond between existing and new concrete is a critical component to the success of the rehabilitation. If the bond between overlay and substrate is not achieved and sustained over time, the traffic and environmental loads will impose excessive stresses onto the structure, stresses that the relatively thin overlay is not designed to withstand by itself. The contribution of the existing structure to the overall strength of the rehabilitated structure is significant. This explains one of the advantages of a BCO, which is its optimum use of the remaining life of the existing pavement.

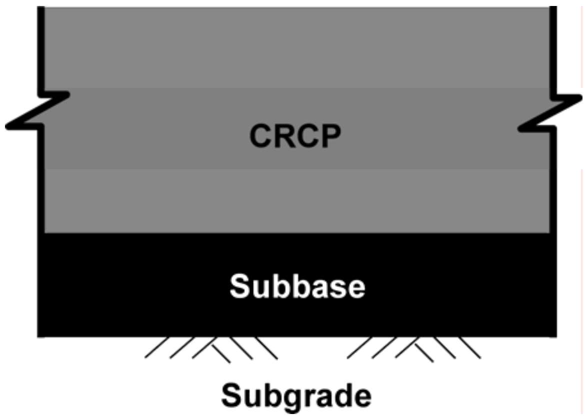


Figure 1.1 Typical existing pavement cross section before rehabilitation

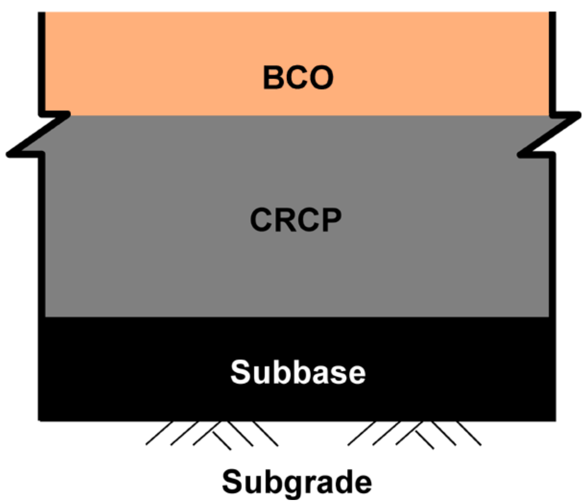


Figure 1.2 Cross section of a rehabilitated CRCP with a BCO

The previous definition of a BCO prompts one to contrast the BCO with unbonded concrete overlays. Unbonded concrete overlays are used to rehabilitate extensively deteriorated asphalt concrete (AC) pavements or portland cement concrete (PCC) pavements, with only a minimum amount of repair of the distresses performed prior to placing the overlay. In this case, the designer purposely places a bond-breaking layer in between the existing substrate and the overlay, generally AC, to ensure that the distresses of the existing pavement will not be reflected in the new overlay. As a result of this, an unbonded concrete overlay has to be much thicker than a BCO, normally between 6 and 12 in., since it does not rely on the stiffness of the existing pavement as a BCO does.

Unbonded concrete overlays are a feasible rehabilitation strategy for many cases. However, they are cost-effective only if the existing pavement is severely deteriorated because of the reduced need for pre-overlay repairs. A more detailed analysis of the applicability of an unbonded concrete overlay versus a BCO will be presented in Chapter 4, which is dedicated to the project selection.

1.2 BCO Advantages

This type of rehabilitation method has proven its effectiveness in many instances, providing an economical and technical solution to the problem of concrete pavement deterioration in heavily urbanized and traveled areas. Because BCOs offer remarkable benefits over other pavement rehabilitation strategies, and because their established viability in the field, usage of BCOs has increased in recent years.

A BCO optimizes the use of existing pavement structure, adding years of service to the remaining life of the facility. This is one of the most notable advantages. The optimal use of the existing structure means that, from an economic standpoint, the overlay adds value to that of the existing infrastructure with no waste of resources. Other rehabilitation strategies, for instance, unbonded concrete overlays or full-depth replacements do not make such an optimal use of the existing pavement. Because more service life is added to the existing structure, a BCO is a means to protect that investment for the benefit of the owners and users of the facility.

A BCO is a relatively low-cost rehabilitation strategy. In several BCO projects developed at the Center for Transportation Research (e.g., El Paso BCO and Fort Worth BCO, Refs 1 and 2, respectively), the costs of a new full-depth replacement pavement have been compared with the cost of a BCO. For the El Paso project, the cost of the BCO was between 46 and 60 percent of the cost of a new pavement. For Fort Worth, the BCO was about 60 percent of the cost of a new pavement.

It takes less time to construct a BCO than it takes to perform a full-depth reconstruction of a pavement, which leads to another advantageous characteristic of a BCO: It expedites construction. This represents a reduction of lane closures, traffic disturbances, pollution, and inconvenience to the public, all of which also imply reducing users' costs. Moreover, this research study demonstrates that an expedited BCO may allow traffic to travel on the overlay as early as 24 hours after its placement, as will be shown in Chapter 9. Therefore, an expedited BCO further maximizes this benefit.

Because it is a thin layer placed on top of an existing pavement, a BCO has another advantage in that potential clearance problems with bridges and other existing structures are minimized. Finally, a BCO, because of the light color of PCC pavement, improves the road visibility at night.

1.2.1 BCO Disadvantages

Most of the limitations of a BCO are related to the project selection stage; thus, they can be avoided with proper project selection. If the drawbacks of a BCO in a particular case are severe, it may not be the optimal rehabilitation strategy.

The BCO reflects the distresses of the existing pavement. If the damage to the original pavement is extensive, the cost of repairs will outweigh the benefits of a BCO, which may not be the optimal solution in that case.

A BCO requires optimum timing in relation to the status of the existing pavement. If a BCO is not built before the existing pavement reaches a more deteriorated condition, the need for repairs will arise, and by then, the BCO may not be an adequate rehabilitation solution.

If the bond between the BCO and the existing substrate is not attained, the structure will not perform as intended. The bond is a critical factor that determines the success or failure of the rehabilitation. In some cases, this bond has been difficult to achieve, mainly because of construction mistakes. Therefore, a higher level of construction inspection is required.

1.3 Objectives

The use of BCOs as a rehabilitation technique for pavements is the focus of this study, and it is exemplified with a specific project. The research objectives are as follows:

- To show the appropriateness of a BCO as a pavement rehabilitation strategy.
- To provide guidelines for the project selection, design and construction.
- To provide techniques and procedures for QC/QA of BCO.
- To demonstrate Objectives 2 and 3 with a specific case study, the implementation of the Fort Worth BCO.

1.4 Scope

This study can be divided into two sections. The first section, including the first seven chapters, addresses BCOs in general. The second part, the implementation section, illustrates a specific application of a BCO case. This report deals primarily with the highway experience of BCO rehabilitations, although the application of the techniques and procedures described herein can be extended to other types of roads, such as city streets.

The guidelines and recommendations provided in this study have been investigated by the Center for Transportation Research (CTR) at The University of Texas at Austin in many different BCO projects across the state, mainly in Houston, El Paso, and Fort Worth, since the early 1980s.

The implementation section is based on the Texas experience with BCOs on CRCP, and it is exemplified with a recent project, the full-scale BCO on Interstate 30 in Fort Worth, conducted by CTR and the Texas Department of Transportation (TxDOT).

A literature review on the historic usage of BCOs is presented in Chapter 2. The literature search encompasses BCOs in Texas and BCOs in other states.

Chapter 3 describes the steps to a successful BCO implementation. Each of the subsequent chapters, from Chapter 4 to Chapter 7, is dedicated to cover one of the steps outlined in Chapter 3.

Chapter 4 is entitled Project Selection. It describes how to identify the need for rehabilitation in an existing pavement, what the available alternatives are when the need arises, and what pavement conditions are suitable for a BCO.

The design of BCOs is featured in Chapter 5, including both the thickness design of the overlay and the reinforcement design.

Chapter 6 details the construction process of a BCO. Important construction aspects that determine the success or failure of the overlay, such as surface preparation, aggregate type, and curing, are analyzed.

Chapter 7 is dedicated to Quality Control and Assurance throughout the BCO process. This entails sampling, testing, and monitoring the overlay over time to ensure that its performance is adequate.

Chapter 8 demonstrates the BCO process described above with an overlay project built in Fort Worth. This project's construction was not flawless, so it is a prime example of how construction problems can affect a BCO, and how a forensic analysis can be conducted to investigate them. The forensic analysis of the Fort Worth BCO is presented in Chapter 9.

Finally, Chapter 10 discusses the results of the study, and Chapter 11 is a summary including recommendations for the use of BCO and future research needs.

2. Literature Review

The historic use of bonded concrete overlays (BCOs) is documented in this chapter. The review encompasses the implementation of BCOs in Texas and nationwide. A section of it is dedicated to BCOs on interstate highways. This retrospective synopsis illustrates how the techniques and procedures for BCOs have evolved through research and experimentation, facilitating and improving the application of this rehabilitation alternative.

2.1 Historic Perspective on the Use of BCOs

Considering construction, maintenance, and rehabilitation costs, the pavement structure is the single most valuable asset of a highway system. Therefore, it has been a concern for pavement engineers to protect this investment in the most efficient way possible. BCOs stemmed from the need to protect this investment in an economical way while keeping traffic disturbances to a minimum.

Portland cement concrete (PCC) overlays were used as early as 1913. In the early 1900s the automobile and truck industry experienced a growth that was hard for the highway and city street pavement designers to predict. The number of vehicles on the roads, their size, and consequently, their weight increased continuously, exceeding the load-carrying capacity of the structures and requiring thicker pavements. Local governments and transportation agencies quickly realized that resurfacing the existing roads, thereby adding thickness to the pavements, was a viable way to keep their roads functional for additional time beyond the original design life of the structure. There are some early instances in which thin overlays (1 to 3 in.) were used, not for the purpose of a structural rehabilitation, but to improve the surface texture of the pavement.

During World War II and in the following years, concrete overlays were instrumental in the successful maintenance of military airport pavements. Many of these pavements were originally designed as 8- to 10-in. thick plain or reinforced concrete slabs. With the continuous growth of air traffic and aircraft weight, those pavements had to be upgraded to accommodate the new load-carrying demands. As a response to this need, the Corps of Engineers implemented a research program to develop procedures for concrete overlay design (Ref 3).

Civil aviation pavements experienced a similar situation. However, unlike the military airplanes, the civil aircraft continued to expand in number and size beyond the 1960s, demanding increasingly thicker overlays (Ref 3).

A wealth of knowledge was acquired on the subject of bonding a BCO to the existing pavement by engineers who started to conduct systematic research just after World War II. The focus of these studies was on the factors that affected performance, for which bonding was identified as a major contributor. In their studies, they documented the condition of the substrate, the treatments applied before the overlay was placed, and the usage of bonding agents.

In the early 1950s, the Portland Cement Association conducted an extensive investigation on the bonding between concrete overlays and old concrete substrates through its Research and Development Laboratories. These early studies, directed by Earl J. Felt (Refs 4 and 5), outlined the fundamentals for attaining a good bond. Felt's research encompassed analyzing many overlays in three different areas: laboratory bond tests, experimental field projects, and existing in-service projects. Surface preparation was identified as the single most important step in the bond strength development process. Good workmanship and quality materials were also recognized as key components of the process. Another finding of the study was that, whenever the old substrate is sound, there is no need for mechanical removal of the concrete surface (Ref 5). He concluded that, in spite of the numerous factors involved in attaining good bond, it is feasible to accomplish it effectively.

In the 1960s, Gillette (Ref 6) studied several BCO projects that were approximately 10 years old. The analysis included overall performance of the overlays, as well as factors affecting the bond between old and new concrete. The performance of the BCOs was outstanding. The following paragraph summarizes the adequacy and benefits of the BCO procedure:

Bonded concrete resurfacing has performed in an excellent manner as a means of strengthening old concrete pavement, providing a new smooth surface, repairing surfaces that have pop-outs, or repairing and patching spalls, scaled areas, etc. (Ref 6).

This study rendered important findings on the topic of bonding, which are discussed herein. These contributions were invaluable for the development of subsequent BCO rehabilitation projects. The study revealed that adequate bond could be attained with normal construction equipment and materials, without the use of any bonding agents. Core samples from projects indicated that bond strength of 200 psi is adequate for a successful BCO. Whenever delamination occurred, it most likely happened soon after the BCO construction. Free water standing on the pavement surface prior to overlay placement was found to be detrimental to the bond. However, some delaminations were found on almost every project he studied. Most of them occurred in small areas, which did not appear to affect the performance of the BCOs for long-term continuous use. The delaminated areas were located by means of the sounding technique. On the subject of discontinuities, it was found that existing cracks and joints in the base pavement would reflect through the overlay. Thus, the joints on the overlay should match the existing joints.

A nationwide trend moving toward concrete overlays where traditionally bituminous mixtures were utilized for resurfacing started in the 1970s. Several states implemented concrete overlays on U.S. highways. This was made possible because of the development of new technologies for concrete paving, in conjunction with the rise in the cost of asphalt, which outpaced the cost of portland cement. Among those innovative technologies, the introduction of scarifying machines with carbide-tipped mandrels was instrumental (e.g., Rotomill). This technology enabled the precise grinding of surfaces, and displaced hand-held pneumatic planers and other devices, and significantly contributed in reducing surface preparation costs to less than $1 per sq. yd (Ref 7).

The Iowa Department of Transportation was the first state agency that experimented with bonded overlays, starting in 1976, and they have constructed many projects ever since. The experience that the Iowa DOT started acquiring in the mid-1960s using thin, bonded, dense concrete overlays to repair deteriorated bridge decks was later successfully applied to the research and implementation of BCOs. The first of these projects was built in conjunction with the city of Waterloo and the Iowa Concrete Paving Association, on U.S. Highway 20, east of Waterloo. The BCO was 2-in. thick and non-reinforced. A lower than normal water-cement ratio was used in the concrete, with the addition of high-range water reducing admixtures to provide workability to the mix. The existing substrate was scarified with the Rotomill machine. The research on the admixtures deemed them to be successful, and the bond achieved was excellent. The shear tests at the interface averaged more than 1,000 psi (Ref 8).

This trend of concrete overlay usage continued in the 1980s, aided by external factors, such as an emphasis on crude oil conservation and increasingly tighter environmental constraints. Nevertheless, the most persuasive reason encouraging highway engineers and agencies turn to concrete overlaying rather than bituminous resurfacing was cost. In this decade, pavement engineers started to base their decisions on total-cost economic analysis (Ref 3), that is, life-cycle costs, which includes initial cost, maintenance and repair costs, and present worth of future rehabilitations during the total life of the structure, including the added life supplied by the rehabilitations. When considering all these components of cost and not only the initial cost, the state transportation agencies realized concrete overlays may be more economical in the long run than asphalt concrete (AC) overlays.

2.2 Interstate Highway BCOs

The Federal-Aid Highway Act of 1956 authorized the construction of the Interstate Highway system, the largest public works program in history. The construction of the initial 41,000 miles of high-quality highways started taking place nationwide. During those early years of the system's life, the main focus was obviously on new pavement construction rather than rehabilitation. However, some existing highways with old pavements —especially in the Northeast— were incorporated into the system, requiring only some overlaying, widening, or both. PCC paving was extensively used in the construction of the new system.

In the 1970s, after 20 years of service, the first interstate pavements approached the end of their service life, and became candidates for BCO rehabilitation. The first BCO on the interstate system was undertaken by the Minnesota Department of Transportation in 1978. This was a pilot project built 10 mi. north of St. Paul, on a 4,200-ft stretch of continuously reinforced concrete pavement (CRCP) on IH-35W. The section was severely spalled and was weakened by chloride corrosion of the reinforcement steel. To prepare the surface, ¼ in. to 1 in. were milled from the existing CRCP; and cement slurry was used to aid the bonding of the 2 to 3-in. thick, non-reinforced BCO (Ref 7).

The findings from this test section were in turn used to design the rehabilitation of 10 mi. of IH-94, in metropolitan St. Paul, one of the most badly deteriorated stretches of the interstate system in the nation at the time (Ref 7).

Representing just over one percent of the nation's highway system mileage, including all urban roadways in the United States (urban, intercity, and rural), the interstate highway system carries nearly one quarter (23 percent) of all roadway traffic, as of figures from 1996 (Ref 9).

The interstate highways were built to accommodate 20 years of traffic growth. By 1985, half of the system had reached the end of its design life, and, by 1995, 90 percent of the system was aged 20 years or older (Ref 9). The original interstate highway system, authorized when the nation's population was less than 170 million, is not much more extensive today, when the nation's population approaches 285 million. Including non-interstate super-highways, expressways, and toll roads, the system now totals 55,000 miles, 30 percent more than the interstate highway system as conceived in the late 1950s, but the nation's population has increased by 70 percent over the same period. The population increase, along with the economic growth, has naturally caused an increase in traffic traveling on the interstate highways, and this has been reflected in the pavements' conditions. In many instances, anticipated usage levels of the system were reached as much as a decade earlier than expected in the original pavement designs.

As of 1996, 40 years after its inception, approximately 60 percent of interstate pavements were rated from fair to poor, according to roughness ratings (Ref 10). Table 2.1, from Reference 9, shows the classification of interstate highway pavement condition.

Table 2.1 Interstate highway pavement ratings

	Rural	**Urban**
Very good	11.0%	9.3%
Good	32.7%	26.3%
Fair	23.7%	23.8%
Mediocre	26.2%	28.3%
Poor	6.4%	12.3%

Fair condition means that the pavement will likely need some improvement in the near future. A mediocre rating indicates that it needs near-term improvement to preserve usability, and a poor-condition pavement needs immediate rehabilitation to restore serviceability. These ratings imply that some of these pavements in need of some type of rehabilitation may be ideal candidates for a BCO repair. Some of these rehabilitation projects have already taken place or are under construction now, including some of the projects analyzed in this study.

2.3 BCOs in Texas

A vast majority of the BCO research in Texas has been conducted on pavements in the Houston area. Heavy traffic is a foremost characteristic of the urban life in this city, which accounts for a sizeable network of concrete pavement roads. The fact that there is a great amount of concrete pavement in the Houston area has provided the district with an extensive expertise in CRCP rehabilitation with BCOs.

The first BCO project in Texas was implemented in 1983 on Interstate Highway 610, the urban section known as the South Loop, which is a major freeway encircling downtown Houston. The project was an experimental BCO on a 1,000-ft CRCP segment, developed by the Texas Department of Transportation (TxDOT) under a cooperative highway research program with the Center for Transportation Research (CTR) at The University of

Texas at Austin and the Federal Highway Administration (FHWA). Built in July and August of 1983, the BCO has delivered excellent performance and is still in service. 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. The surface was prepared by cold milling and sandblasting; portland cement grout was used as a bonding agent for the majority of the section. The existing pavement, built in 1969, consisted of 8-in. thick CRCP on top of a 6-in. thick cement-treated subbase. Table 2.2 shows the factorial with the variables investigated, thickness and reinforcement (Refs 11, 12).

Table 2.2 South Loop factorial

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

A sounding survey conducted in 1990 on this section revealed some minimal delamination of the overlay (Ref 13). Condition surveys conducted in 1996 showed few distresses on the section and no major performance problems (Ref 14).

The success of this first experience led TxDOT to implement a second BCO project, also on the IH-610 Loop in Houston. The section in question consisted of a 3.5-mi. 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-inch CRCP on a 6-in. thick cement-stabilized subbase, was overlaid with a 4-in.-thick BCO in 1986 (Ref 15).

The project was used to experiment with several variables, including reinforcements, coarse aggregates, bonding agents, and existing pavement conditions (various levels of distress). Within the project limits, 10 test subsections, were identified, each one including different combinations of the aforementioned variables. Table 2.3 illustrates the factorial with some of the variables tested and the length of the respective sections.

Table 2.3 North Loop factorial (section lengths in feet)

		Reinforcement Type			
		Steel Fiber		Welded Wire Fabric	
	Aggregate Type	Limestone	Siliceous River Gravel	Limestone	Siliceous River Gravel
Bonding Agent	Grout	-	2,200	1,000	13,400
	None	-	-	-	400

During and after construction, some delamination took place between the BCO and the original pavement. Most of the delaminations occurred within the first 24 hours after placement. Delaminations happened in the presence of adverse environmental conditions during overlay placement, such as high evaporation rates and high daily temperature differentials, and were linked mainly to the sections constructed with siliceous river gravel aggregates, with or without grout. A petrographic study of core samples confirmed the presence of traces of alkali-silica reaction. Even though in some segments the delamination was extensive, it did not continue to deteriorate over time and did not appear to affect performance significantly (Ref 16).

A recent condition survey on this section, conducted in November 2000 as part of a CTR project on the condition of several Houston BCOs, revealed that after 15 years of traffic the performance of the BCO has been excellent. Despite the early delamination problem, those areas have not further deteriorated, and the number and severity of distresses is still minimal (Ref 17).

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. Figure 2.1 shows the location of the first three BCOs in Texas on the IH-610 Loop. Important lessons learned in the IH-610 North project were applied in the construction of this rehabilitation, such as limiting the evaporation rate during construction to less than 0.2 lb./sq.ft/hr. and allowing concrete placement only when the temperature differential expected between placement and the following day is less than 25°F, as adverse environmental conditions surpassing these limits were identified as the primary triggers of the IH-610 North BCO delaminations.

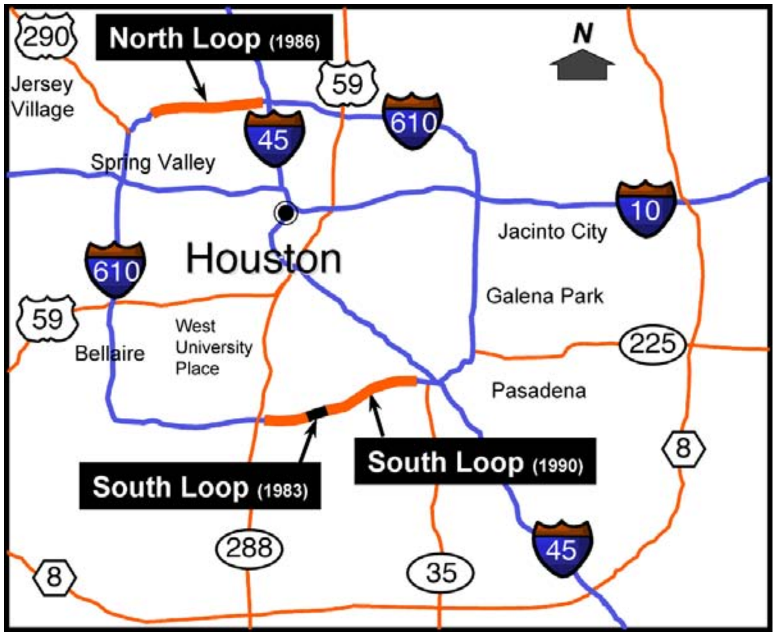


Figure 2.1 Location of first BCOs on IH-610 in Houston

The 8-in. thick CRCP section is about 4 mi. long, and it includes the aforesaid 1,000-ft experimental BCO built in 1983. The approximate project limits are from just east of SH 288 to just west of Telephone Rd. (Ref 18).

This project started in 1989 and was completed in 1990. It consisted of a 4-in. thick BCO with two reinforcement types, wire mesh and steel fibers, with limestone as a coarse aggregate. Portland cement grout, epoxy, and latex-modified Portland cement grout were used as bonding agents in different sections, and two of the sections were placed with no bonding agent (Refs 13 and 18).

The BCO 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. Table 2.4 shows the combinations implemented in each test section (Ref 19).

Table 2.4 South Loop IH-610 experimental factors

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 sections designated as A and B served as control sections for the sections built on each paving date —specifically, Control Section A for Sections 1, 2 and 3, and Control Section B for Sections 4, 5, 6, 7 and 8. Substantial early delaminations occurred in Sections 5 and 6, the sections in which the latex-modified portland cement grout was used as bonding agent. The overlay had to be removed from these sections shortly after construction. Apparently, 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 overlay was placed, the contractor applied new 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 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 from this BCO project is the finding, on the basis of finite element analyses, that most of the debonding is induced at relatively low stresses (under 50 psi) while the overlay is still in its early age. The experiment's results also emphasized the importance of good surface preparation.

The fourth BCO in Texas was placed on IH-10 in El Paso. This project was slightly different from the aforementioned projects, first, because the overlay in question was significantly thicker (6.5 in.) than were previous BCOs in Texas; and second, because the project was intended as an expedited BCO. 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. To say that it is a busy road is an understatement. In a feasibility study, this section was selected for rehabilitation with a BCO in 1993 (Ref 20).

The original section consisted of an 8-in. thick CRCP built in 1965; 8,000 ft in each direction were overlaid in June and July of 1996.

The overlay was planned as an expedited BCO (Refs 21 and 22), which means that expedited paving methods were planned to reduce the normal time between placement and the opening of the lanes to traffic. With this, the overall cost of the project would have been reduced, and the burden to the public caused by lane closures and detours would have been minimized.

However, despite the planning and research invested in the project, construction mistakes caused 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 as the major cause of the debonding problem the high amount of water lost by the overlay before the curing compound was applied. A number of factors contributed to these unusual moisture losses from the concrete. The delay in applying the curing compound in conjunction with high evaporation rates and inadequate surface preparation resulted in a stiff, unworkable mix, which had lost part of its adhesion. The mix had low water content to begin with, because of the higher strength requirement of an expedited BCO. 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 these water losses, the substrate surface should have been prepared by spraying water on it before pouring the concrete (Ref 1).

A delaminated BCO cannot reach its intended service life, because the delaminations impair its capacity to carry traffic and environmental loads. The BCO had to be repaired by means of injected epoxy. The repair work took three weeks to complete, and Falling Weight Deflectometer (FWD) tests confirmed the success of the remedy. A remarkable fact is that even with the high cost of the repair works added to the original BCO cost, it was still less expensive than a full-depth pavement would have been.

The next BCO project in Texas was developed in Houston on Beltway 8, the urban outer loop that surrounds IH-610. The project section, approximately 5.3-mi. long, is located between Greenspoint Drive, just east of IH-45, and Aldine Westfield, near Houston Intercontinental Airport (Figure 2.2). The original 13-in. thick CRCP structure, built in 1984, experienced a severe spalling problem just a few years after construction. By 1995, when this project was undertaken, the CRCP section was in poor condition. A CTR investigation on that pavement concluded that the reason for the spalling was originated by high evaporation rates and high daily temperature differentials that occurred during the construction time. Deflection tests and core samples were extracted to evaluate the structural integrity of the pavement. The tests showed that the spalling problem was only superficial, and it did not affect the load-carrying capacity of the pavement, making it a good candidate for BCO rehabilitation. Thus, a 2-in. thick BCO reinforced with steel fibers was designed and placed in 1996 (Ref 14). No problems have been reported on this BCO to date.

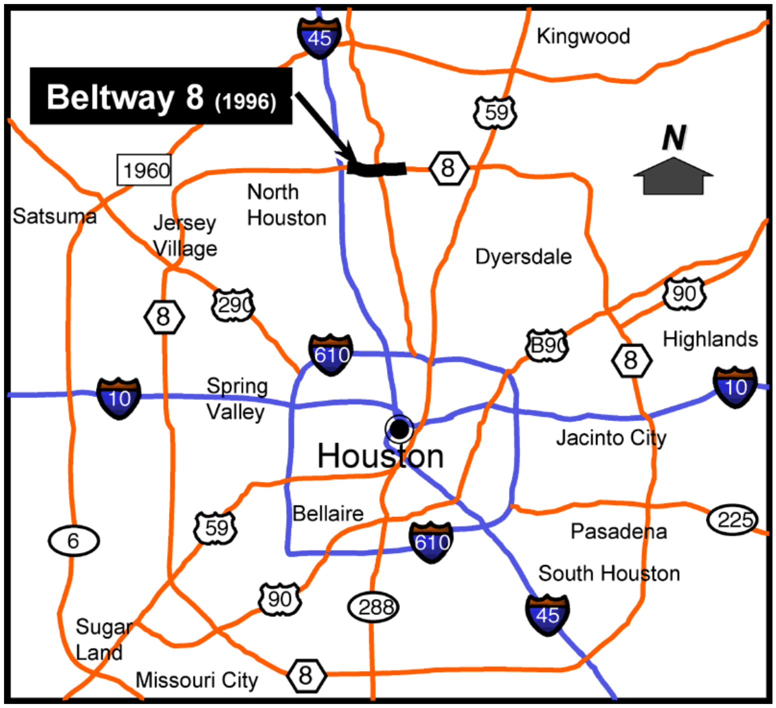


Figure 2.2 Beltway 8 project location in Houston

The positive experience with the Beltway 8 rehabilitation resulted in another BCO project on IH-610, this time in the west part of the loop. The north end of the project is just south of IH-10 near Memorial Park, and the section extends south for 5.5 mi. (Figure 2.3). The original pavement, designed for 20 years, consists of 8 in. of CRCP on 6 in. of cement-stabilized subbase. This section opened to traffic in 1965, and by 1997, when the rehabilitation project started, it had a considerable number of full-depth patches. The extensive repairs that the CRCP had been subjected to over the years prior to the development of this project were due to the heavy traffic volume that this road carries. In 1997, a 5.5-in. thick BCO was constructed (Ref 23).

Figure 2.3 IH-610 project location in Houston

The Fort Worth District undertook the next BCO project in Texas on IH-30 in the west part of town, near the IH-820 Loop (Figure 2.4). The original pavement section was built in 1967, consisting of 8 in. of CRCP over a 6-in. layer of lime-stabilized subgrade. This pavement had been overlaid on several occasions with AC because of low skid resistance. A BCO proved to be a feasible economical and technical way to improve the surface quality of the pavement as well as to extend its service life. A 3.5-in. BCO was placed in the summer of 1998 after the AC overlay was removed. To reduce user costs associated with road closures and delays that happen in high-volume urban highways such as IH-30, an expedited BCO was implemented and traffic was returned to the road about 24 hours after the BCO had been placed. The performance of the overlay was monitored as planned, with condition surveys, in situ sample testing, deflection measurements, and other tests. In February of 1999, a sounding survey revealed the delamination of most of the eastbound outside lane, whereas the westbound lanes were free of delaminations and remained in good condition. A forensic investigation was conducted, with the objective of finding the cause of the delamination problem. The work conducted in the study included the evaluation of the weather conditions at the time of the overlay placement, searching the construction records, and the extraction and testing of cores, including a petrographic analysis, FWD tests and Rolling Dynamic Deflectometer (RDD) tests.

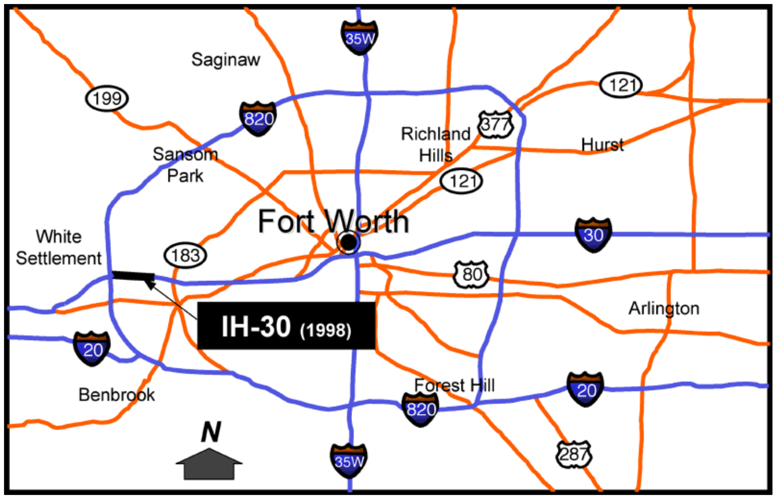


Figure 2.4 IH-30 project location in Fort Worth

These tests showed similar results for both directions; in fact, some of the results of the westbound section, which was not delaminated, were worse than those of the eastbound lanes. The only evidence that led to determining the cause of the eastbound delamination was provided by the petrographic analysis of the cores. The eastbound cores had some debris at the interface between the overlay and the old concrete, whereas the westbound cores were free of debris. Later, meetings with parties involved in the BCO construction revealed that the surface cleaning on the eastbound lanes prior to placement of the BCO was deficient. A great deal of experience was obtained from this project, in which construction mistakes caused the problem, but the concept, appropriateness, and design of an expedited BCO were flawless. If not for the construction errors, the project would have been entirely successful (Refs 2 and 24).

The development of this BCO rehabilitation will be analyzed in detail in Chapter 8, as this project was chosen to illustrate the BCO Implementation in this report. Chapter 9 presents the forensic study of the delamination of this BCO.

During 1997 and 1998, the Houston District and CTR undertook Project LOA-98-0142 to study the feasibility of several BCO rehabilitations of some of Houston's busiest roads. The highways in question included IH-10, two segments of the IH-610 Loop, State Highway (SH) 146, and SH 225. The thickness designs for BCOs were delivered, but to date, none of these projects has been built. However, these resurfacings are scheduled to start in April 2003 and will be the next BCO projects in Texas. The following paragraphs briefly describe the projects.

The IH-10 section that lies between IH-45 and Wayside Dr. in downtown Houston was the first road investigated in this project. The segment is approximately 3 mi. long, including both directions, and the original pavement consists of an 8-in. layer of CRCP on top of 6 in. of cement-stabilized base over a 6-in. lime-stabilized subgrade. The condition surveys for this project were conducted by means of video taken by TxDOT's Multifunction Vehicle. There was a wide range in the pavement conditions of the section; therefore, the BCO design has been developed for the segments in worst condition, and a 6-in. BCO has been designed.

Next in the rehabilitation schedule is the northeast segment of the IH-610 Loop, from IH-45 to IH-10 East. This section is approximately 6.4 mi. long, including both directions. The original pavement consists of 10 in. of CRCP placed over a 6-in. layer of cement-stabilized base and 6 in. of lime-stabilized subgrade. The existing pavement conditions present a high variability along the section, and a 7-in. thick BCO has been designed.

The third phase of the project is the design of the rehabilitation of the southeast segment of the Loop IH-610. The section is approximately 6.2 mi. long, from IH-10 East to IH-45 South. The existing pavement consists of 8 in. of CRCP, 6 in. of cement-stabilized base, and 6 in. of compacted subgrade. For this segment, a 5.5-in. thick BCO has been developed.

The analysis of the SH 146 segment between the Chambers County line and North Main in east Houston is the subject of investigation during the next phase of this project. The existing pavement consists of an 11-in. thick layer of CRCP, on top of a ¾-in. thick asphalt-stabilized base and 6-in. cement-stabilized base, over a 6-in. lime-stabilized subgrade. The conditions of the pavement are very satisfactory, resulting in a thin, 2.5-in. thick BCO design.

Finally, a BCO was analyzed for the rehabilitation of a 4.25-mi. long segment on SH 225 in east Houston, from IH-610 to Redbluff. In this section, the existing pavement is composed of 10 in. of CRCP on 6 in. of cement-stabilized base and 6 in. of lime-stabilized subgrade. This road is in very good condition as well; therefore, the designed BCO is only 3-in. thick.

2.4 Summary

The historic developments presented in this chapter show how the techniques and methods for designing and constructing BCOs have progressed in recent years and how systematic experimentation and research have improved different critical aspects of BCOs. Research on BCOs is a complicated matter because of the number of variables involved, such as surface preparation, type of aggregates, environmental conditions, traffic loads, and so forth. The knowledge acquired through research and the advances in paving technologies have prompted state agencies to embark on more BCO projects to enhance roads, extending their service life in an economical way. Texas has been a prime example of this, becoming one of the leading states in the nation in BCO research and implementation, as illustrated by the projects described herein.

3. Bonded Concrete Overlay Process

The process of a bonded concrete overlay (BCO) encompasses the series of stages that it takes to develop the pavement rehabilitation successfully. This chapter serves as an introduction to the next four chapters of this dissertation. Each of the subsequent chapters is dedicated to cover one of these steps in detail. The result of putting all the steps together will be illustrated with an example in Chapter 8, BCO Implementation.

3.1 Introduction

The process of utilizing a BCO is a complicated one. There are numerous activities that might need to be performed by different agencies, which demands coordination among them. For example, in most cases the designer is independent from the constructor, and both may or may not be part of the state agency conducting the project. Also, some of the activities have to be performed in a particular order, but not all of them need to be conducted sequentially. In fact, one of the components of the process, quality control and quality assurance (QC/QA), is a continuous activity to be handled throughout the process and carried out even after the BCO construction has been finalized.

The BCO process always starts with the need to rehabilitate the roadway. At this juncture, the pavement might show some signs of failure, or might approach the end of its design life, but it is uncertain whether a BCO is the ideal procedure for restoring the pavement.

To determine the most suitable type of rehabilitation, the engineers in charge must conduct a pavement selection process. Once it has been determined that the BCO is the most appropriate rehabilitation alternative, the next step is to design the overlay. Thus the designers must gather information on the current condition of the road as well as historic information such as the amount of traffic that has utilized the facility since its opening. At this stage, several field tests are conducted to assess the structural condition of the pavement. This information is analyzed to determine an overlay thickness design and a reinforcement design.

The next phase is the actual construction of the BCO. The types of materials for the BCO are selected, and the type of cleaning and repairs of the existing pavement are of paramount importance. Special consideration must also be given to the environmental conditions that will prevail during the construction time.

Several tests are conducted during the BCO paving operation to ensure that the overlay has the strength and bonding to perform its function, and these constitute the QC/QA activities of the BCO. Besides testing, the other component of QC/QA is the application of sound workmanship practices and quality construction materials and equipment continuously throughout the process.

The diagram in Figure 3.1 illustrates the stages of the BCO process.

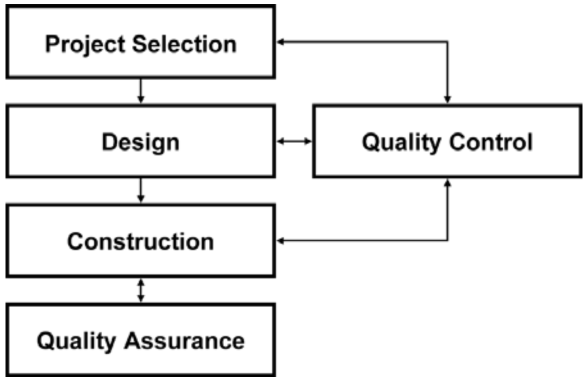


Figure 3.1 BCO development process

The following sections briefly describe each phase, leaving the comprehensive treatment of each stage for the ensuing chapters.

3.2 Project Selection

The BCO development process is triggered by the necessity to rehabilitate the pavement. There are a number of events that can motivate a need for rehabilitation. Among the most common, an increase in traffic from the original design figures might be projected for the facility in the near future, or the pavement may be reaching the end of its design life and there is a need for the facility to be improved. An obvious trigger point is the occurrence of functional or structural inadequacies.

Once the need has been identified, the best rehabilitation alternative must be selected. The pavement conditions are evaluated to determine whether it is suitable for a BCO or whether other rehabilitation options are more appropriate, and this decision is based on technical and economical considerations.

3.3 Design

In the design phase, several variables and properties of the pavement as well as parameters determined by the overlay purpose are analyzed to establish the ultimate output of this stage: the required thickness and reinforcement of the overlay. For instance, if the overlay is being placed for the purpose of structural improvement, as it is in most of the cases, the required thickness of the overlay is a function of the structural capacity of the existing pavement under current conditions and the structural capacity necessary to fulfill future traffic demands. In order to assess the structural capacity of the existing pavement, a comprehensive evaluation of its condition should be conducted, including visual surveys, field non-destructive testing (NDT), and laboratory tests.

3.4 Construction

In the construction phase of the BCO, several activities take place before the concrete is actually placed, and some of these decisions are made collaterally with the thickness design. That is why it was mentioned above that the activities illustrated in the BCO process diagram are not necessarily sequential. In this phase, the following items must be analyzed before the actual placement of the overlay begins:

Materials and aggregate type selection
Milling of existing asphalt layers
Pre-overlay repairs
Subdrainage
Shoulders
Clearances
Traffic control and lane closures during construction
Construction sequence
Entrance and exit ramps
Tapering transitions at project limits and other areas of differing pavement thicknesses
Surface preparation
Surface cleaning
Bonding of the overlay
Environmental conditions
Curing
Time to opening the overlay to traffic

After these BCO considerations have been evaluated and the proper decisions are made, the BCO can be placed.

3.5 QC/QA

Quality is a characteristic of something that satisfies the needs of the customer or that conforms to the standards and specifications established for it (Ref 25). When referring to pavements, quality may imply different attributes for different groups of people, depending on whether it is seen from the standpoint of the public, the owner, or the contractor. For the user, a quality pavement is uniform, durable, and safe. From the owner's perspective, it is cost-effectiveness that defines quality, and from the contractor's standpoint, a quality pavement is one meeting the standards in the most economical way.

QC/QA involves the sampling, testing, and monitoring of the overlay to measure its conformance to standards that ensure that its performance is adequate. QC is a continuous practice conducted throughout the BCO process until the construction is finalized. When the BCO is in place, the QA tests infer the performance level of the overlay.

3.6 Summary

This chapter outlines the BCO development process, which entails the various stages and activities involved in the development of a BCO from its inception to its long-term monitoring after it has been built. The components of the process are presented in detail in the following chapters of this report.

4. Project Selection

Starting with this chapter and in the following three chapters, a detailed discussion of each of the elements of the bonded concrete overlay (BCO) development process is presented.

This chapter features the first stage of the BCO process, Project Selection. After the need to rehabilitate a pavement occurs, the first step toward a successful BCO is evaluating the viable rehabilitation alternatives and selecting the most adequate, from both the technical and economical standpoints. For this purpose, it is necessary to conduct a thorough assessment of the existing pavement conditions and to estimate the cost of the rehabilitation to verify its feasibility, comparing it with the costs of alternative restoration procedures. The importance of the adequate timing of a BCO rehabilitation relative to the current pavement condition is emphasized.

As was explained in Chapter 3, the BCO process is a series of activities conducive to a successful BCO. It is illustrated in Figure 4.1, in which Project Selection has been highlighted.

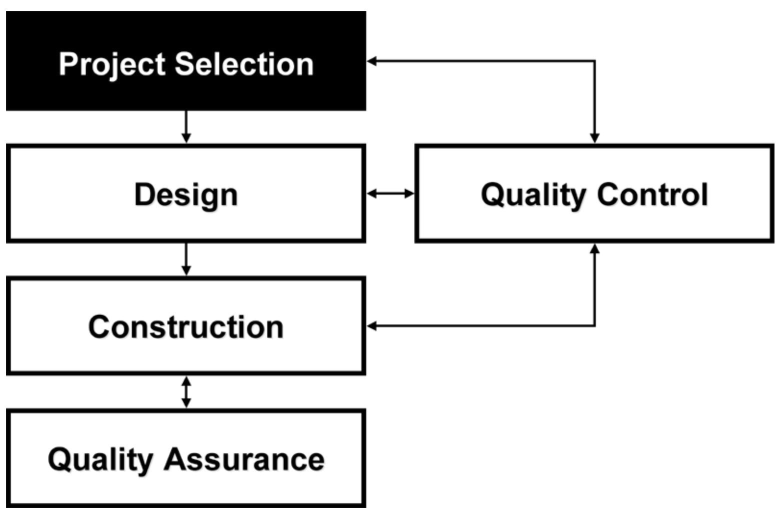


Figure 4.1 Project selection as part of the BCO process

4.1 Need for Rehabilitation

The three major factors influencing the loss of serviceability of a pavement structure are traffic, time, and environment (Ref 26). These factors interact to trigger the need for the pavement rehabilitation. Sometimes the trigger may be a single element, but most of the time is the interaction of factors that signals that the pavement needs rehabilitation. The effects of these factors can be categorized as loads, age, and traffic increases.

4.1.1 Loads

Every pavement is subjected to loads, which cause damage. Loads occur in a pavement even before it has opened to traffic, as a result of the environment and the restraint inherent to the position of the pavement relative to other elements, such as the underlying substrate, adjacent structures, and its own reinforcement. The environment causes contraction and expansion of the materials that compose the pavement; in most cases, they contract or expand at different rates because of their different thermal properties. The environment also makes the materials lose or gain moisture, which in turn causes changes in their composition and volume. These changes are known as environmental loads. Once the pavement opens to vehicle operations, it is subjected to traffic loads. The effects of loads add up as the pavement ages. As a consequence of normal and excessive loads, cumulative traffic, and environmental effects, pavements experience damage, which accumulated effects translate into failure.

Therefore, it is a fact that at some stage of its life, the pavement will show the effects of damage in the form of distresses. An unacceptable level of distress will be the criterion to determine that the pavement has reached a condition of failure.

4.1.2 Age

Pavements are designed to last for a limited period of time, which is determined by the design life. It will not be economically feasible nor physically possible to design a pavement structure that will last forever. Pavement structures are typically designed for periods ranging from 10 years to 40 years. Based upon traffic estimates for the design life, the pavement thickness is determined. Thus, as the facility's service life comes to an end, it is expected that the amount of traffic loads imposed onto the structure will be similar to the number of load applications the pavement was originally designed to withstand. On the other hand, it is known that the properties of materials that constitute pavements change with time. These changes may be beneficial to performance; however, in most cases, the overall influence of age is detrimental to pavement serviceability.

4.1.3 Traffic Increases

Oftentimes, the predicted amount of traffic during the design stage is surpassed well in advance of the end of the pavement design life. An obvious reason for this kind of discrepancy is the inherent difficulty of the traffic prediction task. Also, with growth in population and land development, the usage of the road in question may change from its originally intended purpose to satisfy more ambitious transportation goals, becoming a more heavily traveled road and perhaps connecting to new highways or becoming part of a major corridor that was impossible to predict at the time of design.

4.2 Decision to Rehabilitate

Pavement engineers will seek ways to preserve the integrity of the roadway by means of rehabilitation before considering building a new structure, because rehabilitation means utilizing the existing structure to its fullest possible extent, therefore making better use of the existing infrastructure and optimizing the use of the resources. In a few words, it is the best economical solution unless the structure is in an extremely deteriorated condition.

Because the success of the rehabilitation is dependent on economic as well as technical considerations, at this point the agency must decide whether to embark on a rehabilitation project on the basis of the availability of funds for such an endeavor.

4.3 Type of Rehabilitation

The solution as to how to approach the rehabilitation is not singular. A BCO is just one of the several rehabilitation alternatives, and it is applicable only under certain conditions. If the conditions are not met, the BCO may deliver poor performance and may not fulfill the purpose of its implementation. A BCO is an optimal solution only in certain cases. That is why the other alternatives should be evaluated before a decision is made.

4.3.1 Overlay versus Non-Overlay

When the resources are available, the next decision that the designer faces is whether to use an overlay or to use a rehabilitation method other than an overlay. A feasible alternative is one that addresses the cause of the problem motivating the rehabilitation; therefore, the pavement condition must be investigated before making the decision. The reason for the rehabilitation may be the structural or the functional condition of the pavement. Structural condition refers to whether or not the pavement is fit to support current and future traffic loads over the desired design period. The functional condition encompasses those pavement characteristics related to the way the road serves the user in terms of safety and comfort, such as skid resistance, roughness, appearance, and hydroplaning.

The evaluation of the structural condition involves studying the distress patterns of the pavement, which will provide information about the impact of past traffic loadings. This is assessed by means of a visual condition survey. The visual inspection is normally conducted by personnel with training in distress type identification and with experience on their causative mechanisms. Photographic equipment and audio tape recorders can be advantageously utilized in recording and extracting the data. Historical information on patching, slab replacement, and other repairs are other valuable sources for structural condition assessment. Finally, destructive and non-destructive testing (NDT) methods are extremely helpful in determining the structural integrity of the pavement. Among the NDT procedures, the most common is deflection testing. Destructive testing implies the extraction of samples from the pavement for their laboratory evaluation. All these techniques will be discussed in detail in subsequent chapters. The evaluation of the functional condition requires the measurement of roughness and skid resistance and an assessment of the present serviceability.

A key element to consider is that an overlay can provide structural improvements that are not achievable by non-overlay methods. Non-overlay methods can correct only functional deficiencies; hence, only structurally sound pavements are candidates for rehabilitation without overlay (Ref 26).

There are numerous non-overlay methods available; their applicability depends on the condition they attempt to remedy. Most of them can be used in conjunction with each other or with other techniques. In fact, some of these might be utilized as part of the pavement repairs prior to the placement of a BCO. The discussion of non-overlay methods is beyond the scope of this study; Ref 26 has a comprehensive analysis of their applicability. The most common non-overlay procedures, as listed in Ref 26, are:

Full-depth repair
Partial-depth patching
Joint-crack sealing
Subsealing/undersealing
Grinding and milling
Subdrainage
Pressure relief joints
Load transfer restoration
Surface treatments

4.3.2 Type of Overlay

Once an overlay has been selected over non-overlay methods, depending on the evaluation of the pavement condition, the next resolution involves the type of overlay to apply.

4.3.3 PCC versus AC Overlays

In general, overlays can be classified as asphalt concrete (AC) or portland cement concrete (PCC) overlays. AC overlays are known as flexible, and PCC are referred to as rigid overlays. Both types are applicable to continuously reinforced concrete pavements (CRCP).

The decision as whether to utilize an AC overlay or a PCC overlay depends on the pavement condition as well as economic considerations. Some of the factors to take into account when deciding the overlay type are as follows:

Thin AC overlays are not able to remedy structural deficiencies.
AC overlays represent a smaller initial investment.
PCC overlays, in general, will last longer and require less maintenance.
Considering life-cycle costs, PCC overlays may be more cost-effective.

Thus, conducting a life-cycle cost analysis is advisable in deciding between AC and PCC overlays.

4.3.4 Bonded versus Unbonded PCC Overlays

PCC overlays over CRCP may be bonded or unbonded, as defined in Chapter 1, in which the differences between these overlays were addressed, as well as the advantages and limitations of BCOs. Among PCC overlays, there is a third category, not frequently utilized: the partially bonded overlays. Partial bond exists when a concrete overlay is placed directly on an existing jointed concrete pavement (JCP), where the overlay joints coincide with those of the existing pavement to preclude reflection cracking of the joints through the overlay pavement. For both the partially bonded and the unbonded overlays, the steel requirements and design of the overlay are independent of those of the existing pavement (Ref 27).

To make the decision between bonded and unbonded PCC overlays, it is necessary to define structural and functional failures.

4.3.5 Structural Failure versus Functional Failure

Unbonded concrete overlays, as presented in Chapter 1, are used to rehabilitate severely deteriorated AC or PCC pavements, with only a minimal amount of repairs performed on the distresses prior to placing the overlay. As mentioned earlier, the BCO and the existing pavement behave as a single structural entity whereas both layers remain independent in an unbonded concrete overlay. This difference is normally accomplished by the placement of a thin AC layer between these strata, which ensures that the distresses of the existing pavement will not be reflected in the new overlay.

An unbonded overlay is more cost-effective than a BCO only if the existing pavement is severely deteriorated, because of the unbonded overlay's reduced need for pre-overlay repairs, as opposed to that of a BCO.

Thus, the main factor influencing the choice between a bonded and an unbonded concrete overlay is the current stage of deterioration of the pavement in question —that is, the type of failure motivating the decision to rehabilitate. The types of failure are related to the structural and functional conditions of the pavement, as defined above. A structural failure occurs when a pavement reaches an established level of distress, such as spalling or punchouts. As the main characteristics of functionality in a pavement are safety and comfort for the user, a functional failure refers to that stage at which the pavement has become unsafe or uncomfortable. In terms of serviceability, using the Present Serviceability Index (PSI), Figure 4.2 shows the typical occurrence of structural and functional failure in the lifespan of the pavement, where P_0 and P_t are the initial and terminal serviceability, respectively.

After a structural failure has occurred, a BCO is a feasible rehabilitation alternative. However, when a functional failure has appeared, a BCO is no longer a feasible restoration option, because at that stage, the cost of repairing the extensive damage makes it not cost-effective as compared with other rehabilitation alternatives such as an unbonded concrete overlay. An important drawback of a BCO is that it reflects the distresses of the existing pavement. If the damage to the original pavement is extensive, the cost of repairs will outweigh the benefits of a BCO, which may not be the optimal solution in that case. Therefore, there is a limit in the pavement's deterioration condition in which a BCO can be successfully applied. This limit is after the pavement has reached the stage of structural failure and before it shows functional failures, as is shown at the bottom of Figure 4.2. If the pavement condition gets close to the appearance of functional failure, a BCO may not be ideal.

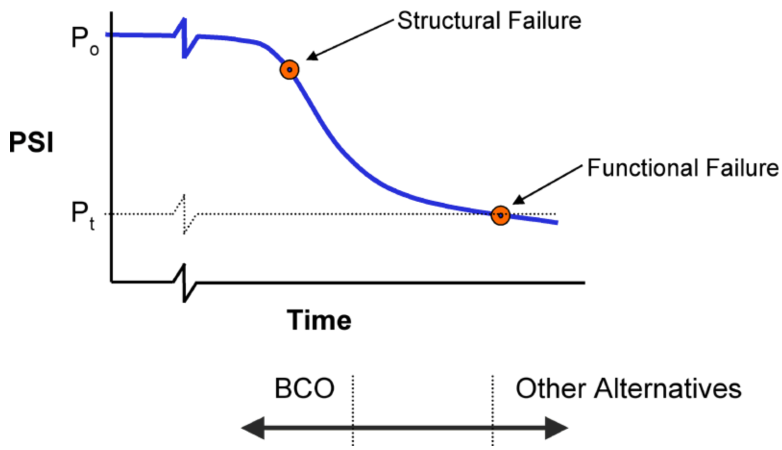


Figure 4.2 Manifestation of structural and functional failure along the PSI curve

There is a stage beyond structural failure but before functional failure, which is illustrated in Figure 4.2, at which it is unclear whether a BCO is applicable. To determine the point at which it is no longer feasible to construct a BCO according to the current conditions of the structure, there are several criteria that the designer should evaluate. These criteria are outlined in Ref 13, and they require field testing such as riding quality, deflections, and condition surveys.

4.3.6 Riding Quality

Riding quality, expressed in terms of PSI, is an indicator of how damaged the pavement is. It is measured by means of a profilometer. The higher the PSI of the pavement, the higher probability there is for a successful application of a BCO. If the PSI is 2.5 or lower, the likelihood of the occurrence of functional failures is high; therefore, at this PSI level, a BCO is not advised and other rehabilitation alternatives should be pursued. PSI measurements within 2.5 and 3 indicate that a BCO is feasible, but only if there is minimum delay in placing it; otherwise the deterioration rate of the pavement will likely make the BCO unsuccessful. For PSI values from 3 to 3.5, there are good conditions for a BCO, and for values above 3.5, the conditions for it are excellent.

4.3.7 Failures

The evaluation should not be based solely on the previous criterion. In fact, many authors recommend evaluating the pavement condition more from a structural standpoint rather than using serviceability criteria, as expressed in the following excerpt from Ref 28:

Evaluating the true condition of the existing pavement is one of the most critical factors in selecting the best overlay option. This evaluation should reflect how the existing pavement will affect the behavior and performance of the overlaid pavement. Such an evaluation should be based on structural or behavioral considerations rather than serviceability considerations.

Addressing this concern, the ideal observable and quantifiable behavioral characteristic is the appearance of failures. The data are collected by condition surveys involving the use of visual inspection to record the type and severity of distress.

A study developed by the Center for Transportation Research (CTR) analyzed the history of failures of approximately 25 CRCP sections in Texas and found that whenever the annual failure rate for a particular pavement was below three failures per mile per year, it was economical to use a BCO, but when the rate surpassed three, an unbonded overlay was the best decision (Ref 29). The study plotted charts similar to that in Figure 4.3 for the pavements investigated. The charts illustrated the development of failures per mile with age for each section. The chart shown in Figure 4.3 is only conceptual, but the actual plots with the projects' data are documented in Ref 29. Of course, every pavement has a different annual failure rate, and the shape of the curve varies from project to project, but the value of three failures per mile per year was found to be a breakpoint for selecting between bonded and unbonded overlays. The reason is that once this rate is reached, the cost of repairs is considered excessive for a BCO. As stated before, an unbonded overlay requires minimum repair of the existing pavement.

To arrive at this conclusion, an economic analysis was performed. The distress quantities were gathered during condition surveys conducted between 1974 and 1978 on CRCPs in Texas, where defects included punchouts and patches. Average cost of repairs as well as user delay costs because patching had to be estimated.

Originally, the breakpoint was defined in the study as the point at which it is better to rehabilitate the pavement than to continue with the routine maintenance activities. This was designated as the point of economic failure: when the current value of maintenance costs and the corresponding user costs occurring over a period of time exceed the cost of the rehabilitation strategy that would last for the same length of time. In other words, the economic analysis entails comparing the current value of a rehabilitation strategy to the current value of continued maintenance. When the latter exceeds the former, the point of economic failure has been reached.

The point of economic failure can also be interpreted as the breakpoint between bonded and unbonded concrete overlays, and this interpretation is assumed in this report as the failure criterion, illustrated in Figure 4.3, for choosing between both types of rehabilitation.

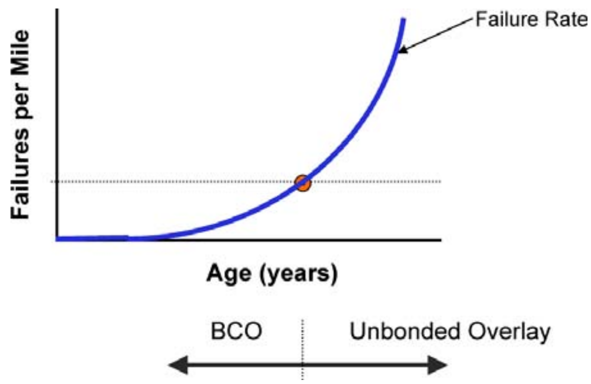


Figure 4.3 Performance curve based on rate of failures per mile per year as criterion for bonded or unbonded overlays

4.3.8 Deflections

An invaluable tool in assessing the structural capacity of the pavement is the measurement of deflections. Deflection measurements are normally made by means of several types of non-destructive testing devices, among which the most common is the Falling Weight Deflectometer (FWD). In the past, other frequently used devices were the Benkelman beam, Dynaflect, and Road Rater, but presently most agencies use FWD.

The criterion that follows, developed in Ref 30, is based on stress calculations and deflection measurements taken at the cracks and at the midspan of pavement slabs. Load transfer is reduced at the cracks, where the transverse stress becomes the critical stress. When the overlay is placed, among other benefits, it reinstates the load transfer capability of the structure. Nonetheless, if the stresses at the bottom of the overlay are still high, cracks will appear in the overlay, the structure will deteriorate, and the original cracks will reflect in the overlay.

For the overlay rehabilitation to be cost-effective, the stresses at the bottom of the overlay must be below the maximum transverse stress at the bottom of the existing pavement; otherwise the overlay will crack.

The ratio of deflections at cracks to deflections at midspan for existing pavements was plotted versus the ratio of maximum tensile stress in the overlay to the maximum transverse stress in the existing pavement. For the stress computations, low and high moduli of elasticity concrete were assumed, as well as three different thicknesses for the existing pavement, 8, 10, and 12 in. An existing pavement stiffness of 4,500 ksi was utilized for the low-modulus concrete, and 6,000 ksi was used for the high-modulus concrete. Their ratios of stresses and deflections are shown in Figures 4.4 and 4.5, respectively. From these plots, it was concluded that when a low-modulus concrete overlay is used, a BCO is feasible when the deflection ratio is less than 1.7 for 8- and 10-in. pavements, and less than 1.85 for 12-in.-thick concrete (Figure 4.4). Similarly, for a high-modulus concrete overlay, the placement of a BCO is advisable if the deflection ratio is less than 1.25 for 8 and 10-in. pavements and less than 1.40 for 12-in. thick pavement (Figure 4.5). These limits are found by the intersection of a stress ratio of 1 with the respective curves.

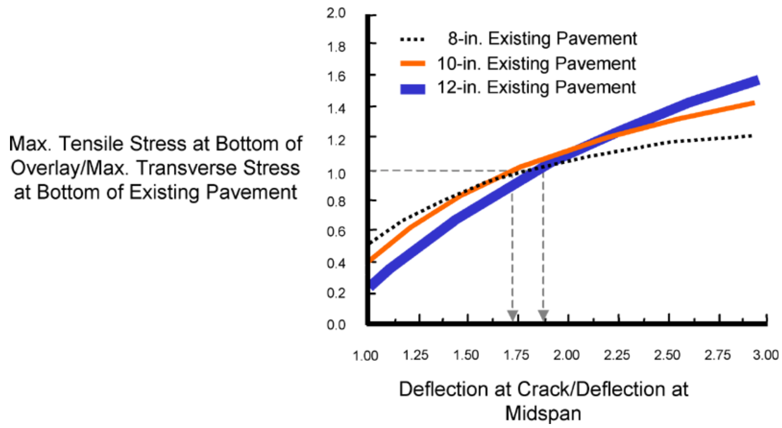


Figure 4.4 Stress ratio versus deflection ratio for low-modulus overlay concrete as criterion for BCO selection

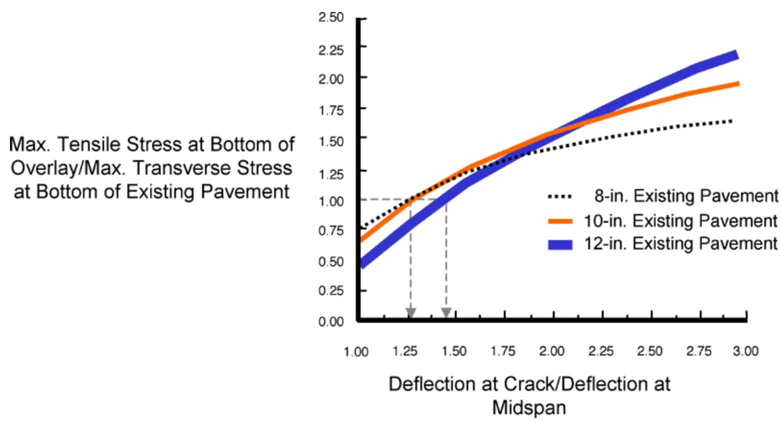


Figure 4.5 Stress ratio versus deflection ratio for high-modulus overlay concrete as criterion for BCO selection

4.3.9 Timing

If a BCO is an appropriate rehabilitation strategy at the project selection stage, it should be noted that it might not be an adequate solution if a considerable amount of time goes by before the BCO is actually constructed. If the deterioration rate during the time elapsed between the project selection stage and the construction stage takes the pavement to a condition of functional failure, by the time the BCO is ready to be built, it may be much more expensive to conduct extensive repairs in the existing pavement. Hence, it is important to place the overlay with minimum delay after the design is ready.

4.4 Summary

The following flowchart (Figures 4.6 and 4.7) summarizes in a simplified way the methodology proposed for the project selection stage.

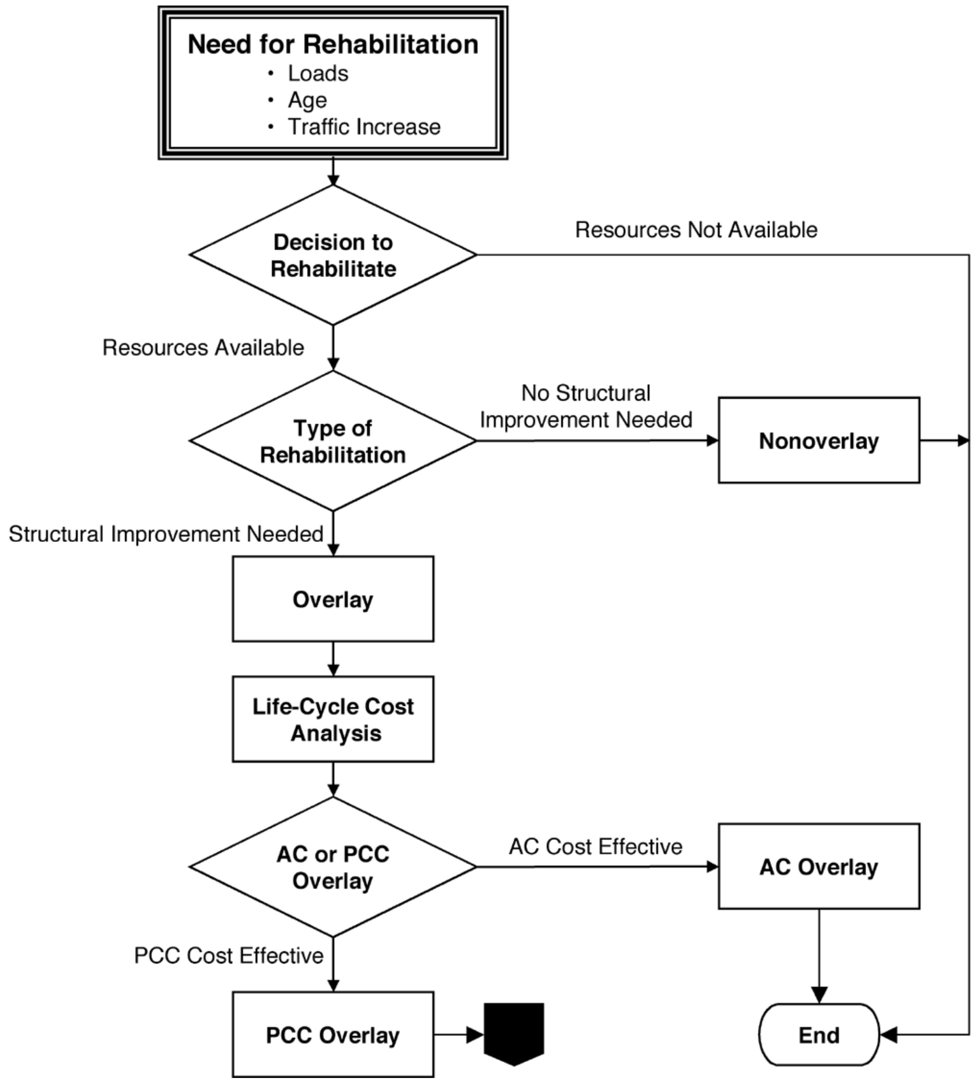


Figure 4.6 Flowchart of the project selection stage

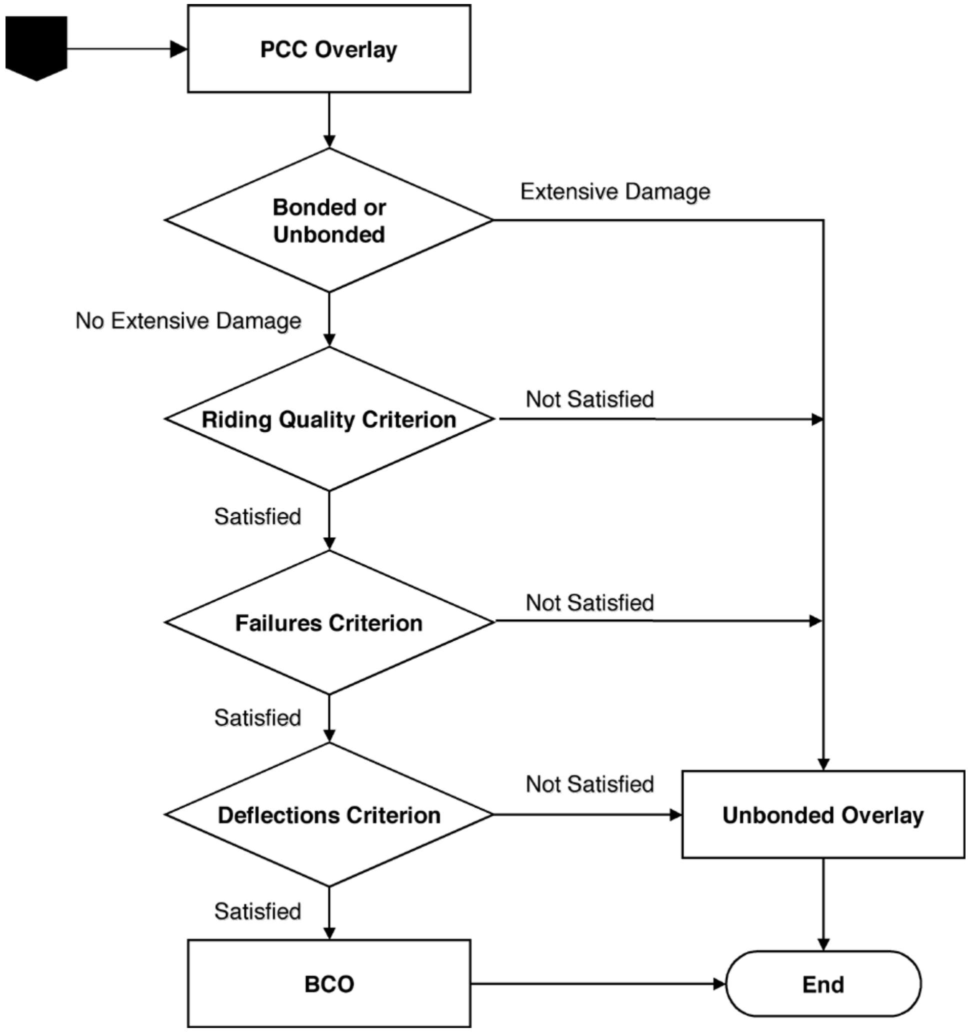


Figure 4.7 Flowchart of the project selection stage

4.5 Afterword

The project selection stage of a BCO may appear to be a series of methodic comparisons, tests, and decisions. Nonetheless, there is room for improvisation, ingenuity, and engineering judgement in every step of the process.

The guidelines outlined in this chapter are not absolute; they utilize subjective judgements as well as probabilities in the involved decisions, making the ultimate success of the project selection a stochastic event. For instance, if one of the criteria for a BCO selection is not met with certainty, according to the engineer's judgement, a BCO may still be a good rehabilitation strategy, but the probability of a successful BCO may be reduced. In other words, the reliability of the project may be diminished.

5. Design

In this chapter, the design stage of the bonded concrete overlay (BCO) is addressed. The design corresponds to the second phase of the BCO process described in Chapter 3, and illustrated in Figure 5.1. Once a BCO has proven to be an appropriate rehabilitation choice for the pavement in question during the project selection stage, the next step is to propose a thickness design and a reinforcement design for the overlay. The thickness design is based on the condition of the existing pavement, the purpose of the overlay, the projected design life for the rehabilitated structure, the historic and projected traffic data, and the material properties of the existing pavement, as well as those of the new overlay.

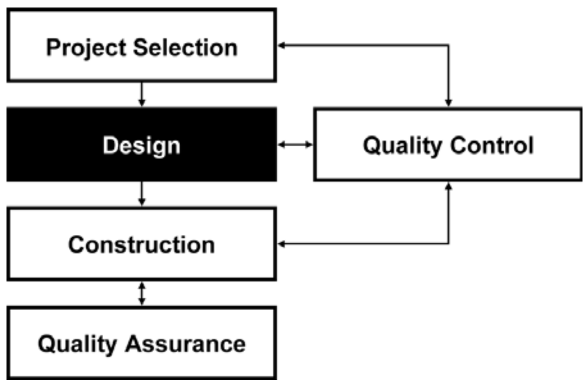


Figure 5.1 Design stage as part of the BCO process

A great deal of the information that is gathered in the project selection stage is utilized in the design phase. In order to accomplish a BCO design in a way that fits the current project conditions, it is important to keep that information updated. The design should occur shortly after the project selection has been finalized. Avoiding delays will minimize the need for collecting the same kind of information again.

This chapter is composed of five major sections. First, an overview of the BCO thickness design is presented, including a review of the design approaches, from a philosophical point of view. The next section deals with the basic design concepts, such as design life, traffic analysis, and remaining life, followed by a section dedicated to testing and evaluation aimed toward characterizing the design input parameters. Then some of the most utilized BCO thickness design methods are discussed. The final section covers reinforcement design for BCOs.

5.1 Overview

The overlay design is not very different from the design of a new pavement. Many of the concepts involved in the design of a new pavement apply to a BCO design as well. Hence, it is worthwhile to conduct a brief review of the pavement design approaches.

5.1.1 Philosophical Approaches to Pavement Design Methods

Pavement design has evolved from art to science. Prior to the 1920s pavement thickness design was based purely on experience (Ref 31). The influence of some of the design variables that were difficult to characterize in a systematic way was either neglected or unknown.

Hence, empirical methods have been used most frequently for pavement analysis and design, many of which have become out of date within a short time. These are based on design equations developed using regression analyses on collected data. An obvious disadvantage of this approach is that the design equations can be applied only to the conditions prevailing at the road test site. If the method is to be used under different surroundings, extrapolation to other conditions must be performed, which reduces accuracy and reliability. A primary example of a data collection effort for the purpose of developing pavement design is the AASHO Road Test, conducted in Ottawa, IL, about 80 mi. southwest of Chicago, between 1958 and 1961. It tested different types of pavement structures under truck loading. Several methods that are still in use are based on data collected during the AASHO Road Tests.

On the other hand, mechanistic design procedures are based on the principles of mechanics. Theory is used to predict stresses, strains, and deflections under a loading system. Most of the mechanistic design approaches make use of empirical data to verify the design model.

Because of the disadvantages of both types of design philosophies, the most sensible approach is to combine the two into mechanistic-empirical methods. A widely known example of this combination is the AASHTO method, in which part of the equation was empirically developed using data from the AASHO Road Test and the rest was based on analytical models. Most mechanistic-empirical methods use the mechanistic part to estimate the stresses in the structure and use empirical information based on laboratory tests and field data to calibrate the model and to estimate distress and long-term performance.

The advantageous use of both design approaches in conjunction with the availability of personal computers and sophisticated methods of material testing have increased the trend toward mechanistic-empirical methods.

Usually, the design of an overlay is similar to that of a new pavement, except that the contribution of the existing pavement is taken into account. Thus, overlay design has been subjected to the same philosophical approaches as new pavement design has been. Prior to 1960, many transportation agencies relied heavily on their experience and engineering judgement in determining the overlay thickness design (Ref 31). However, with the advent of better testing techniques, it has been possible to conduct a more thorough evaluation of the existing pavement, allowing for a better assessment of its contribution to the overall structural capacity of the new rehabilitated structure.

5.1.2 Purpose of the Overlay

The rehabilitation is aimed to remedy a specific situation, which the designer knows from the project selection phase. As noted in the previous chapter, the need for rehabilitation arises as the pavement experiences a decrease in serviceability; this occurs as a consequence of load applications, age, and traffic increases. The interaction of these factors is manifested as structural and functional failures. In most cases, the overlay is placed for the purpose of structural improvement; consequently, the required thickness of the overlay is a function of the structural capacity of the existing pavement under current conditions and the structural capacity necessary to fulfill future traffic demands.

However, if the pavement deterioration is a consequence of non-loading factors(i.e., the purpose of the overlay is functional improvement), the design equations will render a minimal or zero thickness, in which case a minimum constructible BCO thickness will be enough to address the functional deficiency. BCOs as thin as 1 in. have been used successfully on sound pavements (Ref 28).

5.1.3 Basic Design Principle

The foremost structural characteristic that differentiates a BCO from other rehabilitation concepts is that, by definition, the overlay behaves as a single unit in conjunction with the existing pavement. Therefore, the structural capacity remaining in the existing substrate is fully utilized. As such, it is accounted for in the design equations, which contributes to reduce the required thickness of the overlay. This is attainable only if the bond between overlay and substrate is achieved and maintained.

Therefore, if the purpose of the BCO is to remedy structural deficiencies, the design is based on a simple equation. The BCO is designed by determining the additional thickness of concrete needed to carry the anticipated traffic. Thus, the equation is as follows:

$$D_{BCO} = D_f - D \tag{5.1}$$

where D_{BCO} is the overlay thickness, D_f is the required thickness to carry the future traffic if the pavement were constructed new, and D is the effective existing pavement slab thickness. The design methods vary in the way in which the contribution of the existing pavement is determined and the way in which the original thickness is affected by the pavement condition.

5.2 Design Concepts

A great deal of the same types of information and decisions required in new construction projects are also required in rehabilitation projects such as BCOs. Many of the following concepts apply to both new construction and rehabilitation projects.

5.2.1 Design Life

The design life or design period of the overlay is the time for which the rehabilitation is anticipated to last. The design life has a direct correlation with the loads applied over such a period of time; hence, it also means traffic analysis period. It is inherently difficult to predict the traffic expected on a road over a period of time. Furthermore, the longer the period considered, the less reliability the traffic prediction will have. Typically, the design period ranges from 20 to 40 years. It is a common practice to establish longer design periods as the importance of the facility increases. For instance, an interstate highway will have a longer design life than a low-volume road will.

When the BCO rehabilitation is undertaken in conjunction with another upgrade of the facility, such as a road widening, the design periods of both the overlay and the widened lanes should be the same to ensure that the next rehabilitations will not occur at different times.

5.2.2 Traffic Analysis

The traffic analysis estimates the 18-kip Equivalent Single-Axle Load (ESAL) repetitions along the project length from the date the pavement was opened to traffic through the end of the anticipated overlay design period. The projection of the number of ESALs in the future is based on the historical traffic information. The projected number of ESAL applications over the design period is a key input parameter of the overlay thickness design.

5.2.3 Reliability

Reliability is a means to add certainty to the design process; it is defined as the probability that the design will perform its intended function over the design life. This concept was introduced in the AASHTO method in 1986. The reliability level has a large effect on overlay thickness. The designer can select the level of reliability of the BCO design according to the consequences of an eventual failure of the overlay. Higher levels of reliability are associated with the importance of the highway and will result in thicker overlays. The reliability level may be set by the policies of each agency with respect to the highway functional classifications. Field testing has shown that a design reliability level of approximately 95 percent results in overlay thicknesses that conform with guidelines recommended for most projects by state highway agencies.

5.2.4 Overall Standard Deviation

Because there is variability inherent in traffic predictions, material properties, quality control, and environmental conditions during construction, it is reasonable to use a probabilistic approach in the design, rather than a deterministic one. The overall standard deviation adds flexibility to the design; it accounts for all the sources of uncertainty involved in the overlay design, such as the material properties and traffic data. If the information corresponding to the materials characterization and traffic prediction is deemed to be accurate, the data will have a small standard deviation, rendering a thinner overlay. On the other hand, if the information was not collected properly, is not entirely available, or is not trustworthy, the data will have a large standard deviation, resulting in a thicker overlay. The AASHTO Guide (Ref 26) recommends a standard deviation of 0.39 for PCC overlays.

5.2.5 Remaining Life

A concept that has no application in new design but has a decisive influence in the outcome of the overlay design process is the remaining life of the existing pavement. It recounts how much of the initial expected life of the pavement is left after being in service for a number of years.

There are different ways to arrive at an estimate of a pavement's remaining life. The recommended method uses a mechanistic fatigue damage concept, in which repeated loads progressively damage the pavement, reducing the number of future loads the structure will carry before failure. Even though the structure may not show visible signs of damage, every load application represents a reduction of its effective structural capacity, which takes it closer to its ultimate failure. To estimate the remaining life using this idea, it is necessary to determine the amount of traffic the pavement has carried to date and the total amount of traffic the pavement is expected to carry until failure, expressed in ESALs. In terms of the AASHTO design concepts, this condition of failure is reached when the pavement serviceability equals 1.5 (PSI).

The ratio of traffic to date to traffic to failure, in percentage, subtracted from 100 percent will give the percentage of remaining life:

$$RL = 100\left[1 - \left(\frac{n}{N}\right)\right] \tag{5.2}$$

where

RL	=	remaining life, percentage
n	=	total traffic to date, 18-kip ESAL
N	=	total traffic to pavement failure, 18-kip ESAL

The total traffic to pavement failure (N) can be estimated using the design equations for new pavement proposed in Ref 26, presented later in this chapter, using 50 percent reliability to ensure consistency with the AASHO Road Test and the development of these equations.

Another method to assess remaining life, considered to be less precise, is based on the visual condition survey in combination with empirical equations.

5.3 Existing Pavement Conditions and Materials Characterization

The initial evaluation of the existing pavement condition is conducted during the project selection stage. Ideally, all the information gathered at that stage is current, and, therefore, can be used for design purposes.

Condition surveys and deflection test data are necessary for the thickness design. These are among the first tests conducted on any section that is a candidate for rehabilitation. The properties of the materials that constitute the existing pavement are determined from these tests and also from destructive testing.

Proper determination of the existing pavement conditions and materials characterization will allow the designer to determine whether different BCO designs are appropriate throughout the project section, should various subsections with different characteristics be identified.

In this section, the two most frequently used techniques for pavement characterization are presented (i.e., condition survey and deflection testing), followed by a series of properties that are determined from the tests and that are used as design parameters.

5.3.1 Condition Survey

The purpose of the visual survey is to provide data concerning the types of distress present in the pavement and their location, severity, and extension. Locating the section is the first step in conducting the visual evaluation. Location of the section is accomplished by the use of reference markers, which are visible highway route signs commonly found on the side of the road or the median. In Texas, there is a set of rules that addresses the numbering and location of reference markers to ensure consistency across the state, (Ref 32). The use of a global positioning system (GPS) as an alternative means of locating pavement sections is advisable, because the reference marker rules may at times appear to be disregarded on some roads. The distress types to be recorded on CRCP sections, according to Ref 32, are as follows:

Spalled cracks
Punchouts
Asphalt patches
Concrete patches
Average transverse crack spacing

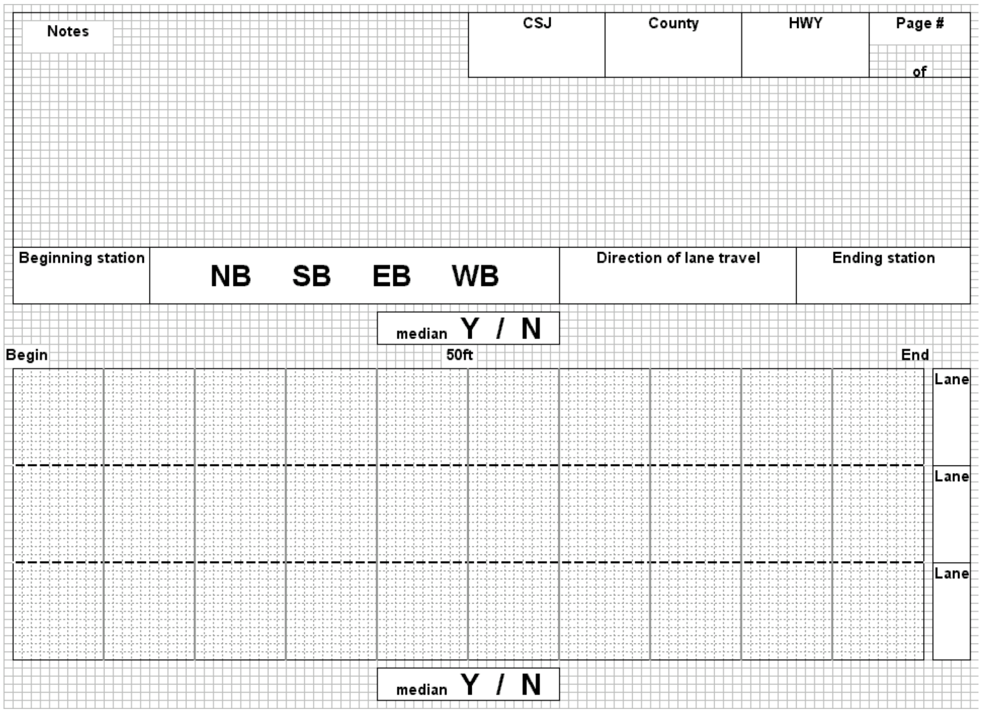
A detailed description of these distress types and guidelines to identify them are presented in Ref 32. A suggested format for recording data is shown in Figure 5.2.

CFTR No.	Direction	Highway	Date	Surveyor	GPS Location
	NB	US 281	10/24/2001	MT	Lat N ° ′ ″
Reference Marker: Start RM_1 to End RM_2	County: Wichita		CRCP	Overlaid / Non-overlaid	Long W ° ′ ″ / Elevation (ft)

Station From	Station To	No.	Number of Lanes 2 / Surveyed Lane	Patches (size in squre feet): ACC 0-50 ft^2	ACC 51-150 ft^2	ACC >150 ft^2	PCC 0-50 ft^2	PCC 51-150 ft^2	PCC >150 ft^2	No. of Punchouts (M-minor, S-severe): M	S	No. of Spalls (M-minor, S-severe): M	S	Total No. of Transverse Cracks	Comments
28.000	29.000	1	R2											10	
29.000	30.000	2	R2											15	
30.000	31.000	3	R2											15	
31.000	32.000	4	R2											18	
32.000	33.000	5	R2											9	
33.000	34.000	6	R2											0	Bridge
34.000	35.000	7	R2											0	Bridge
35.000	36.000	8	R2											4	Bridge
36.000	37.000	9	R2					1						14	
37.000	38.000	10	R2											19	
38.000	39.000	11	R2											18	
39.000	40.000	12	R2						1					14	
40.000	41.000	13	R2											13	
41.000	42.000	14	R2											16	
42.000	43.000	15	R2											13	

Figure 5.2 Sample condition survey form

In some cases, it is convenient to register the approximate location of the distresses in a condition survey form, such as the one shown in Figure 5.3, which allows mapping them and comparing them in successive surveys during the monitoring phase to analyze the behavior of the distresses over time and their possible reflection in the new BCO.


Notes

CSJ | County | HWY | Page # of

Beginning station | NB SB EB WB | Direction of lane travel | Ending station

median Y / N

Begin 50ft End

Lane

Lane

Lane

median Y / N

Figure 5.3 Sample of a condition survey form for mapping distresses

5.3.2 Deflection Testing

Since 1960, the non-destructive deflection testing methods have become more widely known and have earned acceptance as a means to evaluate the in situ pavement conditions (Ref 31).

The most common device for deflection evaluation is the Falling Weight Deflectometer (FWD), shown in Figure 5.4. As was mentioned in Chapter 4, there are other deflection devices that were used in the past, such as the Benkelman Beam, Dynaflect, and Road Rater. Currently, these devices are seldom used by state agencies. The Center for Transportation Research (CTR) at The University of Texas at Austin has used a device called the Rolling Dynamic Deflectometer (RDD) for the semi-continuous recording of deflections along the entire project section, but for an evaluation of the pavement conditions at this stage, a discrete measurement of deflections will render sufficient detail. The specifics of RDD and its usage will be presented in subsequent chapters.

For FWD tests, several loads are applied at each testing location by dropping fixed amounts of weight from a given vertical height above the pavement; different weights are dropped at each time. The pavement response is measured as deflections by sensors (geophones) placed at predetermined distances from the load along the pavement.

Figure 5.4 Falling weight deflectometer

The deflection testing intervals vary depending on the project characteristics. It is common practice to conduct FWD measurements approximately every 400 ft, but this number is to be taken as a general guideline only. The length of the project, its overall condition, and the availability of resources will ultimately determine the frequency and extensiveness of the deflection testing. As was mentioned in Chapter 4, two types of deflection information should be collected: Measurements should be taken along continuous slabs of pavement with no cracks between the deflection sensors (i.e., at the midspan), for elastic layer moduli evaluation; the second type of measurements should be conducted across transverse cracks for load transfer evaluation purposes. For this kind of measurement, it is recommended to arrange the FWD sensors with respect to the crack in the way illustrated in Figure 5.5, in which Sensor Number 4 is positioned on the other side of the crack with respect to the remaining sensors (downside arrangement).

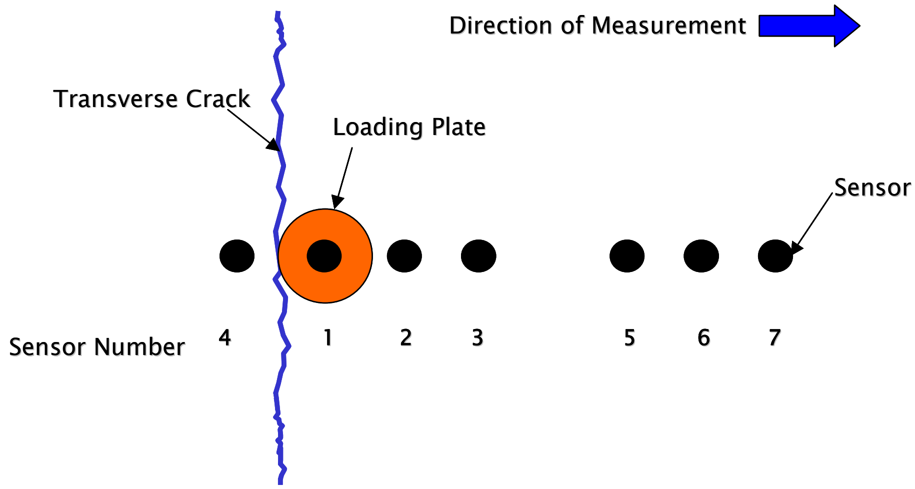


Figure 5.5 FWD downside loading and sensor arrangement for load transfer measurement (plan view)

5.3.3 Modulus of Elasticity

The modulus of elasticity of a pavement is a measure of its stiffness; it is used in characterizing the pavement's load-carrying capacity. It is defined as the ratio of stress to strain. For materials with a linear stress-strain relationship, the modulus is simply the slope of the stress-strain line, but for non-linear materials such as concrete, the modulus estimation is more complicated.

The non-linear behavior of concrete makes it obvious that the conventional concept of linear elasticity does not apply in this case. Therefore, in order to systematically characterize this property, it is necessary to resort to arbitrary definitions based on empirical considerations. For instance, it is possible to define the initial tangent modulus or the modulus tangent to a predetermined point of the stress-strain curve. The secant modulus is more frequently used in laboratory tests.

However, the most common method to arrive at moduli of elasticity values for the pavement structure is through deflection testing. The surface deflection data from FWD collected at the midspan of slabs (i.e., between cracks), are used to calculate the elastic moduli of the pavement layers, by a procedure called backcalculation. Normally, only one predetermined loading level of the FWD is considered, which means that only one of the weight drops is utilized. A load magnitude of approximately 9,000 lbs. is recommended, because it simulates the standard wheel load of an ESAL at one spot, as proposed in Ref 26.

Backcalculation is an iterative process that may be tedious and time consuming; therefore, it is recommended to analyze the FWD data by means of computer programs. The goal of backcalculation is estimating the pavement material layer stiffnesses, finding a set of parameters that corresponds to the best fit of the measured deflection basins, and minimizing the differences between the measured and the calculated deflection bowls. At CTR, a computer program called Modulus (Refs 33 and 34) has been used for this kind of analysis. The program was developed for flexible pavements, but it has been successfully used for interior loading conditions of PCC pavements as well. This program is designed to process data collected with FWD using a linear elastic procedure to generate a database of computed deflection bowls prior to the backcalculation process. The program iterates until the measured and computed deflections converge.

An alternative method is to backcalculate the elastic moduli using the charts and the equation provided in Ref 26.

Modulus values of concrete pavement may also be determined in the laboratory by sample testing. Underlying strata can be tested too, but this is seldom done, as deflection testing is a more economical option and provides more coverage. Many times, laboratory tests are used as a resource to confirm the certainty of the backcalculated moduli.

When both deflection and core testing are available and the resulting elastic moduli are compared, it should be noted that the modulus obtained from the core testing will be lower than the modulus obtained from deflection tests because the stress level applied to the core samples is higher than the stresses that occur during deflection testing and because of the non-linear elastic behavior of concrete. Typically, cores specimens are loaded to approximately 45 percent of their estimated compressive strength with which the secant modulus is calculated, whereas for deflection tests the stress levels are much lower, resembling an initial tangent modulus, as is illustrated in Figure 5.6.

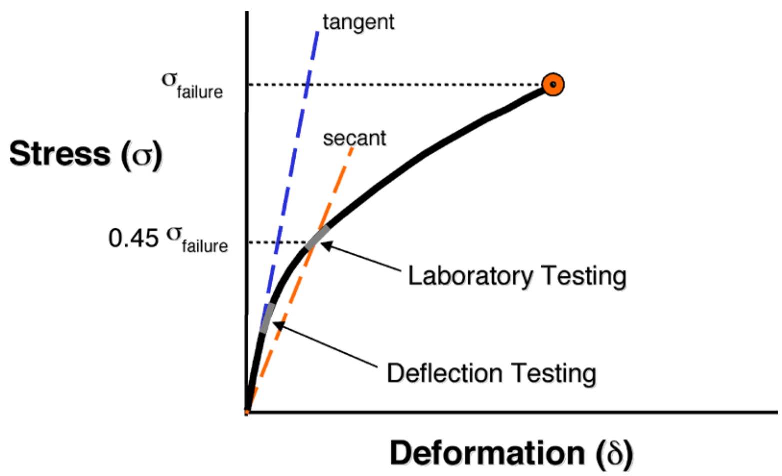


Figure 5.6 Moduli of elasticity for coring and deflection testing

There are several empirical equations that have been developed to estimate the modulus from other concrete properties. An example is the expression shown below, where the concrete modulus, E_c, in psi, is inferred from the compressive strength, f'_c, in psi.

$$E_c = 57{,}000\sqrt{f'_c} \tag{5.3}$$

5.3.4 Subgrade Modulus

From the backcalculation process, the moduli of all the pavement layers, slab, subbase, and subgrade can be estimated. Nevertheless, the subgrade modulus requires additional considerations. Several factors affect the stress-strain relationship of soils, such as the moisture content, confining pressure, and density, making it a very complex characteristic. When referring to subgrades, the elastic modulus of the soil is known as the resilient modulus, M_R. In rigid pavement design, besides the resilient modulus, another concept may be used to characterize the subgrade, known as the modulus of subgrade reaction, k.

The resilient modulus may be estimated by backcalculation of deflection test results, as explained above, which may be the easiest way, but it may also be determined in the laboratory using a triaxial test with repetitive loading. The modulus of elasticity is the relationship between stress and strain; therefore, M_R is the ratio of the repetitive axial deviator stress, σ_d, and the recoverable axial strain, ε_a.

$$M_R = \frac{\sigma_d}{\varepsilon_a} = \frac{\sigma_1 - \sigma_3}{\varepsilon_a} \tag{5.4}$$

The modulus of subgrade reaction is defined as the ratio between an applied pressure and the ensuing deflection. It is determined by a plate-loading test. The plate is 30 in. in diameter, and the load is applied at a predetermined rate until a pressure of 10 psi is reached. The concept of the modulus of subgrade reaction is illustrated in Figure 5.7

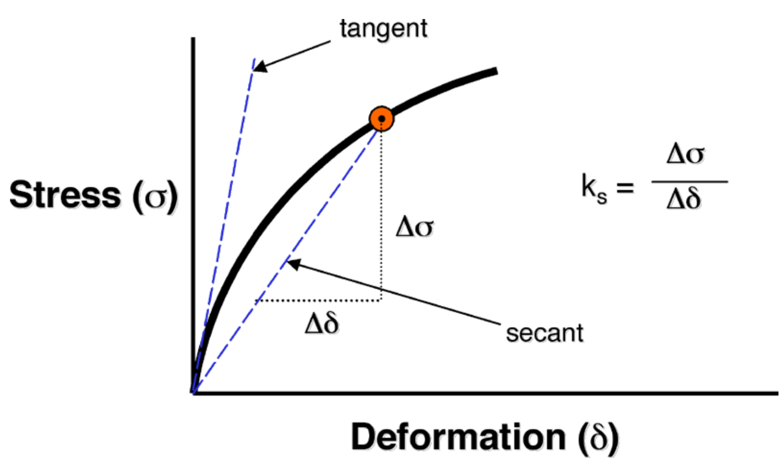


Figure 5.7 Modulus of subgrade reaction

As Figure 5.7 shows, k varies depending on the stress level used, whether the modulus is considered tangent or secant. It also varies depending on the moisture conditions; therefore, it is recommended to analyze k on a seasonal basis. The modulus of subgrade reaction is directly proportional to the roadbed soil resilient modulus; however, there is no unique correlation between both. A mechanistic based conversion from M_R, in psi, to k, in pci, is shown in Equation 5.5 (Ref 26), but it is applicable only if the slab is placed directly on the subgrade, which does not occur very often.

$$k = \frac{M_R}{19.4} \tag{5.5}$$

However, the previous equation renders values of k that are too large —it overestimates k. The composite modulus of subgrade reaction for the case when the pavement slab is placed on top of a subbase, can be determined by means of the chart proposed in Ref 26 (shown in Figure 5.8), which was developed by applying the 30-in. plate loading onto a two-layer system.

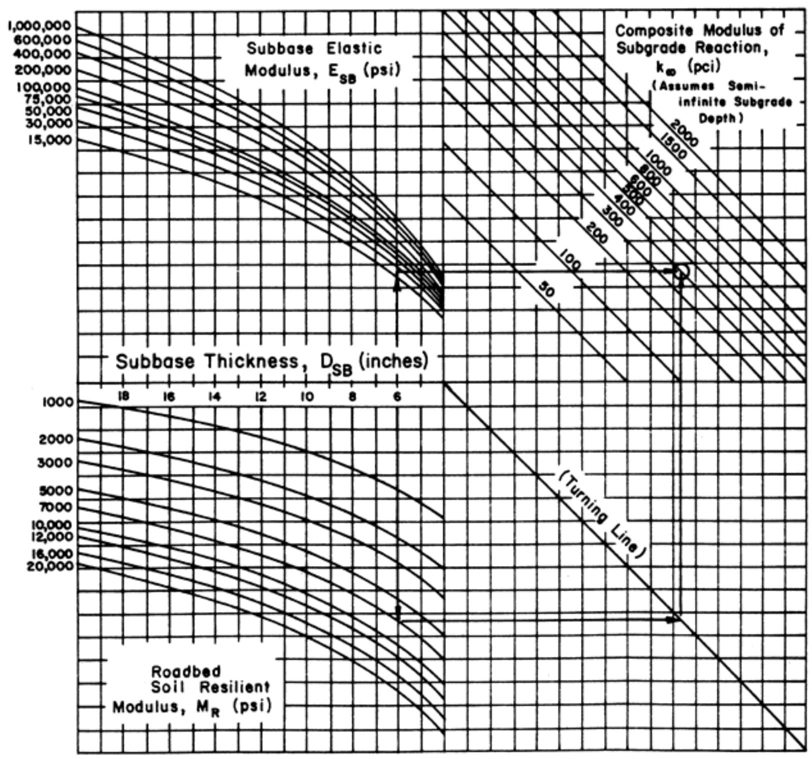


Figure 5.8 AASHTO procedure to estimate the composite modulus of subgrade reaction

5.3.5 Concrete Flexural Strength

For certain applications of concrete, one of which is PCC pavements, it is necessary to know its strength in flexure. Also known as modulus of rupture, the flexural strength of PCC is evaluated by beam-breaking tests. Concrete beams, 6 in. by 6 in. by 18 in., are cast and tests are normally performed at 7, 28, and 90 days. The beam specimens are tested in third-point loading, with the break occurring suddenly with the appearance of a single crack. The value used for design is the mean value of the 28-day test specimens. An alternative way of estimating the modulus of rupture is by means of an approximate relationship with the concrete compressive strength (Ref 27).

$$S'_c = K\sqrt{f'_c} \tag{5.6}$$

where

S'_c	=	PCC modulus of rupture
K	=	constant between 8 and 10
f'_c	=	PCC compressive strength

5.3.6 Load Transfer Efficiency (LTE)

The LTE of a pavement structure refers to its ability to transfer loads across transverse discontinuities such as joints or cracks. A high LTE value indicates that the pavement structure is capable of distributing the loads adequately at the discontinuities. The LTE calculations make use of the deflection data collected across transverse cracks, as was explained before, and as is shown in Figure 5.5. In this study, two different procedures are recommended to calculate LTE, which make use of a deflection ratio. The first procedure is described by the following expression proposed by Teller (Ref 35).

$$LTE = \frac{2W_U}{W_U + W_L} * 100\% \tag{5.7}$$

where

LTE	=	load transfer efficiency (percentage)
W_U	=	deflection on the unloaded slab
W_L	=	deflection on the adjacent loaded slab

If the LTE is zero, it means that no load is transferred from the loaded slab to the adjacent unloaded slab. In the case of perfect load transfer, the load is distributed completely from the loaded slab to the unloaded adjacent slab (i.e., the deflection is the same in both slabs), and LTE is equal to 100 percent.

The second procedure for determining LTE was developed by Darter (Ref 35).

$$LTE = \frac{W_U}{W_L} * 100\% \tag{5.8}$$

where the variables are the same as in Teller's equation.

5.4 Current Overlay Design Procedures

There are a number of BCO design procedures available. The basic concepts of some of the most common procedures are discussed below.

5.4.1 Corps of Engineers

The U.S. Corps of Engineers procedure was originally devised for the design of PCC overlays over PCC airfield runways and taxiways. It was developed using full-scale accelerated test tracks (Ref 36).

This method uses a version of the basic design equation (5.1) presented above, modified by empirical coefficients. In this method, the required thickness of the overlay is the difference between the thickness required for a new pavement and the thickness of the existing slab. In Equation 5.1, instead of the thickness of the existing slab, the thickness considered is the effective thickness. Three models were developed —namely, for the bonded, partially bonded, and unbonded cases, represented by Equations 5.9, 5.10, and 5.11, respectively. Even though the scope of this report covers only the bonded overlay case, the other two are presented as a reference to provide a better understanding of the methodology.

$$h_o = h_n - h_e \tag{5.9}$$

$$h_o = \sqrt[1.4]{h_n^{1.4} - Ch_e^{1.4}} \tag{5.10}$$

$$h_o = \sqrt{h_n^{2} - Ch_e^{2}} \tag{5.11}$$

where

h_o = required overlay thickness

h_n = required theoretical thickness for the design loading, for a new pavement

h_e = existing pavement thickness

C = condition correction coefficient

The values for C are determined by visual inspection and range between 1 (for a pavement in good structural condition) to 0.35 (for a pavement with severe structural defects).

As can be inferred by these equations, it is assumed that for a BCO the condition correction coefficient equals 1, which means that the existing slab is in good structural condition or will be upgraded to this condition. Besides this coefficient, the other difference among these equations is the exponent of the thicknesses, which is related to the bonding characteristics of each type of overlay. For a BCO, the value of the exponent is 1, given that the BCO and the substrate will behave monolithically. In a similar fashion, it is equal to 1.4 and 2 in Equations 5.10 and 5.11, for the partially bonded and unbonded overlays, respectively.

In this method, it is implied that the existing concrete has suffered no fatigue because traffic or other factors, and it is as strong as the concrete in a new pavement, which contradicts the fatigue damage concept explained above and the idea of remaining life. Furthermore, it assumes that the failure mechanism of the overlaid pavement is the same as that of a new pavement.

Failure is defined as the load application level at which cracking or structural breakup first appears, which does not apply properly to highway concrete overlays, where cracking is an inherent occurrence of PCC pavements.

5.4.2 Portand Cement Association

The PCA methodology consists in designing an overlay system that is structurally equivalent to a new full-depth pavement placed on the same subbase and subgrade. Unlike the Corps of Engineers procedure, it uses an evaluation of the existing pavement by means of condition surveys, deflection tests, and in situ sample testing to take its condition into consideration in the design.

The design basis is the analysis of the stresses at the edge of the pavement (Ref 37). The model equates the edge stress at the bottom of the new full-depth pavement (σ_n) with that of the overlaid system at the bottom of the existing pavement (σ_e), as is shown in Figure 5.9.

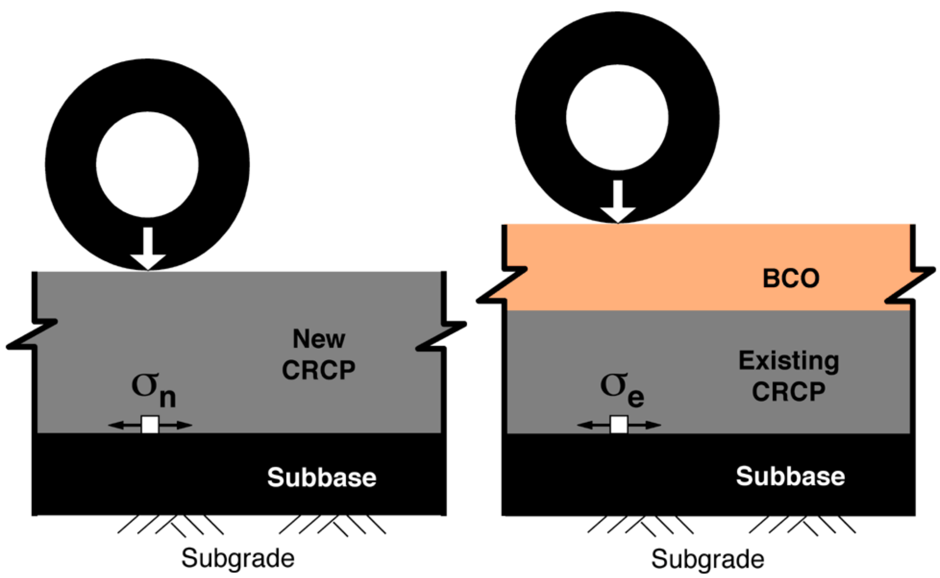


Figure 5.9 Edge stresses for new and overlaid pavement for PCA method design equivalency

Because the new full-depth slab and the existing concrete will have different moduli of rupture, S_c, the equivalency is based on the ratio of stress to modulus of rupture. If the ratio for the overlaid system is the same as that of the new pavement, both pavements will be structurally equivalent.

$$\frac{\sigma_n}{S_{cn}} \geq \frac{\sigma_e}{S_{ce}} \tag{5.12}$$

where

σ_n = critical stress edge in the new pavement

S_{cn} = modulus of rupture of the new concrete

σ_e = critical stress edge in the existing pavement

S_{ce} = modulus of rupture of the existing concrete

In developing this method, a finite element program was used to create a design chart in which the critical tensile stresses due to edge loading in both new pavement and the BCO structure are related to the modulus of rupture of the existing concrete, for which three different ranges of moduli are considered.

For the BCO design, the first step consists of calculating the thickness of the new full-depth pavement, and this can be accomplished by using the PCA design method or another PCC design method. With this thickness and the design chart, the combined thickness of the BCO and the existing pavement is computed, and the BCO thickness is determined by subtracting the existing slab thickness from this value. The maximum recommended BCO thickness is 5 in. When the required thickness exceeds this value the use of an unbonded overlay is more advisable.

In this method, the fatigue consideration is dependent on the procedure used to arrive at the new full-depth pavement thickness. If the PCA method is used, it is assumed that the pavement can withstand an infinite number of applications, as long as those occur under the stress limit established by the method, which is based on plain concrete tests.

5.4.3 RPRDS

The Rigid Pavement Rehabilitation Design System (RPRDS) is the evolutionary result of a series of methods, all of which were originated from a procedure developed for the Federal Highway Administration (FHWA) by Austin Research Engineers (Refs 38 and 39). These methods are characterized by incorporating fully automated and systematic procedures into computer programs and are applicable to both bonded and unbonded concrete resurfacings as well as AC overlays on any type of existing concrete pavement. The procedure is mechanistic-empirical in nature. The mechanistic part consists of the use of elastic layer theory to predict stresses and strains, and the empirical portion comes into play with the incorporation of fatigue relationships developed using the AASHO Road Test data to predict failure.

The first version of this procedure was developed in 1977. It was created as a result of an initiative of the FHWA in the 1970s to rehabilitate the increasingly deteriorating pavements of the interstate highway system, and as a consequence of the realization of the lack of appropriate design methods to accomplish the rehabilitation. At that time, the current design philosophy and procedures failed to consider the structural value of the existing pavement or its remaining life, and omitted a fatigue criterion, which is one of the primary failure mechanisms (Ref 38).

The RPRDS involves an evaluation of the existing pavement conditions by condition surveys and deflection testing as well as destructive testing. It recommends dividing the total project length into shorter design subsections with similar conditions and deflection profiles. A computer module determines statistically whether the selected sections are significantly different. The sections are classified according to the severity of their cracking, making use of the AASHO Road Test crack definitions (Ref 40). Depending on the sections' classification, the program determines the proper overlay design to use for each case, out of 15 possible analyses; 9 of these are applicable to pavements with remaining life, and 1 of those 9 corresponds to the BCO design on CRCP. The analytical model uses linear-elastic layer theory to characterize the subgrade material and to compute stresses and strains. The computer program RPOD1 (Rigid Pavement Overlay Design-1) is used to determine the overlay thickness design that satisfies the fatigue cracking criteria. It computes the stress in the pavement system for an 18-kip ESAL for a range of overlay thicknesses from 3 to 12 in. and calculates the corresponding fatigue life for each case. Then it matches the overlay thickness with the design life supplied by the designer as an input. After the required overlay thickness has been determined, the overlay susceptibility to reflective cracking can be analyzed with another module of the program intended mostly for AC overlays.

The fatigue equation relates the response of the pavement under load (i.e., stress or strain) and the number of load repetitions that the pavement can withstand before reaching the failure criterion, which in this case meant when the pavement first developed opened cracks or spalled cracks to a width of ¼ in. or more over a distance equal to at least one-half the crack length (i.e., Class 3 cracks, according to the AASHO Road Test crack definitions). The form of the fatigue equation is as follows.

$$N_{18} = A\left(\frac{f}{\sigma}\right)^{B} \tag{5.13}$$

where

N_{18}	=	number of load repetitions until failure
F	=	concrete flexural strength
σ	=	calculated concrete tensile stress under design load
A, B	=	regression constants obtained from analysis of fatigue data

If strains are used rather than stresses, the fatigue equation takes the following form.

$$N_{18} = A\left(\frac{1}{\varepsilon}\right)^{B} \tag{5.14}$$

where ε is the computed strain in the concrete under the design load and the other variables are the same as those above.

Regression analyses performed on the AASHO Road Test data resulted in the following fatigue equation, which had an R^2 term of 83 percent.

$$N_{18} = 23{,}440\left(\frac{f}{\sigma}\right)^{3.21} \tag{5.15}$$

The first of the several subsequent improvements to the design procedure is known as the FHWA/Texas Procedure, presented in Ref 41. It was an improvement as well as a simplification of the FHWA method and was developed for the Texas State Department of Highways and Public Transportation (SDHPT) by the Center for Highway Research (later named CTR) at The University of Texas at Austin. The outcome of these modifications was the RPOD2 computer program, which included additional AC overlay designs and simplified the deflections input by using Dynaflect (the most common deflection testing device at the time) loadings as a default.

The next revamping of the method is presented in Ref 29. The improvements were based on data collected on Texas highways after the preceding version of the procedure was implemented. The modified version includes a finite element analysis used to quantify the effect of pavement discontinuities on the tensile stresses obtained from layered theory. It also includes a new fatigue equation derived from the AASHO Road Test data on PCC pavements as well as data from statewide condition surveys in Texas, applying a new failure criterion that corresponds to a higher level of distress. According to the new criterion, failure is reached when 50 ft of cracking occurs per 1,000 sq. ft. This condition relates to a loss of the pavement load-carrying capacity and no remaining life. The ensuing fatigue equation is as follows.

$$N_{18} = 46{,}000\left(\frac{f}{\sigma}\right)^{3.0} \tag{5.16}$$

Attending to the fact that the fatigue damage occurs differently in a pavement after it has been overlaid, a second equation was developed for overlaid pavements, as an overlay reduces the stress concentration factors; for instance, an overlay may provide sealing capabilities against moisture incursion, thus preventing loss of subgrade support, and an overlay may also reduce the warping stresses due to temperature changes. This reduction in stresses translates into an increase in pavement life. The equation for overlaid pavements is as follows.

$$N_{18} = 43{,}000\left(\frac{f}{\sigma}\right)^{3.2} \tag{5.17}$$

Because the overlay improves the resistance to moisture incursion as mentioned above, this fatigue equation may also be interpreted as suitable for modeling good moisture conditions, whereas Equation 5.16 may be understood as applicable to poor moisture conditions.

These fatigue equations present the drawback of having been developed using AC overlays on PCC pavements.

The next step in the improvement of this design methodology occurred when it was incorporated into the RPRDS, which, like its predecessors, included the development of a comprehensive computer program to expedite the design process. The program is called RPRDS-1 (Ref 42). The improvements consist of the creation of cost analyses, including overlay construction costs, maintenance costs, traffic delay costs, and time value of money. The procedure analyzes a large number of feasible pavement rehabilitation alternatives on an economic basis.

The last installment in the procedure's series of enhancements came as part of a software tool developed by CTR, called BCOCAD (Bonded Concrete Overlay Computer-Aided Design), which is presented in detail in Ref 43. BCOCAD, developed for the Texas Department of Transportation (TxDOT), is a BCO design program that incorporates both the AASHTO 93 method, explained below, and BCOPRDS, the new modified version of RPRDS-1.

BCOPRDS is the modification of RPRDS-1 in the direction of BCOs only, thus disabling the previous AC overlay options, and it optimizes the original code. However, the failure criterion and the failure equations remain the same as those in RPRDS-1.

5.4.4 AASHTO

The AASHTO method, outlined in detail in the 1993 AASHTO Guide (Ref 26), is based on the AASHO Road Test, the basic design equation (5.1), the remaining life concept, and thus, fatigue, and the idea of serviceability.

It is mostly an empirical method, because the design equations for the method were derived from regression analyses performed on the Road Test data, but it includes a mechanistic part, in the determination of stresses and strains. Like the PCA method, the AASHTO procedure advocates conducting a comprehensive evaluation of the existing pavement conditions and applying the results as input design parameters for the BCO.

The thickness design equation is as follows.

$$D_{ol} = D_f - D_{eff} \tag{5.18}$$

where

D_{ol} = required thickness of BCO

D_f = slab thickness to carry future traffic

D_{eff} = effective thickness of existing slab

The slab thickness to carry future traffic, D_f, is calculated by means of the standard AASHTO rigid pavement design equation, as if it were a new pavement design:

$$\log_{10} W_{18} = Z_R x S_O + 7.35 x \log_{10}(D+1) - 0.06 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.5-1.5}\right]}{1+\frac{1.624 x 10^7}{(D+1)^{8.46}}}$$

$$+ (4.22 - 0.32 p_t) x \log_{10}\left[\frac{S_c' x C_d x (D^{0.75} - 1.132)}{215.63 x J \left[D^{0.75} - \frac{18.42}{\left(\frac{E_c}{k}\right)^{0.25}}\right]}\right] \quad (5.19)$$

where

W_{18}	=	predicted number of 18-kip ESAL applications
Z_R	=	standard normal deviate
S_O	=	overall standard deviation of rigid pavement
D	=	thickness of pavement slab, in.
ΔPSI	=	difference between initial serviceability, p_o, and terminal serviceability index, p_t
S_c'	=	PCC modulus of rupture, psi
J	=	load transfer coefficient
C_d	=	drainage coefficient
E_c	=	PCC modulus of elasticity, psi
K	=	modulus of subgrade reaction, pci

The first term ($Z_R x S_O$) corresponds to the reliability. The remaining terms on the first line of the equation are the empirical part of the procedure, derived from the data gathered at the AASHO Road Test, whereas the second line, related to stress computations, is the theoretical part, which was added to account for changes in strength and stresses owing to physical constants (e.g., E_c, k) occurring in conditions other than those that existed during the road test.

The effective thickness of the existing slab, D_{eff}, is calculated by applying a condition factor, CF, to the existing slab thickness, D, as in the following expression.

$$D_{eff} = CFxD \quad (5.20)$$

The value of CF can be determined in two ways, either by the use of remaining life or by means of the condition survey. The remaining life relationship with the condition factor appears in the following chart, from Ref 26 (Figure 5.10).

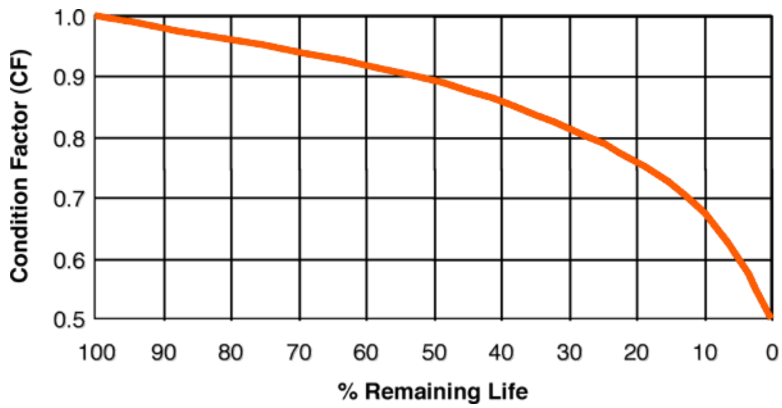


Figure 5.10 Relationship between condition factor and remaining life

The other way to determine CF is by using the three AASHTO adjustment factors, obtained from a condition survey:

a) Joints and cracks adjustment factor (Fjc): adjusts for extra loss in PSI originated by deteriorated reflection cracks that result from unrepaired cracks in the existing pavement prior to overlaying.
b) Durability adjustment factor (Fdur): adjusts for extra loss in PSI of the overlay when the existing slab has durability problems like "D" cracking or reactive aggregate distress.
c) Fatigue damage adjustment factor (Ffat): adjusts for past fatigue damage that may exist in the slab.

The factors range from 0 to 1. When the pavement condition is satisfactory, these factors take the value of one, which means that the condition of the pavement does not affect the effective thickness. However, as the condition of the slab becomes more deteriorated, their values decrease. Guidelines for selecting values for the adjustment factors appear in Ref 26. The condition factor is the combination of these adjustment factors as is shown in Equation 5.21.

$$CF = F_{jc} xF_{dur} xF_{fat} \quad (5.21)$$

Therefore, the value of D_{eff} can be expressed as:

$$D_{eff} = F_{jc} x F_{dur} x F_{fat} x D \tag{5.22}$$

As mentioned earlier, the AASHTO procedure has been incorporated into the BCOCAD computer program (Ref 43).

5.5 Reinforcement Design

The purpose of the steel reinforcement in a CRCP is to hold cracked concrete together and to maintain load transfer; it does not increase the pavement structural capacity. It is recommended that the BCO be reinforced in a fashion similar to that of the existing pavement, unless there is a significant deficiency in the original pavement reinforcement design.

Three main design parameters have to be satisfied in the design of longitudinal reinforcement of CRCPs:

a) Crack width at freezing temperature.
b) Maximum steel stress, at the minimum temperature expected to occur during the design life of the pavement.
c) Cumulative percentage of transverse cracks spaced below 3 ft.

The importance of the transverse crack spacing distribution lies in the fact that cracks spaced below 3 ft could lead to punchouts, the most detrimental distress in CRCPs (Ref 44). It is recommended to limit the maximum crack spacing to 8 ft, to minimize the incidence of spalls. When transverse cracks are spaced farther apart than that, it is likely that the stresses in the steel are high and the reinforcement may undergo permanent deformation, originating wider cracks that are prone to spalling.

Crack width affects CRCP behavior, performance, and durability. Excessive crack width is deleterious to pavement in the following manners (Ref 45):

a) Reducing the slab ability to transfer loads; an open crack will stress the steel more.
b) Extraneous materials may infiltrate into the crack opening, causing spalling and blowups.
c) Allowing water infiltration, which can reduce loss of roadbed support and can lead to steel corrosion.

A maximum crack width limit of 0.04 in. is proposed by AASHTO (Ref 26) to ensure sufficient aggregate interlock and to prevent water penetration.

The greater the amount of longitudinal steel, the narrower the crack widths generally are, because more reinforcement can hold the cracks more tightly by creating a larger bond area between steel and concrete, according to the research presented in Ref 45. Also, more longitudinal steel is related to the incidence of a greater number of cracks. The reason is that the greater quantity of longitudinal steel more effectively restrains slab movement resulting in more cracks (i.e., smaller crack spacing).

For the steel stress, a limiting value of 75 percent of the ultimate tensile strength is recommended to prevent steel permanent deformation and fracture.

Unfortunately, the interaction of those three design parameters is of such a nature that all three are seldom simultaneously satisfied within acceptable limits in a practical design situation. For instance, it is likely that a section where transverse cracks are spaced far apart will have high steel stresses and the crack widths will be large. However, with more cracks the steel stresses will be relieved and the crack widths will be much smaller. If these cracks are spaced too close (say, under a spacing of 3 ft), the likelihood of punchouts developing will be high. The design that would most likely produce the highest level of performance is one in which the best combination of the three design parameters is achieved.

The most widespread longitudinal steel design procedure for CRCP was developed at the former Center for Highway Research at The University of Texas at Austin, now CTR. The method was developed with a computerized system, known as the CRCP computer program. As a supplementary design tool, the equations for the procedure were also presented in the form of nomographs, and this work is published in Ref 46. The method, including the nomographs, was adopted by the AASHTO Guide (Ref 26) and by numerous state and national highway agencies throughout the world. In this method, regression equations were created to model the relationships between the most significant input variables and the aforementioned three key design parameters, expressing the steel design in terms of percentage of the concrete cross-sectional area. Two sets of equations were prepared, following linear and non-linear least squares fits to a set of observations generated by the computer program (CRCP-2 at that time). The improvement in fit obtained with the use of non-linear coefficients was not significant; hence, it was decided to use the equations derived from the linear fit.

The derived equations for the prediction of crack spacing, crack width, and steel stress respectively are as follows.

$$\overline{X} = \frac{1.32\left(1+\frac{f_t}{1000}\right)^{6.70}\left(1+\frac{\alpha_s}{2\alpha_c}\right)^{1.15}(1+\phi)^{2.19}}{\left(1+\frac{\sigma_w}{1000}\right)^{5.20}(1+p)^{4.60}(1+1000\ Z)^{1.79}} \tag{5.23}$$

$$\Delta X = \frac{0.00932\left(1+\frac{f_t}{1000}\right)^{6.53}(1+\phi)^{2.20}}{\left(1+\frac{\sigma_w}{1000}\right)^{4.91}(1+p)^{4.55}} \tag{5.24}$$

$$\sigma_s = \frac{47{,}300\left(1+\frac{\Delta T_F}{100}\right)^{0.425}\left(1+\frac{f_t}{1000}\right)^{4.09}}{\left(1+\frac{\sigma_w}{1000}\right)^{3.14}(1+1000\ Z)^{0.494}(1+p)^{2.74}} \tag{5.25}$$

where

$\overline{X}$	=	average crack spacing, ft
ΔX	=	crack width, in.
σ_s	=	steel stress, psi
f_t	=	concrete tensile strength, psi
α_s	=	steel thermal coefficient, in./in./°F
α_c	=	concrete thermal coefficient, in./in./°F
ϕ	=	bar diameter, in.
σ_w	=	wheel load stress, psi
p	=	percent reinforcement
Z	=	concrete shrinkage strain, in./in.
ΔT_F	=	final temperature change, °F

These equations were incorporated into the CRCP-2 program. The program, created in 1975, has been the subject of a large series of enhancements over time, with ensuing versions of it. Ongoing research has been conducted at CTR since the program inception to improve it; currently, the 10th version of the program, CRCP-10, is available and being calibrated; CRCP-11 is already being developed. Documentation on the latest versions, CRCP-9 and CRCP-10, is available in Refs 47-51.

For the BCO steel design, it is recommended to use the CRCP program. The program has the capability to predict the time history of crack spacing, crack width, steel and concrete stress for a range of material properties, environmental conditions, and layer thicknesses. The first step would be to run it with the existing pavement properties, including the existing steel percentage and past environmental conditions, to try to replicate the existing crack spacing. In the next step, the new pavement structure, with the overlay included, is analyzed with the program, varying the percentage of steel. The output of the program will include the combination of the reflective cracking of the existing pavement plus the new BCO cracking.

The transverse reinforcement can be resolved with the following expression, which relates the percentage of transverse steel, P_s, with the slab length, L (ft), the steel working stress, f_s (psi), and the friction factor, F.

$$P_s = \frac{LF}{2f_s} x100 \tag{5.26}$$

5.6 Summary and Conclusions

Pavement thickness design has been approached with different philosophies that have changed over time. The early methods were based purely on experience, but technological advances have stimulated and facilitated research that has led to the development of more mechanistic approaches. Thus, the trend is toward this kind of method. Nevertheless, the current state of the art in BCO thickness design, even though mostly mechanistic, still requires the use of empirical procedures to establish the failure criteria. According to these philosophies, there are several methods for approaching BCO thickness design. In this chapter, a historical overview of the development of the most widely used design procedures was presented, along with their general equations and input parameters.

Most of the design methods require extensive testing and evaluation of the material properties of the existing pavement as well as assessments of its condition in terms of occurrence and extension of distresses. Materials characterization is a critical aspect of the design in order to adequately assess the structural contribution of the existing pavement and to take it into consideration for the BCO thickness design. Condition surveys and deflection testing are basic tools to characterize the existing pavement.

The current BCO design procedures have conceptual differences and limitations; therefore, there is not an absolute solution to BCO thickness design. One of the main differences across design methods is the failure criterion. The Corps of Engineers procedure was developed for airports and, as such, its failure criterion, based on the appearance of structural cracking, is not applicable to highway pavements. It assumes that the existing concrete has had no fatigue damage and will behave just as new concrete does. Hence, it does not use any evaluation of the existing pavement. The AASHTO method bases its failure criterion on serviceability, whereas the PCA method is based on a stress limit established for plain concrete. The RPRDS procedure is based on a fatigue failure criterion, but the fatigue model was developed using AC overlays on PCC pavements.

From the methods presented, experience in BCO research projects conducted at CTR has shown that it is not advisable to perform the thickness design following only a single approach. Several BCO projects have been designed by following the AASHTO method in conjunction with the RPRDS procedure, such as the project presented in detail in Chapter 8. Some of the projects that were mentioned in Chapter 2, were designed following this dual-method approach, including the El Paso BCO on IH-10 (Ref 1), Beltway 8 in Houston (Ref 14), IH-610 in Houston near Memorial Park (Ref 23), IH-30 in Fort Worth (Ref 2), and the new Houston projects still to be constructed cited at the end of Chapter 2. The advantage of doing this is that both methods use different failure criteria, so the designer can choose between both designs to arrive at the final BCO thickness. The BCOCAD program (Ref 43) is capable of running both procedures at the same time with one set of inputs, thus facilitating the task.

6. Construction

The construction stage of the bonded concrete overlay (BCO) is presented in this chapter. The BCO process, introduced in Chapter 3, encompasses the stages in the development of a BCO project. Figure 6.1 shows how the construction stage is related to the other components of the BCO process.

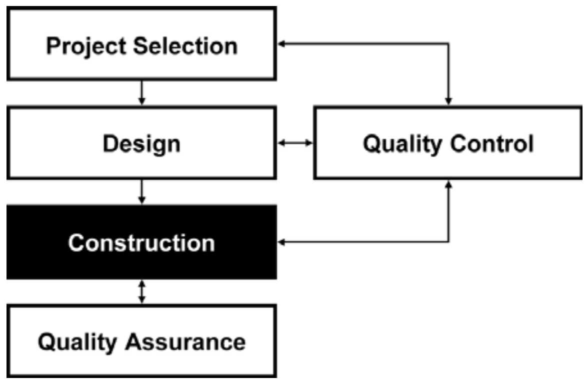


Figure 6.1 Construction as part of the BCO process

The construction of a BCO involves the materials and aggregate type selection, the surface preparation and cleaning, bonding agents, if any, precautions for adverse environmental conditions, and curing of the BCO. A section in this chapter is dedicated to expedited BCOs as a way to minimize traffic disturbances.

6.1 Overview

The objective of a BCO is to produce a cost-effective, durable means to repair a concrete pavement. This implies that the construction should be carried out at a relatively low cost, with minimum traffic disturbances for the users of the facility, and in such a way that the end result is a quality product that with normal maintenance will last in good conditions for its intended service life. To accomplish this, it is necessary to take into account that the overlay will become part of a composite system in conjunction with the existing substrate. The success of the BCO, as is shown in Figure 6.1, is dependent on a series of stages. Construction is one of those stages, which, in turn, is made up by a series of many interrelated steps, such as material selection, surface preparation, construction practices, and so forth. Construction problems manifest themselves as spalling, punchouts, delaminations, and loss of strength, all of which are signs of distresses that often have multiple causes. In most of the cases, these problems are correlated, making it difficult to identify any single underlying cause. One condition, however, is always required for an effective, long-lasting concrete repair such as a BCO: compatibility of the new overlay with the existing concrete.

Following is a definition of materials compatibility from Ref 52:

> Compatibility is the balance of physical, chemical and electrochemical properties and dimensions between repair materials and existing substrate that ensures a repair will withstand all stresses induced by volume changes, chemical and electrochemical effects without distresses and deterioration in a specified environment over a designated period of time.

When analyzing the dimensional compatibility of the materials, it is necessary to consider not only the current proportions of the materials, but also the phenomenon of volumetric changes, which affects every construction material. There are two volumetric change properties of the concrete that are independent from loading: drying shrinkage and thermal expansion. Drying shrinkage is the long-term change in the volume of concrete caused by loss of moisture to the environment after concrete setting. The volume change relative to temperature change is expressed by the coefficient of thermal expansion or contraction. Concrete, like all materials, changes volume when subjected to temperature changes. Volumetric changes of either type create stresses when the concrete is restrained, (e.g., by the substrate, by the steel, etc.). The resulting stresses may be of any type: tension, compression, shear, and so forth, and the result of it may be undesirable behavior such as cracking, spalling, deflections, and delaminations.

Another component of physical compatibility is the modulus of elasticity. Moduli incompatibility may affect the stress distribution, leading to excessive stress concentration at the interface. At early ages, it also induces thermal stresses.

An important chemical compatibility property to consider is alkali content, because of the possibility of alkali-silica reactions within the concrete materials. Many durability problems result from the reaction between the silica in the aggregates (e.g., siliceous river gravel) and alkalis contained in the cement.

Electrochemical compatibility involves considering the way the BCO materials will react to reinforcing steel and other embedded metals such as tie-bars and dowels or to protective coatings or sealers applied on the existing pavement surface as part of its repair prior to the BCO placement. Corrosion of reinforcement steel is an electrochemical process that may occur as a result of dissimilar steel surface conditions such as differences in pH, oxygen concentration, chloride concentration, moisture, and temperature.

Besides material selection, there are many other aspects that should be taken care of in the construction of a BCO. Preparing the surface for a BCO requires milling existing asphalt concrete (AC) layers if the CRCP has been overlaid, repairing the existing pavement distresses, such as shattered slabs, deteriorated and open joints, and spalled cracks, and attaining a proper surface texture that enables the bonding of the new concrete by adhesion and mechanical interlock. Finally, after placing the reinforcement steel and just before the concrete is cast, the surface is cleaned to remove debris and contaminants. For the placement of the concrete, special consideration should be given to the climatic surroundings of the construction site by monitoring the weather variables and knowing how they affect the future performance of the overlay, as the meteorological placement circumstances may be critical for the BCO. Once the concrete has been placed, curing should be performed promptly; the curing methods should be in accordance with the prevailing weather conditions.

6.2 Materials and Aggregate Compatibility

It is of paramount importance that the materials and aggregates of the BCO are compatible with those of the existing pavement, because the intent is for both layers, existing substrate and overlay, to behave as one.

6.2.1 Physical Properties

The basic premise for material compatibility is to use aggregates for the BCO concrete that produce moduli and thermal coefficients lower than those of the materials in the existing slab, which, as is shown in Refs 53 and 54, will result in lower stresses at the interface, regardless of the season of placement. The aggregates make up between 65 and 75 percent of the total concrete volume; therefore, their properties have a definite influence on those of the concrete.

6.2.2 Modulus of Elasticity

Differences in moduli between layers have a significant influence on the thermally induced stresses. The main factors affecting the modulus of concrete are water-cement ratio, aggregate type, and age (Ref 54). Low water-cement ratios are generally associated with higher modulus of elasticity values. High-modulus aggregate will result in high-modulus concrete. Concrete age has an important effect on modulus, especially when referring to early age pavements.

6.2.3 Coefficient of Thermal Expansion

Many of the same factors affecting the modulus of elasticity influence the coefficient of thermal expansion. The type of aggregate used in the mix has a significant impact on the concrete thermal expansion. Aggregates with high coefficients of thermal expansion will produce concretes with low thermal volume stability and vice versa. Normal concrete has a thermal coefficient range from four to six millionths per degree Fahrenheit. Large differences in thermal expansion coefficients between existing and new concrete result in internal stresses, which will impact the durability of the rehabilitation. The thermal coefficient, α, determines the expansion or contraction of the concrete as expressed in terms of thermal strains, ε, owing to a temperature change, ΔT, in the following equation.

$$\varepsilon = \alpha \Delta T \tag{6.1}$$

The temperature that is used as a reference to consider the temperature change is the curing temperature, which is the temperature at which the concrete reaches its final set, when it begins to take thermal stresses and experience shrinkage movements.

With these considerations in mind, it is recommended that the coarse aggregate in the BCO should have a thermal coefficient no higher than that of the coarse aggregate in the substrate. For instance, it is advisable to utilize a limestone aggregate for the BCO concrete if the existing concrete has siliceous river gravel as coarse aggregate, because of the limestone's lower thermal coefficient, but the opposite arrangement will make up for an overlay prone to delamination.

6.2.4 Dimensions

The maximum aggregate size of the BCO concrete should be one third of the overlay thickness. This will ensure a uniform distribution of the concrete constituents when placing the BCO. If the aggregate is larger than one third of the BCO thickness, segregation of the oversized aggregates is likely to occur, especially in areas where it cannot mix properly (e.g., under reinforcement bars). Proper sized aggregate will prevent voids and guarantee good aggregate interlock and bond with the substrate.

As for the reinforcement steel, its dimensions should match those of the existing concrete reinforcement. 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 mix.

6.2.5 Chemical Properties

The chemical composition of the materials used in concrete is a subject of wide variability. Because of this, the occurrence of deleterious chemical reactions such as sulfate attack and alkali-silica reaction is frequent. Furthermore, the hydration process generates internal heat, modifies the chemical composition of the cementitious material and water, and influences the internal moisture condition. When portland cement is mixed with water, it undergoes a series of chemical reactions that result in the hardening of the concrete; these reactions are exothermic(i.e., they generate heat, which consequently causes a temperature rise in the concrete). The properties of the portland cement are in part determined by the type of cement.

Type I cement is most commonly used for general construction where no special properties are needed. If a faster-than-normal strength gain is necessary, Type III cement is the one to choose. This cement has a higher C_3S content, or finer grind, which is responsible for the rapid strength development. For a BCO, it is recommended to use Type I cement, as it produces less heat from hydration and, therefore, reduces the development of thermal stresses, which may lead to tensile cracking. Type III cement may be used if the construction is intended to be an expedited BCO. Further discussion of expedited BCOs will be presented later in this chapter.

One of the most frequent forms of chemical incompatibility in concrete materials produces the phenomenon known as alkali-silica reaction. The reaction between reactive forms of silica in the aggregates and alkalis, such as potassium and sodium oxides (K_2O and Na_2O, respectively), contained in cement is the reason for the occurrence of this deleterious event, which causes many durability problems in concrete. This adverse reaction forms a hydrous alkali-silicate gel around the aggregates in question, causing expansion when the aggregate becomes exposed to moisture. High temperatures, in the range of 50 to 100 °F, accelerate the reaction. With expansion, the surface of the concrete develops map cracking, allowing more moisture to penetrate the concrete, further accelerating the reaction. Appropriate measures to prevent this type of damage to the rehabilitation are to use low-alkali cements, and to carefully choose the aggregate type to be used in the overlay. Low water-cement ratio concrete may be useful in limiting the supply of water necessary for the alkali-silica gel to swell, but it will only slow down the process, not prevent it (Ref 55).

An investigation conducted by the Center for Transportation Research (CTR) at The University of Texas at Austin on the BCO on the IH-610 North Loop in Houston (Ref 56), required the use of petrographic analysis which revealed that this type of chemical incompatibility was occurring at the interface of the BCO. In this study, the existing pavement segment of the analyzed cores showed traces of alkali-silica gel within a thin layer adjacent to the interface and it likely had an impact on the bonding of the two concrete layers.

Other manifestations of chemical incompatibility in concrete are carbonation, chloride penetration, sulfate attack, and exposure to other aggressive chemicals. However, these are related to the presence of extraneous elements that are not more likely to appear in overlays than in new concrete. In other words, the chemical attack is not a result of incompatibility inherent to the construction of the composite system formed by the BCO with the existing substrate.

6.2.6 Electrochemical Properties

The electrochemical incompatibility is mostly manifested in corrosion of the reinforcement steel. Corrosion is an electrochemical process requiring an anode, a cathode, and an electrolyte. A reinforcement steel bar provides the anode and the cathode, and moist concrete forms an acceptable electrolyte. Electrical current flows between the cathode and the anode in the presence of water and oxygen, resulting in an increase in metal volume as iron (Fe) is oxidized into $Fe(OH)_2$ and $Fe(OH)_3$. High-quality concrete and high alkalinity in the concrete will inhibit corrosion. As a result of corrosion, cracking and spalling may occur. Corrosion in the existing pavement should be repaired prior to the BCO placement.

6.3 Milling of Existing Asphalt Concrete Layers

If the pavement in question has been overlaid with AC layers, these layers should be milled prior to BCO placing and prior to surface preparation and repair of distresses. Remnants of AC will hinder the bonding of both PCC layers and are likely to trigger delaminations, because AC works as a bond-breaking layer between PCC layers. Complete milling of these layers will ensure that all surface contaminants such as oil, carbonates and acids are removed.

The most efficient method is by means of shotblasting equipment, such as the Skidabrader machine, although a milling machine, such as the Rotomill could also be used. The Skidabrader, manufactured by Humble Equipment Co. Inc. of Ruston, La., blasts and recycles a steel abrasive media (shot), producing a 6-ft wide swath in a single operation, in a dry, dust-free process that requires no cleanup. The product removed from the surface is loaded into a truck-mounted vacuum unit to be stockpiled for future use as road-building material or fill dirt.

The Rotomill (Figures 6.2a and b), of CMI Corporation of Oklahoma City, Ok, is a cold-milling machine developed in the 1970s; at that time, it was an innovative technology, incorporating mandrels with tungsten carbide teeth, that contributed significantly to drive down the surface preparation costs. It is able to provide cutting widths from 1 to 14 ft and cutting depths from 1 to 18 in. with a profiling accuracy within ± 1/8th in. of referenced grade.

Figure 6.2 Rotomill pavement profiler

Figure 6.3 Rotomill tungsten carbide-tipped mandrel

6.4 Pre-Overlay Repairs

All the major distresses present in the existing pavement should be repaired prior to the overlay placement. The main guideline to follow when performing this work is to assess whether the distress is likely to affect the performance of the overlay within a few years. If that is the case, the distress has to be fixed before the BCO is built.

A concrete repair must replace the damaged concrete and reinforcement, restore structural function, and protect itself and adjacent concrete and underlying strata from aggressive environmental elements. Areas of localized breakup are more susceptible to these elements; therefore, the repair should address any possible attack from extraneous factors.

Spalled cracks, opened cracks, delaminations, punchouts, and deteriorated patches must be repaired. Concrete contaminated with chlorides or carbonation must be removed and patched with PCC. AC patches should be removed and replaced with PCC patches, because AC prevents bonding of the PCC layers. Likewise, no AC should be used for any of the pre-overlay repairs. Most distresses require full-depth repairs, which have to be continuously reinforced with steel bars to ensure continuity between the repair and the existing pavement. This will preserve the load transfer capability of the slab, for which the bars must be properly tied or welded to the reinforcing steel in the existing concrete. Partial-depth repairs, in which the distressed concrete is removed by a combination of sawing and chipping, or by cold milling, are suitable only when the deterioration is limited to the surface of the concrete. A common method of repairing cracks to restore structural capacity and load transfer is the placement of polymer materials into the fracture plane. Liquid adhesives that solidify, such as polyester, epoxy, and acrylic are the most common (Ref 57). Working longitudinal cracks may be repaired by cross-stitching, which involves drilling holes on a 35° angle through the crack, placing No. 6 deformed reinforcing bars, and sealing it with grout. The holes are spaced at approximately 30 in., and the drilling direction is alternated so that the holes intersect the crack at mid-depth (Ref 58).

It is common practice to remove and replace the deteriorated area when the presence of structural distresses is extensive. When the distress is caused by a localized foundation weakness, it is convenient to remove and replace the weak material and to stabilize the foundation before replacing distressed slabs. When voids are detected under existing slabs, grout should be injected to stabilize the pavement.

Additionally, when corroded reinforcing steel is encountered, the degree of damage should be assessed to determine the necessary type of repair. Severe cases will require no further evaluation; a full-depth repair of the slab is prescribed, along with new reinforcing steel. However, for other cases in which corrosion is not widespread, an evaluation is required to establish whether the steel can be cleaned or whether it has to be replaced. Corroded steel is usually found in conjunction with concrete deterioration, such as delamination and spalling. Hence, loose concrete should be removed to allow for such an evaluation of the steel. If the bar has lost more than 25% of its cross section, the steel has to be replaced completely or repaired by means of a supplemental bar spliced to the affected bar or placed parallel to it (Ref 57). If the case is not severe and the steel can be cleaned, the following repair procedure must be conducted. The surface near the bar has to be removed, the perimeter cut by saw or other methods, and the concrete has to be removed to a minimum of ¾ in. under the corroded bar. This clearance will allow for complete cleaning of the steel and exposed concrete, for removal of corrosion and other bond-inhibiting materials, and for a uniform repair to be placed around the bar. This repair will provide a homogeneous electrochemical environment to ensure that corrosion will not occur again.

6.5 Surface Preparation and Cleaning

Surface preparation encompasses the operations conducted on the existing substrate to roughen its texture in such a way that enables the new concrete layer (BCO) to become bonded to it as if both strata were a single structure. Surface cleaning refers to the removal of dust and debris after the surface preparation is complete and prior to the placement of the BCO, to ensure that no foreign elements interfere with the achievement of bonding between both concrete layers.

One of the most critical facets of a BCO construction is surface preparation, because it is highly accountable for the bonding of the overlay to the existing concrete. The bond of a BCO determines to a high degree the success or failure of the rehabilitation. The bond at the interface between the BCO and the existing concrete is the subject of considerable stresses from volumetric changes, shrinkage, impact, vibration, and loading. The goal of surface preparation is to provide a rough surface that favors the bonding of the BCO to the existing pavement by facilitating the bonding mechanisms.

6.5.1 BCO Bonding Mechanisms

A BCO, being a layer subjected to shear stresses at the bond line, gets an essential contribution to its shear bond strength not only from bonding mechanisms attained by the paste adhesion, but also from aggregate mechanical interlocking, which can be accomplished to its fullest potential only by a roughened substrate surface, ergo, by good surface preparation. The tensile bond mechanism is slightly different, but it benefits as well from a rough substrate surface, given that mechanical interlock is supplemented by van der Waals forces, which occur as a result of electrical attraction at the molecular level when the BCO paste penetrates the pore structures in the substrate. Surface preparation operations must create an open pore structure in the existing pavement surface, which will provide capillary suction of the BCO paste into the existing concrete. The absorption of the cement paste into the substrate's pore structure is a critical bonding mechanism. If the pore structure is not attained by surface preparation or if it gets clogged with debris, slurry, or water, the absorption process will be hindered and bond strengths reduced. Hence, surface preparation is decisive both for the shear and tensile bonding of the overlay to the substrate. The shear and tensile bond mechanisms in a BCO are illustrated in Figure 6.3.

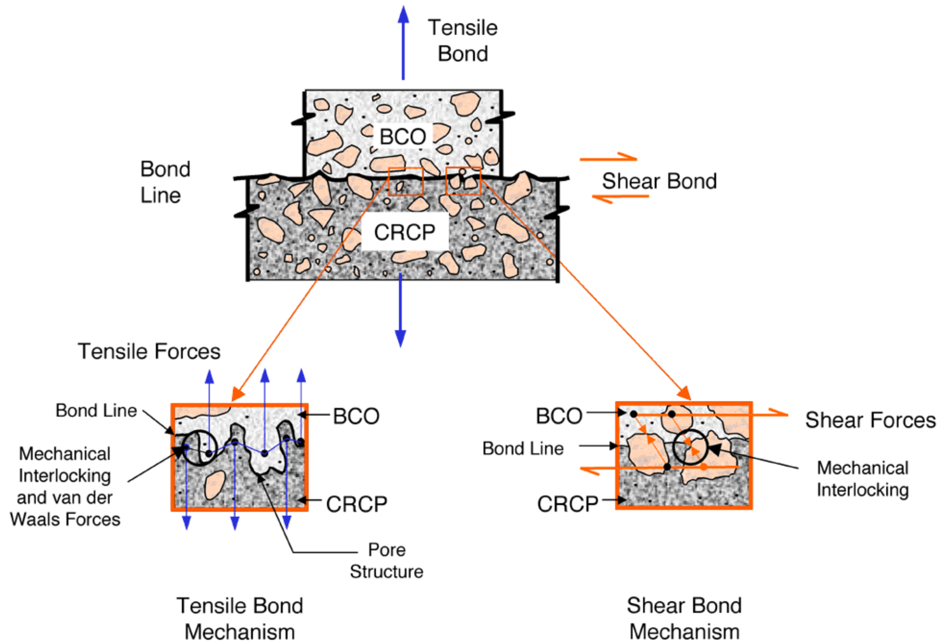


Figure 6.4 Tensile and shear bond mechanisms in a BCO

6.5.2 Surface Preparation Procedures

There are several methods to achieve an open pore structure in the substrate. The most common are as follows.

1. Shotblasting
2. Sandblasting
3. Cold milling

Shotblasting is a relatively new procedure in the paving industry. It was originally developed for the surface preparation of industrial floors. In this procedure, a spinning drum equipped with compressed air blasts tiny steel balls (shot), which impact the surface at an angle to scarify it. A vacuum collects both the shot and the dust. The shot is separated from the dust by magnetic action for continuous reuse. The blaster speed allows control of the level of scarification, wherein lower speeds yield a higher level of scarification. Shotblasting removes the matrix surrounding the coarse aggregates in a uniform way but keeps the aggregate intact. It is a clean procedure that minimizes dust and air pollution. A typical example of shotblasting equipment is the Skidabrader, mentioned in a previous section in this chapter.

Sandblasting is similar to shotblasting, but instead of shot, sand particles are used. However, unlike shotblasting, sandblasting generates airborne dust, and sand may remain on the surface after it is scarified, making it necessary to air blast the surface to remove debris prior to paving. The sandblasted surface finish is not as uniform as that executed by shotblasting operations.

Cold milling removes the top of the substrate to a specified depth by the chipping action of rotating mandrels with sharp tips mounted in a machine such as the Rotomill (Figure 6.2a and b). As a result of its action, the surface texture after cold milling is rougher and more angular than that after sandblasting or shotblasting. Cold milling is the most widespread method for large areas of concrete surface preparation requiring deep scarification. However, it generates a high amount of dust and contamination, which must be removed prior to overlaying.

Cold milling, although an efficient way of removing the grout matrix, has the drawback of fracturing the exposed aggregate, because the procedure relies on breaking the surface. The microfractures of the exposed substrate may be detrimental to its structural integrity (Ref 13), which may increase the potential for delaminations. Nevertheless, the research presented in Ref 19 exhaustively compared the interface strengths for various test procedures (direct shear, direct tension, and pull-out tension tests) for BCOs placed on Loop 610 in Houston, with and without bonding agents, utilizing light and heavy shotblasting and cold milling, and the conclusion was that similar interface shear strengths were attained in milled and blasted surfaces, although heavy shotblasting yielded the best results. The second BCO project in Iowa also investigated bond strengths in shear obtained with cold milling and sandblasting, with the latter method obtaining the higher strengths (Ref 8). Several projects in the past have achieved successful bond by means of a combination of blasting and cold milling procedures. Nowadays, however, it is not necessary to employ more than one method, because it may result in an expensive and tedious operation.

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; Ref 53 reports that the cost for milling was about twice as much as the cost of shotblasting in BCO projects in the Houston area. Judging these experiences on the advantages and disadvantages of the surface preparation procedures, shotblasting is a more recommendable alternative.

The scarification depth and texture should be specified for each project, depending on economical considerations as well as the materials properties, both of the existing pavement and the new overlay. For instance, if the substrate grout paste is relatively soft and the coarse aggregate is especially hard, a light shotblasting will be sufficiently strong to remove the paste to reach the specified depth, leaving the aggregate intact, resulting in a good surface texture. Normally, the depth of surface removal is about ¼ in. into the coarse aggregate (Ref 13). It can also be specified in terms of some standardized texture test method, such as the Sand Patch test. Typical texture readings from this test are between 0.050 in. and 0.095 in. (Ref 19). More information on the Sand Patch test will be presented in Chapter 7.

6.5.3 Surface Cleaning

After the surface preparation operations are finalized and the reinforcing steel is in place, the last cleaning of the surface is done by airblasting just before concrete placement (Figure 6.4). It should be noted that airblasting and waterblasting 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. Airblasting 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 6.5 Final cleaning of the substrate, just prior to placement

6.6 Shoulders

When a BCO is placed on the main lanes, it is generally required to upgrade the shoulders as well. The rationale for resurfacing the shoulders is twofold: it matches the shoulder grade line with that of the traffic lanes, and it may contribute to the load-carrying capacity of the pavement.

To fulfill the first purpose, the shoulder thickness must be equal to the BCO thickness, at least at the longitudinal slab-shoulder joint. This thickness can be used for the entire width of the shoulder or it can be tapered at the outside edge. Tapering the shoulder thickness is done for the sole idea of saving some money on concrete, but the savings may not justify the more difficult construction process of such a shoulder. On the other hand, a shoulder of the same thickness throughout the entire width improves the drainage conditions of the roadway by eliminating the trench and reducing differential frost heave. For these reasons, it is recommended not to perform a separate thickness design for the shoulder, but to build the shoulder using the same design utilized for the overlay throughout the entire shoulder cross section.

To attain the enhancing structural contribution mentioned as the second benefit of overlaying the shoulders, it is necessary that the shoulders be made of PCC and that the shoulders be tied to the main pavement.

PCC shoulders have been constructed next to CRCP main lanes in urban highways for many years, but they appeared in rural highways only in the mid-1960s. Nowadays, because of their contribution to the pavement performance, construction of PCC shoulders for rigid pavements is a standard practice of many agencies.

Therefore, for a BCO, which is intended to become a single structural layer with the existing substrate, it is recommended that tied PCC shoulders be placed onto the underlying shoulders.

Tied PCC shoulders provide support to the edge of the slab, where the stress concentration is critical, thereby reducing the stresses and deflections in the main slab. It is a way to provide continuity to the slab, which reduces fatigue and damage. The loads are better transferred from slab to shoulder and vice versa, so the shoulder structure is benefited as well when encroaching traffic imposes loads on it. Furthermore, the improved load transfer can reduce differential movements at the longitudinal shoulder joint, preventing the shoulder drop-off that occurs when flexible shoulders are utilized. Another advantageous characteristic of tied PCC shoulders is that shoulders can carry main lane traffic during maintenance operations.

Tie-bars are utilized to guarantee adequate load transfer across the longitudinal shoulder joint (Figure 6.5). When the shoulder is built next to an already hardened BCO, the tie-bars can be installed by drilling holes in the edge of the overlay slab; the bars are fixed in the holes by using epoxy or cement grout. The bar should be inserted into the slab and the shoulder over such a length as necessary to develop sufficient bond; a minimum recommended insertion into the slab is about 9 in. If the BCO is being built, tie-bars can be inserted into the plastic concrete near the rear of the slip form paver.

The tied longitudinal shoulder joint should be properly sealed to ensure that no surface runoff infiltrates into the pavement structure.

Figure 6.6 Tie-bar, from hardened pavement to new pavement

6.7 Clearances

One of the benefits of a BCO is that it minimizes clearance problems, because of its inherent relatively low thickness as opposed to that of unbonded concrete overlays. A BCO can be used in areas in which there are clearance problems, where other rehabilitation forms may not be applicable owing to the overall thickness of the rehabilitated structure. Nevertheless, overhead bridge clearances, tunnels, and other clearances with overhead structures such as highway signs and lights, should always be verified to ensure that the intended traffic can negotiate them and that the integrity of those objects is not compromised by traversing vehicles after the placement of an additional layer on top of the existing pavement. If that is the case, three alternatives are recommended; the decision between these options is subject to engineering judgement, according to the increase in height necessary to accommodate the traffic clearance requirements:

Specify more depth of removal for the problem area to be accomplished by the surface preparation operations, provided that the additional depth of existing pavement removed is not performed over an extensive area, and that it will not preclude the pavement structure from withstanding the loadings imposed onto it.

Reduce the BCO thickness in the problem area. This could increase the potential for a higher failure rate in those areas if the required BCO design thickness is reduced by a large amount.

Reconstruct the problem area. This implies removing the existing CRCP and constructing a new section with the specified depth to satisfy the vertical clearance. This is the most expensive of the three alternatives, but it is also the safest in terms of preserving the structural integrity of the rehabilitation procedure.

6.8 Bonding Agents

Several bonding media have been studied in both laboratory and field tests to improve the bond strength between the existing substrate and the new BCO. Felt (Ref 4) concluded that bond strengths, determined by shear tests, of 200 psi or even less may be adequate for a BCO. Ever since the completion of these highly regarded and well-known studies, the value of 200 psi as desirable bond strength has generally been embraced by designers and constructors as a guide for assessing the bond quality of an overlay.

In the past, it was a common practice to use sand-cement grout as a bonding agent, the placement of which required a labor-intensive process. Because of its thick consistency, the grout had to be spread over the existing pavement by workers with stiff brushes or brooms to achieve a thickness of approximately 1/16 in. This grout is made of equal parts of sand and portland cement, mixed with water. Because of its arduous application, sand-cement grout was replaced in many projects by cement grout, which became the preferred bonding medium. Cement grout is obtained by mixing water and portland cement. This grout is easier to mix and faster to apply; a mechanical spraying device operating a short distance ahead of the paver is utilized for this purpose. The performance of sand-cement and cement grouts is reported to be essentially the same (Ref 3).

However, more recent research projects conducted at CTR have demonstrated that, under normal placement conditions, the performance of the BCO is better if no bonding agent is utilized, as long as the surface has adequate texture and has been cleaned as to be completely dry and free of dust, white water, and other debris. Yet if the surface happens to be wet, a PCC grout will assure better bond strength (Ref 13). If a grout is used, the overlay should be placed before the grout dries; otherwise, the bond strength of the overlay may be significantly reduced, because dried grout increases the probability of delaminations (Ref 56).

Special conditions may require the use of epoxy to improve bond strength. One such condition, for instance, is when the surface texture has been treated only by a less expensive shotblasting procedure and, therefore, is not rough enough to guarantee an adequate bond.

The use of epoxy resin materials to achieve bond between a BCO and the substrate is a relatively new technique. Liquid epoxy materials have been reported to provide extremely high bonding strengths in the laboratory, higher than 5,000 psi (Ref 58).

Another measure to consider under less-than-ideal surface conditions is the use of shear connectors or "jumbo nails" to improve the bond and load transfer between the two concrete layers. The nails are installed along the pavement edges and longitudinal saw cuts —the areas more susceptible to debonding— at about 6 in. apart from the edge or joint, with spacing between nails of 15 to 30 in. A smaller nail spacing results in a higher number of cracks of smaller width. A typical layout of nail placement is shown in Figure 6.6.

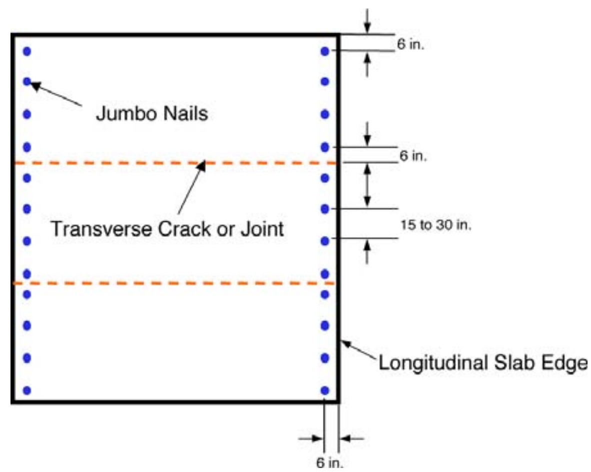


Figure 6.7 Plan view of nail placement

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 BCO when the new concrete is cast. An investigation of the performance of these devices was conducted at CTR prior to the construction of the BCO placed on IH-10 in El Paso, mentioned in Chapter 2. It included laboratory tests and a full-scale experimental BCO in El Paso, in which test sections with nails performed significantly better than those without nails in terms of early-age drying shrinkage cracking and interface bond strength. By using nails, the overlay drying shrinkage transverse cracks will be evenly distributed and their width will be smaller than those developed in a BCO without nails. The complete results of this research are reported in Ref 59.

6.9 Steel Reinforcement Position

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 substrate saves construction time and costs, because it is much easier and economical to lay it over the surface than to place it on chairs at mid-depth. An experiment was conducted at CTR (Ref 11) to determine the effect of the steel position on its bonding to the concrete. 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 6.7.

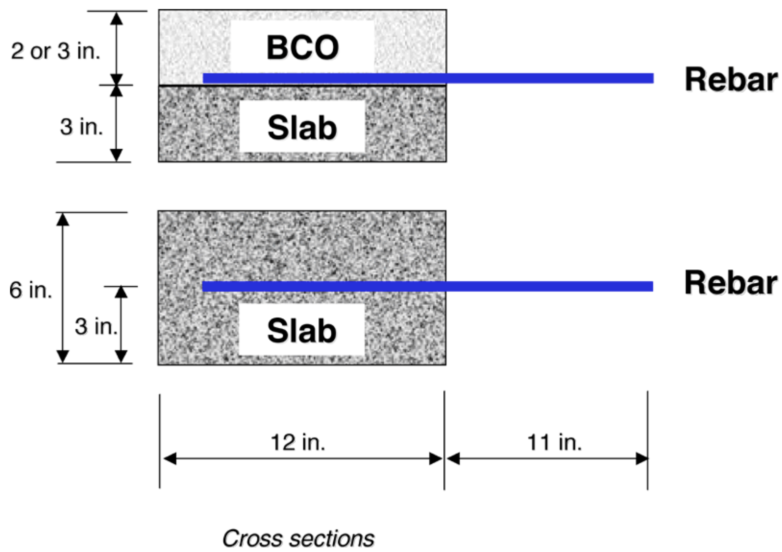


Figure 6.8 Experiment on reinforcement location

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 6.8, saving construction time, labor, and money.

Figure 6.9 Steel placed directly on top of substrate

6.10 Environmental Conditions

Weather conditions prevailing during BCO construction can be critical to the overlay performance; environmental variables that play a key role in the behavior of the overlay are temperature and moisture surrounding the concrete slab. Hot, dry climates pose the most problematic setting for BCO 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 reduces its durability.

Plastic shrinkage is the loss of water from the concrete surfaces to the air and to the underlying subbase during the stage before hydration, when the concrete is still plastic. Spaces between aggregates and paste in fresh concrete are filled with water. However, when water is removed from the concrete by evaporation or other external means, a series of menisci develop, which generate negative capillary pressure that will cause contraction of the paste volume. This contraction, in theory, could be beneficial by causing compaction of the paste, but in reality, the effect of plastic shrinkage is not uniform throughout the concrete mass, and the differential volume changes produce cracking under induced tensile stresses (Ref 55). These stresses are occasioned by restraint produced by the subbase, the reinforcing steel, and certain joints.

Plastic shrinkage cracking occurs more frequently on the horizontal surface of the slab. The shape of a pavement slab, with high ratios of length to thickness and width to thickness, contributes to this phenomenon, given that a huge proportion of the slab is exposed to the environment.

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 favors high water evaporation.

Meteorological information should be monitored at the time of construction by means of a weather station. This device measures the ambient temperature, the relative humidity, and the wind speed (the variables evaporation depends on), plus the fresh concrete temperature, at a specified frequency. The concrete temperature should be measured periodically every time a new concrete truck arrives at the construction site. This is accomplished by means of thermocouples, which are placed in a sample of fresh concrete. Although the concrete temperature is not a meteorological variable, the thermocouple readings can be attached to the weather station readings to have the time-histories of all four variables in a single file. Figure 6.9 shows a weather station at a BCO construction site. The weather station is portable; in this case it is shown mounted in a truck so it can be placed near where the concrete is cast, providing meteorological information specific to the construction site.

Figure 6.10 Weather station

The data collected with the weather station is used to estimate the water evaporation rate. If the evaporation rate surpasses 0.2 lb/sq. ft/hr, the loss of moisture from the slab surface may exceed the rate at which bleed water reaches the top of the concrete, originating negative capillary pressures which trigger plastic shrinkage. Several studies have linked delaminations of BCOs with high evaporation rates during placement (see, e.g., Refs 1, 13, 16, and 54). If this happens, there is a high probability that the delamination of the BCO from the existing surface will occur within the first 24 hours (Ref 13).

The evaporation rate can be estimated from the nomograph published by the Portland Cement Association (PCA, Ref 60), shown in Figure 6.10.

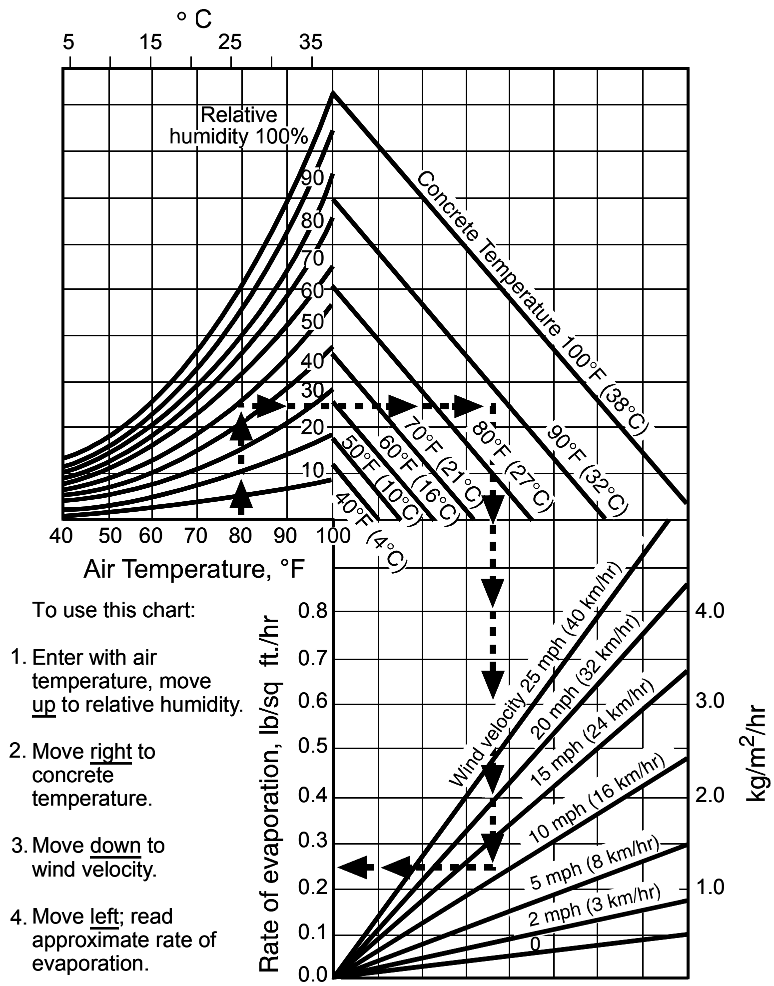


Figure 6.11 Evaporation prediction nomograph

The nomograph is derived from Equation 6.2. It should be emphasized that the rate of water evaporation from the concrete surface computed by these means (Equation 6.2 and Figure 6.10 nomograph) is only an estimation of the evaporation that occurs from a free surface of water, similar to a concrete pavement when no curing has been applied on it.

$$E = 0.0638 * \left(e_c - \frac{RH * e_o}{100} \right) * (0.253 + 0.0960 * ws) \tag{6.2}$$

where

$$e_c = 0.611 * e^{\left[17.269 * \frac{\left(\frac{t_c - 32}{1.8} \right)}{\left(\frac{t_c - 32}{1.8} \right) + 237.3} \right]}$$

$$e_o = 0.611 * e^{\left[17.269 * \frac{\left(\frac{t_a - 32}{1.8} \right)}{\left(\frac{t_a - 32}{1.8} \right) + 237.3} \right]}$$

E = Evaporation rate (lb/sq. ft/hr)
RH = Relative humidity (%)
ws = Wind speed (mph)
t_c = Concrete temperature (°F)
t_a = Ambient temperature (°F)

Should the evaporation exceed the aforementioned critical value, special curing precautions should be implemented. These will be discussed in detail in the section dedicated to curing in this chapter.

Besides evaporation, the concrete temperature should be monitored throughout construction. The influence of temperature on concrete performance has been the subject of numerous investigations. Substantial research on performance prediction for CRCP has been developed at CTR (Refs 44 and 45). The authors of these research projects found that pavements perform differently when placed at different times of the year (i.e., summer placement versus winter placement). Poor performance pavements, manifested by the outcome of their performance indicators such as crack spacing and distress occurrence, are linked to high setting temperatures during summer construction time. Thus, faulty pavement performance is, to a great extent, attributed to adverse placing conditions. Therefore, for BCO placement, the concrete temperature should be kept low, especially when the meteorological conditions are adverse, to avoid high a setting temperature and its deleterious effect on performance. The surface of the substrate immediately prior to overlay placement should be monitored as well, because a surface temperature of 125 °F or above has been found to be unacceptable for paving (Refs 13, 19).

Another environmental issue that may be detrimental for concrete placement is the temperature differential that occurs in the hours following paving. High ambient temperature differentials within 24 hours after placement may cause extensive thermal cracking; daily temperature differentials of 25 °F or above represent an inadmissible condition for paving (Refs 13, 19). Thus, ambient temperature should be monitored before and after placement. The weather forecasts should be analyzed to ensure that the paving conditions will not be harmful to the concrete.

If any of the environmental considerations discussed in this section occur during the schedule of overlay construction, paving should be avoided, unless the conditions can be improved by special curing and other artificial measures, such as cooling the aggregates, using ice for the water in the concrete mix to lower its temperature, and using fly ash to replace cement to lower the heat of hydration.

6.11 Curing

As discussed in the previous section, high evaporation rates are associated with high air temperatures, low relative humidity, and high wind speed. Excessive moisture loss because of evaporation from fresh concrete in its early age may cause loss of tensile strength near the surface of PCC pavements, which leads to spalling, and prevents the development of adequate bond strength in BCOs. To prevent bleed water losses from the surface of the concrete, curing procedures are applied shortly after the concrete placement. This ensures proper hydration of the cement and proper hardening of the concrete.

The following is a definition of curing from Ref 61:

Curing is the maintenance of satisfactory moisture content and temperature in concrete during its early stages so that desired properties may develop.

Curing can be accomplished by a variety of methods, which include the use of a curing compound, membrane curing, curing blankets, evaporative retardants and burlap. The following are definitions from Ref 61:

Curing Compound. - A liquid that can be applied as a coating to the surface of newly placed concrete to retard the loss of water, or in the case of pigmented compounds, also to reflect heat so as to provide an opportunity for the concrete to develop its properties in a favorable temperature and moisture environment.

Membrane curing. – A process that involves either liquid sealing compound (e.g., bituminous and paraffinic emulsions, coal tar cut-backs, pigmented and non-pigmented resin suspensions, or suspensions of wax and drying oil) or non-liquid protective coating (e.g., sheet plastics or "waterproof" paper), both of which types function as films to restrict evaporation of mixing water from the fresh concrete surface.

Curing blanket. – A built-up covering of sacks, mattings, hessian, straw, waterproof paper, or other suitable material placed over freshly finished concrete.

Burlap. - A coarse fabric of jute, hemp, or less commonly flax, for use as a water retaining covering in concrete surfaces; also called Hessian.

At the present, the use of curing compounds has become, by far, the most common curing procedure under ordinary environmental circumstances. Normally, a curing compound is applied by spraying it with a machine, wherewith a fast, smooth, and uniform coverage of the surface is accomplished, as is shown in Figure 6.11. Curing should be spread immediately following screeding of the surface, unless tining is called for.

Figure 6.12 Application of curing compound

Curing may be supplemented by the use of an evaporation retardant, also known as precuring or monomolecular film (MMF). Evaporation retardants, such as Master Builders Confilm or Sealtight Evapre, are compounds that form a thin MMF to reduce rapid moisture loss from the concrete surface prior to curing. Another curing method should be used after the evaporation retardant is sprayed on.

To assess the importance of curing and to evaluate different curing procedures, including the use of evaporation retardant, CTR performed an experiment at a construction site in Houston. The research study investigated curing under adverse weather conditions for concrete placement; hence, it was conducted during the summer (Ref 62).

The experiment consisted of building a few concrete slabs with concrete from the construction job, applying different curing conditions to them, and weighing them several times within the following two days to measure the amount of weight lost by them through evaporation. With this information, it was possible to determine how effectively the curing procedures prevent evaporation from the fresh concrete and to test whether the application of MMF to retard evaporation, prior to the application of the usual curing compound, is beneficial.

In this experiment, three curing conditions were evaluated:

- Application of a common curing compound.

- Use of monomolecular compound prior to the normal curing compound application. The monomolecular compound application is also known as precuring.
- No curing.

Six slabs, 16 in. by 16 in. by 5.5 in. each, were cast. The slabs were treated with different curing procedures according to Table 6.1.

Table 6.1 Curing treatment applied to specimens (Ref 62)

Slab	Precuring	Curing
1	3	3
2	3	3
3	3	3
4		3
5		3
6		

Precuring was applied to slabs 1, 2, and 3, i.e., the monomolecular compound was sprayed prior to the application of the normal curing compound. The MMF was applied immediately after the initial weighing of the slabs, and the normal curing compound was applied after the sheen had disappeared. Figure 6.12 shows the six slabs after the curing was applied. Slab 6 is easily distinguishable, in the center of the picture, as the only specimen without any curing, by its gray surface, unlike the bright reflective surface of the other slabs, characteristic of the curing compound.

Figure 6.13 Specimens after curing (Ref 62)

The slabs were weighed at three different times on the day the experiment started, three times the next day, and one more time two days after, for a total of seven observations.

One of the slabs precured with the MMF had the best performance regarding moisture lost to the environment throughout the curing process. However, two other slabs that were precured showed no significant differences when compared with the two specimens that were cured with the normal curing compound only. Therefore, the precuring compound is effective on occasions in reducing the evaporation from the surface of fresh concrete, but it cannot be considered absolutely necessary, as the normal curing compound provided similar results.

As expected, the slab with no curing exhibited the greater water losses. This confirms that the utilization of a curing process applied appropriately and in a timely manner is essential in preserving the moisture contents of the concrete during its early stages.

Another study conducted by CTR in El Paso (Ref 63), investigated the moisture losses in pavements with various curing conditions and their effect on in situ strength. One of the curing conditions evaluated was the use of MMF. The conclusion regarding the use of MMF is summarized in the following paragraph, quoted from Ref 63:

> Normalized tensile strength comparisons between slabs where curing compound is applied at sheen loss both with and without MMF showed that elimination of MMF does not significantly affect strength. Results vary between ages but not to a large degree.

In terms of water losses, the MMF did not appear to help the concrete retain more water.

When the climatic conditions exceed the hazardous boundaries for paving specified in the previous section, special curing must be applied. Recommended special curing procedures are summarized in Table 6.2. The worst environmental circumstances demand the utilization of more aggressive and effective curing procedures.

Table 6.2 Recommended curing for bonded concrete overlays

Condition	Recommendation
Evaporation below 0.1 lb/sq. ft/hr	Membrane curing
Evaporation above 0.1 lb/sq. ft/hr but below 0.2 lb/sq. ft/hr	Membrane curing, plus evaporation retardant or fogging or wet mats, in place for 12 hours
Evaporation over 0.2 lb/sq. ft/hr	Membrane curing, plus wet mat curing or fogging or other methods approved by the engineer, in place 36 hours
Temperature drop in next 24 hours less than 25°F below temperature at time of paving	Membrane curing
Temperature drop in next 24 hours more than 25°F below temperature at time of paving	Membrane curing plus wet mats for 36 hours, or other methods as approved by the engineer

6.12 Expedited construction

If the construction duration of a BCO is critical, several procedures may be implemented to minimize it; this is called an expedited BCO.

Duration of construction is critical mostly in urban areas or on highways with heavy traffic. Because of their importance, these highways should not experience major traffic disturbances, but construction generally represents prolonged lane closures, delays, detours, and severe inconveniences for the users of the facility, with its consequential elevated user costs. However, a BCO is inherently a quick construction process, because it requires only a limited number of operations. Many times, a BCO is selected over other rehabilitation alternatives for this advantage. An expedited BCO takes this concept further; by utilizing special materials, the road can be opened to traffic in a minimal time after placement. With older paving methods, concrete pavements had to be cured for 5 to 14 days before the highway could be opened for traffic (Ref 21). On the other hand, an expedited BCO can be opened within 6 to 24 hours after placement (Ref 26). Fast-curing concrete has been utilized in the paving industry since the 1980s, originating expedited paving. Besides the obvious advantage of minimizing users' delay and inconvenience, expedited paving techniques also reduce the overall cost of the project.

To make this possible, the BCO is normally constructed with a high-early-strength PCC mix, which is attained by using Type III cement, as opposed to the normal cement (Type I). As was noted earlier in this chapter, Type III cement produces a higher heat of hydration than does normal cement. As such, when the concrete has gained some hardening, it may originate thermal stresses if the rate of heat generation is greater than the rate of heat dissipation. These stresses can lead to tensile cracking.

In general, if Type III cement is not utilized, the mix is supplemented by the use of superplasticizers.

> Superplasticizer. – A water reducing admixture capable of producing large water reduction or great flowability without causing undue set retardation in concrete (water reducing admixture). The effect is due to factors other than air entrainment (Ref 61).

For expedited BCO construction, the water-cement ratio should not exceed 0.35 (Ref 1). Another type of admixture that may be added is an air entrainment agent to increase workability:

> Air-Entraining Agent. – An addition for hydraulic cement or an admixture for concrete or mortar which causes air, usually in small quantity, to be incorporated in the form of minute bubbles in the concrete or mortar during mixing, usually to increase its workability and frost resistance (Ref 61).

In terms of strength, it is recommended that the BCO fulfill certain requirements before it is opened to traffic. The following three criteria are established in Ref 1:

- Splitting tensile strength of at least 500 psi,
- Compressive strength of at least 3,500 psi,
- Bond strength of 175 psi obtained from pull-off tests, or of 350 psi obtained from guillotine tests.

It is recommended to choose one of these criteria, preferably splitting tensile strength to determine whether the BCO has attained enough strength to be ready for traffic.

6.13 Summary

In this chapter, the most critical aspects for a successful BCO construction are addressed. The following is a list of recommendations on different aspects of BCO construction that summarizes the contents of this chapter:

a) The materials selected for a BCO should be compatible with those of the existing pavement.

b) Coarse aggregates in the BCO should have a thermal coefficient lower than or, at most, equal to that of the coarse aggregates of the substrate.
c) The maximum aggregate size should be one third of the BCO thickness to avoid segregation, prevent voids, and ensure proper aggregate interlock and bond with the substrate.
d) All AC layers and patches from the existing CRCP must be removed because they will break the bond between the substrate and the new BCO and are a likely cause of delaminations.
e) All major distresses in the existing pavement should be repaired prior to BCO placement. PCC should be used for patches, and for partial- and full-depth repairs.
f) Repairs should be continuously reinforced to ensure continuity between the repair and the existing pavement and to preserve load transfer capabilities.
g) Voids detected under existing slabs should be stabilized with grout prior to the replacement or repair of existing damaged slabs.
h) Corrosion problems may require full-depth repairs with new steel. Other minor corrosion cases may be fixed by localized concrete removal and steel replacement (if the bar has lost more than 25 percent of its cross section), or by cleaning all corrosion from the steel and surrounding the steel with new concrete.
i) The preferred method of surface preparation is shotblasting. Cold milling delivers good surface texture, but it may cause microcracking in the substrate.
j) All dust and debris should be eliminated from the substrate surface by airblasting just prior to placing the BCO.
k) The BCO shoulders should be made of PCC, tied to the main lane, with tie-bars to transfer loads. Tied PCC shoulders provide support to the slab edge, where the stress concentration is critical.
l) If placing a BCO causes localized overhead clearance problems, three action paths could be followed: 1) the depth of removal achieved by surface preparation procedures can be increased, or 2) the BCO thickness can be slightly reduced in a particular area, or 3) the problem area can be reconstructed. The first two alternatives represent a compromise in terms of the total necessary thickness specified by design and, thus, a structural capacity reduction, whereas the third option represents more labor and cost, but may be the best structural solution.
m) No bonding agents should be utilized between the substrate and the BCO under normal conditions. Special circumstances may warrant the use of epoxy to improve bond strength and/or shear connectors.
n) Steel reinforcement can be placed directly on top of the substrate instead of at mid-depth of the BCO with no diminishment in performance.

o) Placement of concrete should be avoided if the following environmental conditions occur:

    1) Water evaporation rate exceeds 0.2 lb/sq. ft/hr
    2) Substrate temperature of 125 °F or higher
    3) Daily temperature differentials of 25 °F or higher expected for the 24-hr period following concrete placement

p) If any of the above circumstances arises, special curing can be applied to proceed with the BCO placement.
q) Regardless of the weather conditions, curing should be applied immediately after screeding. Delaying this operation may be harmful to the BCO and could result in crack spalling and delaminations.
r) Type III cement may be used for expedited BCO construction; otherwise, a superplasticizer may be added to the mix.
s) Air-entraining admixtures may be added to the PCC mix as well.
t) An expedited BCO should attain a pre-established strength requirement before being opened to traffic.

7. Quality Control and Quality Assurance

Quality Control and Quality Assurance (QC/QA) in the bonded concrete overlay (BCO) process refer to the series of quality measures, practices, and tests conducted throughout the BCO development to ensure that the overlay has the strength, bonding, and overall adequacy to perform its intended objective.

As introduced in Chapter 3, Figure 7.1 contains a chart with the BCO process showing where QC and QA fit within the context of the process and their interactions with each of the other stages.

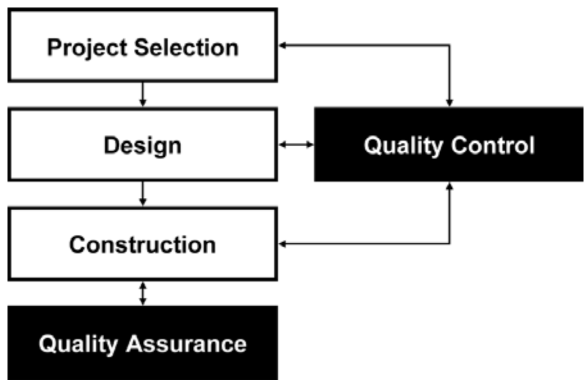


Figure 7.1 QC/QA as part of the BCO process

As the figure illustrates, Quality Control (QC) includes practices, procedures, and tests in the first three stages of the BCO process, and Quality Assurance (QA) refers to the tests conducted during construction, and after the construction has been finalized, a process that is also known as monitoring.

7.1 Introduction

The following definitions from Ref 61 clarify what the concepts of QC and QA encompass.

> Quality Control. – Actions taken by a producer or contractor to provide control over what is being done and what is being provided so that the applicable standards of good practice for the work are followed.
>
> Quality Assurance. – Actions taken by an owner or his representative to provide assurance that what is being done and what is being provided are in accordance with the applicable standards of good practice for the work.

Both QC and QA involve monitoring, but for different purposes, as will be explained below. As the definitions illustrate, a fundamental difference between these two concepts is who is in charge of performing the quality actions and practices, and of conducting tests. QC is performed by the contractor, to ensure that his product satisfies quality standards, whereas QA is the verification by the owner –or the owner's agent– of the adequacy of the contractor's work in accordance to quality standards. Both are forms of what is known as "supervision," which is done from within in the case of QC and by an outside agency or the owner himself in the case of QA.

Everyone has a notion of what quality means, but it entails different features according to the subject in question and context. Quality is a characteristic of something that satisfies the needs of the customer or conforms to the standards and specifications established for it (Ref 25). When referring to pavements, quality may imply different attributes for different groups of people, depending on whether it is seen from the standpoint of the public, the owner, or the contractor. For the user, a quality pavement is uniform, durable, and safe. From the owner's perspective, it is cost-effectiveness that defines quality, and from the contractor's standpoint, a quality pavement meets the standards in the most economical way.

QC is a continuous practice conducted throughout the BCO process until the construction is finalized. Besides requiring good, sound practices for project selection, design, and construction, both QC and QA involve sampling and testing of the overlay to measure its conformance to standards that ensure that its performance is adequate. When the BCO is in place, the QA tests monitor the performance of the overlay.

QC and QA are geared toward meeting a standard of quality, as is indicated in the aforementioned definitions, and toward verifying it in an objective way. In the case of pavements and BCOs, the standard of quality is expressed as a specification.

7.2 Specifications

A specification is an agreement, wherewith a buyer and a supplier establish a description of a product to be delivered. Specifications have always existed as part of the buyer-supplier relationship, ranging from simple verbal agreements to written legal contracts. In the case of construction, the owner of the facility is the buyer and the contractor building it is the supplier, and their relationship is bound by a specification. For them, the specification constitutes a legal contractual document.

The specification can be viewed as the rules set by the owner describing what needs to be done or accomplished by the contractor. A specification can also be viewed as an insurance policy for both parties. For an owner, there is the risk of accepting a product that may be defective and may not perform its intended purpose over the course of its lifetime. The specification is aimed at minimizing this risk. For a contractor, the specification ensures that nothing beyond what it establishes shall be demanded from the contractor's work.

Traditional construction specifications contained mostly methods that the owners wanted the contractors to use, as well as instructions to the contractor. However, as the knowledge about pavement behavior and performance has increased, the specifications have become more oriented toward establishing only an end-result, judged on the performance of the pavement, leaving the choice of method and the way to implement it in the contractor's hands (Ref 64). This is what characterizes and distinguishes method specifications from end-result and performance-oriented specifications.

The current trend of agencies and industry for highway projects is toward changing the specification philosophy, from method to performance-oriented, because it is an advantageous move for the construction industry; it requires less owner's inspection and promotes engineering innovation and creativity on the contractor's side. Performance-oriented specifications control those characteristics that contribute to successful performance of the finished product. But the performance of a pavement or overlay cannot be gauged immediately after it has been built; the way to assess it is by means of performance predictors. A great deal of the successful implementation of this kind of specification is based on the reliability of pavement performance predictors.

Specifications, of any type, set the standards and requirements by which quality is measured.

7.3 Quality Control

As mentioned before, it is the producer or the contractor who is in charge of QC. Whoever is in charge of the project selection, design, and construction is the agent responsible for the QC. In the past, the idea of QC has been restricted to the construction phase, because this stage of the project is normally carried out by a third party, the contractor, who has been selected from a group of bidders as the one that offered the most economic pricing for the construction, and hence, the contractor seeks every opportunity to restrict costs incurred in construction and to maintain a profitable stance, which may hamper quality. Also, construction is the most visible stage of the project. Nevertheless, QC should be present from the inception of the project. Unfortunately, this is not always the case. It is not rare that the project selection and design stages are poorly defined, intangible, and ill conceived, unstructured, and spontaneous. Needless to say, under these circumstances, QC is likely to be neglected.

Quality is achieved by individuals performing work functions carefully and in conformance with the requirements. As such, QC is the responsibility of these individuals. These functions include planning, coordinating, developing, checking, reviewing, testing and scheduling the work (Ref 65). The QC tasks and responsibilities, broken down by stage of the BCO process, are assigned as follows:

- Project Selection: the owner of the highway to be overlaid has the primary responsibility for the implementation of QC practices. Of course, planners and engineers participating in the decision should apply QC actions themselves.
- Design: the engineer or team of engineers designing the BCO is in charge of making sure that QC is applied.
- Construction: the contractor or in-house construction force is responsible for QC.

It is during the construction phase where the testing is conducted. However, QC tests are to be performed at the discretion of the contractor.

Regardless of the amount and the strictness of the testing conducted, quality starts with sound practices, whichever the stage the BCO process is at.

7.3.1 Sound Practices

In order to meet specifications, regardless of their type and stringency, sound practices applied during the planning, design, and construction are required in the BCO process. Independently of the specifications, only good practices throughout the process will guarantee the accomplishment of a quality BCO that fulfills its intended purpose.

Sound practices are aimed at preventing the appearance of distresses in the BCO throughout its entire design life. A quality BCO is discernable by its lack of distresses. To be able to produce it, the individuals participating in the BCO process need to know what causes distresses and how these develop in the BCO. The following distress classification, presented in Tables 7.1, 7.2 and 7.3, organizes distresses according to the time in the life of the BCO at which they occur; these tables have been adapted from Ref 64 to include BCO distresses. Table 7.1 shows a list of the most common distresses that manifest during construction and shortly thereafter in a BCO and their possible causes.

Table 7.1 Primary distress types and causes

Distress Type	Cause
Delamination	• Inadequate surface preparation • Inadequate surface cleaning • Excessive evaporation at placement • Excessive daily temperature differential at placement • Excessive substrate temperature at placement • Inadequate mix proportion design • Inadequate curing • Poor bonding agent selection or application
Excessive Transverse Cracking	• Excessive drying shrinkage due to poor curing and large temperature differential during first 72 hours • High thermal coefficient coarse aggregates • Inadequate or late joint formation (spacing, sawing, dowel bar alignment)
Longitudinal Cracking	• Excessive drying shrinkage due to poor curing and large temperature differential during first 72 hours • High thermal coefficient coarse aggregates • Inadequate or late joint formation (spacing, sawing, tie-bar design)
Spalling	• Excessive shrinkage due to defective curing • High thermal coefficient coarse aggregates • Early or improper joint sawing • Excessive crack widths • Dowel bar misalignment • Reinforcement too close to surface
Plastic Shrinkage Cracking	• Excessive shrinkage due to high evaporation rate and defective curing techniques/compounds
Scaling	• Improper finishing techniques
Ponding	• Improper cross slope and poor surface finish
Low Ride Quality	• Excessive initial roughness due to paving operations
Low Surface Friction	• Inadequate initial surface texture • Soft fines indicated by high acid insolubles • Aggregate abrasion

In a similar fashion, Table 7.2 shows secondary distresses, i.e., those that appear during the service life of the overlay, and the factors that cause the distresses.

Table 7.2 Secondary distress types and causes

Distress Type	Cause
Delamination	• Fatigue of concrete from wheel loadings • Environmental loading
Transverse Cracking	• Fatigue of concrete from wheel loadings • Reflection cracking from substrate
Longitudinal Cracking	• Fatigue • Reflection cracking from substrate • Inadequate shoulder design • Loss of load transfer
Spalling	• Delamination extension after repeated loading • Reactive aggregate • Inadequate air entrainment in mix • Wide crack openings • Infiltration of extraneous elements in cracks or joints
Corner Break	• Fatigue • Improper substrate repair prior to overlaying • Loss of substrate support • Loss of load transfer
Joint Sealant Failure	• Sealant aging • Improper joint design and material selection
Low Surface Friction	• Continued polishing owing to load applications

Likewise, Table 7.3 lists distresses that manifest after secondary distresses have occurred, i.e., tertiary or severe distresses and their causes.

Table 7.3 Tertiary or severe distresses types and causes

Distress Type	Cause
Pumping	• Joint sealant failure • Water intrusion to substrate • Substrate erosion from loads and presence of water • Punchouts increasing deflection, i.e., pumping action
Faulting	• Inadequate load transfer • Pumping action resulting in loss of support
Slab/Lane Separation	• Inadequate reinforcement, tie bars • Steel corrosion • Infiltration of extraneous elements in cracks or joints
Punchouts	• Loss of load transfer • Small transverse crack spacing • Longitudinal cracking between transverse cracks resulting in small blocks of concrete • Delaminations • Loss of substrate support • Loss of adequate bond development distance
Low Ride Quality	• Increase in roughness owing to cumulative effect of other distress types

The recommendations presented at the end of Chapter 6 constitute a set of guidelines for QC in the construction stage, which will satisfy its ultimate objective: to hinder the occurrence of distresses in the BCO.

7.4 Quality Assurance

QA is the judgment or supervision by the owner or an agent over how the BCO is constructed. This phase is tied to the construction stage of the BCO process. Evidently, an overlay project is of such a magnitude that the quality of its construction cannot be judged by a single measure once the BCO is finalized. Furthermore, the construction entails an assortment of activities, oftentimes performed by different contractors, at different stages, making it conducive to evaluate the quality of each activity separately as the construction progresses. This enables the owner to properly reward the contractor who conscientiously observes good quality practices, and, at the same time, it allows the highway agency to make up for economic loss when the lack of good quality practices results in a sub-par product. For this to be possible, the specification must identify quality characteristics or measures.

7.4.1 Quality Measures

A quality measure is a variable that defines the overlay's design characteristics. A quality measure should be a reliable predictor of pavement performance. Quality measures are selected specifically for each project, and, therefore, specifications are specific for any particular project.

The most important properties of a quality measure are as follows (Ref 66):

Performance-related: the quality measure has to be strongly correlated with the performance of the constructed product.

Consistency: a consistent quality measure always increases (or decreases) as the level of performance increases.

Effectiveness: the quality measure is sufficiently sensitive as to detect differences in expected performance.

Examples of quality measures are: BCO bond strength, BCO thickness, concrete elastic modulus, and concrete strength. The next step is deciding how much testing is necessary for each of the quality measures.

7.4.2 Sampling

A BCO is a continuous product, constructed in a continuous way. It is constituted of many components: cement, aggregates, steel, substrate, and so forth. Pavement quality testing has to rely on testing of small samples of each of those components, wherefrom calculated statistics shall represent the population properties, which in turn, are utilized as acceptance quality criteria. For this rationale to be valid, the samples need to be independent, which is achieved by random sampling. The usage of stratified random sampling, instead of just random sampling, represents a step further in the refinement of sampling methods. Stratified random sampling ensures that the samples will not be concentrated in a single area or belong to a single construction time. To apply stratified random sampling, it is necessary to introduce the concepts of lot and sublot.

7.4.3 Lots and Sublots

A lot is a discrete quantity of constructed pavement to which an acceptance quality procedure is applied. A lot is generally equal to one day's production or less. Payments to the contractor are often on a lot basis (Ref 67). In special cases, a lot can be determined by events such as starts and stops in construction operations, or by changes in equipment, materials, or personnel (Ref 64).

A sublot is a portion of a lot. A lot is divided into sublots of approximately equal size. In stratified random sampling, at least one random sample should be taken for testing from each identified sublot, with each member of the population having an equal probability of being sampled, which makes the procedure statistically valid (Ref 67). Typically, the number of sublots per lot varies from three to five. This allows for sublots to ensure the presence of a normal distribution for the sample means (Ref 64).

The recommended sample size for QA criteria is at least four stratified random samples from a lot size of a single production day (Ref 64). QC sample sizes depend on economic and operational policies and considerations. For instance, if a contractor is implementing a new procedure, the sampling conducted for the new procedure might be more frequent than the sampling for a well-tested and known procedure for which the contractor has more confidence.

The concept of stratified random sampling in highway construction is illustrated in Figure 7.2, taken from Ref 64, in which the samples are taken at random locations within the sublot, and at least one sample is taken from each sublot. This concept can be applied to both destructive and non-destructive testing (NDT).

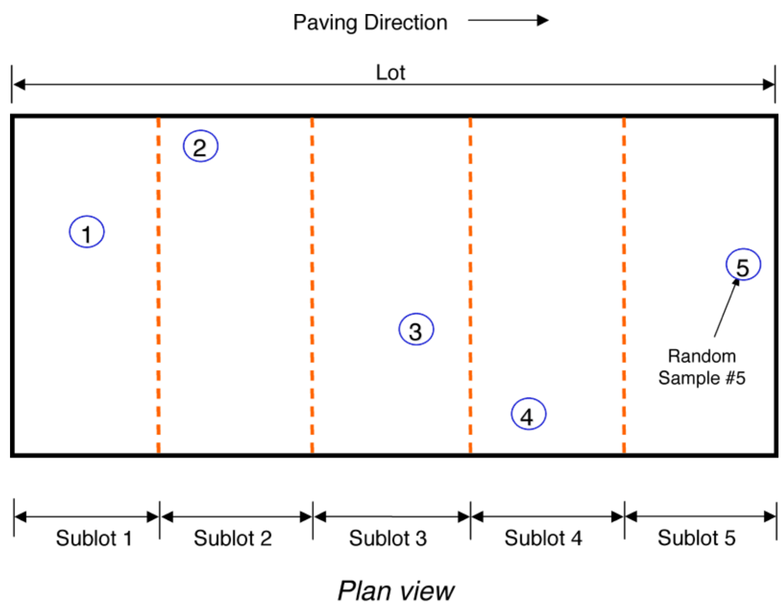


Figure 7.2 Stratified random sampling concept in pavement construction (Ref 64)

7.4.4 As-Designed Target Values and Variability

The design of the BCO specifies what the design values for the quality measures should be. However, for quality assurance purposes, the project's specifications cannot be based solely on a single design value for each quality measure, because variability plays an important role.

Variability occurs in construction materials, in construction equipment and its calibration, and in human operation. Conversely, if variability of the construction is to be judged by testing, it has to be acknowledged that variability is an inherent characteristic of the tests as well; its sources are the testing procedures, sampling methods, testing equipment, testing personnel, and operation. The occurrence of variability has to be taken into account in the specifications. Stringent specifications are characterized by allowing a small amount of variability around the target value.

The data collected in the testing procedures are described by their statistics. The target value for the mean of the samples tested should be the design value for that quality measure. The quality measure is a random variable, X, for which central tendency and dispersion measures are calculated and are used to determine acceptance or rejection in accordance with the quality standards established in the project specifications. Those statistics are estimates for the properties of the population; Equations 7.1 and 7.2 calculate the sample mean, $\overline{X}$, and variance, s_X^2, estimates of the population mean, μ_X, and variance, σ_X^2, respectively.

$$\overline{X} = \frac{\sum_{i=1}^{n} X_i}{n} \tag{7.1}$$

$$s_x^2 = \sum_{i=1}^{n} \frac{\left(X_i - \overline{X}\right)}{n-1} \tag{7.2}$$

Another useful measure of the dispersion of a random variable is the coefficient of variance, CV:

$$CV = \frac{\sqrt{s_x{}^2}}{\overline{X}} \tag{7.3}$$

Quality measures, as random variables, are assumed to be normally distributed. To make comparisons between test results and design target values, it is necessary to test the statistical hypothesis that the mean and variance obtained from the tests correspond to the population; the population of each of the quality measures is described by its mean and variance target values established in the project specifications. The statistical hypothesis testing is accomplished by the t test and the F test. These tests will provide basis for acceptance or rejection of the contractor's tests results. The target values established in the project specifications define the quality levels for which the owner or agency is willing to pay 100 percent of the bid price.

Table 7.4 presents a list of some common PCCP test parameters with their respective coefficients of variance. These values have been measured in actual paving projects and are published in Ref 67. The values shown herein are provided as an indication and a guide of what has been achieved in the past in terms of variability.

Table 7.4 Coefficients of variance for some test variables (quality measures, Ref 67)

Test Variable	Coefficient of Variance
Initial Serviceability Index	6.7%
Thickness	4%
Concrete Modulus of Rupture	10%
Load Transfer Factor	5%
Drainage Coefficient	10%
Concrete Elastic Modulus	10%
Modulus of Subgrade Reaction	35%

7.5 Testing

The most common types of tests conducted on BCOs, both for QC and QA purposes, are as follows:

- Wet mix tests
- Weather monitoring
- Condition surveys
- Deflection testing
- Concrete strength
- Concrete modulus of elasticity
- Concrete thermal expansion
- Overlay thickness
- Delamination detection
- Bonding strength
- Texture evaluation

In the following pages, some innovative testing techniques and devices, as well as some that are not very widespread as part of the BCO testing procedures, are described in more detail.

7.5.1 Wet Mix Tests

These tests are conducted soon after a concrete batch arrives at the construction site. Performing these tests is a common practice everywhere concrete is used. Slump, air content, and unit weight are measured according to test methods ASTM C 143 (Ref 68), ASTM C 231 (Ref 69), and ASTM C138 (Ref 70), respectively. The slump test is illustrated in Figure 7.3.

Figure 7.3 Slump test

7.5.2 Weather Monitoring

A detailed description of the purpose and implementation of the monitoring of meteorological conditions during the BCO construction was presented in Chapter 6. To summarize, weather monitoring should detect adverse climatic conditions for paving, which are as follows:

Water evaporation rate in excess of 0.2 lb/sq. ft/hr
Substrate temperature of 125 °F or higher
Daily temperature differentials of 25 °F or higher expected for the 24-hr period following concrete placement

If any of these situations occur, placement of concrete should be avoided, unless special precautions, such as those discussed in Chapter 6, are implemented.

7.5.3 Condition Surveys

A discussion of condition surveys for the design phase was presented in Chapter 5. The same distress types as those surveyed for design are surveyed for QC/QA purposes. Likewise, the procedures and formats utilized for the design stage surveys apply for QC/QA.

7.5.4 Deflection Testing

A detailed description of deflection testing was presented in Chapter 5, as an activity to be performed for design of the BCO. The same principles and equipment as those introduced for the design phase are utilized for QC/QA testing. In Chapter 5, a description of the most common deflection testing device, the Falling Weight Deflectometer (FWD) was presented, along with the procedures for modulus backcalculation and load transfer evaluation, which are also implemented as part of the QC/QA testing.

Repeated deflection tests should be conducted at approximately the same locations to enable the assessment of the BCO condition over time. Periodical deflection evaluation of the BCO constitutes a fundamental part of the overlay continuous monitoring.

In Chapter 5, it was also mentioned that the Rolling Dynamic Deflectometer (RDD), a relatively new type of equipment, might be of valuable use in certain projects, given that this device is capable of recording deflections in a semi-continuous way, as opposed to the conventional FWD discrete deflection measurement. This may be especially useful if there is certainty that delaminations are present; this device may be used to determine delamination locations. A description of the RDD follows, with information and schematics from Ref 71.

7.5.5 Rolling Dynamic Deflectometer

The RDD is a truck-mounted device that measures semi-continuous deflection profiles of pavements, sampling approximately every 3 ft. It gives much more comprehensive deflection data than devices currently in use, such as the FWD and the Dynaflect. Semi-continuous deflection profiles provide more ways of assessing the in-place structural adequacy of pavements.

The RDD, developed by Dr. Kenneth Stokoe at The University of Texas at Austin, consists of a vibroseis truck, which is typically used as a wave source for exploration geophysics, modified to apply dynamic loads through a pair of loading wheels. Mounted on the truck is a servo-hydraulic vibrator with a 7,500-lb reaction mass, which is driven hydraulically to generate vertical dynamic forces as large as 70,000 lb peak-to-peak over a frequency range of about 5 to 100 Hz. Figure 7.4 shows a picture of the RDD. Figures 7.5 and 7.6 show schematics of the RDD configuration and a cross-sectional view of the RDD loading and measurement apparatus, respectively.

Figure 7.4 RDD truck

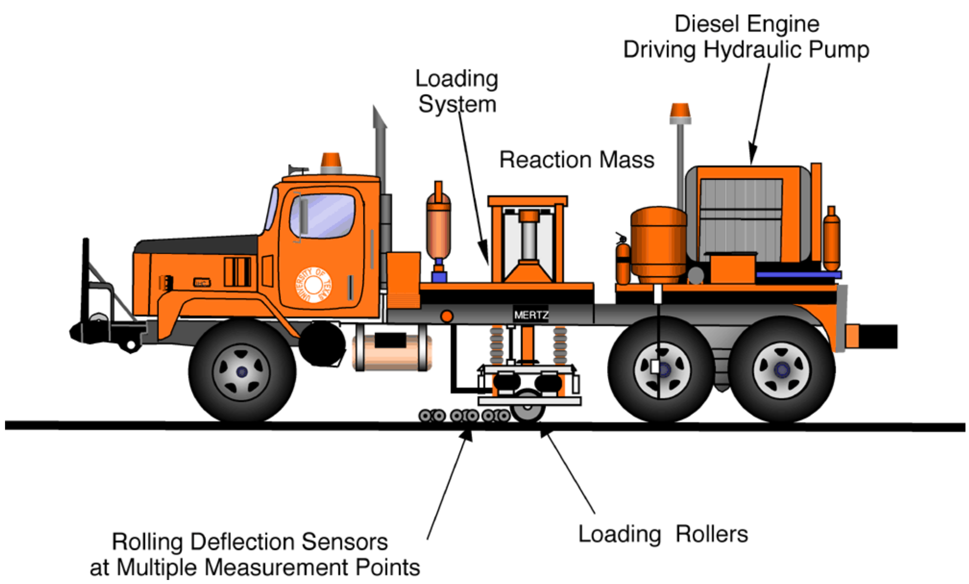


Figure 7.5 Configuration of the RDD

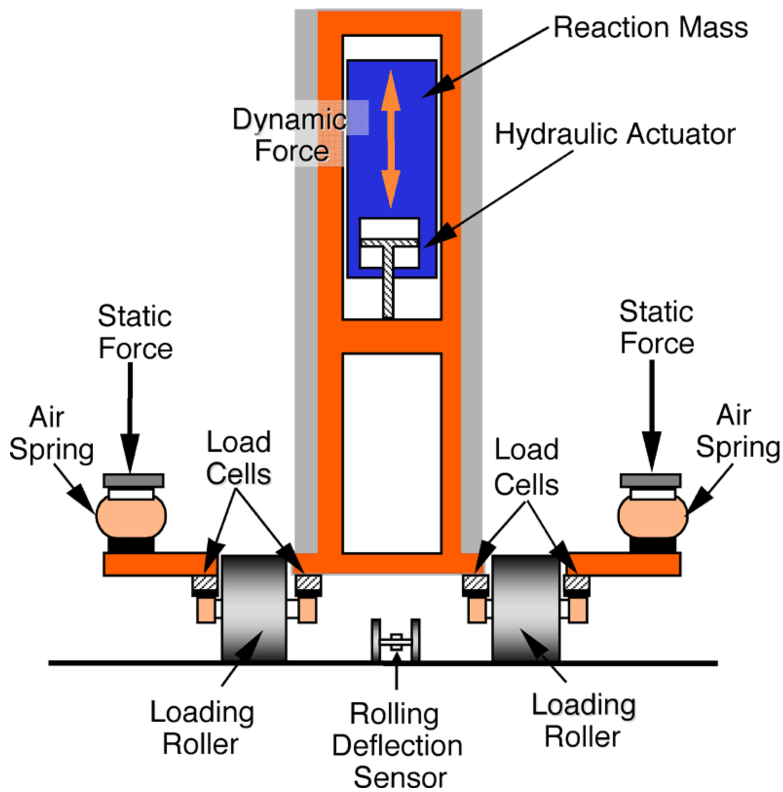


Figure 7.6 Front cross-sectional view of RDD loading and measurement systems

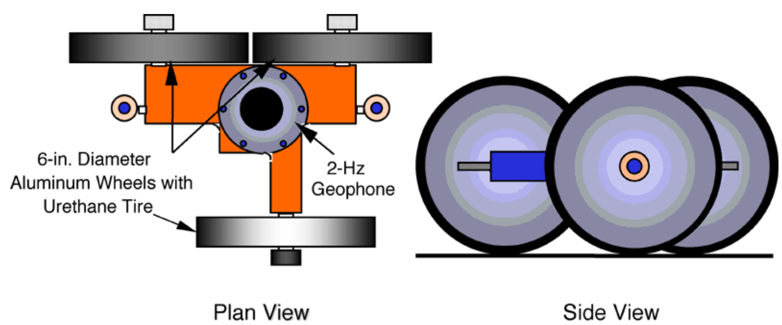


Figure 7.7 Plan view and side view of RDD rolling sensor

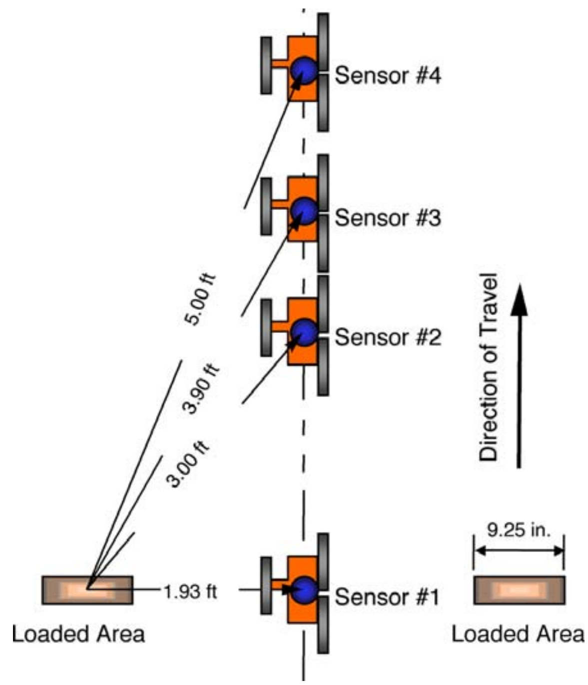


Figure 7.8 Rolling sensors configuration

Figure 7.9 RDD rolling sensors

Figures 7.7, 7.8, and 7.9 illustrate the rolling sensors.

7.5.6 Strength of Concrete

Concrete strength tests can be performed in manufactured samples such as cylinders and beams, as well as in core samples. In-situ samples have the advantage over cylinders and beams of allowing for the testing of the concrete that is actually in the overlay, rather than testing just the concrete that was delivered at the construction site, which is the case of molded specimens. Hence, core samples are trustworthier for representing the properties and conditions of the in-place pavement, although it should be acknowledged that the drilling process itself might introduce variables affecting the strength results of extracted samples.

Compressive, flexural, and splitting tensile strength are common tests for concrete pavement. In the case of manufactured samples, cylinders are used for compressive and splitting tensile strength testing, whereas beams are tested for flexural strength.

The maturity method, an NDT, may be used to estimate the compressive strength of concrete.

The procedures for molded specimen preparation and storage are established in ASTM C 31 (Ref 72). The following excerpt indicates the size and shape of the specimens:

> *Cylindrical Specimens*—Compressive or splitting tensile strength specimens shall be cylinders cast and allow setting in an upright position, with a length equal to twice the diameter. The standard specimen shall be the 6 by 12 in.

> *Beam Specimens*—Flexural strength specimens shall be beams of concrete cast and hardened in the horizontal position. The length shall be at least 2 in. [50 mm] greater than three times the depth as tested. The ratio of width to depth as molded shall not exceed 1.5. The standard beam shall be 6 by 6 in.

The in situ specimens are cut from hardened concrete as specified in ASTM C 42 (Ref 73). It is common to obtain lower strengths from specimens cut from the top of the concrete placement and higher strengths from specimens cut from the bottom of the concrete placement as is shown in Ref 14, owing to the loss of moisture from the top of the concrete surface to the environment while curing.

7.5.7 Compressive Strength

Compressive strength testing of cylinders is specified in ASTM C 39 (Ref 74). The results of this test are intended to measure the concrete's ability to withstand a uniaxial compressive force; however, the stress conditions to which the sample is subjected during the test is much more complex than those for uniaxial compression: there is friction between the bearing faces of the testing machine and the specimen, which restrains the specimen laterally, thereby inducing lateral compression in the specimen ends (Ref 75). Besides this consideration, many factors affect the outcome of the test, such as the specimen end conditions, its size and aspect ratio (length to diameter), its diameter to aggregate ratio, the sample moisture and temperature conditions, the loading direction versus casting direction, the testing machine properties, and the loading rate. Results from cylinders and beams are not comparable, because of the differences in geometry of the specimens.

7.5.8 Tensile Strength

There is no standard procedure for a direct determination of the tensile strength of concrete. However, it is the concrete's tensile strength the property that establishes its resistance to cracking. Because of its importance, there are procedures to indirectly gauge the tensile capacity of concrete. The splitting tensile strength test and flexural strength tests, which are among the most common of these procedures, are described below. It should be noted that results from each test method are specific to that procedure and cannot be used interchangeably.

7.5.9 Splitting Tensile Strength

This test is standardized by ASTM 496 (Ref 76). In it, the cylindrical specimen is placed on its side, and a diametrical compressive force along its length is applied. The specimen is loaded until failure, which occurs along a vertical plane running through its axis. If the test specimen were a homogenous material, which concrete is not, the application of a load perpendicular to the axis of a cylinder in a diametrical plane would produce a uniform tensile stress over that plane. This theory assumes linear elastic behavior, which is not entirely true for concrete. The tensile strength is calculated as follows:

$$T = \frac{2P}{\pi\, Ld} \tag{7.4}$$

where

T = splitting tensile strength, psi

P = maximum applied load, lb

L = length of the specimen, in.

d = diameter of specimen, in.

The length of the specimen does not affect the results; hence, it is common practice to cut core samples transversely and to test them at different depths of the slab, which also enables testing of the BCO and the existing pavement from a single core. Cores having a diameter of 4 in. have been observed to produce splitting tensile strengths about 10% higher than those obtained from 6-in. diameter samples (Ref 75).

7.5.10 Flexural Strength

The flexural strength of concrete is obtained from testing beams by either the third-point loading method (ASTM C 78, Ref 77), or by the center-point loading method (ASTM C 293, Ref 78). The difference between these two procedures is the location of the load application. Test results from center-point loading are higher than those obtained from third-point loading, with discrepancies of 15 percent being common (Ref 75).

Sometimes it is useful to estimate a concrete strength value from the results of another test. The ratio of splitting tensile strength to compressive strength ranges between 0.08 and 0.14. The ratio of flexural strength test results obtained by third-point loading to compressive strength lies between 0.11 and 0.23, and these ratios are even higher for center-point loading flexural strength to compressive strength (Ref 75).

7.5.11 Maturity Method

Maturity in concrete technology refers to the extent of the development of those properties of a cementitious mixture dependent on cement hydration and pozzolanic reactions. Maturity depends on the previous curing history of concrete, which is given by its temperature and moisture content through time. The procedure for estimating concrete strength, described in ASTM C 1074 (Ref 79), is based on the development of a maturity function, a mathematical expression to account for the combined effects of time and temperature on strength gain of a concrete mixture. The maturity function converts the temperature history to a maturity index that is indicative of strength development. This function is specific for each concrete mixture. The basic assumption of the method is that samples of a given concrete mixture will attain equal strengths if their maturity indices are equal.

The concrete temperature is measured by means of thermocouples or with maturity meters. A recommended time interval for temperature recording is 30 minutes for the first 48 hours; longer intervals may be used afterward.

This test is used to determine whether a pavement has gained enough strength to be opened to traffic, because its ability to carry traffic is a function of its strength; hence, the method is helpful in expedited BCO projects. For instance, for the El Paso BCO project, cited in Chapter 2, it was determined that, according to laboratory and field tests, the BCO was ready for traffic at 600 to 700°C hours (Ref 1).

A major drawback of the method is that it has to be supplemented by other indicators of the potential strength of the concrete mixture.

7.5.12 Modulus of Elasticity of Concrete

The standard test method for static modulus of elasticity of concrete is performed according to ASTM C 469 (Ref 80), which is a compressive procedure applied to concrete cylinders and cores. The outcome of the test is the determination of the chord modulus of elasticity (stress to strain ratio), and the Poisson's ratio (lateral to longitudinal strain ratio) may be estimated. The chord modulus is gauged between a level of stress corresponding to 40 percent of the ultimate concrete strength for the higher point and a strain of 50 millionths for the lower point. This determination is intended to obtain the approximate average concrete modulus throughout a customary working stress range. The modulus is calculated with the following equation:

$$E = \frac{S_2 - S_1}{\varepsilon_2 - 0.000050} \tag{7.5}$$

where

E	=	chord modulus of elasticity, psi
S_2	=	stress corresponding to 40 percent of ultimate load, psi
S_1	=	stress corresponding to a longitudinal strain, ε_1, of 50 millionths, psi

7.5.13 Thermal Expansion of Concrete

Concrete has a positive coefficient of thermal expansion, which expresses the linear change per unit length divided by the temperature change. Although a value of 5.5 millionths/°F is extensively used, thermal expansion is a complex phenomenon resulting from the interaction of materials, moisture, and temperature. Because concrete is subjected to a wide variety of moisture and temperature conditions, and the properties of its component materials vary broadly, there has not been a standard test procedure purposely developed for the coefficient of thermal expansion. However, in practice, the test is conducted according to the following procedure. The cores are prepared following the guidelines established in ASTM C 341 (Ref 81), which is a length change test for cores, where the dimensional change occurs as a consequence of effects other than external loading and temperature changes; therefore, as originally intended, this particular test is not applicable for thermal dimensional changes. Holes are drilled at the center of the ends of the core, and gage studs, such as those specified in Ref 81, are glued with epoxy at both ends. Approximately half of each gage stud should extend out of each end of the core. The length of the gage studs extending out of the core and the length of the cores should be registered at room temperature.

After the epoxy bonding the gage studs to the core is sufficiently cured, the specimen is placed in an oven and heated overnight. The change in length of the steel gage studs and the change in length of the core are measured and are used to obtain the coefficient of thermal expansion for each core (Ref 2).

7.5.14 BCO Thickness

The measurement of overlay thickness may be performed by destructive and non-destructive methods. The destructive way is to measure it directly from core samples; this procedure is done with a three-point calipering device, capable of making a length measurement at the center of the upper end of the core and at eight additional equally spaced intervals along the top of the core, following the guidelines of ASTM C 174 (Ref 82). Thickness estimation from non-destructive procedures involves seismic methods such as impact echo, described in the Delamination Detection section.

7.5.15 Delamination Detection

Several NDTs are available to locate delaminations. Deflection testing is one of them. In this section, impact echo, a seismic technique, is presented, along with a very common, effective and not very refined procedure: sounding.

7.5.16 Impact Echo

The impact echo technique is a non-destructive sonic test for the evaluation of concrete member integrity and thickness. It involves introducing mechanical energy, in the form of a short stress pulse, into the structure (Refs 83, 84, and 85). The test identifies echoes from flaws, voids, reinforcing bars, and/or the opposite side of the sound concrete member, or any change in medium, such as the substrate or subbase. Therefore, this procedure is ideal for BCO delamination detection as well as for thickness estimation of the BCO.

The stress pulse is originated at the point of impact, and it propagates in all directions rather than as a focused beam, and because of that, the reflections may arrive from many directions. This is the major drawback of the test, and for this reason, impact methods were initially used only for testing piles. In that case, the pile boundaries act as a guide for the waves and confine the energy within the pile. When the pulse strikes a reflector within the test object, which can be any flaw or discontinuity, the wave (echo) is reflected back to a receiver, and a surface displacement is recorded.

In isotropic solids, elastic theory indicates propagation of three types of waves: compression, shear, and surface waves. Compression waves, also known as dilatational or P-waves, are those in which the particle motion is parallel to the direction of propagation. Shear, distortional, or S-waves are those in which the particle motion is perpendicular to the direction of propagation. In the surface waves, also known as Rayleigh or R-waves, the motion is along the surface of the solid. Wave propagation in heterogeneous materials such as concrete is a more complex phenomenon, but the relationship between wave speed and the properties of a material can be understood using elastic theory. Figure 7.10 illustrates the impact echo technique.

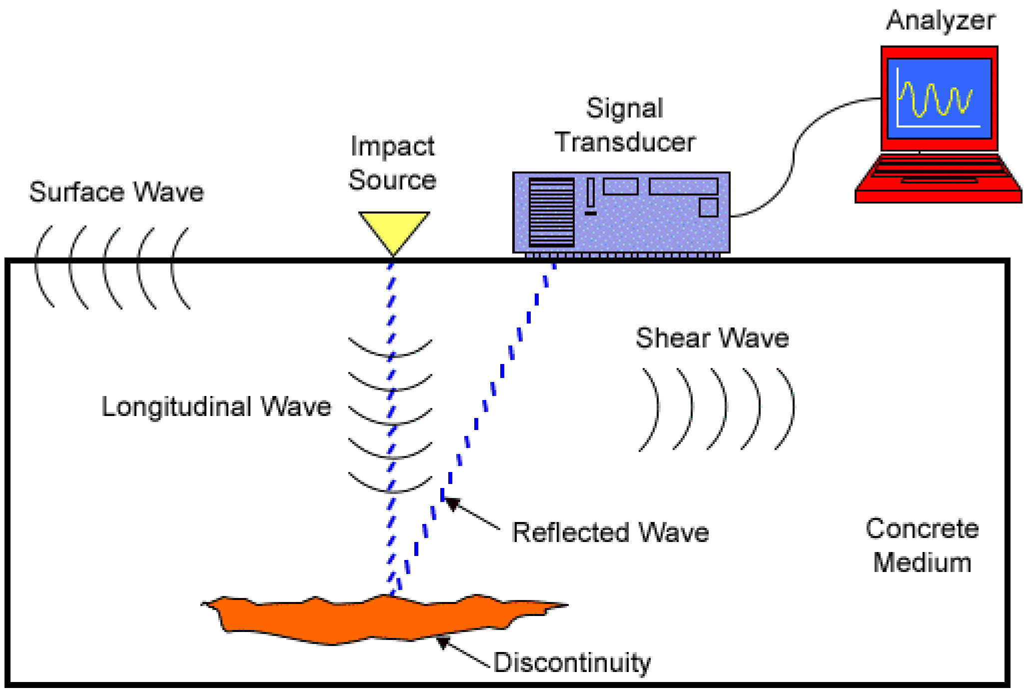


Figure 7.10 Schematic of the impact echo technique

The interpretation of waveforms from the test can be done using two different approaches, depending on the shape of the test structure.

For long, slender structures, such as piles, in which the geometry of the structure helps to keep the waves propagating within its relatively narrow boundaries, the following equation is used:

$$T = \frac{1}{2} \Delta t \, C_p \tag{7.6}$$

where

T	=	depth of the test object
Δt	=	round trip travel time
C_p	=	P-wave speed

The P-wave speed is calculated with an impact echo measurement on a part of the structure with known thickness.

The reflections recorded at the top of the surface are easy to interpret, because there is enough time between the generation of the stress pulse and the reception of the reflected wave from the bottom surface or a discontinuity.

For thin structures, such as a BCO or any slab or wall, an alternative approach is followed, which consists of the frequency analysis of the displacement waveforms: the stress pulse generated by the impact travels back and forth between the bottom (or a discontinuity) and the top surface, and every time it arrives at the top surface, it produces a displacement. The waveform is periodic, with the period being equal to the travel path, 2T, divided by the P-wave speed, C_p. The frequency (f) is the inverse of the period:

$$f = \frac{C_p}{2T} \tag{7.7}$$

As in the previous case, the P-wave speed is calculated with an impact echo measurement on a part of the structure with known thickness. The thickness or distance to a flaw or interface is calculated as follows:

$$T = \frac{C_p}{2f} \tag{7.8}$$

The equipment necessary for this test includes three basic components:

- Impact source
- Receiving transducer
- Digital processing oscilloscope or waveform analyzer

The selection of the impact source is a key element in the success of the test. The duration of the impact determines the frequency content of the stress pulse. If the contact time is short, then the pulse contains higher frequency components and smaller defects can be detected. Also, short duration impacts are better in facilitating the location of shallow defects.

For pile evaluation, hammers are the impact source of choice. Hammers produce long contact times (more than 1 ms), which are appropriate for slender structures. However, for thin, slab-like structures, small steel spheres and spring-loaded spherically-tipped impactors are used, which produce shorter duration impacts (20 to 60 μs). For steel spheres, the contact time is proportional to the diameter of the sphere.

Geophones and accelerometers are used as receiving transducers.

7.5.17 Sounding

Sounding is a technique to verify the bonding between the BCO and the substrate. It is a manual technique, which unfortunately is also time consuming and labor-intensive. The surveyor locates delaminations by holding a steel bar vertically and dropping it onto the overlay. The presence of a delaminated area is identified by a characteristic hollow sound as the bar impacts the overlay. With quiet surroundings, the procedure is very reliable. Sounding is illustrated in Figure 7.11.

Figure 7.11 Sounding for delamination detection

It is recommended that delaminated areas be demarcated with spray paint as soon as they are found. It is common for delaminations to start near the edge of the slab and spread toward the center of it, as isshown in Figure 7.12.

Figure 7.12 Common delamination pattern delineated with spray paint

7.5.18 Bond Strength Evaluation

There are several procedures to measure the bond strength of a BCO. This is a subject of capital importance for a BCO. Delamination tests and bond strength are critical components of the QC/QA of a BCO.

7.5.19 Slant Shear Method

This laboratory test, based on ASTM C 1042 (Ref 86), is the most common method for determining bond strengths of epoxies and latex systems used with concrete. It was originally developed to measure the effectiveness of latex in bonding fresh concrete to hardened concrete. The test makes use of a metallic cylindrical mold, into which a dummy section, made of hard material and equal to half the volume of the cylinder, is precast in a position that is skewed 30° from the vertical (Figure 7.13). The remaining half of the mold is filled up with the overlay mortar. In the case of a BCO, the dummy portion of the cylinder is intended to simulate the substrate. When the overlay mortar cures, the entire specimen is removed from the mold and tested in compression. The specimen fails along the bond line. However, the test is not an accurate simulation of the actual conditions that occur with a BCO; at the interface of the test specimen, the compressive force produces a shear component parallel to the skewed interface and a normal component perpendicular to the interface. The normal component provides more friction than actually occurs in shear failures of BCOs (Ref 19).

The bond strength is calculated by dividing the load applied at failure by the area of the bonded surface.

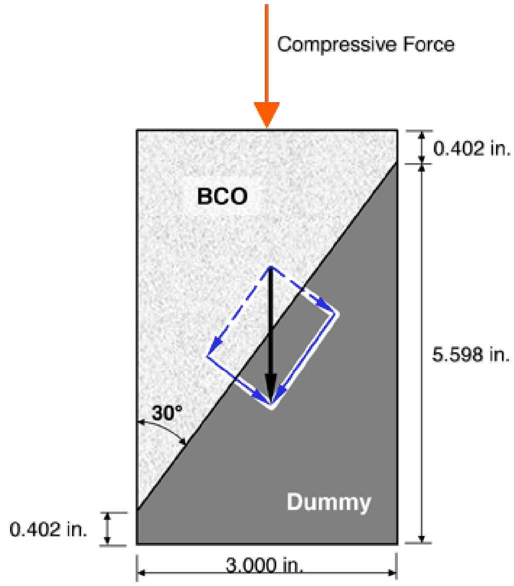


Figure 7.13 Schematic of slant shear test

7.5.20 Pulloff Test

This tensile bond test was originally developed in the United Kingdom in the mid-1970s (Ref 87). It is similar to the ACI method for determining the adhesion of epoxies to concrete substrates (ACI 503R, Ref 88), from which it was adapted for the testing of BCOs. In this field test, a core is drilled but not extracted. The drilling is stopped slightly below the depth of the BCO; the BCO is left with its original bond to the substrate intact. Then a circular steel plate is glued to the top surface of the BCO using epoxy resin. After the epoxy has cured, a tensile force is applied to the steel disk with a mechanical or hydraulic racking device, pulling the core away from the pavement. Because the tensile strength of the bond between the epoxy and the BCO is greater than that between BCO and substrate, the latter fails in tension. The pulling device records the tensile strength to pull the BCO core from the substrate (Figure 7.14).

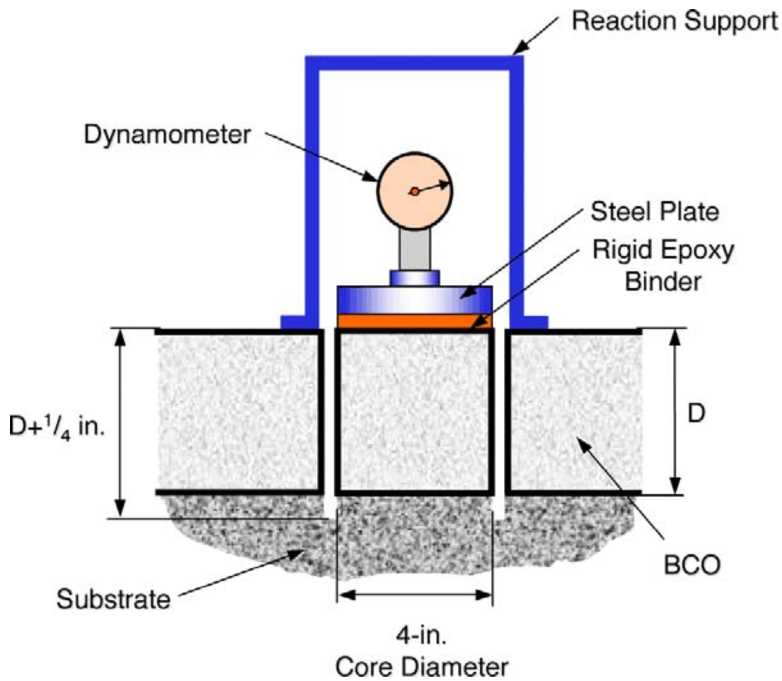


Figure 7.14 Schematic of pulloff test

The stress at failure is a direct measure of the tensile strength of the concrete. The test is relatively simple to perform and gives reproducible results.

7.5.21 Texture Evaluation

The most commonly recommended procedure for texture evaluation is the sand-patch method. It is the simplest, least expensive, and most reproducible test among those available (Ref 19).

7.5.22 Sand-Patch Method

The sand-patch method is based on ASTM E 965 (Ref 89), and it has been incorporated into TxDOT testing procedures as Tex-436-A, "Measurement of Texture Depth by the Sand-Patch Method" (Ref 90). It describes a field procedure for determining the average texture depth of a selected portion of a concrete pavement surface. The test consists of pouring the contents of a cylinder of known volume filled with silica sand of a specific gradation and spreading them onto the previously cleaned and brushed, dry pavement surface. The diameter of the sand patch is measured at four locations and the texture depth is determined as follows:

$$T = \frac{4V}{\pi D^2} \tag{7.9}$$

where

T = texture depth, in.

V = volume of sand, $in.^3$

D = average diameter of the sand patch, in.

The texture depth may be used to determine the pavement skid resistance capability. The sand-patch technique may be applied on the substrate, prior to the overlay placement, as an evaluation of the effectiveness of the surface preparation operations. As a result of it, additional milling or blasting of the substrate may be recommended.

7.6 Summary

The basic concepts of QC/QA have been discussed in this chapter. A fundamental premise for a quality BCO is to implement sound practices at every stage of the BCO process. QC involves good practices, applied throughout the BCO process, and tests, applied mostly during and after construction, the purpose of which is to insure that a quality BCO is planned, designed, and constructed. In the construction stage, QC is done at the discretion of the contractor, whereas the owner, or the owner's agent verifies the QA testing. Specifications set the standards against which quality is measured, by means of QA testing. Some of the most common testing procedures were mentioned, and a number of innovative tests were presented in more detail.

8. Implementation

The process of a bonded concrete overlay (BCO) was presented in the previous five chapters. In this chapter, which may be considered the second part of this report, a particular example that addresses the several stages of the process is discussed in detail. The project chosen for this purpose is a recent rehabilitation on an interstate highway in Texas. This project epitomizes the BCO experience at its fullest, with an expedited overlay in conjunction with a road-widening project showing the application of the state-of-the-art in BCO research. But it also typifies construction mistakes that emphasize the significance of proper execution of every critical step for a successful BCO. The fact that the project was not exempt from blemishes presented an additional challenge to research the cause of the problem. The ascertainment of the delamination cause is presented in Chapter 9.

8.1 Overview

The full-scale BCO project on Interstate Highway 30, in Fort Worth, presented in this section, represents an ideal opportunity to illustrate the BCO process. It was undertaken by the Center for Transportation Research (CTR) at The University of Texas at Austin, in conjunction with the Texas Department of Transportation (TxDOT).

The research activities specific to this project (i.e., the project selection), began in August of 1994, with the data collection starting the following month. A design for a new BCO was finalized in the summer of 1996. Five contractors bid on the project in March of 1997. The construction of the road-widening phase started in November of 1997. After some construction delays, the BCO was built in the summer of 1998. Delamination of part of the BCO manifested shortly thereafter. CTR and TxDOT embarked on a forensic study, presented in the following chapter, to investigate the cause; the study concluded in the fall of 2000.

8.2 Project Selection

By the end of last century, large metropolitan areas such as Fort Worth had various sections of the interstate highway system approaching the end of their pavement lives. The Fort Worth district has many miles of continuously reinforced concrete pavements (CRCP) that were constructed in the late 1950s, 1960s, and 1970s, which gave excellent performance throughout the years. One of these pavements was the section presented herein, which, at the inception of the project, was in need of some type of rehabilitation and was at the optimum point of application of a BCO to extend its useful life. Therefore, the construction of a BCO in the Fort Worth District offered the potential of providing years of additional service life at minimum life-cycle cost.

The rehabilitation of a roadway always causes traffic disturbances and implies user-associated costs and an increase in pollution problems. In order to keep all these to a minimum while providing an adequate repair, a strategy for the rehabilitation was devised, which included expedited construction. The economical and technical feasibility of a BCO as a solution for rehabilitation for the heavily urbanized and traveled pavement sections on Interstate Highway 30 was the first task in this study.

The project is located in Tarrant County (Fort Worth District) on IH-30. The original roadway section was constructed in 1967 and consisted of a pavement structure of 8 in. of CRCP over a 6-in. layer of lime-stabilized subgrade. In 1975, the pavement received a 4.5-in. thick hot mix overlay to correct some longitudinal roughness and surface polish. A plant mix seal was applied in 1981 to improve the surface texture. In 1993, the accumulated hot mix layers were removed because of their excessive deterioration. A 2-in. asphalt concrete (AC) overlay was placed to provide an interim acceptable riding surface until the rehabilitation took place. This layer was removed prior to overlaying. The existing pavement structure at the beginning of the project is shown in Figure 8.1.

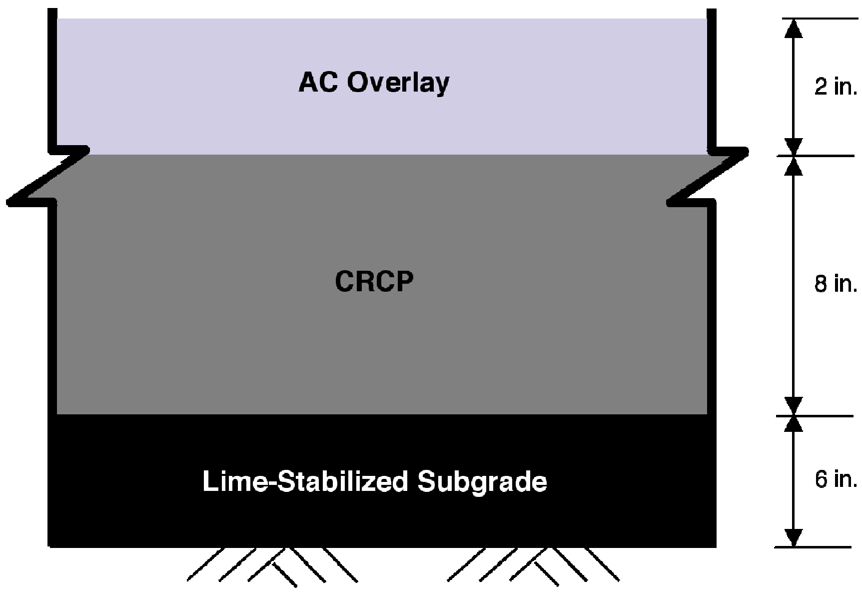


Figure 8.1 Existing cross section at the beginning of the project

The section is approximately 1.3 mi long, including bridges and ramps. There are 6,930 ft of roadway and 170 ft corresponding to bridges. It is located in Fort Worth, (Figure 8.2) on IH-30 between Loop 820 (West Loop) and Las Vegas Trail. The section in question lies between station markers 962 + 00 and 1033 + 00. Each station represents 100 ft. Figure 8.3 shows the project location on the west side of Fort Worth.

Figure 8.2 Map showing location of project in Fort Worth, Texas, as indicated by the star (★)

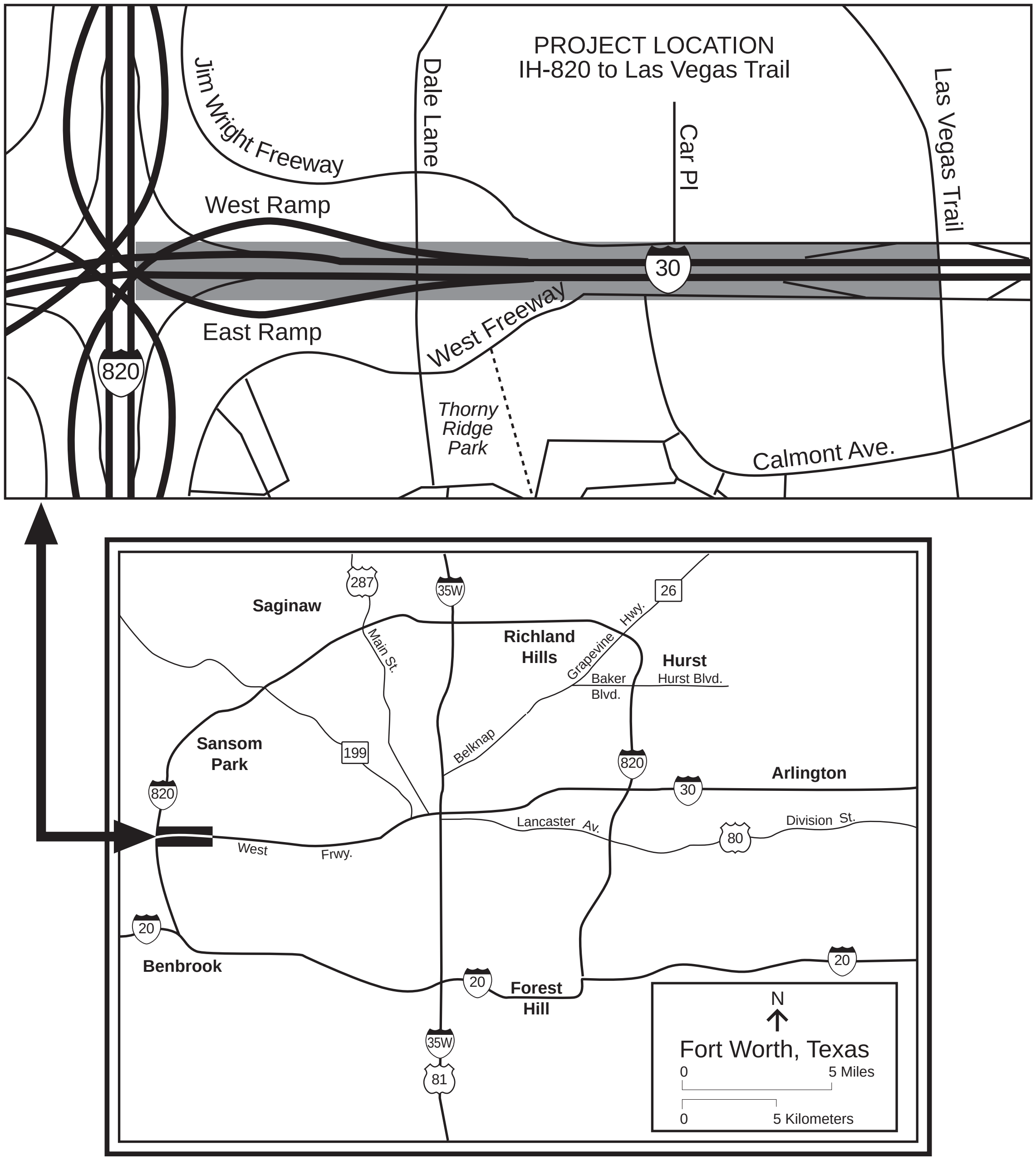


Figure 8.3 Project location on IH-30 in western Fort Worth, just inside IH-820

The rehabilitation of the existing pavement included, because of projected traffic demands, an increase in the freeway capacity. Consequently, the project also included widening the cross section to provide more lanes. For this, a new CRCP was constructed, spanning up to 48 ft, which added one lane in each direction. After the widening and rehabilitation were finished, the resulting cross section was a 48-ft portland cement concrete pavement (PCCP) slab for each traveling direction. In certain areas, both directions are joined to form a continuous 96-ft PCCP cross section.

In summary, the factors that triggered the rehabilitation decision were age, given that the original pavement was almost 30 years old at the time, traffic increase, shown by the need for increased capacity, and loading, which had caused excessive deterioration of the AC overlay removed in 1993, a rough profile, and the need of a provisional riding surface until the rehabilitation could take place.

A BCO was selected over non-overlay methods given that the purpose of the rehabilitation was to increase the pavement structural capacity and increase its service life. A BCO was preferred over an AC overlay because the district needed a long-lasting rehabilitation that could improve the existing structural capacity of the pavement. A BCO was chosen over an unbonded overlay because of the clearance requirements at three structures along the project section, and because the current condition of the CRCP did not warrant an unbonded overlay, which is normally required when the pavement is at a latter stage of deterioration.

The decision of selecting a BCO as the rehabilitation strategy implied that the structural capacity of the current pavement at that time would be fully utilized, allowing for an optimal use of the existing infrastructure, and would be enhanced by providing it with additional service life.

8.2.1 Evaluation

A visual inspection conducted when the AC layer was milled off indicated that the CRCP section was structurally sound. A thin rehabilitation method was necessary to provide vertical clearances at three structures. These conditions indicated that a thin BCO was the most desirable option. The evaluation conducted at this stage was based on deflection measurements and testing of in situ samples, which are discussed below.

8.2.2 Deflection Testing

Falling Weight Deflectometer (FWD) tests were conducted in April of 1994. The collected surface deflection data were analyzed using a microcomputer-based procedure called RPEDD1 (Rigid Pavement Evaluation Program; Ref 91), wherewith the backcalculated layer properties were obtained.

8.2.3 Load Transfer Efficiency (LTE)

The LTE of a pavement structure, as introduced in Chapter 5, refers to its ability to transfer loads across transverse discontinuities such as joints or cracks. A pavement structure with a high value of load transfer efficiency indicates that the loads are adequately distributed at the discontinuities.

The average LTE obtained for the Fort Worth project section was 98.5 percent, which is an indication of good behavior of the pavement regarding load distribution.

8.2.4 Deflection Results

The outcome of the backcalculation process is the modulus of elasticity for the component layers of the structure; the mean results obtained from the RPEDD1 program are presented in Table 8.1.

Table 8.1 Modulus of elasticity backcalculated from pavement deflections

Modulus of Elasticity (ksi)		
Slab	Subbase	Subgrade
4,703	58.7	26.8

8.2.5 In Situ Samples

Twenty core specimens were extracted from the eastbound and westbound lanes of the IH-30 section. The samples were tested by the Construction Materials Research Group at The University of Texas at Austin. The cores had a diameter of 4 in. and a typical length of 8 in. Six cores were tested for modulus of elasticity, four were tested for coefficient of thermal expansion, four were tested for splitting tensile strength, and eight for compressive strength. Table 8.2 shows the length and tests performed on the cores.

Table 8.2 Tests performed on cores

Core Number	Average Length (in.)	Direction	Tests Performed
1	8.125	Westbound	Splitting tensile
2	8.016	Westbound	Modulus, Compressive
3	8.000	Westbound	Thermal coefficient
4	8.125	Westbound	Compressive
5	7.922	Westbound	Splitting tensile
6	8.125	Westbound	Modulus, Compressive
7	7.875	Westbound	Splitting tensile
8	7.813	Westbound	Modulus, Compressive
9	7.750	Westbound	Thermal coefficient
10	7.063	Westbound	Damaged in testing
11	7.719	Eastbound	Thermal coefficient
12	7.578	Eastbound	
13	7.609	Eastbound	Modulus, Compressive
14	8.063	Eastbound	Splitting tensile
15	7.906	Eastbound	Modulus, Compressive
16	7.875	Eastbound	Splitting tensile
17	7.781	Eastbound	Splitting tensile
18	7.781	Eastbound	Thermal coefficient
19	7.688	Eastbound	Modulus, Compressive
20	7.531	Eastbound	

The length of each core was measured four times and averaged. The mean pavement thickness was 7.817 in., and its standard deviation was 0.254 in. None of the cores appeared to have been cracked. Core 7 had an indentation from reinforcing steel at mid-depth. The diameter of the cores was also measured, being within 1/16 in. of the nominal 4-in. diameter in all the cases.

8.2.6 Testing Program

Four different tests were performed on the cores as presented in Table 8.2: modulus of elasticity, coefficient of thermal expansion, splitting tensile strength, and compressive strength. The tests were performed according to the guidelines presented in Chapter 7.

8.2.7 Modulus of Elasticity

Measured core moduli varied from 3.53 to 4.29 million psi. The mean value was 3.99 million psi, and the standard deviation was 0.318 million psi. The coefficient of variation was 8 percent. These are secant moduli values at approximately 45 percent of the compressive strength of the cylinder (Table 8.3). Note that the mean value for the backcalculated modulus was 4.7 million psi.

Table 8.3 Elastic modulus of cores

Core Number	Modulus (million psi)
2	4.04
6	4.29
8	4.27
15	3.53
19	3.83
Mean	3.99

As expected, because of the higher stresses applied to the pavement cores as opposed to the level of stress prevailing during the deflection tests, these moduli are significantly lower than those obtained from the backcalculation procedure. This was explained in Chapter 5, and it was illustrated in Figure 5.6, which is reproduced here as Figure 8.4. In it, the conceptual concrete stress-strain curve, where the two different slopes of the curve (elastic moduli), corresponding to coring and deflection testing, are compared.

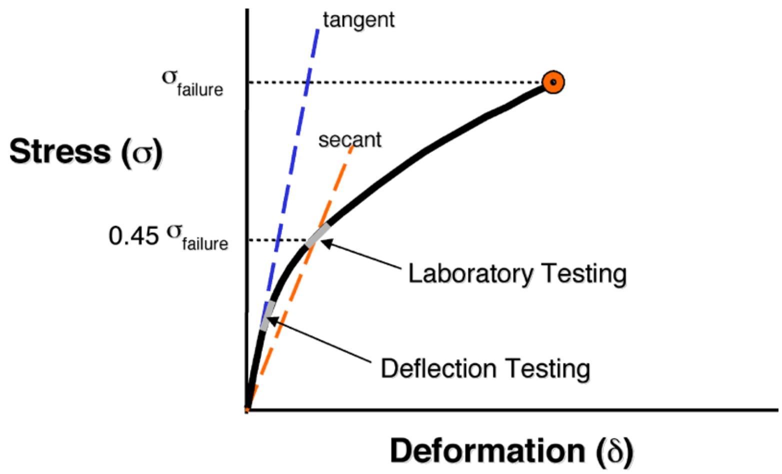


Figure 8.4 Moduli of elasticity for coring and deflection testing

8.2.8 Coefficient of Thermal Expansion

The cores were heated to 201 to 210 ºF and cooled to room temperature (72 to 80 ºF) seven times. The coefficient of thermal expansion of the cores tested varied from 2.39 x 10^{-6} in./in. ºF to 7.25 x 10^{-6} in./in. ºF. The mean value was 4.53 x 10^{-6} in./in. ºF, the standard deviation was 1.62 x 10^{-6} in./in. ºC, and the coefficient of variation was 36 percent. A possible explanation for such a high coefficient of variation in this test could be that the gage studs were affixed to the cores with epoxy, and this may have caused the variability.

8.2.9 Splitting Tensile Strength

Splitting tensile strength varied from 576 to 729 psi. The mean value was 633 psi, the standard deviation was 56.2 psi, and the coefficient of variation was 9 percent. Table 8.4 presents the splitting tensile strength test results.

Table 8.4 Splitting tensile strength of cores

Core Number	Splitting Tensile Strength (psi)
1	608
5	616
7	729
14	576
16	601
17	669

8.2.10 Compressive Strength

Compressive strength varied from 4,737 to 5,924 psi. The mean value was 5,236 psi, the standard deviation was 442 psi, and the coefficient of variation was 8 percent. Normally, the compressive strength of the cores is expected to be approximately 85 percent of that of the cylinders (f'c); therefore, the cylinder's f'c can be predicted by dividing core strength by 0.85 (Ref 55). Similarly, the elastic modulus may be predicted by multiplying 57,000 times the square root of the cylinder strength, in psi (Ref 55).

The predicted and measured modulus values are compared in Table 8.5. Values from Table 8.3 are shown in the last column for comparison. The mean predicted value was 4.47 million psi, and the mean measured value was 3.99 million psi. Except for Core 8, the predicted moduli values are significantly higher than those measured.

Table 8.5 Compressive strength test results and predicted values

Core Number	Compressive Strength (psi)	Predicted f'c (psi)	Predicted E (million psi)	Measured E (million psi)
2	4,783	5,627	4.28	4.04
6	5,430	6,389	4.56	4.29
8	4,737	5,572	4.26	4.27
15	5,924	6,969	4.76	3.53
19	5,209	6,129	4.46	3.83

8.2.11 Summary of Materials Characterization

The moduli of elasticity obtained from the cores were not as high as those backcalculated from the deflection measurements. This is because the stresses applied to the pavement in the deflection test are lower than the stresses placed on the cores. Lower stresses lead to higher moduli, because at that point, the concrete is tested at an earlier stage of the stress-strain curve, still in the elastic interval, and therefore, the slope of the curve (i.e., the modulus of elasticity) is higher. Discontinuities in the pavement (cracks) performed adequately regarding load transfer, reaching an average value of 98.5 percent LTE for the entire project section. This value indicates an excellent behavior of the cracks at transferring loads.

The test results from the cores appeared to be reasonable, except for a few results from the coefficient of thermal expansion test. The coefficient of variation for that test was very high (36 percent). The use of epoxy to affix the gage studs may have influenced the results. The coefficients of variation from all the other tests were less than 10 percent.

Nevertheless, the results revealed that the CRCP was in adequate condition to accommodate a BCO, with both deflection and core testing showing structurally sound concrete pavement. Consequently, it was deemed appropriate to proceed to subsequent stages of the BCO process.

8.3 Design

The following paragraphs discuss the traffic analysis, remaining life, overlay thickness design, and reinforcement design.

8.3.1 Traffic Analysis

The traffic data obtained from the Fort Worth District for the project segment of IH-30 consisted of the anticipated average daily traffic (ADT) volumes for the years 1996, 2016, and 2026. Also, a tabulation showing traffic analysis, in 18-kip equivalent single axle loads (ESALs) through the year 2026, was provided by the district, for one direction and a 65 .percent directional distribution. The traffic information is summarized in Tables 8.6 and 8.7.

Table 8.6 IH-30 ADT estimation (both directions)

Year	ADT
1996	58,800
2016	89,600
2026	102,000

Table 8.7 IH-30 ESAL estimation (one direction)

Design Period	Years	ESALs
1996-2016	20	14,652,000
1996-2026	30	23,812,000

With this information and assuming a compound growth, a 1.45 percent annual growth rate was calculated for the ESALs. The analysis periods chosen for the thickness design of the overlay were 30, 40, and 50 years. ESAL estimations were performed for the different analysis periods using a 65 percent directional distribution and a 40 percent lane distribution factor, and the results are presented in Table 8.8.

Table 8.8 IH-30 estimated number of ESALs

Analysis Period (years)	Both Directions	Single Direction	Design Lane
30	36,073,259	23,447,619	9,379,047
40	51,999,387	33,799,602	13,519,841
50	70,391,462	45,754,450	18,301,780

8.3.2 Remaining Life

The accumulated traffic from the opening of the pavement until the present was used to estimate the remaining life of the pavement. Several methods exist for this estimation.

In this case, the mechanistic fatigue model proposed in the AASHTO Guide was implemented (Ref 26). Having the total number of ESALs to date and the AASHTO design equation (Equation 5.19), the remaining life of the pavement section was estimated. The above-mentioned equation allows the designer to calculate the total number of ESALs to failure. With Equation 5.2, shown again here as Equation 8.1, the ratio of ESALs-to-date to ESALs-to-failure, in percentage, subtracted from 100 percent will give the percentage of remaining life:

$$RL = 100\left[1 - \left(\frac{n}{N}\right)\right] \tag{8.1}$$

where

RL = remaining life, percentage

n = total traffic to date, 18-kip ESAL

N = total traffic to pavement failure, 18-kip ESAL

The total traffic to pavement failure (N) is defined by the minimum acceptable Present Serviceability Index (PSI). To calculate the remaining life, the total number of ESALs since the facility was built (1967) up to the present was estimated. The traffic information obtained from the district was extrapolated back to the year 1967, because historical traffic data back to that year could not be obtained, and the total number of ESALs to date (n) was estimated to be 6,099,900. The total traffic to pavement failure (N) determined from the AASHTO design equation (Equation 5.19), using 50 percent reliability, was 28,520,000 ESALs. With these figures, the remaining life of the pavement section was estimated to be 80 percent. The high remaining life estimation is supported by the fact that the number of pavement defects in the condition survey (i.e., punchouts or patches) was minimal and that the materials testing indicated soundness of the pavement.

8.3.3 Overlay Thickness Design

The BCO was designed using two procedures: the AASHTO 1993 design method (Ref 26) and a mechanistic overlay design method called Texas Rigid Pavement Overlay Design (RPOD) which was incorporated into the Rigid Pavement Rehabilitation Design System (RPRDS; Ref 42), developed by CTR. Both procedures were explained in detail in Chapter 5. The AASHTO method is an empirical method, because some empirical factors must be identified to classify the pavement strata, climate and drainage conditions. The RPRDS method is mechanistic-empirical; it makes use of elastic layer theory and regression equations developed through a finite element model. The empirical part of the procedure incorporates fatigue damage relationships developed from the AASHO Road Test in order to predict failure. The computer program developed by CTR, called BCOCAD (Bonded Concrete Overlay Computer Aided Design), was utilized for the design (Ref 43). This program, also introduced in Chapter 5, calculates the overlay thickness by both methods simultaneously using a common set of inputs. The input variables can be classified into the following categories:

a) Materials information for each layer

Modulus of elasticity

Flexural strength (PCC only)

Poisson's ratio

b) Traffic information

Average daily traffic

18-kip ESALs –past and future

Analysis period

c) Other design information

Remaining life

Roadway cross section (width and thickness)

Reliability

Serviceability

The following general design parameters were considered in developing the design:

Desired level of reliability	99.5%
Serviceability index	
After overlay construction	4.5
At the end of performance period	2.5
Performance periods	30, 40, and 50 years
Overall standard deviation	0.39

As for the existing pavement materials inputs, the following considerations were made to account for the variations in thickness and tensile strength found in the data.

The thickness of the existing pavement varied substantially along the length of the project. Cores taken from the existing concrete showed a mean thickness of 7.817 in. with a standard deviation of 0.254 in. Assuming a t-distribution with 19 degrees of freedom, (20 cores, n = 20 - 1 = 19), and a 99.5% reliability ($\alpha = 0.005$), $t_{99.5}$ equals 2.861. Using this value, the thickness of the existing pavement to use in overlay design equals 7.817 - 2.861 * 0.254 = 7.09 in.

No direct flexural strength tests were performed on the existing concrete. To estimate the concrete flexural strength, the correlation shown in Equation 8.2, proposed in Ref 26, was used to calculate S'_c, in psi, from the splitting tensile strength, IT, also in psi.

$$S'_c = 210 + 1.02\ IT \tag{8.2}$$

Six cores were tested for splitting tensile strength. The values of flexural strength for the cores estimated with the previous correlation equation are shown in Table 8.9.

Table 8.9 Flexural strength estimation

Core Number	Splitting Tensile Strength, psi	Flexural Strength, psi
1	608	830
5	616	838
7	729	954
14	576	798
16	601	823
17	669	892

The correlated values of S'_c had a mean of 856 psi, with a standard deviation of 57 psi. To perform a design with a 99.5 percent reliability using the RPRDS method, a 99.5 percent value of S'_c must be calculated. With six cores, the degrees of freedom, n is equal to 5. The corresponding t value is $t_{99.5}$ = 4.032. Using this value, the design flexural strength, S'_c, equals 856 - (4.032) * 57 = 625 psi.

The design results, calculated with BCOCAD, for the 1993 AASHTO Design Procedure and the RPRDS method are shown in Table 8.10 and Figure 8.5.

Table 8.10 Overlay thickness designs

	Design thickness (in.)	
Performance Period (years)	1993 AASHTO Method	RPRDS Method
30	1.7	2.0
40	2.4	3.0
50	2.9	3.5

From these results, a 3.5-in. thick BCO with a 50-year performance period was deemed the optimum selection for the Fort Worth pavement rehabilitation. The design cross section is presented in Figure 8.6.

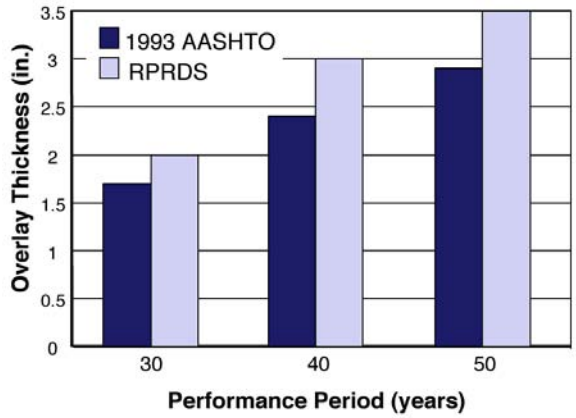


Figure 8.5 Overlay thickness design

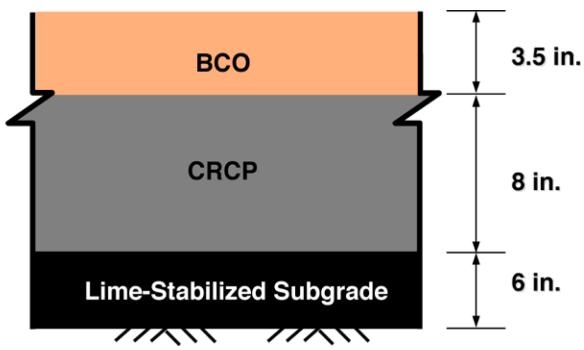


Figure 8.6 Proposed cross section for the BCO

8.3.4 Reinforcement Design

This section shows the longitudinal and transverse reinforcement design, as well as the design of tie-bars for construction joints. The design involved both the reinforcement for the bonded concrete overlay and the reinforcement for the new CRCP that was built enclosing the original depressed median. The steel design was performed using the analysis programs CRCP8 (Ref 92) and JRCP6 (Ref 93), developed at CTR, which calculate critical performance indicators such as steel stress, crack spacing, crack widths, and joint movement in the pavement for assumed loading conditions and design. The design was based on optimizing the performance indicators to provide a constructible pavement that will perform well over the design life. The following assumptions were made regarding required inputs for the analyses:

1. Construction was scheduled for the summer. Summer placement translates into the worst-case condition with regard to environmental loads on the pavement.
2. The concrete mix was designed to contain limestone as coarse aggregate. This assumption was based on the aggregate availability in that region of Texas in

which the project is located. The type of coarse aggregate in a mix determines several of the concrete properties relevant to design.

3. The annual minimum temperature was calculated using Fort Worth weather records for the past 25 years. The 99 percentile value for annual minimum temperature was calculated to be 0 °F. Maximum environmental stresses occur at the minimum temperature.
4. All the newly constructed portland cement concrete pavement (PCCP) was 11.5-in. thick. The new CRCP was 11.5-in. thick with a flexible subbase. The BCO was 3.5-in. thick on top of the existing 8-in. CRCP, which in turn is supported by a lime-stabilized subgrade.

The following are assumptions and input variables utilized for the reinforcement design:

Concrete Properties

Thickness:	11.5 in.
Elastic modulus (28 days):	4,800,000 psi
Tensile strength (28 days):	540 psi
Thermal coefficient:	6×10^{-6}/°F
Drying shrinkage (28 days):	0.00032

Steel Properties

Elastic modulus:	29×10^{6} psi
Yield strength:	60 ksi
Thermal coefficient:	5×10^{-6}/°F

Other Parameters

Subbase type:	flexible
Setting temperature:	120 °F
Minimum concrete temperature	
3 days after set:	90, 84, 80 °F
Annual minimum temperature:	0 °F

Two cases were considered for transverse steel design. The first one corresponds to the configuration of two unconnected 48-ft PCCP slabs, meaning that both directions of the road are separated; for the second case, the full 96-ft section is connected. It was proposed that the two 48-ft sections should remain unconnected where possible because this reduced the amount of reinforcement in the new 11.5-in. CRCP and eliminated the tie-bars at the center. The tie-bars between the new 11.5-in. CRCP and the existing 8-in. CRCP are essential, however, and cannot be eliminated. These bars keep the new pavement from shrinking away from the existing slab, preventing a dangerous and detrimental longitudinal gap, and providing the necessary load transfer.

The PCCP slab at the joint with the new asphalt shoulder is expected to move a maximum of 0.2 to 0.4 in. annually as it expands and contracts in summer and winter. A joint sealant was specified between the asphalt and the PCCP slab to prevent a gap from opening up, which would allow water into the supporting structure of the pavement.

Sensitivity analyses graphs of the longitudinal and transverse steel are presented below. Figure 8.7 relates both crack spacing and steel stress with longitudinal steel percentage and bar number. The steel percentage for this design was 0.6 percent, and number 6 bars were chosen. For that amount of steel and bar diameter, the mean crack spacing is around 5.2 ft, which is within the allowable limits of 8 ft (maximum crack spacing), and 3 ft (minimum crack spacing), according to Ref 26. These crack spacing limits are established in order to minimize the incidence of crack spallings and the potential for the development of punchouts, respectively. The crack distribution diagram illustrating those limits is presented in Figure 8.8.

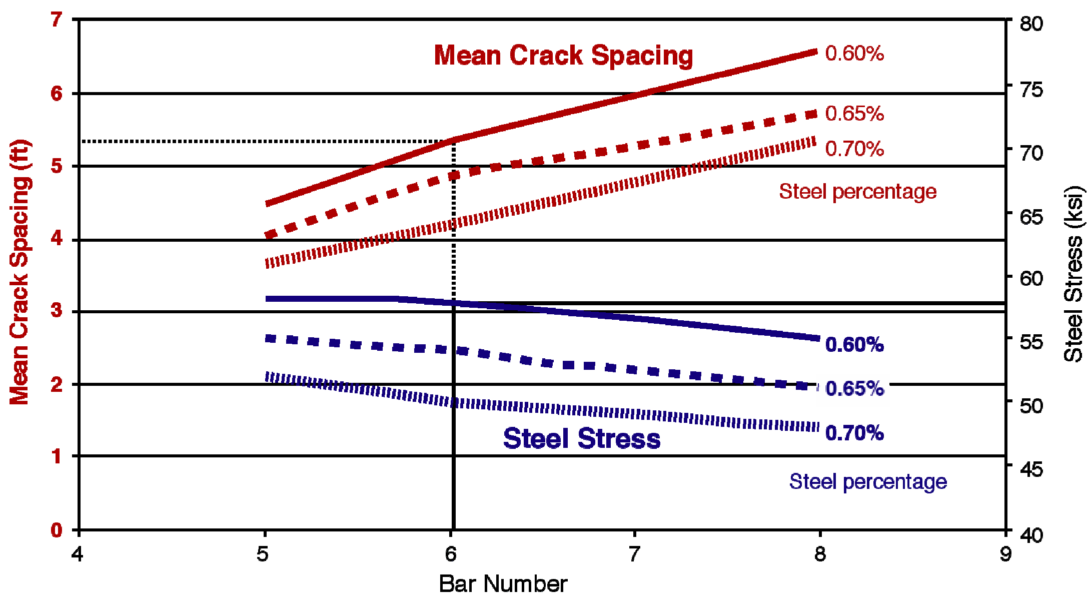


Figure 8.7 Longitudinal steel sensitivity analysis considering mean crack spacing and steel stress

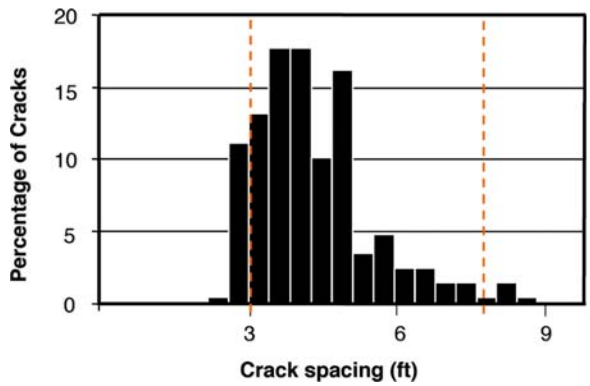


Figure 8.8 Crack distribution diagram

In Figure 8.9, a relationship for the crack width and longitudinal steel stress with the bar number and percentage of steel is illustrated. The crack width, for 0.6 percent steel and number 6 bars, is slightly above 0.04 in., and the steel stress is 56.5 ksi, just within the allowable limit.

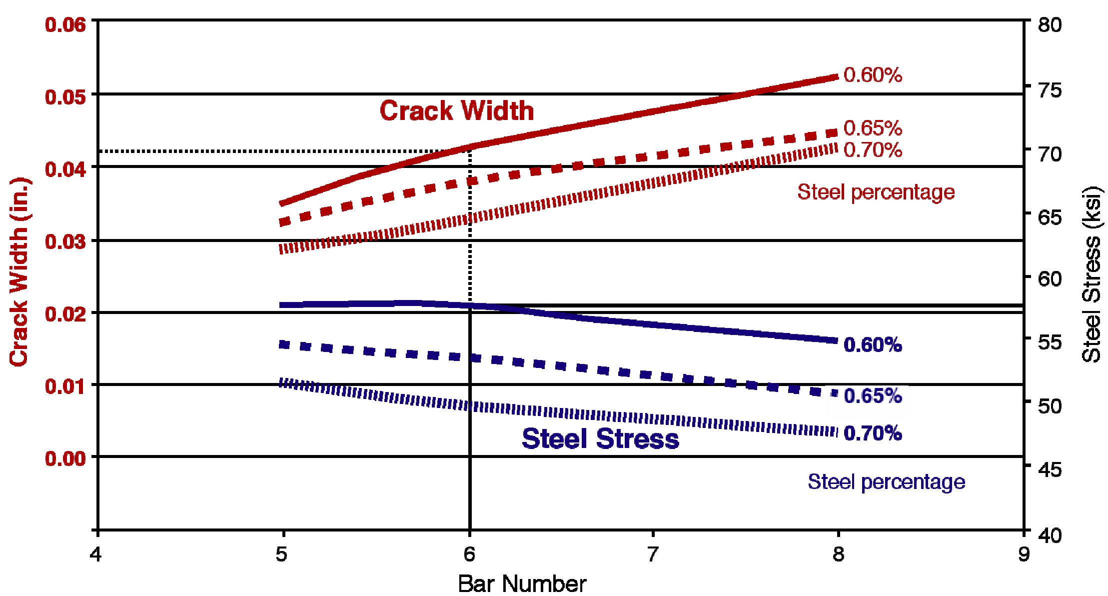


Figure 8.9 Longitudinal steel sensitivity analysis considering steel stress and crack width

Transverse steel designs are indicated on the transverse steel design chart (Figure 8.10), in which the tie-bars and reinforcement are linked by the encircled red numbers to the details shown in the cross section drawings that present the steel design results (Figures 8.11 and 8.12).

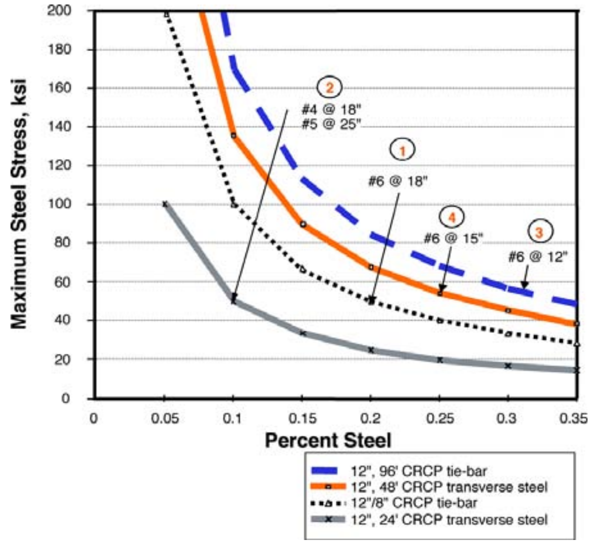


Figure 8.10 Transverse steel sensitivity analysis

8.3.5 Reinforcement Design Results

The reinforcement design results are presented in Figures 8.11 and 8.12.

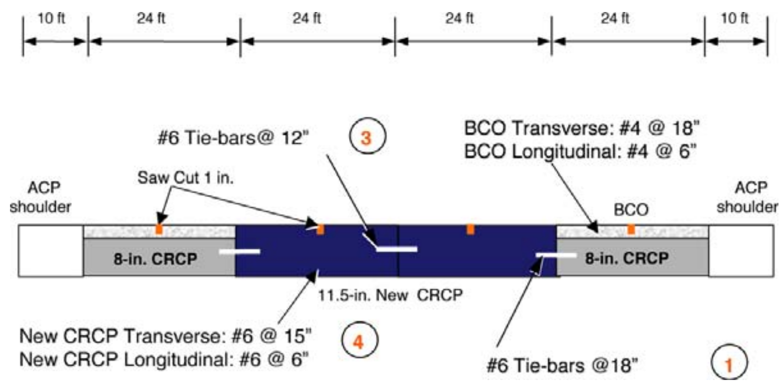


Figure 8.11 96-ft section steel design results

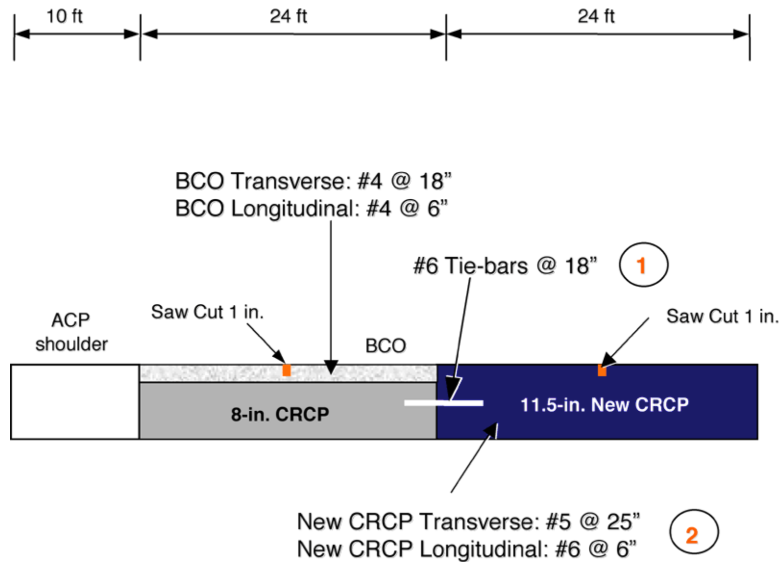


Figure 8.12 48-ft section steel design results

Table 8.11 summarizes the results of the reinforcement design:

Table 8.11 Reinforcement design summary

Reinforcement Description	Tied 96-ft Section		Independent 48-ft Sections	
	Size	Spacing	Size	Spacing
BCO Transverse	#4 bars	18 in.	#4 bars	18 in.
BCO Longitudinal	#4 bars	6 in.	#4 bars	6 in.
New CRCP Transverse	#6 bars	15 in.	#5 bars	25 in.
New CRCP Longitudinal	#6 bars	6 in.	#6 bars	6 in.
Old/New CRCP tie-bars	#6 bars	18 in.	#6 bars	18 in.
New CRCP tie-bars	#6 bars	12 in.	-	-

8.3.6 Summary of Design

The overlay thickness designs were performed following the AASHTO method and RPOD mechanistic method. A 3.5-in. thick BCO was recommended, which corresponded to a 50-year performance period.

The reinforcement design was performed using the programs CRCP8 and JRCP6, for both the BCO and the new CRCP. There were two cases for the new CRCP steel design, one with both directions of the road in separate pavement structures (two 48-ft wide sections), and the other one with a single structure (a 96-ft wide section).

8.4 Construction

As mentioned before, the CRCP rehabilitation design with a BCO was one of the two major components of this project. The other part of the project consisted of constructing new lanes. The road was widened to the inside by enclosing the original depressed median. One lane and one shoulder were built in each direction over the grassy area that used to be the median of the road.

The original roadway configuration is illustrated in Figure 8.13. It consisted of two lanes in each direction, with an inside and an outside shoulder on each side. The widening part added 48 ft to the width of the road, including two lanes and two shoulders.

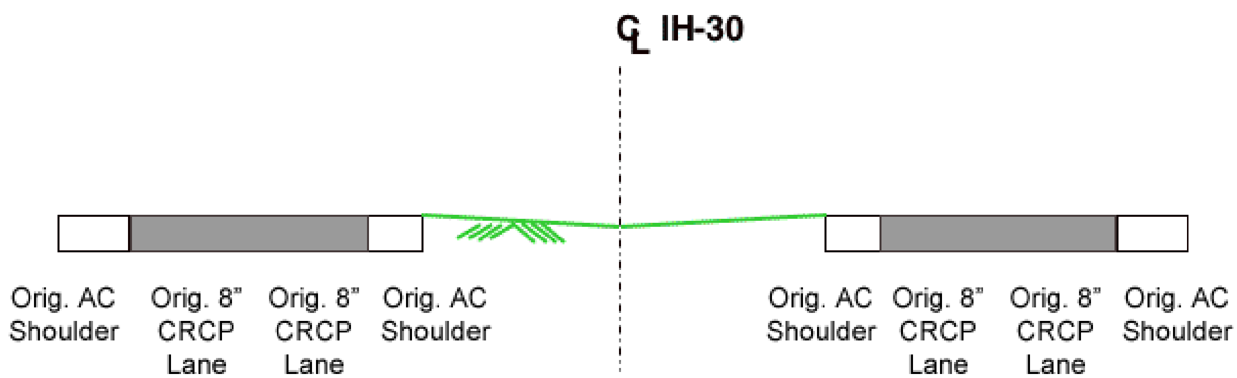


Figure 8.13 Original pavement cross section

The project construction was scheduled in different phases. The first phase was the widening of the roadway using the median area, and the second phase was the BCO placement on the existing pavement.

8.4.1 Phase I: New CRCP Construction

During the first phase of the project, the construction took place in the median and on the inside shoulders of the road, where the new 11.5-in. thick CRCP was constructed. The depressed median was filled to grade with a selected embankment material, and the top 8-in. layer was lime treated. The original median sloped downhill toward the centerline of the highway with an approximate grade of 13:1. On top of the subgrade, a 4-in. thick hot mix asphalt cement subbase and the 11.5-in. concrete slab were built. The existing AC on the inside shoulders of the road was removed to accommodate the new CRCP slab. Concrete traffic barriers were placed at the edges of the inside lanes to separate the construction area from the lanes open to traffic. During that phase, traffic utilized the existing main lanes and the outside shoulders. This allowed for 30 ft to be opened to vehicle travel in each direction (Figure 8.14). This phase took place in the fall of 1997.

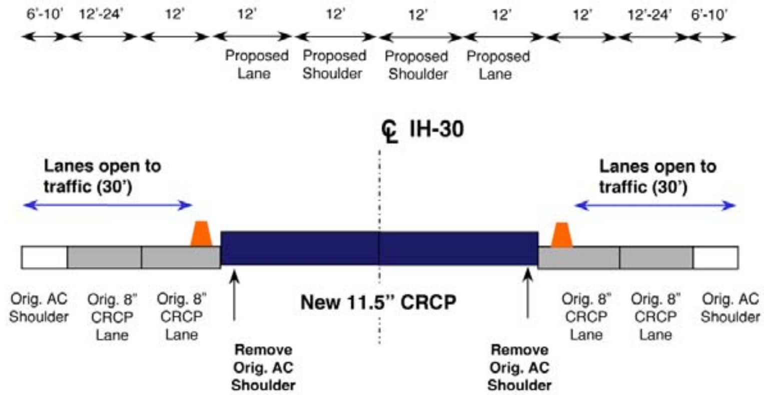


Figure 8.14 Phase I: Construction of median section

8.4.2 Phase II: BCO Construction

For the second phase of the project, the new CRCP in the middle was finished and made ready to accommodate traffic. Concrete traffic barriers were placed at the edges of the new CRCP section to allow a 40-ft section to be open to traffic. In this phase, the rehabilitation of the existing pavement took place. The 3.5-in. thick BCO was placed over the existing 8-in. thick CRCP structure. The outside AC shoulders were overlaid with asphalt to match the grade of the BCO (Figure 8.15).

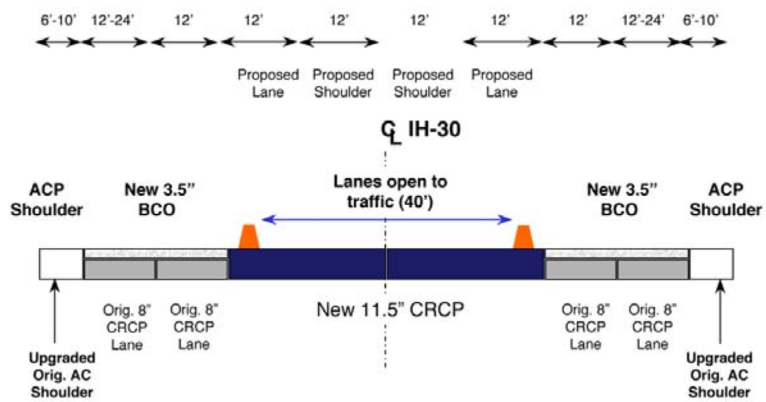


Figure 8.15 Phase II: BCO construction

The concept of an expedited overlay played a major role in minimizing traffic disturbances. Traffic was placed back on the overlay shortly after it was completed. At the end of the construction work, the facility had three full-width primary lanes in each direction with inside and outside shoulders in both directions.

In accordance with the concept of an expedited overlay, the BCO construction was done over several weekends, one section at a time, so as not to disturb the busy weekday traffic. The recently finished BCO section was opened to vehicular traffic by noon on Sundays, approximately 24 hours after the concrete was placed.

8.4.3 Preconstruction Activities

Before the overlay could be placed, the first major operation was the removal of the existing AC overlay. The AC layer was removed with the Skidabradder machine. The removal of the AC overlay was completed for the westbound section on May 11, 1998. The Skidabradder equipment, mentioned in Chapter 6, is designed to remove a ½ in. deep swath in one cut, and the cuts are approximately 6 ft wide. The machine uses a metallic shot that is sprayed on the surface at velocities permitting removal of the concrete. A condition survey was performed on the same day, shortly after the asphalt was removed. No major distresses were recorded during this survey; thus an excellent surface was prepared to accommodate the BCO. Prior to the paving operations, the debris was removed with an air compressor.

8.4.4 Overlay Placement

The BCO construction was scheduled for the following weekend (May 16 and 17) after the cleaning operation, but it had to be postponed because of problems with the mix slump at the concrete plant. These problems were attributed to a dirty coarse aggregate and were solved when the aggregate supplier was changed. The aggregate problems took several weeks to resolve, therefore delaying the overlay construction. The construction took place over several weekends, with the cleaning process and placement of the reinforcement steel starting on Friday night, and the concrete placement starting early on Saturday mornings in order to provide a minimum of 24 hours of curing prior to opening the overlay to traffic. Figure 8.16 is a generalized layout of the timing for the various placement operations.

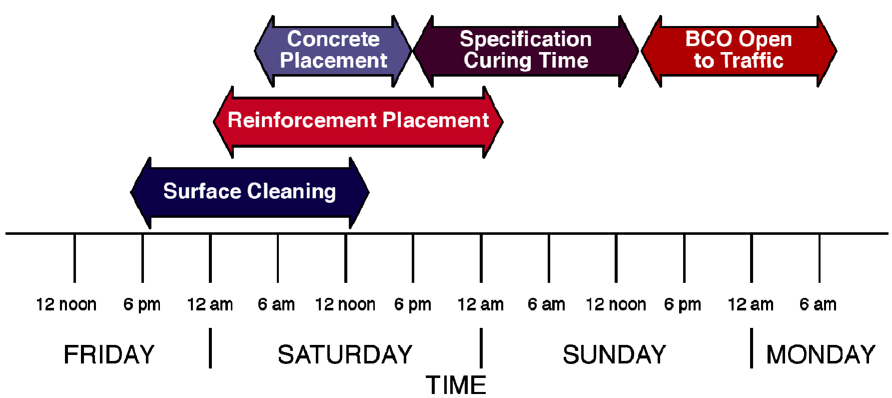


Figure 8.16 Sequencing and timing of construction operations

The BCO placement started with the westbound direction, at the easternmost end of the section, and progressed westbound on that side of the road. After the westbound BCO was finalized, the work began on the eastbound lanes, from the westernmost end of the project. The eastbound lanes were constructed on June 20, June 27, and July 11, 1998, and the eastbound lanes on July 18, and July 25, 1998.

At the end of each weekend's placement, an AC ramp was constructed to provide a transition for traffic from the completed BCO to the existing CRCP.

During construction, there was a weather station on site in one of the TxDOT vehicles. The information collected with the weather station included the ambient temperature, relative humidity, wind speed, and the fresh concrete temperature.

Figures 8.17 and 8.18 illustrate some aspects of the BCO placement activities. Figure 8.17 shows the end of the BCO placement on the weekend of June 20, 1998, before the temporary AC transition ramp was placed.

Figure 8.17 End of construction on June 20

Figure 8.18 shows the finishing operations on the overlay, which included tining.

Figure 8.18 Finishing operations

8.5 Quality Control and Quality Assurance (QC/QA)

CTR and TxDOT personnel supervised the construction activities during the first days of construction. Good construction practices and tests were observed. However, the CTR staff could not be present on all the eastbound lanes' placement dates. The original QC/QA plan for the IH-30 BCO outlined the following field tests:

- Condition survey: to monitor the development of various types of distresses, such as transverse and longitudinal cracks, spalls, and punchouts. The surveys were done on the outside lanes. Figure 8.19 is a plan view of the IH-30 section illustrating the surveyed lanes.
- Present Serviceability Index (PSI) and Profile Tests: to evaluate the riding quality and changes in profile of the pavement on the overlaid lanes (see Figure 8.19).
- Delamination detection: by means of the sounding technique on the overlaid lanes.
- Deflection Tests: to be measured with FWD on the outside lanes.
- Seismic Pavement Analyzer (SPA): The SPA and its portable version (PSPA) were used to measure the pavement stiffness. The PSPA also helped in testing delaminated areas. These tests were conducted at the centerline of the outside lanes.

- Coring: from the wheel path of the outside lane. The laboratory tests conducted on the samples were direct tensile bond, density, and splitting tensile strength.

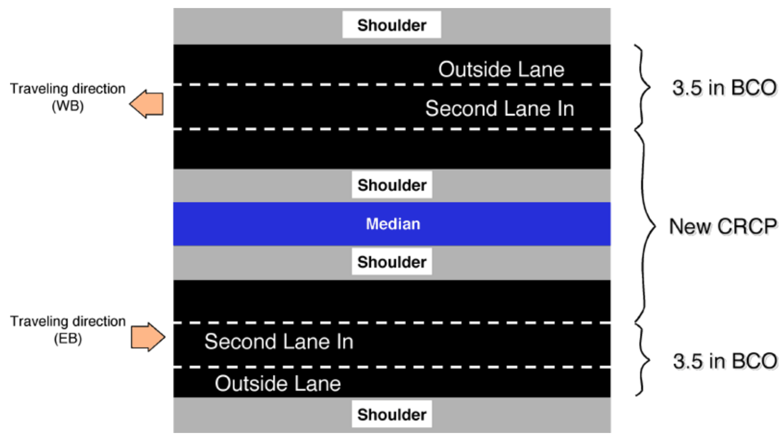


Figure 8.19 Plan view of IH-30

8.5.1 Condition Surveys

On Sunday afternoon, after each section was placed, and before the pavement was opened to traffic, cracks and delamination surveys were conducted. No cracks were detected during these initial surveys. The sounding technique was used to search for delaminations. Each section that was placed was sounded approximately every 6 ft for delaminations. No delaminations were detected and no other distresses were seen in the pavement at that time.

Nevertheless, the presence of a delamination problem in the eastbound lanes was initially detected while conducting a crack survey, deflection tests, and SPA and PSPA tests on February 4, 1999, on the eastbound lanes, as part of the monitoring plan outlined above. However, on that date, the extent of the delamination could not be determined and a new delamination survey was scheduled.

8.5.2 Delamination Survey

On February 23, 1999, a delamination survey was conducted on the eastbound outside lane, by means of the sounding technique. These tests were focused only on the eastbound lanes, as no distresses were found in the westbound direction.

The sounding tests were performed by two surveyors, starting at both edges of the outside lane and moving toward the center of the lane. After a delaminated area was found on either edge, the sounding procedure continued to the center to delineate the area, which was subsequently marked with paint. Figure 8.20 shows a segment in good condition.

Figure 8.20 Eastbound BCO segment with no delaminations

The delamination mechanism followed a typical pattern, starting at the edge of the lane and continuing to the center of the lane. In some cases, there were only isolated delaminations at the edges with limited extension to the center of the lane, whereas in others, the delamination extended over an entire transverse section across the lane. Figures 8.21 through 8.24 illustrate the delamination pattern, with the delaminated areas delineated by paint.

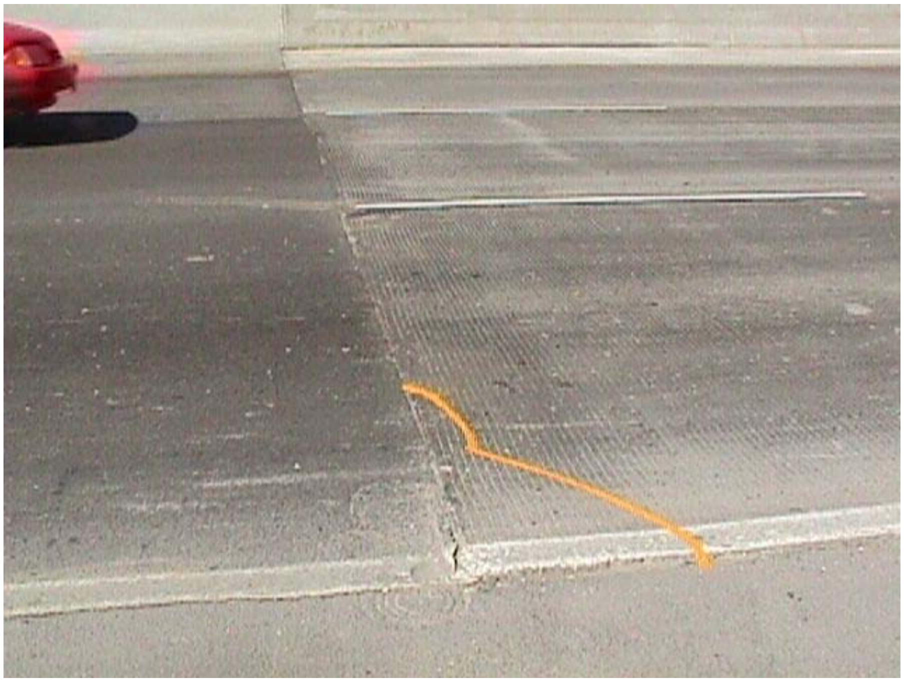


Figure 8.21 Delamination at the edge at the start of the eastbound BCO (station 972+00)

Figure 8.22 Delamination of the outside lane with extensive transverse cracking

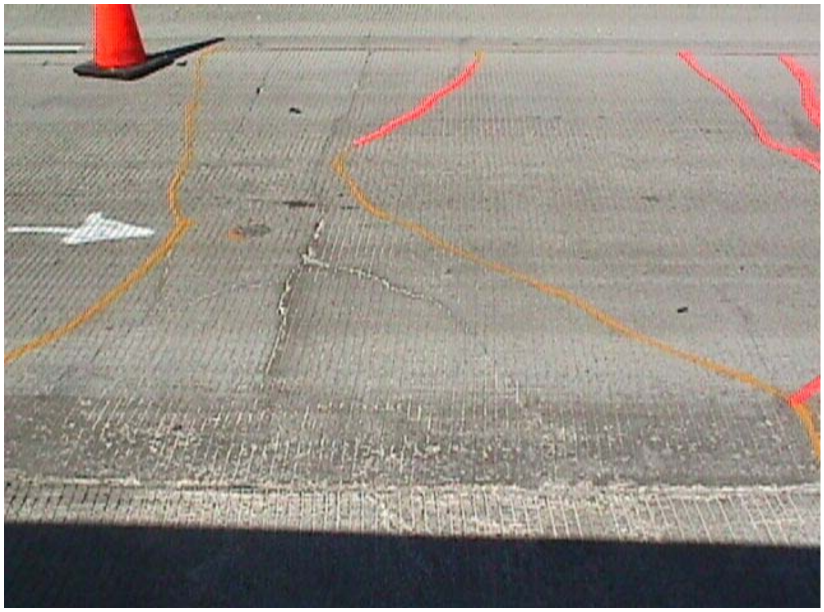


Figure 8.23 Delamination across the lane, where a core sample was extracted

Figure 8.24 Extensive delamination starting at the edges and spreading toward the center of the lane

8.5.3 Delamination Locations

Figure 8.25 shows those locations on the eastbound lanes that presented at least one delaminated area across the lane transverse section. The area could have been at either edge, both edges, the middle of the lane, or a delaminated section covering the entire lane. It also shows the location of cracks in an attempt to establish a correlation between the delaminated areas and the transverse cracks.

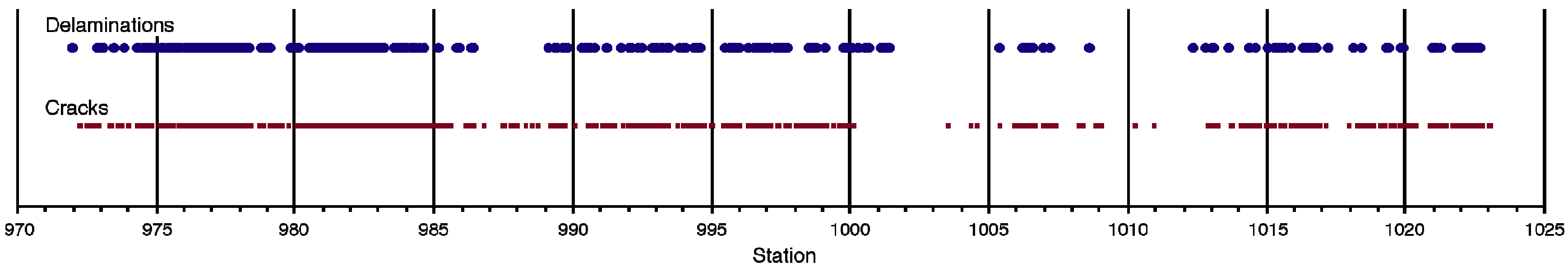


Figure 8.25 IH-30 eastbound delaminated area and transverse crack locations

8.5.4 Condition Survey and Sounding Test Discussion

The majority of the eastbound outside lane presented at least some delamination across its section. Most of the delaminations coincided with cracks, and although the majority of the cracks seemed to be minor, there were a few of them that were spalled. The delamination of the eastbound lanes was a severe problem that led to the implementation of a forensic study to find the cause of the problem. Hence, the subsequent QA testing that took place after the delamination problem arose was undertaken following a forensic approach.

8.6 Summary

The Fort Worth BCO on IH-30 is featured in this chapter as an exemplification of the implementation of the research presented in the preceding chapters. The stages of the BCO development process —namely project selection, design, construction, and QC/QA— are illustrated in detail, as applied to this particular research project. QA tests revealed the occurrence of a delamination problem involving part of the project section. A critical research task was the establishment of the source of the problem, which led to the development of a forensic investigation, presented in the following chapter.

9. Forensic Study

In this chapter, the forensic investigation aimed toward finding the cause of the delamination of the bonded concrete overlay (BCO) project on IH-30 in Fort Worth, Texas is presented. This rehabilitation and widening project was introduced in Chapter 8, with which the BCO development process was demonstrated. The delamination of part of the eastbound lanes' overlay was found during routine monitoring procedures after the overlay had been placed. Extensive testing was required to enable the researchers to find an explanation for the delamination problem. A wide variety of possible causes were investigated. Most of them did not prove to be the source of the problem, but it was only by searching different options that a final conclusion could be reached. This offered a unique opportunity to conduct additional research and increase the knowledge in the area of BCOs.

9.1 Introduction

At the time the first monitoring tests were conducted after the placement of the Fort Worth BCO, the performance of the westbound lanes was excellent, but the eastbound lanes showed a delamination problem that deteriorated over time. Apparently, the placement conditions for both directions had been very similar. The first delaminations were detected during a condition survey, as was shown in the previous chapter, and were confirmed later by sounding tests. The author and a technician from the Center for Transportation Research (CTR) were present during the first construction weekends, during which the westbound lanes were overlaid. However, after the first construction activities were confirmed to be running smoothly, it was deemed unnecessary to continue with the job site visits for the remaining weekends, and only the CTR technician attended the construction to perform visual surveys and sounding tests after the BCO placement activities. The eastbound lanes were placed in the last two weekends of construction. At this point, there was no indication as to what might have caused the problems in the eastbound lanes. CTR and the Texas Department of Transportation (TxDOT) undertook a forensic investigation attempting to determine the cause of the delamination. Several hypotheses were evaluated to find an explanation for the problem. Those investigated as part of the forensic study were as follows:

- Adverse weather conditions during construction, coupled with suboptimal curing precautions.
- Inadequate strength of the concrete.
- Errors or oversights during construction.
- Inadequate surface preparation.
- The time between end of construction and returning the traffic back onto the overlay before it had gained enough strength to carry traffic loadings.

In the following pages, these hypotheses are evaluated.

9.2 Construction Weather

Weather conditions during the overlay construction could have been critical to the overlay performance, especially because the concrete placement occurred in the summer months.

9.2.1 June 20, 1998 (Westbound Placement)

The overlay construction began on June 20, 1998, with the westbound lanes. The section that was built on that day lies between stations 1011+80 and 1024+78.

The paving operations began at 7:15 a.m. The weather station started working at 5:30 a.m. with the equipment check. Figure 9.1 shows the calculated evaporation rates during paving on June 20. The dashed line shows the critical evaporation rate (0.2 lb/sq.ft/hr). As the figure shows, this value was surpassed during most of the paving time. For values above this rate, special curing should be applied, such as fogging, which was used in this case. The ambient temperatures recorded during construction are shown in Figure 9.2. Unfortunately, no weather information was recorded for the day following construction; therefore, the daily temperature differential could not be verified from this plot.

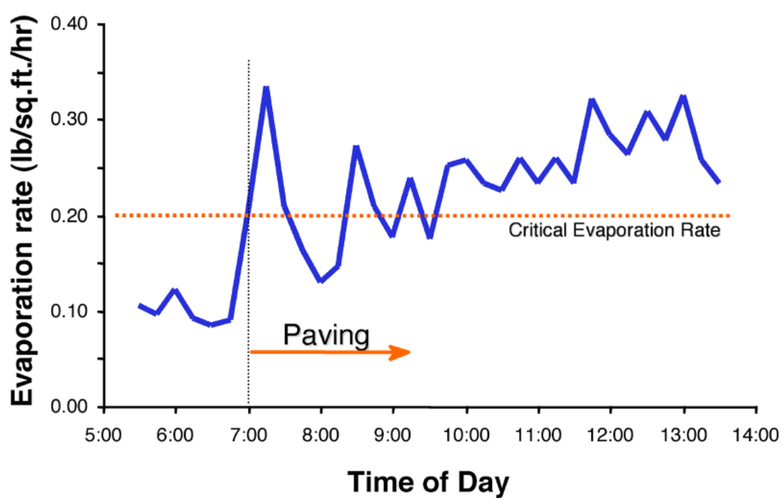


Figure 9.1 Calculated evaporation rates from weather station data for June 20 (westbound placement)

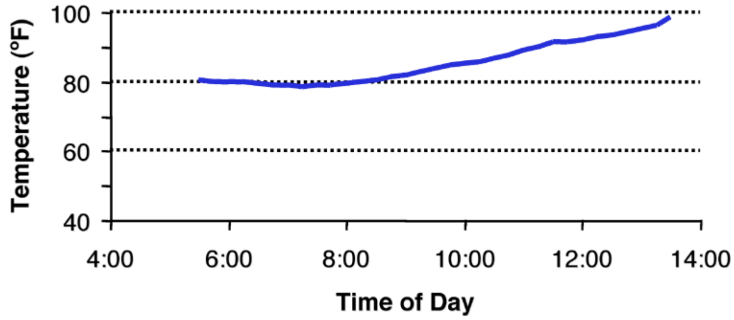


Figure 9.2 Ambient temperatures for June 20 (westbound placement)

9.2.2 June 27, 1998 (Westbound Placement)

Construction continued on the following weekend, June 27. The westbound section that was completed is between stations 1012+30 and 999+09. The concrete placement started just after 4 a.m. and had to be finished at 10 a.m., because of an accident on the access road in the westbound direction. Figure 9.3 shows the calculated evaporation rates for June 27, and Figure 9.4 plots the ambient temperature during construction. Evaporation rates during paving stayed well below the critical value of 0.2 lb/sq.ft/hr, and the daily ambient temperature differential could not be verified because of the unavailability of weather records for the days following construction.

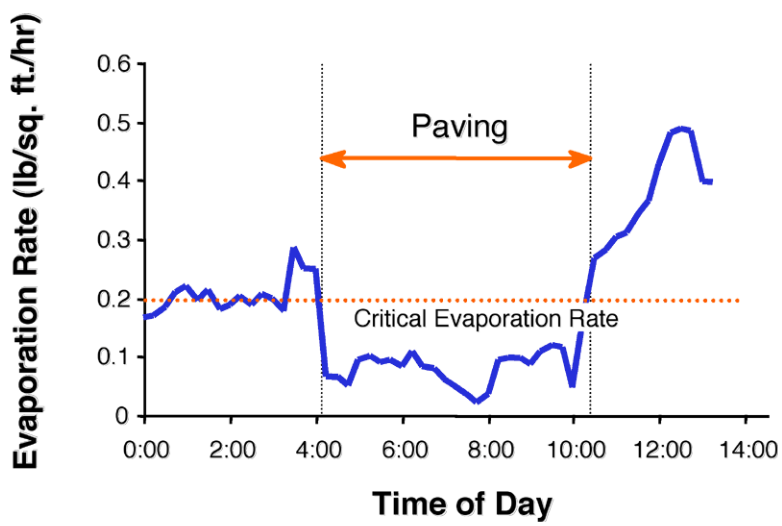


Figure 9.3 Calculated evaporation rates from weather station data for June 27 (westbound placement)

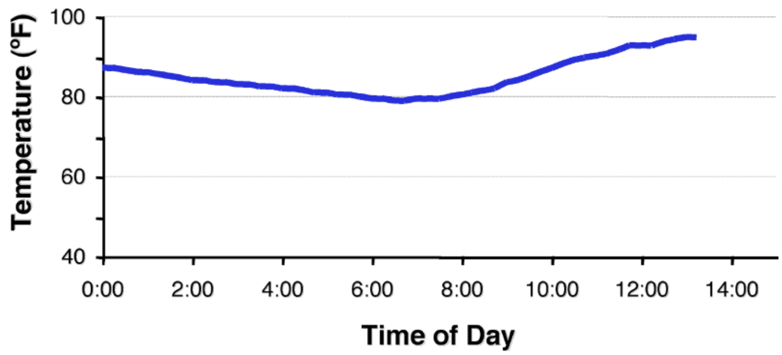


Figure 9.4 Ambient temperatures for June 27 (westbound placement)

9.2.3 July 11, 1998 (Westbound Placement)

The overlay construction continued on July 11, 1998, at station 999+09 and finished at 1:30 p.m., with the completion of the westbound lanes at station 972+00. Figures 8.30, 8.31, and 8.32 show the calculated evaporation rates for the construction day, a day after construction, and two days after construction, respectively. Figure 9.5 indicates that during paving, evaporation rates were not critical, except for a few minutes in which the evaporation rate slightly surpassed the critical value.

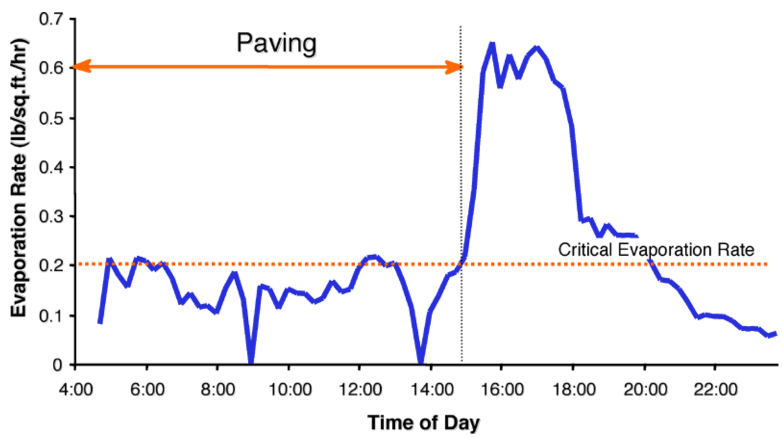


Figure 9.5 Calculated evaporation rates from weather station data for July 11 (westbound placement)

Figure 9.6 displays the daily temperature variations for the construction day and for the following two days; the daily temperature differential following construction did not exceed 25 °F.

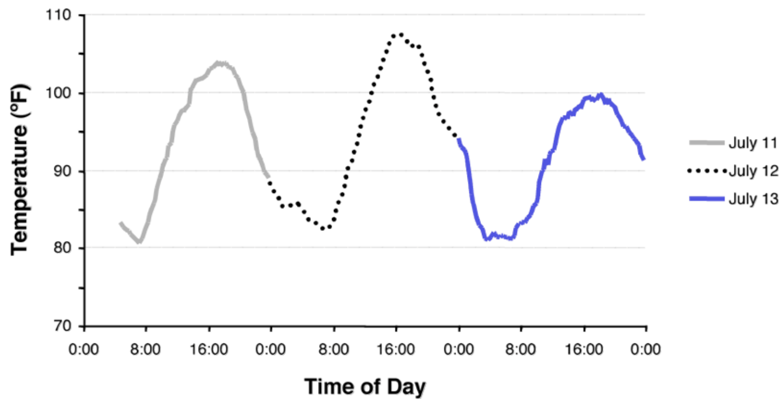


Figure 9.6 Ambient temperatures for July 11-13 (westbound placement)

9.2.4 July 18, 1998 (Eastbound Placement)

July 18 was the first day of placement of the eastbound lanes' BCO. Construction started at station 972+00 and finished for the day at station 1002+86. The evaporation rate plot is shown in Figure 9.7. Critical evaporation rates were not reached while the paving activities lasted.

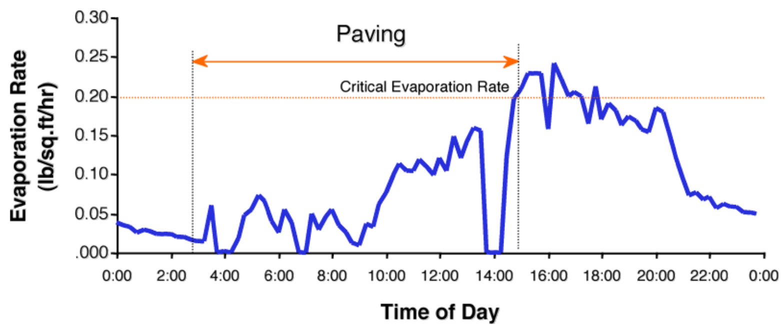


Figure 9.7 Calculated evaporation rates from weather station data for July 18 (eastbound placement)

Daily temperature differentials did not reach critical levels during this construction period as is shown in Figure 9.8, because they did not surpass the 25 °F differential from one day to the next.

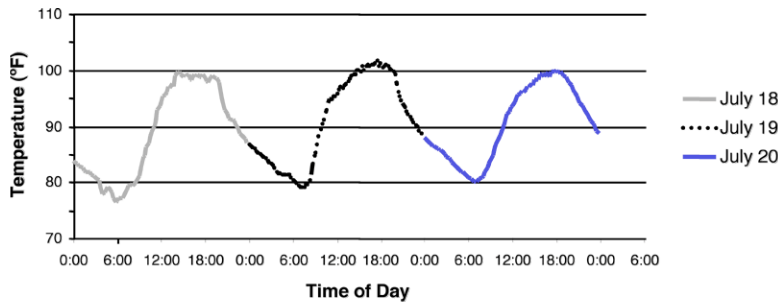


Figure 9.8 Ambient temperatures for July 18-20 (eastbound placement)

9.2.5 July 25, 1998 (Eastbound Placement)

The last section of the BCO was completed on July 25 at 11:45 a.m. The section that was constructed lies from station 1002+86 to station 1024. Unfortunately, no weather station records were available for this date.

9.2.6 Summary of Weather Conditions

The weather conditions were generally acceptable for all the periods of overlay placement. However, on the first construction day the conditions were critical, resulting in potentially undesirable evaporation rates for the fresh pavement. Daily temperature differentials did not appear to have been hazardous for the pavement. Weather conditions at the placement time were thought to be the cause of the eastbound delamination problem that was detected, but neither evaporation rates nor daily temperature differentials surpassed critical values, except for the first westbound construction date. Nevertheless, weather conditions could have been a harmful factor on the last construction weekend (eastbound placement), for which no weather information was available.

9.3 In Situ Samples

As part of the QC/QA plan, core samples were taken from both directions of the rehabilitated pavement section. The cores included both the overlay and the original pavement.

The original plan for sampling was to extract fifteen cores in each direction from the centerline of the outside lanes of the new overlay, which corresponds to drilling approximately every 450 ft. along the 1-mi long stretch. However, because of traffic control difficulties with entrance and exit ramps, that number of samples could not be attained. Most of the cores were drilled at the same spots on which FWD and SPA measurements were performed.

Twelve cores were extracted from the westbound outside lane on Thursday, January 23, 1999. Figure 9.1 shows the total thickness and the BCO thickness for these core samples, as well as comments on the condition of the cores. Nine specimens had some steel reinforcement in them, and one core had a crack in the BCO. None of the samples was delaminated when extracted, and their overall condition was very satisfactory.

Table 9.1 Westbound cores thickness

Core #	Total Thickness (in.)	BCO Thickness (in.)	Comments
1	$10\ ^{3}/_{4}$	$4\ ^{3}/_{4}$	No steel
2	$12\ ^{1}/_{4}$	5	Rebar in BCO
3	$11\ ^{1}/_{4}$	$4\ ^{1}/_{8}$	Rebar in BCO and CRCP
4	11	$4\ ^{3}/_{4}$	Rebar in BCO and CRCP
5	11	$3\ ^{3}/_{4}$	Rebar in BCO and CRCP
6	$10\ ^{1}/_{2}$	$3\ ^{3}/_{4}$	Rebar in BCO
7	$11\ ^{1}/_{4}$	$4\ ^{5}/_{8}$	Rebar in BCO and CRCP
8	12	$4\ ^{3}/_{8}$	No steel
9	$11\ ^{3}/_{4}$	$4\ ^{5}/_{8}$	Rebar in BCO and CRCP
10	$11\ ^{3}/_{8}$	$4\ ^{1}/_{2}$	Rebar in BCO. BCO cracked
11	12	$4\ ^{7}/_{8}$	Rebar in CRCP
12	11	$4\ ^{1}/_{4}$	No steel

Figure 9.9 illustrates one of the westbound cores, in which both the overlay and the old pavement are clearly distinguishable.

Figure 9.9 Westbound core #5

Twelve cores were obtained from the eastbound lane on Thursday, February 4, 1999. The thicknesses and comments about the cores are shown in Figure 9.2. Unlike the westbound specimens, by the time the eastbound cores were extracted, it was evident that there was a delamination problem in the eastbound lanes, and this is reflected in the condition of the cores.

Table 9.2 Eastbound cores thickness

Core #	Total Thickness (in.)	BCO Thickness (in.)	Comments		
1	$10\,{}^{3}/_{4}$	$3\,{}^{3}/_{4}$	Rebar in BCO and CRCP.	No delamination.	
2	$11\,{}^{1}/_{2}$	$4\,{}^{3}/_{4}$	Rebar in BCO and CRCP.	BCO delaminated.	
3	$11\,{}^{1}/_{4}$	$3\,{}^{7}/_{8}$	Rebar in BCO and CRCP.	BCO delaminated.	
3-A	11	$3\,{}^{3}/_{4}$	Rebar in CRCP.	BCO delaminated.	BCO cracked
4	$10\,{}^{1}/_{2}$	$3\,{}^{1}/_{2}$	Rebar in CRCP.	No delamination.	
5	12	$4\,{}^{1}/_{4}$	Rebar in BCO and CRCP.	No delamination.	Bottom of core is bigger: $4\,{}^{1}/_{8}$
6	$11\,{}^{7}/_{8}$	$4\,{}^{7}/_{8}$	Rebar in BCO and CRCP.	BCO delaminated.	
7	$11\,{}^{7}/_{8}$	$4\,{}^{1}/_{2}$	Rebar in BCO.	No delamination.	
8	$11\,{}^{1}/_{2}$	$4\,{}^{1}/_{8}$	Rebar in BCO and CRCP.	BCO delaminated.	CRCP cracked
9	12	5	Rebar in BCO.	No delamination.	
9-A	$10\,{}^{1}/_{2}$	4	Rebar in CRCP.	No delamination.	
10	$11\,{}^{5}/_{8}$	4	Rebar in BCO and CRCP.	No delamination.	

Eastbound cores 3-A and 9-A were drilled after all the other cores had been extracted. Their location was between those of cores 3 and 4, and cores 9 and 10, respectively; hence their designation with a number and a letter. Figure 9.10 shows one of the delaminated cores.

Figure 9.10 Debonded eastbound core

9.3.1 Testing Plan

The cores were tested for splitting tensile strength, density, and direct tensile bond pulloff strength by the Construction Materials Research Group.

9.3.2 Splitting Tensile Strength

Both the overlay and the existing pavement portions of the cores were tested for splitting tensile strength. The results for the westbound lane overlay and the existing pavement are shown in Tables 9.3 and 9.4, respectively.

Table 9.3 Westbound BCO splitting tensile strength

Specimen ID	Tensile Strength (psi)
WB 1	910
WB 2	790
WB 3	1,050
WB 4	935
WB 5	840
WB 6	855
WB 7	890
WB 8	780
WB 9	825
WB 10	core was cracked
WB 11	920
WB 12	845
Mean	876
Std.Dev	73
C.V.(%)	8.35

Table 9.4 Westbound original pavement splitting tensile strength

Specimen ID	Tensile Strength (psi)
WB 1	645
WB 2	755
WB 3	625
WB 4	core was cracked.
WB 5	580
WB 6	705
WB 7	780
WB 8	695
WB 9	655
WB 10	695
WB 11	core was cracked
WB 12	710
Mean	685
Std.Dev	60
C.V.(%)	8.75

The results for the eastbound lane overlay and the existing pavement are shown in Tables 9.5 and 9.6, respectively.

Table 9.5 Eastbound BCO splitting tensile strength

Specimen ID	Tensile Strength (psi)
EB1	935
EB2	740
EB3	755
EB3-A	core was cracked
EB5	970
EB6	790
EB7	1,050
EB8	710
EB9	805
EB9-A	780
EB10	785
Mean	832
Std.Dev.	112
C.V.(%)	14

Table 9.6 Eastbound original pavement splitting tensile strength

Specimen ID	Tensile Strength (psi)
EB1	550
EB2	750
EB3	780
EB3-A	650
EB4	785
EB5	675
EB6	610
EB7	620
EB8	core was cracked
EB9	855
EB9-A	645
EB10	710
Mean	694
Std.Dev.	91
C.V.(%)	13

Table 9.7 presents a summary of the splitting tensile strength results for both directions of IH-30, including the original pavement as well as the overlay.

Table 9.7 Summary of splitting tensile strength tests results

	WB		EB	
	Original CRCP	Overlay	Original CRCP	Overlay
Mean	685	876	694	832
Std.Dev	60	73	91	112
C.V.(%)	8.7	8.4	13.1	13.5

The results of the core testing revealed that the strength of the pavement was adequate. Figure 9.11 shows a plot of the results, in which the cumulative frequency of the splitting tensile strength of the cores is displayed. It shows that the BCO concrete in both directions was stronger than the original pavement, with similar results in both directions.

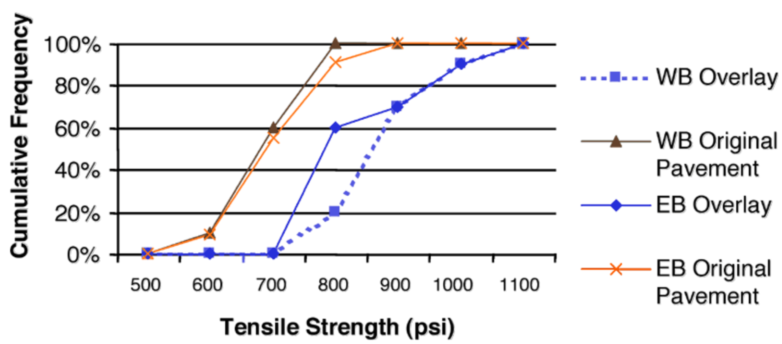


Figure 9.11 Splitting tensile strength cumulative frequency

9.3.3 Direct Tensile Bond Strength

This test, also known as the pulloff test, measures the strength at the interface between the original pavement and the overlay. When the cores from the eastbound lane, were extracted some of them came out debonded (i.e., the overlay layer was delaminated from the original pavement). Such cases have zero bond strength. Tables 9.8 and 9.9 present the pulloff test results for the cores taken from the westbound and eastbound lanes, respectively.

Table 9.8 Bond strength of westbound cores

Specimen ID	Bond Strength (psi)
WB 1	200
WB 2	110
WB 3	200
WB 4	core cracked
WB 5	210
WB 6	170
WB 7	160
WB 8	220
WB 9	epoxy failure at cap
WB 10	50
WB 11	120
WB 12	110
Mean	155
Std. Dev	56
C.V.(%)	35.89

Table 9.9 Bond strength of eastbound cores

Specimen ID	Bond Strength (psi)
EB1	170
EB2	Already debonded
EB3	Already debonded
EB3-A	Already debonded
EB4	130
EB5	90
EB6	Already debonded
EB7	70
EB8	Already debonded
EB9	170
EB9-A	30
EB10	230
Mean	74
Std. Dev	83
C.V.(%)	111.70

There is a noticeable difference between the bond strength of the cores for the westbound and eastbound lanes. Most of the eastbound cores, if not delaminated (40 percent of the cores had zero bond strength), proved to be very weak at the interface. Only one of the westbound cores (WB10) had very low bond strength. The chart in Figure 9.12 shows the cumulative frequency of the bond strength of the cores.

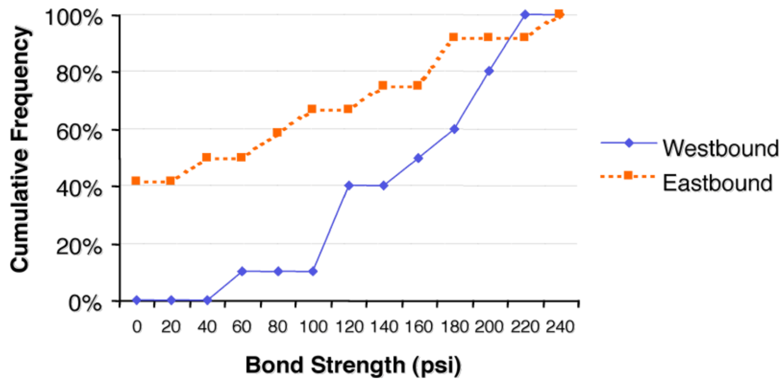


Figure 9.12 Bond strength cumulative frequency

9.3.4 Density

For the density test, fragments of both the overlay and the existing pavement were tested. Not all the specimens were tested. The westbound and eastbound density test results are presented in Tables 9.10 and 9.11.

Table 9.10 Westbound density test results

Specimen ID	Density (pcf)	
	CRCP	Overlay
WB 1	140	139
WB 2	138	141
WB 3	141	140
WB 4	*	142
WB 5	*	139
WB 6	138	137
WB 7	141	142
WB 8	140	*
WB 9	138	137
WB 10	*	Cracked
WB 11	*	*
WB 12	141	141
Mean	140	140
Std.Dev.	1	2
C.V.(%)	1.00	1.41

* Core not tested

Table 9.11 Eastbound density test results

Specimen ID	Density (pcf)	
	CRCP	Overlay
EB1	*	140
EB2	138	142
EB3	142	142
EB3-A	143	135
EB4	145	142
EB5	142	139
EB6	140	137
EB7	*	145
EB8	*	141
EB9	143	138
EB9-A	141	140
EB10	132	139
Mean	141	140
Std.Dev.	4	3
C.V.(%)	2.76	1.81

* Core not tested

The results were very similar in both directions. Likewise, densities were almost the same between the CRCP and the overlay. There was very small variability among the results, as the coefficients of variation show. The figures were very satisfactory, demonstrating that the pavement had no problems regarding its density. Figure 9.13 shows the cumulative frequency of the density tests.

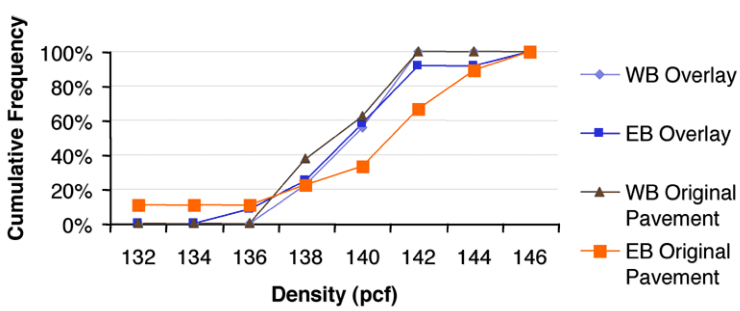


Figure 9.13 Density test cumulative frequency

9.3.5 Summary of In Situ SampleTesting

The destructive tests performed on the IH-30 pavement revealed that the problem causing the eastbound lanes' delamination was related only to the interface between the original CRCP and the overlay. The bond strength at this interface was very low or non-existent in the eastbound cores as compared with the westbound samples. The bond strength of the eastbound cores was less than half of the strength of the westbound cores. Splitting tensile strength tests performed on the cores showed that the concrete had adequate strength in both traveling directions. Likewise, the density tests yielded positive results. Therefore, the concrete itself was not the source of the problem, because its properties were satisfactory; it was the interface that was defective. At this point, further testing was deemed necessary to ascertain the cause of the delamination. Petrographic tests were proposed to be conducted on the remnants of these cores.

9.4 Deflection Testing

Deflection tests were originally scheduled as part of the monitoring plan before the delamination problem appeared, because deflections are an essential indication of the structural integrity of the pavement. Initially, only FWD testing was planned. However, as the forensic investigation progressed and the results were still unclear, it was considered appropriate to incorporate the use of the Rolling Dynamic Deflectometer (RDD) to evaluate deflections in a semi-continuous way along the project section.

9.4.1 FWD Testing

Westbound FWD tests were conducted on January 21, 1999, and the eastbound tests were performed on February 4, 1999. The FWD measurements were taken at approximately 225-ft. intervals, taking two sets of deflections at every interval. One was for LTE evaluation, and the other set of measurements was taken at the midspan between two cracks, as mentioned in Chapter 5. These deflections were used to backcalculate the moduli of elasticity of the pavement layers. In the next few paragraphs, the FWD results are presented.

9.4.2 Westbound Deflections

Midspan westbound deflections indicated a very satisfactory load-carrying capacity by the pavement structure, as is shown in Figure 9.14, in which the measurements correspond to Sensor 1. The low deflections suggest good structural integrity of the section, where the average deflection was 2.3 mils and the highest measurement was 3.5 mils.

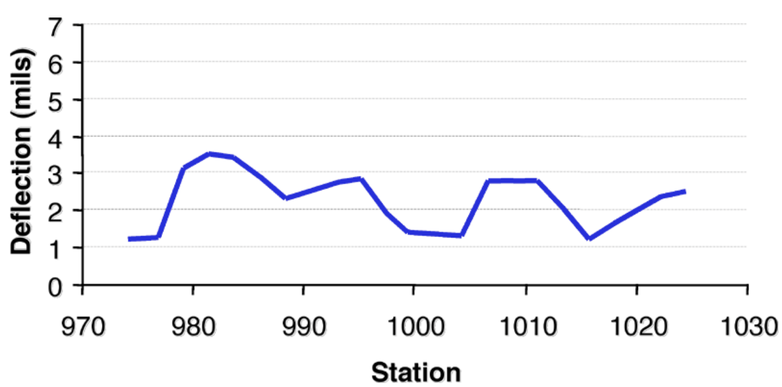


Figure 9.14 Midspan westbound deflections

The LTE also supports the statement about the good structural quality of the section, with average values of 95 percent and 91 percent, respectively, for the Teller and the Darter equations, introduced in Chapter 5 as Equations 5.7 and 5.8, respectively. Figure 9.15 illustrates the LTE variation along the westbound section for both calculation procedures. The westbound LTE values are shown also in Table 9.12.

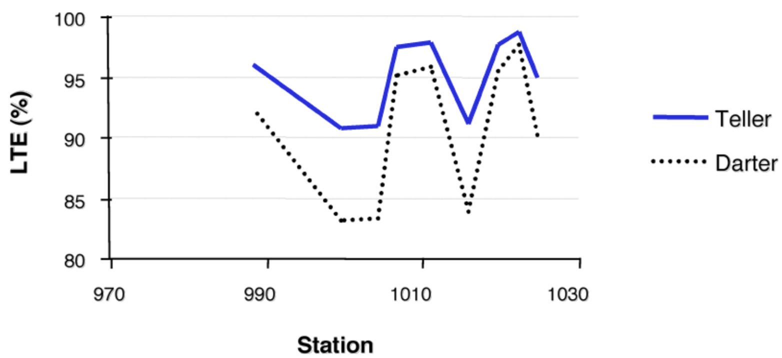


Figure 9.15 Westbound load transfer efficiency

Table 9.12 Westbound LTE values by station

Station	LTE	
	Teller	Darter
1024.78	94.87	90.24
1022.28	98.80	97.62
1019.76	97.65	95.41
1015.76	91.18	83.78
1011.17	97.84	95.77
1006.70	97.45	95.04
1004.35	90.91	83.33
999.52	90.79	83.13
988.42	96.02	92.34
Mean	95.06	90.74
Std. Dev.	3.27	5.88
Coeff.Var.(%)	3.44	6.48

9.4.3 Eastbound Deflections

Unlike the westbound lanes, the eastbound BCO did not perform as well in the deflection testing, averaging 2.8 mils, but with a standard deviation of 1.4 and a coefficient of variation of 50 percent. These large variations in the midspan first sensor deflections can be seen in Figure 9.16.

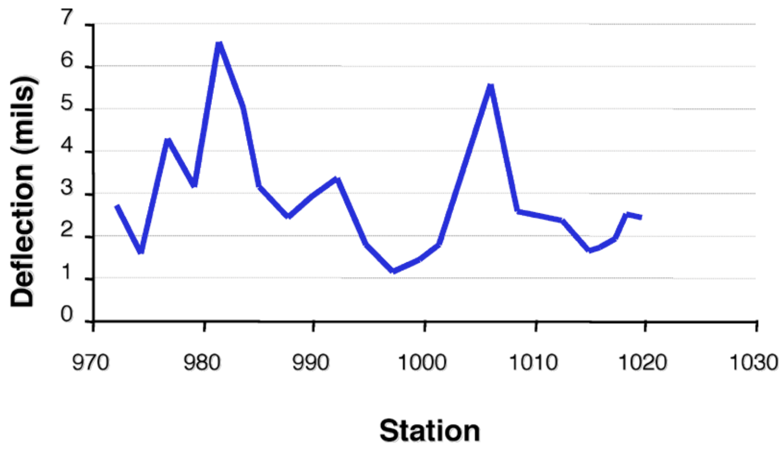


Figure 9.16 Midspan eastbound deflections

Likewise, the eastbound LTE values were poor, indicating a lack of structural soundness in the pavement, with the suboptimal load transfer capability reflecting the delamination of the BCO. Average LTE values for the section were 84 percent (Teller) and 75 percent (Darter). These are shown in Figure 9.17.

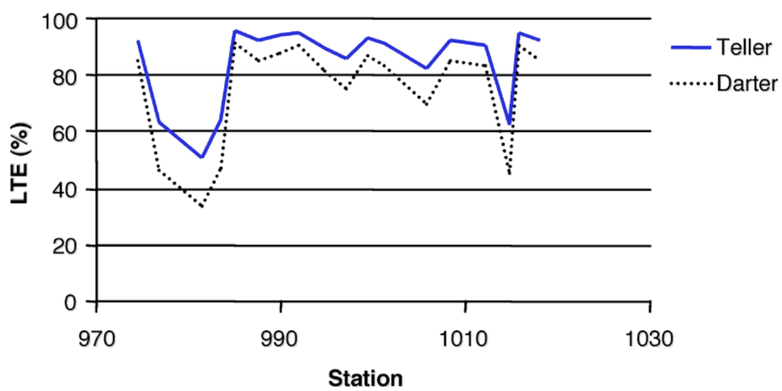


Figure 9.17 Eastbound load transfer efficiency

Eastbound LTE values by station are presented in Table 9.13.

Table 9.13 Eastbound LTE values by station

Station	LTE	
	Teller	Darter
974.51	91.57	84.44
976.89	62.94	45.92
981.51	50.07	33.40
983.66	64.16	47.24
985.16	95.14	90.72
987.60	91.55	84.41
989.89	93.40	87.63
992.11	94.63	89.81
994.78	89.29	80.65
997.11	85.59	74.81
999.54	93.00	86.92
1001.32	90.70	82.98
1006.01	81.70	69.07
1008.36	91.55	84.41
1012.37	90.53	82.69
1014.86	62.15	45.08
1015.87	94.88	90.26
1018.28	91.49	84.31
Mean	84.13	74.71
Std. Dev.	14.02	18.45
Coeff.Var.(%)	16.66	24.69

9.4.4 Westbound versus Eastbound Deflection Comparison and Subgrade Analysis

It was evident that the eastbound deflections reflected the delamination problem of the BCO by showing higher values throughout most of the overlaid section. However, in this analysis it was already known that there was a problem, and the objective was to find why it occurred. To ascertain whether the delamination could have occurred as a consequence of poor subgrade conditions, both westbound and eastbound deflections were plotted together; similar deflection patterns could lead to a conclusion that a poor subgrade was most likely to occur at the same locations in both directions (Figure 9.18). Also, the most distant sensor from the source load in the FWD device, Sensor 7, would reflect the subgrade properties more accurately: the farther the measurement is taken from the load, the less the amount of upper layer properties is reflected in the deflection. Hence, by plotting Sensor 7, it was intended to characterize the subgrade properties. This graph is presented in Figure 9.19.

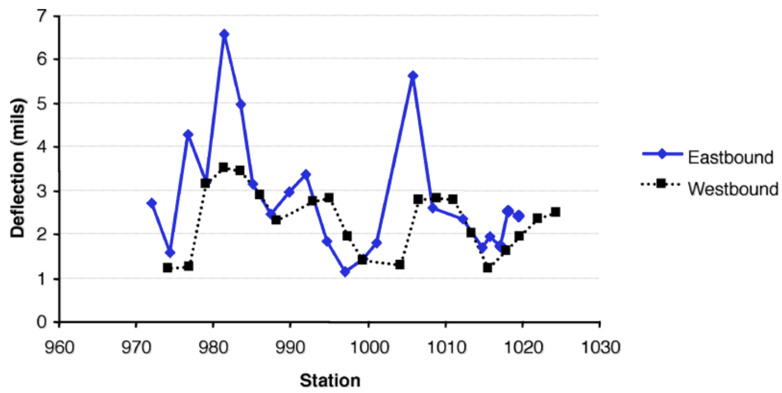


Figure 9.18 Westbound and eastbound FWD deflections, Sensor 1

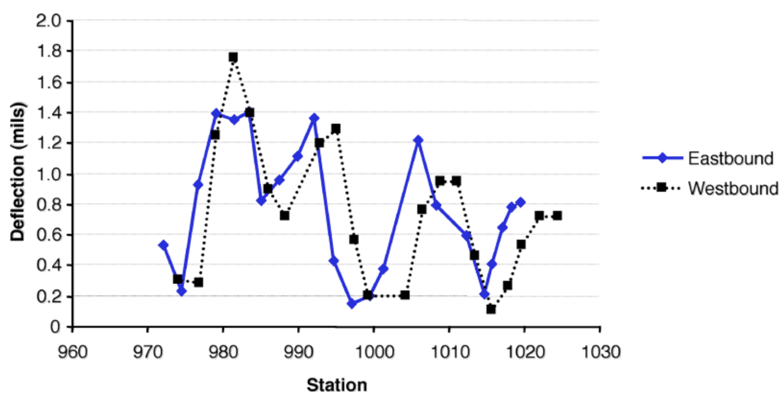


Figure 9.19 Westbound and eastbound FWD deflections, Sensor 7

The plot of Sensor 1 (Figure 9.18) shows a similar deflection pattern for both directions, with the obvious difference being the peaks in the eastbound direction. Figure 9.19, featuring the subgrade properties, indicates practically no difference between eastbound and westbound. With this evidence, it was concluded at this point that there was no reason to believe that the subgrade could have been a decisive factor in the eastbound BCO delamination. It is interesting to note how in both plots it seems that the westbound deflections are slightly shifted to the right with respect to the eastbound measurements. This could have been the result of an error in locating the stations at the time the deflections were taken.

Another way to demonstrate that the cause of the delamination was not the subgrade is to plot both Sensor 1 and Sensor 7 deflections by location (station), with eastbound and westbound in different axes, as is shown in Figure 9.20. Ideally, all the deflection points should fall along the 45° dotted line, which would mean that the deflections are similar in both directions regardless of the location. This was true for Sensor 7, which implies that the subgrade was similar in both directions, but did not hold true for Sensor 1, and this is because of the delamination problem of the overlay in the eastbound lanes.

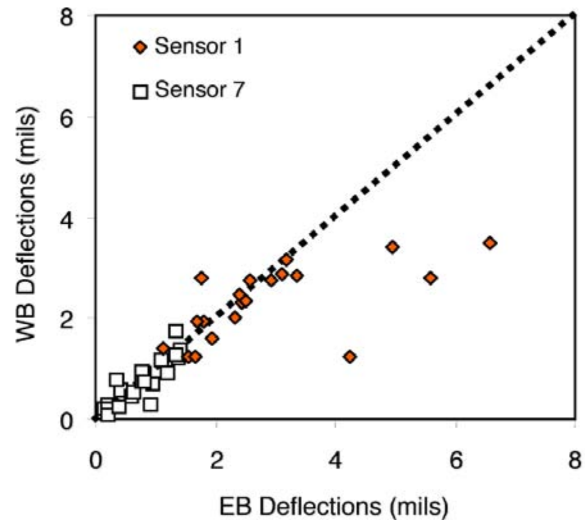


Figure 9.20 FWD tests in both directions, Sensors 1 and 7

9.5 Modulus Backcalculation

9.5.1 Westbound

As was expected, the westbound moduli of elasticity of the pavement layers indicated that the structure had adequate stiffness. These, including the subgrade moduli, are shown in Figure 9.21. The moduli means, standard deviations, and coefficients of variation for the four structure layers are presented in Table 9.14.

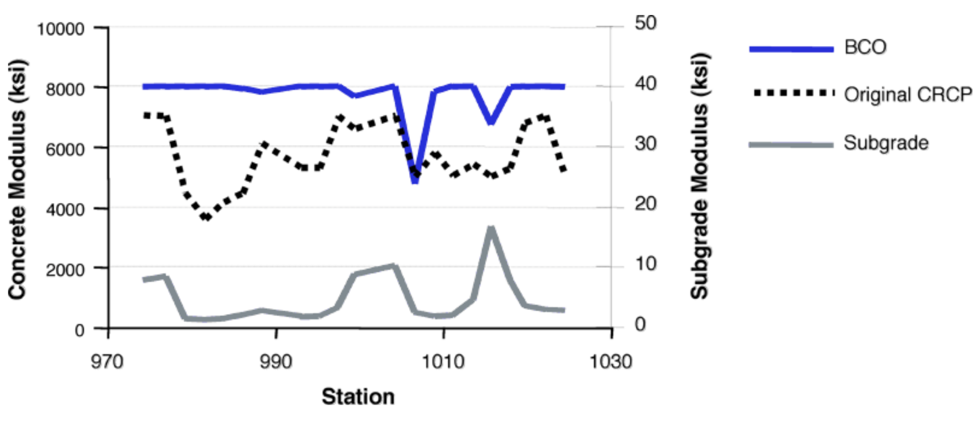


Figure 9.21 Westbound moduli of elasticity

Table 9.14 Westbound moduli means, standard deviations, and coefficients of variation (ksi)

	BCO	CRCP	Subbase	Subgrade
Mean	7747	5640	73	4.6
Std.Dev.	736	1061	23	4.0
Var. Coeff(%)	10	19	31	87

These figures are indicative of a good quality pavement structure. However, it is worth noting the high variability of the moduli of the subgrade layer.

9.5.2 Eastbound

As the backcalculated eastbound moduli came from the deflections, the moduli reflected the delamination problem, as is illustrated in Figure 9.22, which shows weak layer stiffness for the overlay. Averages and variabilities of these backcalculated stiffnesses are presented in Table 9.15.

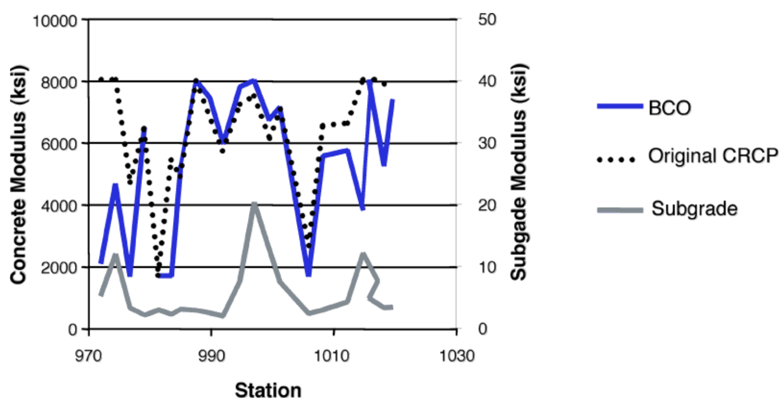


Figure 9.22 Eastbound moduli of elasticity

Table 9.15 Eastbound moduli means, standard deviations, and coefficients of variation (ksi)

	BCO	CRCP	Subbase	Subgrade
Mean	5459	6507	73	5.8
Std.Dev.	2349	1764	26	4.7
Var. Coeff(%)	43	27	36	81

These BCO moduli are lower than the westbound ones, but when considering the underlying structure, the eastbound pavement seemed to be in better shape than the westbound. In fact, the hypothesis that the lack of support from the subgrade could have been a reason for the delamination occurrences in the eastbound lanes proves to be untrue. The eastbound subgrade appeared to be stiffer than the westbound subgrade (Figure 9.23), although there is high variability associated with the subgrade on both sides of the road. Therefore, from these moduli, it can be concluded that the debonding of the eastbound BCO was not a result of the performance of the underlying structure (i.e., original pavement, subbase, and subgrade), but it was an effect of the overlay itself.

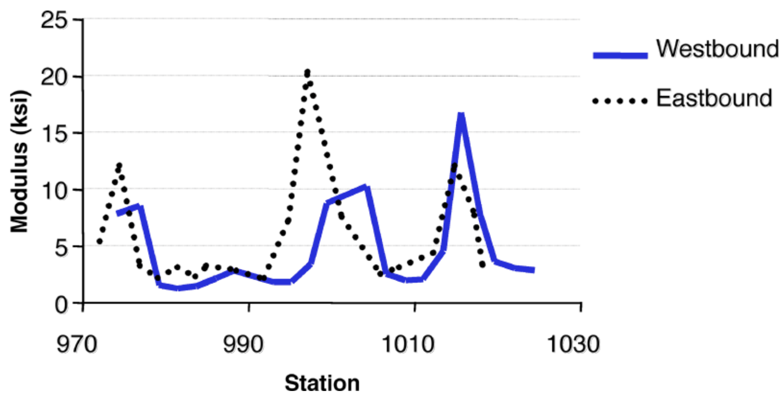


Figure 9.23 Subgrade moduli comparison

9.5.3 Rolling Dynamic Deflectometer (RDD) Testing

The use of the RDD was considered for this project when looking for a more continuous deflection profile of the road that could shed some light in the forensic investigation. After the FWD tests were concluded and analyzed, and more information was deemed necessary to draw more definitive conclusions, the RDD was incorporated as a valuable resource to characterize the roadway condition.

To have a basis for comparisons, both sides of the road were evaluated with the RDD. The first testing day was August 24, 1999. On that date, two westbound lanes were tested, including one of the BCO lanes. As the eastbound lane testing began, the RDD operators experienced some mechanical difficulties with it, ending up with the RDD truck breaking down and the eastbound testing being terminated for the day. Only about one third of the eastbound BCO was tested that day. The testing had to be repeated once the equipment was repaired. Fixing the RDD truck took a few months, because it had to be taken to Oklahoma where it was manufactured. The eastbound testing was rescheduled for January 25, 2000.

Besides that unfortunate incident, there were some differences between the westbound and eastbound testing. For some unknown reason, the westbound deflection data file was incomplete: for the westbound center lane data the measurements stop at 4,028.52 ft from the east end, whereas in the westbound inner lane the last measurement was taken at 4,931.45 ft from the east end, which corresponds to station number 972, where the project starts. This was a minor issue, considering that the main focus of the investigation was the eastbound direction, but another more significant problem was that eastbound and westbound deflections were tested under different loading conditions and that, although the sensor arrangement remained consistent, some of the data from a sensor could not be utilized. The westbound direction was measured under a 16-kip load, whereas the eastbound lanes were subjected to a 15-kip load while evaluating deflections. Also, for the eastbound measurements, the frame that holds the sensors was tilted, causing Sensor 4 to not make good contact with the ground, rendering the data from that sensor useless. All these differences were taken into account when analyzing the following RDD plots. Figure 9.24 shows both westbound and eastbound RDD measurements.

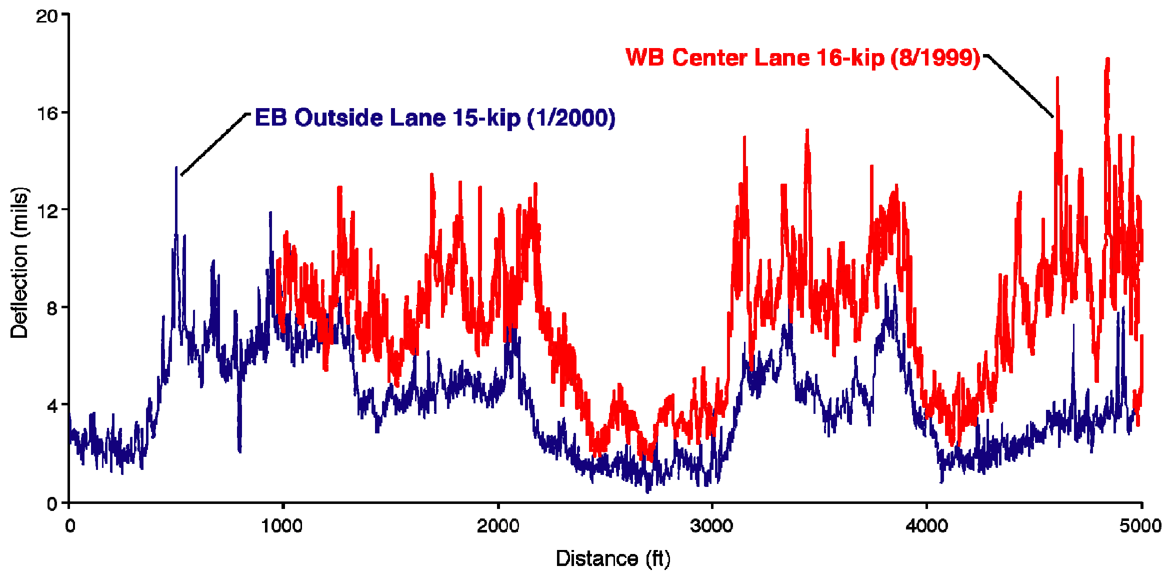


Figure 9.24 RDD deflection measurements

The shortcomings of the RDD data detailed above are evident in the previous plot. However, it gives a good idea of the deflection behavior and the structural condition of both traveling directions. The deflection pattern is very similar in both directions, which suggests some trustworthiness in the data.

An interesting analysis would be a comparison between FWD and RDD deflections, showing data taken from both apparatuses on the same lane, not only to look for patterns in the deflections, but also to see the advantages and disadvantages of both devices. In Figure 9.25 this plot is presented for the delaminated eastbound BCO. In this graph, the delaminated areas found during the sounding survey have been plotted at the bottom, to verify whether there is a relationship between delaminations and deflections. Because the delaminations were spread throughout almost the entire eastbound lane, it was not possible to establish such a relationship. From this picture, it is evident how useful a continuous delamination profile is, because the FWD intervals can lead to the wrong assumption that every deflection in between intervals should be about the same as the measured deflections at either side. This assumption is not true in this case. For example, there is an area with higher deflections that were not detected by the FWD —the peak between 3,500 and 4,000 ft from the start of the eastbound RDD deflections (Figure 9.25).

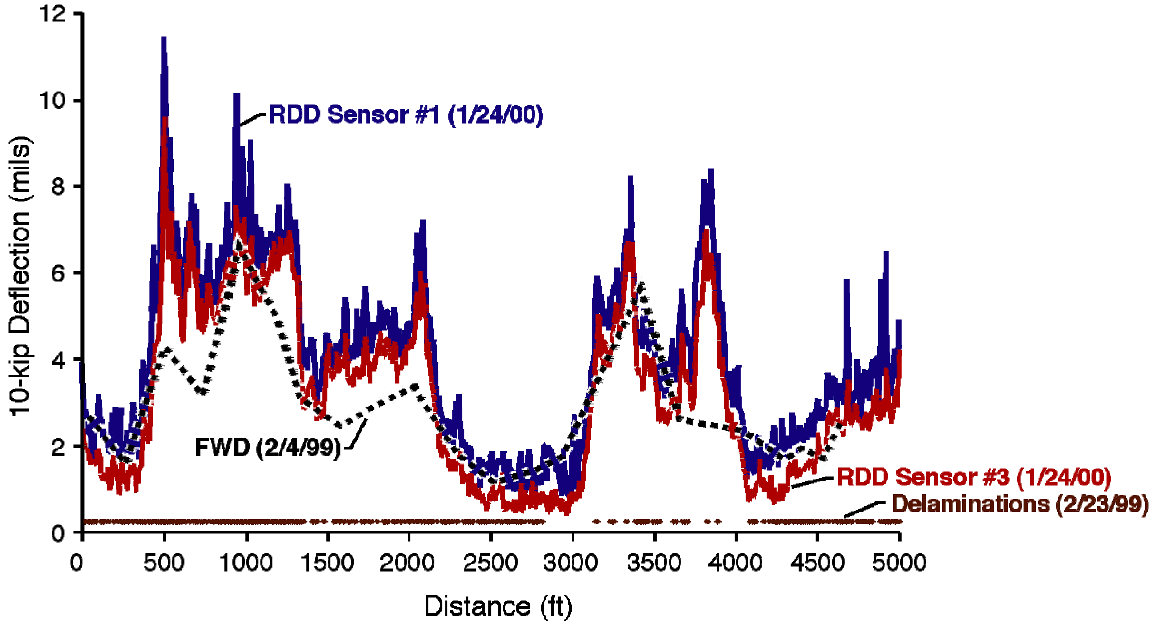


Figure 9.25 Eastbound FWD and RDD deflections and delaminations

To produce the previous graph, the RDD data had to be normalized to 10 kips to make it comparable to the deflections obtained with FWD. Still, RDD deflections are slightly higher, but the patterns of deflections from both FWD and RDD are very much alike, which confirms the reliability of both sets of data.

9.6 Construction Records

The next phase of the forensic study was to investigate the construction records. The purpose of looking at these records was to search for anything that might have been overlooked during construction, or other details that could hint about possible QC mistakes. The Fort Worth District provided the Center for Transportation Research with a considerable amount of documents archived during the BCO construction. Two issues were studied: the time to opening the BCO to traffic, and its probability of delamination. Also, the results of pulloff tests taken right after the BCO was constructed were found in the construction records.

9.6.1 Time to Opening to Traffic

A primary concern in this project was the prompt opening of the road to traffic after the overlay had been placed just the day before, approximately 24 hours after the work had been finalized for the day. This was done in order to minimize user costs associated with the closure of the road, and it was one of the important research objectives of this project. The construction records were used to retrieve the dates and times at which each BCO segment was started, finalized, and opened to the traffic the next day, by section number. The hypothesis was that a shorter period of time between the end of the overlay construction and the opening to traffic could be associated with delaminations, because the concrete might have not been allowed to gain enough strength before carrying traffic loads. Because the westbound lanes did not present any cases of delaminations, they could provide a good reference. With the construction record information on the times to opening of each segment to traffic, the charts in Figures 9.26 and 9.27 were prepared for the westbound and eastbound directions, respectively. The vertical axes show the time from when the BCO was finished to the time it was opened to traffic. The horizontal axes show the time of the day (time of construction) associated with each construction date and the station number. For these plots, it was assumed that the construction works had a linear progression from start to end each day. The westbound overlay was completed in three weekends, therefore, there are three diagonal lines showing the construction time versus the time to opening to traffic. The lines are straight because of the linear construction progression assumed.

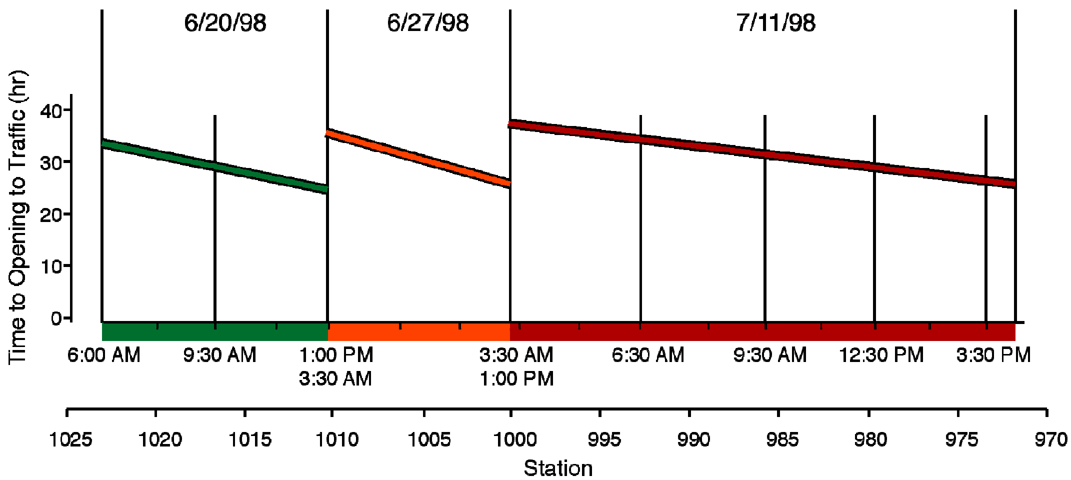


Figure 9.26 Westbound BCO time to opening to traffic

In a similar fashion, Figure 9.27 displays two diagonal lines corresponding to the two eastbound construction weekends. In this graph, the cracks and delamination locations from the February 1999 survey have been added in an attempt to correlate them to the time to opening to traffic.

It is important to note that the westbound BCO progressed from station 1024+78 to station 972+00 and the eastbound construction was done the opposite way; therefore, the station axes are reversed.

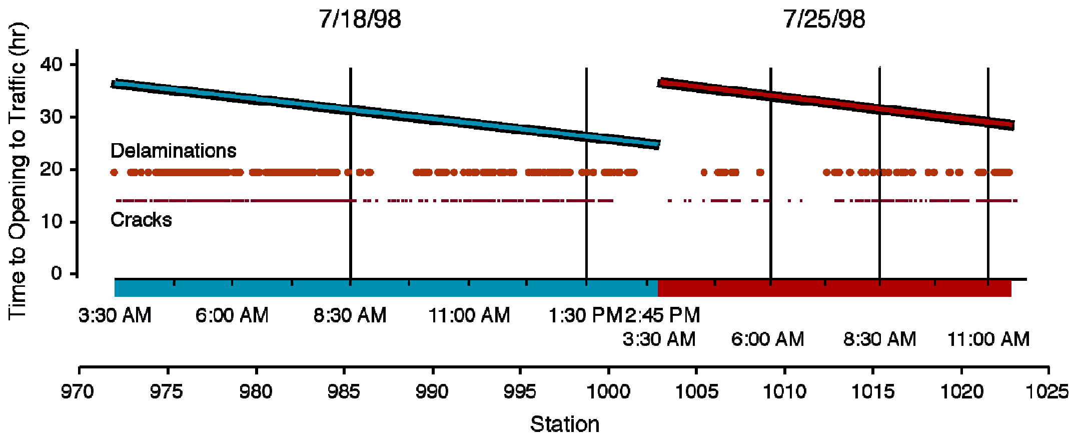


Figure 9.27 Eastbound BCO time to opening to traffic

What these charts proved was that the time to opening to traffic had no correlation to the occurrence of the eastbound delaminations because those times were very similar in both directions. For the westbound direction, the times to opening to traffic ranged from 24.2 to 36.5 hours. For the eastbound, those times ranged from 26.5 to 36.5 hours.

9.6.2 Probability of Delamination

To get a better understanding of how the time to opening to traffic could have had any influence on the eastbound delamination, the constructed segments were divided into smaller sections, and their probability of delamination was calculated.

The probability of delamination was estimated by counting the number of cracks within that section and the number of delaminations occurrences within the section. These cracks and delaminations counts come from the February 1999 visual and sounding survey. It is important to note that the presence of delaminations was associated with the occurrence of transverse cracks as was mentioned earlier in this chapter. The percentage of delaminated cracks was calculated by dividing the number of delamination occurrences by the number of cracks within each section, and this was utilized as the probability of delamination within a section. For the westbound segment, all the sections have a zero probability of delamination, because no debonding was detected. Tables 9.16 and 9.17 show the westbound and eastbound segments, respectively, with their estimated time to opening to traffic and probabilities of delaminations.

Table 9.16 Westbound probabilities of delamination, by segments

Section begins at Station	Time to Opening hr	Probability of Delamination %
1024+ 75	31.0	0
1020	28.5	0
1015	25.8	0
1011+ 75	33.7	0
1010	32.5	0
1005	28.5	0
1000	24.8	0
999+ 21	24.2	0
995	36.5	0
990	34.0	0
985	32.0	0
980	29.5	0
975	27.5	0
972+ 30	26.2	0

Table 9.17 Eastbound probabilities of delamination, by segments

Section begins at Station	Cracks	Delaminations	Time to Opening hr	Probability of Delamination %
972	37	22	36.5	59
975+ 20	51	37	35.3	73
978+ 70	46	28	34.0	61
982	79	34	32.8	43
985+ 40	16	4	31.5	25
988+ 80	24	14	30.3	58
992+ 10	33	19	29.0	58
995+ 40	43	24	27.8	56
998+ 80	13	13	26.5	100
1003	7	1	36.4	14
1006+ 10	19	8	35.1	42
1009+ 20	2	0	33.9	0
1012+ 30	23	10	32.6	43
1015+ 40	33	13	31.4	39
1018+ 60	33	9	30.1	27
1021+ 70	18	11	29.0	61

Figure 9.28 shows a plot of the probabilities of delamination for both the westbound and eastbound segments that were tabulated above.

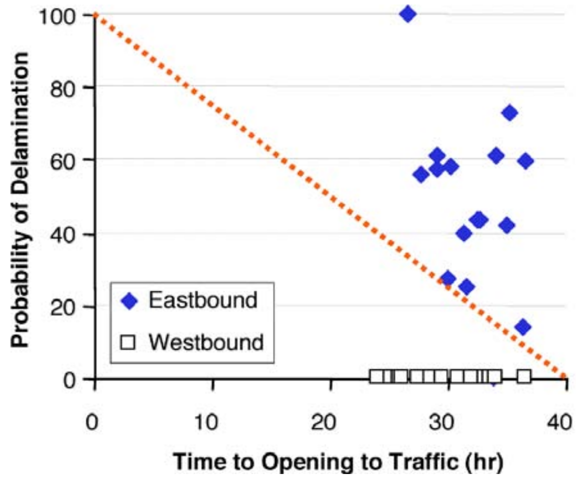


Figure 9.28 Probability of delamination in terms of time to opening the BCO to traffic

The diagonal dotted line in Figure 8.57 is the expected trend of the data: higher probabilities of delamination associated with shorter times to opening to traffic. Evidently, the data do not follow that pattern; hence, the hypothesis that expediting the BCO could have been a factor in the delamination problem was discarded.

9.7 Pulloff Tests

These tests were performed approximately 24 hours after the BCO construction was finalized, before opening the road to traffic. TxDOT personnel, as part of the BCO normal QA activities, conducted them on the site. The main interest in these tests was to determine whether any signs of the debonding of the overlay could have been detected as early as 24 hours after placement. Their results were later compared with those obtained from the cores taken after the BCO delamination was noticed.

9.7.1 Westbound

Only three tests were conducted on the westbound lanes on July 12, 1998, the last westbound construction weekend. All of them correspond to the second lane in (i.e., the left lane of the BCO; see Figure 8.19). The results are shown in Table 9.18.

Table 9.18 On-site westbound pulloff tests

Sample #	Bond strength (psi)	Date	Location	Lane
1	119.4	7/12/98	972+ 55	center
2	79.6	7/12/98	972+ 55	center
3	87.5	7/12/98	972+ 55	center
Average	95.5			
Std. Dev.	21.1			
C. of V.(%)	22.0			

9.7.2 Eastbound

Eight tests were conducted on the eastbound lanes, four for the first segment poured on July 19, and the other four during the last eastbound construction weekend on July 26; all of them were from the outside lane, the lane that later showed the worse distresses and delaminations. Table 9.19 contains the results.

Table 9.19 On-site eastbound pulloff tests

Sample #	Bond strength (psi)	Date	Location	Lane
1	119.4	7/19/98	1003+ 25	outside
2	95.5	7/19/98	1003+ 25	outside
3	47.7	7/19/98	1003+ 25	outside
4	47.7	7/19/98	1003+ 25	outside
5	79.6	7/26/98	unknown	outside
6	35.8	7/26/98	unknown	outside
7	99.5	7/26/98	1023+ 25	outside
8	95.5	7/26/98	1023+ 25	outside
Average	77.6			
Std. Dev.	30.2			
C. of V.(%)	39.0			

Even 24 hours after placement, the eastbound overlay was weaker, regarding bond strength, than the westbound. Eastbound average strength was approximately 80 percent of westbound strength in these tests. This concurs with the research presented in Chapter 2, from Gillette (Ref 6), who, as early as the 1960s, found that most delaminations happen in the early age of the BCO. Other studies, also mentioned in Chapter 2 (Refs 1, 13, 16, and 22), have confirmed that statement.

There is a considerable difference between the average bond strength obtained from these tests and the average bond strength obtained from the cores tested in 1999, presented earlier in this chapter. In the cores tested in 1999, the eastbound average bond strength was approximately 50 percent of the estbound strength. Therefore, this is an indication that the delamination problem appeared early in the life of the overlay and increased over time; this was also noticeable in the surveys and pictures showing how the deterioration process progressed with time.

9.8 Petrographic Analysis

The purpose of a petrographic analysis is to determine the formation and components of a concrete sample and to classify its type, condition, and serviceability. The petrographic analysis attempts to answer two objective questions about the concrete: "What is its composition?" and "How is it put together?" (Ref 57). Petrographic examination includes the identification of mineral aggregates, aggregate-paste interface, and integrity of the cement paste, determining whether it has been subjected to chemical processes and physical changes. These include freezing-thawing cycles, sulfate attack, alkali-aggregate reactivity, carbonation, and so forth. It also distinguishes the types of concrete present in the sample. In this case, it was considered that the petrographic analysis of some of the cores, including parts of the BCO and the original pavement, could detect the presence of foreign elements in the pavement and could give an indication of the quality of the concrete, and that this might have been helpful in trying to identify the cause of the eastbound BCO debonding.

Four cores from the eastbound lanes of IH-30 were initially submitted to TxDOT's Materials and Testing for petrographic analysis. Later on, some samples from the westbound lanes were subjected to the same type of testing for comparison purposes.

9.8.1 Eastbound Samples

The four eastbound cores were taken to TxDOT for testing in February 2000. One of them is illustrated in Figure 9.29.

Figure 9.29 Eastbound core for petrographic testing

The following is an excerpt from the report that was issued in April 2000 with the eastbound petrographic results:

The four cores were broken or debonded at the contact between the new and old concrete. The broken or debonded surfaces of two of the cores had accumulations of fines at the contact (it is undetermined whether these deposits had accumulated during the coring operation or were residues from the milling process). Accumulation of debris was observed at the interfaces of broken cores and an effort was made to identify whether the debris was from the coring operation. A brushing action was applied to remove the debris; however, this action was not successful, which indicates the debris was not from coring operations. Instead, it is believed that the debris existed before coring, which indicates that the surface preparation might have caused this accumulation of debris.

Water/Cement ratio: Normal in new overlaid concrete. Appeared slightly high in the old concrete.

Air Content: The percent of entrained air appeared to be approximately 1% in old concrete. Estimates on the new overlay concrete indicated about 2-3% air content.

Paste Content and Appearance: Normal for new concrete. The old concrete paste had signs of degradation near the surface.

Concluding Comments: The quality of the new concrete is satisfactory. The strength of the old concrete may be compromised due to the higher water/cement ratio and some degree of degradation of the paste. One of the cores clearly debonded (no broken aggregate or detached paste) at the interface between the old and new concrete. There was abundant accumulation of fines (evidence that the surface may have been dirty, possibly due to the milling process) on the surface at the interface. A second core had poor bond between the overlaid concrete and the existing pavement. The other two cores had satisfactory bonding at the interface between the two concrete slabs; however, they broke approximately one-sixteenth of an inch within the old concrete from the interface. This indicates that the old concrete had lower shear strength than the new overlay or bond strength at the interface. Based on the above observation, the following factors may have contributed to the debonding problem:

- The surface may have been insufficiently cleaned which led to a poor bonding between new and old concrete.
- The strength of the old concrete might not be sufficient to withstand the shear stresses developed near interface between old and new concrete.
- Even though it was not substantiated, it is probable that the debris from milling operation caused poor bond.

9.8.2 Westbound Samples

Three cores were sent to TxDOT's Materials and Testing after the eastbound testing and results were finalized, in May 2000, all of them in good condition, with no signs of debonding. The purpose of the westbound petrographic investigation was to determine whether there were significant differences in the westbound cores, and to establish whether the debris at the interface of the eastbound cores could be singled out as the foremost reason for the BCO delamination. These three cores were extracted specifically for the purpose of this test, whereas the eastbound cores had been taken in advance for other purposes.

The report with the petrographic analysis was finished in June 2000, with the following results:

The three westbound cores had similar batch design parameters as the four eastbound cores that were submitted for analysis previously. The three cores were cut and polished to examine and characterize the bonding between the existing concrete and the new overlay.

Water/Cement ratio: Normal in new overlaid concrete. Appeared slightly high in the old concrete.

Air Content: The percent of entrained air appeared to be approximately 1% in old concrete. Estimates on the new overlay concrete indicated about 2-3% air content.

Paste Content and Appearance: Normal for new concrete. The old concrete paste had signs of degradation near the surface.

Concluding Comments: The three westbound cores had no debonding problems. Bleed water channels (oriented parallel to the surface) were observed in the old concrete very near the interface. There were not any fines observed at the interface between the old and new concrete.

From these results, the only difference between the cores in both directions is the debris found in the eastbound samples, leading to the conclusion that a surface preparation problem definitely triggered the eastbound delamination of the overlay.

9.9 Meetings

Once the important conclusion from the petrographic investigation was reached about the inadequate surface cleaning on the eastbound lanes, the focus of the forensic study was to determine why it happened and why it occurred only in the eastbound BCO, because the construction process was supposed to have been the same in both directions. To try to answer those questions meetings with people involved with the construction were arranged, in addition to the routinely scheduled project meetings.

The first of these took place in the Fort Worth District Office on July 19, 2000. TxDOT personnel involved in the construction of the overlay met with CTR personnel in charge of the forensic study. CTR made a presentation to the district on the findings of the forensic investigation. The most important comments and conclusions at that meeting regarding the BCO delamination were as follows:

- There were problems with the coarse aggregate that led to the change of supplier for the eastbound concrete.
- Free water on eastbound surface draining toward the outside lane may have carried the debris that caused delamination.
- For the repair of delaminated areas, epoxy injection was an option. However, epoxy may stick to dirt, if the delaminations are left without repair for a long time. It should be applied to a small area first, to verify its adequacy.

- There is a considerable difference between the average bond strength obtained from the pulloff tests 24 hours after placement and the average bond strength obtained from the cores tested in 1999. (The average eastbound strength was approximately 80 percent of the westbound strength in the tests 24 hours after placement, whereas in the cores tested in 1999, the eastbound average bond strength was approximately 50 percent of the eastbound strength).
- No bonding agents were applied to the interface. Previous studies have shown that the bonding of the overlay to a clean, well-prepared surface is as good as the bonding attained using grout or other bonding agents.
- BCO rehabilitation is an excellent option for concrete pavements in Fort Worth, given the good quality of the pavements in the district.

- It was suggested to post signs on the highway for trucks to use the middle lane instead of the outside lane to prevent more damage to the delaminated outside lane.

Also, it was agreed to have a new meeting with the inspector, contractor (Champagne Webber), Skidabradder personnel, and other parties involved in the construction.

That meeting was scheduled for September 25, 2000, at the North Tarrant County Office. On that occasion, Mr. Carl Cortes, from Champagne Webber, provided valuable comments that offered insight into the presence of debris in the eastbound lanes prior to overlaying:

- For the westbound lanes, because there was more time available before pouring concrete, the shotblasting cleaning was done two or three times, whereas for the eastbound lanes, there was time for shotblasting only once.
- The reinforcement steel placement for the eastbound took place just before the BCO was placed (i.e., the paving machine was right behind the crew in charge of placing the steel).
- The overall slope of the westbound section is downhill from the perspective of the paving operations, whereas the eastbound lanes are mostly uphill, causing water with debris to stay in front of the concrete.
- The shotblasting machine is 12 ft wide, so every time the surface is cleaned, there is certain overlap with the previous swath, except for the edge, where the machine cleans only once. This concurs with the fact that the outside lane is where most of the delaminations had been found.
- The way of cleaning the surface with the shotblast is from the inside lane to the outside lane, in a 500-ft pattern. Thus, if there is not enough time to clean before the paving occurs, the outside lane does not get proper cleaning.
- During the first weekends of construction (westbound paving), there was no time pressure, but for the eastbound construction, the paving occurred in a hasty way.
- The conditions of the original pavement prior to the BCO placement were no different from one direction to another in terms of distresses. According to the maintenance supervisor, there were no cracks in the original pavement.

All these comments support the statement that the eastbound surface was not cleaned adequately and that the problem manifested more severely in the outside lane because of the nature of the construction. The explanations given for the deficient cleaning are sensible and congruent with the findings of the other parts of the forensic investigation, especially with the petrographic testing.

Finally, a conference call with Mr. Mike Swain, from Skidabradder on September 27, 2000, provided more evidence as to how the eastbound lanes lacked an adequate surface preparation. The following are his comments:

1. The contamination on the surface of the original pavement is the worst he had ever seen:

- There was dirt around the guardrails, near the shoulder, that was blown onto the surface.
- Some areas of the original pavement were repaired prior to the overlay placement. As a result of these operations, there was “whitewater” (waste water from sawing) that was running over the surface.
- Placement operations of the overlay reinforcement steel cause contamination of the surface to be overlaid.
- Ideally, the better surface preparation and conditions would occur with a non-reinforced overlay, because the cleaning could be accomplished just before the overlay placement.
- All these conditions were worse on the eastbound side of the highway.

2. General comments on the Skidabradder machine

- The equipment is designed to cut ½-inch deep in one cut.
- The machine performs continuous cleaning of the shot.
- The cuts are approximately 6 ft wide.
- There is a need to establish a specification as to what is clean enough for placement
- The air coming out of the compressor may be another source of contamination for the surface.
- All the operations should be performed before the cleaning.

These comments again concur with the conclusion of the poor surface preparation causing the debonding of the overlay.

The meetings and conference call were considered very positive steps toward the closure of the forensic investigation, because everything that was mentioned by the construction team supported the previous findings and explained why the problem happened.

9.10 Conclusions

This chapter, and the preceding one showcase a BCO project that offered the opportunity to study all the features of a BCO process presented in the first part of this dissertation, from project selection to QC/QA. The delamination of part of the overlay originated an additional forensic investigation, as was shown in this chapter. The conclusion of the study is that even when every other facet of the project is carefully planned and developed, a single aspect that is not well taken care of may render a negative outcome.

The various areas investigated in this study have shown that the delamination occurred solely as a result of a construction problem, which can be easily avoided in the future, and that the delamination was caused neither by the overlay design nor by the fact that the project was an expedited BCO. The concept of an expedited BCO was implemented to return the traffic back to road shortly after construction, but it did not involve accelerating the other stages of the construction process; therefore, it had no impact on the fact that the surface preparation was overlooked. Furthermore, the excellent performance of the westbound overlay confirmed that an expedited BCO is a viable means of rehabilitating a pavement without causing prolonged traffic delays. In the Fort Worth case, state-of-the-art research was applied and a great deal of new research was developed, all of which has been presented in this dissertation.

10. Discussion of Results

This chapter recapitulates the results of the bonded concrete overlay (BCO) Implementation, discussed in Chapter 8, and the forensic study, shown in Chapter 9, and extends recommendations for overlay repair.

10.1 Implementation Results

The Fort Worth full-scale BCO project on IH-30 offered the opportunity to study all the stages of a typical BCO project, developing the Texas experience with BCOs; the fact that the project included the widening of the road and expediting the BCO, as well as the forensic study, represented a unique inducement for the researchers to conduct additional investigation and to deepen the knowledge on this type of rehabilitation. Certainly, the delamination of the eastbound lanes occurred as a result of a construction mistake that caused major inconvenience, but from the research standpoint, the forensic study enhanced the value of the project, because it presented the challenge and opportunity to endeavor in an investigation of a greater extent.

Age, projected traffic increases, and loading prompted the Fort Worth District to undertake this rehabilitation project on IH-30, just inside the IH-820 urban loop in the west part of the city. The conditions were ideal for a BCO, because the original 8-in. thick continuously reinforced concrete pavement (CRCP) was structurally sound, as the deflection and in situ sample testing revealed. The rehabilitation alternative needed to be thin because of the existence of structures with clearance issues along the project stretch.

An estimated remaining life of 80 percent attested to the structural adequacy of the original CRCP; this estimation was derived from the traffic analysis. The BCO was designed using two procedures: the AASHTO 1993 design method (Ref 26), and a mechanistic overlay design method, the Texas Rigid Pavement Overlay Design (RPOD) that was incorporated into the Rigid Pavement Rehabilitation Design System (RPRDS, Ref 42). Performance periods of 30, 40, and 50 years were analyzed using a 99.5 percent level of reliability and a 3.5-in. thick BCO for a 50-year performance period was selected as the final design. The reinforcement design was performed using the programs CRCP8 and JRCP6, for both the BCO and the new CRCP.

Construction was scheduled in two phases. The first stage comprised the widening of the road with the construction of the new 11.5-in. thick CRCP where the original grassy median used to be. The second phase was the BCO construction. To make this possible, the existing AC overlay was removed, exposing the CRCP, which was in good condition to be overlaid. Segments of the project length were overlaid on weekends and opened to traffic on Sunday afternoons, fulfilling the purpose of an expedited construction, and allowing the BCO to cure for at least 24 hours. It took five weekends to construct both traveling directions of the 1.3-mile stretch of BCO.

Several tests were conducted on the new overlay to monitor its quality and performance, including condition surveys, sounding, and deflection tests. During one of the routine surveys the delamination of some portions of the BCO was detected. The presence of delaminations was confirmed in a subsequent survey and sounding testing. A forensic investigation attempted to unveil the reason for the mishap.

The process of establishing the cause for the BCO delamination problem proved to be a major task, because the results of the different aspects analyzed in the forensic study did not provide any definitive conclusions as the investigation progressed. The various hypotheses formulated at the beginning of the study kept being discarded as untrue, but not as completely useless, because they served the purpose of narrowing the number of possible reasons, and helped to prepare a new hypothesis every time, which was closer to the true source of the problem. The process involved analyzing in detail many areas of the BCO construction. Hopefully, the results of this forensic analysis will help avoid similar mistakes in the future, and will help improve construction procedures and specifications to construct better overlays. In the next few paragraphs, the outcome of the forensic study is discussed.

The weather that prevailed during the five construction weekends was certainly not ideal for concrete placement, as could be expected from a summer construction job in Texas; but for the most part, it was generally acceptable. Only the first weekend represented a threat for the BCO performance. Nevertheless, despite the adverse evaporation rates on those days, the westbound section that was placed fortunately had no apparent signs of damage, as was ascertained during the subsequent monitoring activities. Weather conditions were considered as a possible factor in the eastbound BCO delamination, but the analysis of the weather station information confirmed otherwise.

In-situ samples of the pavement tested for splitting tensile strength, bond strength, and density showed that the concrete itself, for both the BCO and the original pavement, had good strength and good overall quality, and that the problem causing the delamination of the BCO was related only to the lack of strength at the interface between the overlay and the old pavement. This conclusion was evident as both the splitting tensile strength and density tests yielded very satisfactory results. But it was the pulloff test that showed that the eastbound cores lacked appropriate bond strength.

Both Falling Weight Deflectometer (FWD) and Rolling Dynamic Deflectometer (RDD) tests consistently exhibited the weakness of the eastbound BCO as compared with the westbound BCO, as an obvious consequence of the delaminations. Moduli of elasticity backcalculations from FWD deflections proved that the subgrade structural support was adequate but not uniform throughout the section, and that it was relatively similar in both directions; this provided evidence to discard the hypothesis that the eastbound delaminations could have been caused by the poor condition of the subgrade stratum. In fact, the eastbound subgrade was, on average, slightly stiffer than the westbound subgrade.

The investigation of the construction records indicated that the delamination was not connected with the fact that the BCO had an expedited construction, meaning that the early opening to traffic, just about 24 hours after the paving duties were finalized, had no apparent influence on the failure of the eastbound section. The shorter times to opening to traffic for the various segments were supposed to be correlated with the occurrence of delaminations, but the records proved otherwise. The sections with a higher probability of delaminations were not necessarily those that had an earlier opening to the traffic.

The petrographic analysis was the part of the forensic investigation that provided the more definite conclusions on the reason for the debonding of the BCO. These conclusions fit with what other parts of the study had demonstrated and support their findings. The presence of debris at the interfaces of the eastbound cores between the BCO and the old pavement showed that the problem was at the interface, leading to the conclusion of poor surface preparation prior to the BCO placement. The fact that the debris was found only in the eastbound cores makes it more evident that the surface preparation is the leading cause of the delamination.

The subsequent meetings with the Texas Department of Transportation (TxDOT) personnel, and other parties involved in the BCO construction, confirmed the previous conclusion, and provided more insight as to why it happened only in the eastbound lanes, when the construction conditions were assumed to have been very similar for both sides of the road. Those comments can be summarized in the following statements:

The eastbound uphill slope of the road contributed to the accumulation of debris carried by surface water just ahead of the paving machine. The hastiness with which the eastbound paving occurred was also a major contributor to the faulty cleaning of the road, with the reinforcement placement happening slightly ahead of the concrete pouring. The overall condition of the eastbound lanes was dirtier than the westbound, and there was more time to prepare the westbound surface adequately before overlaying.

Another important conclusion of the study is that the delamination occurred merely as a consequence of a construction problem, and not as a design failure. This confirms that bonded overlays are a reliable and economical way of pavement rehabilitation, as long as special attention is given at critical construction stages such as the surface preparation.

10.2 Recommendations for Overlay Repair

When facing a delamination problem, in general, there is a wide spectrum of alternatives as to the direction of the actions to take, which include the following:

- Do nothing
- Seal cracks and spot patch
- Selective epoxy injection
- Extensive epoxy injection
- Remove and replace overlay

After a thorough discussion with the Construction Materials Research Group of The University of Texas at Austin, and on the basis of earlier CTR experiences with debonded overlays in El Paso (Ref 1) and Houston (Ref 16), it was concluded that any attempts to bond the overlay and the existing pavement at that late date would have been futile. Although these operations were successful in El Paso, the procedure was expensive and the step was accomplished shortly after placement. The consensus in the case of the Fort Worth BCO was that so much foreign material and debris would be present that a successful bond would be difficult to attain.

Therefore, it was recommend to pursue one of the following two options:

Let the section continue to deteriorate, with spot patching as needed, until it became either a safety issue (i.e., car damage from potholes) or an economic burden, if the patching expenditures became excessive.

Remove the overlay and replace with a new BCO.

The merit of the first approach is that a major expense would not be required initially (as would be the case of the removal and replacement of the BCO), and that it would allow the assessment of whether the rate of deterioration increased or decreased. If the latter case occurred, permanent patches would correct the problem. This was the case in Houston, where the problem showed quickly, but stabilized after patching. However, the delamination in Houston was not as extensive as it was in this case.

A reason not to favor this option could be that the delamination and deterioration processes had increased with time. The decaying progression is dramatically illustrated in the images in Figures 10.1 (from February 1999). Figures 10.2 and 10.3 (photographs from January 12, 2000) and Figures 10.4 and 10.5, taken during a condition survey on January 25, 2000, provide some more evidence of the deterioration process when compared with the pictures shown in Chapter 8, which were taken during the February 1999 condition survey.

Figure 10.1 BCO delaminations on February 23, 1999

Figure 10.2 BCO deterioration

Figure 10.3 BCO deterioration

Figure 10.4 BCO deterioration

Figure 10.5 BCO deterioration

These images show that some patching took place between January 12 and January 25, 2000.

As for the second option, remove and replace the overlay, it had the advantage of still achieving the intended 30-year design for the pavement structure. Obviously an unbonded surface places higher stresses in the existing pavement, thus deteriorating it at a faster rate than was predicted in the design. This option would eliminate that problem altogether. Another advantage would be not having to deal with a continuous assessment and repair schedule.

In light of the BCO deterioration, the second option, to remove and replace the overlay, was considered the most viable alternative.

11. Conclusions and Recommendations

This chapter summarizes the most relevant aspects of this investigation. The first part presents general conclusions, linked to the objectives of this work, as was explained in Chapter 1. Specific recommendations pertaining to the stages of the bonded concrete overlay (BCO) are presented in the second part, followed by recommendations for future developments in the area of BCOs.

11.1 Conclusions

There has been a significant increase in the use of BCOs as an economical way to extend the life of a pavement structure. This increase is attributed to improvements in construction equipment and procedures, to the excellent performance given by the overlays, and to the advances in several research areas impacting BCOs, especially bonding, because achieving it is so critical for BCOs. Surface preparation, climatic conditions at the time of placing, and curing have been identified as significant construction factors affecting bond, early-age strength gain, and long-term performance.

The following paragraphs show conclusions associated with the four main objectives of this study:

The appropriateness of a BCO as a pavement rehabilitation strategy has been established, indicating what conditions have to occur for a BCO to be successfully implemented. This was explained in Chapter 1, with the definitions, advantages, and disadvantages of BCOs; in Chapter 2 with a historical review of the development of BCOs, as well as the research that has enabled its widespread utilization; in Chapter 3, in which the process for BCO implementation is outlined; and in Chapter 4, in which a rational methodology consisting of a series of steps to make the decision for a BCO over other alternatives, i.e., the project selection, is presented in detail and synthesized in a flow chart.

A series of guidelines resulting from years of research and experience in many BCO projects for selection, design, and construction are established in Chapters 4, 5, and 6, respectively.

Techniques and procedures for Quality Control and Quality Assurance (QC/QA) on a BCO are presented in Chapter 7. Common QC/QA tests are outlined, and some innovative procedures are also detailed.

The deployment of those techniques and procedures at the various stages of the BCO process is exemplified with a BCO implementation project, the full-scale BCO on IH-30 in Fort Worth, which encompassed the application of previous experiences with BCO projects, as well as the development of new research.

In addition, Chapter 9 presents a detailed set of procedures and tests for the evaluation of delaminated BCOs.

11.2 Recommendations

The following guidelines and procedures are applicable to the stages of the BCO Process.

11.2.1 Project Selection

The following conditions warrant the use of a BCO:

There is a need for rehabilitation as a result of age, current or projected traffic increases, distresses from carrying loads, or a combination of these.

It is more cost-effective than other alternatives. Adequate resources are available.

Pavement is structurally sound.

No functional failures have occurred. No extensive damage exists.

There is a need to improve the structural capacity of the pavement and to add service life to it.

The riding quality, failures, and deflection criteria are met.

Construction of the BCO can take place with minimum delay, before the pavement might deteriorate to a functional failure condition.

11.2.2 Design

There is not an absolute solution to BCO thickness design. The extremes of the spectrum of design methodologies feature the mechanistic approach, on one side, and the empirical approach, on the other. The development of research has enabled the advancement of mechanistic methods.

It is advisable to choose a method that utilizes an evaluation of the current conditions of the existing pavement (i.e., materials characterization), to assess the structural contribution of the existing pavement and to take it into consideration for the BCO thickness design.

The recommended design procedure is to follow the AASHTO method in conjunction with the RPRDS procedure. Both methods have different failure criteria, so the designer can choose between both thickness designs to arrive at the final BCO thickness.

11.2.3 Construction

The materials selected for a BCO should be compatible with those of the existing pavement.

Coarse aggregates in the BCO should have a thermal coefficient that is lower than, or, at most, equal to that of the coarse aggregates of the substrate. Thus, limestone is the most preferred type of aggregate for a BCO.

The maximum aggregate size should be one third of the BCO thickness to avoid segregation, prevent voids, and ensure proper aggregate interlock and bond with the substrate.

All AC layers and patches should be removed from the existing CRCP because they will break the bond between the substrate and the new BCO and are likely cause of delaminations.

All major distresses in the existing pavement should be repaired prior to BCO placement. PCCP should be used for patches, and for partial, and full-depth repairs.

Repairs should be continuously reinforced to ensure continuity between the repair and the existing pavement and to preserve load transfer capabilities.

Voids detected under existing slabs should be stabilized with grout prior to the replacement or repair of existing damaged slabs.

Corrosion problems may require full-depth repairs along with new steel. Other minor corrosion cases may be fixed by localized concrete removal and steel replacement (if the bar has lost more than 25 percent of its cross section) or by cleaning the steel from all corrosion and surrounding it with new concrete.

The preferred method of surface preparation is shotblasting. Cold milling delivers good surface texture, but it may cause microcracking in the substrate.

All dust and debris should be eliminated from the substrate surface just prior to placing the BCO by means of airblasting.

The BCO shoulders should be made of PCCP, and tied to the main lane, from which tie-bars will transfer loads to it; tied PCC shoulders provide support to the slab edge, where the stress concentration is critical.

If placing a BCO causes localized overhead clearance problems, three action paths could be followed: 1) the depth of removal achieved by surface preparation procedures can be increased, 2) the BCO thickness can be slightly reduced in a particular area, or 3) the problem area can be reconstructed. The first two alternatives represent a compromise in terms of the total necessary thickness specified by design and thus, a structural capacity reduction, whereas the third option represents more labor and cost, but may be the best structural solution.

No bonding agents should be utilized between the substrate and the BCO under normal conditions. Special circumstances may warrant the use of epoxy to improve bond strength and/or shear connectors.

Steel reinforcement can be placed directly on top of the substrate instead of at mid-depth of the BCO with no diminishment in performance.

Placing of the concrete should preferably happen under cool temperature conditions, low winds, and high humidity to minimize evaporation. Periods of large temperature changes are especially detrimental for bonding during the BCO construction. A large temperature drop from day to night when the concrete is still in the process of gaining strength may cause cracking and debonding, especially at slab corners. Placement of concrete should be avoided if the following environmental conditions occur:

Water evaporation rate exceeds 0.2 lb/sq. ft/hr

Substrate temperature of 125 °F or higher

Daily temperature differentials of 25 °F or higher expected for the 24-hr period following concrete placement.

If any of the above circumstances arises, special curing can be applied to proceed with the BCO placement.

Regardless of the weather conditions, curing should be applied immediately after screeding. Delaying this operation may be harmful to the BCO and could result in crack spalling and delaminations.

Type III cement may be used for expedited BCO construction; otherwise, a superplasticizer may be added to the mix.

Air-entraining admixtures may be added to the PCC mix as well.

An expedited BCO should attain a pre-established strength requirement before being opened to traffic.

11.2.4 Delaminations

Delaminations are a common and harmful occurrence in BCOs. The following findings and recommendations pertain to this type of distress.

Delaminations hinder the ability of the BCO to carry traffic loads. Because the BCO is intended to behave as a single structural unit with the existing pavement, a separation between both layers jeopardizes its performance.

Delaminated areas are associated with cracks, starting at the edges of the slab and extending toward the center.

Most delaminations happen early in the life of the overlay, during the first 12 to 48 hours following placement. Delaminations may or may not deteriorate once they have started, depending on the initial extent of the delamination. Two projects illustrate these cases: first, the Houston North Loop project developed delaminations early in its life, but after repairs, the delamination problem did not extend and the debonding did not affect the BCO performance. In the second case, the Fort Worth project, the delaminations appeared early and continued to deteriorate with time.

Sounding tests should be conducted shortly after placement.

The bonded condition of the overlay should be restored soon after the problem manifests.

11.2.5 QC/QA

A fundamental principle for a quality BCO is to implement good practices at every stage of the BCO process.

Specifications set the parameters against which quality is measured, by means of QA testing. Current specifications tend to be performance-oriented as opposed to method specifications.

There is a need for a standardized field bond test. Currently, the most reliable procedure is the pulloff test, which is a modified version of another test.

Whenever possible, it is recommended to use tests conducted on in situ samples of concrete rather than on tests of molded specimens, because the former reflect the properties of the concrete that is placed as part of the structure, whereas the latter will show only the properties of the concrete delivered to the job site.

A value of 200 psi for the interface shear strength is considered a safe standard that ensures the BCO can withstand fatigue from traffic loading and environmental loading. Once that bond strength is attained, it will endure. However, it is critical to achieve it during the early-age of the BCO.

11.2.6 Recommendations for BCO Future Research and Improvement

Results from the implementation of BCOs, the project featured in Chapters 8 and 9, as well as other projects reviewed in Chapter 2, reveal that there is a great potential for BCO implementation as a means to extend the pavement service life.

Some future developments that may enhance the implementation of a BCO and facilitate its success could be:

To develop a specification on surface preparation. This is an area of opportunity to improve the implementation of BCOs in the future. A current recommended procedure is to expose the coarse aggregate of the substrate before placing a BCO, which is correct, but a formal specification needs to be devised.

To develop surface cleaning specifications. The Fort Worth Project, presented herein, emphasized the relevance of a sequence of operations that need to take place between the time of surface preparation and the placement of the concrete. Those operations are presented in this study; they need to be followed as quality standards.

The creation of a standardized field bond test.

To continue the research on the use of the Rolling Dynamic Deflectometer (RDD) to make it possible to analyze its measurements in a more expedited way. Continuous deflection profiles have proved to be a valuable tool in the investigation of delaminations.

To disseminate the knowledge on BCO construction quality practices and procedures, especially among agencies and contractors. This will provide agencies with a better understanding of BCO adequacy and applicability during project selection and design, and will provide both, agencies and contractors, with the means for successfully executing the BCO construction and QC/QA.

References

1. Delatte, N. J., S. F. Grater, M. Trevino-Frias, D. W. Fowler, and B. F. McCullough, "Partial Construction Report of a Bonded Concrete Overlay on IH-10, El Paso, and Guide for Expedited BCO Design and Construction" Research Report 2911-5F, Center for Transportation Research, The University of Texas at Austin, December 1996.

2. Trevino M., B. F. McCullough, and T. Krauss, "Full-Scale Bonded Concrete Overlay on IH-30 in Ft. Worth, Texas," Research Report 572-1, Center for Transportation Research, The University of Texas at Austin, October 2000.

3. Hutchinson, R. L., "Resurfacing with Portland Cement Concrete," National Cooperative Highway Research Program, Synthesis of Highway Practice 99, Transportation Research Board, Washington, D. C., December 1982.

4. Felt, E. J., "Resurfacing and Patching Concrete Pavements with Bonded Concrete," Highway Research Board Proceedings, Washington, D. C., January 1956.

5. Felt, E. J., "Repair of Concrete Pavement," Journal of the American Concrete Institute, Proceedings Vol. 57, August 1960.

6. Gillette, R. W., "A 10-Year Report on Performance of Bonded Concrete Resurfacings," Highway Research Record 94, Maintenance Practices, Highway Research Board, Washington, D. C., January 1965.

7. "Shift Toward Concrete Seen in Minnesota Overlay Test," Engineering News Record, Vol. 201, August 17, 1978.

8. Knutson, M. J., "Iowa's Bonded Portland Cement Concrete Overlays," Iowa Concrete Paving Association, West Des Moines, IA, November 1977.

9. Cox, W., and J. Love, "40 Years of the US Interstate Highway System: An Analysis, The Best Investment a Nation Ever Made," http://www.publicpurpose.com/freeway1.htm, June 1996.

10. Comeau, C. M., "Condition and Performance of the Interstate System After 40 Years," Public Roads, Washington, D. C., 1996.

11. Kailasananthan, K., B. F. McCullough, and D. W. Fowler, "A Study of the Effects of Interface Condition on Thin Bonded PCC Overlays," Research Report 357-1, Center for Transportation Research, The University of Texas at Austin, October 1984.

12. Bagate, M., B. F. McCullough, D. W. Fowler, and M. Muthu "An Experimental Thin-Bonded Concrete Overlay Pavement," Research Report 357-2F, Center for Transportation Research, The University of Texas at Austin, November 1985.

13. McCullough, B. F., and D. W. Fowler, "Bonded Concrete Overlay (BCO) Project Selection, Design, and Construction," Research Report 920-6F, Center for Transportation Research, The University of Texas at Austin, November 1994.

14. Trevino, M., "Rehabilitation of Beltway 8 Pavement Spalling with Concrete Overlays," Master's Thesis, The University of Texas at Austin, August 1996.

15. Koesno, K., and B. F. McCullough, "Evaluation of the Performance of the Bonded Concrete Overlay on Interstate Highway 610 North, Houston, Texas," Research Report 920-2, Center for Transportation Research, The University of Texas at Austin, December 1987.

16. Teo, K., D. W. Fowler, and B. F. McCullough, "Monitoring and Testing of the Bonded Concrete Overlay on Interstate Highway 610 North in Houston, Texas," Research Report 920-3, Center for Transportation Research, The University of Texas at Austin, February 1989.

17. Trevino, M., "Condition Survey of Bonded Concrete Overlay on IH-610 North in Houston," Technical Memorandum 0147-1, Center for Transportation Research, The University of Texas at Austin, December 2000.

18. Koesno, K., and B. F. McCullough "Design Analysis for Rehabilitation of CRCP on Southeast Quadrant of Houston Loop 610," Research Report 920-1, Center for Transportation Research, The University of Texas at Austin, October 1986.

19. Whitney, D. P., P. Isis, B. F. McCullough, and D. W. Fowler, "An Investigation of Various Factors Affecting Bond in Bonded Concrete Overlays," Research Report 920-5, Center for Transportation Research, The University of Texas at Austin, June 1992.

20. Allison, B. T., B. F. McCullough, and D. W. Fowler, "Feasibility Study for a Full-Scale Bonded Concrete Overlay on IH-10 in El Paso, Texas," Research Report 1957-1F, Center for Transportation Research, The University of Texas at Austin, January 1993.

21. Wade, D. M., D. W. Fowler, and B. F. McCullough, "Concrete Bond Characteristics for a Bonded Concrete Overlay on IH-10 in El Paso," Research Report 2911-2, Center for Transportation Research, The University of Texas at Austin, July 1995.

22. Huddleston, J. L., D. W. Fowler, and B. F. McCullough, "Effects of Early Traffic Loading on a Bonded Concrete Overlay," Research Report 2911-3, Center for Transportation Research, The University of Texas at Austin, September 1995.

23. Nelson, R., M. Trevino, S. Grater, and B. F. McCullough, "Rehabilitation of IH-10 and IH-610 with Concrete Overlays," Research Report LOA 97-01-1, Center for Transportation Research, The University of Texas at Austin, February 1997.

24. Trevino M., and B. F. McCullough, "Monitoring and Forensic Study on Full-Scale Bonded Concrete Overlay on IH-30 in Ft. Worth, Texas," Research Report 572-4, Center for Transportation Research, The University of Texas at Austin, March 2002.

25. Johnson, R. H., and R. T. Weber, "Buying Quality," Franklin Watts, New York, NY, 1985.

26. AASHTO Guide for Design of Pavement Structures, American Association of State Highway and Transportation Officials, 1993.

27. Yoder, E. J., and M. W. Witczak, "Principles of Pavement Design," John Wiley & Sons, Inc., New York, NY, 1975.

28. Barenberg, E. J., “Rehabilitation of Concrete Pavement by Using Portland Cement Concrete Overlays,” Transportation Research Record 814, 1981

29. Taute, A., B. F. McCullough, and W. R. Hudson, “Improvements to the Materials Characterization and Fatigue Life Prediction Methods of the Texas Rigid Pavement Overlay Design Procedure,” Research Report 249-1, Center for Transportation Research, The University of Texas at Austin, November 1981.

30. Van Metzinger, W. A., B. F. McCullough, and D. W. Fowler, “An Empirical-Mechanistic Design Method Using Bonded Concrete Overlays for the Rehabilitation of Pavements,” Research Report 1205-1, Center for Transportation Research, The University of Texas at Austin, January 1991.

31. Huang, Y. H., “Pavement Analysis and Design,” Prentice-Hall, Englewood Cliffs, NJ, 1993.

32. “Pavement Management Information System Rater’s Manual,” Texas Department of Transportation, Design Division-Pavements, June 1999.

33. Uzan, J., T. Scullion, C. H. Michalak, M. Paredes, and R. L. Lytton, “A Microcomputer Based Procedure for Backcalculating Layer Moduli From FWD Data,” Research Report 1123-1, Texas Transportation Institute, Texas A&M University, July 1988.

34. Scullion, T., and C. Michalak, “Modulus 4.0: User’s Manual,” Research Report 1123-4, Texas Transportation Institute, Texas A&M University, January 1991.

35. Wei, C., and B. F. McCullough, “Development of Load Transfer Coefficients for Use with the AASHTO Guide for Design of Rigid Pavements Based on Field Measurements,” Research Report 1169-3, Center for Transportation Research, The University of Texas at Austin, February 1992.

36. Army Airfield and Heliport Rigid and Overlay Pavement Design, Department of the Army Technical Manual, TM 5-823-3, 1968.

37. Tayabji, S. D., and P. A. Okamoto, “Thickness Design of Concrete Resurfacing,” Third International Conference on Concrete Pavement Design and Rehabilitation, Purdue University, West Lafayette, IN, 1985.

38. Treybig, H., B. F. McCullough, P. Smith, and H. von Quintus, “Overlay Design and Reflection Cracking Analysis for Rigid Pavements,” Volume 1-Development of New Design Criteria, Research Report No. FHWA-RD-77-66, Federal Highway Administration, Washington, DC, 1977.

39. Treybig, H., B. F. McCullough, P. Smith, and H. von Quintus, “Overlay Design and Reflection Cracking Analysis for Rigid Pavements,” Volume 2-Design Procedures, Research Report No. FHWA-RD-77-67, Federal Highway Administration, Washington, DC, 1977.

40. The AASHO Road Test, Report 5, Pavement Research, Special Report 61E, Highway Research Board, 1962.

41. Schnitter, O., W. R. Hudson, and B. F. McCullough, “A Rigid Pavement Overlay Design Procedure for Texas SDHPT,” Research Report 177-13, Center for Highway Research, The University of Texas at Austin, May 1978.

42. Seeds, S. B., B. F. McCullough, and W. R. Hudson, "A Design System for Rigid Pavement Rehabilitation," Research Report 249-2, Center for Transportation Research, The University of Texas at Austin, January 1982.

43. Rasmussen, R. O., B. F. McCullough, and J. Weissmann, "Development of a Bonded Concrete Overlay Computer-Aided Design System," Research Report 2911-1, Center for Transportation Research, The University of Texas at Austin, January 1995.

44. Won, M., K. Hankins, and B. F. McCullough, "Mechanistic Analysis of Continuously Reinforced Concrete Pavements Considering Material Characteristics, Variability and Fatigue," Research Report 1169-2, Center for Transportation Research, The University of Texas at Austin, March 1991.

45. Suh, Y. C., K. Hankins, and B. F. McCullough, "Early-Age Behavior of Continuously Reinforced Concrete Pavement and Calibration of the Failure Prediction Model in the CRCP Program," Research Report 1244-3, Center for Transportation Research, The University of Texas at Austin, March 1992.

46. Noble, C. S., B. F. McCullough, and J. C. M. Ma, "Nomographs for the Design of CRCP Steel Reinforcement," Research Report 177-16, Center for Highway Research, The University of Texas at Austin, August 1979.

47. Kim, S. M., M. Won, and B. F. McCullough, "Three-Dimensional Nonlinear Finite Element Analysis of Continuously Reinforced Concrete Pavements," Research Report 1831-1, Center for Transportation Research, The University of Texas at Austin, February 2000.

48. Kim, S. M., M. Won, and B. F. McCullough, "CRCP-9: Improved Computer Program for Mechanistic Analysis of Continuously Reinforced Concrete Pavements," Research Report 1831-2, Center for Transportation Research, The University of Texas at Austin, February 2001.

49. Kim, S. M., M. Won, and B. F. McCullough, "CRCP-9 Computer Program User's Guide," Research Report 1831-3, Center for Transportation Research, The University of Texas at Austin, February 2001.

50. Kim, S. M., M. Won, and B. F. McCullough, "CRCP-10 Computer Program User's Guide," Research Report 1831-4, Center for Transportation Research, The University of Texas at Austin, February 2001.

51. Kim, S. M., M. Won, and B. F. McCullough, "Transformed Field Domain Analysis of Pavements Subjected to Moving Dynamic Tandem-Axle Loads and Integrating their Effects into the CRCP-10 Program," Research Report 1831-5, Center for Transportation Research, The University of Texas at Austin, August 2001.

52. Emmons, P. H., A. M. Vaysburd, and J. E. McDonald, "Concrete Repair in the Future Turn of the Century-Any Problems?," Concrete International, March 1994.

53. Van Metzinger, W. A., J. R. Lundy, B. F. McCullough, and D. W. Fowler, "Design and Construction of Bonded Concrete Overlays," Research Report 1205-4F, Center for Transportation Research, The University of Texas at Austin, January 1991.

54. Lundy, J. R., B. F. McCullough, and D. W. Fowler, “Delamination of Bonded Concrete Overlays at Early Ages,” Research Report 1205-2, Center for Transportation Research, The University of Texas at Austin, January 1991.

55. Mindess, S., and J. F. Young, “Concrete,” Prentice-Hall, Inc., Englewood Cliffs, NJ, 1981.

56. Suh, Y. C., J. R. Lundy, B. F. McCullough, and D. W. Fowler, “A Summary of Studies of Bonded Concrete Overlays,” Research Report 457-5F, Center for Transportation Research, The University of Texas at Austin, November 1988.

57. Emmons, P. H., “Concrete Repair and Maintenance Illustrated: Problem Analysis, Repair Strategy, Techniques,” Kingston, MA, 1993.

58. Whiting, D., A. Todres, M. Nagi, T. Yu, D. Peshkin, M. Darter, J. Holm, M. Andersen, and M. Geiker, “Synthesis of Current and Projected Concrete Highway Technology,” SHRP-C-345. Strategic Highway Research Program, National Research Council, Washington, DC, August 1993.

59. Choi, D., “An Experimental Investigation of Interface Bond Strength of Concrete Using Large Powder-Driven Nails,” Doctoral Dissertation, The University of Texas at Austin, May 1996.

60. Portland Cement Association, Design and Control of Concrete Mixtures, Skokie, IL, 1988.

61. AASHTO/FHWA Industry Joint Training, Construction of Portland Cement Concrete Pavements, American Concrete Pavement Association, Publication No. FHWA HI-96-027, Arlington Heights, IL, October 1996.

62. Trevino, M., “Evaporative Retardant Study,” Technical Memorandum 0147-3, Center for Transportation Research, The University of Texas at Austin, August 2000.

63. Ramaiah, S., “Estimating In Situ Strength of Concrete Pavements Under Various Field Conditions,” Research Report 1700-2, Center for Transportation Research, The University of Texas at Austin, May 2001.

64. Grater, S., “An Investigation toward Performance-Oriented Specifications for Portland Cement Concrete Pavement,” Doctoral Dissertation, The University of Texas at Austin, December 1996.

65. O’Brien, J., “Construction Inspection Handbook: Quality Assurance/Quality Control,” Van Nostrand Reinhold, New York, NY, 1989.

66. Weed, R. M., “Practical Framework for Performance Related Specifications,” Transportation Research Record 1654, Washington, DC, 1999.

67. AASHTO Guide for Design of Pavement Structures, Vol. 2, American Association of State Highway and Transportation Officials, 1986.

68. ASTM C 143/C 143M–00 Standard Test Method for Slump of Hydraulic-Cement Concrete, West Conshohocken, PA, October 2000.

69. ASTM C 231–97 Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method, West Conshohocken, PA, March 1997.

70. ASTM C 138/C 138M–01a Standard Test Method for Density (Unit Weight), Yield, and Air Content (Gravimetric) of Concrete, West Conshohocken, PA, May 2001.

71. Stokoe, K. H., and J. A. Bay, "Development and Preliminary Investigation of a Rolling Dynamic Deflectometer," Research Report 1243-3, Center for Transportation Research, The University of Texas at Austin, September 1995.

72. ASTM C 31/C 31M–00 Standard Practice for Making and Curing Concrete Test Specimens in the Field, American Society for Testing and Materials, West Conshohocken, PA, October 2000.

73. ASTM C 42/C 42M–99 Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete, West Conshohocken, PA, May 1999.

74. ASTM C 39C/C 39M-01, Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens, American Society for Testing and Materials, West Conshohocken, PA, March 2001.

75. Carrasquillo, P. M., "Concrete Strength Testing," Significance of Tests and Properties of Concrete and Concrete Making Materials, ASTM STP 169C, Philadelphia, PA, 1994.

76. ASTM C 496-96, Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens, American Society for Testing and Materials, West Conshohocken, PA, March 1996.

77. ASTM C 78-02, Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading), American Society for Testing and Materials, West Conshohocken, PA, March 2002.

78. ASTM C 293-02, Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Center-Point Loading), American Society for Testing and Materials, West Conshohocken, PA, April 2002.

79. ASTM C 1074-98, Standard Practice for Estimating Concrete Strength by the Maturity Method, American Society for Testing and Materials, West Conshohocken, PA, March 1999.

80. ASTM C 469-94, Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression, American Society for Testing and Materials, West Conshohocken, PA, July 1994.

81. ASTM C 341-96 Standard Test Method for Length Change of Drilled or Sawed Specimens of Hydraulic Cement Mortar and Concrete, American Society for Testing and Materials, West Conshohocken, PA, January 1997.

82. ASTM C 174/C 174M-97, Standard Test Method for Measuring Thickness of Concrete Elements Using Drilled Concrete Cores, American Society for Testing and Materials, West Conshohocken, PA, March 1997.

83. Sansalone, M., and N. J. Carino, CRC Handbook on Nondestructive Testing of Concrete. Chapter 12: Stress Wave Propagation Methods, CRC Press, Boca Raton, FL, 1991.

84. Olson, L. D., "NDE of Structural Concrete with Stress Waves," Proceedings, Nondestructive Evaluation of Civil Structures and Materials, October 1990.

85. Limaye, H. S., and G. J. Klein, “Investigation of Concrete Arch Bridges Using the Impact-Echo Method,” Proceedings, Nondestructive Evaluation of Civil Structures and Materials, October 1990.

86. ASTM C 1042-99, Standard Test Method for Bond Strength of Latex Systems Used with Concrete by Slant Shear, American Society for Testing and Materials, West Conshohocken, PA, May 1999.

87. Malhotra, V. M., “Nondestructive Tests,” Significance of Tests and Properties of Concrete and Concrete Making Materials, ASTM STP 169C, Philadelphia, PA, 1994.

88. ACI 503R-30, Use of Epoxy Compounds with Concrete, Appendix 1, Field Test for Surface Soundness and Adhesion, ACI Committee 503 Report, ACI Manual of Concrete Practice Part 5, American Concrete Institute, Detroit, MI, 1986.

89. ASTM E 965-96, Measuring Surface Macrotexture Depth Using a Sand Volumetric Technique, American Society for Testing and Materials, West Conshohocken, PA, January 1997.

90. Texas Department of Transportation, Test Method Tex-436-A, “Measurement of Texture Depth by the Sand Patch Method,” Manual of Testing Procedures, Volume II, Materials and Tests Division, Austin, TX, September 1995.

91. Uddin, W., A. H. Meyer, and W. R. Hudson, “A User’s Guide for Pavement Evaluation Programs RPEDD1 and FPEDD1,” Research Report 387-12, Center for Transportation Research, The University of Texas at Austin, July 1985.

92. Won, M., T. Dossey, S. Easley, and J. Speer, “CRCP Version 8.0 Program User’s Guide,” Center for Transportation Research, The University of Texas at Austin, January 1995.

93. Dossey, T., S. Easley, S. Grater, and J. Speer, “JRCP Version 6.0 Program User’s Guide,” Center for Transportation Research, The University of Texas at Austin, January 1995.