Rapid Rehabilitation/Replacement of Bridge Decks

Final Report

to
Alabama Department of Transportation

on
Research Project 930-376

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ABSTRACT

The ALDOT has over 4,830 m (3 miles) of major interstate bridges (3 to 5 lanes wide with approximately 55,740 m^2 (600,000 ft^2) of deck) near downtown Birmingham with significant levels of deck cracking and deterioration. The rehabilitation or replacement (R/R) of these decks is obviously a matter of great concern because of the enormous cost and potential for disruptions of traffic.

The objective of this research work was to identify the most viable rapid bridge deck rehabilitation or replacement (R/R) options which can be implemented under staged construction/concurrent traffic conditions. The objective was achieved by analyzing and synthesizing the results of a review of the literature, a mail questionnaire survey to all State DOT's in the U. S., telephone discussions with DOT bridge and maintenance engineers in over half the states in the U. S., in-person meetings with select personnel of the ALDOT from hands-on bridge maintenance and inspection personnel to bureau chiefs of the primary player bureaus, site visits to the Birmingham bridges, discussions and meetings with bridge deck product industry representatives, and site visits to four states to observe and discuss their rapid bridge deck rehabilitation practices.

Execution of this work led to the following conclusions and recommendations:

1. A study should be immediately initiated to investigate and decide on the best means of meeting the excessive interstate traffic load through Birmingham.

2. Immediately initiate a study to determine the remaining fatigue/service life of the Birmingham interstate bridge support girders.

3. If results of the girder remaining fatigue study indicate a remaining life of 15 - 25 years then execute a structural condition assessment program to determine if the decks are sufficiently sound to rehabilitate via overlay.

4. Use an AL79 bridge near Birmingham which is scheduled to be taken out of service in 1999 to help determine the state and best course of action for the Birmingham bridges.

5. Place and monitor the performances of four candidate deck replacement/ rehabilitation "test sections" described in the report.

6. If the results of girder remaining fatigue life study and the deck assessment program indicate rehabilitation via overlay, then place and monitor the performances of two candidate deck overlay "test sections".

7. Immediately expand the scope of study to begin implementing the above recommendations.

FORWARD

This report was prepared under a cooperative agreement between the Alabama Department of Transportation (ALDOT); the U. S. Department of Transportation, Federal Highway Administration (FHWA); the Highway Research Center (HRC); and the Engineering Experiment Station at Auburn University. The PI is grateful to the ALDOT and HRC for their sponsorship and support of the work.

The PI is grateful for the assistance of many people in the ALDOT for giving of their time and expertise in helping with the research reported. Special thanks are due to Fred Conway, Mitch Kilpatrick and Randall Mullins of the ALDOT.

The PI is also grateful to personnel of state DOT offices throughout the U. S. and to bridge deck material, overlay product, and prefabricated deck panel designers and manufacturers for sharing their experience and expertise in bridge deck rehabilitation during the course of this research.

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1. INTRODUCTION

1.1 Statement of Problem

Because of weather/environment exposure coupled with heavy truck wheel loadings and high tire pressures, bridge decks are subject to the most severe loading of all bridge components. This usually results in a deck service life which is less than the other major components of bridges. In Alabama the primary types of deck deterioration are:

- early drying and thermal shrinkage cracking
- weathering from freeze-thaw, wet-dry, hot-cold
- impact and fatigue from truck traffic

Alabama has many bridges which have good substructures and superstructures, but deteriorated decks which need rehabilitating or replacement. Unfortunately, many of these bridges are on heavily traveled interstate highways, e.g., those in the Birmingham, Alabama area, and any rehabilitation or replacement (R/R) scheme must be implementable in a rapid manner with minimal interference with highway traffic. Identification of R/R schemes which are effective and workable for the high traffic volume bridges in the Birmingham area was the impetus and purpose of this research.

1.2 Project Objectives

The overall objective of this research was to identify effective and cost efficient design and construction strategies and procedures for rapid rehabilitation or replacement of bridge decks to include those decks which must be rehabilitated or replaced under conditions of concurrent traffic. Specific sub-objectives of the research work toward that end were:

(1) to identify strategies for rapid rehabilitation or replacement of bridge decks which are at various levels of deterioration from various sources of deteriorations common to Alabama, and which are applicable for implementation under concurrent traffic conditions.

(2) to analyze and evaluate the candidate strategies in (1) above to assess which are most appropriate for the types and levels of deterioration, and the operating conditions found in Alabama.

(3) make recommendations which are appropriate for use during the design (including material selection), and construction phases of rapid bridge deck rehabilitations and replacements under conditions of concurrent traffic.

1.3 Research Work Tasks

The research work tasks to accomplish the above objectives are outline below:

1. Meet with select personnel of the Alabama Department of Transportation (ALDOT) to confirm the common causes of bridge deck cracking and deterioration in Alabama, the operating conditions which deck rehabilitations and replacements must be implemented under, and the R/R strategies and procedures which they currently employ.

2. Review the literature on bridge deck rehabilitation and replacement to identify strategies and procedures which are appropriate for decks in Alabama.

3. Survey other state DOTs to identify the strategies and practices being employed in the U.S. in bridge deck rehabilitation and replacement.

4. Survey other countries, and particularly the OECD member countries, to determine their deck rehabilitation and replacement strategies and procedures.

5. Visit select bridge rehabilitation and replacement sites to verify, judge, and/or identify modifications to the strategies and procedures which are being implemented.

6. Analyze the strategies and procedures currently being used by other agencies to identify those most appropriate for Alabama types and levels of deteriorations and operating conditions, and to assess their applicability under concurrent traffic and/or rapid R/R conditions.

7. Based on (1) - (6) above, identify and develop efficient and effective rehabilitation and replacement strategies and procedures for bridge decks in Alabama which are applicable for rapid implementation under concurrent traffic conditions.

8. Meet with select personnel of the ALDOT, bridge contractors, and concrete repair contractors to discuss and refine the strategies and procedures identified in (7).

9. Make recommendations which are appropriate for use by ALDOT Bureaus and Divisions Offices, and by bridge contractors, in implementing the rehabilitation and replacement strategies and procedures identified and developed.

1.4 Scope of Work

The results, conclusions, and recommendations made in this report are all based on a review of the literature, a mail questionnaire survey, meetings, and phone conversations with the state DOT personnel throughout the country and with industry leaders in the private sector, site visits to bridge sites in Birmingham and to rapid deck overlaying and replacement sites in four states. Information gleaned from these sources were analyzed and synthesized for applicability to the Birmingham bridge deck situation, and recommendations as felt appropriate by the authors were made. This work and its results are what are presented in this report.

2. BACKGROUND AND LITERATURE REVIEW

2.1 Background

As indicated in the research work plan, the first step needed in addressing the appropriate rehabilitation or replacement actions for the Birmingham, AL bridge decks was to meet with select personnel of the ALDOT to confirm the common type and causes of cracking of the decks, and to determine the operating conditions under which deck rehabilitations or replacements must be implemented. Thus, early in the project the PI met with ALDOT's Bridge Engineer, Fred Conway and Maintenance Engineer, Mitch Kilpatrick, to discuss the state and problems with the Birmingham bridge decks. Also, in April 1997, Dr. Ramey visited ALDOT's Third Division Office in Birmingham and met with Division Maintenance Engineer, Bill Davis, and Maintenance Operation Engineer, Mike Mahaffey, and later visited many "typical" deck damaged bridges in the Birmingham area. A summary of the primary information gleaned from that visit follows.

1. The decks of primary concern are located on I-65 and I-59/20 routes through Birmingham and are typically steel girder — concrete deck superstructures where,

    - simple spans are typically 40' - 80' and composite
    - continuous spans are typically 3 span ≈ 70'-100'-70' and noncomposite
    - typical girder spacing is 8'
    - typical deck thickness is 6 ½"

    A more detailed description of the Birmingham interstate bridges and the state of their decks is given in Chapter 4.

2. Most of the larger and very obvious transverse cracks in the top of the decks occurred very early in the life of the bridge and have grown in width and length as the thin decks flex considerably under heavy traffic. These cracks were probably formed as early thermal and drying shrinkage cracks. It should be noted that this is the same conclusion reached by University of Alabama in Birmingham (UAB) researchers (19)in an earlier study.

3. The concrete decks are also badly cracked with hairline cracks in both directions in both the top and bottom of the deck. Very few of these cracks appear to go all the way through the deck. By placing a finger across the underside hairline cracks, one can feel movement on most of the cracks.

4. The bridges are under a very heavy traffic volume.

5. One can feel the bridges deflecting under truck traffic when standing on the deck.

6. Bill Davis indicated that he has not noticed any significant deck crack growth or other indications of a significant rate of deterioration over the past 10 years or so. Mike Mahaffey indicated that he did think that the decks were getting progressively worse (greater cracking and increasing crack width). Note, the Deck-Structural Condition item on the biennial bridge inspection reports was later extracted and plotted for the life of some typical Birmingham bridges to assess the rate of the deck cracking and deterioration. This is shown in Chapter 4.

7. ALDOT has not collected any load-deflection data on any of the damaged bridges. However, they indicated that Auburn's Dr. Stallings has. In checking back at Auburn, Dr. Stallings and graduate student, Eric Stafford, have load-deflection data at each girder under a calibrated truck loading for two bridges--one simple span and the other continuous span. Their work was done during 1994 and could provide the basis for assessing further structural deterioration via repeating a subset of their load-deflection testing five years later i.e, in 1999, if so desired.

8. ALDOT has a little experience with concrete deck overlays. They have had material manufacturers place 2 thin polymer concrete (PC) overlays on 2 bridges near Pell City. Both overlays are approximately 1/4" thick and appear to have been applied with 2 applications of a polymer monomer followed by a broadcasting of fine aggregates. On one of the bridges, the initial overlay came unbonded almost immediately and had to be redone. Today, which is approximately 10 years later, one of the thin PC overlays is badly debonding in large regions and the remainder should be removed. The other overlay, which is the same age but placed by a different manufacturer, appears to be in mint condition. The overlay, because of its exposed aggregate texture, causes more road-tire noise as vehicles cross the bridge than a nonoverlayed deck. This seems to be a minor negative feature. However, it appears to bother Mike Mahaffey and he indicated it appears to bother some Birmingham residents as well.

9. Mike Mahaffey dislikes the use of concrete overlays. He indicated that their bridge decks must withstand loadings such as large rolls of sheet steel falling off of low-boy trucks, and that under such loading overlays will debond and present maintenance problems form the time of debonding until replacement. Thus, he recommends deck replacement rather than trying to rehabilitate the deck via overlaying.

Later, in April 1997, Dr. Ramey met with the ALDOT Chief Engineer, Mr. Ray Bass and Bridge Engineer, Mr. Fred Conway and a group of bridge contractors to solicit the contractors input and suggestions on the Birmingham bridge decks. The contractors recommended that the bridges be widened first, then shift some of the traffic to the new portion and replace the existing decks in stages (a couple of lanes at a time).

If widening of many of the Birmingham interstate arteries through the city due to high traffic volumes is anticipated or planned in the foreseeable future (0-20 years), then the bridge contractors' solution of widening the damaged bridges appears to be a good one. If widening of the interstate arteries is not likely, then alternate solutions such as staged overlaying to buy additional time to assess traffic growth and/or to have most of the bridge major components reach the end of their service lives simultaneously seems more appropriate. Traffic volume demands and ALDOT's laneage upgrading plans for I-65 and I-59/20 in Birmingham need to be determined as they have a significant impact on actions which should be taken on bridge deck rehabilitation.

In later discussions with Mr. George Ray, Chief of the Transportation Planning Bureau, it was determined that the ALDOT is now in the process of planning an additional traffic lane in each direction on I-65 (on the outside of existing lanes) through the Birmingham area. They are also planning the same thing for 120/59 except that it is undecided whether the added lanes will be on the inside or outside or some combination of these. It appears that deterioration of the existing Birmingham bridge decks precipitated or accelerated their planning in this regard. However, Mr. Ray indicated that the interstate system (I-65 and I20/59) through Birmingham is about to or over capacity at this time, and additional lanes are needed now. He indicated that even if the existing decks were in mint condition, a lane addition rehabilitation would be needed in the near future because of heavy traffic conditions.

The ALDOT's plan at this time appears to be to execute bridge lane additions first in order to carry some of the traffic, and then execute some sort of deck, or deck and superstructure, rapid rehabilitation or replacement in a staged construction sequence. Thus, our research task of identifying rapid rehabilitation and/or replacement schemes remain viable in order to address the deteriorating existing bridge deck/superstructure problem.

Based on discussions with ALDOT bridge and maintenance engineers and bridge inspectors, it appears that ALDOT's primary concerns about the Birmingham I-65 and I-59/20 bridge decks are as follows:

1. Inadequate traffic lanes and traffic capacity (on I-65 and on I-59/20 from the I-59/20 juncture to the I-65 interchange in particular).

2. Significant levels of live load deflections and out-of-plane movement of the deck superstructure system.

3. Significant level and rate of increase of deck cracking and deterioration which is requiring ever increasing maintenance attendance in the form of surface spalls and potholes (which generally require full-depth patches), is probably reducing the bending stiffness in both the longitudinal and transverse directions and leading to greater deflections and cracking, and will eventually lead to deck punching shear failures.

4. Extensive state of fine cracking on the deck undersides with a concern for future underside spalling problems which would create a safety hazard, and additional maintenance requirements.

5. Past history of fatigue problems with deck support girders (at the locations of transverse diaphragms) and a concern that the girders may be approaching their fatigue limit/life and need to be replaced.

6. Past history of fatigue problems with diaphragms and diaphragm-to-girder connections.

The typical failure chronology for bridge decks in Alabama appears to be as follows:

- A significant level of early transverse shrinkage cracking
- Growth in width of transverse cracks due to crack movement and abrasion from traffic and environment loadings
- Development of longitudinal cracks at girder edges due to poor longitudinal distribution of truck tire loadings (due in part to extensive transverse cracking)
- Reduced bending stiffness in both the transverse and longitudinal directions due to crack growth which in turn leads to increased deck cracking.
- Local surface spalling requiring ever increasing maintenance attendance
- Eventual deck punching shear failures

Punching shear failures have occurred on some ALDOT bridge decks in the past, e.g., on some I-59 decks near Gadsden, AL and on an AL79 bridge deck near Birmingham. These have been the only deck structural failures in Alabama to the PI's knowledge. Punching shear is a highly localized failure mode and, while obviously not desirable, is good in the sense that it will not typically lead to catastrophic accidents and is relatively easy to repair.

It has been observed in Alabama deck punching shear failures, that they occur in regions of relatively new and growing longitudinal deck cracking (transverse deck cracking is quite prevalent at almost all locations). Thus, inspecting for significant longitudinal deck cracking and identifying effective and efficient repair schemes for such cracks is believed to be a key factor in avoiding deck punching shear failures.

2.2 Literature Review

Summaries of specific rehabilitation strategies, methods and procedures commonly used for highway bridge decks are presented in Chapter 3. A brief review of the literature pertaining to bridge deck deterioration and rehabilitation in general is presented below.

Since deck cracking appears to be the initiating point for most bridge deck deterioration in Alabama, the causes of deck cracking were of particular interest. Bridge deck concrete shrinks as it drys out and cools down, and since it is constrained externally by the bridge longitudinal girders (whether intentionally by shear lugs or unintentionally by adhesion and friction), and internally by the deck reinforcing steel, shrinkage stresses develop which may, and usually do, cause micro and macro shrinkage cracks in the deck. A cooperative study by PCA with ten state DOTs in 1970 (54) found that

- transverse cracking was the predominate mode of deck cracking
- transverse cracking appeared to increase somewhat with age and increasing span length
- combinations of transverse and longitudinal cracking was the most detrimental as these often lead to surface spalls, potholes, or deck punching shear fractures
- on decks supported by steel girders, transverse cracking usually occurred at relatively short intervals throughout their length, regardless of being simple or continuous span structures
- transverse cracks typically occur directly over transverse rebars

Fig. 2.1 shows an example of such transverse cracking which is believed to be primarily caused by drying shrinkage, and a combination of resistance to subsidence when the concrete is in the plastic state and later concrete tensile stress concentrations due to the presence of the top transverse rebars (see Figs. 2.2 -2.3). Results of recent surveys (10, 58, 74) indicate that restrained shrinkage of concrete is the leading cause of bridge deck cracking. The restraint may take the form of internal reinforcing steel, external deck/girder shear connectors, and girder/abutment connections. The drying shrinkage of concrete is caused by a loss of moisture either from evaporation or hydration, and is also affected by the relative paste volume, the aggregate type, and the relative humidity.

There appears to be a consensus of opinion of ALDOT engineers that concrete shrinkage is the primary cause of bridge deck cracking, and in turn, deck cracking is the primary cause of premature deck deterioration and reduced durability. The primary causes of this are felt to be the concrete mixture design and poor quality concrete curing. Thus to effectively mitigate early shrinkage cracking of bridge decks, improvements are needed in the

- concrete mixture design
- concrete curing requirements
- development of relatively simple and reliable methods to assess concrete durability at the time of mixture acceptance and at the time of producing and placing the concrete.

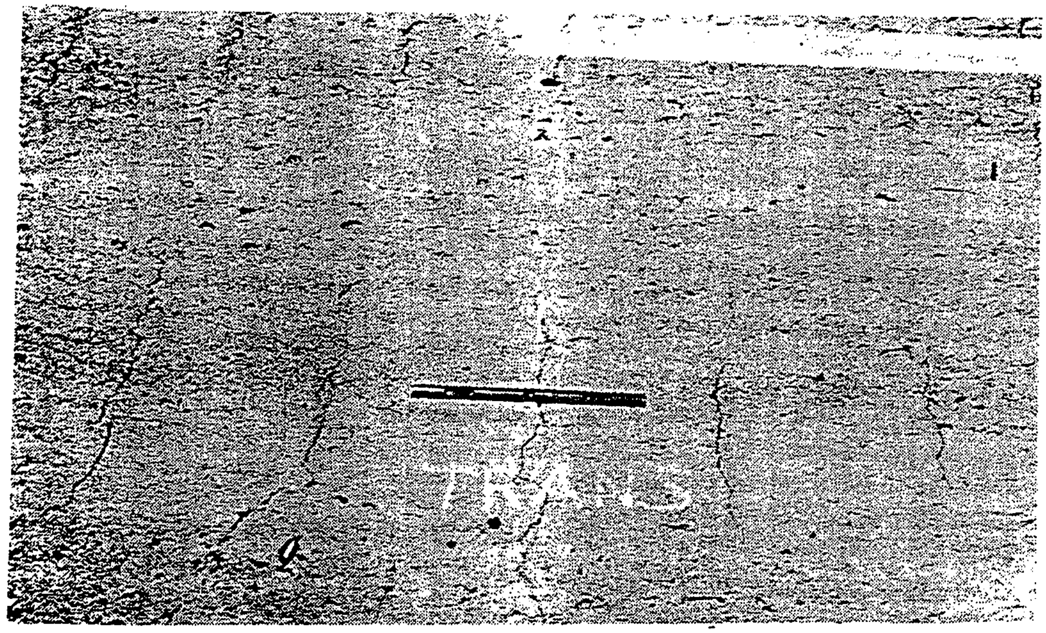


Fig. 2.1. Deck with Truss Reinforcement-Transverse Cracks Developed Only Where Truss Bars are Near Top of Slab (54).

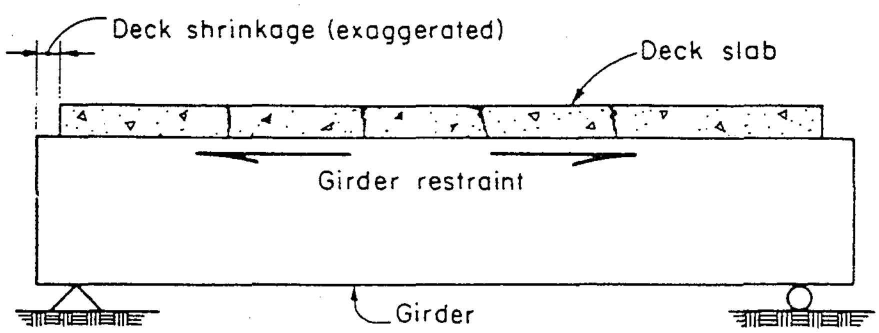


Fig. 2.2. Girder Restrain to Volume Changes (54).

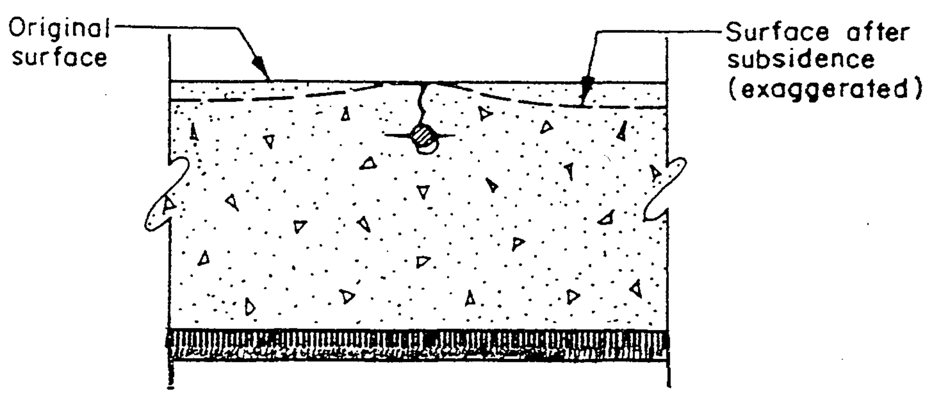


Fig. 2.3. Resistance to Subsidence of Concrete by Top Reinforcement (54).

Fortunately, the mode of concrete deck structural failure is one of punching shear rather than a more extensive and catastrophic 1-way slab flexural failure. Csagoly et al. (23) recently reported the results of a laboratory testing program to assess the behavior and failure mode of concrete bridge decks. They used reduced scale test models and load tested with a concentrated load (to simulate a truck tire loading). They found that as the deck deflected, the internal deck arch became shallower and increased the deck concrete arching stresses. When the deflection was half of the deck depth, the arch essentially flattened out and became compressively unstable under further loading. When the maximum deflection exceeded half the deck depth, punching shear failure occurred (see Figs. 2.4 - 2.7). Csagoly et al. also performed punching shear proof load testing in the field to 5 times the maximum anticipated wheel load.

Dorton, et al. (25) conducted laboratory and full field testing on bridge decks in Canada subjected to point loads and found in both cases that the decks transmitted point loads to the support girders by internal arching action rather than flexural action. They conducted punching shear load testing of the field decks with support girder diaphragms in-place and removed, and found there was a significant increase in deck deflection when the diaphragms were removed. They did not carry the field testing to failure, but estimated a punching shear reduction when the diaphragms are removed.

Beal (11) also reported the results of a reduced scale model experimental testing of the load capacity of concrete bridge decks. His results indicated punching shear failures (rather than flexure) from simulated wheel loadings at load levels at least six times those of design loads.

Kato and Goto (42) conducted laboratory testing on a 15' x 6' x 6" bridge deck model to assess the effects of water infiltration of cracks on deck deterioration. They purposely generated significant top surface cracking by inducing plastic shrinkage cracking when the model deck was first cast (see Fig. 2.8a). The interesting thing about their testing is that after several point load applications, the bottom surface of the model deck looked very much like the underside of the I-65 and I-59/20 decks in Birmingham (see Fig. 2.8c, d). These results along with personal observations of the large longitudinal and transverse deflections of the Birmingham I-65 and I-59/20 decks would lead one to believe that the cause of the extensive cracking is flexure due to truck traffic.

The AASHTO Manual for Bridge Maintenance (2) recommends that "decks with severe cracking should be sealed with a good waterproofing membrane and overlayed with asphaltic concrete".

A new type of spring pin shear connector was recently developed in London (57) for strengthening and extending the fatigue life of existing bridge decks constructed of steel girders

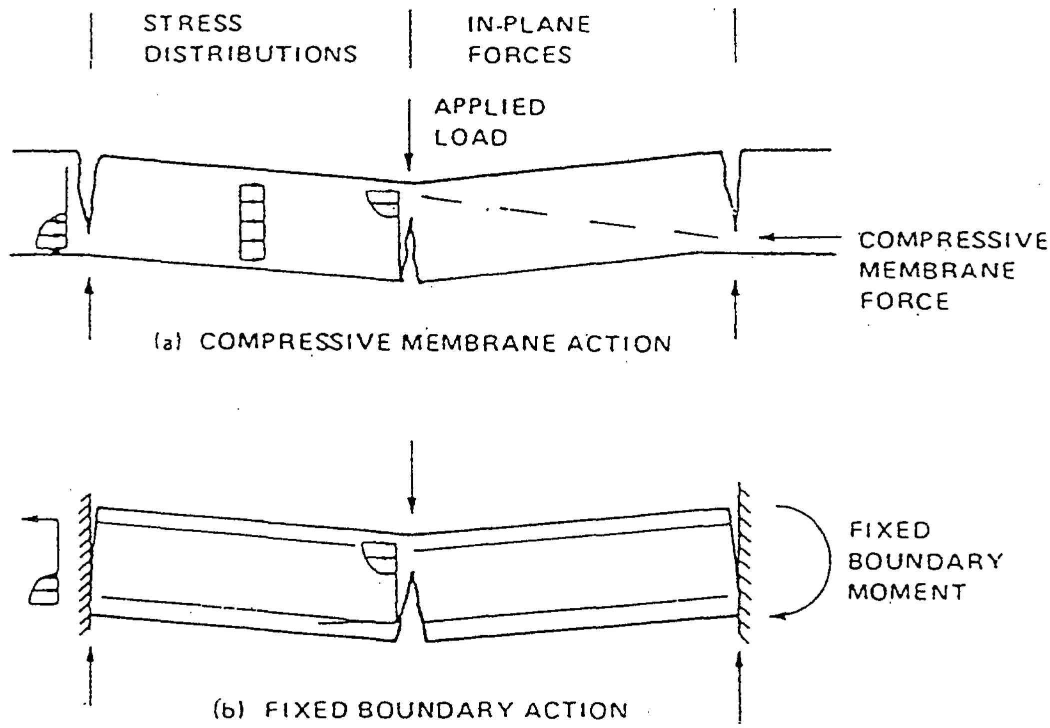


Fig. 2.4. Arcing Action in Deck Slabs (23).

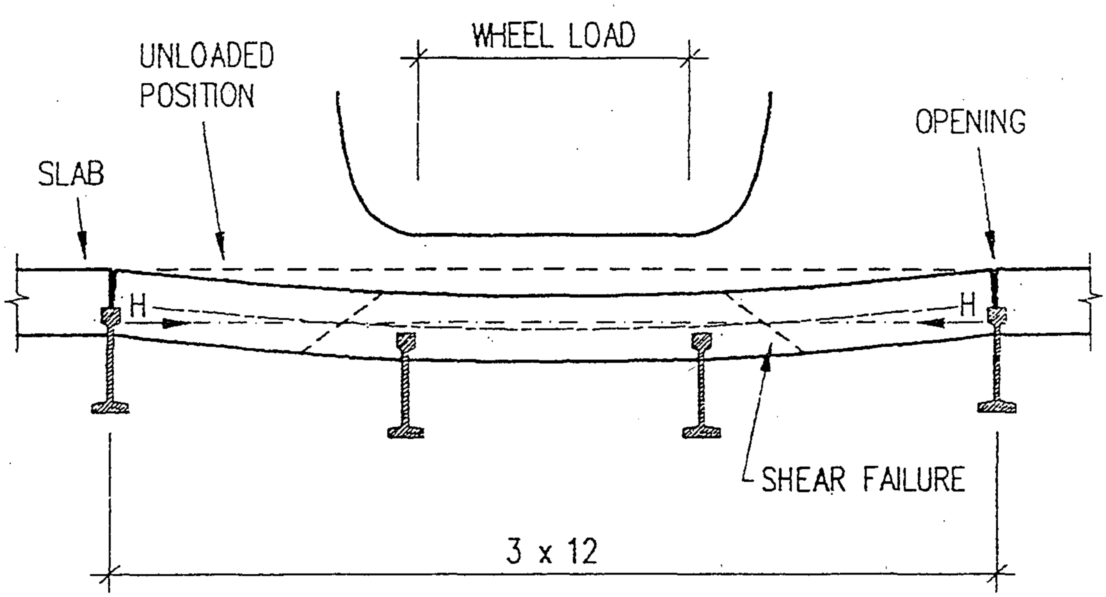


Fig. 2.5. Punching Shear Failure Mode (23).

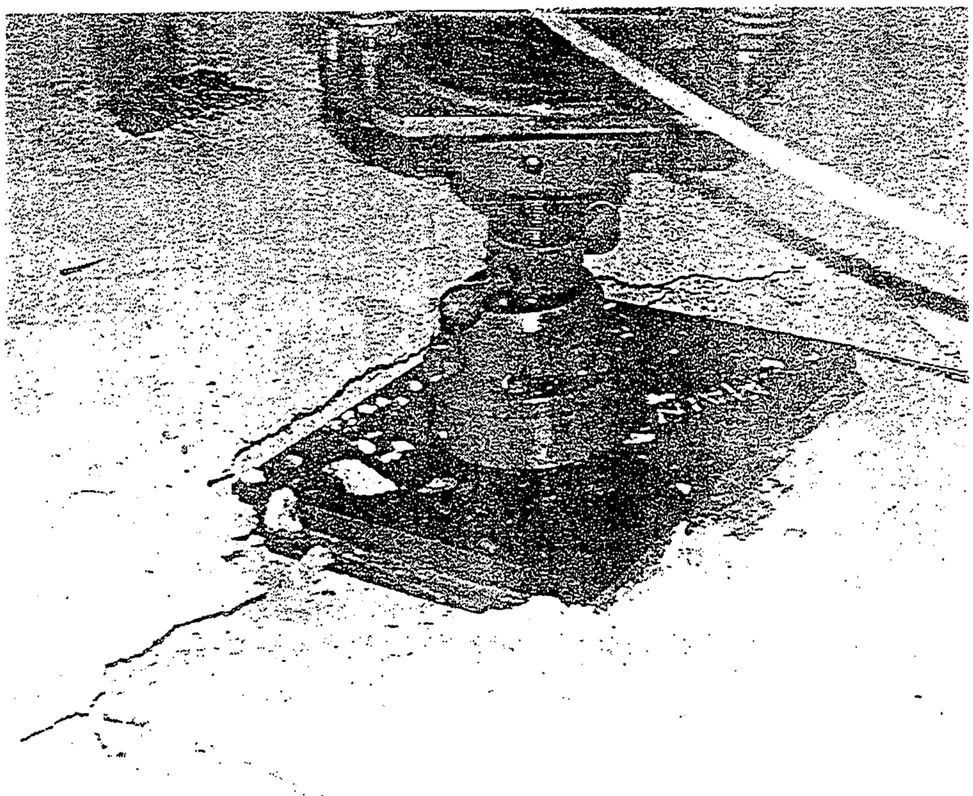


Fig. 2.6. Slab Under Punching Failure (23).

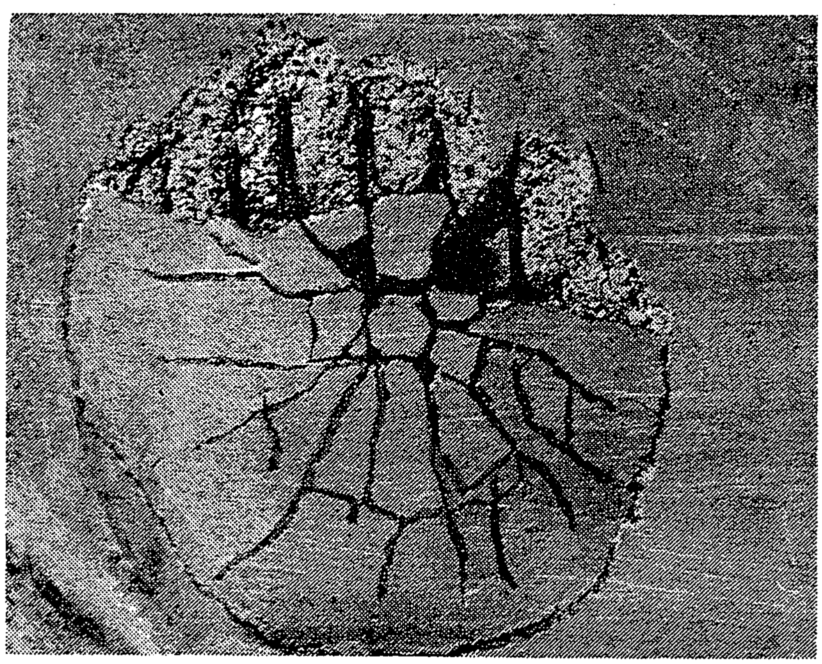


Fig. 2.7. Punching Shear Failure - Underside of a Slab.

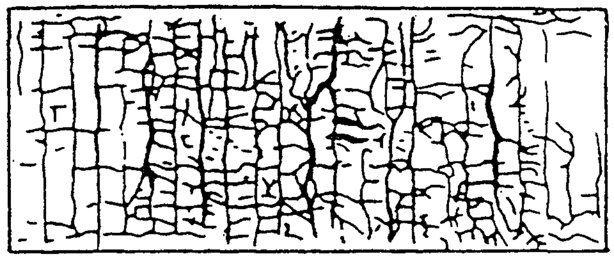
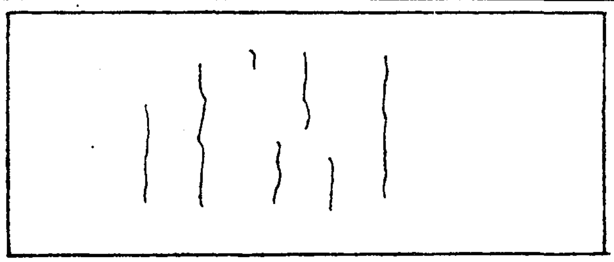
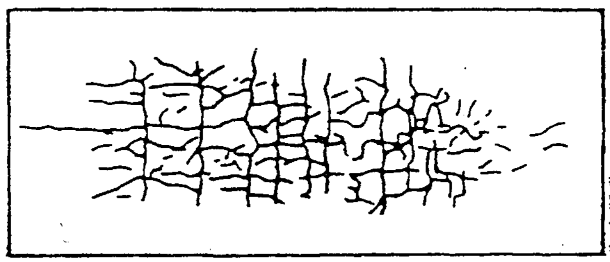
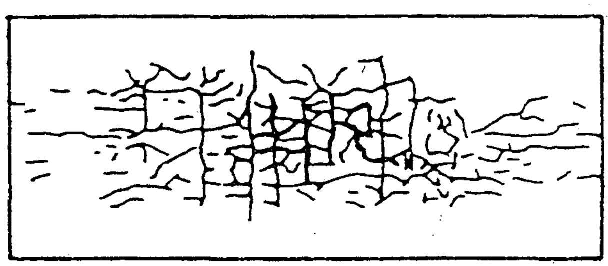
(a) Before loading (top surface of slab). Thick lines represent cracks more than 0.5 mm wide. Cracks purposely created by plastic shrinkage.	
(b) Before loading (bottom surface of slab). Transverse penetrating cracks were observed.	
(c) After loading of 78 kN loads at 21 places around slab. Grid pattern cracks developed (bottom surface of slab).	
(d) After fatigue test with 1.2 x 10^7 repetitions of 93 kN loads (bottom surface of slab).	

Fig. 2.8. Change in Cracks of 15' x 6" x 6" Slab Specimen (42).

supporting and composite with reinforced concrete deck slabs. The development was precipitated by a recently required upgrading of the viaducts carrying London's Docklands Light Railway (DLR) shown in Fig. 2.9. More specifically, the fatigue life of the standard welded studs which provided composite actions between deck slab and twin steel plate girders needed to be extended via the addition of additional shear connectors.

Technically the easiest solution was to drill out holes through the deck slabs from above and install new 19 mm stud connectors in clusters of three of four to optimize the hole size and economies of installation. Practically, this would be extremely disruptive and costly to undertake with a live service operation.

The upwards installation of new shear connectors through the top flange of the steel girders was considered as a viable option. Several types of connectors were considered, including 20 mm diameter spring steel tension pin fasteners. These offered the advantage of a readily achieved force fit into a hole drilled up through the steel flange and lower section of the concrete deck slab with no requirement for traffic-sensitive grouting, glueing, welding or traffic disruption.

Tension pins are now a universally accepted fastener. They are compressible hollow spring steel tubular pins with a chambered driving nose. A force fit is obtained by jacking or hammer driving the pin into a drilled with slightly smaller diameter. There are two types, either a 'Spirol' pin which relies upon a force fit by tightening during driving of its 2 1/4 turn spiral coil, or a cylindrical pin with a longitudinal slot, which closes on driving (see Fig. 2.10).

The number of spring pin fasteners required as additional shear connectors for the composite beam decks is a function of the fasteners stiffness. Thus, the stiffer the spring pin fastener the less drilling and pin insertion would be required on-site, with obvious economic benefits. This gave a strong incentive during testing to produce the stiffest fastener.

The spiral 'Spirol' type spring pins (Fig. 2.11) had somewhat better properties as shear connectors than the slotted pins and were chosen for installation. Close tolerance holes were diamond drilled between the existing welded stud connectors and cadmium plated 'Spirol' pins were jacked into place with the assistance of a special lubricant (Fig. 2.12). All operations were undertaken without interfering with the frequent train service. The pins were inserted 2 mm into the steel flange and the holes filled with an approved sealant prior to local repainting.

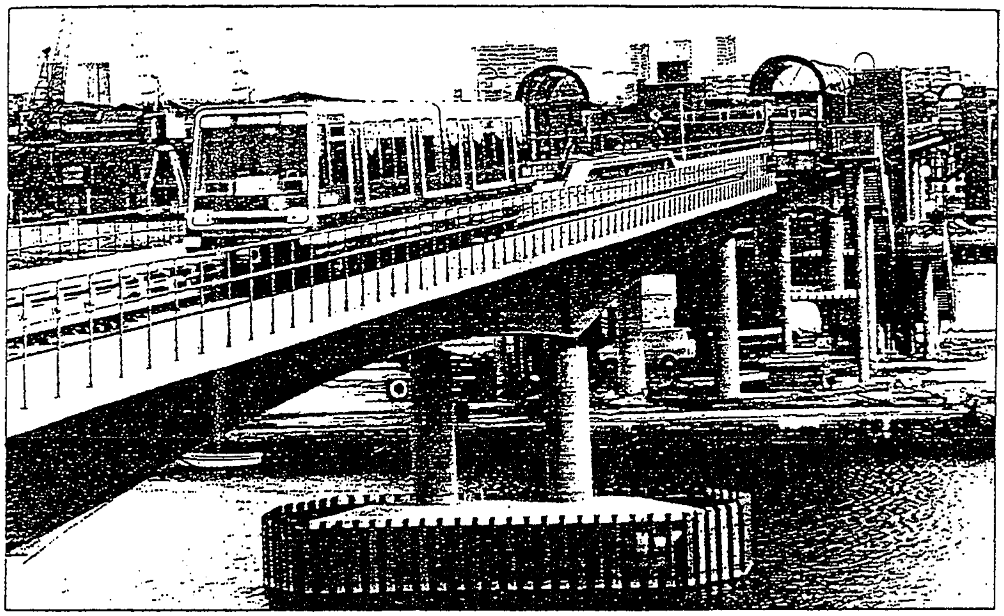


Fig. 2.9. The Original DLR Viaduct (57).

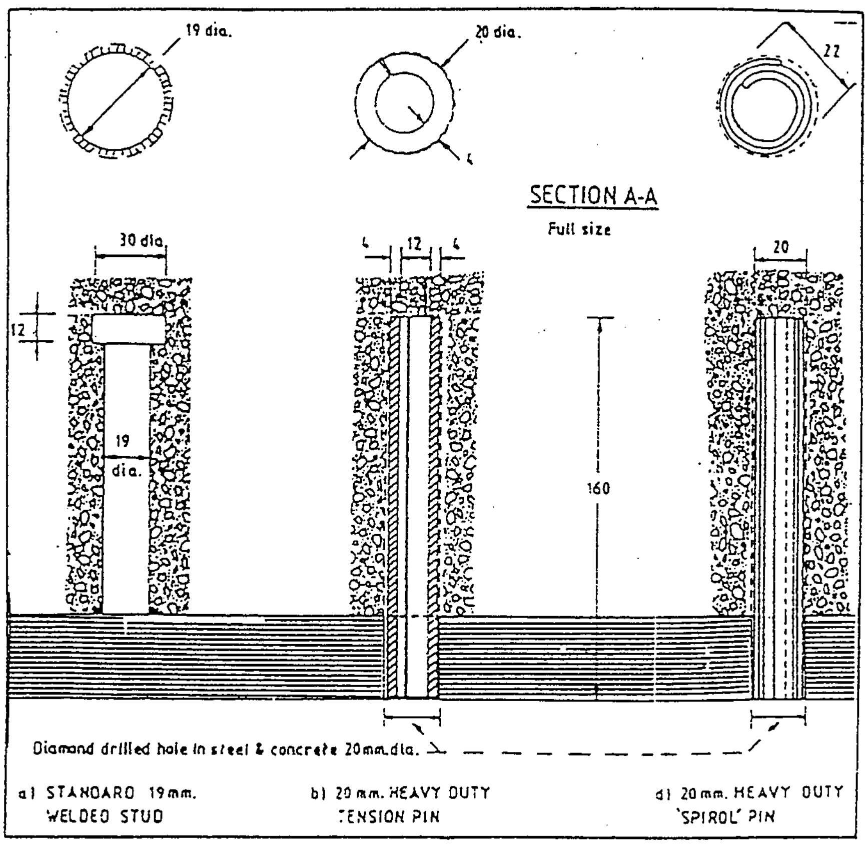


Fig. 2.10. Welded Stud and Spring Pins Tested (57).

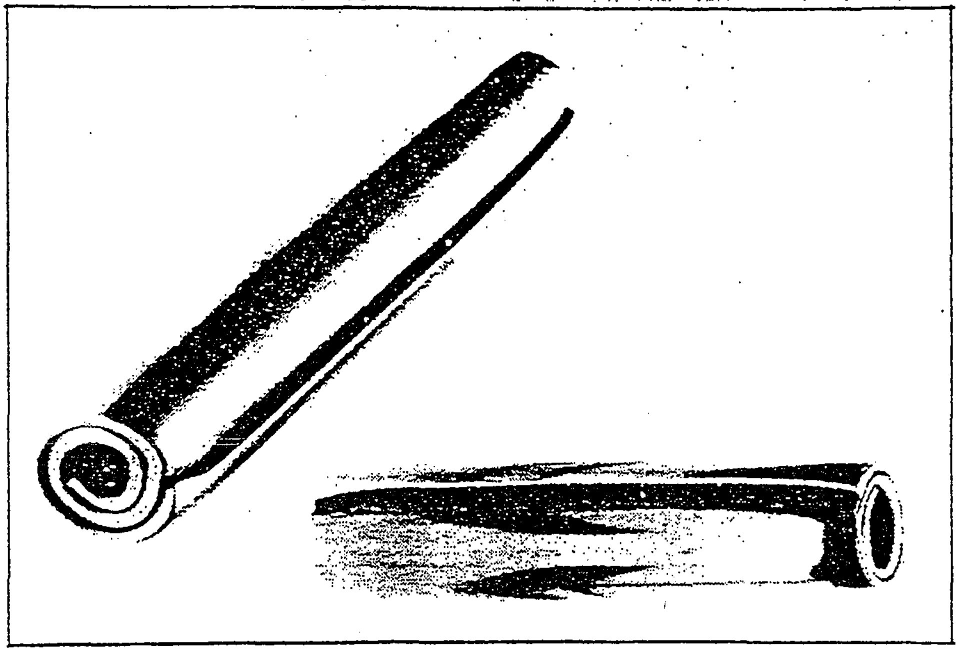


Fig. 2.11. 'Spirol' Tension Pin (57).

Fig. 2.12. Jacking in 'Spirol' Pins for DLR Upgrading (57).

Bridge decks can be effectively protected and/or rehabilitated by deck overlays which (14)

- Protect against the impact of heavy trucks and the further intrusion of chlorides, gasoline, acids, solvents, oils, and other contaminants
- Prevent carbonation
- Correct uneven surfaces created by wear
- Provide a nonskid riding surface
- Create a uniform appearance

An overlay should (14)

- Have ample strength
- Be a good sealant
- Resist freeze-thaw
- Adhere well to concentrate substrate
- Be easy to apply
- Be cost effective

Chamberlain and Weyers (20) state that among the treatments in the mainstream of current practice for rehabilitating bridge decks, only latex-modified concrete (LMC) and low-slum dense concrete (LSDC) overlays have been used frequently enough and long enough to provide reliable estimates of how they will perform. The first LMC bridge deck overlay was placed in 1957, and the first LSDC overlays in the early 1960s. By 1977, twenty one states reported one or both overlays as a standard practice for deck rehabilitation, and by 1989, that number had increased to 37 states. Chamberlain and Weyers summarized the findings of their field performance investigation as follows:

> The use of thin, high performance concrete overlays to rehabilitate corrosion-damaged concrete bridge decks in the United States and Canada has been one of the highway industry's success stories of the last 20 years. Experience suggests that these treatments have the potential for extending the useful life of the riding surface of decks for considerably longer than had previously been thought. Variations in climate, traffic volume, and overlay type and thickness appear to be far less important determinants of their performance than the methods used to prepare the deck before the overlay is placed. When concrete removal criteria are based on half-cell potential rather than present damage, when removal of chloride contaminated concrete is extended to below the rebar, and when the substrate is sandblasted to remove microcracking prior to cleaning, service life potentials of 30 to 50 years are likely.

Ramirez (61) reported the results of his field evaluation of three types of polymer concrete overlays, i.e, a 3/4" thick polyester resin overlay, and 1/4" - 3/8" thick multiple layer epoxy binder and epoxy-urethane co-polymer binder overlays. Each type of overlay was placed on two separate bridge decks, and evaluated after five years. The polyester resin overlays experienced construction problems and later exhibited moderate amounts of cracking, spalling and debonding. Both the epoxy and epoxy-urethane are providing good long-term performance and are recommended by Ramirez for standard use in overlaying bridge decks.

The New York State Department of Transportation (DOT) has found that deep concrete removal, and the quality of the removal and reconstruction have the greatest potential for extending the life of a repaired or renovated concrete bridge deck. Their estimates of extension of service life for various deck renovation schemes are shown in Table 2.1.

CALTRAN commonly uses both thick polyester polymer concrete (PPC) overlays (¾" - 3") and thin PC overlays (1 layer of methacrylate primer/sealer with broadcasted sand). Their typical thick PC overlay is 3/4". Because of the relatively high cost of PC, when overlay thicknesses greater than 3" are needed, CALTRAN uses a portland cement based overlay material.

Table 2.1. New York State DOT Estimates of Extension of Service Life

Rehabilitation Scheme	**Estimated Extension of Service Life (years)**
Asphalt Overlay	4
Asphalt Overlay with membrane (resurfaced after 11 years)	22
Concrete Overlay with select deep removal*	25
Concrete Overlay with 100% deep removal*	35
Deck Replacement	40

*Deep removal is to level below the top rebar mat.

CALTRAN indicate that they have been placing thin PC overlays for improving skid resistance and for preventative deck maintenance (healing of deck cracks to mitigate entry of water and in turn mitigate later deck delaminations and spalling). They estimate that they have placed 50 thin PC overlays a year for the past 12 - 13 years (approximately 600 overlays) and have experience delaminations problems on only 1 or 2 of these decks. CALTRAN's cost estimates for the methacrylate thin overlay treatment are as follows:

Methacrylate thin overlay in place	- \$1.50/ft^2
Deck preparation - sand or shot blasting	- \$1.00/ft^2

To CALTRAN, the primary attractive features of the methacrylate overlay treatment are its excellent crack healing, bonding to concrete, and skid resistance. Even after the overlay wears out from abrasion, they feel that they have protected the original concrete deck from water penetration and concrete spalling.

To CALTRAN, the primary attractive features of the thick PPC overlays are their rapid strength gain (and minimum lane closure time), and their excellent bonding to existing deck concrete. If the existing deck has no delaminations and/or other reasons for surface concrete removal then CALTRAN prepares the deck simply by sand or shot blasting as for a thin overlay. If removal of a portion of the deck top surface is required, it is normally accomplished by hydrodemolition. CALTRAN's cost estimates for thick PPC overlay placements are as follows:

Thick PPC Overlay in place	-	\$110.00/ft^3
Deck preparation - sand or shot blasting only	-	\$1.00/ft^2
Deck preparation - hydrodemolition	-	\$65/ft^3

Thus the unit cost of a 3/4" thick overlay on a deck prepared by shot blasting would be:

Deck preparation	-	\$ 1.00/ft^2
3/4" PPC overlay in place	-	\$ 6.88/ft^2
		\$ 7.88/ft^2 or \$70.88/yd^2

In telephone discussions with CALTRAN's Michael Lee, Chief of Structure Maintenance Design, regarding their rehabilitation approach to bridge decks such as those in Birmingham (he had detailed photos of the Birmingham decks showing the state of cracking), he stated that CALTRAN would do the following.

1. Test for ASR problem.
2. Chain/sound top of deck for delams.
3. Because of severe bottom of deck cracking, "chain" or equivalent the underside of the deck for delams.

4. If there is an ASR problem, or if there is significant delamination of the underside of the deck, then replace the deck.

5. If there is not an ASR problem and there is not significant underside delaminations, and the extent of the top surface delaminations are reasonably low (say 20% or below), then remove the delaminated concrete and overlay the deck with polyester PC.

6. If the only damage is significant deck cracking, then seal/heal the cracks via squeezing on a low viscosity methacrylate and broadcasting aggregate (CALTRAN calls this a surface treatment and others call it a thin PC overlay).

It should be noted that if No. 6 above is the case, then following the crack sealing/healing with a ¾" PPC overlay may be a better approach in that a thicker overlay will stiffen the deck and should somewhat reduce future deflections and cracking.

Michael Sprinkel (3) reported on the performance of polymer concrete bridge deck overlays ranging in ages from 6-19 years, based on the data obtained in the SHRP Project C103. The type and ages of the polymer concrete overlays that they evaluated are given in Table 2.2. Sprinkel's conclusions were:

- Multiple-layer epoxy, multiple-layer epoxy-urethane, and premixed polyester polymer concrete overlays can provide a skid resistant wearing and protective surface on bridge decks for 20 years or more.

- Multiple-layer polyester overlays have a life of about 10 years.

- The data for methacrylate slurry overlays is inclusive.

Table 2.2. Age of Polymer Concrete Overlays at Time of Assessment (3).

Overlay Type	**Age (Years)**	**Average Age (Years)**
Multiple-layer Epoxy	9, 9, 19	12
Multiple-layer Epoxy Urethane	11, 8, 8	9
Premixed Polyester	11, 12, 11	11
Multiple-layer Polyester	10, 11	11
Methacrylate Slurry	9, 6, 6	7

The project PI, Dr. Ramey, had several conversations with Michael Sprinkel, Research Manager, Virginia Transportation Research Council about SHRP Project C103 and their findings on deck overlays. In the course of these conversations, they also discussed the Birmingham bridge decks (Mr. Sprinkel was provided photos showing the state of cracking of the decks). Sprinkel's comments and recommendations on the Birmingham decks were as follows:

1. Use of low viscosity gravity filled polymer crack sealer/healer and a PC overlay would probably buy some time.

2. The full depth cracks will reflect through no matter what type overlay is used, and generally people do not like to see cracks in a new overlay.

3. The bottom of the deck extensive hairline cracking is of concern to him. He feels this cracking pattern and extensiveness is indicative of (1) a concrete ASR problem or (2) the deck concrete had a lot of water in it and shrunk considerably with time. If it is an ASR problem, we should replace the deck. If it was excessive water in mixture, we may still want to replace the deck because of low quality concrete. He is worried about bottom spalling in the future and dropping chunks of concrete onto vehicles below.

4. Virginia DOT has used prestressed deck panels to speed up deck construction or replacement. The type that they used were essentially S.I.P. forms which act compositely with a 5" cast-in-place top portion. Virginia DOT stopped using these as they resulted in longitudinal cracks (at panel ends) vertically above each bridge girder. Texas and some other states still use these panels.

As part of their NCHRP investigation on "Rapid Replacement of Bridge Decks", Tadros, et al (68) conducted a nationwide survey of bridge owners, designers, and contractors to determine current practices for deck replacement as well as possible improvements in deck replacement procedures. Based on their survey, they determined the relative importance ranking of various influencing factors on bridge deck replacement systems shown in Table 2.3. Tadros, et al. also developed two new prestressed panel rapid deck replacement systems. These are described and discussed in Chapter 3.

In 1994 the New York Thruway Authority (NYTA) placed three rapid deck "test" replacement systems at night under concurrent traffic conditions. The three systems were:

- 7½" precast concrete deck panels
- Half-filled steel grid panels
- Exodermic panels (approximately 18' x 6.5')

and were economically competitive. The NYTA's later assessment of the three systems was as follows:

- The poured-in-place/injected concrete support shoulders needed for the precast concrete panels failed in short order under the pounding traffic loading.

Table 2.3. Relative Importance of Various Influencing Factors on Bridge Deck Replacement Systems (68)

FACTOR	RELATIVE[1] IMPORTANCE
Structural Performance	10
Structural Requirements	7.9
Traffic Control During Construction	7.9
Protection Measures For New Deck	7.0
Deck Material	6.9
Volume of Traffic & Importance of Crossing	6.9
Life Cycle Cost	6.9
Method of Removal & Installation	6.8
Equipment & Level of Skill Required	6.0
Relative Initial Cost	6.0
Cost of Bridge Partial or Full Closure	5.9
Contractor's Availability & Experience	5.4
Composite & Noncomposite Design	4.9
Possible Future Replacement	4.6
Girder Material	4.3
Sources of Deterioration	4.1
Contractor's Incentive to Accelerate Work	3.5
Environmental Restrictions	3.4
Night Construction	1.9
Innovative Features	1.0

[1]10 is considered most important and 1 is least important

- The half-filled steel grid panels performed very well, but the field bolting required to connect the panels was very time consuming and slowed construction time.

- The Exodermic panels performed very well, except that the top wearing surface left something to be desired.

The NYTA is currently preparing to replace the bridge deck on a major bridge (over 250,000 ft^2 of deck) under concurrent traffic conditions (lane down time from 8:00 pm - 6:00 am) and will use Exodermic panels based on the results of the 1994 "test" replacement systems mentioned above. These panels will be of the new Exodermic design and will be 23' wide x 12' long. The NYTA plans to place a thin wearing surface of a proprietary product which is similar to a thin EPC overlay, once the panels are in place. Exodermic deck panels are described and discussed in Chapter 3.

Brinckerhoff (14) recommends that remaining service life should be evaluated for all structures for which deck replacement is contemplated. The AASHTO has published "Guide Specifications for Fatigue Evaluations of Existing Steel Bridges" (1), to help evaluate the remaining fatigue life for existing steel bridges. The formulae presented in this publication are reported to be difficult and not straightforward, and rely heavily on empirical factors and adjustments. Brinckerhoff (14) reports other methods of determining remaining fatigue life. He describes the procedure outlined by Dr. John Fisher in the Guide to 1974 AASHTO Fatigue Specifications as follows:

> The first step in determining the remaining fatigue life is the determination of the number of actual stress cycles that have occurred over the history of the structure. Fisher developed the methodology to derive the number of stress cycles given the average daily truck traffic (ADTT) and known life (days) of the structure. This can be expressed as:
>
> $$N = (ADTT)DL(\alpha)^3/C$$
>
> where
>
> N = Actual number of stress cycles
>
> $ADTT$ = Average daily truck traffic
>
> DL = Life to date, in days
>
> α = Member stress factor: 0.8 for transverse members and 0.7 for longitudinal members
>
> C = A factor relating actual stress range to the gross vehicle weight distribution
>
> Miner's linear fatigue damage equation, described in the same article, provides a method of determining the remaining availability cycles at a given stress range:
>
> $$n_1/N_1 + n_2/N_2 = 1$$

where n_1 = The actual number of cycles that have occurred at the historical stress range, calculated above

N_1 = The allowable number of cycles at the historical stress range, drawn from Figure 2.12

n_2 = The available number of cycles at the future stress range

N_2 = The allowable number of cycles at the future stress range, drawn from Figure 2.12

When the actual remaining cycles have been determined, Fisher's design life formula can then be used again to determine the remaining life, replacing historical data with expected future data.

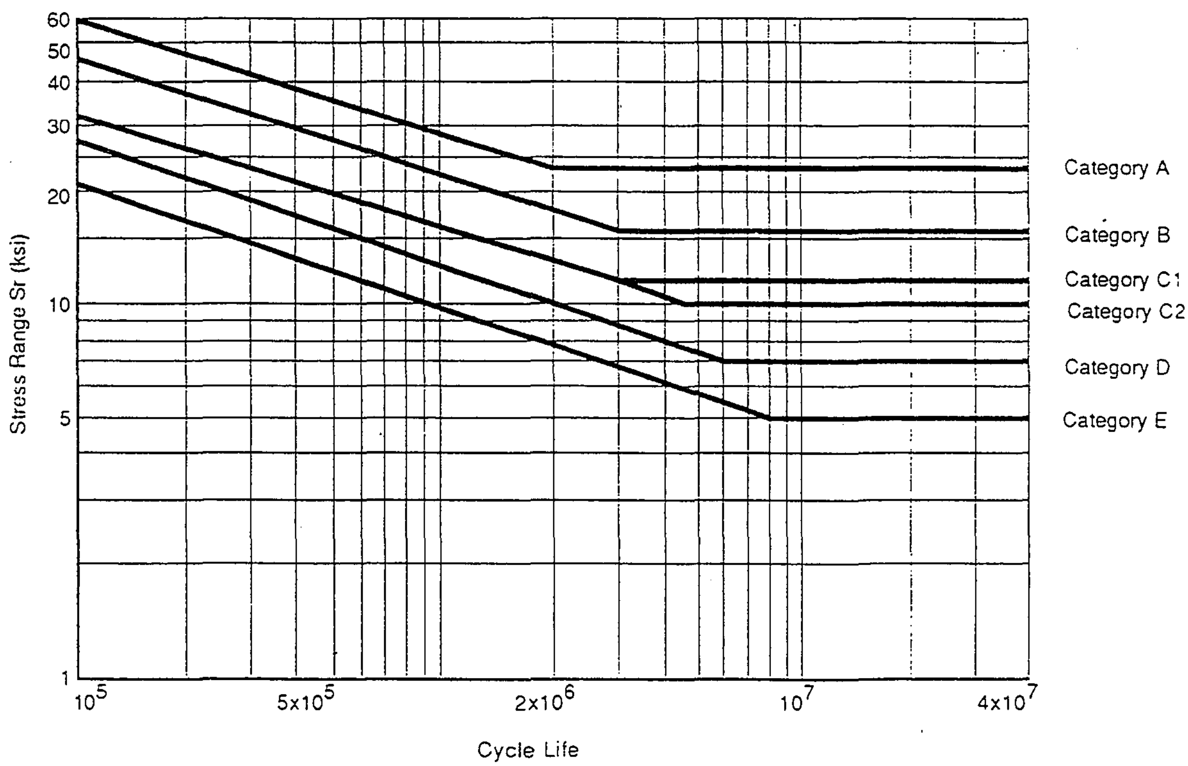


Fig. 2.12. Number of Cycles vs. Stress Range (14).
(As presented in 1974 AASHTO Fatigue Specifications)

3. BRIDGE DECK REPLACEMENT AND REHABILITATION OPTIONS

3.1 General

Beyond minor spall area, loose joint angle iron, clogged drainage pipe, and other such repairs, there comes a time in a bridge's history when its state of deck deterioration has reached such significant proportions, that a major decision must be made on whether to

1) rehabilitate the deck
2) rehabilitate the deck and superstructure
3) rehabilitate the whole bridge
4) replace the deck
5) replace the deck and superstructure
6) replace the whole bridge

The parameters and considerations that go into making that decision are numerous, interrelated, and quite complex. They are also beyond the scope and purpose of this investigation.

This study is focused on what are the most feasible (technically, constructionally, and economically) options for bridge deck replacement or rehabilitation which can be implemented under conditions of concurrent traffic (or minimum "close-down" time). Hence, a discussion of commonly used and newly emerging options for bridge deck replacement and rehabilitation are presented below. Also, because some degree of superstructure stiffening/strengthening is generally needed or appropriate when performing a major deck rehabilitation or replacement project, viable options in this area are also discussed.

3.2 Deck Replacement Options

In the case of the Birmingham bridges, if a deck replacement option is taken, an additional support girder should probably be added between each existing girder to reduce the flexibility and live load deflections of the bridges.

A prerequisite for deck replacement are superstructure and substructure systems that can be expected to be in service for a long period of time. The available replacement systems vary in weight between approximately 22 and 100 pounds per square foot. The lighter weight systems, when used to replace heavier deck types, can improve dead-load stresses, allow larger than original live loads, and even introduce the possibility of roadway widening without complete bridge replacement.

The most common deck replacement options include

- Cast-in-place concrete
- Precast concrete
- Steel grid (open or filled)
- Steel orthotropic
- Aluminum isotropic
- Exodermic
- Inverset
- FRP (Fiber Reinforced Polymer) composite decks

Each of these deck replacement options is discussed below.

Cast-In-Place Concrete. Cast-in-place (CIP) concrete using normal weight or light weight concrete appears to be the most popular redecking option. Most bridge owner/operators consider cast-in-place concrete decks their norm for new construction, and utilize this type of construction for deck replacement as well whenever possible. The AASHTO LRFD Bridge Design Specifications gives an alternate design method for full depth CIP bridge decks. This design is based on an empirical method rather than the conventional transverse strip-continuous beam design. This method allows for much less transverse reinforcement to be used, due to the low transverse tensile bending stresses developed in bridge decks due to traffic loads. This reduces the chance of corrosion in rebar, and therefore reduces the chance of spalling.

Advantages:

- Low cost
- Good durability
- Ready availability of materials
- Well-known construction methods
- Allows for field adjustment of riding surface profile
- Excellent riding, wear, skid resistance surface

Disadvantages:

- Relatively high weight/unit weight
- Manufactured in the field—can result in reduce quality
- Slow construction speed and high cost of field forming and placement of steel rebar
- Significant curing time required—can result in not getting traffic back on roadway quickly

It should be noted that the use of fiber reinforced CIP deck slabs is currently being explored in Canada, and is based on the fact that, when subjected to concentrated wheel loads, a significant arching action develops in reinforced bridge decks. This causes the deck slab to fail in punching shear rather than flexure, and thus, bridge decks may/should be designed for punching shear instead of flexure. When this is done, some or all of the reinforcing steel is replaced with steel or polypropylene fibers. In order to ensure arching action, a lateral restraint is attached to the deck slab by welding a transverse steel strap to the top flanges of the girders. Advantages of using fiber reinforced decks is that corrosion of the deck steel will be diminished, and reduction in deck rebar allows for faster construction. Also, some states allow the used of epoxy coated welded wire fabric (WWF) in place of reinforcing steel in CIP deck slabs. This reduces time and cost of placing reinforcement by achieving coverage of large areas in a short time. It should be noted that the fatigue resistance of WWF is less than that of reinforcing bars although limited testing shows no problems to date.

Precast Concrete. Modular precast concrete involves placing precast panels and attaching them to the superstructure through blockouts filled with cast-in-place concrete. Shear studs or roughened concrete surfaces then provide the necessary horizontal shear connection.

Precast panels which are of prestressed concrete are normally used, and these may or may not be post-tensioned longitudinally after they are placed. Panels, which are 4 to 5 ft in width, are placed next to one another transverse to the stringers. A female-female keyway, filled with epoxy mortar, is used to join the panels together. Should the panels be used in conjunction with a steel superstructure, voids in the panels allow for stud shear connectors to be placed on primary members once the panels are in place. After installation of the studs, the voids, like the keyways, are filled with epoxy mortar (see Figs. 3.1 and 3.2).

Advantages:

- Excellent quality control of deck panels
- Allows prefabrication of deck panels to speed up the deck replacement
- Provides an almost immediate riding surface for traffic
- Relatively low cost (though higher than CIP concrete)
- Allows potential for longitudinal post-tensioning and a nearly crack free deck and stiffer deck

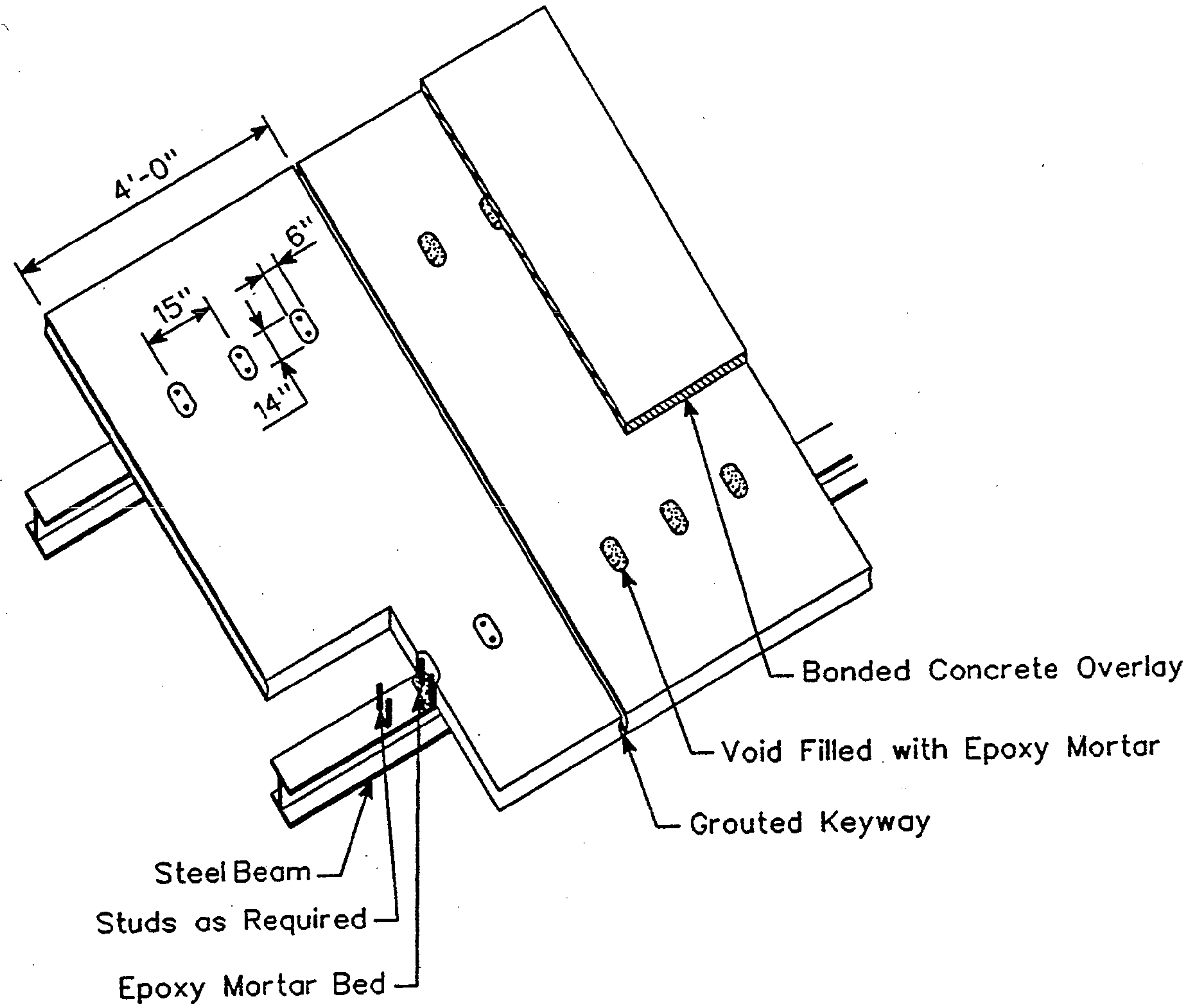
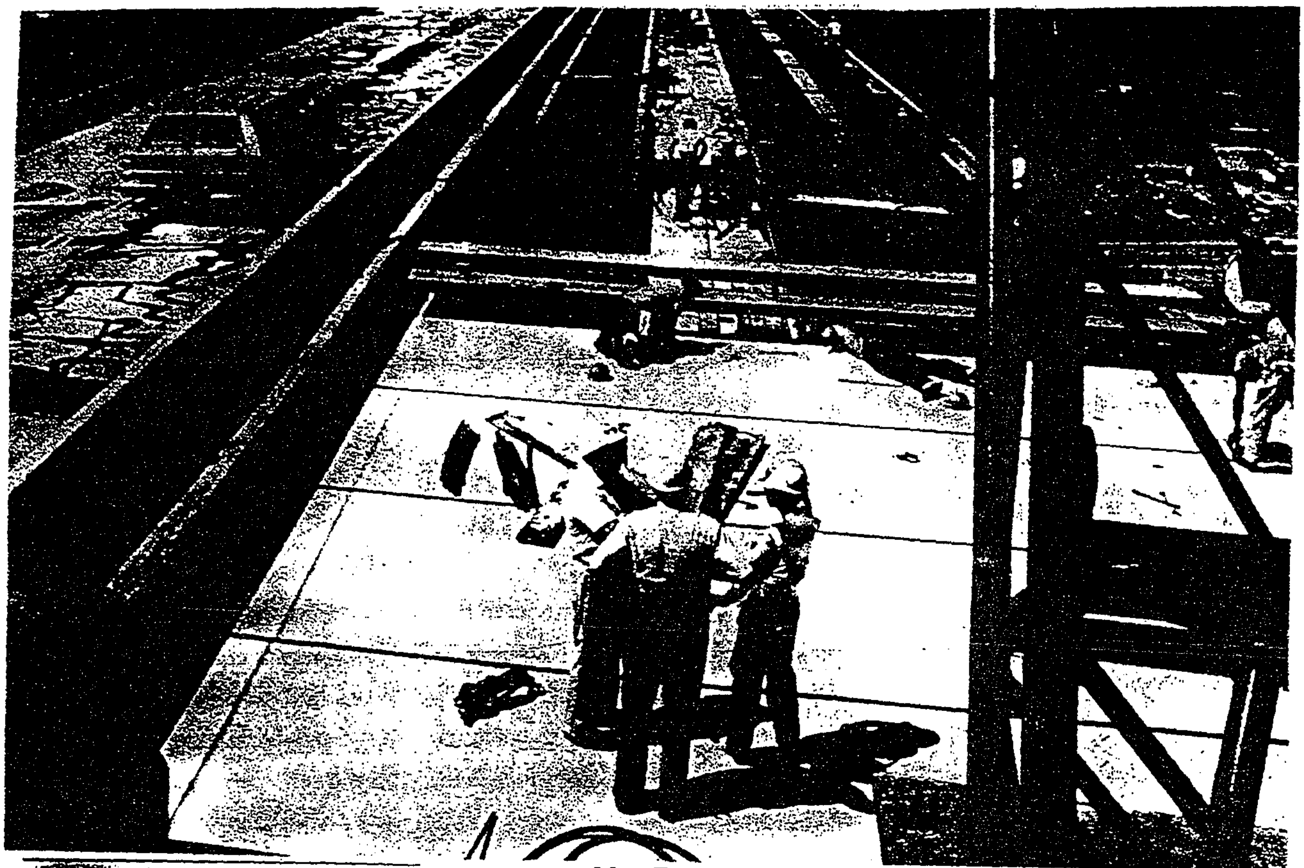
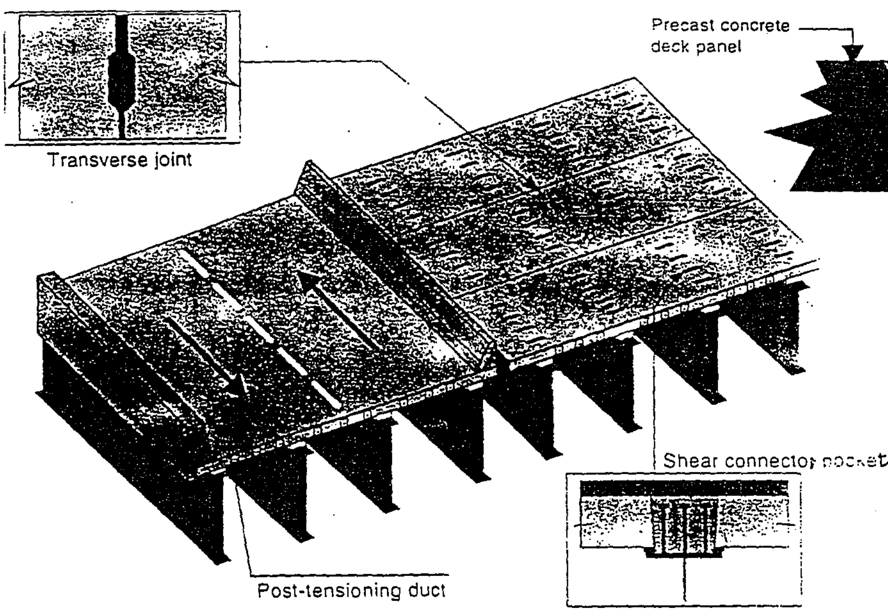


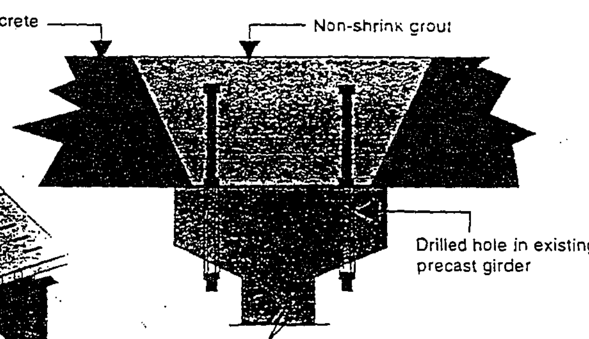
Fig. 3.1. Precast Concrete Deck Panels.



... and new decks co-exist with minimal interference at the Pennsylvania Turnpike's Clark's Summit Bridge re-decking project.



Layout of the bridge deck reconstruction system.



Connection between precast panels and precast girders.

Fig. 3.2. Precast Panel Layout, Placement, Connections.

Disadvantages:

- Added design and construction complexity of details required for horizontal shear transfer/connections to support girders
- Many cold joints in final deck surface which could leak and cause problems, or require a wearing surface
- May be impractical for skewed bridges

The University of Nebraska recently developed two precast/prestressed concrete deck replacement systems as part of a 3 ½ year NCHRP project. One is a continuous precast prestressed SIP system, and the other is a full depth precast prestressed system. Each of the new systems is described below.

NU Continuous Precast Prestressed SIP System. This system consists of a 4.5" thick precast continuous panel with a 4.5" thick CIP concrete topping. Figure 3.3 shows the cross section of this system, and Figure 3.4 shows the plan view. This system is continuous in both the longitudinal and transverse directions, which eliminates reflective cracks. The portion over the girder line is kept open to accommodate shear studs. This system utilizes transverse joints with reinforced pockets for longitudinal reinforcement. Figure 3.5 shows the transverse joints and Figure 3.6 shows the details of the reinforced pocket. The leveling device used to level the panel is detailed in Figure 3.7. The construction time for this system was 20% faster than a conventional SIP system and 60% faster than a conventional CIP system. Due to cost estimates, the system is comparable to the CIP deck system. The CIP topping slab utilizes epoxy coated welded wire fabric for reinforcement. A summary of the construction steps for this system is as follows (68).

- clean the girder surface by grit blasting
- glue the grout barrier to the edges of the top surface of the girders
- install the precast panels, insert the leveling device, and adjust the panels with the leveling devices
- install a backer rod between adjacent panels to prevent leakage during the casting of the CIP topping slab
- fill the 8" gaps over the girders with a flowable mortar mix or a rapid set, non-shrink grout
- install the #4 bar splices in the pockets and adjust the spiral bars into position
- install the welded wire fabric reinforcement for the CIP topping slab
- place the CIP topping concrete and cure it - the shear keys and pockets will be cured at the same time when the CIP topping concrete is cast

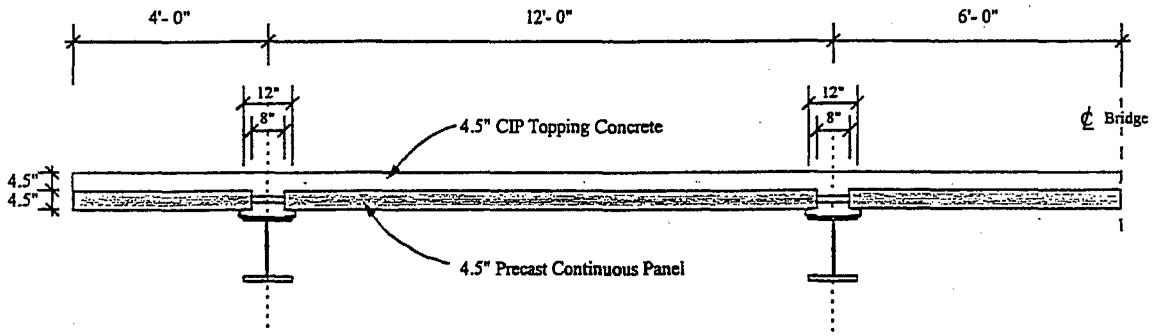


Figure 3.3. Cross-Section of the Continuous SIP System.

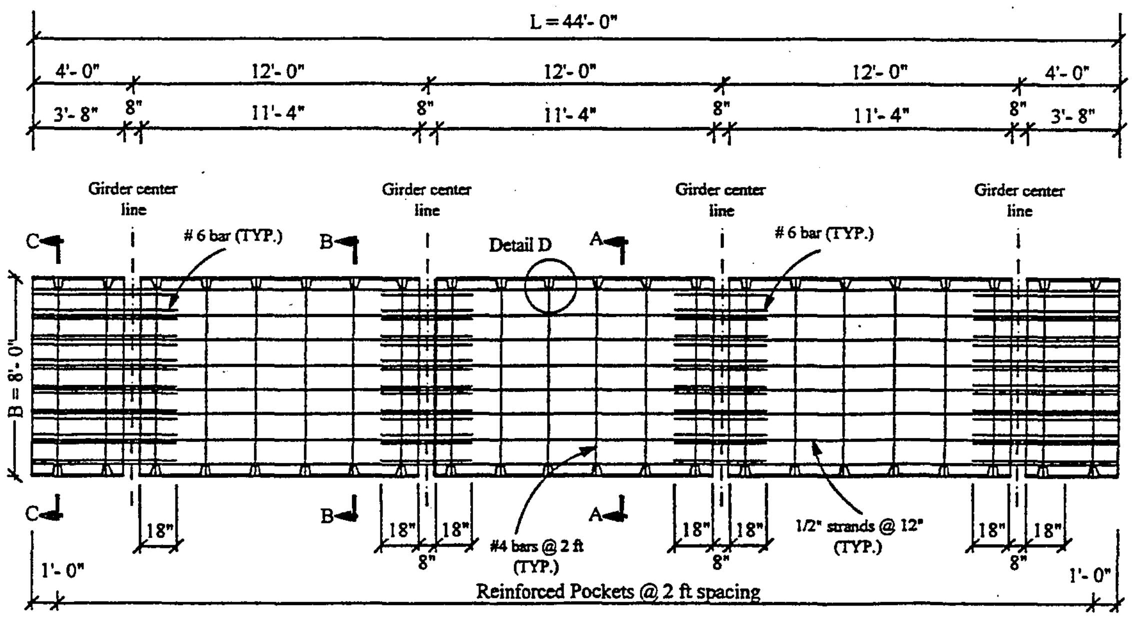


Figure 3.4. Plan View of the Continuous SIP System.

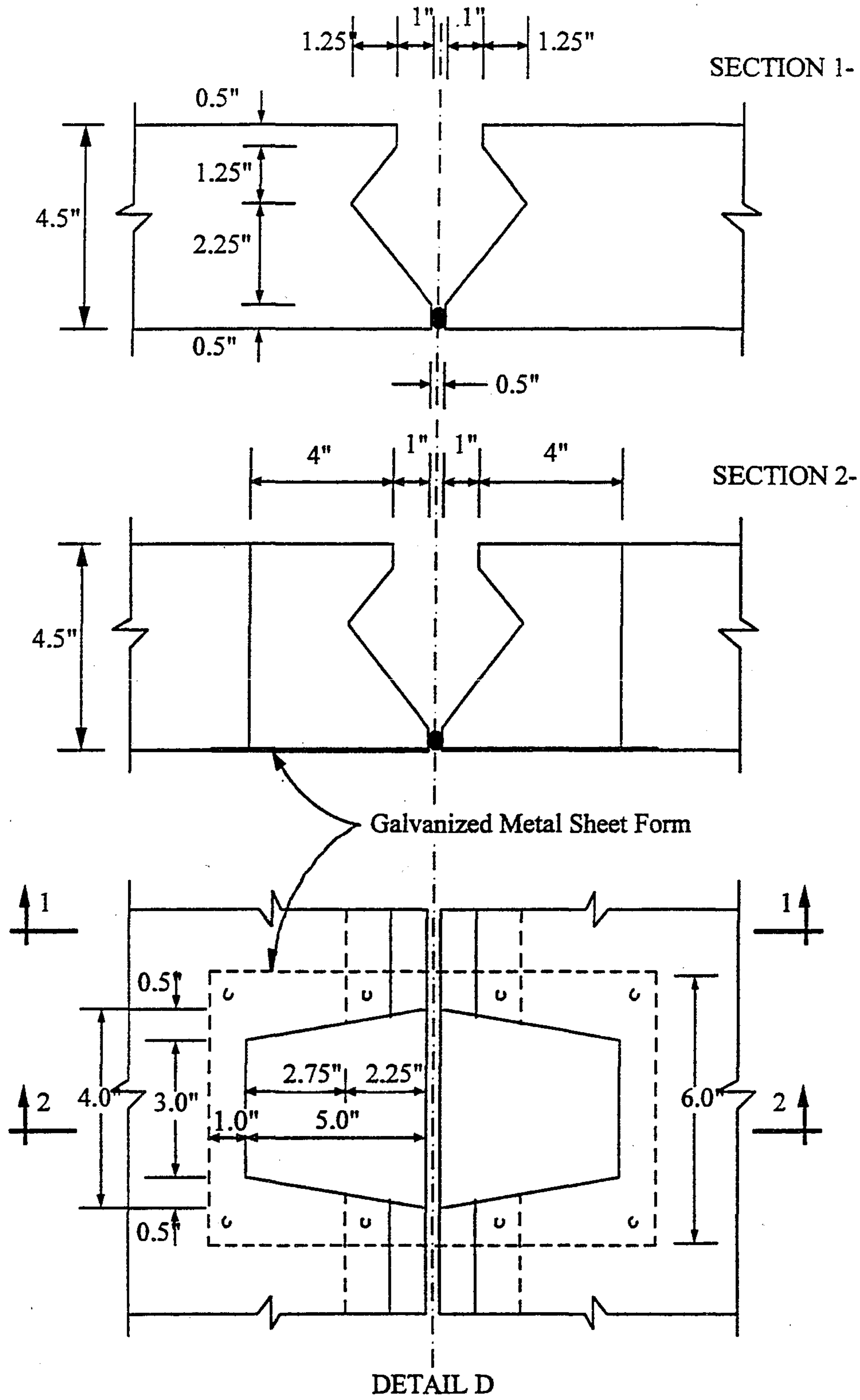


Figure 3.5. Transverse Joint Details.

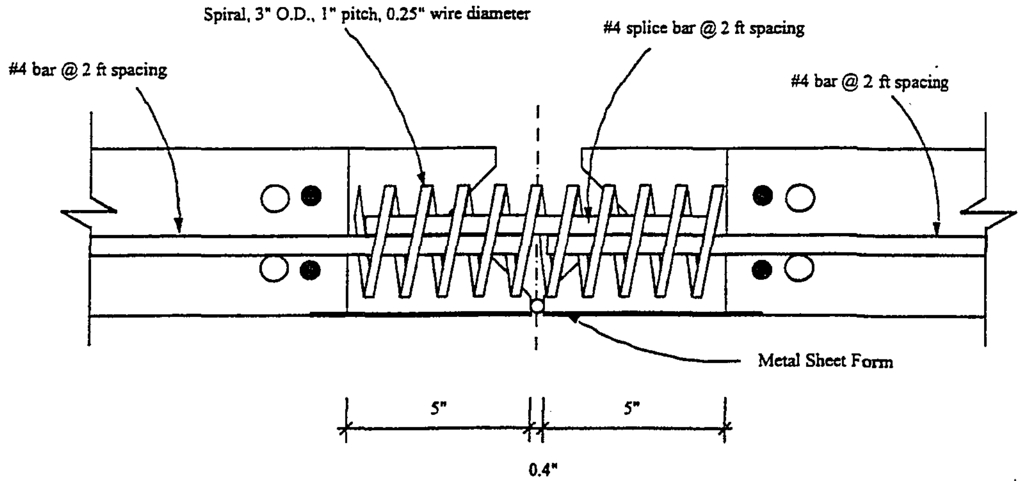


Figure 3.6. Reinforced Pocket Detail.

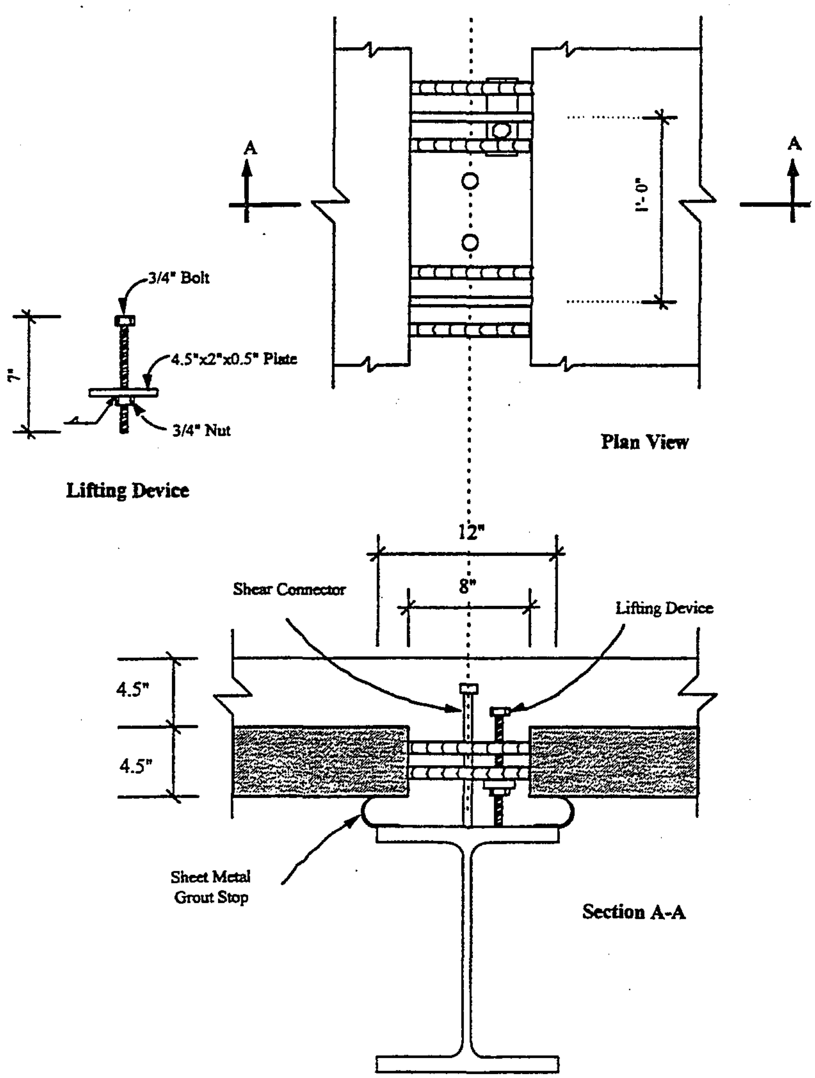


Figure 3.7. Leveling Device.

The system was conceived when the research team recognized several disadvantages of conventional systems utilizing precast subpanels and CIP toppings (i.e. the need for forming overhangs, handling a large number of precast panels, formation of reflective cracks over transverse joints between SIP panels, and development problems with prestressing reinforcement). There are several advantages as compared to conventional SIP systems (68).

Advantages

- relatively inexpensive materials in the panels
- using this system with welded wire fabric reduces the construction time by 55% compared with the full depth system reinforced with conventional reinforcement
- no field forming for the deck overhangs was needed and fewer precast panels would have to be handled
- SIP panel could be crowned to form for the cross slope of the bridge
- SIP panels were continuous in the longitudinal direction resulting in a better load distribution and eliminating reflective cracks at the transverse joints
- under cyclic loading, the system exhibited only minor top cracks over the girder
- due to creep of the prestressed concrete and continuity of the SIP panels over the girder lines, the CIP topping gains some compression stress which helped to reduce the effect of the service tension stresses at the negative moment zones. It helps also to close the cracks over the girder lines after removing the service loads.
- the strands were fully developed at maximum positive moment sections - this prevents the system from having sudden one-way shear failure which was reported in the conventional SIP panel system
- the system exhibited almost double the capacity of the conventional SIP concrete panel system

NU Full Depth Precast Prestressed Concrete System. An overview and typical cross-section of the new NU full depth precast prestressed system, as well as the typical steel arrangement, are shown in Figures 3.8 - 3.10. This system is composed of transversely pretensioned and longitudinally post-tensioned concrete panels which may extend the full width of the bridge, welded headless shear studs, welded threaded shear studs, grout filled shear keys, leveling bolts, and threaded bars for post-tensioning. The details of the system are shown in Figures 3.11 - 3.13. Joint details were a key consideration to avoid water leakage through the joints. A rapid set non-shrink grout was found to fill shear keys and shear connector pockets, that correspond to the properties needed to reduce construction duration, transfer live loads, and prevent water leakage.

Also, details to avoid a free edge loading should be developed for temporary opening of the deck to traffic. The details must either create continuity at the joint between new and existing decks or keep the loading location away from the edge (79).

If the bridge needs to opened to traffic for 24 hours a day, some lanes of the bridge must be used for traffic while the rest of the lanes are under construction. In this situation (transverse segmental construction), the bridge decks would have temporary and permanent longitudinal joints, and details for such joints should be developed for this type of construction (79).

As already mentioned, longitudinal post-tensioning was utilized in the NU system. The details of the longitudinal post-tensioning system are in shown Figure 3.14. This system was found to be adequate to close the transverse cracks at the joints between the panels. The shear connectors for the system consist of welded headless studs and welded threaded studs with nuts. Figure 3.13 shows the shear connector details. The system developed was clamped to steel girders by way of the nuts threaded on the studs. If concrete girders were used, a design utilizing either inserts or drilled and grouted anchors for shear connectors would apply.

In the past, using fulldepth precast deck panels usually made it necessary for some type of overlay to be used, after placement of panels, to provide for a smooth riding surface and good geometric profile. However, in a telephone conversation with the project PI, University of Nebraska Professor M. K. Tadros indicated that the Nebraska DOT and a Nebraska bridge contractor were planning a deck replacement where full depth prestressed, precast panels are cast ½" too thick to allow for later grinding to provide a smooth riding surface and a good geometric profile. This, of course, would preclude the need for an overlay on the new panels.

The general procedure for the deck construction would start with removing the old deck and installing the new shear studs. Then, the new deck panels could be erected and adjusted to grade by the leveling bolts, and tied down by the threaded studs. Then, the keyways between panels would be grouted, the grout would be allowed to cure, and the panels would be longitudinally post-tensioned. Finally, the pockets over the tops of the girders would be grouted to achieve composite action. It should be noted that the system was the fastest of all deck replacements studied by the University of Nebraska (68).

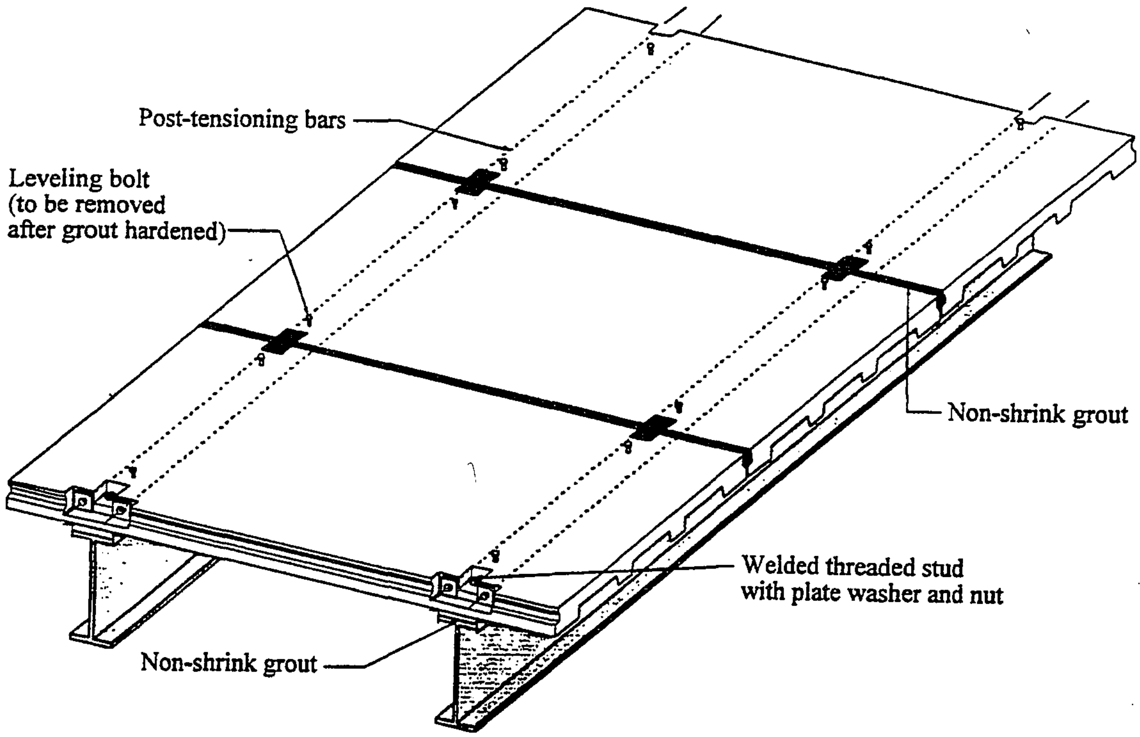


Figure 3.8. Overview of the Precast Prestressed Concrete Panel System.

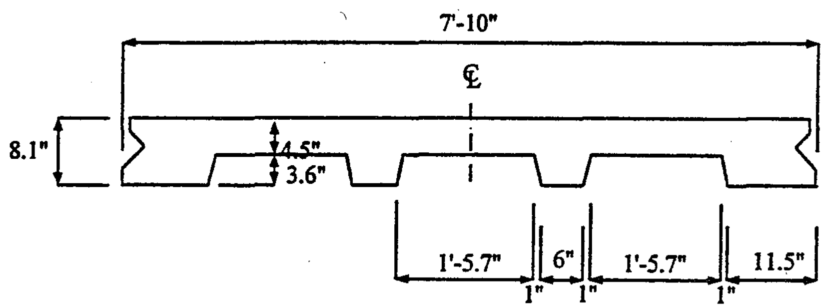


Figure 3.9. Typical Transverse Cross-Section of Precast Panel.

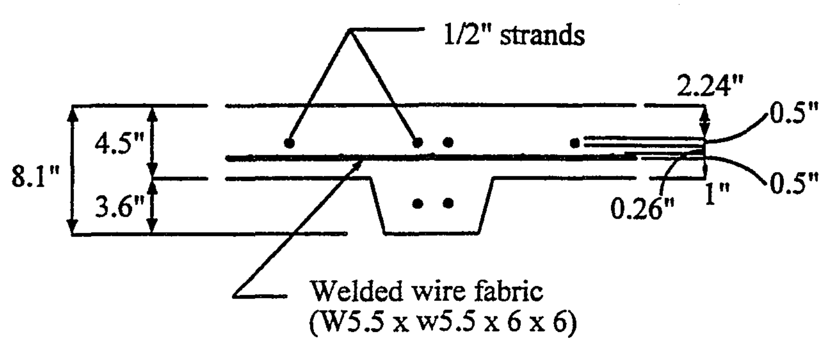


Figure 3.10. Typical Steel Arrangement.

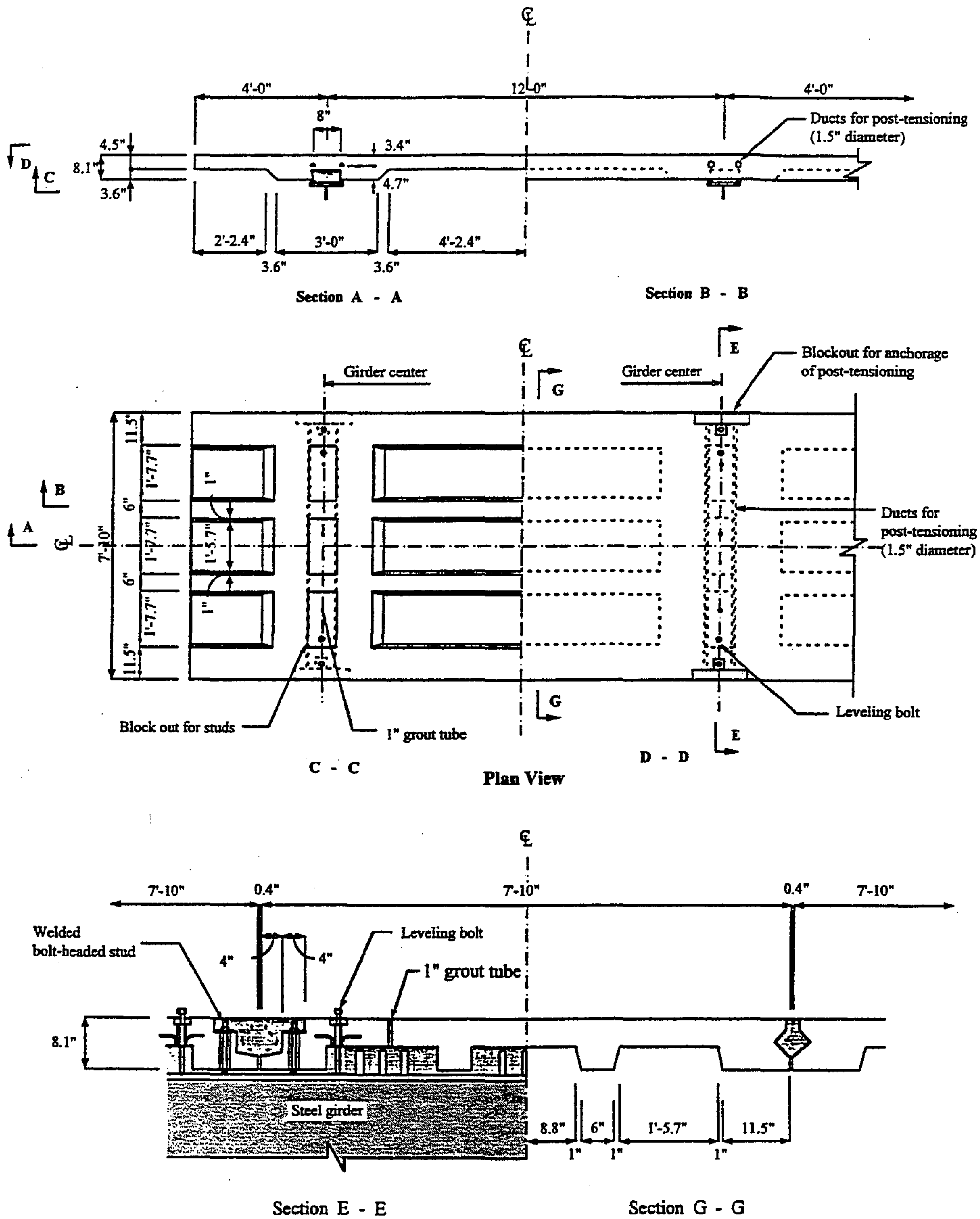


Figure 3.11. Details of Precast Prestressed Concrete Panel System.

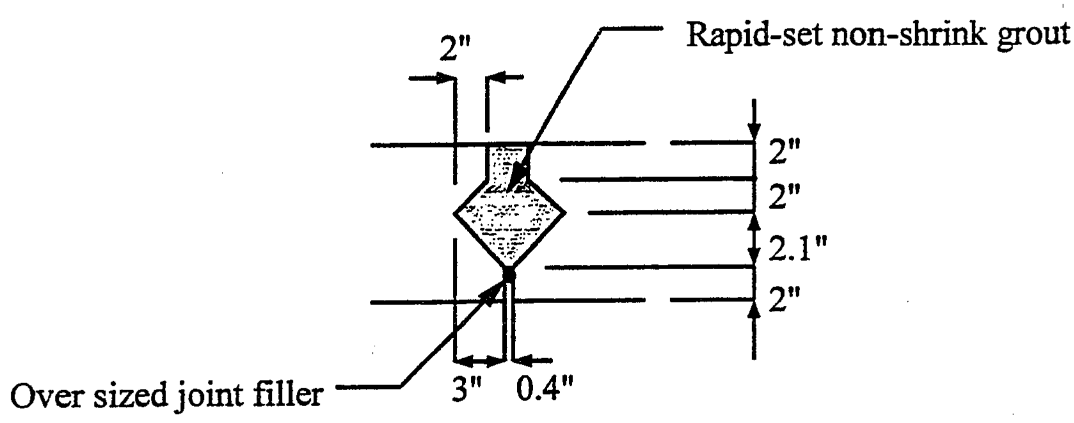


Figure 3.12. Transverse Shear Key Detail.

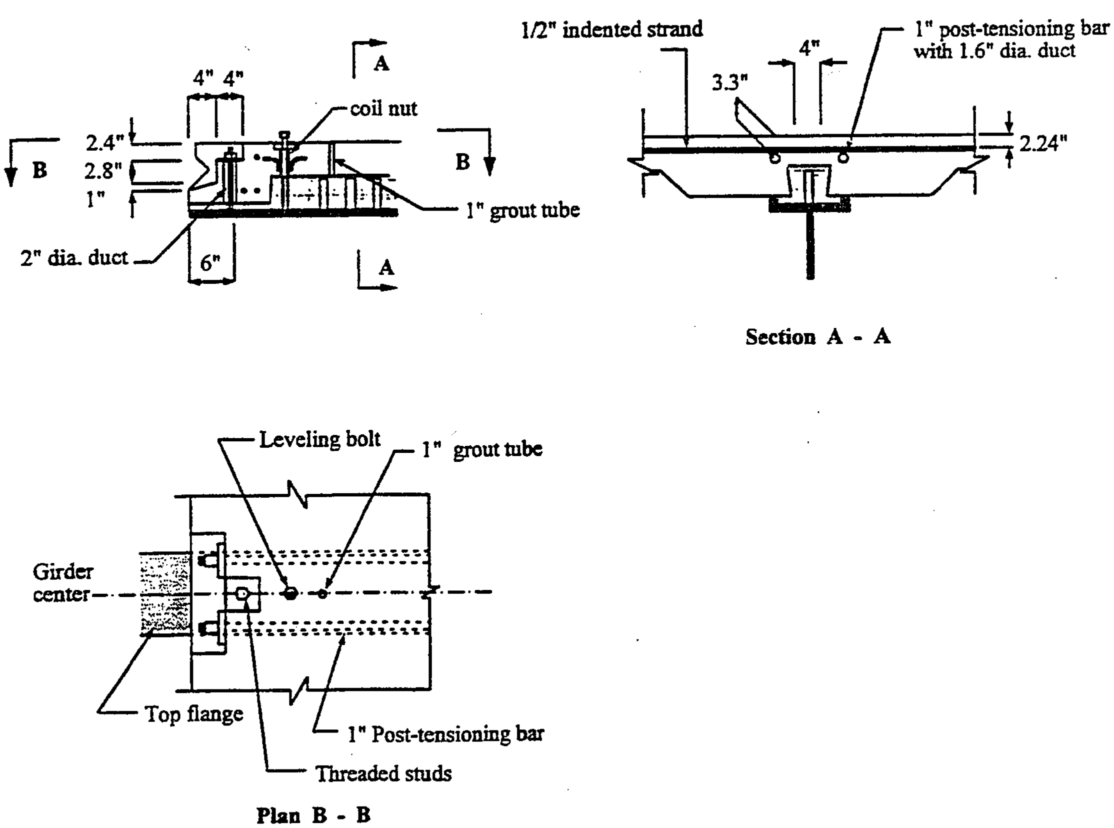


Figure 3.13. Shear Connection Detail.

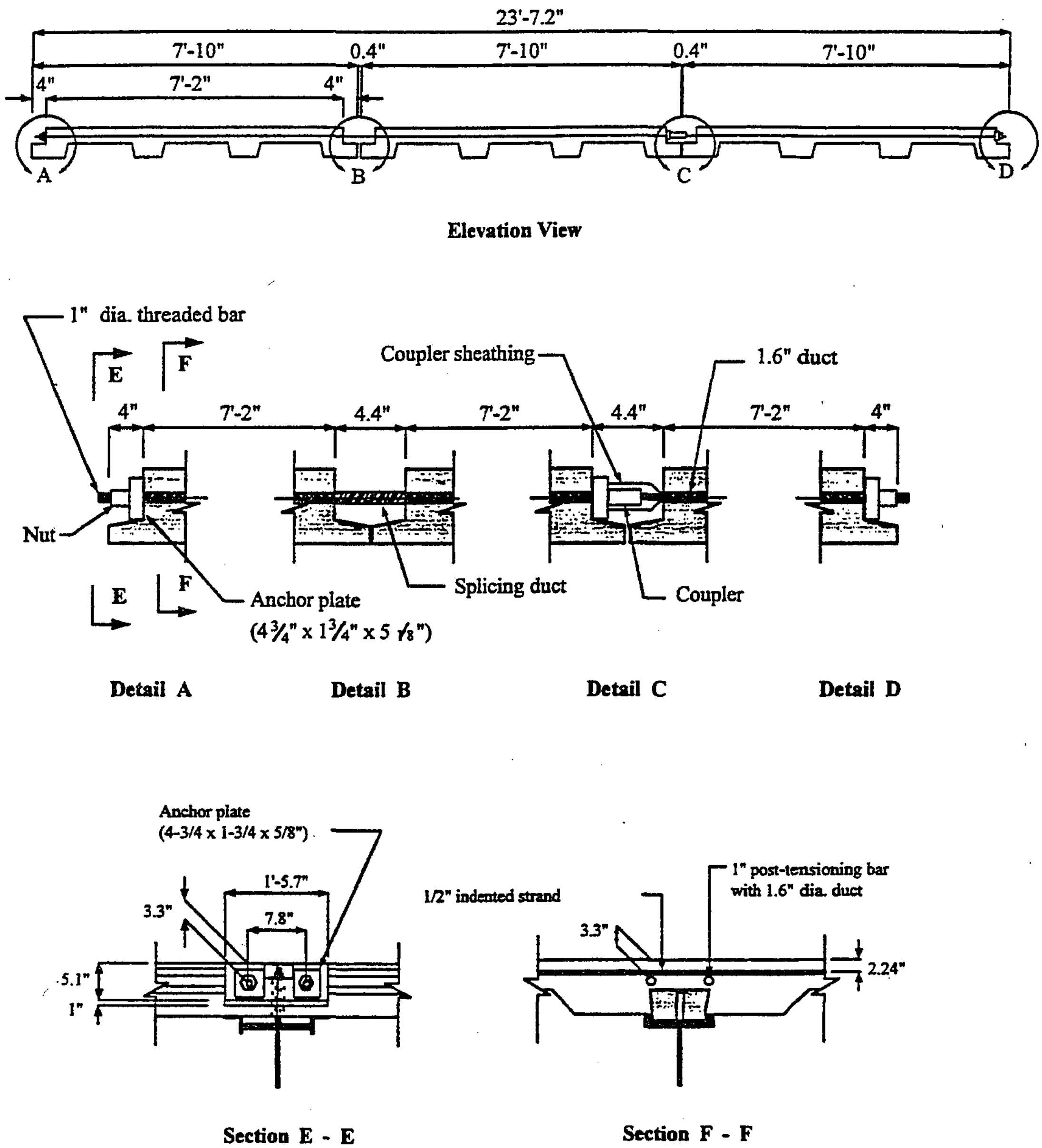


Figure 3.14. Longitudinal Post Tensioning Detail.

Advantages:

- two-way prestressing results in controlled cracking in both the longitudinal and transverse directions
- compares favorably with other full depth systems
- 10% thinner and 20% lighter by weight than conventional CIP or precast reinforced solid concrete sections
- very fast relative construction time

Disadvantages:

- procedure has never been used in practice - contractor unfamiliarity
- may require a bonded overlay or grinding of top to achieve a smooth riding surface

In the early stages of their research, the University of Nebraska researchers performed a comparison of three different prefabricated deck systems: Conventional precast reinforced concrete, Exodermic, and the University of Nebraska full depth precast, prestressed system. The parameters of the comparisons were deck thickness, equivalent solid thickness, weight of deck, and dead load saved. The results of the comparison are shown in Table 3.1.

Open Steel Grids. This grid system is popular when dead loads and wind resistance must be reduced to an absolute minimum, as in movable structures. Normally open grids are not used in redecking applications unless the existing deck was also open grid.

Advantages:

- Relatively low deck dead load
- Low wind resistance
- Free-draining surface
- Provides an immediate riding surface as soon as modules are in place
- Provides an easy connection to steel support girders for composite action

Disadvantages:

- Poor skid resistance (see Fig. 3.15)
- Poor riding quality—promotes fish-tailing, tracking, and is noisy
- Provides no weather protection for underneath supporting elements
- Susceptible to corrosion and failures at the grid intersections

Filled Steel Grids. The filled grids are generally placed without concrete fill, attached to the supporting elements, and then filled, either half-depth or full-depth, with concrete in place. The grid functions as a reinforcement cage, preassembled and ready for placement of concrete. It also offers an immediate riding surface, in the event the roadway must be opened to traffic before concrete can be placed. Concrete fill is then either placed to the top of the grid or overfilled above the grid to form an integral wearing surface. When concrete is not overfilled, a separate wearing surface, either dense concrete or latex-modified concrete, can be placed after the original concrete fill cures. The integral or separate wearing surface is necessary to avoid cupping of the concrete within the grid squares, along with the loss of skid resistance and rideability associated with riding directly on a steel grid surface.

Advantages:

- Allows prefabrication of deck panels to speed up the deck replacement
- Provides easy connection to support girders for composite action
- Provides an almost immediate riding surface
- Fairly well known construction methods
- Provides a good riding surface

Disadvantages:

- Requires several construction stages and thus replacement time is somewhat slowed down
- Has potential for delamination of concrete wearing surface at the top of the grid

Steel Orthotropic System. An orthotropic deck is a steel plate with stiffeners attached underneath it. The plate is stiffened in two directions: longitudinally and transversely. Longitudinal to the bridge, open or closed rib systems are used (see Fig. 3.16 to stiffen the deck plate. Floor beams are used to provide stiffness in the transverse direction. Since the stiffness of the ribs vary from the floor beam, the system is said to be anisotropic. The term orthotropic is derived from the orthogonal (*ortho*) placement of the stiffeners and the anisotropic (*tropic*) behavior.

An orthotropic deck acts as the top flange of the primary members and, compared to a concrete deck, adds little dead load to the superstructure. Because of this reduced weight, a bridge equipped with an orthotropic deck can carry large loads. This becomes a major advantage in large span bridges, since dead load represents a major part of the moment due to bending. Since this is not the case with shorter span structures of less than 200 ft, they rarely are equipped with such decks since the benefit of reduced dead load is not offset by the increased fabrication costs.

Table 3.1 Comparison of Prefabricated Deck Systems

Type	Precast Panels with CIP Concrete	Exodermic	UN Full-depth Prestressed Panel
Thickness of deck, in (mm)	9.0 (230)	9.6 (245)	8.1 (205)
Equivalent solid thickness, in (mm)	9.0 (230)	5.9 (150)	5.7 (145)
Weight of deck psf (kPa)	110 (5.3)	75 (3.6)	70 (3.4)
Dead load saved, psf (kPa)	--	35 (1.7)	40 (1.9)

Figure 3.15. Scabber Roughened Grid Deck (for Better Skid Resistance or Concrete Bonding).

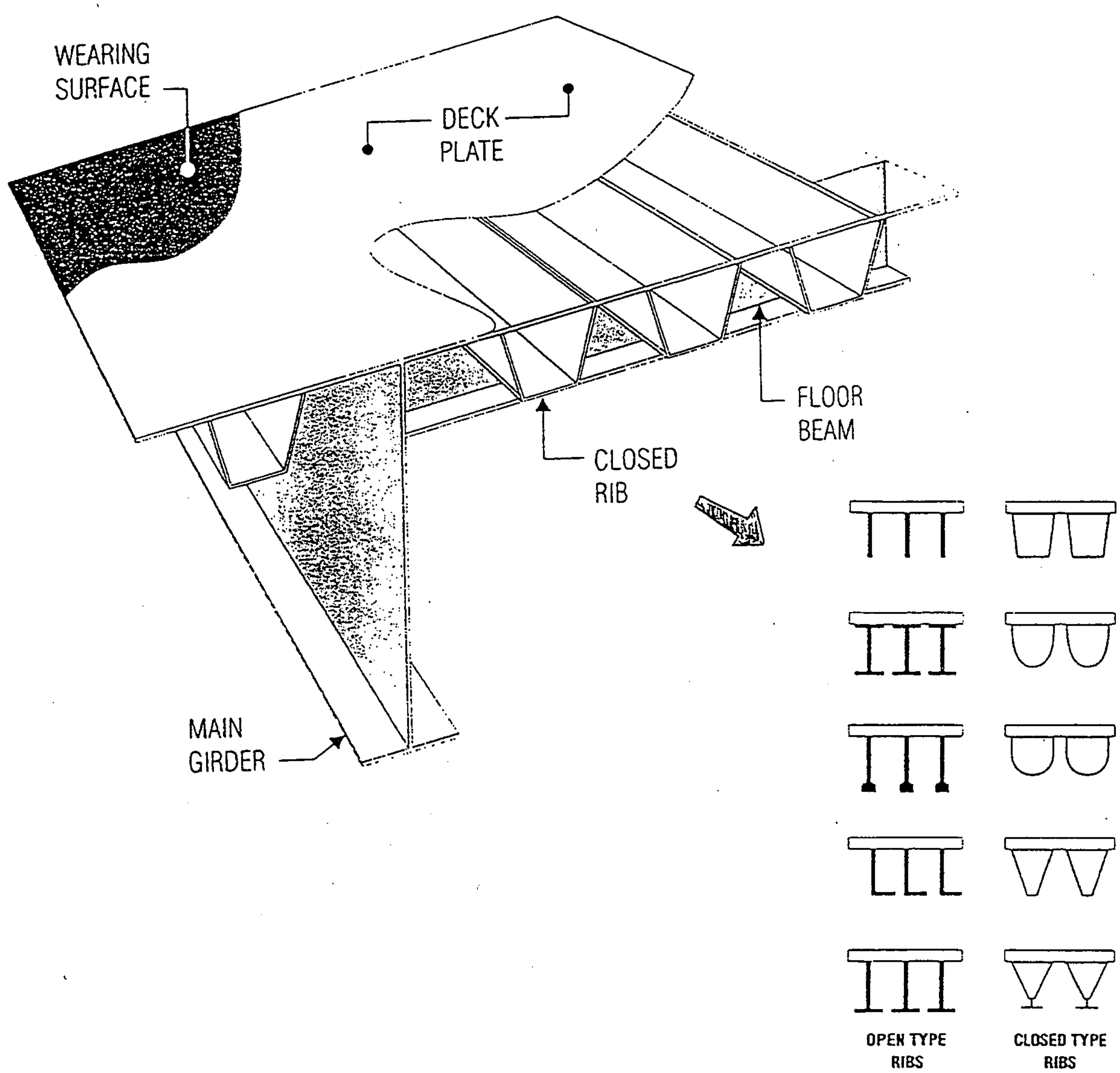


Figure 3.16. Isometric Illustration of Steel Orthotropic Deck.

The main components of steel orthotropic decks have exhibited excellent longevity, but problems can arise with surfacing materials and connections. Historically, problems that appear shortly after installation have been associated with the surfacing materials; later in the service life of the deck, problems associated with welded connections can develop.

- *Shoving and rutting* are basic surfacing problems that have arisen, and can be easily identified during a visual inspection or through discussions with maintenance personnel. Shoving and rutting imply a breakdown in the bond between the surfacing and the steel plate that causes the surfacing to migrate across the deck, forming high and low points. The ridges formed create difficulties for drainage (ponding) and the resulting irregular surface increases the impacts of vehicular loads. Eventually, these problems result in a rough riding surface and chronic maintenance problems. They can be eliminated by resurfacing with close attention to surfacing materials and placement procedures.
- *Connections* are the other major source of problems in orthotropic decks. Welds have cracked and bolts have loosened, since, as in the case of the grid decks, composite action is developed for live load between the deck and its supporting elements.

A properly designed orthotropic deck can provide an excellent riding surface, with square foot weights approaching those of the open grid systems. An advantage of an orthotropic scheme is that the modular construction can be accomplished at night to minimize traffic impacts. When this is done the section is paved after all erection is complete.

Riding surfaces have caused some problems with orthotropic systems. More recent installations have used epoxy asphalts, rather than the earlier bituminous asphalts. The epoxy asphalts have generally performed better than the bituminous, which were frequently subject to bond failures in the form of shoving and rutting of the pavement surface. It appears that epoxy asphalts properly mixed and placed, combined with adequate deck plate thicknesses, can provide long-term, comfortable, riding surfaces.

Advantages:

- Low weight
- Allows prefabrication of deck panels to speed-up the deck replacement

Disadvantages:

- High cost (generally several times the cost of other alternatives)
- Requires the addition of a deck wearing surface
- Susceptible to wearing surface bonding problems leading to "shoving" and "rutting" which in turn lead to high maintenance and early replacement activities
- Susceptible to connection problems within the prefabricated panels and at the panel connections.
- Added design and construction complexity
- Matching of existing deck elevation in staged replacement is difficult

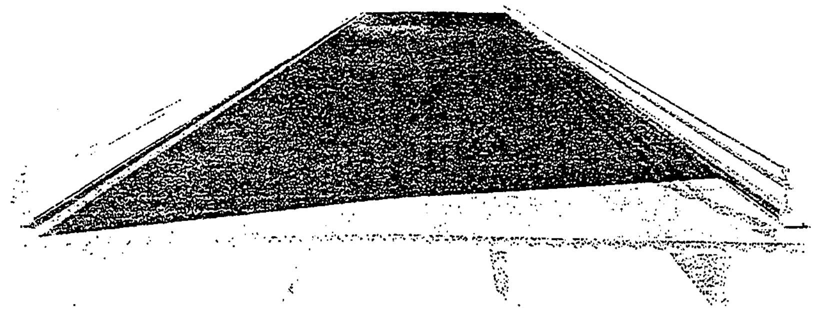
Aluminum Isotropic Deck System. Reynolds Metals Company has developed a nearly isotropic aluminum deck system for use on highway bridges. The new deck is nearly isotropic rather than orthotropic, comprised entirely of extrusions welded together at the sides, providing continuity in the top and bottom flanges. After welding, the webs of the extruded parts form repeating triangles, creating trusses perpendicular to the extruded direction. The extrusions generally run longitudinally (parallel to the girders and to traffic). The completed proprietary deck is connected securely to the underlying girders so that composite action is developed between the deck and the girders. The decks can be fabricated in panels as large as can be shipped, which for highway shipments is about 14' wide by 120' long, and are shop-coated with a ⅜" nominal thickness epoxy-based wearing surface.

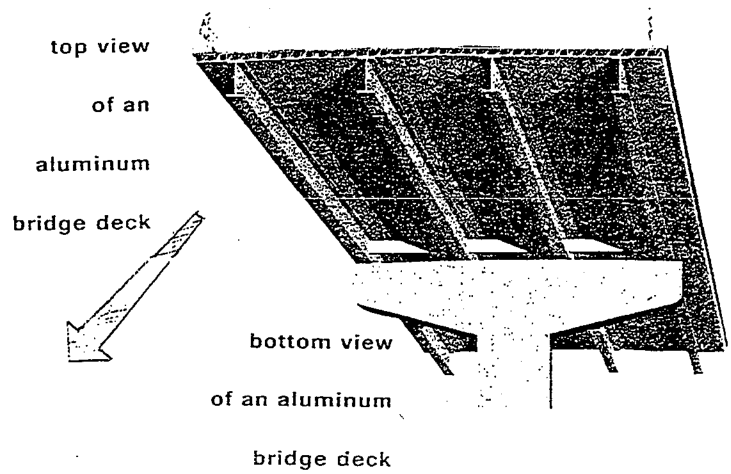
Isometric sketches and cross-sectional views of the Reynolds decks are shown in Fig. 3.17 Details and information about these decks are given below.

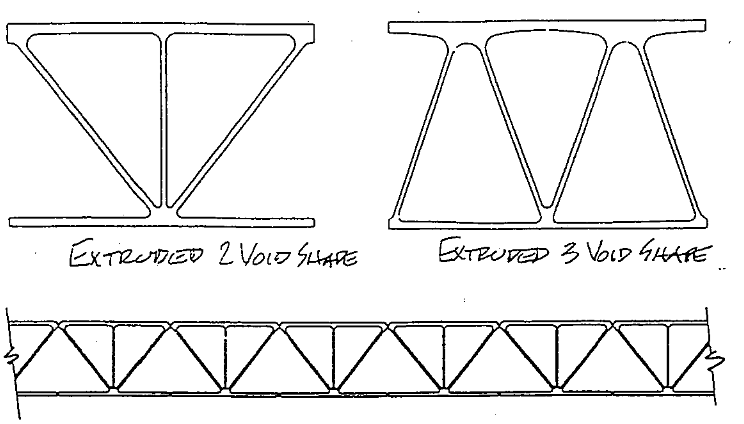
- Deck panels are approximately isotropic
- Deck panels are 8" in depth and weight 22 psf (rather than 100 psf of 8" concrete decks)
- Deck panels are joined along their longitudinal edges at the site with bolted splices that provide full continuity to the deck
- Decks can be prefabricated in panels as large as can be shipped (approximately 14' wide by 120' long for highway shipment)
- Decks are shop-coated with a ⅜" nominal thickness epoxy-based wearing surface
- Decks can be bolted or grouted to supporting steel girders to achieve composite action (see Fig. 3.20)
- The 2 void shape extrusion was developed and tested first. The 3 void shape extrusion is the newer design and is stiffer, yields lower bending stresses, and is the recommended shape for heavy truck loadings (see Fig. 3.17)
- Deck panels have very low maintenance costs (no painting, rusting, or crack repair)

Constructability - The prefabricated panels, which are relatively lightweight and have the wearing surface in-place, are excellent candidates for rapid deck replacement under concurrent traffic conditions. The deck panels are relatively easy to connect to the girders to develop composite action (see Fig. 3.18).

Cost - The initial cost of aluminum deck panels is higher than other decking alternatives. However, their low maintenance cost and projected long service life renders their projected life-cycle-cost as being very competitive. It should be noted that lower on-site construction cost and the speed in which the deck can be reopened to traffic are very important "cost savers".







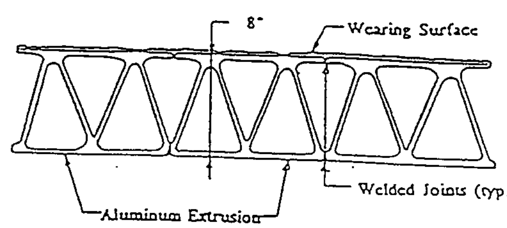
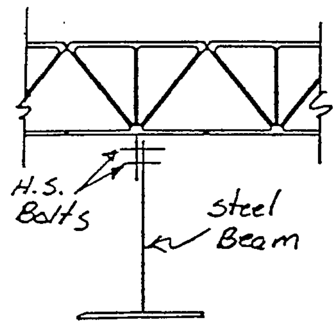
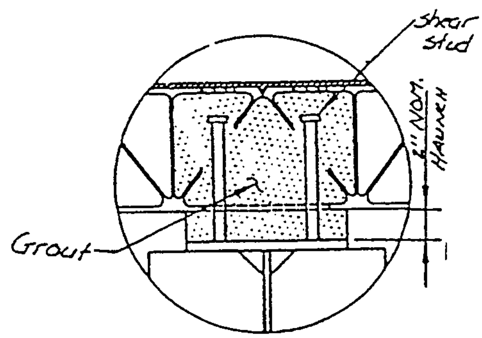


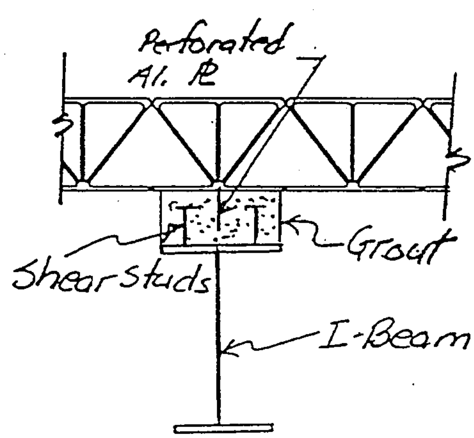
Figure 317. Reynolds Aluminum Isotropic Bridge Decks.



Bolted/Shop Connection



Grouted/Field Connection



Grouted Field Connection

Figure 3.18. Aluminum Isotropic Deck-To-Girder Connections.

Comments - The excellent ductility and fatigue strength of aluminum deck panels are such that stiffening of the Birmingham steel girder superstructure should not be necessary.

Advantages:

- Extremely light weight (approximately 22 psf vs. 90 - 100 psf for concrete decks)
- Exhibit approximately isotropic behavior
- Deck panels are joined along longitudinal edges at site to provide continuity
- Deck panels can be fabricated in panels as large as can be shipped and handled in the field to expedite rapid deck replacement
- Decks come shop-coated with an epoxy based wearing surface
- Deck panels can be grouted by existing support girders to achieve composite behavior
- Low maintenance cost (no rusting/painting or concrete cracks to repair)
- Excellent for rapid deck replacement under concurrent traffic conditions

Disadvantages

- High initial cost
- Specialized construction techniques/capabilities are needed for emplacement
- Panel end splices (transverse seams) to achieve continuity present problems

Exodermic Deck System. An Exodermic or "unfilled, composite steel grid" bridge deck is comprised of an unfilled steel grid 3" to 5" deep, with a 3" to 5" reinforced concrete slab on top of it. The upper portion of the main bearing bars extend up into the reinforced concrete slab, making the slab composite with the steel grid (see Fig. 3.19). The composite action is accomplished by the holes drilled into the upper portion of the main bearing bars. I should be noted that this is a new design (as of 1998). In the former design, tertiary bars were used with welded vertical shear studs (see Fig. 3.20). The revised design is superior to the former in that it eliminates the tertiary bars, making the deck easier to fabricate and install. It is estimated that the new design will save the owner over $2 per square foot versus the original design. The concrete portion of an Exodermic deck can either be cast-in-place (the grid panels act as the form work), or precast, where rapid construction is critical.

Lightweight and modular, Exodermic bridge deck systems have been used since 1984, and are increasingly being used to re-deck existing structures with minimal interruption to traffic. Also, reducing dead load can help achieve higher load ratings.

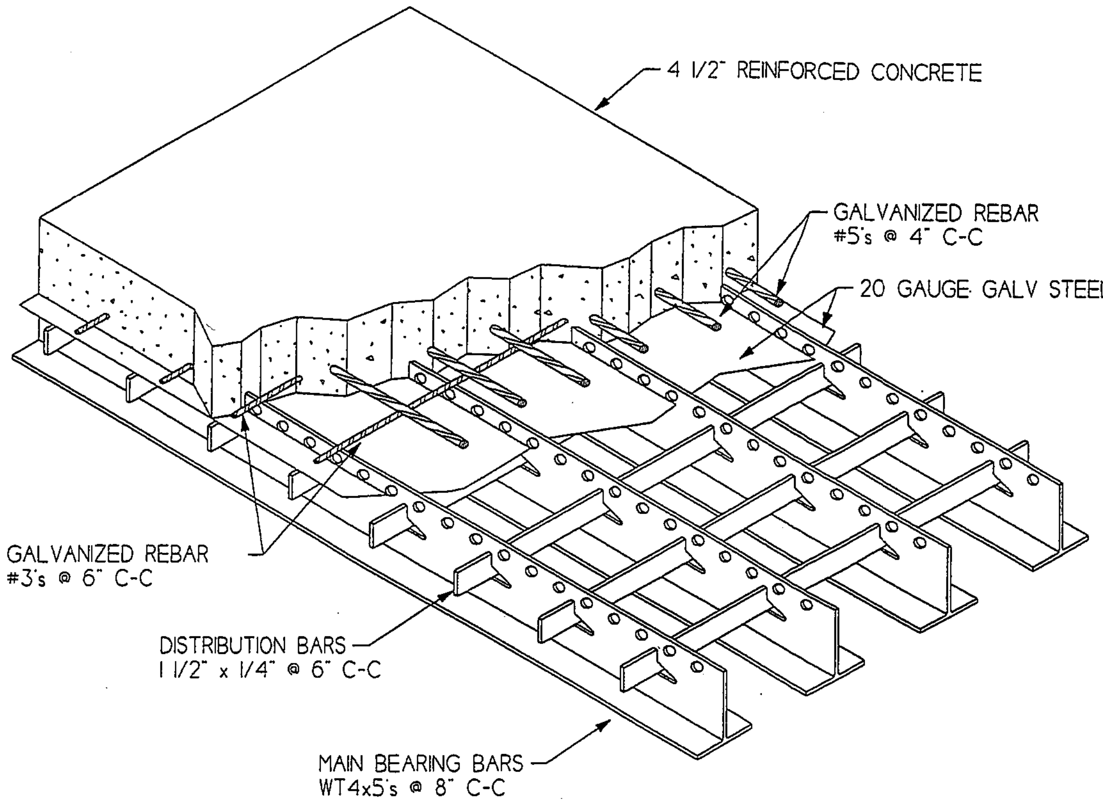


Figure 3.19. New (1998) Exodermic Deck Panel.

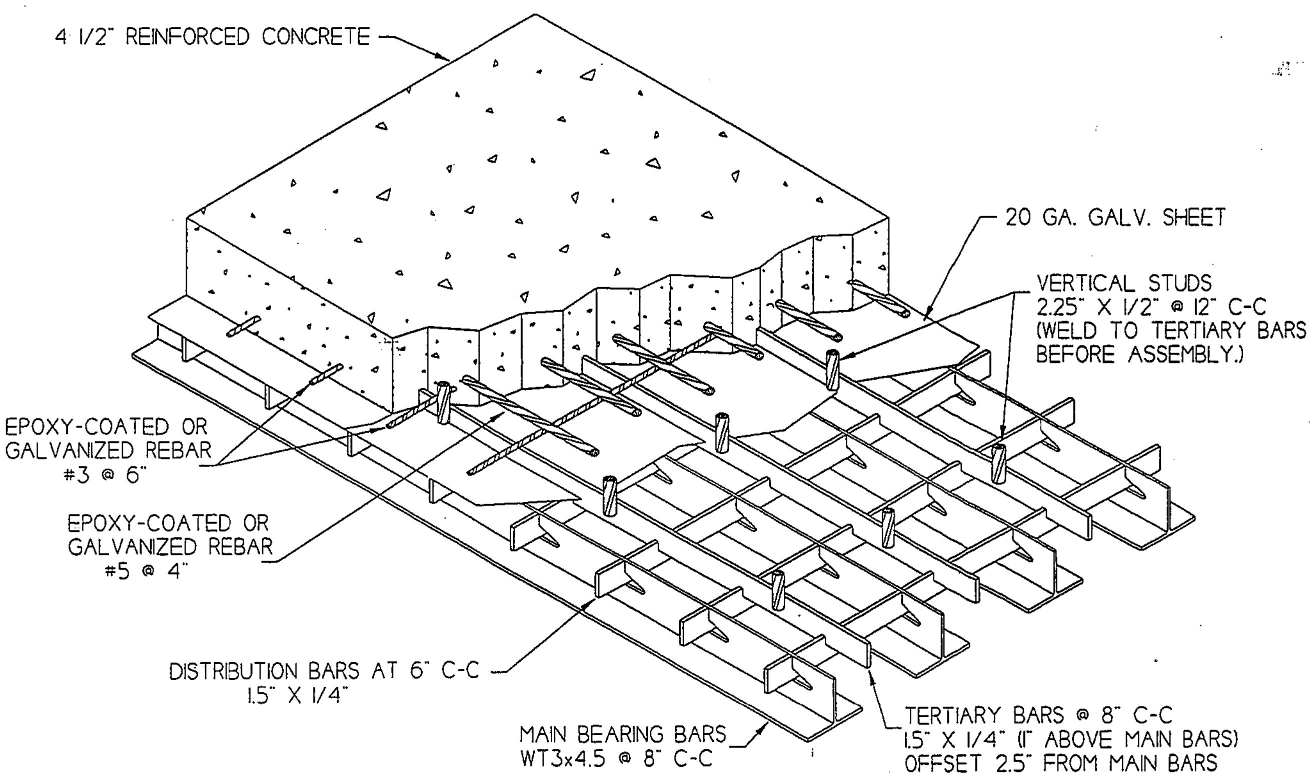


Figure 3.20. Original Exodermic Deck Panel

In a standard reinforced concrete deck, in positive bending, the concrete at the bottom of the deck is considered 'cracked' and provides no practical benefit. Thus, the effective depth of the slab is reduced, and the entire bridge -- superstructure and substructure -- has to carry the dead load of this 'cracked' concrete. In an Exodermic deck, essentially all of the concrete is in compression and contributes fully to the section. The main bearing bars of the grid handle the tensile forces at the bottom of deck. Because the materials (steel and concrete) in an Exodermic deck are used more efficiently than in a reinforced concrete slab, an Exodermic design can be substantially lighter without sacrificing stiffness and strength.

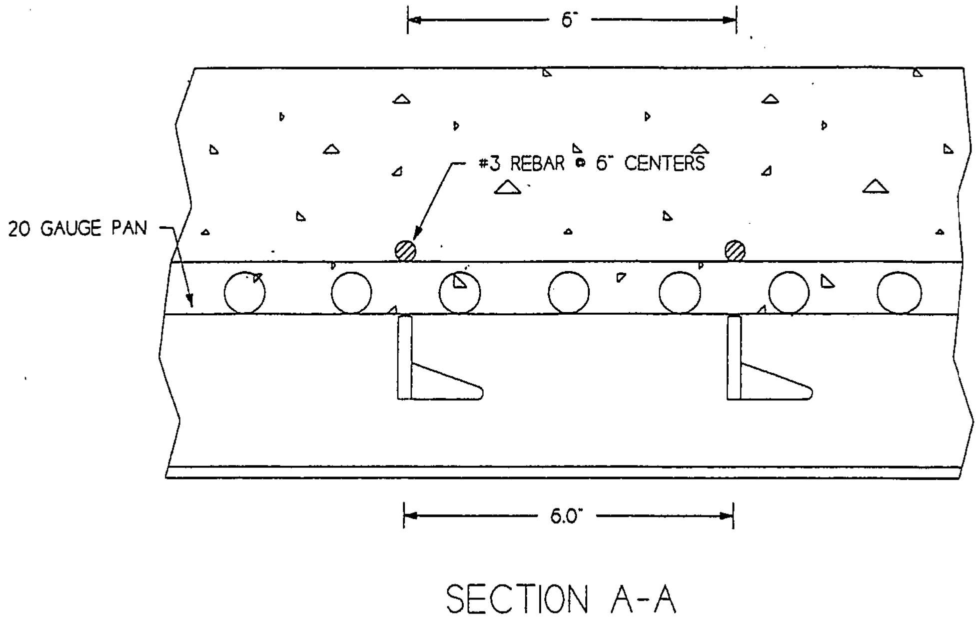
In negative bending, the reinforcement in the concrete handles the tensile forces, as it does for a conventional concrete deck. Rebar in the concrete component of an Exodermic deck can be specified to handle the negative moment, where the deck is continuous over supporting beams, or where a cantilever is required.

Exodermic decks can be specified to accommodate the particular requirements of a specific bridge. To date, overall deck thickness on different projects has ranged from 6.75" to 10". Figure 3.21 shows details of the 7.5" thick Tapan Zee Bridge Deck. The concrete component of an Exodermic deck can be precast, or cast-in-place. In the latter case, the steel grid can be thought of as a super stay-in-place form, where the strength of the steel grid panels permits longer spans, if necessary, and elimination of half the concrete.

During precasting, blockouts are used so that there is no concrete over the grid in the areas that will be over floor beams and stringers (see Fig. 3.22). During erection, headed shear studs are welded through these openings in the deck onto the stringers or floor beams below. These portions of the deck are filled full-depth with rapid-setting concrete which captures the shear connectors and the grid and rebar of the deck modules, locking them together and making them composite. No field welding of the grid to supporting beams, or grid panels to each other is required.

Advantages:

- Significantly lighter weight than reinforced concrete slab
- Shear keys between panels permit rapid field erection
- Top mat of rebar permits splicing or lapping rebar with rebar in existing deck, and for providing continuity where adjacent panels are connected in negative moment regions
- Possible to block-out precast concrete from area that will be above beams in the bridge, permitting rapid field placement of rapid-setting concrete to achieve composite behavior



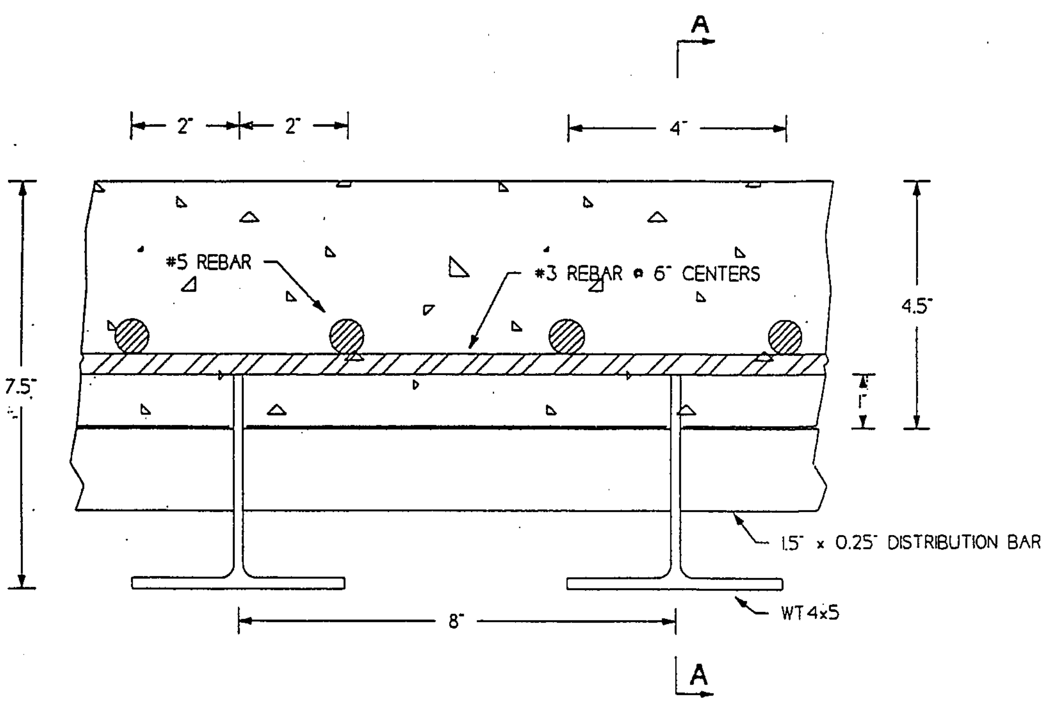
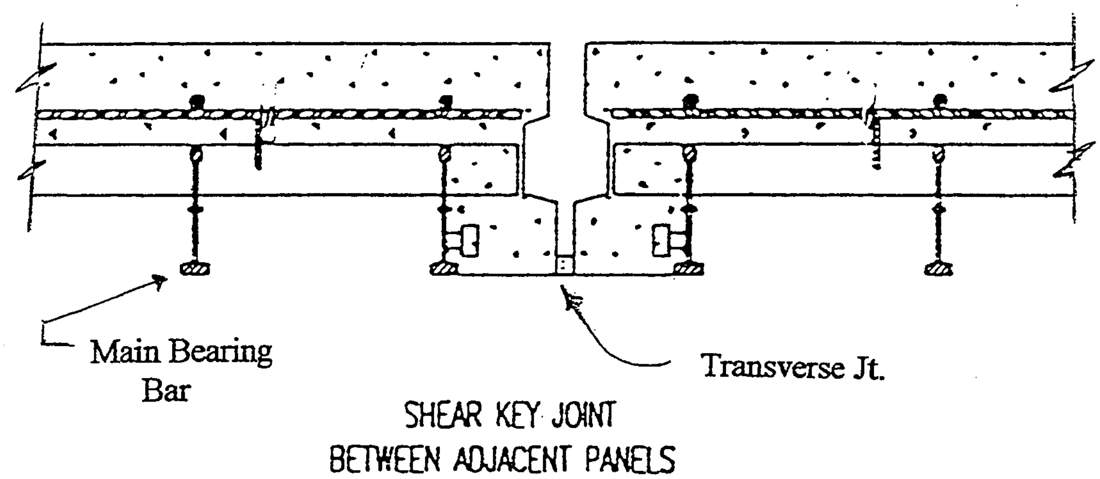


Fig. 3.21. Enlarged Detail of New Exodermic Panel.



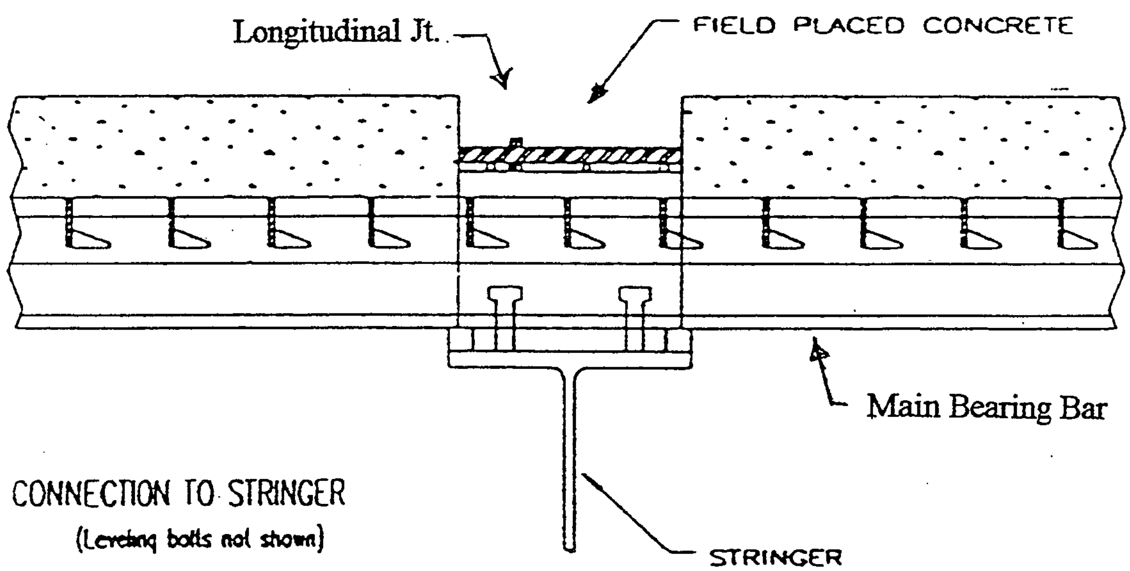


Fig. 3.22. Panel-to-Panel and Panel-to-Stringer connections for Exodermic Decks.

- Reinforced concrete surface provides excellent riding surface
- Simplified construction staging and reduced total construction time

Disadvantages:

- Contractor unfamiliarity
- System may be labeled as proprietary despite its availability from multiple, independent suppliers
- Higher cost than most alternatives except steel orthotropic or aluminum isotropic decks

Inverset Deck System. This system was used by the New York State DOT. It is a prefabricated system that can be easily connected at the site. A pair of fabricated beams are connected by diaphragms and have diaphragm connection plates for field installed diaphragms. The beams have shear stud connectors as required. See Figure 3.23 for a view of the prefabricated beam pair with shear studs and diaphragms. The system is prefabricated in the following manner: the beam pair is inverted, and forms are connected to the bottom. See Figure 3.24 for a view of the inverted beam pair and forms ready for concrete casting. Concrete is cast in the forms, and the combined weight of the forms and concrete produces a prestressing effect on the beams. While the concrete is curing, a deflection control is placed under the system at midspan. Finally, when the system is turned over, after curing, the compressive stresses in the bottom flange reverse to near zero, and compression stresses result in the concrete deck. This system allows for rapid replacement, but is not a well known procedure.

FRP Composite Deck System. One other deck replacement option that should be mentioned is the modular deck system made of Fiber Reinforced Polymer (FRP) composite materials. This is a modular deck system that is very lightweight compared to its concrete and steel counterparts. Decks of this kind have been under development and evaluation for only about three years (since 1995). Two such bridge decks exist in West Virginia, as a result of a partnership of academia, government, and industry. These are the Laurel Lick Bridge and the Wickwire Run Bridge, complete in May 1997 and September 1997, respectively. These were made with the pultruded components shown in Figure 3.25. A cross-section of the Wickwire Run Bridge and the Laurel Lick Bridge deck can be seen in Figures 3.26 and 3.27, respectively. There are some distinct advantages to the FRP bridge decks. They are reportedly very resistant to fatigue, easy to transport and install, very lightweight, and somewhat corrosion resistant. In fact, it is reported that, with improvement in design and construction methods, many deck panels may perhaps be placed in the operations of one night.

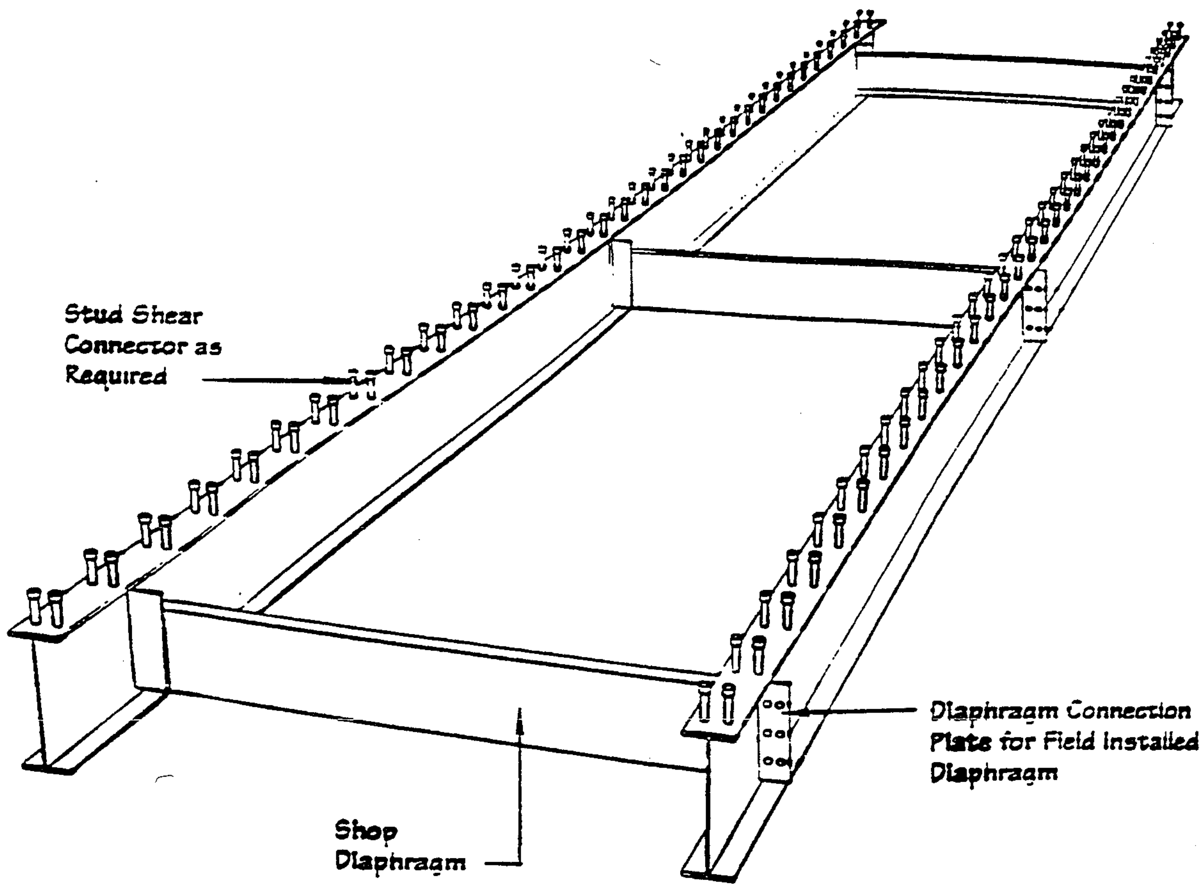


Figure 3.23. Prefabricated Beam Pair.

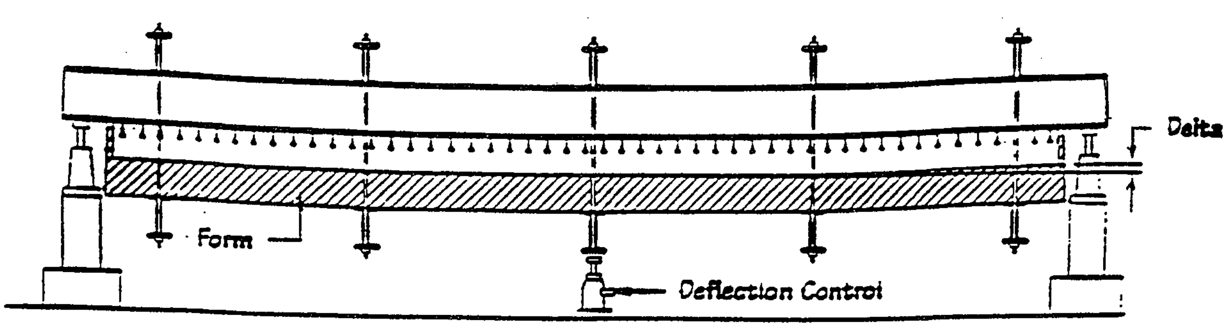


Figure 3.24. Inverted System Ready for Concrete Casting.

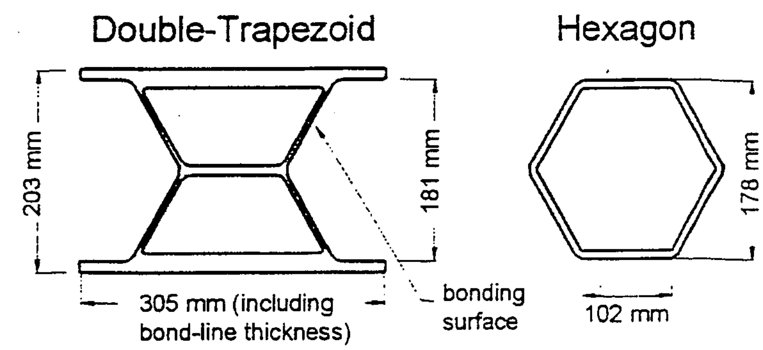


Figure 3.25. Pultruded Deck Components.

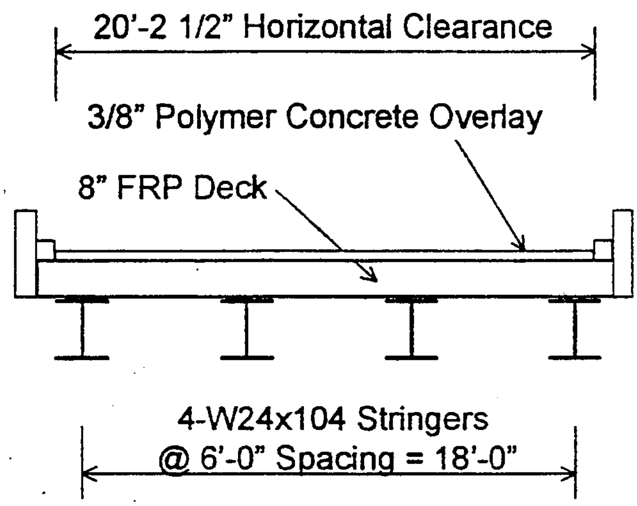


Figure 3.26. Wickwire Run Bridge Cross-Section.

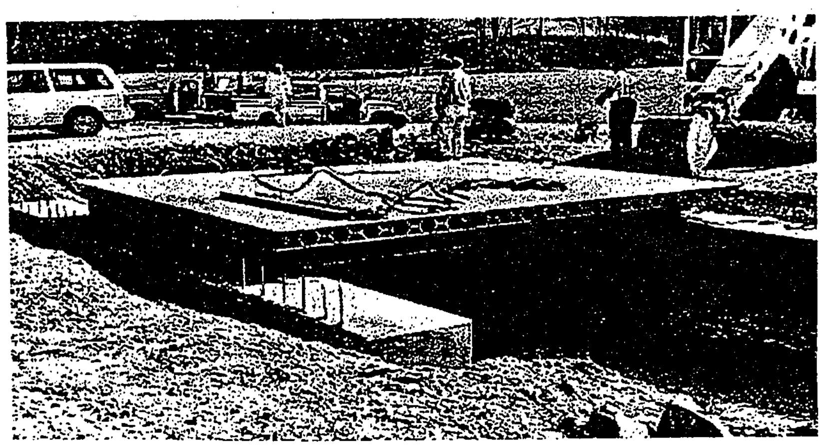


Figure 3.27. Construction of Laurel Lick Bridge in WV.

While these qualitites sound very attractive, it must be noted that this very new system has drawbacks that will make it impractical in its present (1998) form. One important drawback is that these bridge deck systems are not proven for use in high volume highway bridges. This means that there could be many unforseen problems such as inadequate strength of connections for large truck loading. Also, these systems are mainly considered proprietary at this time, which might make them expensive and impractical for use by government agencies. Another problem with using FRP composites for bridge deck is that they exhibit little ductile behavior and, thus, may not provide for load redistribution and visual displacement warnings if overload conditions ever occur (63). And, as always with a new system, contractor unfamiliarity is a definite problem.

3.3 Deck Rehabilitation Options

By far the most common bridge deck rehabilitation option used today is that of placing a deck overlay. The overlay may be placed on the existing deck as it is (after surface bonding enhancement actions are taken), or the top ½" - 1½" of the existing deck may be scarified/milled off in preparation for the overlay. Thus, the primary options for deck rehabilitation are simply the options on what kind of overlay to employ.

Current viable overlay options and other options for deck rehabilitations are as follows:

1. Protective Coating and Sealants

   This is not viewed as a viable option for the Birmingham bridges as the extensive deck cracks are live and active cracks which will recrack at the same or adjacent location if they are simply sealed.

   Due to their high flexibility, some type of structural or partial structural rehabilitation is needed on the Birmingham bridges.

2. Seal and Weld Cracks

   This rehabilitation option is not viable (by itself) for the Birmingham bridges for the same reasons indicated in number 1 above.

3. Overlays

   a. Low - slump dense concrete (LSDC) (≈ 1¾" thickness)
   b. Latex - modified concrete (LMC)
   c. Polymer concrete
   d. Silica fume concrete (SFC)
   e. Fiber reinforced concrete
   f. Water proofing membrane/bituminous wearing surface

4. Strengthen/Stiffen Existing Deck
    a. Seal and weld cracks (see # 2 above)
    b. Overlays (see #3 above)
    c. Use of FRP strips - possibly on top and bottom and in both directions
    d. Use ¼" steel or aluminum plate (with epoxy wearing surface already bonded) epoxied to deck top surface
    e. Use 1½" prestressed high performance concrete panels/sheets epoxied to deck top surface.
    f. Add transverse steel or FRP strips to girder top flanges to improve deck "arch action" in transferring loads laterally to the support girders

5. Combinations of (2) - (5) above.

Brief discussions of the above overlay and other deck rehabilitation options are provided below. Detailed discussion regarding placement of the Georgia DOT structural overlay, Kentucky DOT RSLMC overlay, and the California DOT polymer concrete overlay are given in separate chapters later in this report.

Low-Slump Dense Concrete (LSDC) Overlay. LSDC overlays were first developed by the Iowa Department of Transportation. The normal slump of the overlay mix is 1" or less, and the recommended minimum overlay thickness is 1¾" with 2" being preferred.

Advantages:

- Uses materials and construction procedures which are familiar to bridge construction contractors
- Has a track record of good performance when properly applied
- Uses a rich coat of cement mortar as a bonding compound (can use epoxy if desire)
- Low cost

Disadvantages:

- Requires relative thick and heavy overlay (≈ 1¾" - 2")
- Skid resistance of LSDC overlays is reported to be somewhat poor
- Workability of the mix (because of the low w/c ratio) leaves something to be desired
- Overlay is quite sensitive to curing because of the low w/c ratio

Latex-modified Concrete (LMC) Overlay. Latex modified concrete is a mixture of concrete with a latex-emulsion admixture (approximately 15% latex solid by weight of cement). A widely used latex-modifier is a styrene butadiene type, which was developed by Dow Chemical Company in 1957. The typical thickness of a LMC overlay is 1¼".

Advantages:

- Thinner and lighter than a normal portland cement concrete overlay
- Latex additive makes the concrete less permeable and with improved bonding and self healing properties
- Has a track record of excellent performance
- Requires no bonding agent – the latex provides sufficient bond.

Disadvantages:

- Requires a mobile concrete batching/mixing unit
- Results in a significant increase in deck DL
- Most common problems reported are
  - development of moderate mapcracks
  - bonding failure near the deck joints

Polymer Concrete (PC) Overlay. A polymer concrete overlay protective system, developed by Brookhaven National Laboratory for FHWA has been used as an experimental project in several states in the U.S. The overlay consists of an application of monomer resin to the deck-surface, followed by an application of the fine aggregate. The process is repeated until four layers have been placed. The overlay is relatively impermeable and skid resistant. Generally, the resin is sprayed over the deck and fine aggregate is covered over the resin. After polymerization, the excessive aggregate is removed and the process is repeated for other layers. The four layers produce a thickness of about ½". While this particular polymer concrete overlay is a viable system, there are other similar systems in which polymer concrete is used. The thickness, method of placement, and type of polymer concrete used vary with each system. It should be mentioned that the three different polymer resin systems used are acrylic (or methyl methacrylate mixtures), polyester, and epoxy.

All types of thin polymer overlays should exhibit a tensile bond to properly prepared concrete high enough to fail the substrate concrete (250-350 psi, by ACI 503R). Aggregates used in thin polymer overlays should be as hard as possible for wear resistance. Also, the curing time can vary from 1 to 12 hours depending on the type of binder resin used and temperature conditions. The application methods usually employed for thin polymer overlays are premixed and broom and feed. In the premixed method, also known as the slurry method, the materials are thoroughly mixed and then placed. In the broom and seed method the overlay is built up using different layers, such as the aforementioned method developed by the Brookhaven National Laboratory. Of all the polymer overlays used, the thickness usually ranges from 1/4" to 3/4".

Advantages:

- Very rapid setting overlay requiring minimal traffic interruption
- Provides a highly impermeable deck surface
- Requires minimal surface preparation
- Fastest of the overlays systems to install–provides the least interruption to traffic

Disadvantages:

- High cost
- Handling problems may occur as the materials sets quickly and is quite sensitive to ambient weather conditions
- Coefficient of thermal expansion is much larger than deck concrete. This incompatibility can cause debonding problems and/or tension cracks in the overlay
- Brittleness of epoxy over time–newer generations of epoxy have greatly improved in this area
- Construction unfamiliarity

Silica Fume Concrete (SFC) Overlay. Whereas the overwhelming majority of concrete bridge deck overlays to date have been LSDC or LMC, silica fume (microsilica) concrete overlays are beginning to be looked at by some highway agencies. SFC is a conventional portland cement concrete with the addition of approximately 7-12 percent microsilica solids by weight of the cement.

Advantages:

- Provides a dense, high strength, and low permeability overlay
- Moderate cost
- Good workability (with HRWR)
- Can be transit or site mixed
- Transported to the deck, consolidated, and screeded by conventional finishing machine

Disadvantages:

- High-range water reducers (HRWR) required
- Use of HRWR may restrict transit time to the job site
- No track record of proven performance

Fiber Reinforced Concrete (FRC) Overlay. FRC employs either steel or polyester fibers to improve the tensile strength of the concrete. The fibers are intended to improve the resistance of the overlay to cracking, and to impact loadings. FRC overlays are not widely used partially because the introduction of the fibers normally renders the concrete more permeable. In the presence of chloride ions, this accelerates corrosion and thus deterioration. Since most overlays are used in the northern states where deicing salts are widely used, this is a major disadvantage for the overlay material.

Advantages:

- Reduced overlay cracking
- Potential for superior durability in areas (such as Alabama) where concrete permeability and chloride intrusion are not major issues or problems

Disadvantages:

- Extra cost of fibers
- Greater handling workability problems because of fibers (note, since Alabama does not tine its decks, the fibers do not greatly affect the concrete surface texturing work)
- Higher concrete permeability
- Relatively longer period of time to cure and thus to have deck closed to traffic

Bituminous Overlay. Many bridge decks in the northern U.S. are designed and built with a protective asphaltic overlay on top of a reinforced concrete slab. Whether implementing on a new bridge deck, or as part of a deck rehabilitation, a bituminous overlay begins with the placement of a bituminous fabric water proofing membrane. Normally a primer is applied to the concrete deck to provide better adherence. A tack/bonding coat is applied to the membrane, and followed by a bituminous concrete (asphalt) layer of approximately 2-3 inch thickness. LMC overlays have largely replaced bituminous overlays as the most popular and widely used overlay system.

Advantages:

- Low cost
- Ease/familiarity of construction and installation
- Ease of "seaming" lanes longitudinally in staged construction

Disadvantages:

- Lower durability/longevity–especially in high traffic volume locations
- Sometimes poor bonding of overlay in areas near expansion joints

3.4 Stiffening/Strengthening of Existing Girder Options

Stiffening and strengthening of existing bridge girders may be needed to:

1. Support additional DL resulting from the addition of a deck overlay
2. Improve vibrational characteristics of the bridges and to reduce LL cyclic fatigue stresses and LL impact loadings
3. Reduce LL deflections

It is anticipated that some sort of deck strengthening and stiffening, e.g., via addition of an overlay, will also be implemented to complement the girder rehabilitations.

Some common and newly evolving methods to stiffen/strengthen existing deck support girders are as follows:

1. Bonding of FRP to bottom flange
2. Welding or bolting of additional bottom flange plates
3. Addition of King-post truss support members
4. Addition of extra girders between existing girders
5. Addition of external post-stressing to existing girders
6. Additions of shear/moment capable splices at ends of SS to make support girders and deck continuous
7. Addition of extra girder support points via addition of intermediate transverse support girders.

Each of these girder/supperstructure stiffening/strengthening options is briefly discussed below.

FRP Plates. Advanced composite materials have the potential to stiffen and increase the strength and service life of the Birmingham, Alabama steel girder bridges with badly cracked concrete decks. Additionally, use of these materials is compatible with the need for a rapid rehabilitation procedure. Due to their extreme high strength, light weight, and epoxy bonding attachment, advanced composite materials would be much faster than attachment of steel plates to the existing girders. For example, for a desired stiffness, a carbon composite plate would weight approximately one-tenth that of a steel plate.

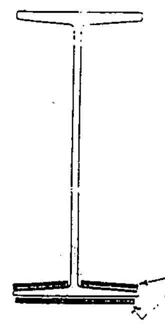
The type of advanced composite material girder rehabilitation envisioned is as shown in Fig. 3.28 with the plates applied to the bottom flange over the middle ¾" of the span (for both simple and continuous span construction). A major issue with this type of rehabilitation is the long term performance of the epoxy attachment of the composite plates to the steel flange. The newest generation of epoxies appear to do an excellent job in providing a strong and durable attachment. Figures 3.29 - 3.30 show photographs of an old I24 x 80 steel beam reinforced with multiple ¼" x 1½" FRP plates in a laboratory test set-up.

Steel Plates. Adding of steel plates is very similar to adding of FRP plates discussed above, except that the method of attachment is by welding or bolting or epoxying. Photographs showing some of strengthening of existing structures by the addition of steel plates is shown in Figs. 3.31 - 3.33.

King-Truss System. The King-Truss type prestressing system (see Fig. 3.34) procedure works by tensioning the strands that are connected below the bottom flange with one or more posts. Threaded end-connections are provided so that proper tension can be induced into the system.

Additional capacity can be obtained by changing the configuration of the truss and by adjusting the tension in the bottom chord. The installation should be monitored by controlling the number of turns of the nuts at the anchors and by measuring the deflection induced in the existing member. Advantages and disadvantages of post-stressing as opposed to adding beam flange plates are given below.

Figure 3.28. Steel Girder Stiffened with Composite Plates.



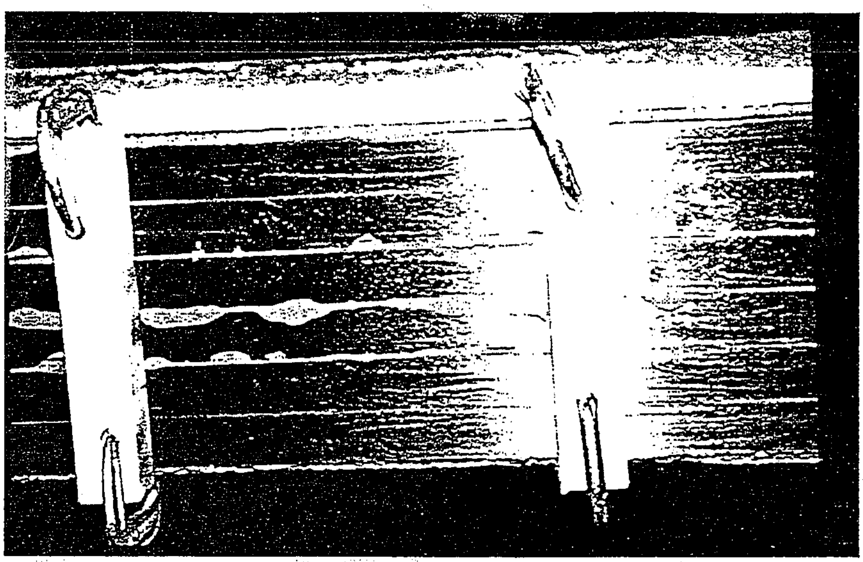


Figure 3.29. FRP Rehabilitation Girder - Bottom Flange View.

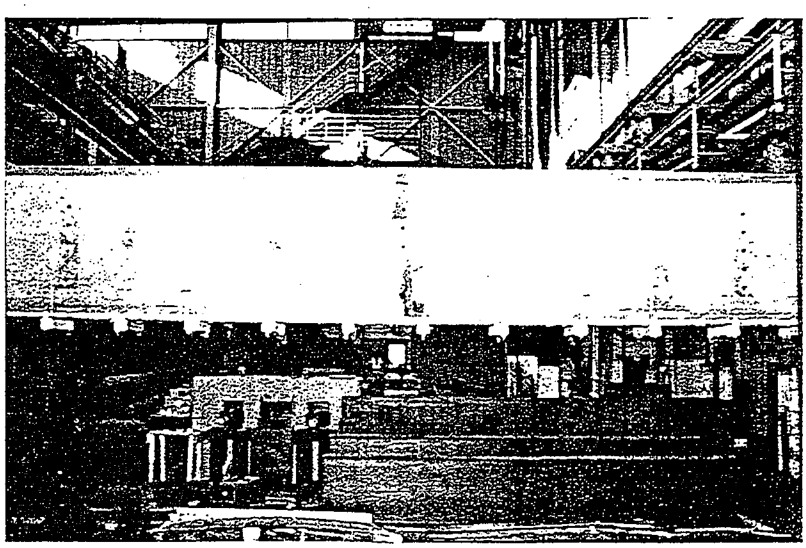


Figure 3.30. FRP Rehabilitated Girder - Side View.

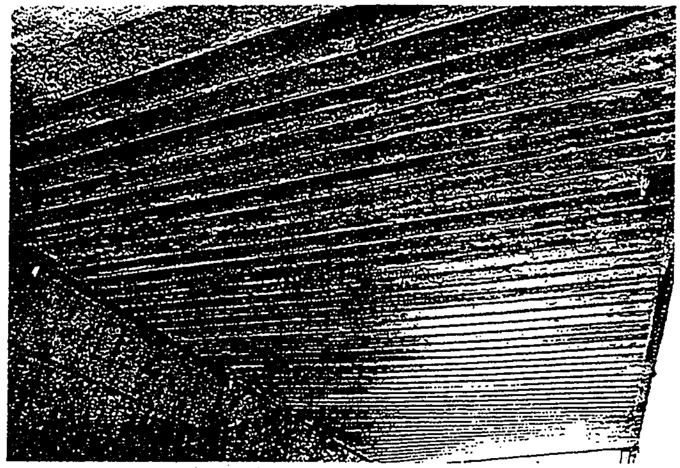


Figure 3.31. Strengthening a Bridge to Accommodate an Increase Live Load-Carried Out Without Stopping Traffic Flow.

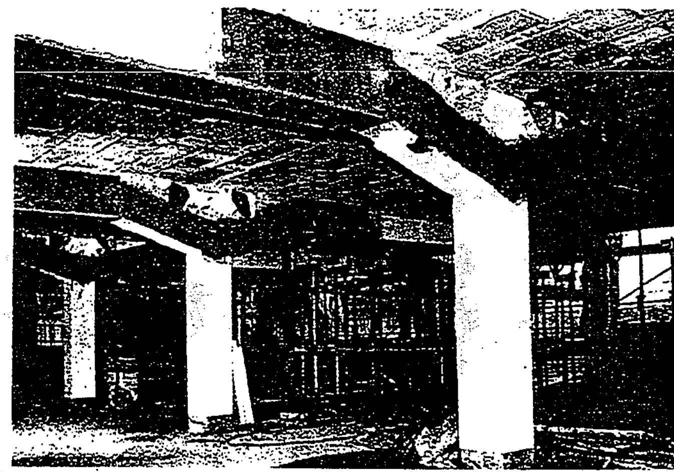


Figure 3.32. Strengthening of Beams to Accept Increase Shear Forces.

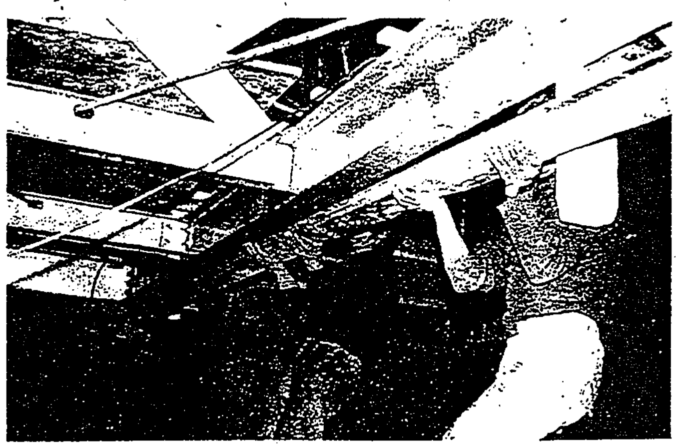


Figure 3.33. Strengthening of Steel Structures-Increasing the Stiffness of a Galvanized I-Beam.

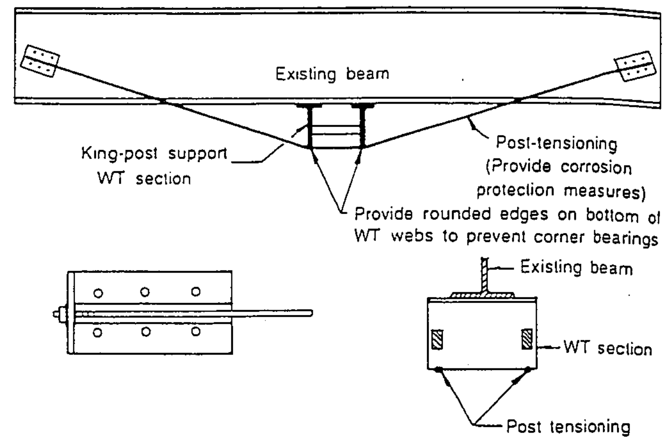


Figure 3.34. King-Truss Type Prestressing System.

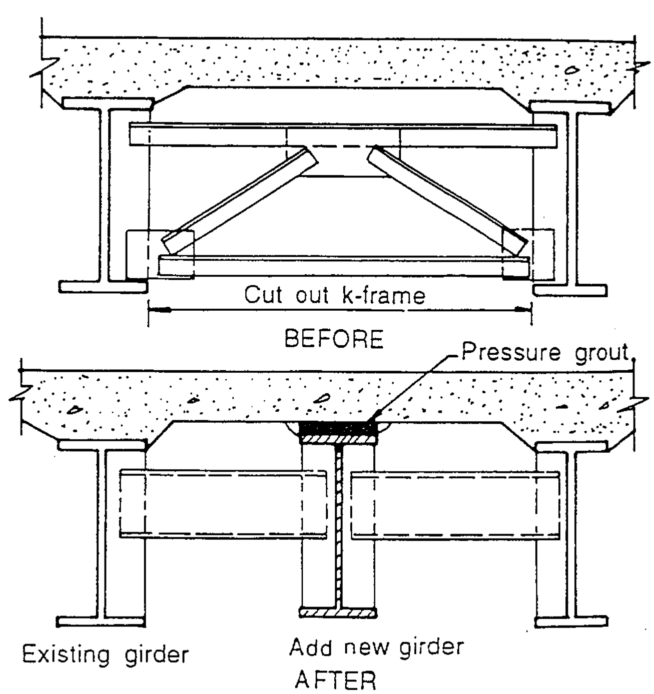


Figure 3.35. Adding Additional Girder.

Advantages:

- It may be more economical
- Normal traffic may be maintained and if detour is needed, the period will be short
- Jacking of the beams to stress-free the members (for the purpose of connection of new flange plates) is eliminated
- In many cases, such as with existing riveted plate girders, increasing the section modulus may not be feasible whereas the King-Truss system offers a feasible (and perhaps economical) solution

Disadvantages:

- Relaxation of the steel tendon can occur
- Tendons need to be protected against corrosion
- Without relieving the axial compression force, the beam will act as a beam-column and the deflection may cause a significant change in stress distribution
- Possible cracking of the concrete deck
- Greater underneath clearance is required to install system

Extra Longitudinal Girders. An inadequate structural system can be strengthened by placing addition stringers or floor beams 'between' the existing members (see Fig. 3.35). New girders can be installed from below the structure by jacking. Any gap between the top flange of the beam and the underside of the deck slab can be filled by drilling holes through the deck slab and pressure grouting or by pressure grouting from below.

External Post-Stressing. Addition of an external post-stressing system to stiffen and strengthen a bridge girder system is a viable option for both steel and concrete girder systems. It is particularity attractive for box girder systems where the post-stressing tendons can be concealed and protected on the inside of the box girders. Figures 3.36 - 3.37 show layouts and photos of some external post-stressing systems.

Convert SS to Continuous. Splicing of support girders at intermediate piers/bents to render them continuous is an excellent way to stiffen a bridge superstructure and reduce deflections and stresses. In doing this, care must be taken to analyze the bridge to ensure that the new load path/distribution system created is safe. In addition, the deck should also be made continuous at the intermediate supports. It

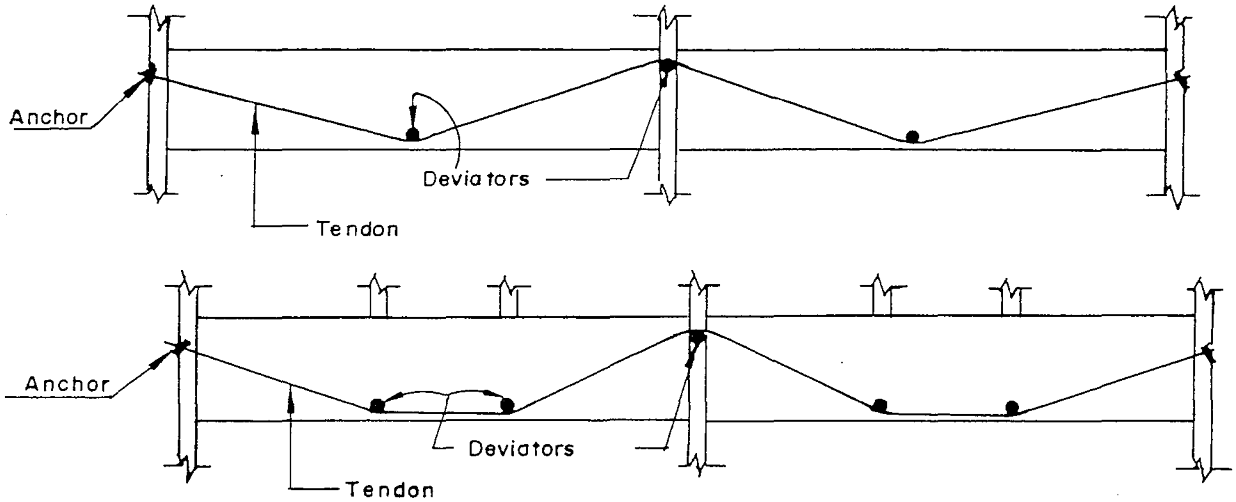


Figure 3.36. Basic Arrangements of External Tendons.

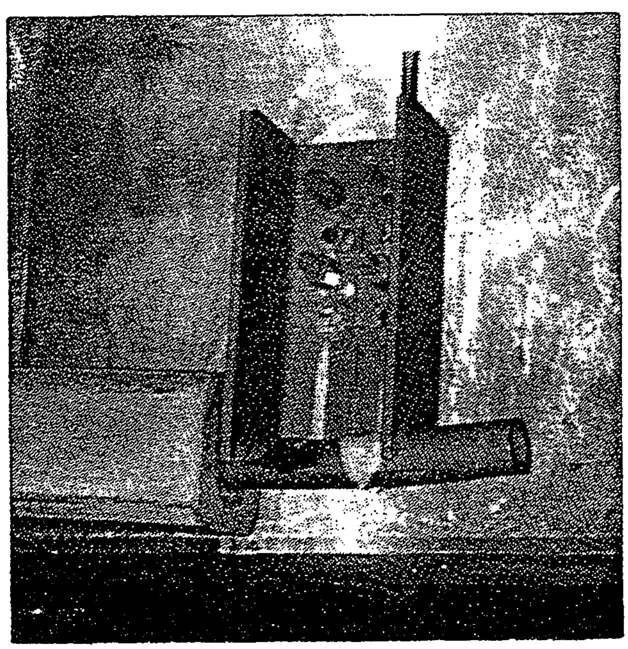


Figure 3.37. Structural Steel Bracket Bolted to the Side of a Beam at a Low Point; the PT is Encased in Concrete for Protection.

should be noted that in addition to stiffening and strengthening the superstructure, this rehabilitation will remove the deck and girder joints and one line of bearing supports at each intermediate support. Since deck joints and bearing supports are both components causing bridge durability problems and high maintenance cost, this action should also reduce future maintenance cost and increase bridge service life. Figure 3.38 shows a sketch of this rehabilitation action.

Extra Transverse Support Girders. Under certain favorable conditions where the vertical clearance and the geometric requirements allow, the load carrying capacity can be increased by shortening the effective span length of the bridge. Installation of auxiliary piers or a transverse floor beam system with a main girder would convert a simple span bridge in to a continuous span bridge and reduce the effective span length (see Fig. 3.39).

3.5 Closure

Current and newly emerging options for replacing or rehabilitating bridge decks, and for stiffening/strengthening the bridge girders have been identified and discussed in this chapter. Next, for application of this information to the Birmingham, Alabama bridge deck problem, one must determine the following for the Birmingham, Alabama bridges.

- the structural form and geometrics
- the type and extent of deck damage
- the source/cause of the deck damage
- the time and traffic flow constrains under which the deck replacements or rehabilitations must be made.

The synthesis of this information with the viable deck replacement/rehabilitation options discussed in this chapter is then needed to identify appropriate candidate replacement/rehabilitation options. This will be reported on in the ensuing chapters.

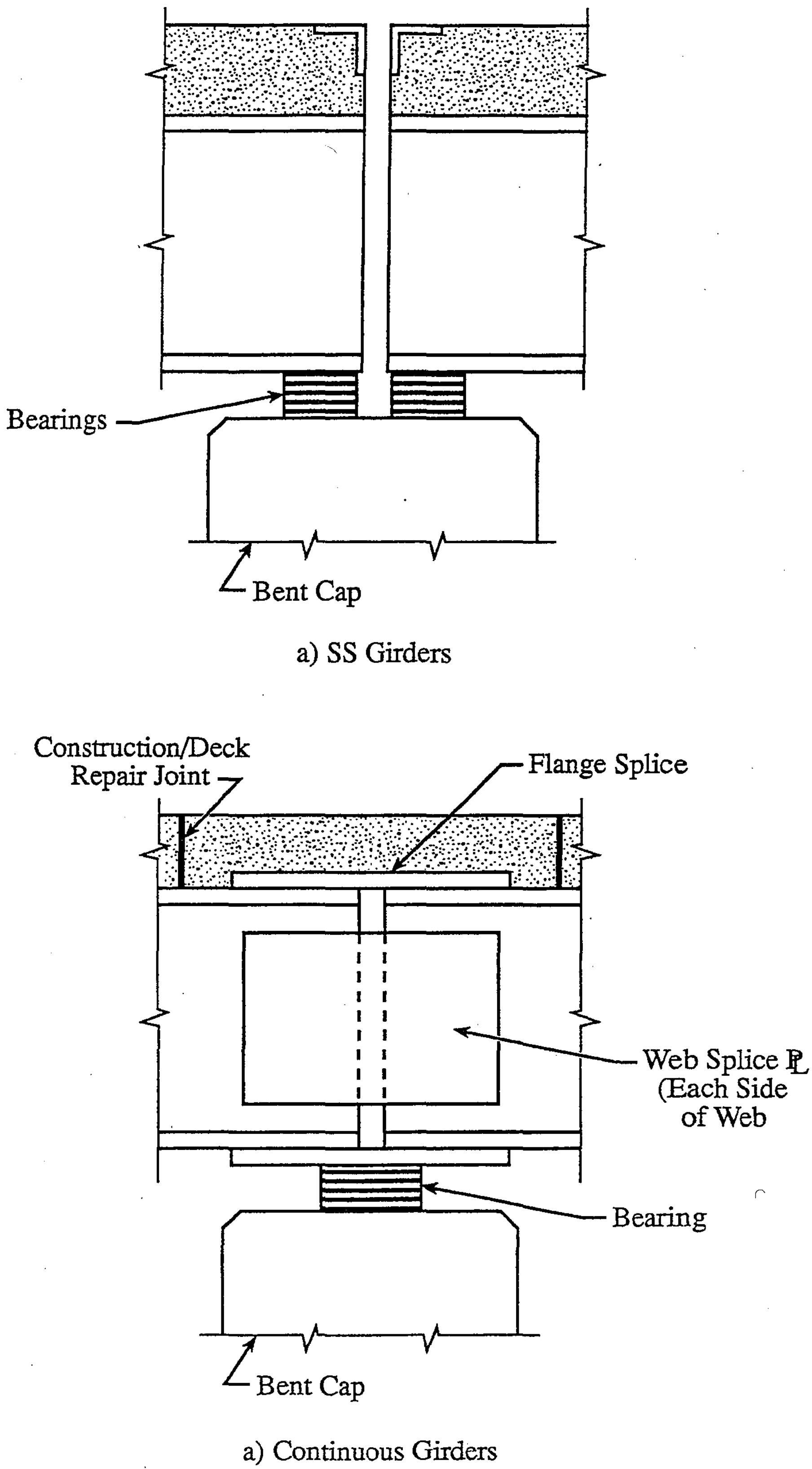


Figure 3.38. Conversion of SS to Continuous Girders.

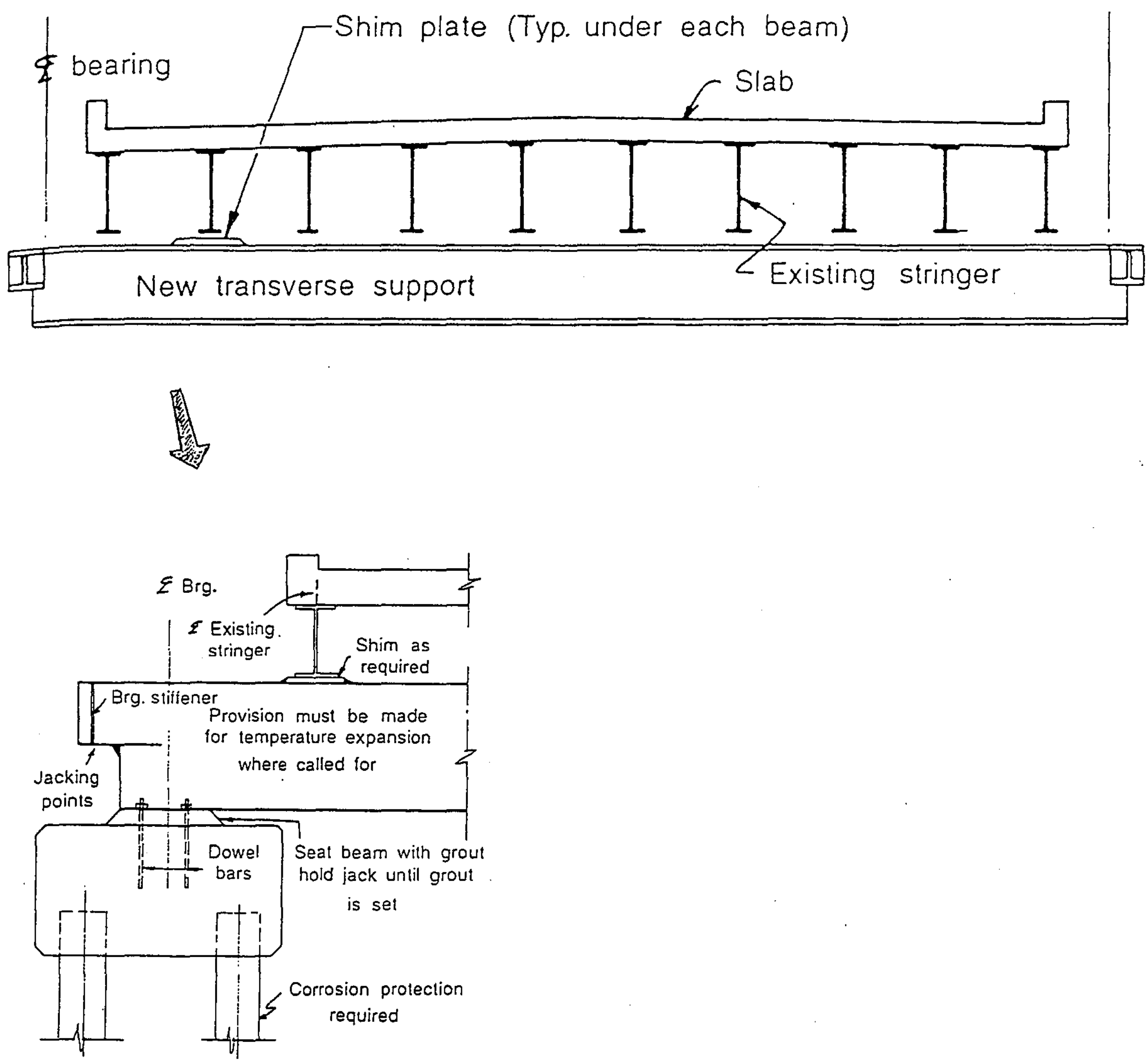


Figure 3.39. New Transverse Beam.

4. STATE/DESCRIPTION OF TYPICAL BIRMINGHAM INTERSTATE BRIDGE DECKS

4.1 General Background

Alabama has many bridges which have good substructures and superstructures, but deteriorated decks which need rehabilitating or replacement. Unfortunately, many of these bridges are on heavily traveled interstate highways in urban areas, and any rehabilitation or replacement (R/R) scheme must be implementable in a rapid manner with minimal interference with highway traffic. For example, there are numerous (approximately 80) bridges in the Birmingham area which are part of the I-65 and I-59 interstate highway system through the city which are approximately 25 years old and have badly cracked concrete decks. It appears that these cracks are primarily the result of

- early drying and thermal shrinkage
- early concrete obstructed settlement
- thin and flexible decks (approximately 6 ½" thickness)
- light and flexible steel girder superstructure
- heavy traffic volume (≈ 77,000 ADT in 1995)
- heavy truck loading (8% in 1995 with many estimated as being overloaded)

Various people have estimated the remaining deck life to be anywhere from 10-20 years, with 15 years being a common mean value quoted. The tremendous volume of traffic that these bridges carry, the lack of any convenient alternate routings, and the rapid rate of deterioration (decks have drop from a Condition Rating of 8 to 5 over their present 25 years of operation/age) have the Alabama Department of Transportation (ALDOT) management apprehensive and concerned about the best course of action to take for these decks.

4.2 Primary Finding of UAB Birmingham Bridge Deck Study

During the period 1992 - 1994, the ALDOT contracted researchers in the Department of Civil Engineering at the University of Alabama at Birmingham (UAB) to (1) investigate and determine the causes of excessive damage to bridge decks in the Birmingham area, and (2) make recommendations to achieve more durable bridge decks.

The UAB researchers performed condition surveys as well as analytical and experimental analyses on 10 Birmingham bridge decks (5 with severe cracking damage and 5 largely undamaged for control).

General information about the bridges investigated is provided in Table 4.1. As can be seen in the fourth column of Table 4.1, the average age of the bridge studied was approximately 40 years.

Some of the primary results of the UAB condition surveys are summarized in Table 4.2. As can be seen in Table 4.2 the bridges with the severely damaged decks were on the unfavorable side relative to the required values for most of the conditions shown. Relative to the undamaged decks, the bridges with the damaged decks were on the unfavorable side of every condition assessed.

The UAB researchers determined that severe transverse cracking at the top surface above the transverse rebars was the predominant form of deck damage. Their assessment of the primary causes of the severe transverse cracking, and their recommended corrective actions were:

- Excessive deck slenderness, i.e., excessively large values of deck span/thickness ratios. The design deck thickness for 7 of the 10 decks was 6 1/4", and was 6 1/2" for 2 others. Even worse, the mean as-constructed thickness for these 9 decks was 5.61", with a range of 5" - 6 1/4". So, indeed the decks are very thin. The UAB researchers recommended increasing the deck thickness so that the design tensile stresses do not exceed the concrete cracking strength.

- Excessive truck loadings in number of the trucks and in weight of trucks.

- Insufficient longitudinal rebar in both the top layer (shrinkage and temperature rebar) and the bottom layer (load distribution rebar). The researchers recommended increasing the percentage of top longitudinal rebar and in placing it on top of the transverse steel to improve its effectiveness and to reduce transverse cracking over the top transverse bars.

- Poor quality, gradation, and shape of the deck concrete coarse aggregate. Decks of hard, crushed and well graded aggregate performed better than decks of softer/absorptive, rounded (river gravel) and poorly graded aggregate. The researchers found that river gravel (rounded aggregate) was used as the coarse aggregate in 4 of the 5 severely damaged decks. They recommended using only crushed stone coarse aggregate, and that coarse and fine aggregate be well graded.

- Poor construction QC/QA by construction contractors and ALDOT inspectors. Large variations in deck thicknesses, rebar spacing and rebar cover were detected during the field condition surveys. In most every case, the variation or error was on the side of reducing the deck strength, stiffness and durability. The UAB researchers recommended additional or improved inspection be conducted during construction to assure the as-built bridge is the same as that shown in the construction documents.

4.3 Results of AU Birmingham Bridge Study

During the period 1994 - 1996, the project Co-PI, Dr. Mike Stallings, performed a field and laboratory study on the effects of diaphragms on multigirder steel bridges. As part of the field study, two interstate bridges in Birmingham were visually inspected to determine the existing conditions of the girders and the concrete deck. The selected bridges were a nine steel girder 24.5 m simple span bridge on I-65, and an eight steel girder 3 span continuous bridge with span lengths of 22 m, 32 m, and 22 m on I-59. The girders of the simple span bridge were composite (via shear lugs) with the concrete deck, and the continuous span bridge was non-composite. Girder spacing for both bridges was approximately 8 ft. as can be seen in Figs. 4.1 and 4.2.

The top and underside were inspected to observe the extent and type of cracks present in the decks. The underside of the concrete decks were closely inspected with the aid of a lift truck. Due to the high volume traffic on the bridge, the top side had to be inspected from a safe distance. To determine the extent of the cracking in the deck, a total of four sections in the 24.5 m simple span bridge were selected for crack mapping. Two sections were marked at midspan between girders 3 and 4 and girder 4 and 5, and two sections were marked between the same girders near supports. The sections at midspan had a width equal to the girder spacing and a length of 3 m (1.5 on either side of midspan). The sections near the supports had a width equal to the girder spacing and a length of 2 m.

Figures 4.19 and 4.23 display the resulting cracking patterns at midspan after all cracks had been identified and highlighted. As can be seen from the figures, the underside of deck is significantly cracked in a grid-like pattern with longitudinal and transverse cracks being on the average of 150 -300 mm apart. The cracks were generally hairline cracks with up to 20% having widening 1 - 2 mm at the surface. Some of the transverse cracks showed notable water seepage indicated by the white stalactite deposits. No significant differences were noticed between the sections marked at midspan and near the supports concerning crack widths or crack patterns.

A total of ten sections in the 76 m 3-span continuous bridge were marked to inspect the extent of cracking in the deck. The observed crack patterns were generally the same as observed in the simple span bridge with approximately 20% having widened to about 1 mm. The extent of cracking near the support appeared to be somewhat less than that at midpsan. Some water seepage was observed in the transverse cracks as indicated by the efflorescence. As heavy vehicles crossed the bridge, the cracks could be observed to open and close.

The 24.5 m simple span and the 76 m 3-span continuous bridge appear to have amount and type of deck cracking that is typical of others similar bridges in Birmingham. Some bridges were observed to have somewhat more deck cracking while others had less. Bridges with low volume traffic lanes appeared to have less cracking under those lanes. In the continuous spans, deck cracking in the positive moment regions was a little more pronounced than the negative moment regions.

4.4 Current Study Findings on Condition of Birmingham Bridge Decks

In April 1997, the project PI made several trips to Birmingham to visit bridge sites and to visit with bridge maintenance and inspection personnel to discuss and document the state of the Birmingham I-65 and I-59/20 bridge decks. The results of these visits are presented below primarily in graphical and photographic form. Brief narrative introductions and discussions are give below, but the best descriptions of the state of the bridge decks are felt to be shown in Figs. 4.3 - 4.30.

4.4.1 General Bridge Descriptive Parameter Values. General descriptions and characteristics of some typical major bridges of concern are shown in Table 4.3. Overview photographs of the two I-59 bridges, and I-65 bridge in Table 4.3 are shown in Figs. 4.3 - 4.6. Note that the I-59 bridges carry 1-way traffic and are located in the Birmingham Central Business District (CBD). The I-65 bridge carries 2-way traffic with the traffic separated by a barrier rail.

Several bridges on I-65 through Birmingham were widened to add an additional lane on the inside (on the left side when looking in the direction of travel) in both the north and south bound directions around 1985-1987. The 1,666' long bridge over 1st Avenue South and the RR tracks was included in this widening. The new lanes plus shoulders plus inside barrier rail appear to be in good condition and probably should be left in place in the even of a deck replacement. If they are left in place, it would significantly reduce the time, cost and traffic disruptions of a deck replacement.

Randall Mullins of ALDOT's Bride Design Bureau indicated that the new section of the deck was designed using the old design, but was constructed with SIP metal deck forms. Therefore, the thickness of the newer section is 6.5" thick exclusive of the protrusions of the ribs of the stay-in-place forms. These protrusions are included in the weight of the deck dead load when designing the girders. Actually, the specific quantity used for dead load is an additional inch in thickness of concrete over the area of the deck, but this additional inch is not considered in the structural capacity of the deck. Since the newer section was designed the same as the older section, there are no shear lugs to provide composite action. However, Mr. Mullins feels that the newer section is probably behaving compositely, due to friction and adhesion between the deck and girders.

4.4.2 Bridge Deck Performance and Condition Rating. Bridge deck performance/deterioration for the typical major bridges identified in Table 4.3, as reflected by deck-structural and deck-inspector's condition ratings, are shown in the form of Condition Rating vs. Age plots in Figs. 4.7 - 4.10. Note in these figures that for the first 15-years the deck condition ratings were quite good (as would be expected). However, after approximately 15 years, the deck condition ratings began to drop dramatically. Figures 4.7 and 4.8 for the I-59 bridges indicate that deck condition ratings will probably reach a level of 4 in approximately 3 years (from the last inspection/data point which was 1996). This coupled with projections from the Penn DOT, which indicate a usable life of 10 years after reaching a CR = 4, would translate into a remaining usable life (from 1997) of approximately 12 years. For the I-65 bridge (see Figs. 4.9 and 4.10) the deck condition rating appears to have stabilized at CR = 5 at the present time. How long this will continue is unknown; however based on the Penn DOT projection of 10 years of usable life after reaching a CR = 4, it would appear that the I-65 bridge should have a remaining usable life of 12 years or more.

Deck condition ratings parallel deck crack width growths (of the largest cracks) as can be seen in Figs. 4.11 - 4.13. Note in these figures that maximum deck crack widths remained below 16 mils for approximately the first 15-years, then in periods of 4, 6, and 8 years respectively for each of the three bridges, the maximum crack grew to widths in excess of 125 mils. This is approximately an order of magnitude increase over a period, on average, of 4 years. This is a dramatic rate of crack growth and is probably the main reason that the deck condition ratings dropped dramatically over this same period of time.

It should be noted that in later discussions with ALDOT bridge inspectors and maintenance personnel, the dramatic drops in condition ratings (see Figs. 4.7 - 4.10) and increases in deck crack widths (see Figs. 4.11 - 4.13) at around 17 years of age were attributed primarily to changes in inspection evaluation and appraising procedures, rather than abrupt changes in the behavior and performances of the bridges. However, ALDOT 3rd Division bridge inspectors did indicate that the significant changes occurred over a relatively short period of time.

4.4.3 Photographic Display of Deck Cracked Condition. A photographic portrayal of the state of cracking of the I-65 and I-59 bridges described in Table 4.3 and Figs. 4.3 - 4.13 of the previous sections are shown in Figs. 4.14 - 4.30. Figures 4.14 - 4.23 are of the I-65 bridge with 2-way traffic (separated by a barrier rail). Figures 4.24 - 4.29 are of the I-59/I-20 bridge through the Central Business District (CBD). It should be noted that the underside of the I-59 bridge decks were very similar to those of the I-65 bridges shown in Figs. 4.18 - 4.23. It is very evident in Figs. 4.14 - 4.30 that the bridge decks are showing very significant cracking deterioration. It can also be observed in the photos that, based on a lack of efflorescence, most of the cracks do not fully penetrate the deck.

ALDOT bridge inspectors indicated that about 5 years ago they began to see longitudinal cracks in the top of the deck above the edges of the support girders. These cracks are continuing to grow in length and width, and are beginning to combine with the older transverse cracks (which are almost everywhere) to form surface spalls as shown in Fig. 4.30.

4.5 Deflection Comparison for Two Birmingham Interstate Bridges

A deflection comparison between experimental and theoretical results was made for two Birmingham, AL, interstate bridges. The experimental results were obtained by Dr. M. Stallings of the Civil Engineering Department at Auburn, and his graduate student Eric Stafford in 1994 and as part of their ALDOT funded field study of diaphragm behavior and performance in multigirder steel bridges. A special truck, which is described later, was used to apply loads to the bridges while deflectometers were used to determine the deflections. The theoretical results for deflections, using the special truck loading case, were obtained by Randall Mullins of ALDOT by using computer programs which will be discussed later.

The reasons for making this comparison were twofold. The first reason was to establish benchmark values of load-deflection responses for future comparisons to assess structural degradation of the bridge girder-deck superstructure with time. The current structural condition of the superstructure can be estimated based on a comparison of theoretical and experimental deflections; however, structure idealization and modeling assumptions would probably render such comparisons of limited value. However, changes in the bridge load-deflection behavior with time should provide an excellent indicator of structural degradation of the bridge superstructure. Thus, if structural deterioration of the superstructure is of significant concern, an experimental re-evaluation of the bridge load-deflection behavior can be performed at a future date and compared with the behaviors observed from Stallings' and Stafford's 1994 testing.

The second reason for making this comparison was to determine how effective it would be to add shear studs to the girders to ensure composite action with the deck and thus enhance stiffness of the superstructure with a deck replacement. If the comparison of the non-composite, continuous span bridge reveals that the bridge is actually closely resembling composite behavior, then composite action is effectively being accomplished by friction and adhesive forces. This would suggest that shear studs might not be effective in stiffening the superstructure. However, if the comparison reveals that the noncomposite bridge is indeed not acting in a composite manner, then the addition of shear studs should be effective for stiffening and strengthening purposes in the case of a deck replacement.

4.5.1. Bridge Descriptions. The two bridges tested were both in Birmingham, AL, with one being a simple span (see Fig. 4.1) and the other a continuous span (see Fig. 4.2). The simple span bridge has shear studs to make the deck composite with the girders. It spans 24.5 meters over 2nd Ave. South along I-65 in the northbound lanes. The bridge has a 165 mm thick concrete deck on 9 rolled steel wide flange girders spaced at 2.44 meters transversely. Seven girders are W920 x 289 with 33mm x 267mm x15.8mm cover plates welded to the bottom flange. The other two are W920 x 488 rolled steel sections. These two are girders 8 and 9, located on the west side of the span (see Figure 4.1). The bridge carries 5 lanes of traffic which are each 3.66 m wide.

The continuous bridge is a 76 meter three-span bridge on I-20/59 westbound between 18th and 19th streets, which carries 4 lanes of traffic. This bridge has a 165 mm thick reinforced concrete deck with no shear studs to make it composite with the stringers. There are 8 steel wide flange girders made up of 3 different sizes spliced together: W920 x 289, W920 x 342, and W920 x 365 (see Fig. 4.2).

4.5.2. Field Testing for Deflections. During the field testing, which was conducted in the Summer of 1994, deflections were induced by using two identical load testing trucks provided by the Alabama Department of Transportation Bridge Rating and Load Testing Section. Both trucks were characteristic of short heavy trucks in that they each had 3 axles. A schematic of the test truck axle configuration and axle weights is given in Figure 4.31. Only one of the trucks was used in obtaining the particular deflections used in this comparison.

Vertical deflections of the girders were measured with demountable deflectometers which were essentially calibrated cantilever beams. The deflectometers were mounted at midspan of each girder using C-clamps to secure them firmly against the bottom flange. For the simple span bridge, the deflections in this report were obtained from girders two, three, and four (see Figure 4.1). To obtain the maximum deflection in girder two, the loading was positioned in lane 1. To obtain the maximum deflections in girders three and four, the loading was applied in lane 2 (see Figure 4.1). In all cases, the test truck was moved slowly across the bridge and the deflection at the center of each girder was monitored and the maximum value recorded (but not the corresponding longitudinal location of the test truck). For the continuous span bridge, only the maximum mid-span deflection (of the center span) was used in this comparison. This was the deflection of girder 4 corresponding to loading in lane 2 (see Figure 4.2). The loading was applied as explained before.

4.5.3 Theoretical Deflection Values. The theoretical values for deflections were obtained by using two ALDOT computer analysis programs, BRUFEM and BRASS. BRUFEM was used to obtain the theoretical deflections on the simple span bridge. The acronym BRUFEM stands for Bridge Rating Using Finite Element Methods. This program was developed by the University of Florida and the Florida Department of Transportation.

The theoretical deflections of the continuous span bridge were obtained by using the program BRASS, which was developed by the Wyoming Department of Transportation. The acronym BRASS stands for Bridge Rating and Analysis of Structural Systems. This program gives the maximum deflection for the full width of the bridge at a given longitudinal position and for a given load. In other words, the program places the load at different points across the width of the bridge until the maximum deflection is found. For this comparison, the maximum deflections at mid-span of the center span were used. One of these deflections was obtained by using a composite computer model. The other was obtained by using a non-composite computer model.

4.5.6 Comparison of Deflections. For the simple span bridge, the three deflections that were obtained experimentally and theoretically were compared. Table 4.7 shows the results of this comparison, and it can be seen from this table and the corresponding bar graph in Fig. 4.32 that the theoretical deflections were quite close to those obtained experimentally.

For the continuous span bridge, the maximum deflection at mid-span was compared to the two theoretical deflections obtained by BRASS. Both Table 4.8 and Figure 4.33 show that the deflection measured in the field was much closer to the value obtained by using the composite computer model. This suggests that the three span continuous bridge is acting in a composite manner via deck-to-girder friction and adhesive forces.

Thus, results of the maximum girder deflection load testing conducted by Stallings and Stafford in the Summer of 1994, indicate that both the 24.5m simple span bridge and the 76m three span continuous bridge are behaving in a composite manner. This is as it should be for the simple span bridge since it was constructed with shear lugs to achieve composite behavior. However, the continuous span bridge does not have shear lugs, but is apparently achieving composite behavior by friction and adhesive forces. This suggests that if decks are replaced on the continuous span bridge, and shear lugs are added to the girders for composite action, no significant stiffening of the superstructure and reduction in deflections will occur since the superstructure is currently acting in a composite manner. However, if the decks are replaced, shear lugs should be placed on the girders to assure composite behavior.

The load-maximum deflection behavior of the simply supported I-65 bridge and the 3-span continuous I20/59 bridge in Birmingham, AL documented here (and in Ref.(999)) provides a convenient source of comparison for assessment of future structural degradation of these bridges. The load testing described here can be repeated in 5, 10 etc. years and compared with the 1994 load-deflection values to assess structural degradation, of the bridge superstructures.

Table 4.1. General Information About UAB Decks Studied

Damage Category	Study No.	Location	Const. Date (age)*	Span Type/ Length(s)	Girder Type	Deck Cond. Rating	Deck Damage	Transverse Crack Spacing (Crack Width)
Severely Damaged Decks	1	I-59 over Little Canoe Creek	1961 (34)	simple 34'	RC	3	advanced deck damage, sudden localized deck failures	24" (0.04")
	2	I-65 over US 31 near Blountsville	1959 (36)	continuous 30 degree skew 60'/55'/59'	RC	4	severe transverse cracking	24" -
	3	US 78 over Locust Fork of the Warrior River	1962 (33)	continuous 141'/180'/140'	steel	5	severe transverse cracking	18" - 30" (0.05")
	4	AL 269 over TCI Railroad	1954 (41)	continuous 134'/164'/225'	steel	5	moderate transverse cracking	55" (0.04")
	5	AL 269 over Copeland Ferry	1955 (40)	continuous 184'/184'	steel	5	severe transverse cracking	26" (0.05")
Undamaged Decks	6	AL 79 over Five Mile Creek	1956 (39)	simple 45 degree skew 34'	RC	6	minimal damage (spalling @ exp. jt.s)	-
	7	US 78 over Kelly Creek	1955 (40)	simple 45 degree skew 46'	RC	8	minimal damage	-
	8	US 31 over Bishop Creek	1955 (40)	simple 42'	RC	7	minimal damage	-
	9	US 31 over Black Creek	1955 (40)	simple 40'	RC	7	minimal damage	-
	10	AL 269 over Southern Railroad	1945 (50)	simple 36'	RC	7	minimal damage	-

*At time of survey in 1995.

Table 4.2. Summary of UAB Condition Survey Results

Damage Category	Bridge Study No.	Measured vs. Required Deck Thickness ($D_{meas.}/D_{req.}$)	Measured Deck Span (Transverse) to Thickness Ratio ($S/D_{meas.}$)	Measured vs. Required Concrete Strength ($f'_{c\ meas.}/f'_{c\ req.}$)	Design Tensile Stress vs. Modulus of Rupture ($f_{t\ Design}/f'_{r}$)	Total Truck Loading Intensity (Σ ($W_{recorded}/W_{legal}$))	Measured vs. Required Distribution Rebar Spacing ($S_{meas.}/S_{req.}$)	Measured vs. Required Shrinkage/ Temperature Rebar Spacing ($S_{meas.}/S_{req.}$)	Concrete Coarse Aggregate
Severely Damaged Decks	1	0.81	14.9	1.25	2.22	5,500	1.2	0.9	smooth quartz and sandstone gravel
	2	0.85	14.5	-	1.81	10,500	1.9	-	polished quartz gravel
	3	1.00	10.7	1.75	1.24	20,500	1.5	1.5	smooth sandstone gravel
	4	0.81	20.6	1.70	2.07	2,500	2.4	1.8	slag
	5	0.88	13.4	1.88	1.59	2,500	1.7	1.2	smooth sandstone gravel
Undamaged Decks	6	0.80	13.0	2.15	1.61	7,500	1.2	0.9	crushed limestone
	7	0.96	10.9	-	1.11	100	1.6	0.8	pea gravel
	8	0.96	10.9	2.73	1.00	1,200	0.9	1.0	crushed limestone
	9	0.88	11.8	2.10	1.35	800	1.2	0.7	slag
	10	0.88	11.8	2.14	1.34	500	1.2	0.7	slag

Table 4.3. Bridge General Descriptive Parameter Values

Parameter	I-59 Bridges		I-65 Bridges[e]
	South Bound	**North Bound**	
Bin No.	10670	10617	14407
Structure No.	I-59-37-24.1A	I-59-37-24.1B	I-65-37-10.8
Location	CBD[a]	CBD[a]	RR's & 1st Ave. S.
Year Built	1972	1972	1970
Age	25[b]	25[b]	27[b]
Length	6592'	6632'	1666'
Max Span	104'	104'	105'
Width	45.5'	45.5'	148'
Traffic	1-way	1-way	2-way
Structure Type	Steel continuous multi-girder	Steel continuous multi-girder	Steel continuous multi-girder
Deck-Girder Design	Noncomposite	Noncomposite	Noncomposite
Girder Spacing	≈ 8'	≈ 8'	≈ 8'
Deck Thickness	≈ 6½"	≈ 6½"	≈ 6½"
ADT	76,600[c]	76,600[c]	133,480[c,d]
% Trucks	8%[c]	8%[c]	6%[d]

[a]Central Business District
[b]As of 1997
[c]1995 values
[d]Note that this is 2-way traffic
[e]One new lane plus shoulder added on inside of bridge w/each direction around 1986.

Table 4.4. Bridge Inspection/Performance/Deterioration Data for I-59 South Bound Bridge Near Birmingham Civic Center (BIN 10670)

Inspection Date	Bridge Age	Condition Rating		Cracking/Deterioration Observed
		Deck Structure	Overall	
Sept. 1979	7	7	8	Hairline cracks with seepage
Mar. 1982	10	7	8	Hairline cracks with seepage
Sept. 1984	12	7	8	Hairline cracks with seepage
Feb. 1986	14	7	7	Transverse Class 1 cracks with seepage
Mar. 1988	16	7	7	Spotted areas of deck repair; transverse Class 2 cracking with seepage
Mar. 1990	18	7	6	Class 1-4 transverse cracking
Mar. 1993	21	6	6	Class 1-4 transverse cracking
Jun. 1994	22	5	5	Heavy wear on deck with some rebar exposed; raveling at construction joints; span #88 has 4 potholes (1' x 1' x 2"); Class 5 transverse, longitudinal and map cracking
Aug. 1996	24	5	5	Wear on deck; potholes; Class 5 longitudinal, transverse and map cracking

Table 4.5. Bridge Inspection/Performance/Deterioration Data for I-59 North Bound Bridge Near Birmingham Civic Center (BIN 10671)

Inspection Date	Bridge Age	Condition Rating		Cracking/Deterioration Observed
		Deck Structure	Overall	
Sept. 1979	7	7	8	Transverse cracks with efflorescent seepage
Mar. 1982	10	7	8	Transverse cracks with efflorescent seepage
Sept. 1984	12	7	8	Transverse cracks with efflorescent seepage
Feb. 1986	14	8	8	Transverse Class 1 cracks with seepage
Mar. 1988	16	7	7	Spotted areas of spalling; spotted areas of deck repair; Class 1 transverse cracking with seepage
Mar. 1990	18	7	6	Heavy wear on deck; pothole in deck (48" x 24" x 3") span #59; Class 1-4 transverse cracking with seepage
Jul. 1990	18			Truck lost roll of steel on bridge causing 11 holes in deck
Mar. 1993	21	6	6	Collision damage on deck @ span #85
Nov. 1994	22	6	6	Heavy wear on deck; random areas of moderate spalling; numerous Class 1-5 transverse cracks
Nov. 1996	24	5	5	Potholes; Class 5 cracks with seepage

Table 4.6. Bridge Inspection/Performance/Deterioration Data for I-65 Bridge Over US 11 and Railroads in Birmingham (BIN 14407)

Inspection Date	Bridge Age	Condition Rating				Cracking/Deterioration Observed
		Deck Structure		Overall		
		SBL	NBL	SBL	NBL	
Oct. 1970	0	9	9	9	9	Hairline cracks with water seepage
Jan. 1973	3	8	8	8	8	Transverse hairline cracks with efflorescence seepage
Jan. 1976	6	8	8	8	8	Hairline cracks with efflorescence
Mar. 1978	8	7	8	8	8	Transverse cracks with efflorescence
Jun. 1980	10	7	8	8	8	Transverse cracks with efflorescence
May 1982	12	7	8	8	8	Transverse cracks with efflorescence
Oct. 1984	14	7	8	8	8	Transverse cracks with efflorescence
Jun. 1986	16	7	7	7	7	Heavy transverse Class 1 and 2 cracking in SBL; transverse cracks with efflorescence in NBL
Jun. 1988	18	5	5	5	5	Heavy transverse Class 1-5 cracking with heavy seepage
May 1989	19	6	6	6	6	Minor potholes in deck
Jul. 1990*	20	7*	7*	7*	7*	Moderate wear on deck; map cracking with seepage span #11
Jul. 1992	22	5	5	5	5	Small potholes @span #7 (12" x 3" x 1"); heavy wear with aggregate exposed; pothole in expansion joint @ BT #12 in NBL (36" x 8" x 6"); numerous Class 5 transverse cracks; Class 4 transverse and longitudinal cracking span #5 and 7 in NBL
Apr. 1994	24	5	5	5	5	Heavy wear on deck with aggregate exposed; numerous small potholes; numerous Class 1-5 transverse cracks
Jun. 1996	26	5	5	5	5	Heavy wear; numerous pot-outs and potholes; Class 1-5 transverse and longitudinal cracks

*Increase in condition ratings between 5-89 and 7-90 was due to repairs made during this period.

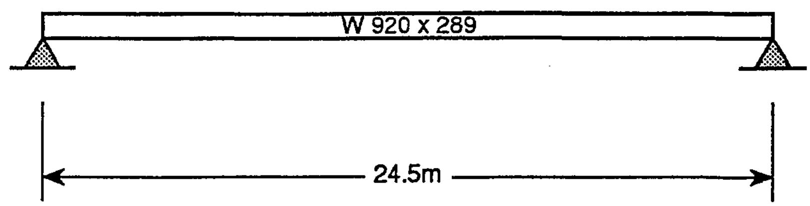
Table 4.7. Experimental and Theoretical Deflections for 24.5m Simple Span Bridge

Girder No.	Loading Location	Maximum Center Line Deflection	
		Theoretical (mm)	Experimental (mm)
2	lane 1	11.73	11.1
3	lane 2	11.63	13.9
4	lane 3	8.78	11.1

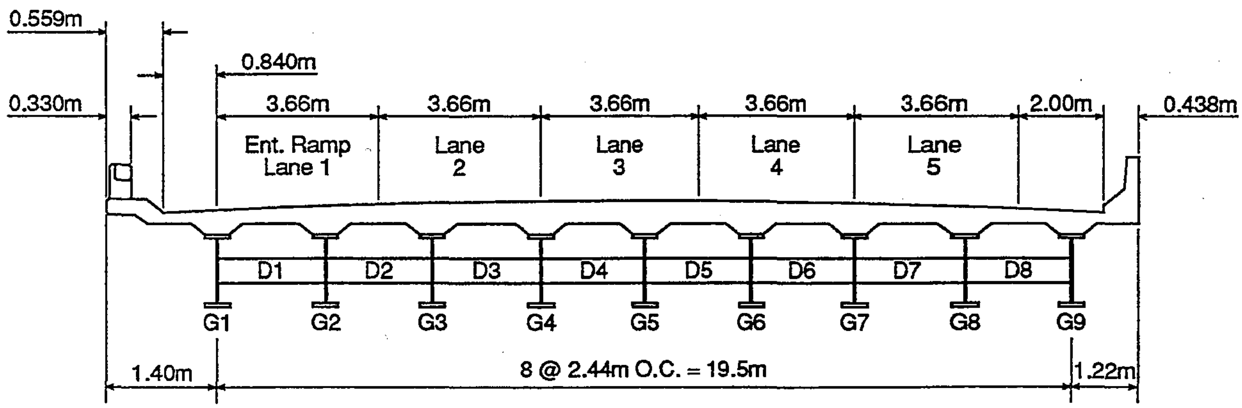
Table 4.8. Maximum Deflections of 76m Three-Span Continuous Bridge

Girder No.	Loading Location	Maximum Center Line Deflection of Center Span		
		Experimental (mm)	Theoretical Composite (mm)	Theoretical Non-composite (mm)
4	lane 2	15.7[1]	32.9[2]	76.7[2]

[1]Diaphragms Out
[2]Not necessarily girder four or loading in lane 2 (rather, maximum deflection across width of bridge)

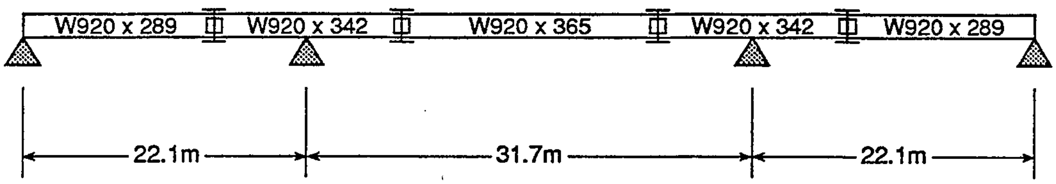


a. Bridge Span / Elevation

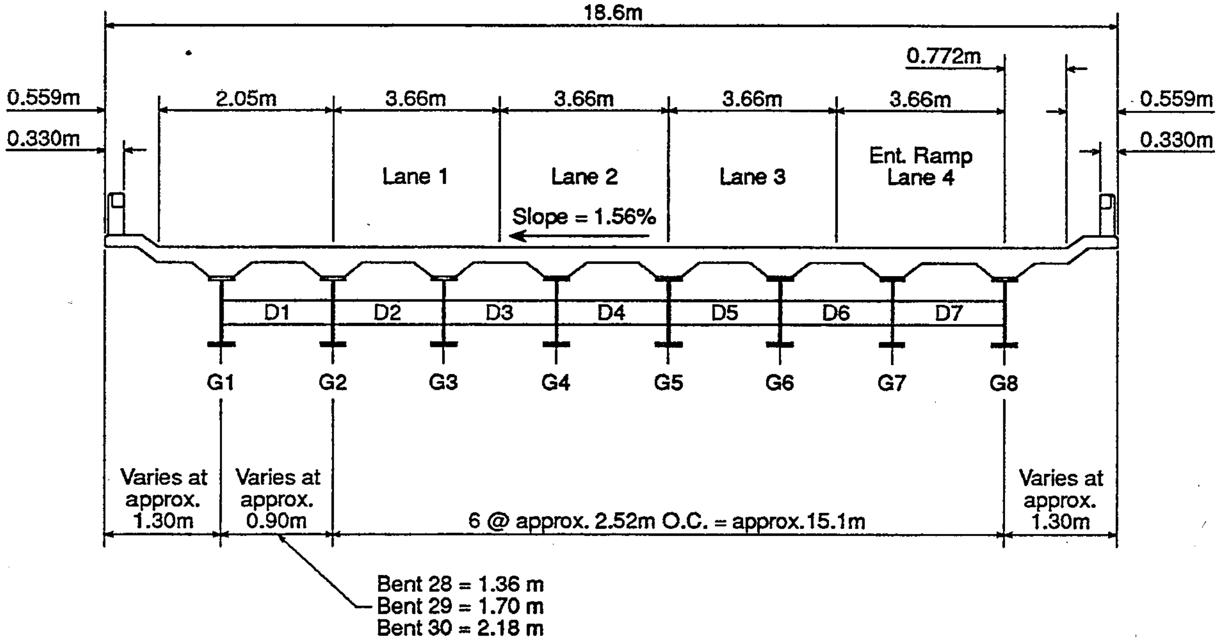


b. Bridge Cross Section (Looking into Traffic)

Fig. 4.1. 24.5 m Simple Span I-65 Bridge in Birmingham, AL



a. Bridge Span / Elevation



b. Bridge Cross Section (Looking into Traffic)

Fig. 4.2. 76 m 3-Span Continuous I-20/59 Bridge in Birmingham, AL

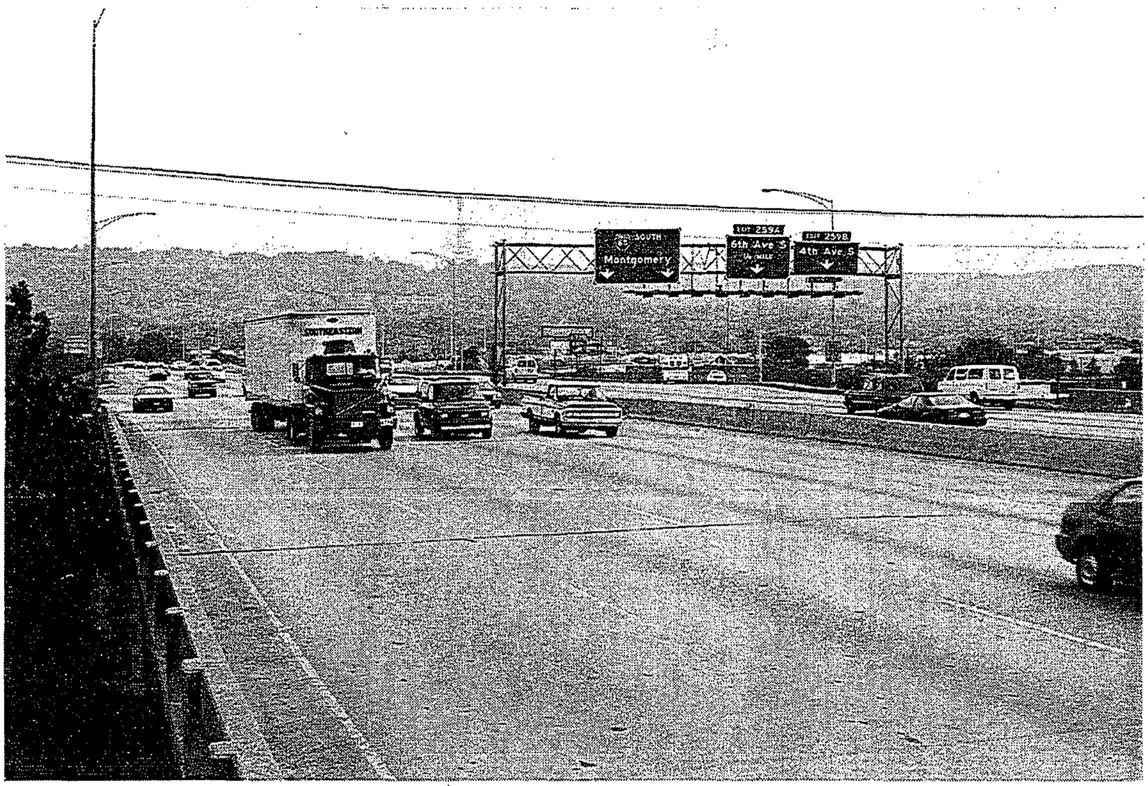


Fig. 4.3., Overview of I-65 Bridge Near Downtown Birmingham Looking South

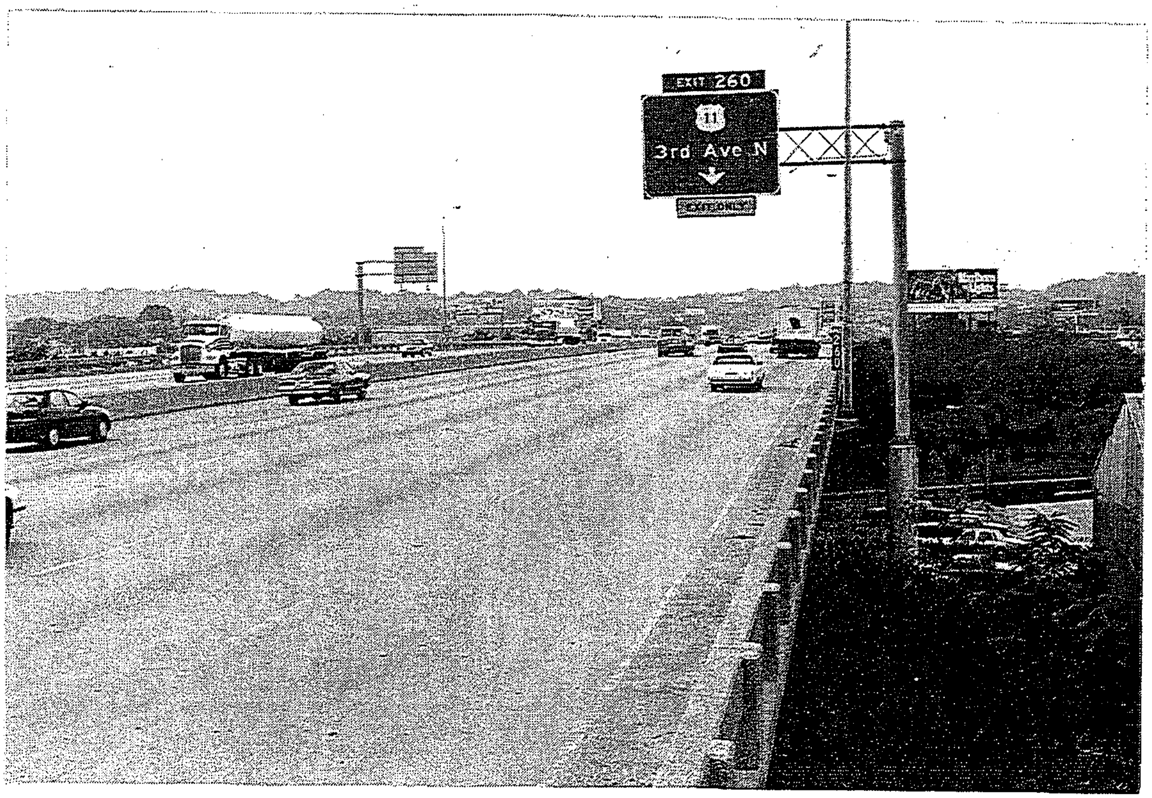


Fig. 4.4. Overview of I-65 Bridge Near Downtown Birmingham Looking North

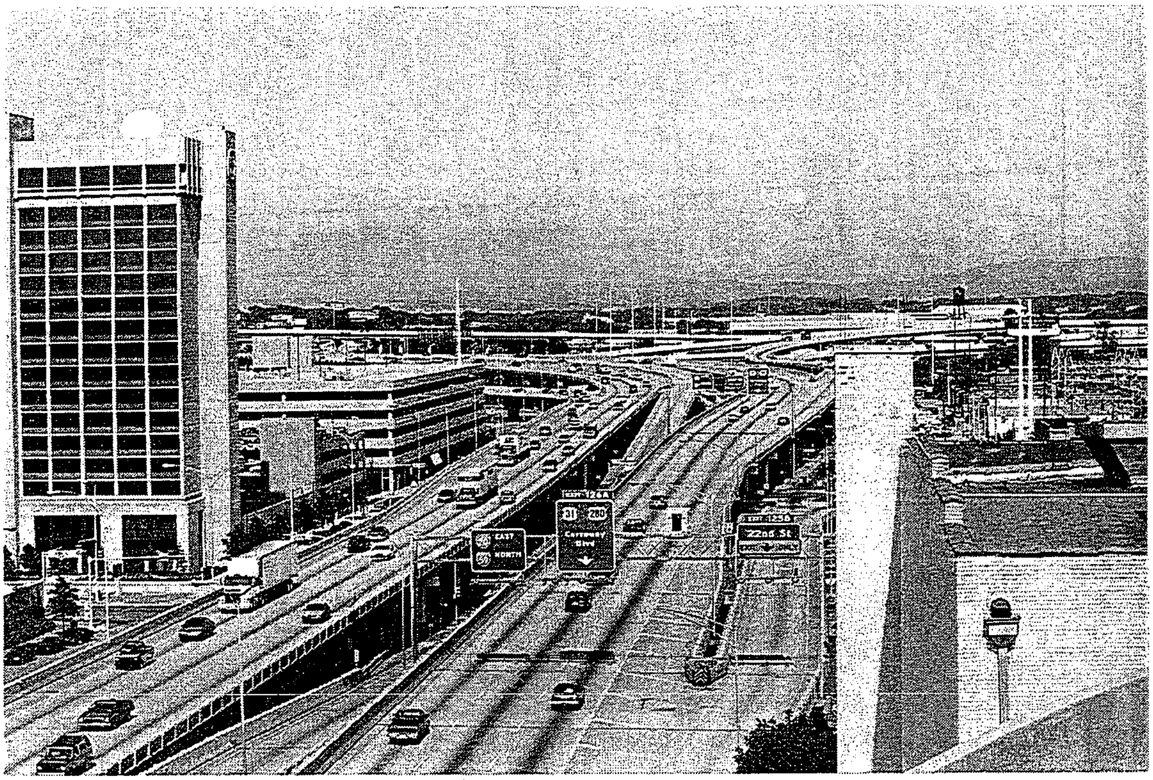


Fig. 4.5. Overview of I-59 Bridge in Birmingham CBD Looking North

Fig. 4.6. Overview of I-59 Bridges in Birmingham CBD Looking South

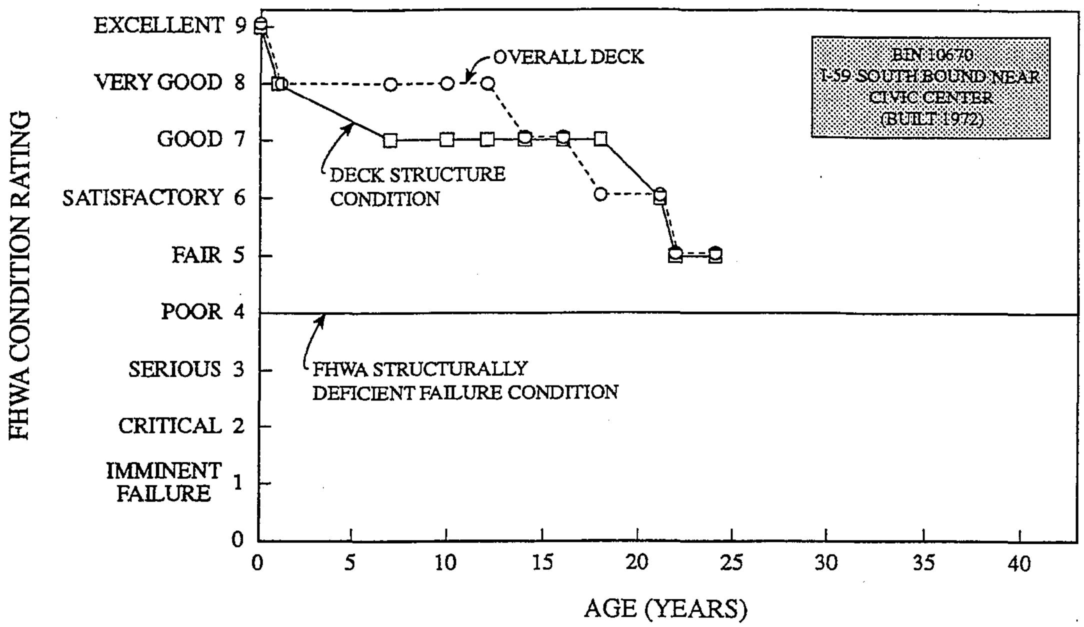


Fig. 4.7. Deck Structure and Overall Deck Condition Rating vs. Age Curve for Birmingham, AL Bridge - BIN 10670

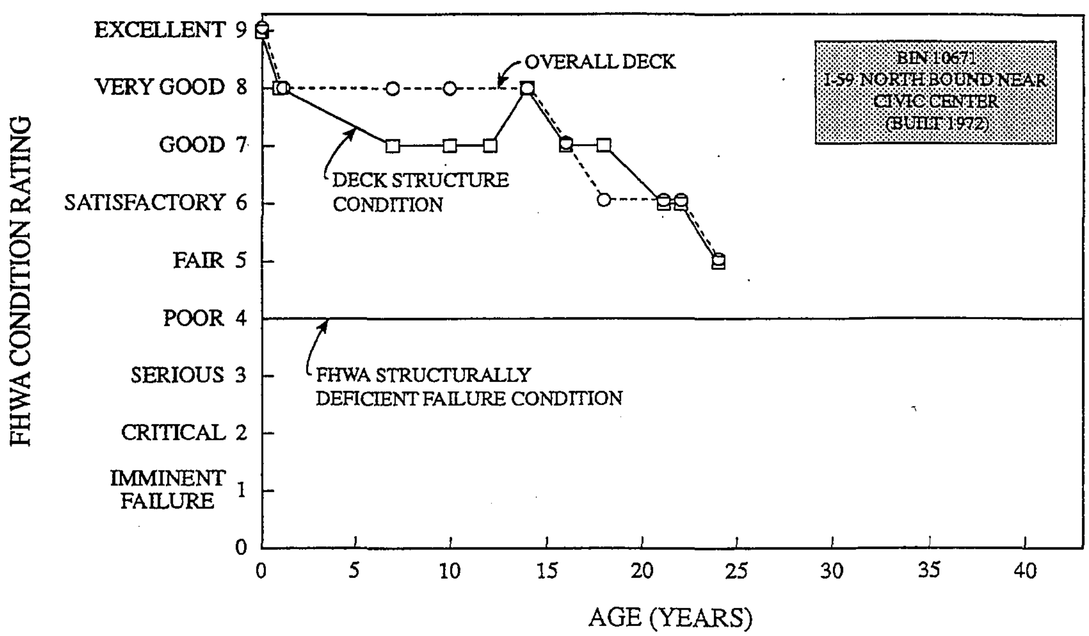


Fig. 4.8. Deck Structure and Overall Deck Condition Rating Age vs. Age Curve for Birmingham, AL Bridge - BIN 10671

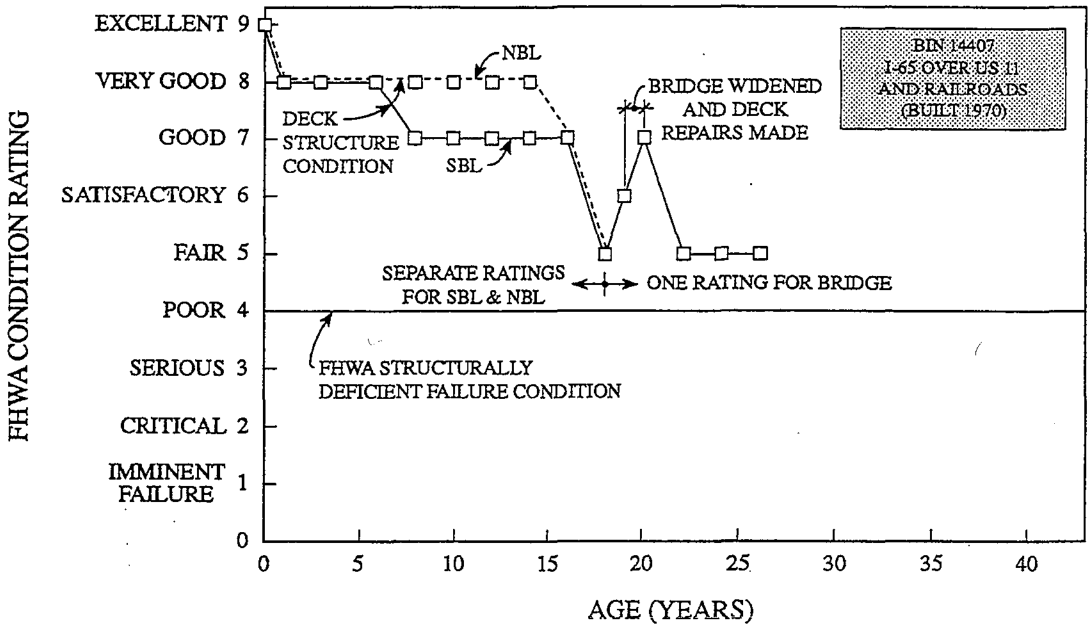


Fig. 4.9. Deck Structure Condition Rating vs. Age Curve for Birmingham, AL Bridge - BIN 14407

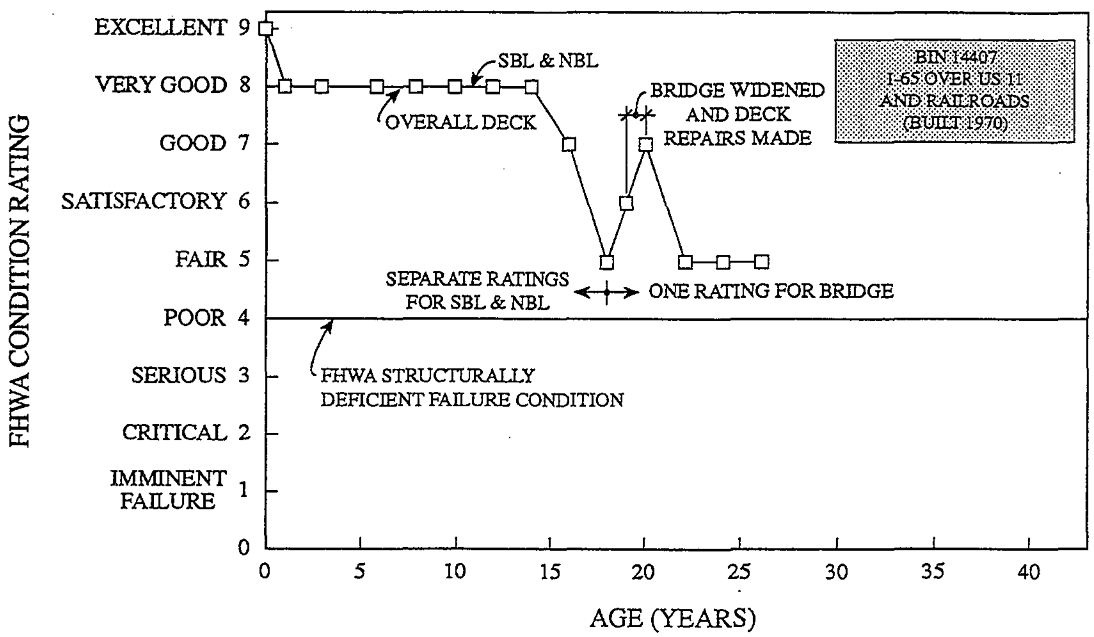


Fig. 4.10. Overall Deck Condition Rating vs. Age Curve for Birmingham, AL Bridge - BIN 14407

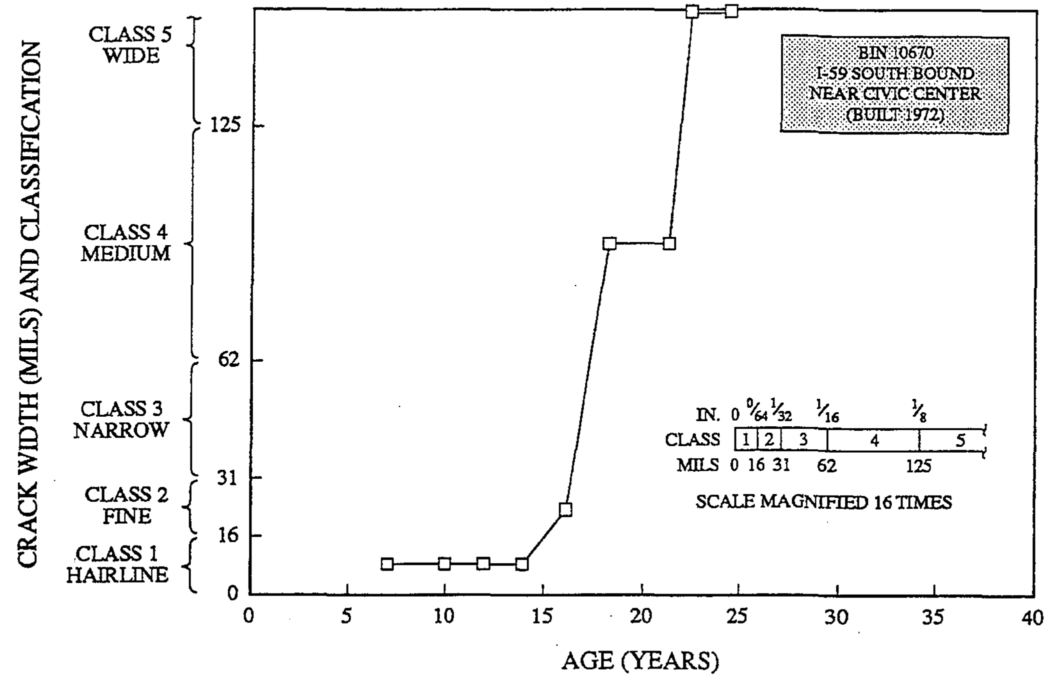


Fig. 4.11. Maximum Deck Crack Width Growth for Birmingham, AL Bridge - BIN 10670

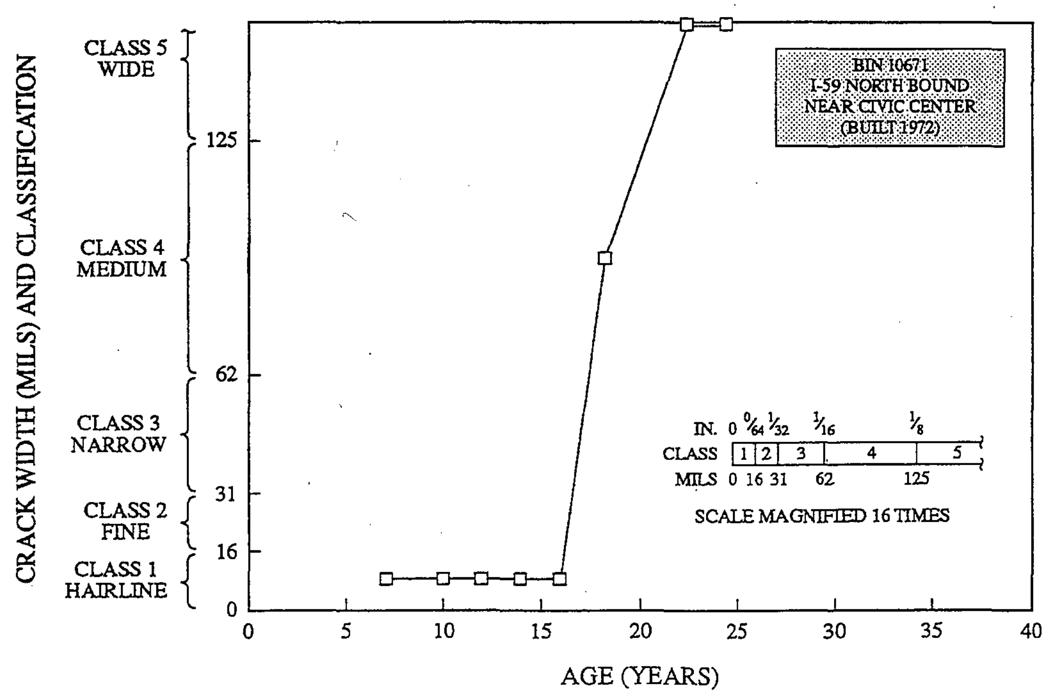


Fig. 4.12. Maximum Deck Crack Width Growth for Birmingham, AL Bridge - BIN 10671

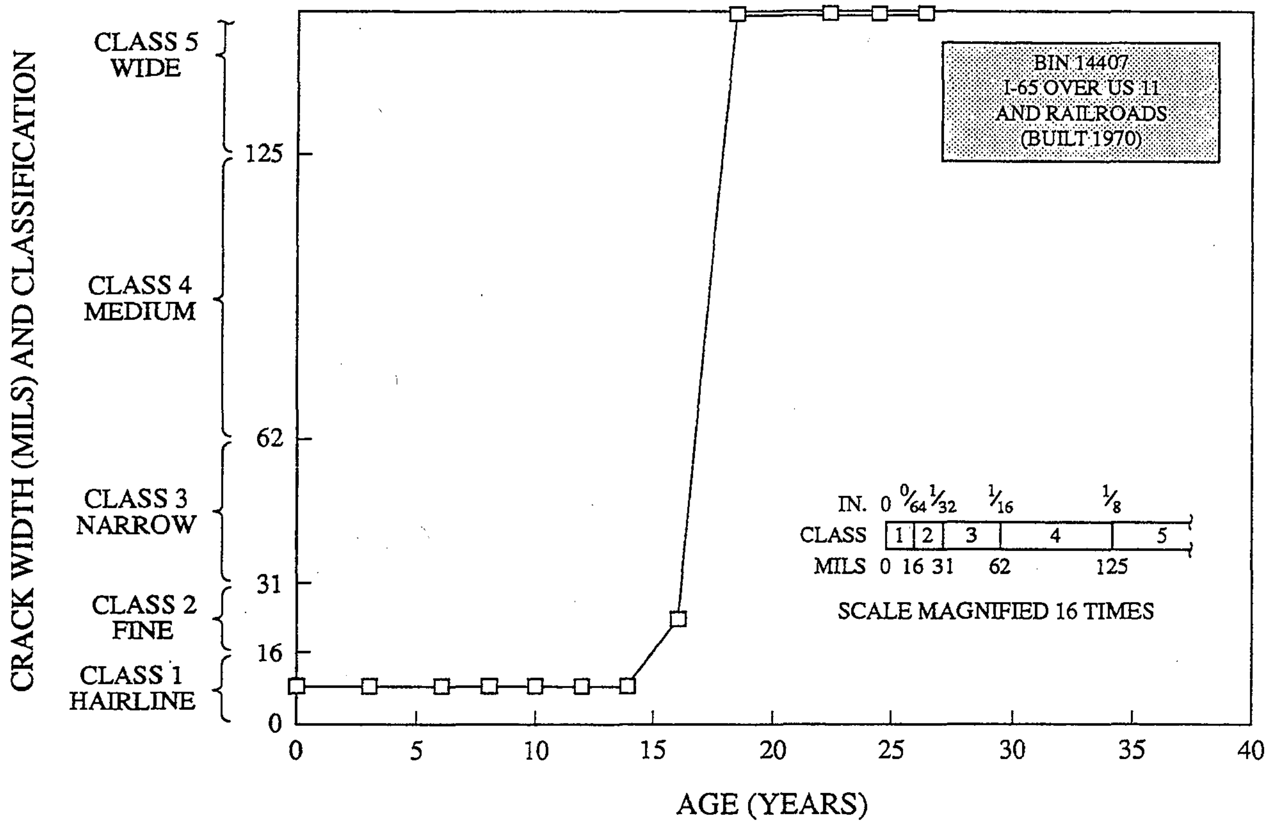


Fig. 4.13. Maximum Deck Crack Width Growth for Birmingham, AL Bridge - BIN 14407

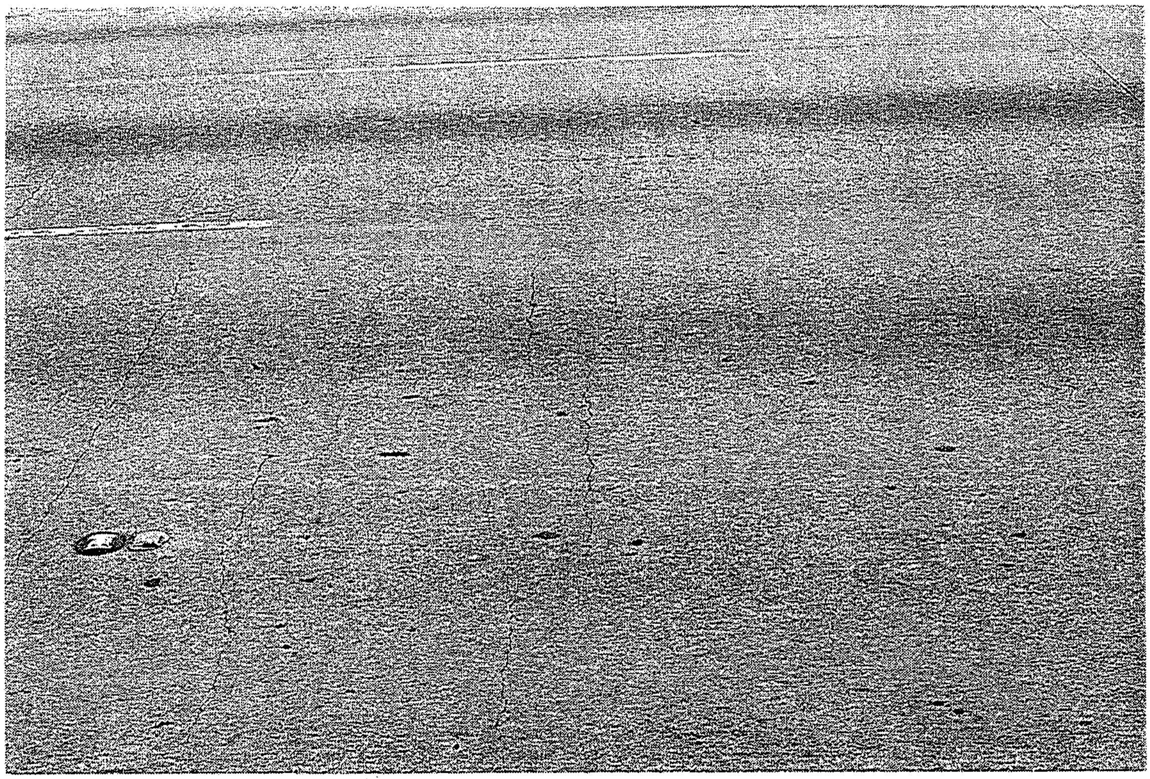


Fig. 4.14. Extensive Transverse Cracking on I-65 Bridge

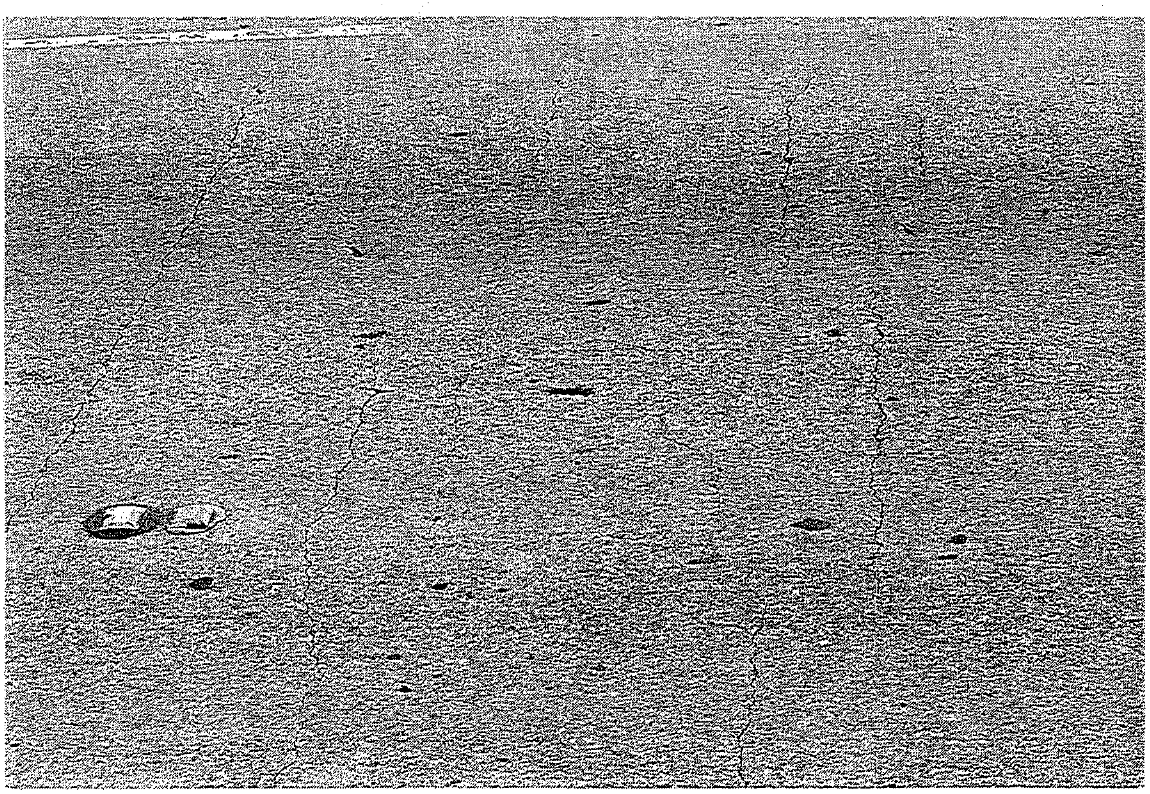


Fig. 4.15. Close-up of Transverse Cracking on I-65 Bridge

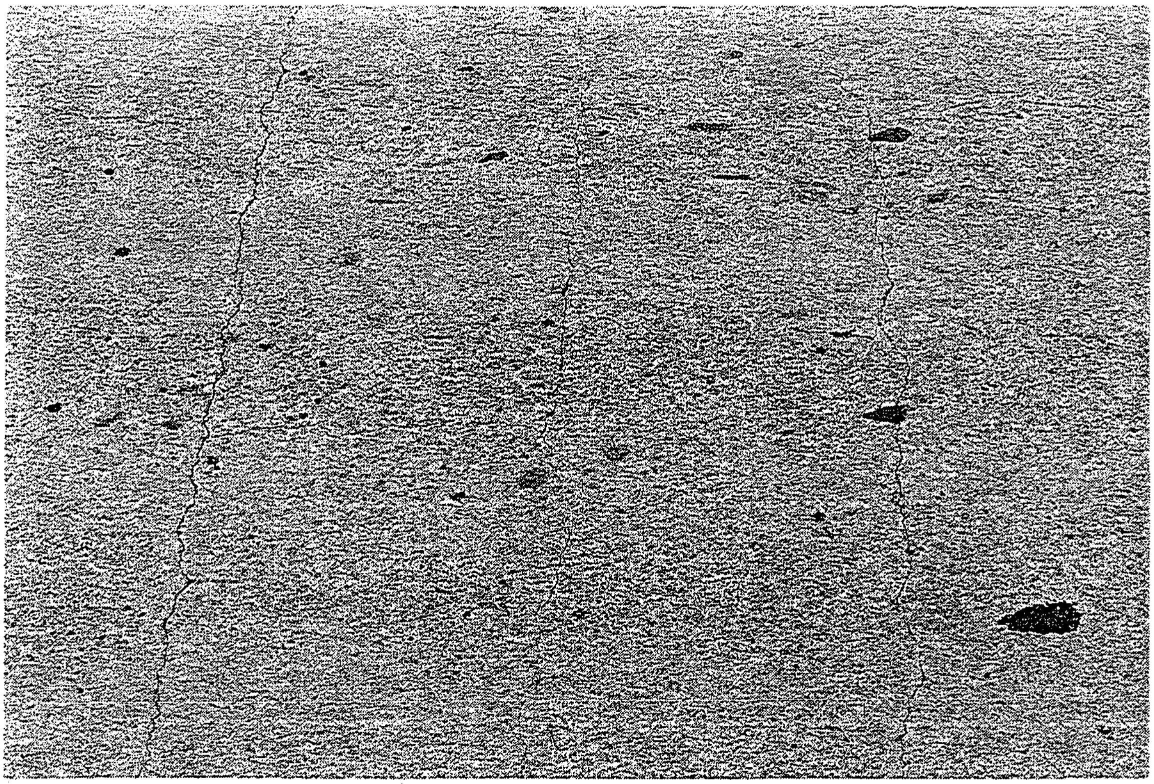


Fig. 4.16. Further Close-up of Transverse Cracking on I-65 Bridge

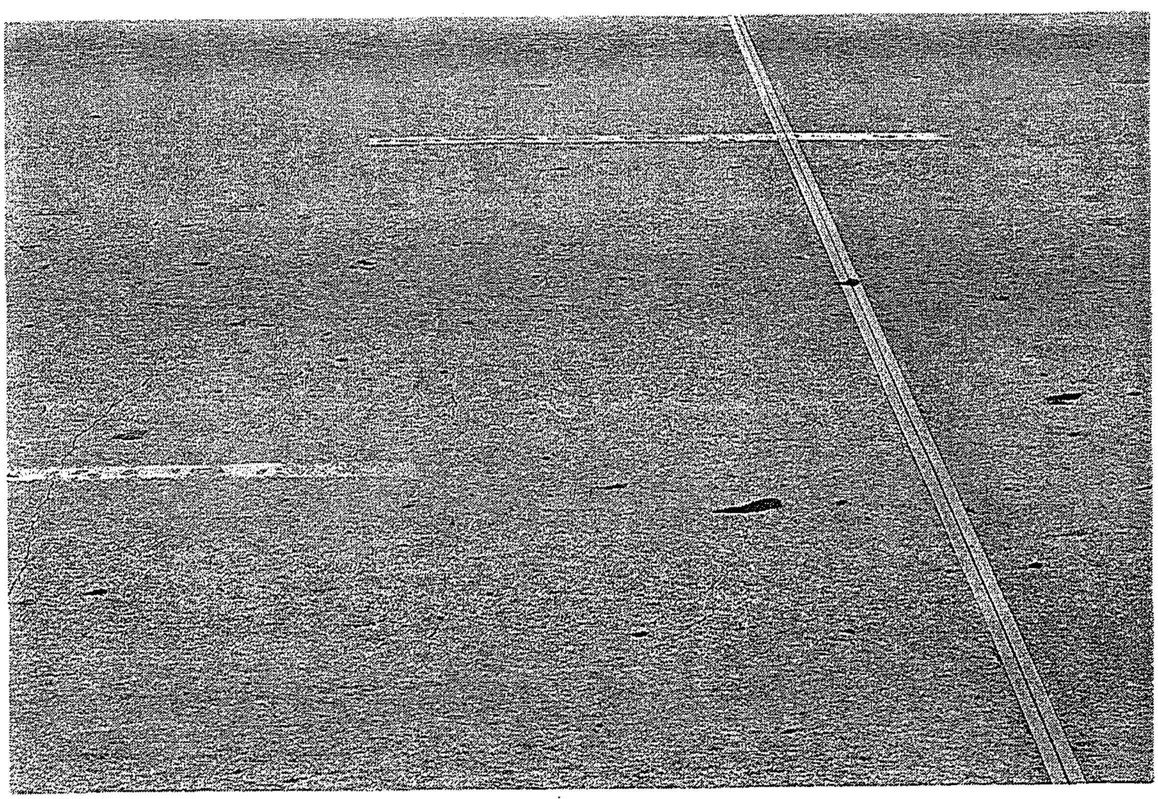


Fig. 4.17. Transverse Cracking Near End of 3-Span Continuous I-65 Bridge

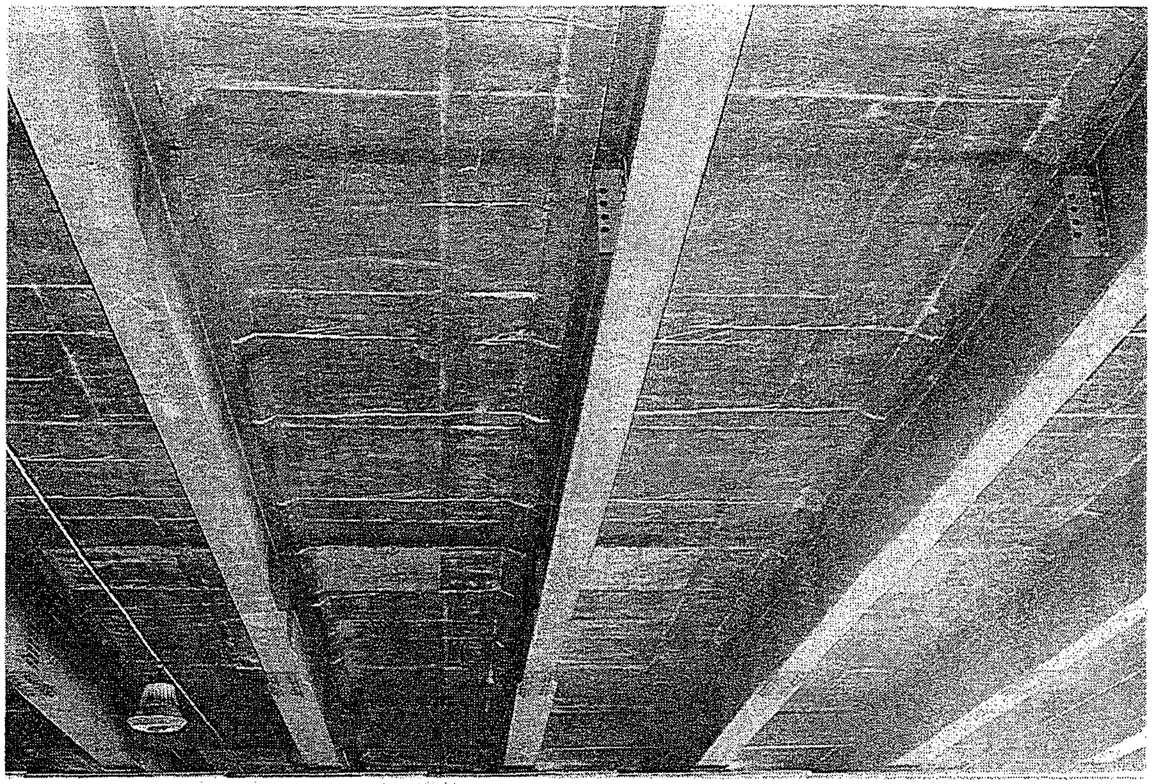


Fig. 4.18. Underside of I-65 Bridge

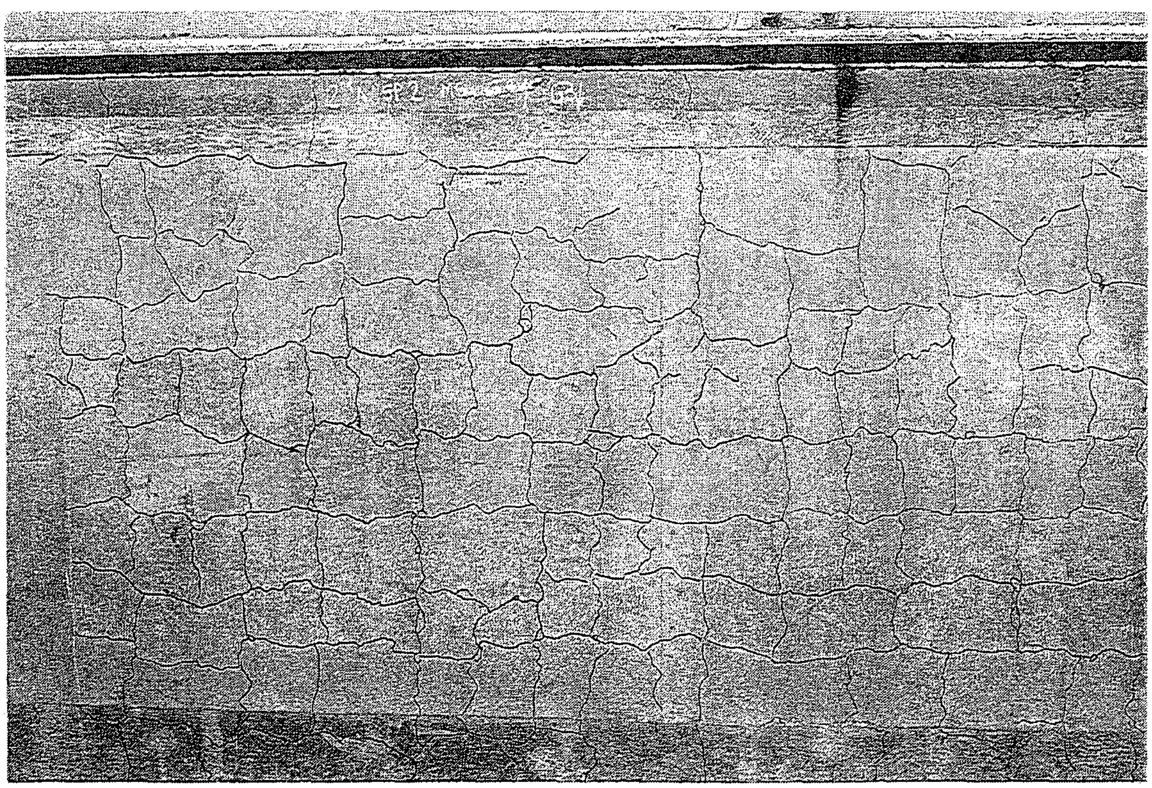


Fig. 4.19. Close-up of Underside of I-65 Bridge at Midspan with Hairline Cracks Highlighted

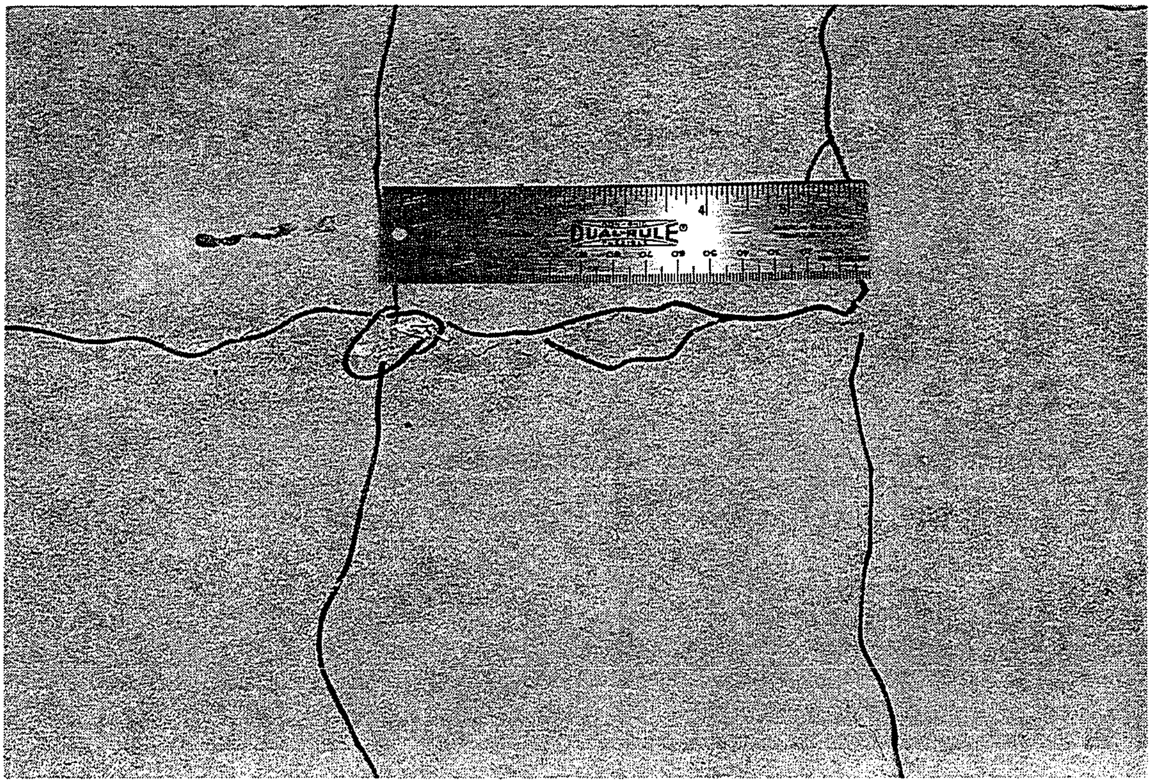


Fig. 4.20. Further Close-up of Underside of I-65 Bridge with Hairline Cracks Highlighted

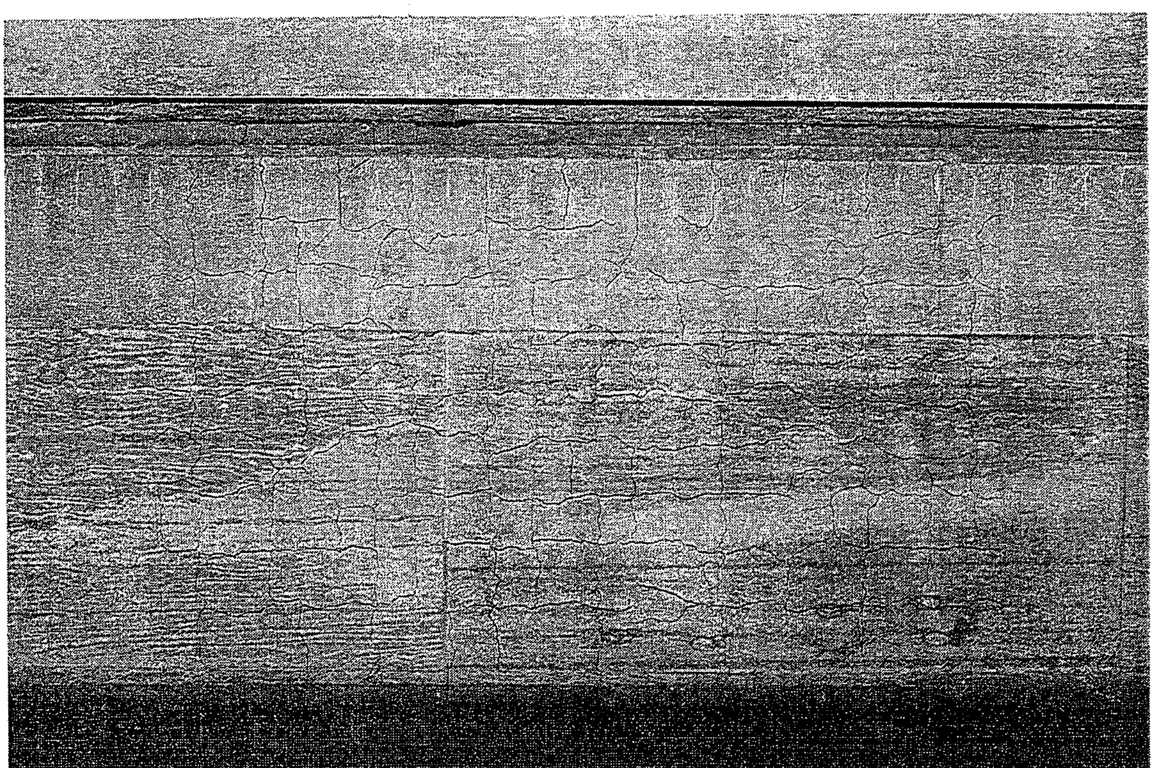


Fig. 4.21. Underside of I-65 Bridge with Hairline Cracks Highlighted

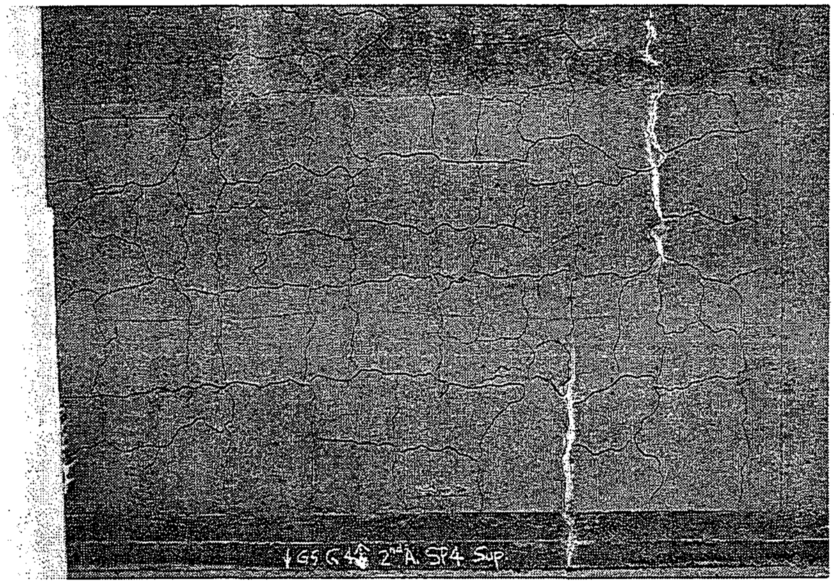


Fig. 4.22. Underside of I-65 Bridge Near a Support Point

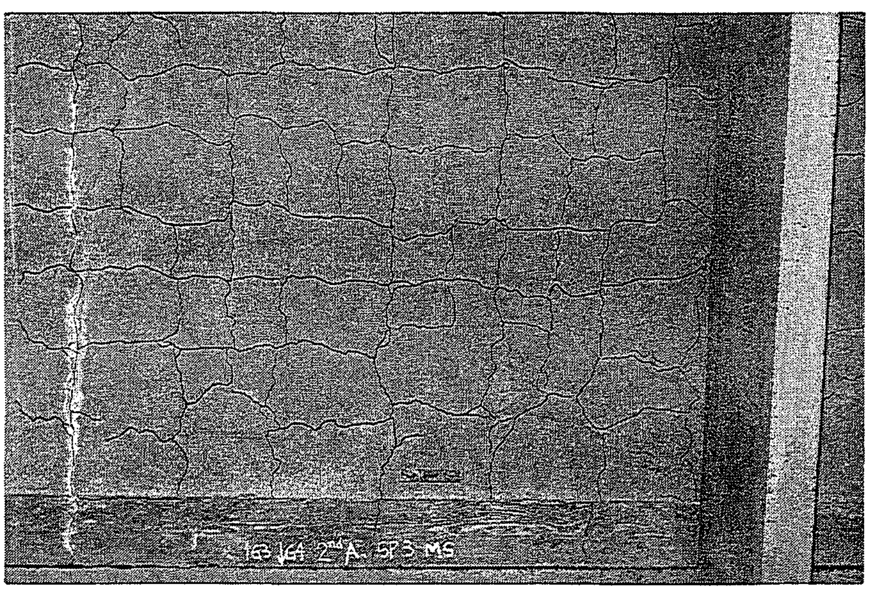


Fig. 4.23. Underside of I-65 Bridge Near Midspan

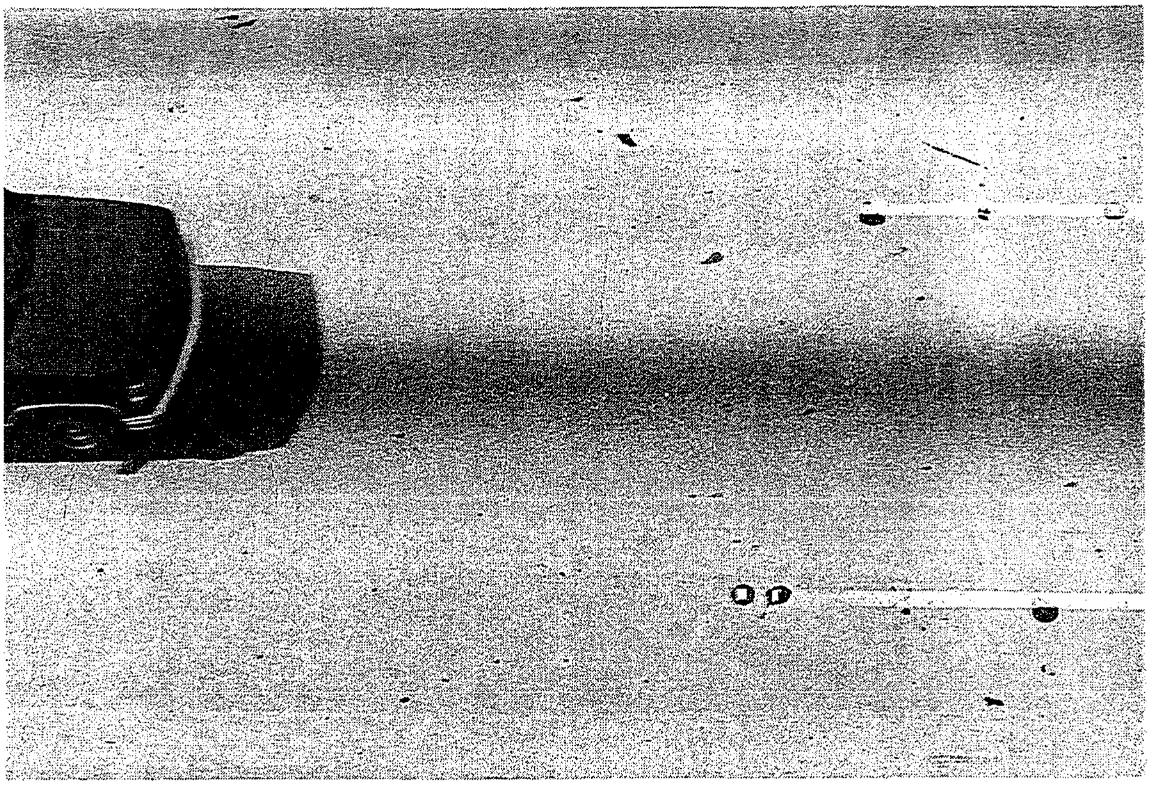


Fig. 4.24. Transverse Cracks in Top of Deck of I-59 Bridge

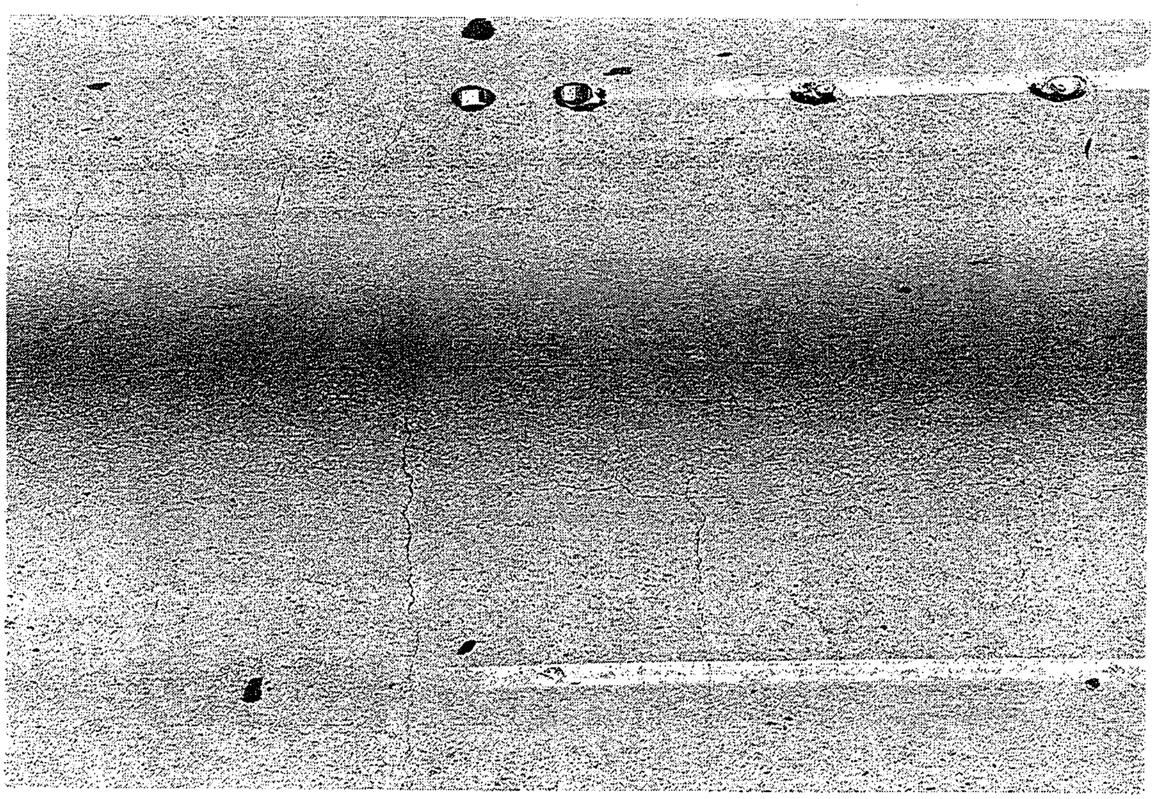


Fig. 4.25. Top of Deck Cracking of I-59 Bridge

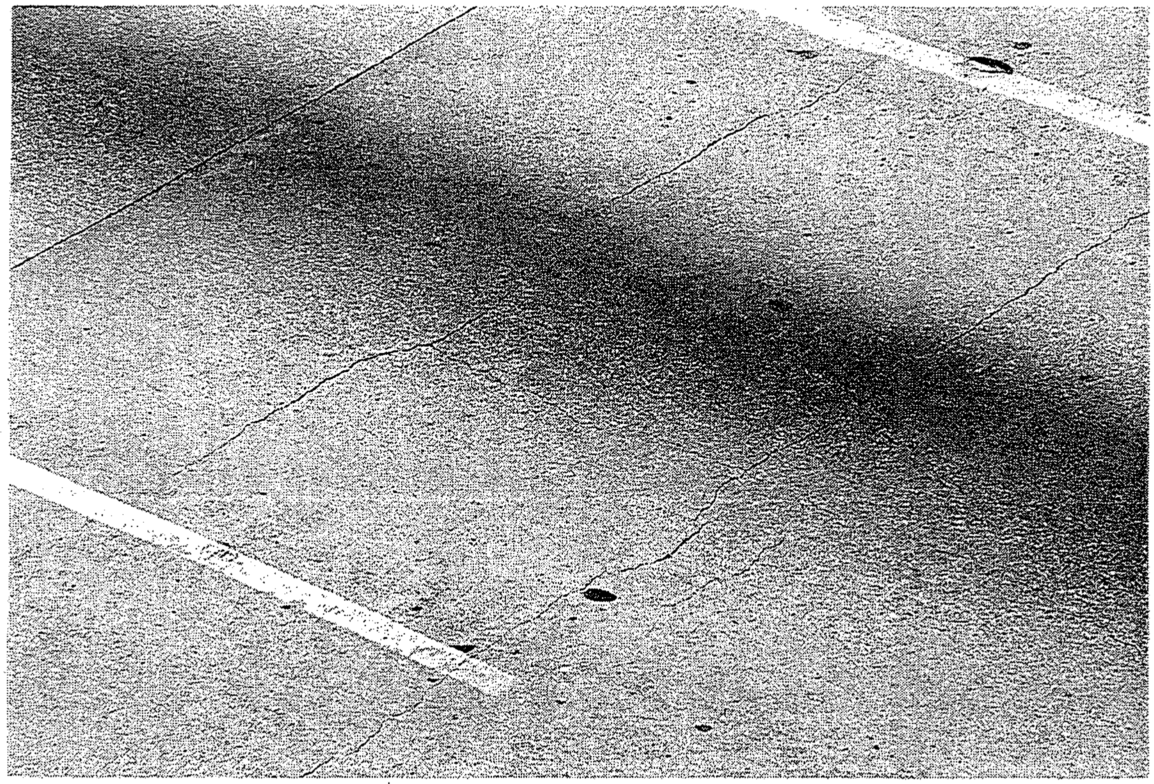


Fig. 4.26. Top of Deck Cracking Near End of Span of I-59 Bridge

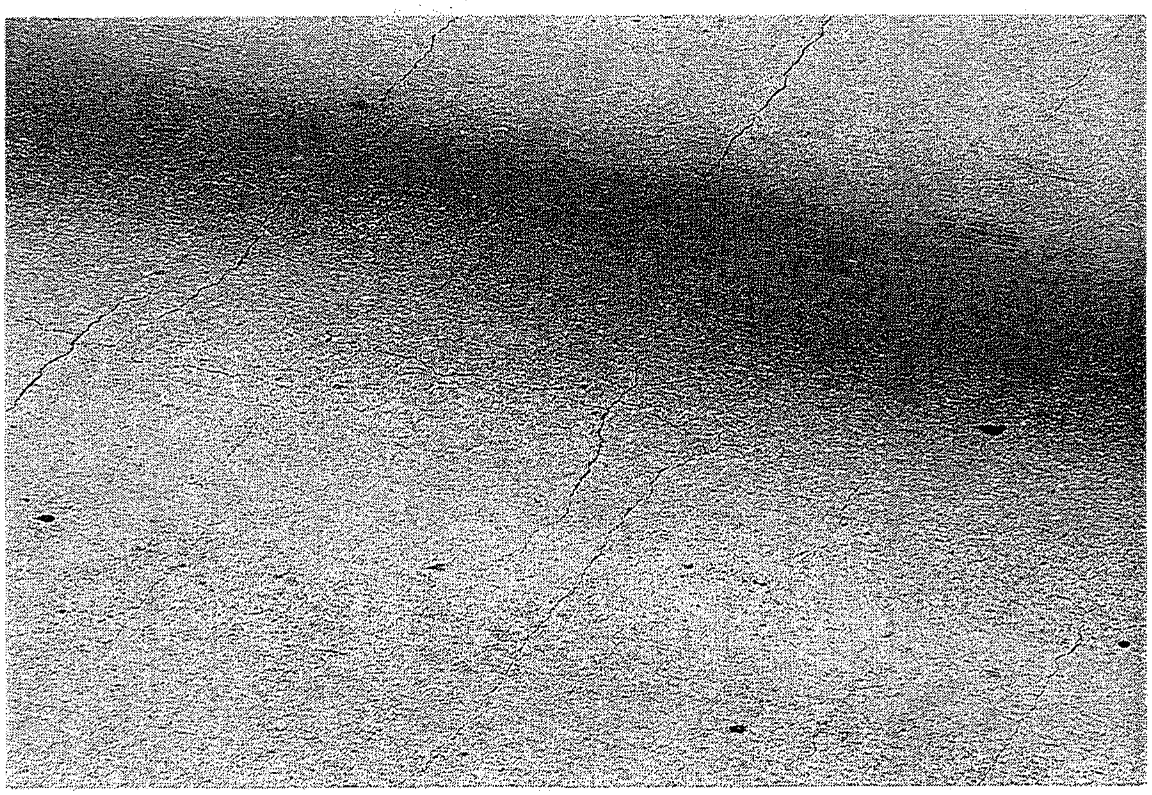


Fig. 4.27. Close-up of Top of Deck Cracking of I-59 Bridge

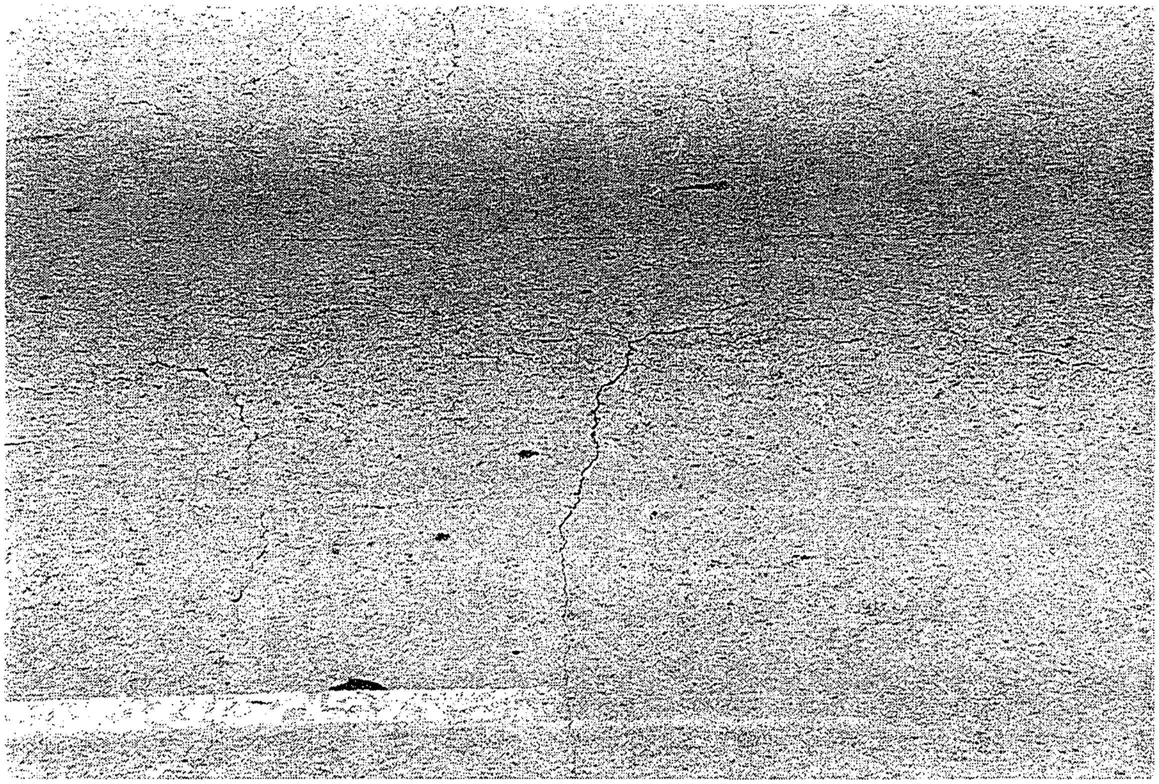


Fig. 4.28. Additional Close-up of Top of Deck Cracking of I-59 Bridge

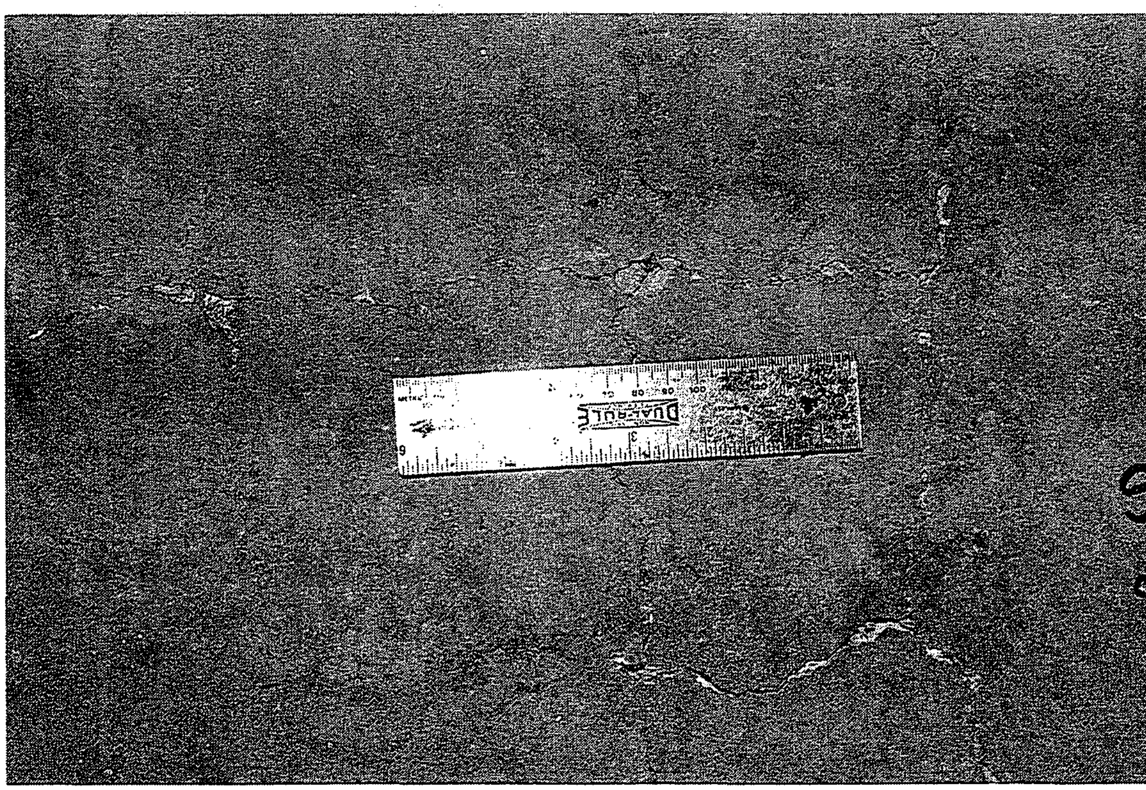


Fig. 4.29. Close-up of Underside of I-59 Bridge Deck with Cracks not Highlighted

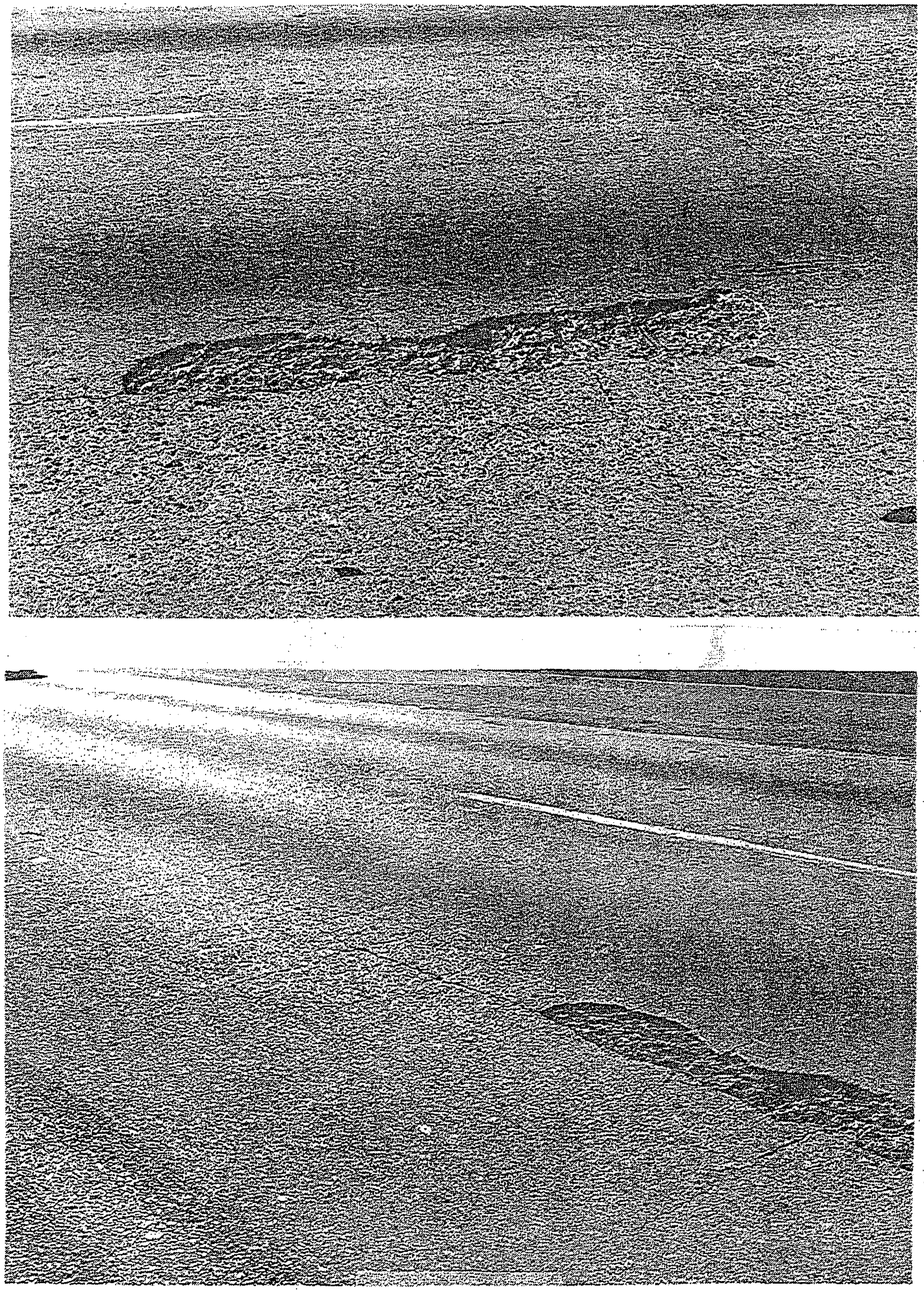


Fig. 4.30. I-65 Deck Surface Spall

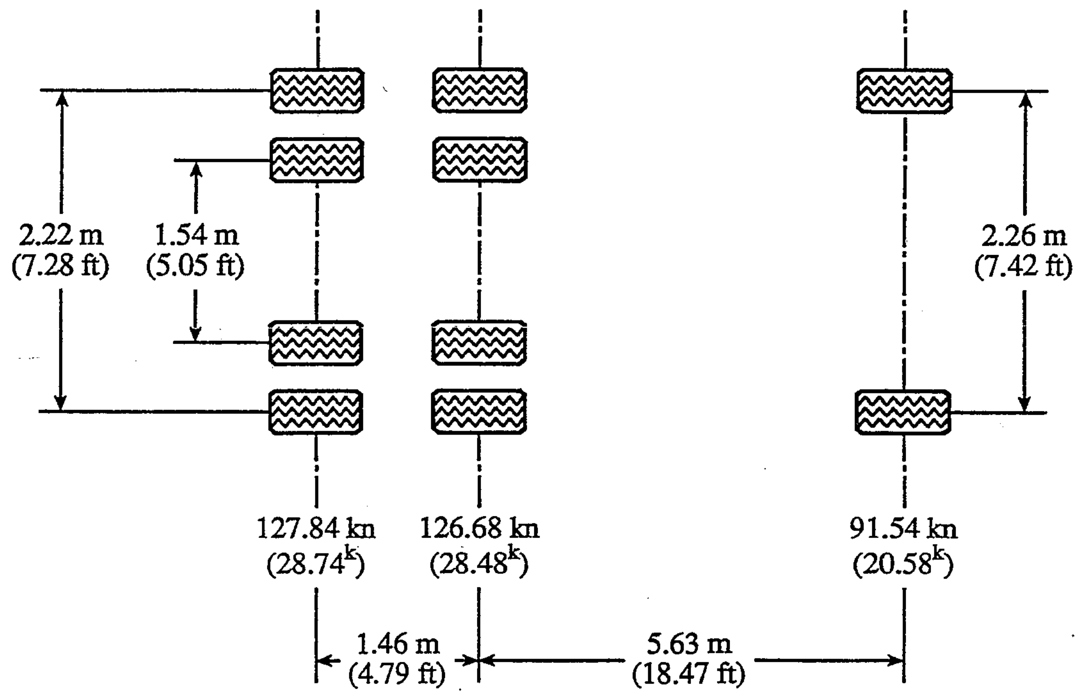


Figure 4.31. Test Truck Axle Configuration and Weights

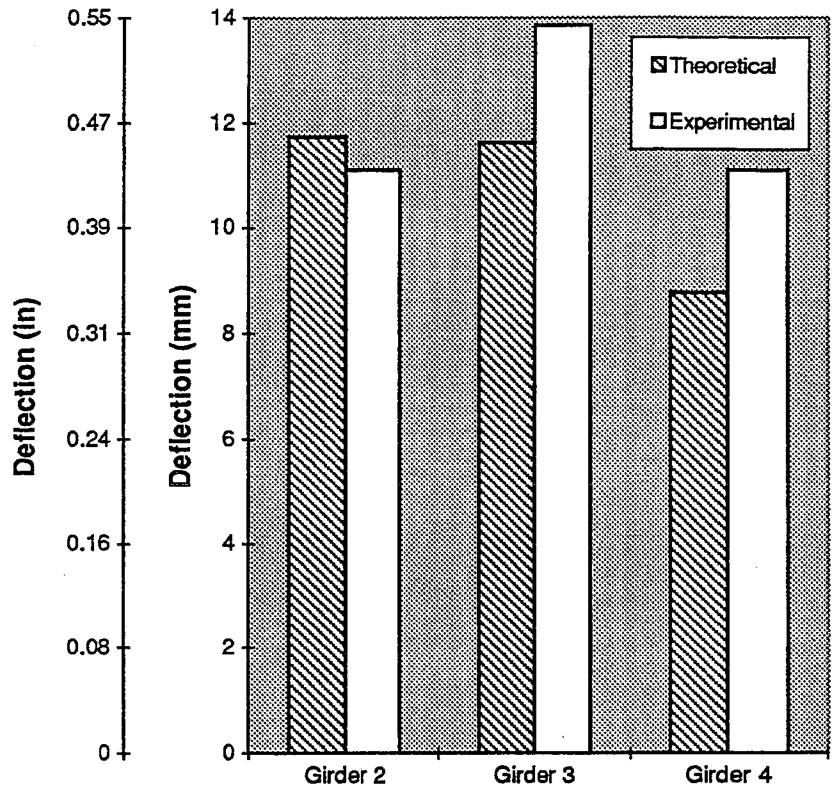


Figure 4.32. Deflections for 24 m Simple Span Bridge

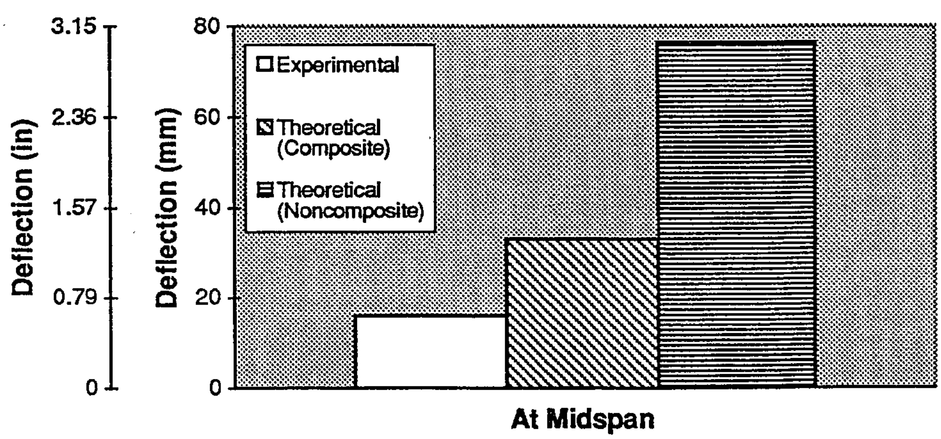


Figure 4.33. Deflections for 76 m Continuous Span Bridge

5. RESULTS OF DECK REPLACEMENT/OVERLAY SURVEY

5.1 General

In an effort to identify what other highway agencies in the U.S. and in other countries do in regards to rapid replacement or overlaying of deteriorated bridge decks, the PI prepared a relatively short survey questionnaire. Prior to sending the questionnaire out, it was reviewed by Mr. Fred Conway, Bridge Bureau Chief, and Mr. Mitch Kilpatrick, Maintenance Bureau Chief of the ALDOT, and their recommendations were incorporated.

The questionnaire focused on three things, i.e.,

1. information pertaining to rapid replacement of bridge decks
2. information pertaining to rapid overlaying of bridge decks
3. information pertaining to overlaying of severely cracked bridge decks (such as those in Birmingham, AL)

Also, the survey questions were fairly broad with the plan being that once the broad picture was clear, the PI could contact the appropriate states and countries for further details where needed.

5.2 Survey of State DOTs

A copy of the survey questionnaire and transmittal letter are shown in Appendix A. They were sent to the Bridge Bureau Chiefs in each of the 50 states during the first week of August 1997, with a requested return date of September 15, 1997. Forty-one (41) states participated in the survey for an 82 percent response. Results of the State DOT survey are presented in Section 5.4.

5.3 Survey of Other Countries

The survey questionnaire and transmittal letter sent to other countries was the same as that sent to the State DOTs with minor changes such as replacing the word <u>state</u> with <u>country</u>. These too were air mailed out in the first week in August 1997 with a requested return date of September 15, 1997. At the suggestions of the Organization for Economic Cooperation and Development (OECD) headquarters office in Paris, France, the survey questionnaire was sent to the membership of OECD's Steering Committee for Road Transport Research . Represented on this committee are the following countries

Australia	Japan
Austria	Luxembourg
Belgium	Mexico
Canada	Norway
Czech Republic	New Zealand
Denmark	Netherlands
Finland	Portugal
France	Spain
Germany	Sweden
Greece	Switzerland
Hungary	Turkey
Iceland	United Kingdom
Italy	United States (questionnaire not sent to U.S.)

Survey participation by the international community was not good (12 of 25 responded with only 8 being able to provide the information requested). However, this was primarily the fault of the PI failing to get the survey form in the "right hands". The members of OECD Road Transport Research Committee were too far removed from the specifics sought in the questionnaire, and this required much rerouting of the questionnaire (in some cases 2 and 3 times). As a result, many of the questionnaires probably never reached those with the expertise needed to prepare a response. Results of the international community survey are presented in Section 5.5.

5.4 Results of State DOT Survey

As indicated earlier, 82 percent of the states participated in the survey and were very gracious in sharing their experiences and expertise on rapid bridge deck replacement and overlaying. The PI contacted all survey responders with follow-up questions and clarifications where needed or felt to be potentially beneficial. The State DOT responses to the survey questionnaire are summarized in Tables 5.1 and 5.2 and Figures 5.1 - 5.5.

Table 5.1 provides a rather comprehensive summary regarding the employment of rapid bridge deck replacements, overlaying, overlaying on badly cracked decks, and approximate service life on a state-by-state basis. Figure 5.1 summarizes the state responses to the three primary survey questions. This figure reflects that for the participating states, about half have executed rapid deck replacements in a staged construction and concurrent traffic setting, and about half have not. The figure also reflects that almost all states (80% of those participating) have executed rapid deck overlaying, and that most states (56%) have employed overlays on badly cracked decks.

Figure 5.2 reflects that all states which have performed rapid deck replacements in a staged construction and concurrent traffic setting have done so using cast-in-place (CIP) reinforced concrete, while in addition, 15 percent of the states have also done the replacements using precast and prestressed concrete panels with a CIP topping. Other deck replacement systems have been close to nil.

Figure 5.3 reflects that the deck overlay materials of choice in the past are

- latex modified concrete (LMC)
- low slump dense concrete (LSDC)
- micro silica modified concrete (MSMC)

In discussions with the state DOTs it appears that overlays of both LMC and LSDC are decreasing in use, and MSMC is increasing in use (this may be do in part because of FHWA incentives in that direction). It should be noted that based on number of states utilizing the material, the polymer concrete overlays hold a small percentage of the "overlay market"; however, because of Caltran's sole and extensive use of polyester polymer concrete overlays, polymer concrete would probably share a significant percent of the total overlay square footage currently in-place on bridge decks. Also, it should be noted in Figure 5.3 that asphalt with water-proofing membranes share a significant percentage of the "overlay market". The percentages shown in Figure 5.3 do not sum to 100% as many state use several types of overlay material.

Figure 5.4 reflects results similar to that of Figure 5.3 for employment of overlays on badly cracked decks, and the same comments are applicable. However, note in Figure 5.4 the fairly high percentage of asphalt (without membrane). This is mostly result of states providing a "temporary fix" until the deck can be replaced. The percentages in Figure 5.4 do not sum to 100% for the same reason as stated earlier.

Figure 5.5 summarizes the approximate years of service life of the overlay materials being used by the participating states. In this figure, the shaded bars represent the range of reported service life and the large "target" dots are the approximate mean service life. The figure reflects a very good service life for LSDC overlays, a good service life for LMC, a very good service life for PPC, and a fair service life for asphalt with a waterproofing membrane. The service life performances of the RSLMC and MSMC can not be assessed at this time as these overlays have typically only been in service from 0-7 years. Note in Figure 5.5 the large range of service life for LMC overlays. Some states are not too pleased with their LMC overlays and others feel that it is the best overlay material on the market. The New Jersey DOT, which is most pleased with its LMC overlay, indicated that they had only experienced bond failures with LMC when the temperature at the time of placement was out of specifications, or when the overlay is placed on certain proprietary patching materials to which it will not bond.

Table 5.2 summarizes the work time schedules typically employed by the participating states when executing a rapid deck replacement or overlay. Note for the deck replacement, a normal weekday work schedule is reported as being most widely used (with one state reporting the use of 2 daylight shifts), and an extended number of days in the work week being the second most widely used. However, for overlay work, working at nights and on weekends are the predominate work time schedules. The significant percent of overlaying during normal week day work schedules are most probably for those cases where traffic is lighter and closing of a lane during normal work days does not present a significant problem. Again, the percentages in this table do not sum to 100% as many states use more than one of the work time schedules.

Table 5.1. Summary of Survey Questionnaire Responses from State DOTs on Rapid Bridge Deck Replacement or Overlaying Under Concurrent Traffic Conditions

STATE	Have Executed Rapid Deck Replacement	Type Deck Replacement Employed	Have Executed Rapid Deck Overlaying	Type and Thickness of Overlay Employed	Have Executed Overlaying of Badly Cracked Decks	Type and Thickness of Overlay Employed	COMMENTS
Alaska	No	--	Yes	1½" micro-silica modified concrete (MSMC)	No	--	Overlays usually placed at night, 8:00 p.m. - 4:00 am. Overlays are relatively new (around 5 years old) and performing well.
Arizona	No	--	No	--	No	--	--
Arkansas	No	--	Yes	Dense concrete (LSDC)	No	--	Performances of dense concrete overlays have varied from good to below average. Usually replace deck rather than overlay.
California	No	--	Yes	3/4" polyester polymer concrete (PPC)(may increase thickness to 3") 3"- 8" portland cement concrete	Yes	3/4" polyester polymer concrete (PPC) (may increase thickness to 3")	Use PPC almost solely for overlays of thicknesses ¾" - 3". Use a methacrylate primer for bond enhancement and crack sealing/healing. Have achieved 13 year and counting service life with PPC. Have achieved 20 + years of service life with portland cement concrete.
Connecticut	Yes	CIP RC Have used some precast concrete deck slabs	Yes	Membrane waterproofing with 2½" bituminous concrete overlay. Have used a small amount of LMC overlays.	No	--	Overlay work is usually restricted to week nights between 10:00 p.m. - 6:00 am. 8 - 10 year service life on bituminous concrete overlays.
Colorado	Yes	CIP RC	Yes	2" Asphalt	No	--	Achieving 30 year service life with CIP deck replacements. Achieving 20 year service life with 2" asphalt overlays with membranes. In high traffic areas, asphalt requires a lot of maintenance. Have used silica fume concrete in high traffic areas for last 5 years and they are performing well.

Table 5.1 Summary of Survey Questionnaire Responses from State DOTs on Rapid Bridge Deck Replacement or Overlaying Under Concurrent Traffic Conditions (Cont'd.)

STATE	Have Executed Rapid Deck Replacement	Type Deck Replacement Employed	Have Executed Rapid Deck Overlaying	Type and Thickness of Overlay Employed	Have Executed Overlaying of Badly Cracked Decks	Type and Thickness of Overlay Employed	COMMENTS
Delaware	Yes	CIP-RC with SIP metal forms	Yes	1½" - 2" LMC	No	--	20-25 year service life on LMC overlays
Florida	No	--	No	--	No	--	--
Georgia	No	--	Yes	Fast setting Type III cement as a structural overlay (extends below top mat a minimum of ¾").	No	--	Have used 2½" - 8" asphalt overlays, ½" methyl methacrylate overlays (3 years and counting), polymer concrete overlay (5 years and counting). Have also used structural overlays on badly cracked decks (6 years and counting).
Hawaii	No	--	No	--	No	--	--
Iowa	No	--	Yes	1½" dense concrete (require about 7 days of moist curing)	Yes	1½" dense concrete Membrane with 2" asphalt	20 years and counting service life with dense concrete overlays unsatisfactory service life with asphalt overlay.
Idaho	No	--	No	--	Yes	1½" micro-silica modified concrete, 1½" LMC, LSDC	20 years and counting service life with LMC. Have only been using micro silica for about 5 years. Had some application problems with LSDC about 20 years ago, and stopped using and went to LMC
Illinois	Yes	CIP RC (with epoxy coated rebar)	Yes	2-2¼ " concrete • •Dense concrete • •Micro silica concrete • •LMC 2-2 ¼" Bituminous ¼" EPC	No	--	15-20 year service life with concrete overlays. 10-15 year service life with bituminous overlays. Currently evaluating ¼" epoxy polymer concrete overlays.
Kansas	Yes	CIP RC	Yes	1½" silica fume concrete 2¼" dense concrete	Yes	1½" silica fume concrete 2¼" dense concrete	Expect 30 year life for CIP RC decks. Have achieved up to 30 year life with dense concrete overlays. Have used silica fume overlays only in the past 5 years - expect a 30 year life.
Kentucky	No	--	Yes	1¼"-1½" LMC 1¼"-1½" RSLMC	Yes	1¼"-1½" LMC 1¼"-1½" RSLMC	In highly trafficked area, typically achieving 10-12 year service life with LMC overlays. Just began to use RSLMC this year.

Table 5.1 Summary of Survey Questionnaire Responses from State DOTs on Rapid Bridge Deck Replacement or Overlaying Under Concurrent Traffic Conditions (Cont'd.)

STATE	Have Executed Rapid Deck Replacement	Type Deck Replacement Employed	Have Executed Rapid Deck Overlaying	Type and Thickness of Overlay Employed	Have Executed Overlaying of Badly Cracked Decks	Type and Thickness of Overlay Employed	COMMENTS
Louisiana	No	--	Yes	1½" high density portland cement concrete and silica fume concrete	No	--	Have only begun to use silica fume concrete overlays (within last 6 months). These overlays require 3-4 day curing.
Maryland	Yes	CIP RC with epoxy coated rebar (25 year life w/o epoxy coated rebar and 25 years and counting with epoxy bars)	Yes	1¼" LMC	Yes (If deck concrete is sound and has low chloride content)	1¼" LMC	15 years and counting service life on LMC overlays. Have used 2½" low slump concrete overlays with unsatisfactory results (approximately 18 year service life).
Michigan	No	--	No	--	Yes	Bituminous concrete	Bituminous concrete overlay is considered temporary until deck can be replaced.
Minnesota	Yes	CIP RC	Yes	2" LSDC 1½" LMC	No (Not if looking for long service life)	--	Typical service life of original RC decks are around 25 years. Typical service life with either the LSDC or LMC is 20-25 years. Have used 3/4" latex mortar overlays (10 year service life) and 3" bituminous overlay with membrane (10 year service life).
Missouri	Yes	3" prestressed panels with a 5½" RC slab integral with panels on top (less expensive) CIP RC with SIP forms	Yes	1¼" EPC 2" concrete • LMC • LSDC • Silica Fume 2" Asphalt	No	--	Deck replacements are accomplished by lane closures with longitudinal construction joints. Use EPC for special applications. Look for 10 + years from concrete overlays. Look for 7-10 years from asphalt overlays.
Mississippi	No	--	No	--	Yes	1/4"-3/8" Resurf II	Resurf II gave satisfactory performance for a period of only 6-18 months.

Table 5.1 Summary of Survey Questionnaire Responses from State DOTs on Rapid Bridge Deck Replacement or Overlaying Under Concurrent Traffic Conditions (Cont'd.)

STATE	Have Executed Rapid Deck Replacement	Type Deck Replacement Employed	Have Executed Rapid Deck Overlaying	Type and Thickness of Overlay Employed	Have Executed Overlaying of Badly Cracked Decks	Type and Thickness of Overlay Employed	COMMENTS
North Carolina	Yes	CIP RC with SIP forms 3½" prestress panels with a RC topping (presoak panels prior to placing CIP concrete).	Yes	1¼" LMC 1¼" RSLMC	Yes	1¼" LMC	LMC overlays have been used by NCDOT for approximately 8-10 years and are performing satisfactorily. RSLMC has only been recently used for one urban setting.
North Dakota	Yes	CIP RC	Yes	1½"-2" low slump concrete (Iowa mix)	Yes	1½"-2" low slump concrete (Iowa mix)	Have achieved 25 year service life with low slump concrete overlays. Life varies with traffic and salt application conditions. Service life approximately 20 years for adverse conditions and 25 years and counting for less severe conditions.
Nebraska	No	--	No	--	Yes	2" high density concrete (LSDC) 2" silica fume concrete	Have achieved service life of 10 years and counting with dense concrete. Had problems placing and curing silica fume overlays-they have only been in place for about 2 years. Also had cracking 1 year after placement which they attribute to autogenous shrinkage.
New Hampshire	Yes	CIP RC Precast/prestressed concrete panels	Yes	1¼" LMC 1¼" high density concrete Micro silica concrete	No	--	Have stopped using LMC overlay because of cost. Achieving about 20 years service life with LMC and dense concrete. Oldest micro silica concrete are only 5 years old.
New Jersey	Yes	CIP RC	Yes	1¼" LMC Membrane waterproofing with asphalt	Yes	1¼" LMC	Achieving service life of approximately 25 years with RC replacement decks. Achieving 20-30 year service life with LMC with an average of around 25 years. They do not consider our Birmingham decks as badly cracked

Table 5.1 Summary of Survey Questionnaire Responses from State DOTs on Rapid Bridge Deck Replacement or Overlaying Under Concurrent Traffic Conditions (Cont'd.)

STATE	Have Executed Rapid Deck Replacement	Type Deck Replacement Employed	Have Executed Rapid Deck Overlaying	Type and Thickness of Overlay Employed	Have Executed Overlaying of Badly Cracked Decks	Type and Thickness of Overlay Employed	COMMENTS
New Mexico	Yes	CIP RC with SIP forms	Yes	2" LMC	Yes	2" LMC	Achieving 15-25 years with deck replacements. Achieving 10-15 years with LMC overlays.
New York	Yes	Exodermic (designed by manufacturer) Superstructure replacement using ≈ 10' wide inverset panels. CIP early strength RC	Yes	6 ½" early strength concrete (with rebars) 2 ½" asphalt (8-10 years) 1½"-2" LMC 1½ "-2" MS Concrete 3"-3 ½" High density MS concrete (newest)	Yes	Same as for rapid overlaying	Exodermic deck are noncomposite with girders and have only been in place for 2 years. Contractor was able to place 6 panels (21' (2 lanes) by 8') each day. 6½" concrete overlays are not performing satisfactorily. Shrinkage and thermal stresses caused cracking and lifting of overlay from existing deck surface. Have a wide range of service lives on LMC (5-25 years) 5 years ago switched to MS overlays. 2 years ago switched to HDMS structural overlays (but not because of any debonding problems). Projecting 40 year life for these overlays.
Nevada	No	--	Yes	3/4" PPC (low cost resin) LSDC (10 year life) LMC (1-3 years)	No	Haven't overlayed, but have used high molecular weight methacrylate to seal cracks.	3/4" polyester polymer concrete is the Caltran PPC overlay material. PPC overlays have 7 years and counting service life. They anticipate a 25 year service life. Had debonding problems with LMC. Had shrinkage cracking problems with LSDC.
Ohio	Yes	CIP RC 3" precast panels with a 5" CIP topping (with rebar) High Performance Concrete (HPC)	Yes	1¼"-3" micro silica, LMC, or dense concrete (Thin overlays seem to perform better).	Yes	Same as for rapid overlaying	Have difficulty with cracking with precast panel deck replacements. Have changed to CIP HPC for deck replacements. Currently using only micro silica or LMC overlays (eliminated dense concrete because other materials are more impermeable). Service life of deck replacements are 20-30 years. Service life of overlays are approximately 15 years.

Table 5.1 Summary of Survey Questionnaire Responses from State DOTs on Rapid Bridge Deck Replacement or Overlaying Under Concurrent Traffic Conditions (Cont'd.)

STATE	Have Executed Rapid Deck Replacement	Type Deck Replacement Employed	Have Executed Rapid Deck Overlaying	Type and Thickness of Overlay Employed	Have Executed Overlaying of Badly Cracked Decks	Type and Thickness of Overlay Employed	COMMENTS
Oklahoma	Yes	CIP RC with 2½" cover, epoxy rebar and sealer	Yes	1½" high density concrete 3" asphalt with membrane	Yes	2" asphalt without membrane (for needed life <5yrs.) 3" asphalt with membrane (for longer service life)	Have achieved service life of 22 years and counting with high density concrete overlays. Have achieved service life of 22 years and counting with CIP RC decks.
Pennsylvania	No	--	Yes	Silica fume concrete 1½" LMC	No	--	Silica-fume concrete overlays with 1 day strength of 4000 psi have only been in place for 3 years. These are structural overlays and are performing well. LMC is their standard bonded overlay material and is providing a service life of 15-20 years. Longitudinal construction joints seem to be the problem area.
Rhode Island	Yes	CIP RC	Yes	3" Type I cement 3" silica fume concrete 3" asphalt	No	--	Achieving about 15 year service life on asphalt overlays. New overlays (past 2-3 years) are all silica fume and performing well.
South Dakota	Yes	CIP RC	Yes	2" low slump dense concrete (LSDC) 1½" LMC	Yes	2" low slump dense concrete (LSDC)	20 years and counting on service life of RC deck replacements. 20-25 years on LSDC and LMC overlays. Appear to have changed from LMC to LSDC overlays. This spring they used a LSDC with polyprofiline fibers on some badly deteriorated decks.
Texas	Yes	CIP RC on precast prestressed panel for a total deck thickness of 8" (standard)	Yes	2" dense concrete (LSDC)	Yes	2" concrete 2" dense concrete	15-20 year service life on overlays. 10-15 year service life on overlays on badly cracked decks.
Utah	No	--	Yes	3" Asphalt over water-proofing membrane	Yes	3" Asphalt over waterproofing membrane	5-7 year service life on asphalt overlay.

Table 5.1 Summary of Survey Questionnaire Responses from State DOTs on Rapid Bridge Deck Replacement or Overlaying Under Concurrent Traffic Conditions (Cont'd.)

STATE	Have Executed Rapid Deck Replacement	Type Deck Replacement Employed	Have Executed Rapid Deck Overlaying	Type and Thickness of Overlay Employed	Have Executed Overlaying of Badly Cracked Decks	Type and Thickness of Overlay Employed	COMMENTS
Virginia	Yes	CIP RC	Yes	1½" LMC 1½" Silica fume concrete Thin (≈3/8") polymer concrete RSLMC	No	--	Achieving around 20 year service life from LMC. Currently using silica fume (past 5 years) and it is performing well. Any place where have a free edge (e.g. transverse and longitudinal joints) it will begin to unravel with time. They suggest going down to top mat of steel at these locations. Thin PC overlays have not performed well. One $3,000,000 project has all come up in 3-4 years. The best performance has been a 9 year life with a rigid EPC.
Vermont	No	--	No	--	Yes	2" Type I concrete Bituminous overlays	Achieving service life of 10 years and counting on Type I concrete overlays
Washington	Yes	CIP RC Have designed a prestressed system using the new Nebraska U. panels but have not yet built it	Yes	1½" LMC 1½" MSMC 3/8" EPC 1.8" Asphalt with membrane	Yes	1½" LMC 1½" MSMC 3/8" EPC 1.8" Asphalt with membrane	Approximately 10 year service life with asphalt overlay, 20 years with LMC, 5 years with EPC, and too early to say with MSMC.
Wisconsin	Yes	CIP RC (average life of existing decks is around 35 years)	Yes	2" Iowa mix (low slump dense concrete) 2" micro silica concrete (relatively new, 3-4 years) 2" + Asphalt with membrane	Yes	2" Asphalt	Typical service life with Iowa mix overlays is about 18 years. 5 year service life on temporary asphalt overlays. 15-20 year service life with asphalt overlays with membrane.
Wyoming	No	--	Yes	1¼"-Full Depth LMC 1¼" -Full Depth Silica fume modified concrete	Yes	1¼" LMC or silica fume 3/8" methacrylate and aggregate (MPC)	Approximately 15 year life on LMC overlays. 10 years and counting life on silica fume modified concrete. For decks with just cracking problems, they use thin methacrylate overlay. For spalling and delam decks, they use 1¼" LMC or silica fume concrete. 10 year and counting life on thin MPC overlays.

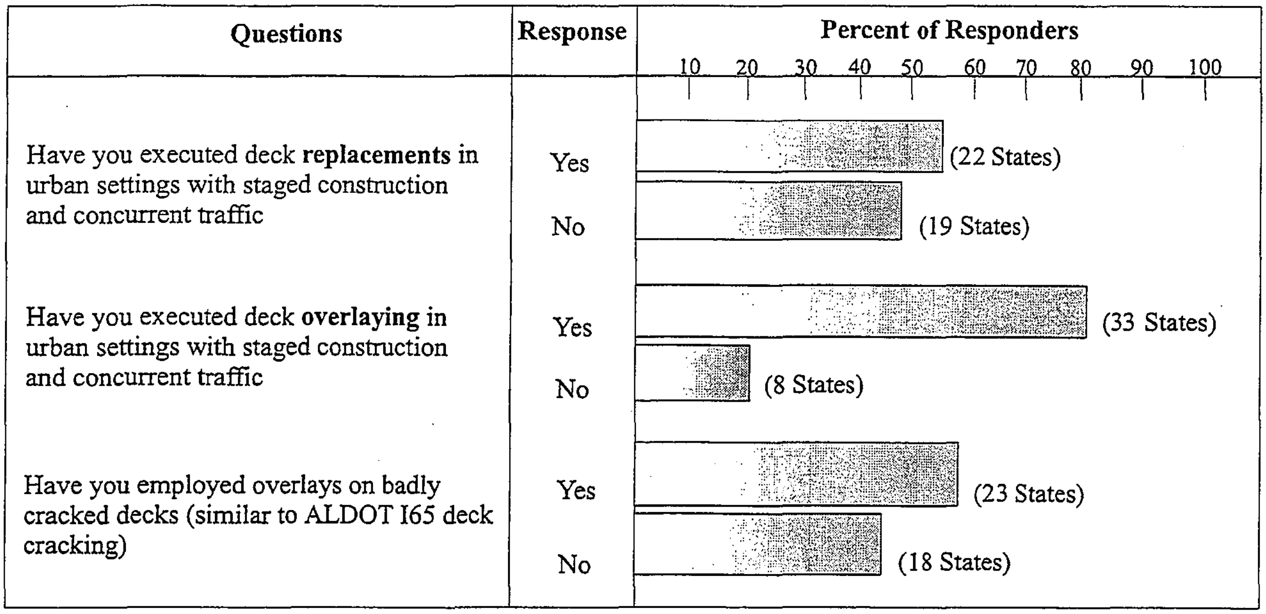
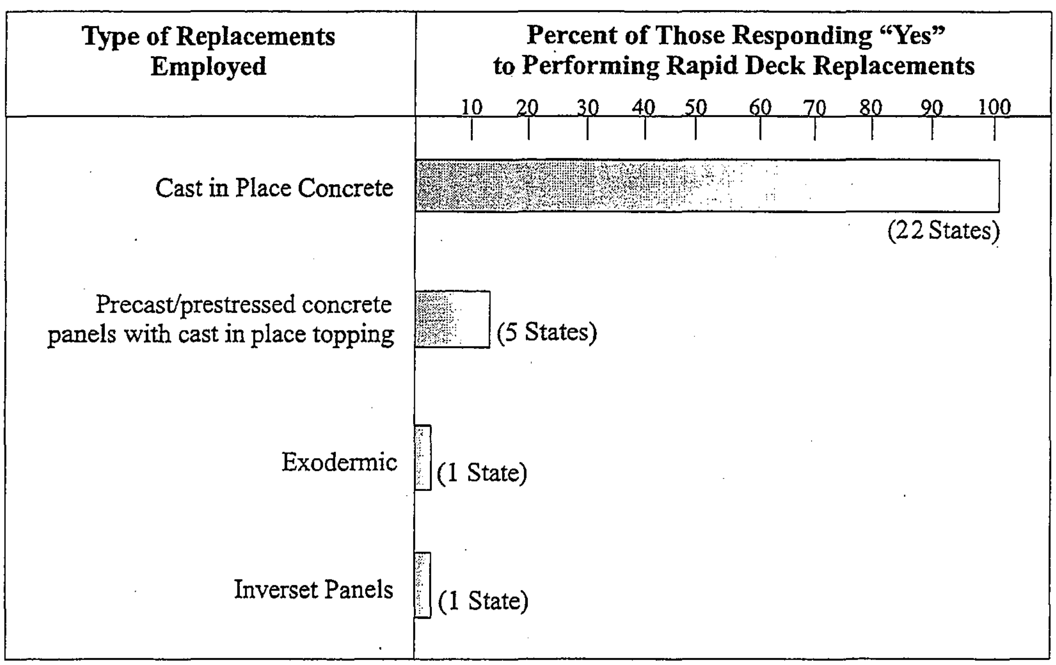


Figure 5.1. Summary of State DOT Responses to Primary Questions on Survey Questionnaire



Figures 5.2. Summary of Type of Deck Replacements Employed by Other States in Urban Setting with Staged Construction and Concurrent Traffic

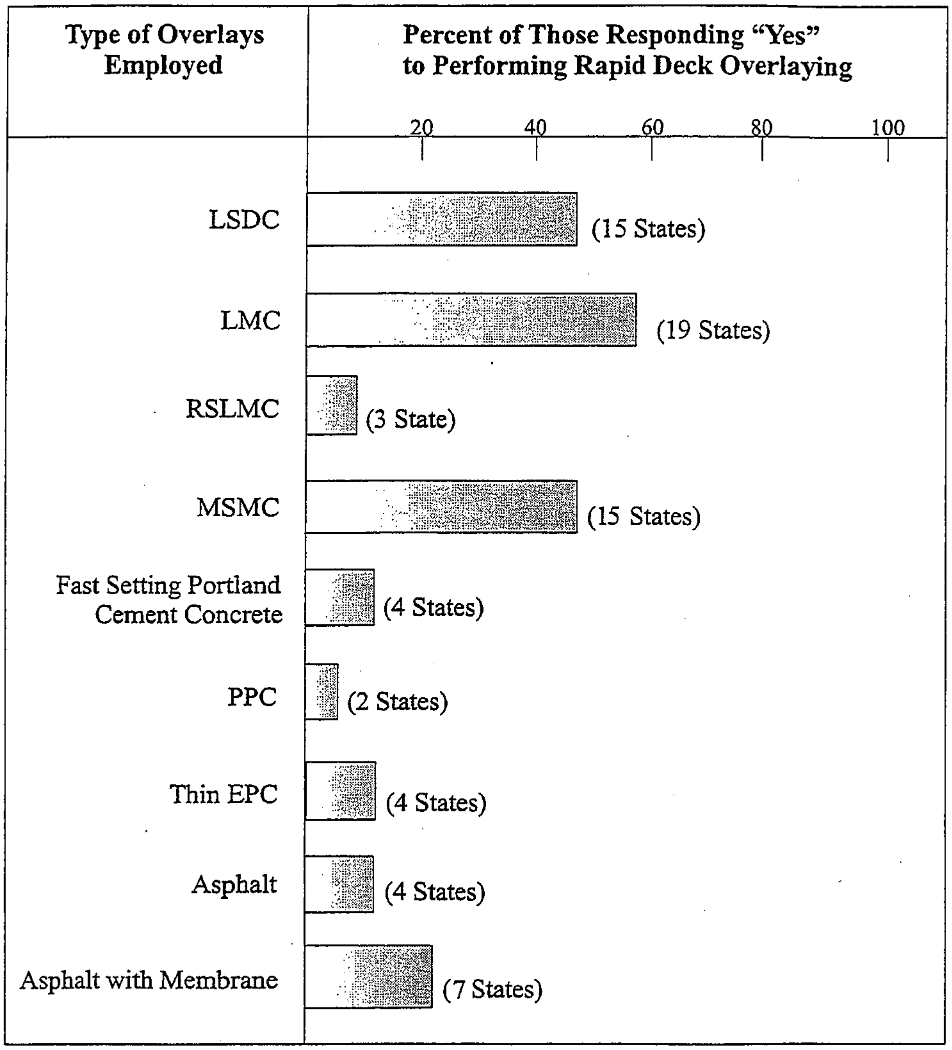


Figure 5.3. Summary of Type of Deck Overlays Employed by Other States in Urban Settings with Staged Construction and Concurrent Traffic

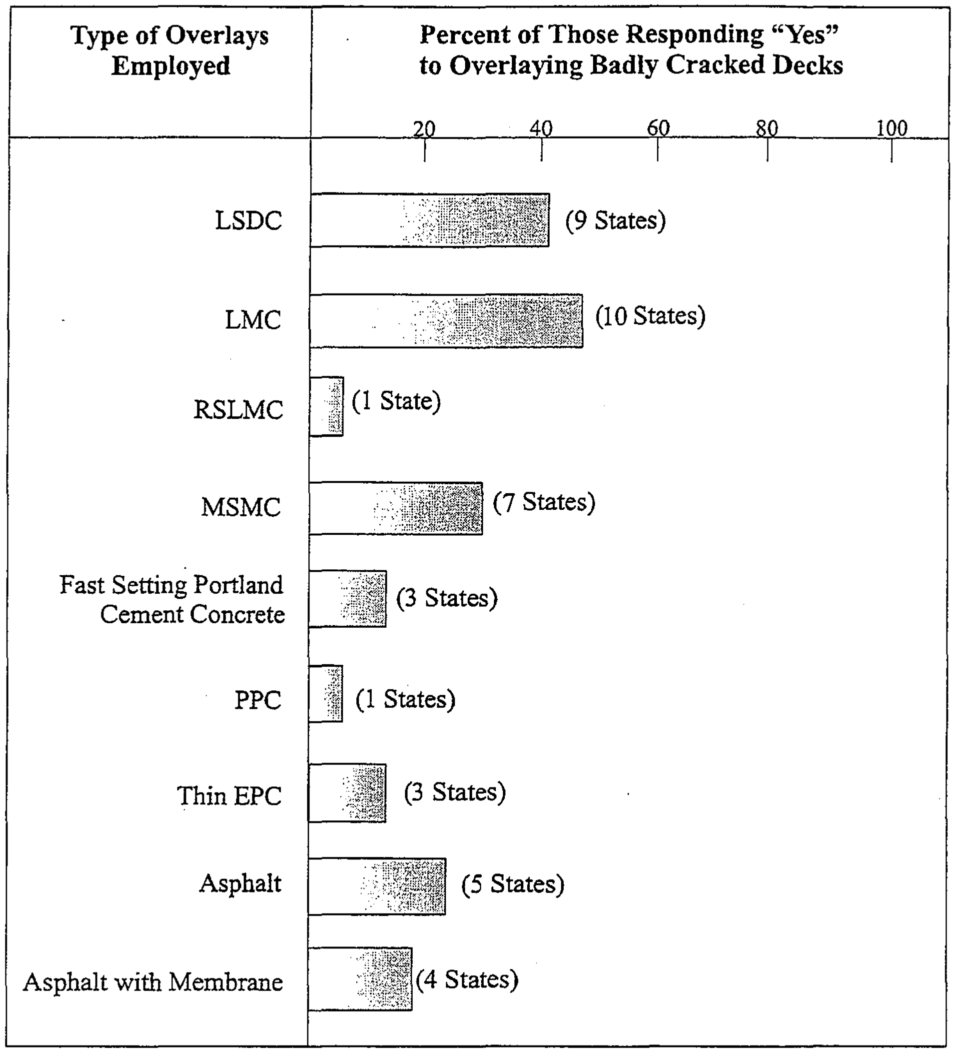


Figure 5.4. Summary of Type of Deck Overlays Employed by Other States on Badly Cracked Decks

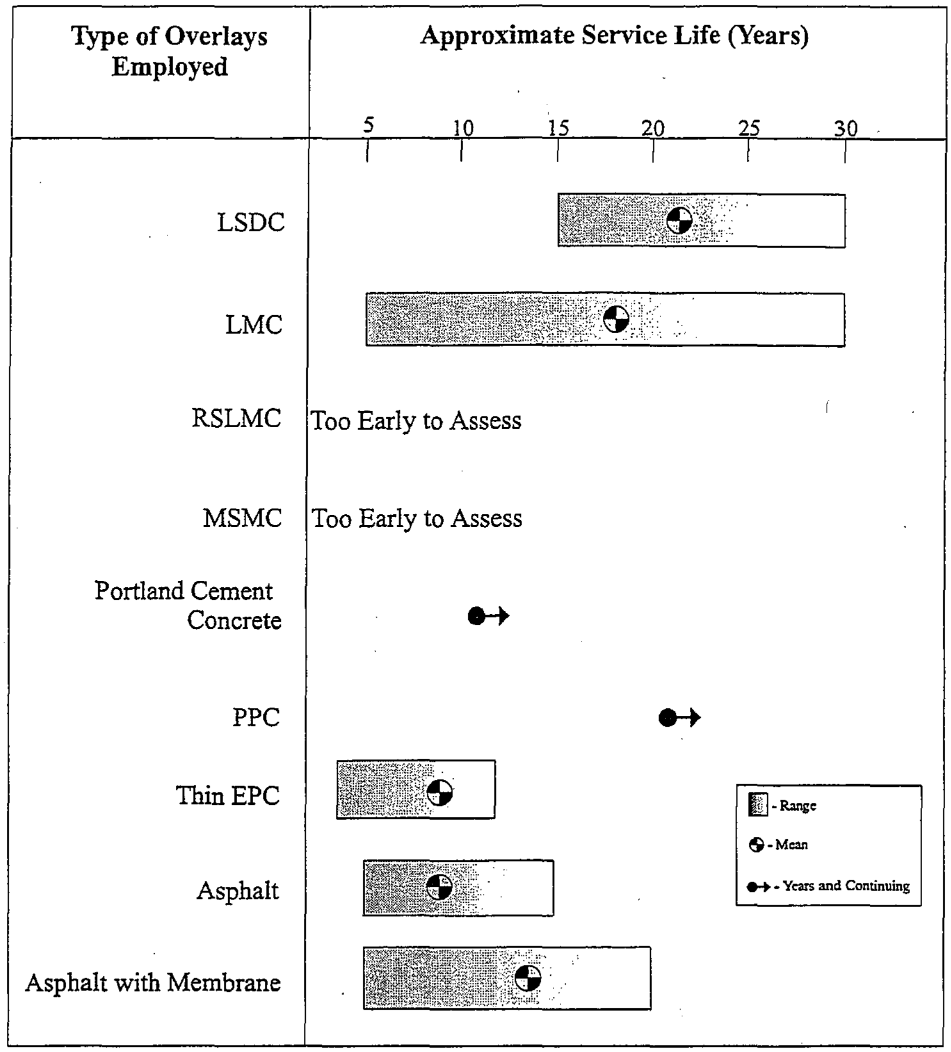


Figure 5.5. Summary of Approximate Service Life of Bridge Deck Overlays

Table 5.2. Summary of Work Time Schedules for Rapid Deck Replacements and Overlaying

Work Time Schedule	Rapid Deck Replacement	Rapid Deck Overlay
Nights (generally 8 p.m.-5 a.m.)	3 states (14%)*	13 states (41%)**
Weekends (generally 8 p.m. Friday - 5 a.m. Monday)	2 states (10%)	12 states (38%)
Modified Weekday (generally 9 a.m. -3 p.m.)	2 states (10%)	1 state (3%)
Normal Weekday	7 states (33%)	10 states (31%)
Extended Week (generally 7-days a week)	4 states (19%)	2 states (6%)
Varies Depending on Location and ADT	4 states (19%)	3 states (9%)
No Response	4 states (19%)	4 states (12%)

*Percentage of those responding "Yes" to performing rapid deck replacements.
**Percentage of those responding "Yes" to performing rapid deck overlaying.

5.5 Results of International Survey

Unfortunately completed survey information was received from only eight other countries. As explained earlier this appears to be the result of the survey questionnaire not getting to the "right people" in many of the cases rather than an unwillingness to cooperated and share expertise on the part of the international community. In addition to the eight participating countries whose input will be summarized below, four other countries, i.e.,

- Australia
- Belgium
- Canada
- Sweden

provided a response indicating contacts to make to gather the requested information. Due to a lack of time, the follow-up contacts were not made.

Table 5.3 provides a summary on employment of rapid bridge deck replacements, overlaying, and approximate service life on a country-by-country basis. As can be seen in Table 5.3, six of the eight countries have performed rapid deck replacement in a staged construction and concurrent traffic environment with the most common material being CIP reinforced concrete. Six of the participating countries have also performed rapid deck overlaying, with no particular material being the overlay material of choice. Three countries, Greece, Japan, and Mexico have performed rapid deck overlaying on badly cracked decks. Mexico employs a thick (6") portland cement concrete overlay, and Japan employs a thin overlay (of unspecified material) and a lamination of the deck bottom with steel plates or carbon fiber sheets.

Because of the low number of countries participating in the survey, further breakdowns of the responses, analogous to Figures 5.1 - 5.5 and Table 5.2 for the states, were not made. Comparisons within Table 5.3 can be made to identify the most common practices, procedures and materials used in the rapid replacement and overlaying of bridge decks by the participating countries.

Table 5.3. Summary of Survey Questionnaire Responses from Other Countries on Rapid Bridge Deck Replacement or Overlaying Under Concurrent Traffic Conditions.

COUNTRY	Have Executed Rapid Deck Replacement	Type Deck Replacement Employed	Have Executed Rapid Deck Overlaying	Type and Thickness of Overlay Employed	Have Executed Overlaying of Badly Cracked Decks	Type and Thickness of Overlay Employed	COMMENTS
Finland	No	--	Yes	Asphalt with water proofing membrane (≈ 4" thick)	No	--	Service life for asphalt wearing course is 5-10 years on major highways and 10-15 years on other roadways (Note, they use studded tires during winter). Expected service life of waterproofing membrane is 35-45 years.
France	No	--	No	--	No	--	Traffic is not allowed on bridges during concreting work to avoid damage due to vibrations and deflections, and thus achieve a good quality rehabilitation.
Germany	Yes	CIP RS, Prestress Concrete, Compound Steel	Yes	2¾" Asphalt	No	--	Service life of 80 years for replacement decks (same as original decks). Service life of 15-20 years on asphalt overlays.
Greece	Yes	Type not specified 10 year service life	Yes	Type not specified	Yes	Type not specified	Perform rapid deck replacement and overlay work at nights and on weekends.
Japan	Yes	CIP RC Precast/prestressed panels with CIP concrete on top Filled steel grid Steel orthotropic	Yes	5/16"-7/16" thin overlay of steel fiber reinforced concrete with ultra rapid setting cement.	Yes	5/16"-7/16" thin overlay with 3/16" steel plates or carbon fiber sheets on underside.	If deck damage is severe and replacement cannot be employed, they recommend both an overlay and lamination of the deck bottom with carbon fiber sheets.
Mexico	Yes	CIP RC	Yes	6" Portland cement concrete	Yes	6" Portland cement concrete	Achieving 15 year service life with concrete overlays for both light and severely cracked decks.
Switzerland	Yes	Concrete	Yes	5/16"-9/16" concrete	No	--	Achieving 12 year service life with thin concrete overlays. They work two daylight shifts to accelerate deck replacement or overlaying.
United Kingdom	Yes	CIP-RC	No	--	No	--	To minimize damage to concrete, keep traffic as far from work area as possible. They recommend for our Birmingham bridges to •rehabilitate to make deck composite with girders •increase deck thickness to improve robustness (6.5" is too thin for 8' girder spacing).

6. GEORGIA DOT STRUCTURAL BRIDGE DECK OVERLAY CONSTRUCTION SEQUENCE

6.1 General

On the weekend of August 8 - 10, 1997 Dr. Ramey visited the job site of a bridge deck overlay project in Atlanta, Georgia as a guest of the Georgia Department of Transportation (DOT). The bridge was located on 1-285 over the Chattahoochee River and was 844 feet long and 64 feet wide. The primary descriptive parameters for the bridge are shown in Table 6.1.

A structural overlay (as opposed to a bonded overlay) was placed on the bridge after removing the top portion of the existing deck concrete by hydrodemolition to a level of at least 1/2" below the lower bar of the deck upper mat. The overlay went from that level to a minimum of 11/2" above the top bar of the upper mat. If all conditions had been ideal, this would translate into a 3" thick (11/2" cover + 1/2" bar + 1/2" bar + 1/2" clearance) overlay. However, conditions were not ideal and the average thickness of the overlay was probably more like 41/2". The total thickness of the deck was increased by 3/4" from its original thickness via the overlay.

L. C. Whitford Co. Inc., from New York was the contractor for the entire project, and Jet Blasting from New Jersey was the concrete hydrodemolition subcontractor. Whitford was contracted by the Georgia DOT to overlay 2 bridges in the Atlanta area with a total deck surface area of 6702 square yards for a total cost of $2 million. The cost was all inclusive from traffic control to final grooving, and translated into a unit cost of $298 per square yard or $33 per square foot.

Because of the importance of the 1-285 route and the heavy traffic load that it carries, the work had to be done in stages under concurrent traffic conditions. Basically this consisted of restricting the overlay work to weekends only, i.e., from 9:00 p.m. on Fridays until 5:00 a.m. the following Monday. The overlay work is being staged over 10 weekends, and an 11th weekend will be used to finish all deck joint work and to groove the deck. The specific work to be done during each of the 11 stages, (i.e., the 11 weekends), and the traffic control/bridge lane closures to accomplish the work is shown in Figure 6.1.

Table 6.1. Bridge General Descriptive Parameter Values.

Parameter	Value		
Structure No.	Structure ID 121-0242-0 / Location ID 121-00407D-013.78C		
Location	I-285 - South bound lanes on western side of I-285 "loop" over Chattahoochee River		
Year Built	1968; Widened in 1982[a]		
Age	29 years[b]		
Length	844 ft.		
Number of Spans	12		
Span Lengths	46', 57' 3",	3 Span Continuous 88' 6", 110' 0", 88' 6"	66' 9" (5), 67' 0", 53' 0"
Width	64' 4" Gutter to Gutter		
Traffic	1-way		
Number Traffic Lanes	4		
Right Shoulder Width	10.0'		
Left Shoulder Width	6.5'		
Structure Type	Steel continuous multi-girder and steel multi-girder simple spans		
Deck-Girder Design	46' span-noncomposite; all others composite		
Girder Spacing	7' original, widening varies (5' 9" - 7' 6")		
Deck Thickness	7" original design; 8¼" widening		
ADT	174,701 (1995)		
% Trucks	12 (1995)		

[a]Widening in 1982 "closed in middle" of two bridges, adding 4 lanes (2 on each bridge)
[b]As of 1997

The primary construction sequence and tasks for each weekend during the overlaying work were as follows:

1. Marshal supplies and equipment near the site, place the traffic control system, move supplies and equipment onto bridge, and set-up lighting.
2. Remove top 3/4" - 1 " of deck concrete by rotamilling and simultaneously set-up water/slurry containment system for hydrodemolition work.
3. Remove remaining portion of deck concrete to a level of at least 1/2" below the top mat by hydrodemolition.
4. Finish preparing deck for overlaying by jack hammering and hand preparation and repair work.
5. Set rails for and prepare self-driving vibratory screed.
6. Overlay deck with rapid-setting Type III Portland cement concrete mixture.
7. Place quality 24-hour wet curing system.
8. Put away materials and equipment and clear from deck, clean and flush the deck.
9. Remove deck curing system (after 24 hours assuming 3000 psi concrete is achieved) and finish clearing and cleaning the deck.
10. Remove traffic control system and reopen all lanes.

Photographs showing the overlay work in progress during Stage 5 are presented and discussed in the following section.

6.2 Photographic Display and Discussion for Deck Overlay Work

A photographic display of the state of deck cracking prior to overlaying, the overlaying process in progress, and the resulting structural overlay are shown in Figs. 6.2-6.34. Note in Figs. 6.2 and 6.3 that the original deck is in a severe state of transverse cracking. Also, one section of the deck had a significant amount of spalling and delaminations (a section of bad/"dirty" concrete) and concrete patches. This area of bad concrete is one of the reasons GADOT chose to use a structural overlay rather than a bonded overlay, i.e., they did not want to spend a lot of money bonding an overlay to a poor quality bridge deck. It can be noted in Figs. 6.2 and 6.3 that sometime in the past, the GADOT had used a low viscosity methacrylate, or equivalent, crack sealer/healer in an attempt to arrest the deck cracking problem. In general, this treatment appeared to have worked fairly well, but additional cracks opened up. Also, the methacrylate did not fully penetrate the crack, but rather penetrated to about the depth of the top rebar mat. It should be noted that in areas of low rebar cover, concrete subsidence during consolidation left a slight undulating or scrub board appearance with some subsidence cracking.

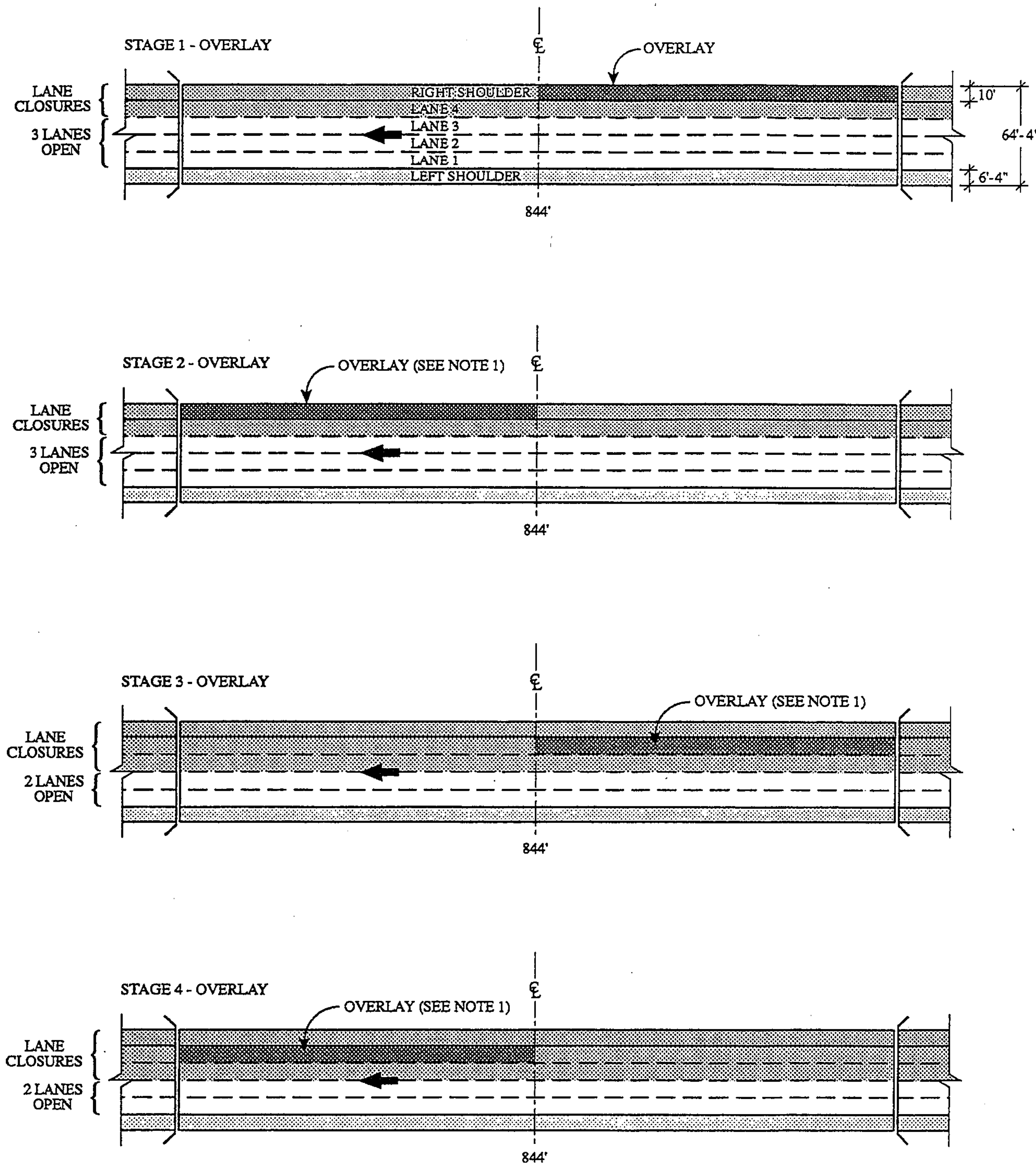


Fig. 6.1. Staged Construction Sequence for I-285 South Bound Bridge over Chattahoochee River Structure ID 121-0242-0.

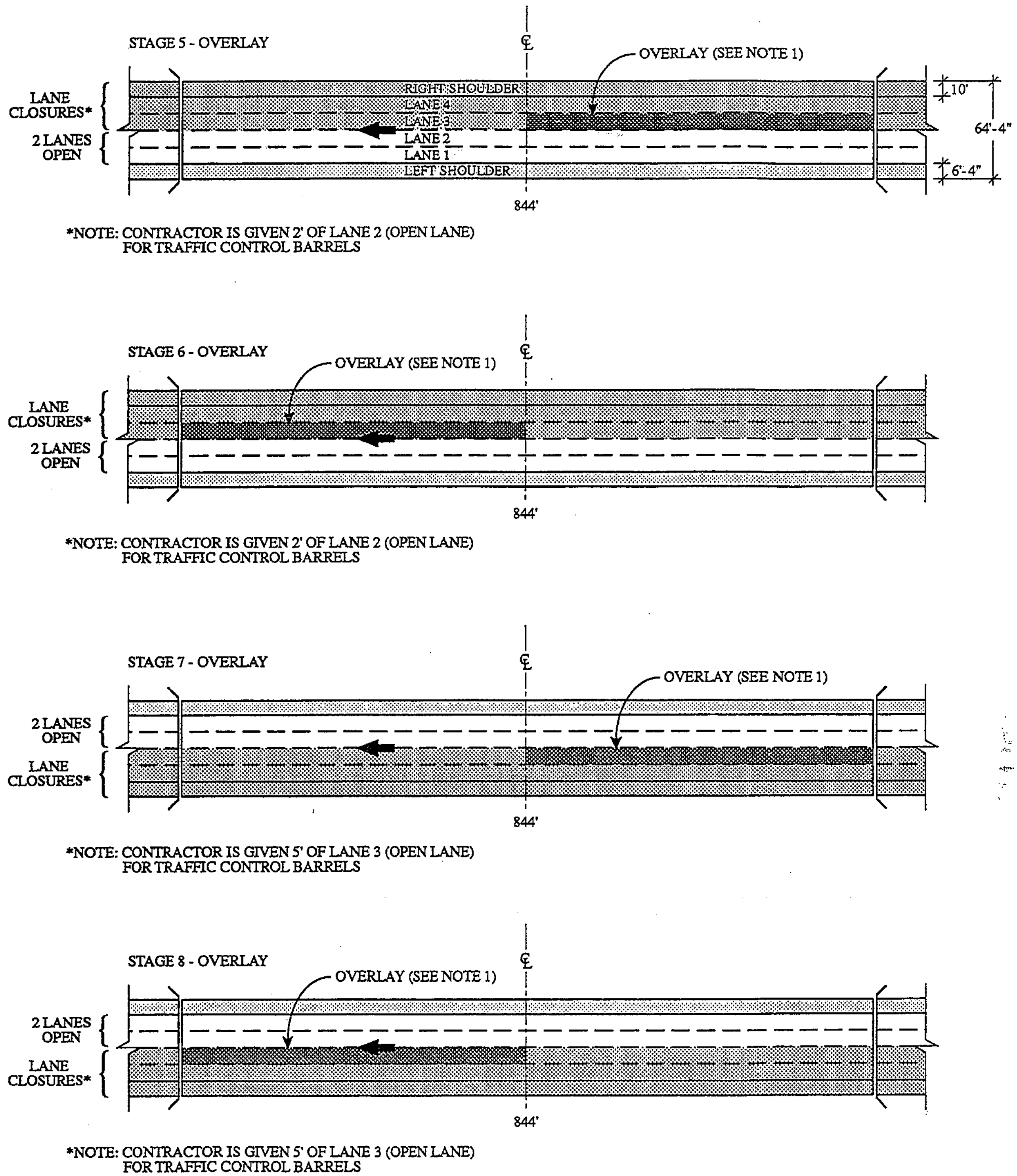
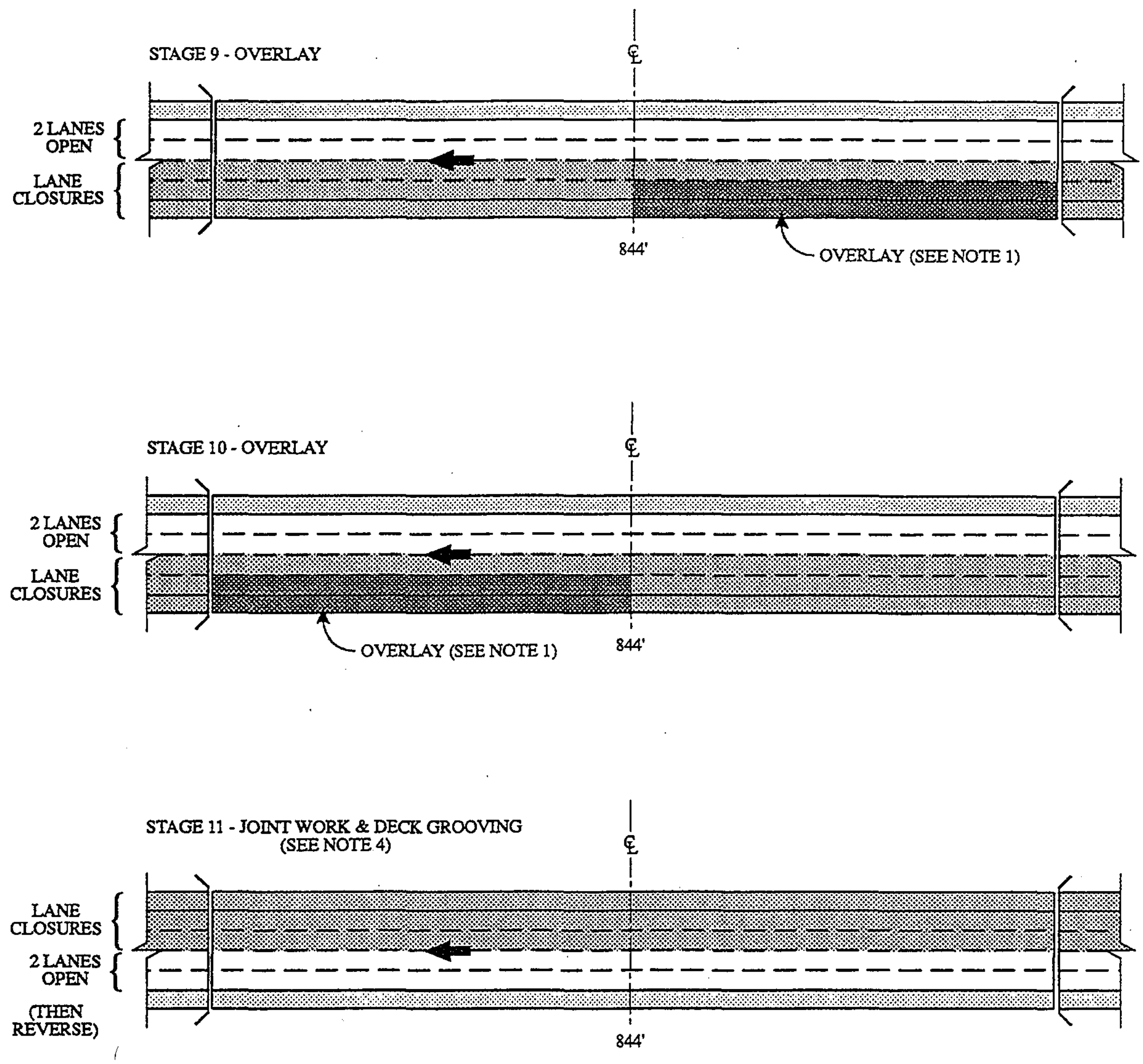


Fig. 6.1. Staged Construction Sequence for I-285 South Bound Bridge over Chattahoochee River Structure ID 121-0242-0 - Continued.



NOTES:

1) A SAW-OUT WAS MADE 3" BACK INTO THE PREVIOUSLY PLACED OVERLAY PRIOR TO PLACING THE ADJACENT OVERLAY STRIP.

2) A POLICE CAR WAS ON-SITE FOR COMPLETE TIME THE TRAFFIC CONTROL WAS IN FORCE, I.E., BEGINNING FRIDAY NIGHT AT 9:00 PM UNTIL MONDAY MORNING AT 5:00 AM.

3) TRAFFIC CONTROL WAS PROVIDED FOR 2 LANE HIGHWAY BELOW BRIDGE DURING THE TIME THAT HYDRODEMOLITION WORK WAS TAKING PLACE ON BRIDGE IMMEDIATELY ABOVE THE 2 LANE HIGHWAY.

4) IN STAGE 11, HALF THE BRIDGE WILL BE CLOSED ON SATURDAY MORNING AND ALL DECK JOINTS IN THAT HALF WILL BE FINISHED. IN THE AFTERNOON THE OPEN AND CLOSED HALFS WILL BE REVERSED AND THE REMAINING PORTIONS OF THE JOINTS WILL BE FINISHED. ON SUNDAY, THE SAME PROCEDURE WILL BE FOLLOWED FOR GROOVING THE DECK.

Fig. 6.1. Staged Construction Sequence for I-285 South Bound Bridge over Chattahoochee River Structure ID 121-0242-0 - Continued.

Figures 6.4 and 6.5 show the strip to be overlayed after completion of the rotamilling and before the hydrodemolition work. The rotamilling is done to save time and money in that it can remove the top 1 " of concrete much quicker and cheaper than by hydrodemolition. However, the top transverse steel in many locations on the deck proved to only have about 1/2" of concrete cover. This created a problem of the rotamilling machine damaging or tearing out the rebar. In these locations, the rotamilling was eliminated and all deck concrete removal was by hydrodemolition. Figures 6.14 and 6.15 show some slightly rusted splice bars that were used to replace bars damaged or removed by the rotamilling equipment. The contractor employed a rotamilling machine which milled a 6' wide strip and thus two passes of the machine were needed for each overlay strip.

Figures 6.6 and 6.7 show the water/slurry containment system needed for the hydrodemolition work which is very water and water/slurry intensive. This system was inplaced on the bridge each Friday night while the rotamilling was being done. The water barrier on the roadway was a 6" diameter rubber hose inflated with water along with polyethylene paper and sand bags. The baffled detention/settlement basins shown in Figs. 6.8 and 6.9 are at the low end of the bridge. They were inplaced before the lane closing and deck work began.

Figures 6.10 - 6.13 show the hydrodemolition units in action. Two units were used in a staggered manner (see Fig. 6.13) with each unit demolishing a 6' strip of the deck. The water pressure can be adjusted to demolish the concrete to the desired depth. As indicated earlier, this is a water intensive operation and a good water containment system needs to be in place before beginning. The strength/hardness of the concrete and the depth of removal were such that the units moved at an average speed of about 24 ft. per hour.

Figures 6.14 - 6.19 show the deck strip (to be overlayed) after passage of the hydrodemolition unit, and after the decomposed concrete and rubble have been flushed out by a large "fire hose". Note in these figures the large variation in thickness of demolished and removed concrete, and in turn the remaining concrete. It ranges from about 7" in Fig. 6.16 (complete "blow out") to about 11/2" near the left edge of Fig. 14. In Fig. 16, the rubble has not yet been flushed out because the "blow-out" hole first needs to be formed. This will be done by placing a piece of plywood on the underside and trying it to the deck rebar. Note the repaired, i.e., formed, "blow-out" in Fig. 6.19, and also the large variation of concrete removal in the region of that photograph. In one of the earlier strip hydrodemolition work, the contractor had a major "blow-out" requiring four 4'x 8' sheets of plywood to form. This occurred in a region of the deck where the Georgia DOT has had bad spalling and delamination problems and an area that they view they have bad or "dirty" concrete. The Georgia DOT was not upset by this large "blow-out" (or the other smaller "blow-outs") as they view that they have gotten rid of some bad concrete and have replaced it with excellent concrete (the overlay concrete).

Figures 6.20 - 6.23 show removal of the hydrodemolition demolished deck concrete. This is achieved primarily by flushing the demolished area with a large fire hose a short distance behind the hydrodemolition units. This can be seen in the distance in Fig. 6.21. The material is flushed onto a lane which has already been overlayed and removed by a small front-end loader and loaded on a dump truck as can be seen in Figs. 6.22 and 6.23. All of the lanes shown in Figs. 6.20 - 6.23 which are closed to traffic have already been overlayed.

Figure 6.21 shows the jack hammering work taking place after the demolished concrete has been flushed out. Even though the hydrodemolition units did an excellent job of demolishing the deck top concrete, on an overall or macro basis, there were many areas where removal to a minimum depth of 1/2" below the top mat longitudinal rebar was not achieved. This removal had to be done by jack hammering as shown in Fig. 6.21. The removal of the top concrete was very jack hammer intensive. There were prolonged periods of time when 10 jack hammers were in operation on the strip being prepared.

After the contractor felt that he had accomplished the concrete removal, a Georgia DOT inspector inspected the prepared area in detail and marked areas of further concrete removal and/or areas requiring rebar repair with spray paint as indicated in Fig. 6.24. The contractor then made a final pass on the strip, jack hammering and spicing in rebar where identified by the inspector. After this final pass of the jack hammers, a final water flushing pass was made, and this was followed by a walk through to remove larger pieces of rubble missed by the water flushing. The last step was to air blow the strip to remove entrapped pockets of water and to dry the strip. Figure 6.25 shows an area near the end of the strip ready for placing the overlay. Again, note the large variation in depth of the removal concrete, i.e., from about 2½" to 7" (at "blow-outs") and everywhere in between.

Figures 6.26 and 6.27 show the screed rails being set and the vibratory screed being prepared for placing the overlay. A burlap soaking tank is shown in the foreground in Fig. 6.26 in preparation for the 24-hour wet curing phase. In Fig. 6.27, the left existing concrete is the previously placed lane overlay, and the right concrete is the old existing deck. On the right side, a strip of 3/4" plywood is laid on top of the old existing deck and nailed to the transverse 2" x 4" rail supports shown. The screed height is adjusted to just touch the new overlay on the left and the top of the 3/4" plywood strip on the right. This results in the new overlayed deck being 3/4" thicker than the old deck. Later in preparation of the next lane for overlaying, a saw-cut will be made 3" back into the overly being prepared in Fig. 6.27 to assure removal of the temporary 3/4" overlay on the old existing deck on the right hand side.

Figures 6.28 and 6.29 show the overlay being placed, vibrated and screeded. Prior to placing the concrete, the edge of the previously placed strip is painted with epoxy to enhance the bonding of the strips. The screed unit consisted of a leveling auger in front and followed by a roller vibratory screed. The overlay concrete was a rich Type III cement mixture with a w/c of 0.4 and superplactized to yield a job site slump of 41/2" - 7" and a 24-hour minimum compressive strength of 3000 psi. The Georgia DOT inspector indicated that they had been achieving a 4000 psi strength in 24-hours. The Georgia DOT required the contractor to place the concrete at night to better control the concrete temperature, premature setting, workability, and to reduce early thermal shrinkage cracking.

Fogging units were on hand but were not used as the night temperature was cool and the humidity was high. After the concrete had hardened sufficiently, a broom finish was applied followed by 2 layers of wet burlap and a polyethylene covering as shown in Figs. 6.30 and 6.31. Soaker hoses were placed under the polyethylene covering to keep the burlap and overlay wet. Note the quality of the curing/coverings in so far as staying in place in the right photo of Fig. 6.30. Large trucks were passing within 5 ft. and blasting the coverings with air without removal, ripping, etc. of the curing coverings.

Figure 6.32 shows some transverse cracks in a previously placed overlay strip (not the strip shown in Figs. 6.4 - 6.31). The Georgia DOT concrete engineer feels that the overlay concrete being used is volumetrically very stable with little to no shrinkage. He believes the cracks shown in Fig. 6.32 are cracks reflecting through from the existing deck. This appears to be the case as some nearby cracking in the adjacent nonoverlayed lanes could be seen, and the cracking was not uniformly spaced throughout the deck as one would expected with shrinkage cracking. It should be noted that all experts indicate that existing deck live or working cracks will reflect through all types of bonded overlays. It appears that to a lesser degree this is also true for the Georgia DOT structural overlay.

Owners and project mangers for the overlay contractor, L. C. Whitford Co., Inc., indicated that they have been installing this type of bridge deck overlay for approximately 12 years in and around New York, and that the overlays are performing beautifully.

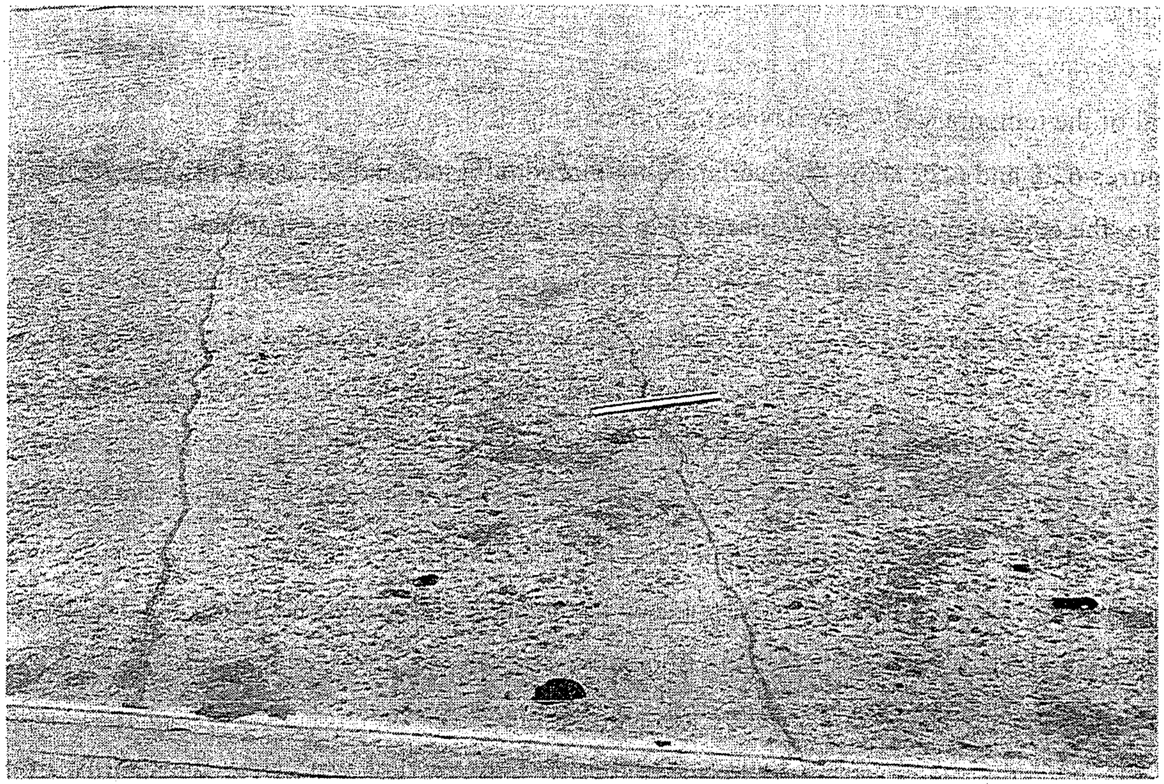


Fig. 6.2. Typical Transverse Deck Cracking

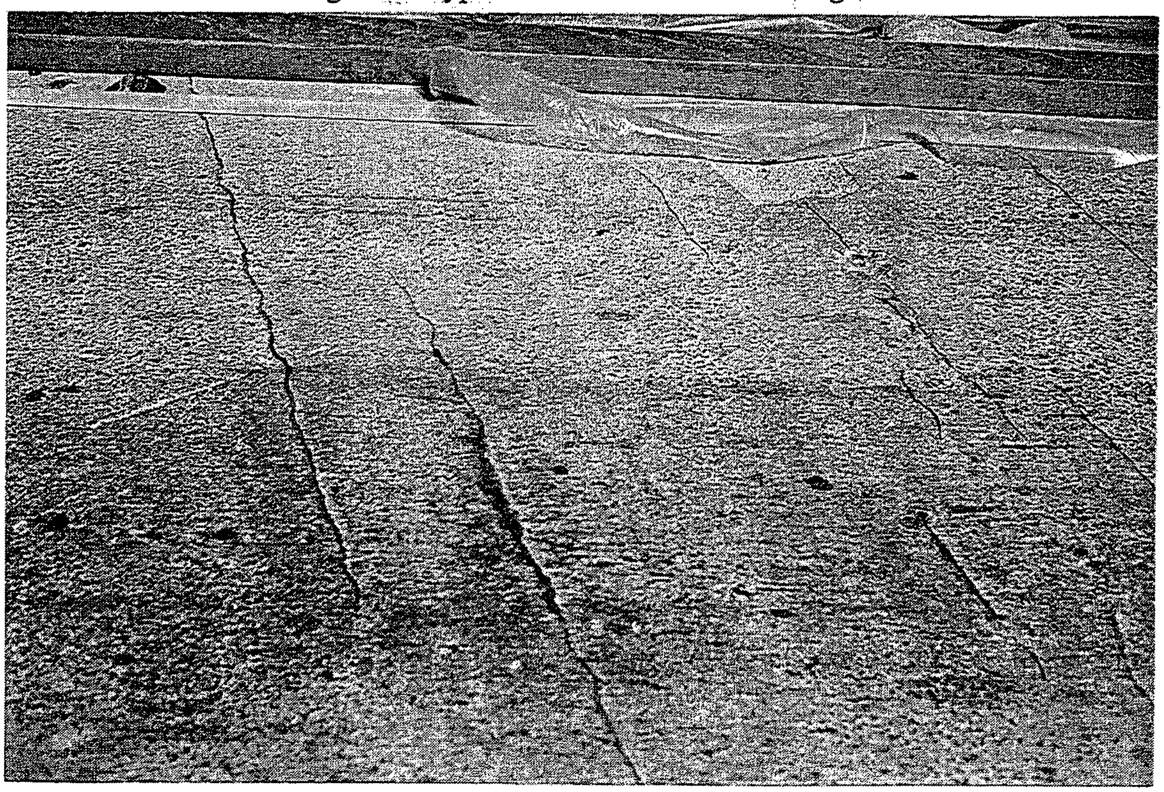


Fig. 6.3 Severe Transverse Deck Cracking

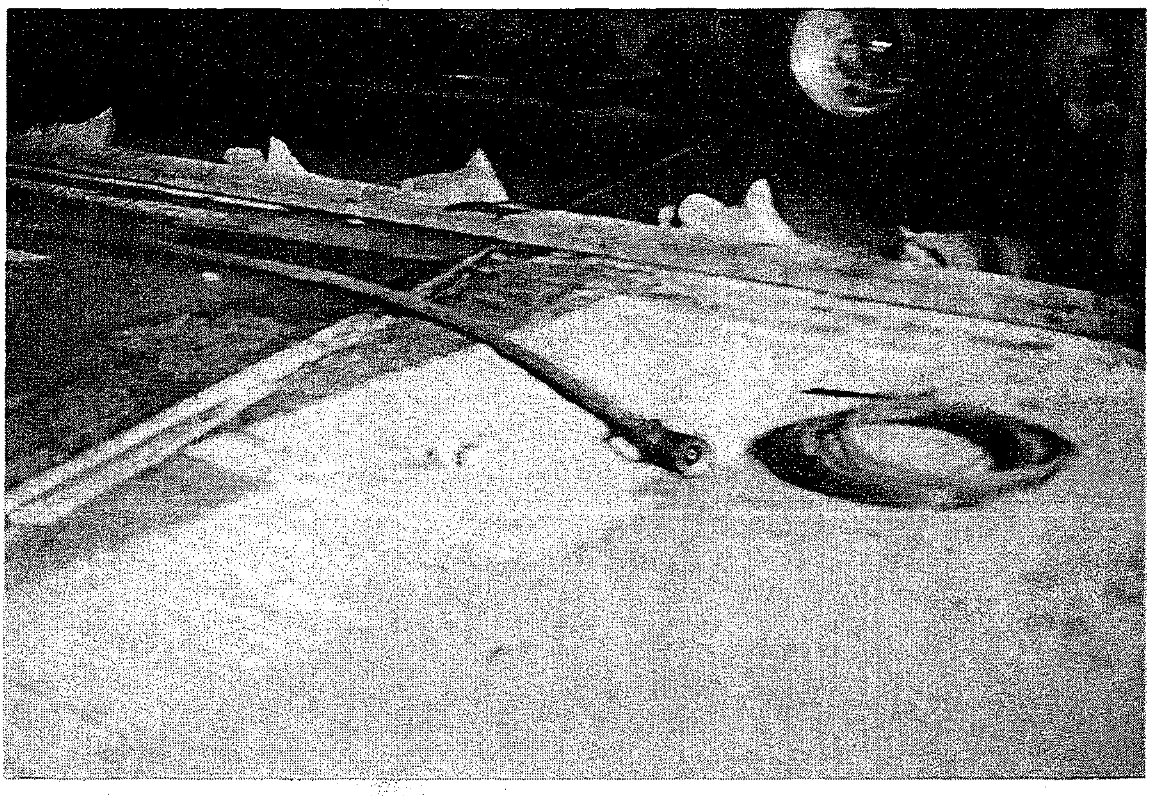


Fig. 6.4. Beginning of Deck Rotamilling for an Overlay Strip

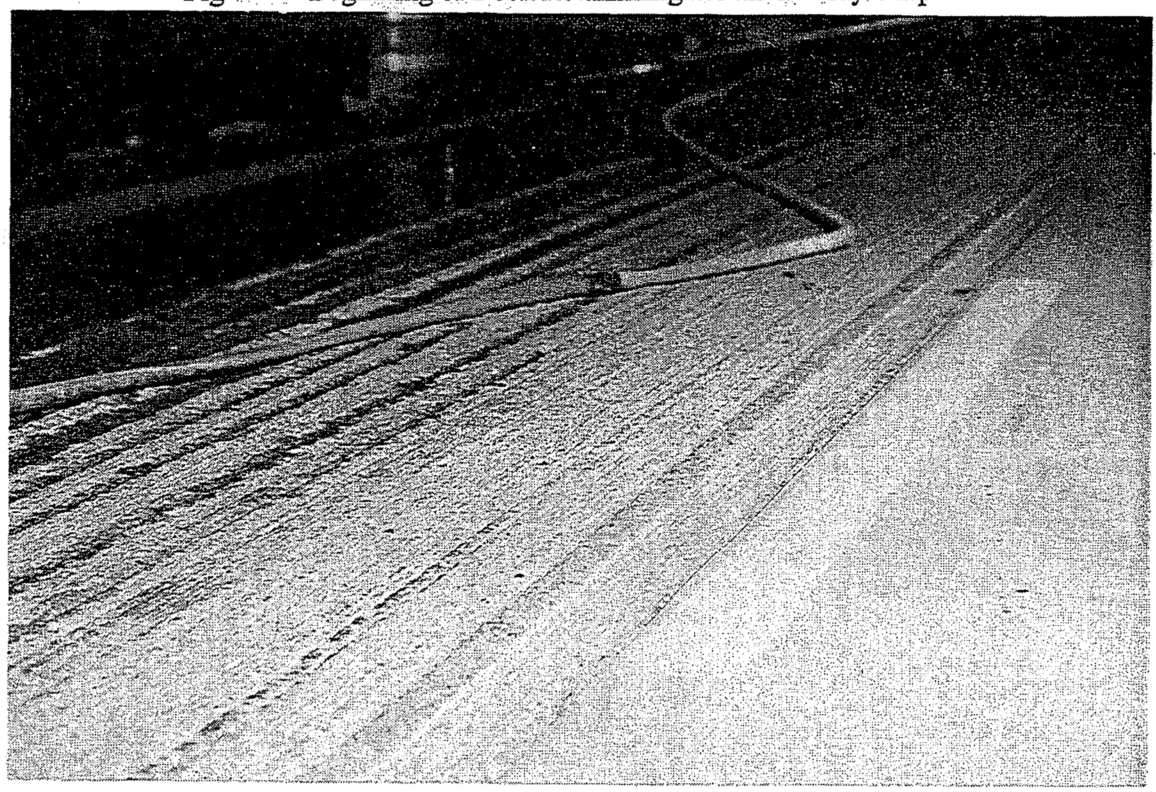


Fig. 6.5. Overlay Strip After Rotamilling and Before Hydrodemolition

Fig. 6.6. Water/Slurry Containment System on Bridge Looking Toward Hydrodemolition Units

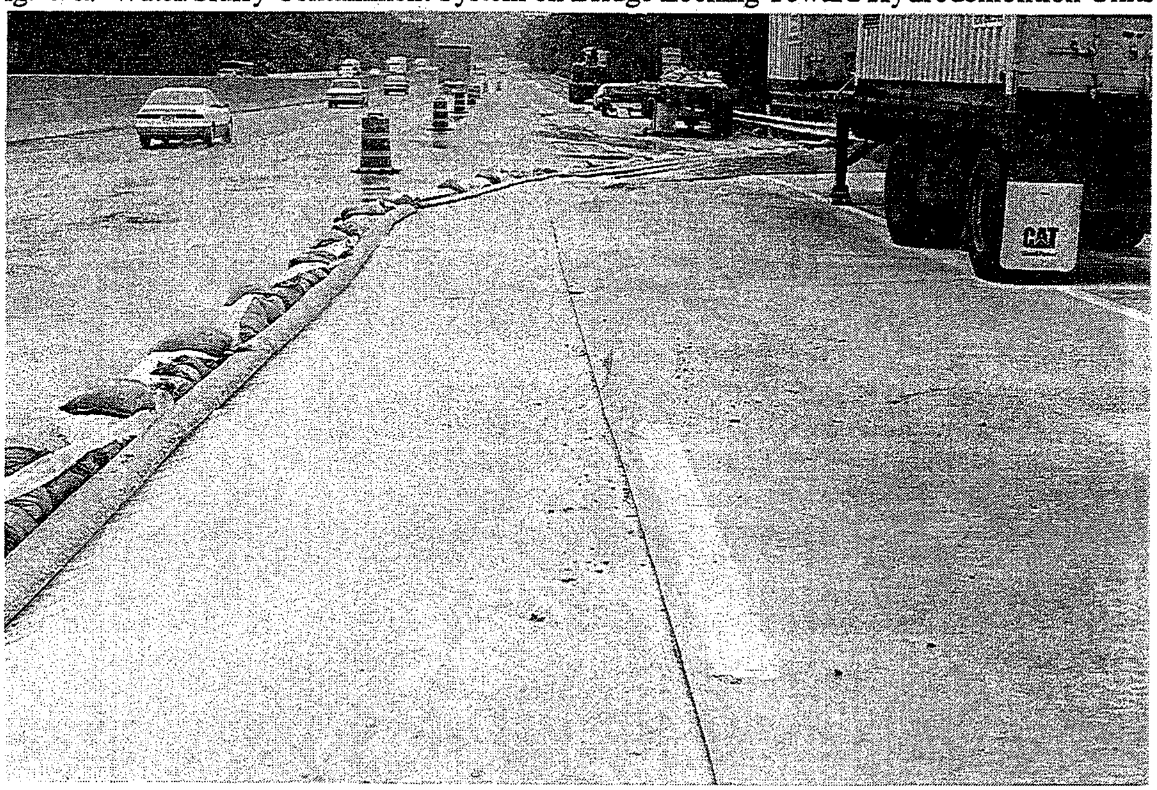


Fig. 6.7. Water/Slurry Containment System on Bridge Near Settlement/Detention Basins

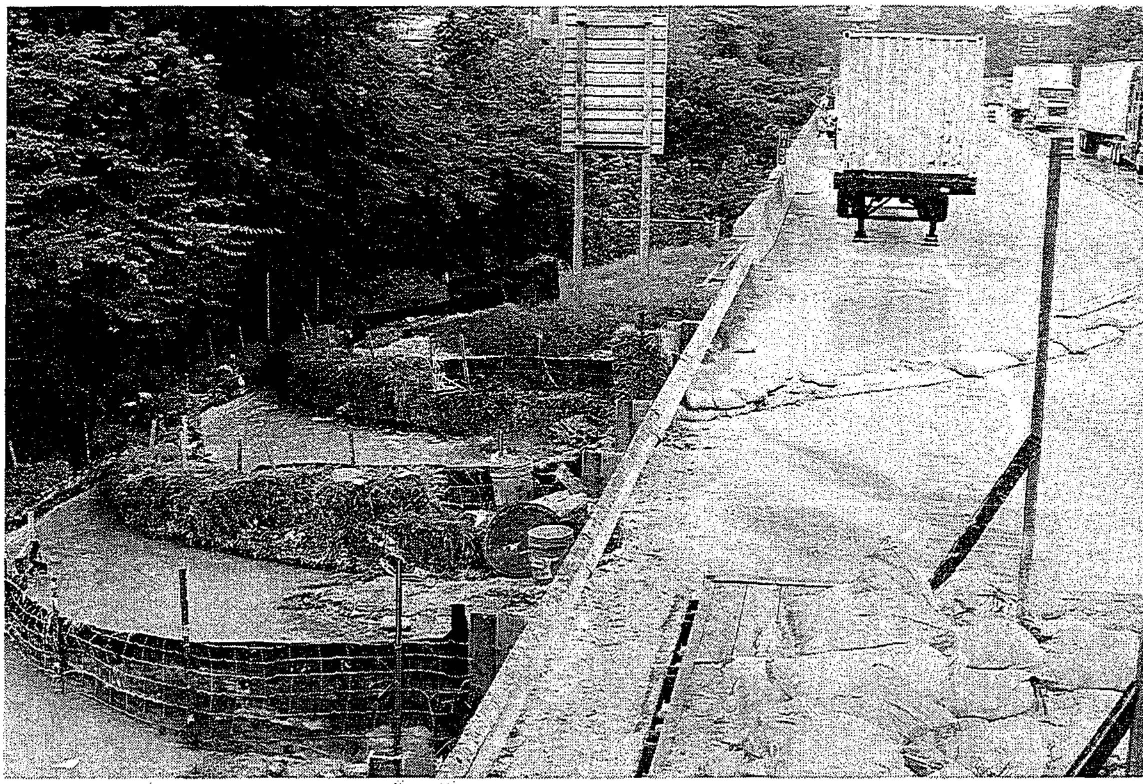


Fig. 6.8. Water/Slurry Containment System and Detention/Settlement Basins

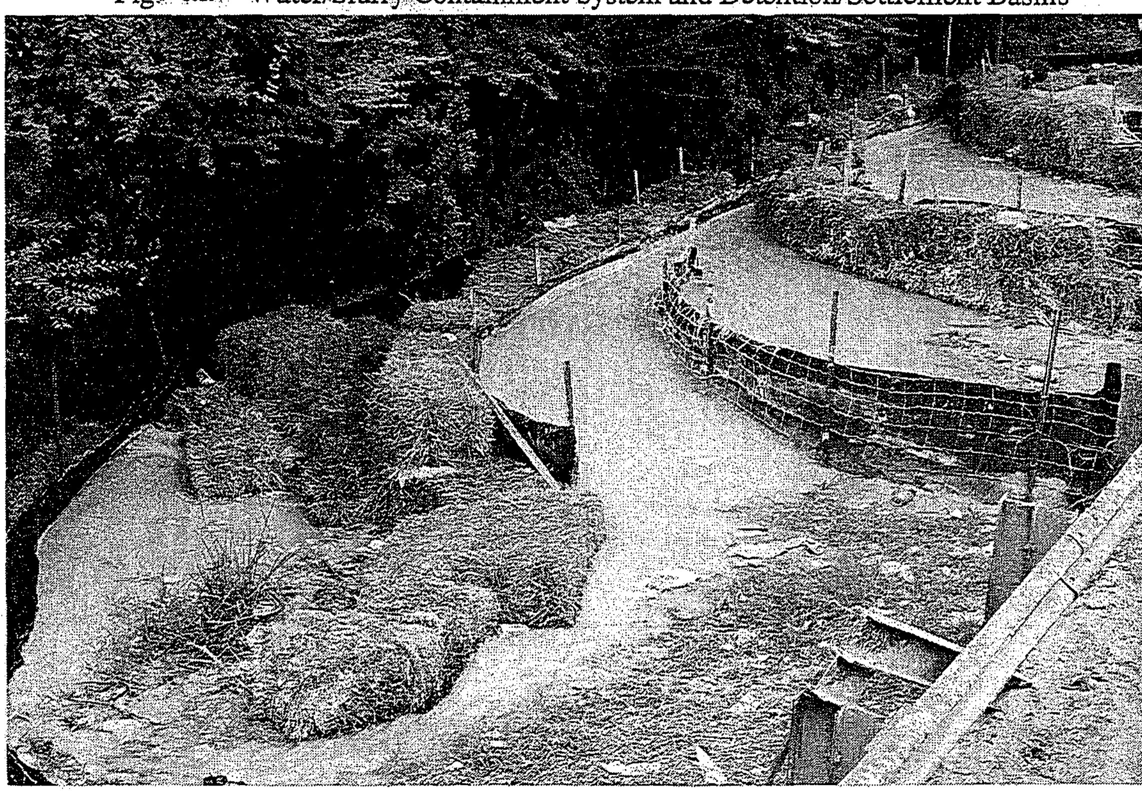


Fig. 6.9. Water/Slurry Baffled Settlement Basins

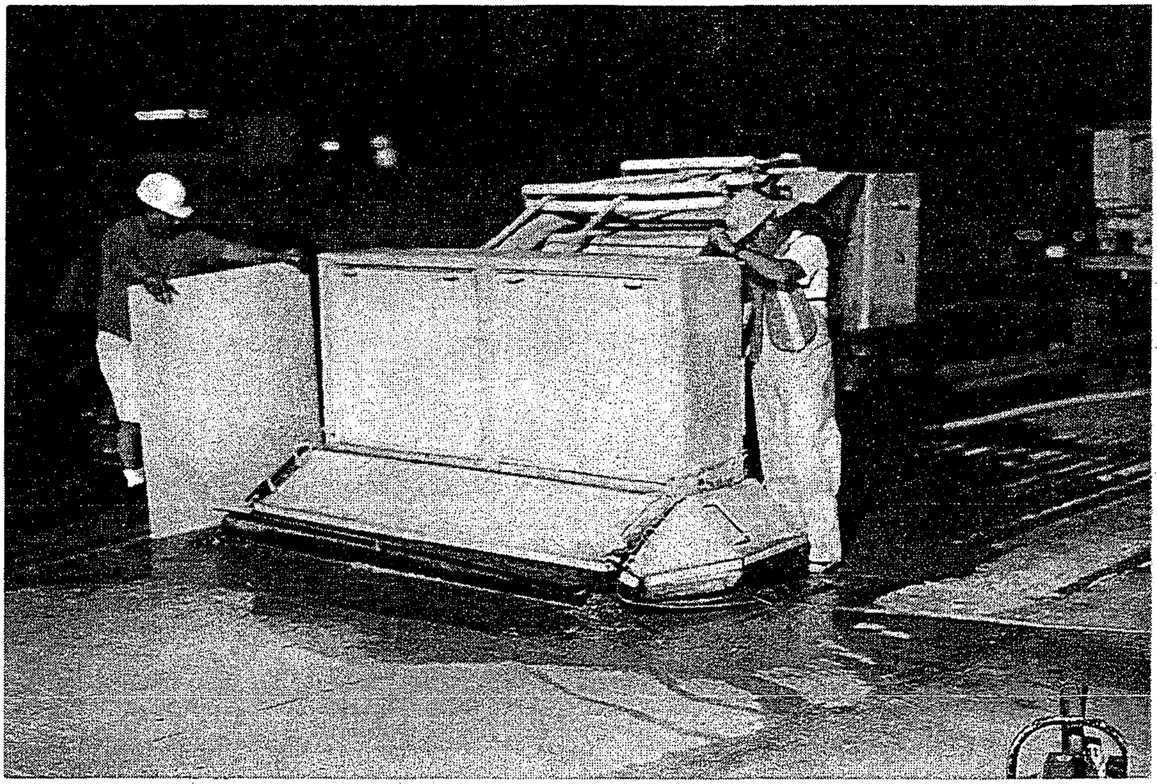


Fig. 6.10. Front View of Hydrodemolition Unit at Beginning of Strip Removal

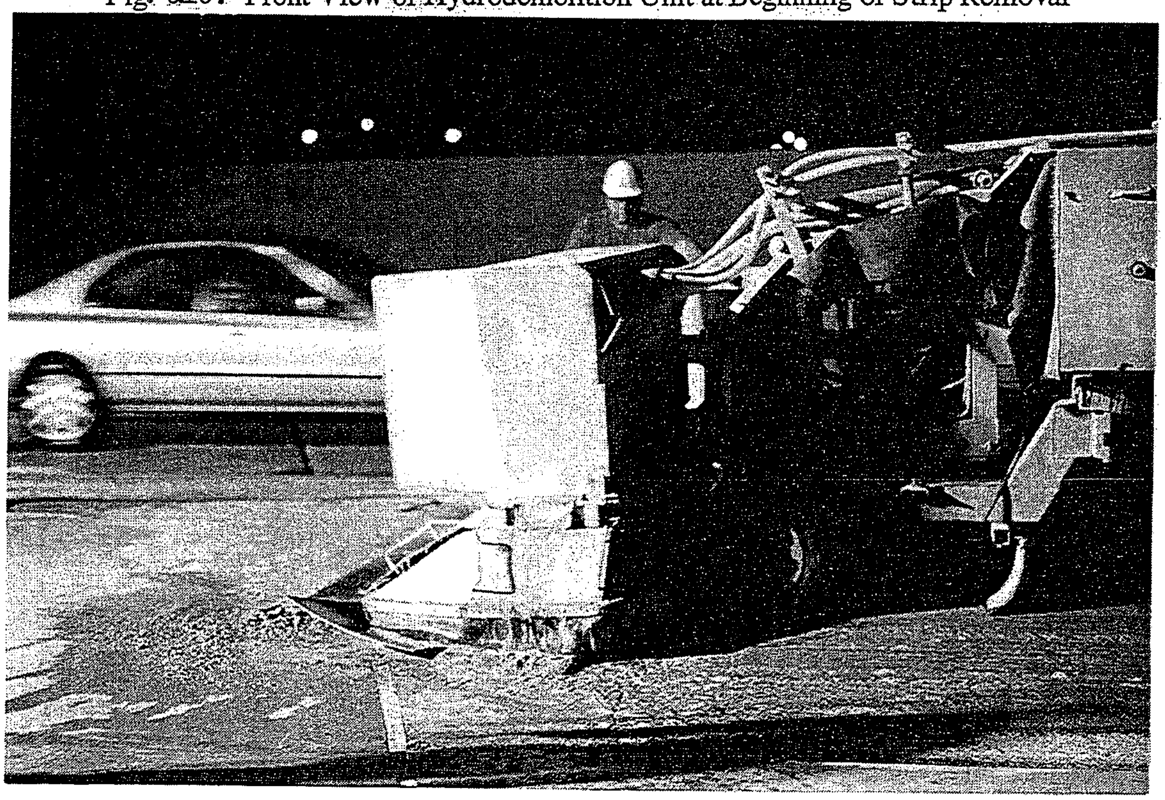


Fig. 6.11 Side View of Hydrodemolition Unit at Beginning of Strip Removal

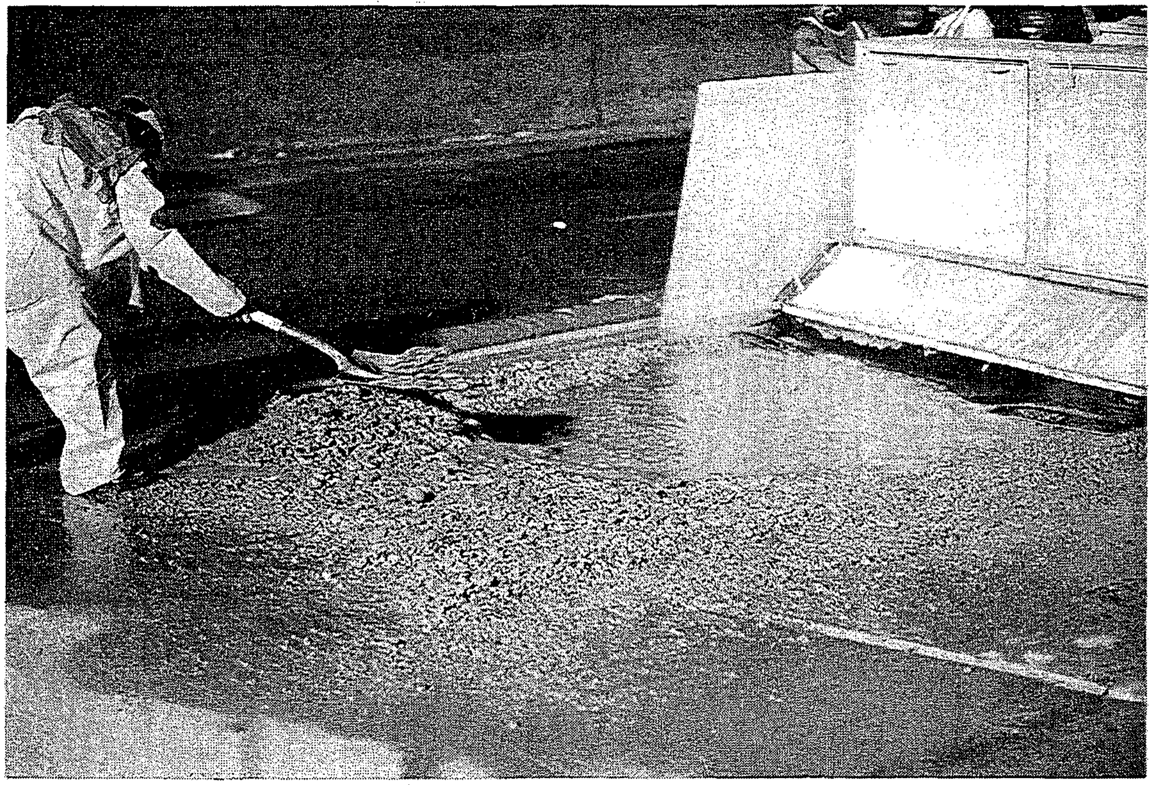


Fig. 6.12 Decomposed Concrete Mixture/Rubble and Water Remaining After Passage of Hydrodemolition Unit

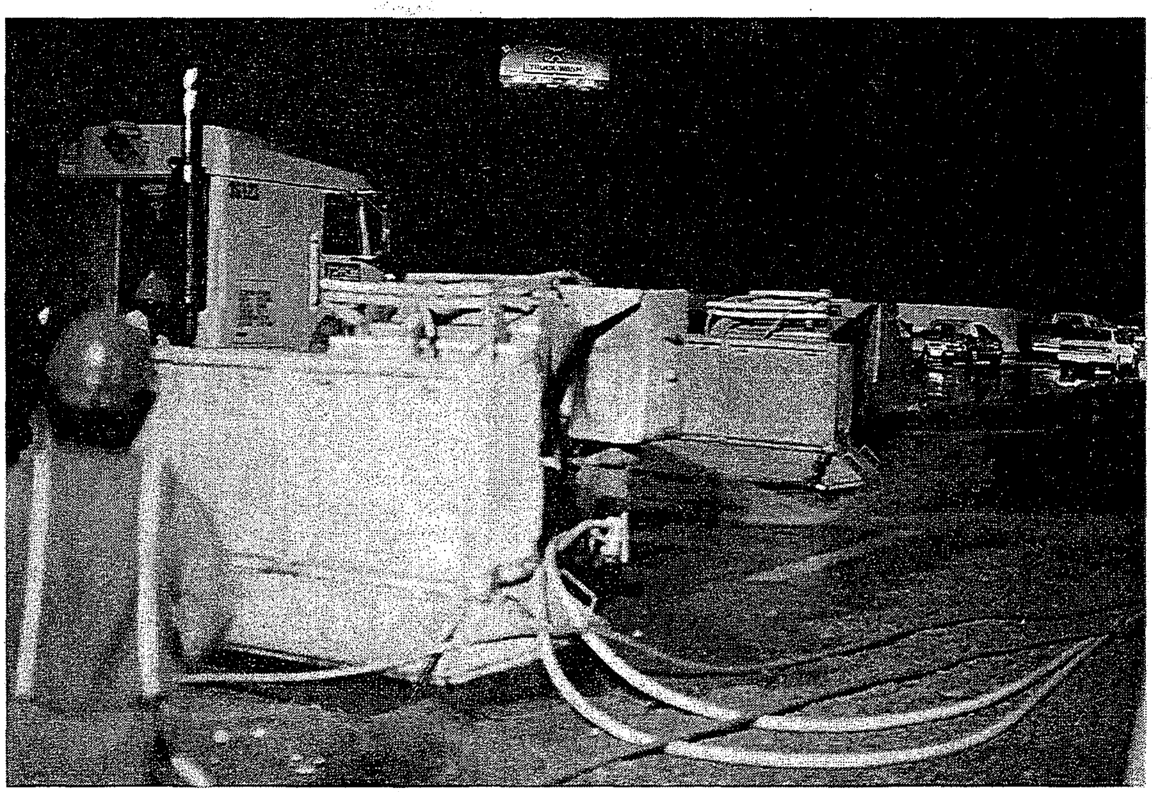


Fig. 6.13 Two Hydrodemolition Units Moving in Staggered Manner Down Lane to be Overlayed

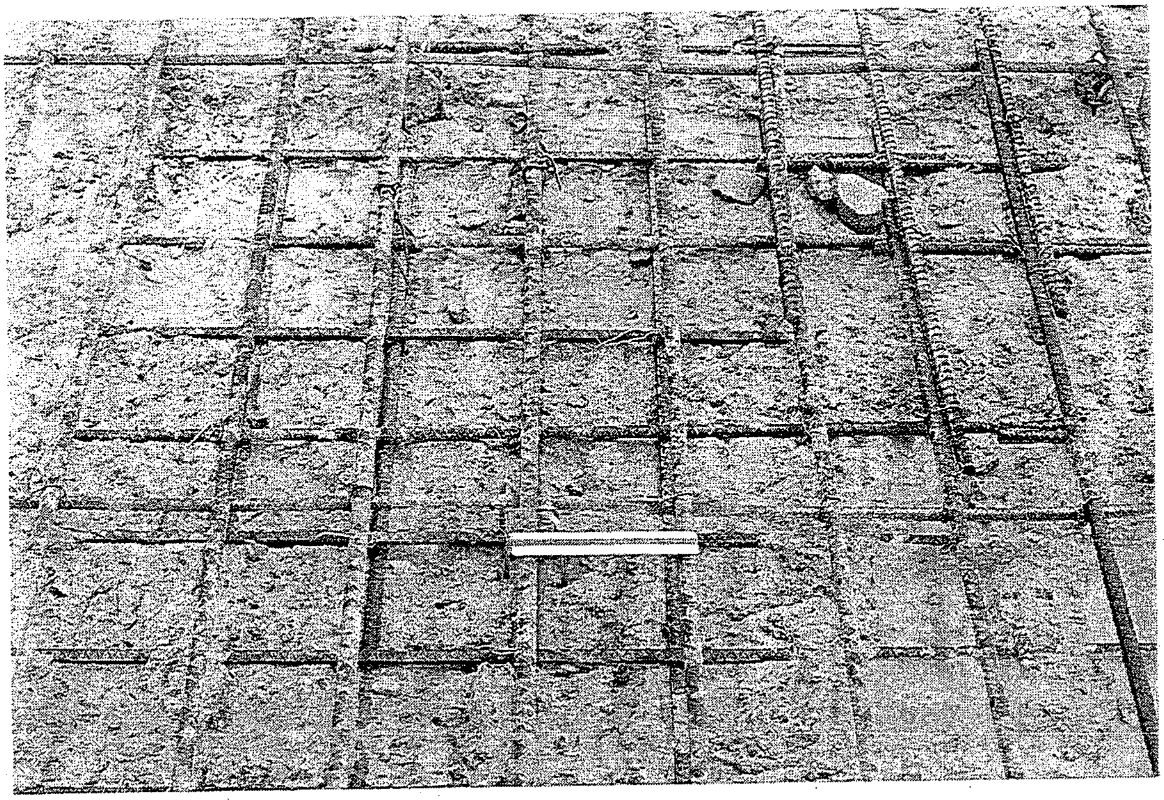


Fig. 6.14. Exposed Rebar After Hydrodemolition Showing Some Rebar Damage (Splices) from Rotamilling

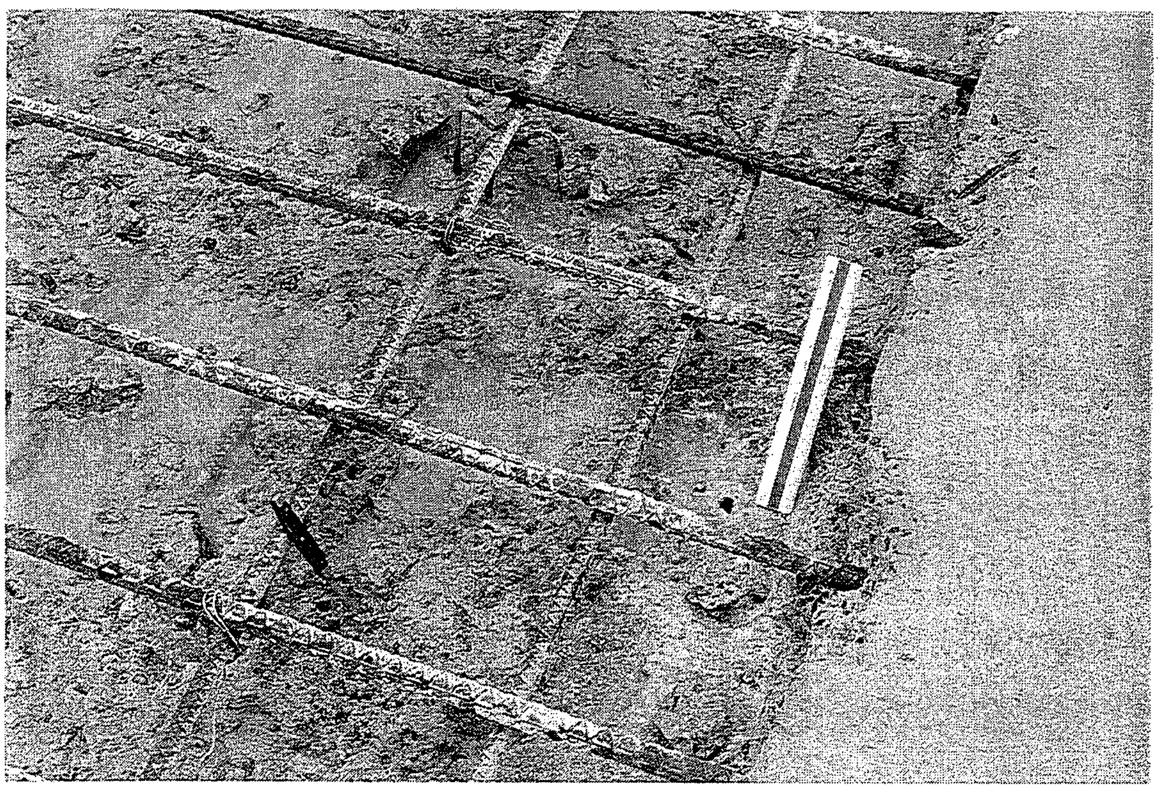


Fig. 6.15. Exposed Rebar at Edge Showing Some Rebar and Overlay Damage from Rotamilling

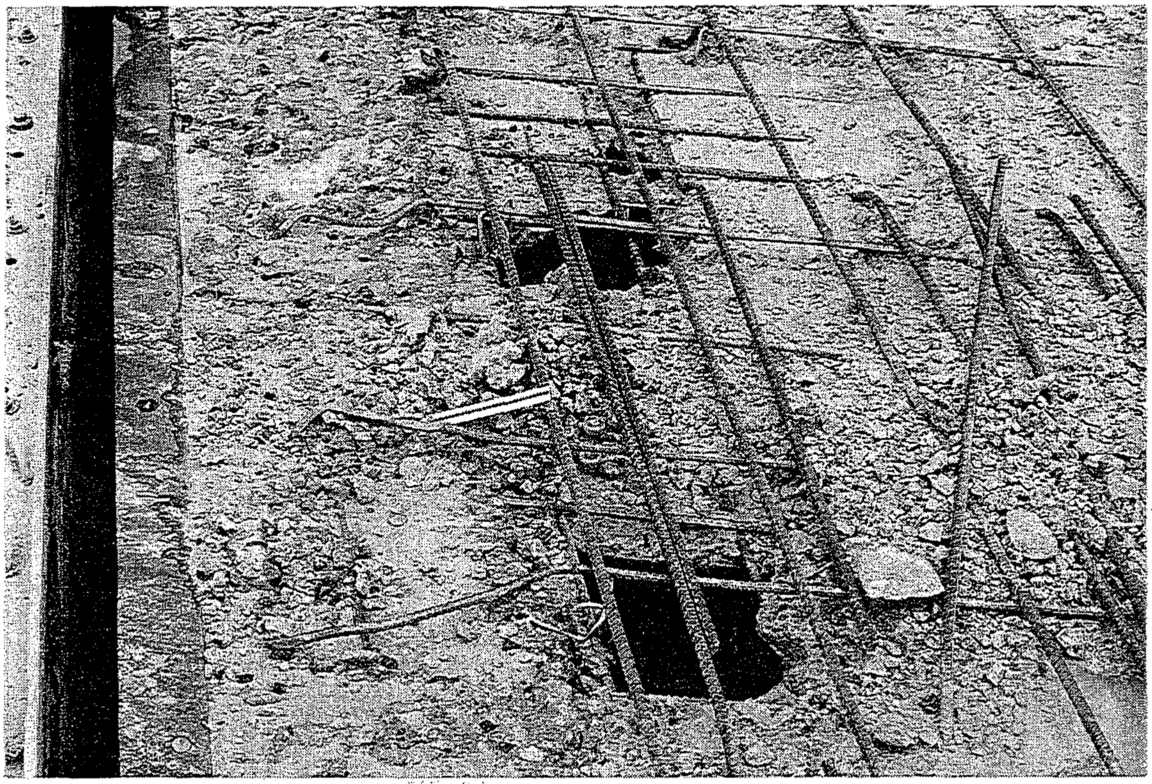


Fig. 6.16. Hydrodemolition Portion of Deck Near an Expansion Joint with Two Sizeable "Blow-Outs"

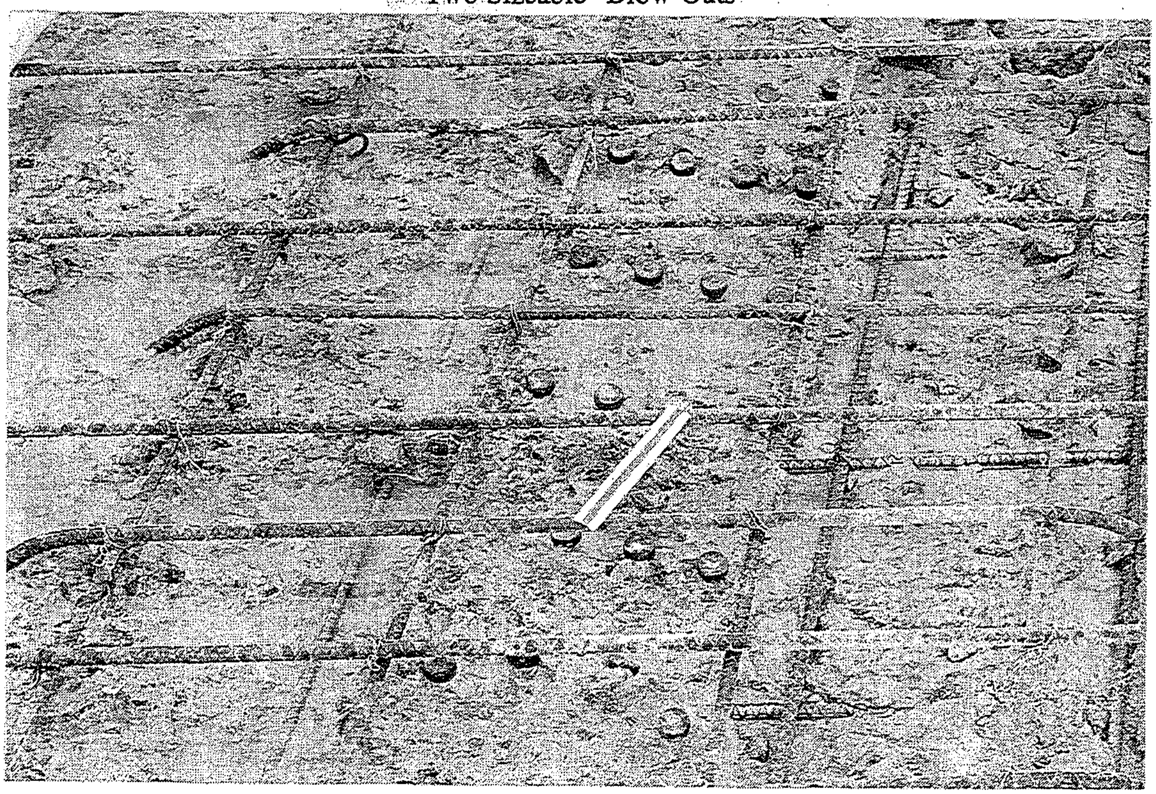


Fig. 6.17. Results of Hydrodemolition at One Region Exposing Portion of Lower Rebar Mat and Girder Shear Lugs

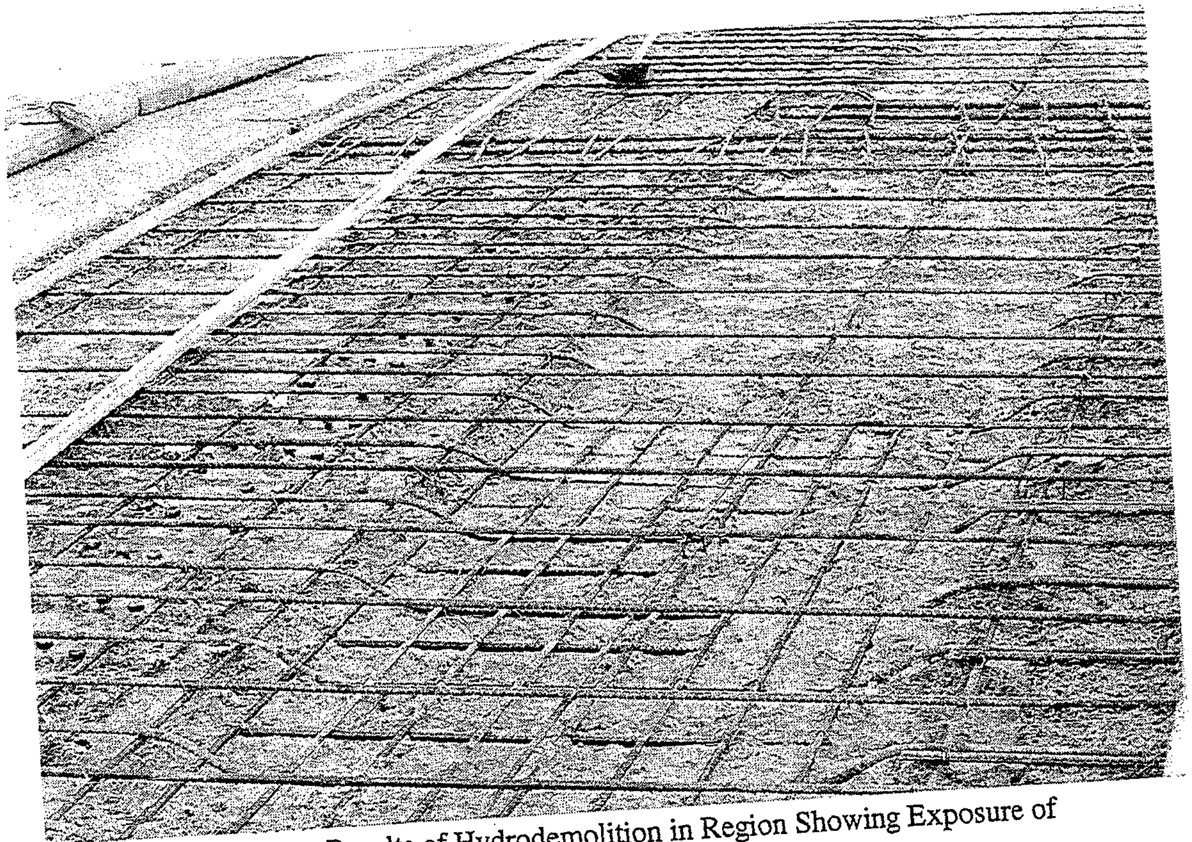


Fig. 6.18. Results of Hydrodemolition in Region Showing Exposure of Some Truss Bars, Bottom Bars and Shear Lugs

Fig. 6.19. Results of Hydrodemolition Showing Large Variation in Concrete Removal and Small "Blow-Out" (Already Formed)

Fig. 6.20 Strip to be Overlayed Just After Completion of Hydrodemolition - Looking North

Fig. 6.21. Strip to be Overlayed Just After Completion of Hydrodemolition - Looking South

Fig. 6.22. Demolished Concrete Being Removed by Small Front-End Loader

Fig. 6.23. Front-End Loader (Behind Truck) Placing Demolished Concrete in Dump Truck

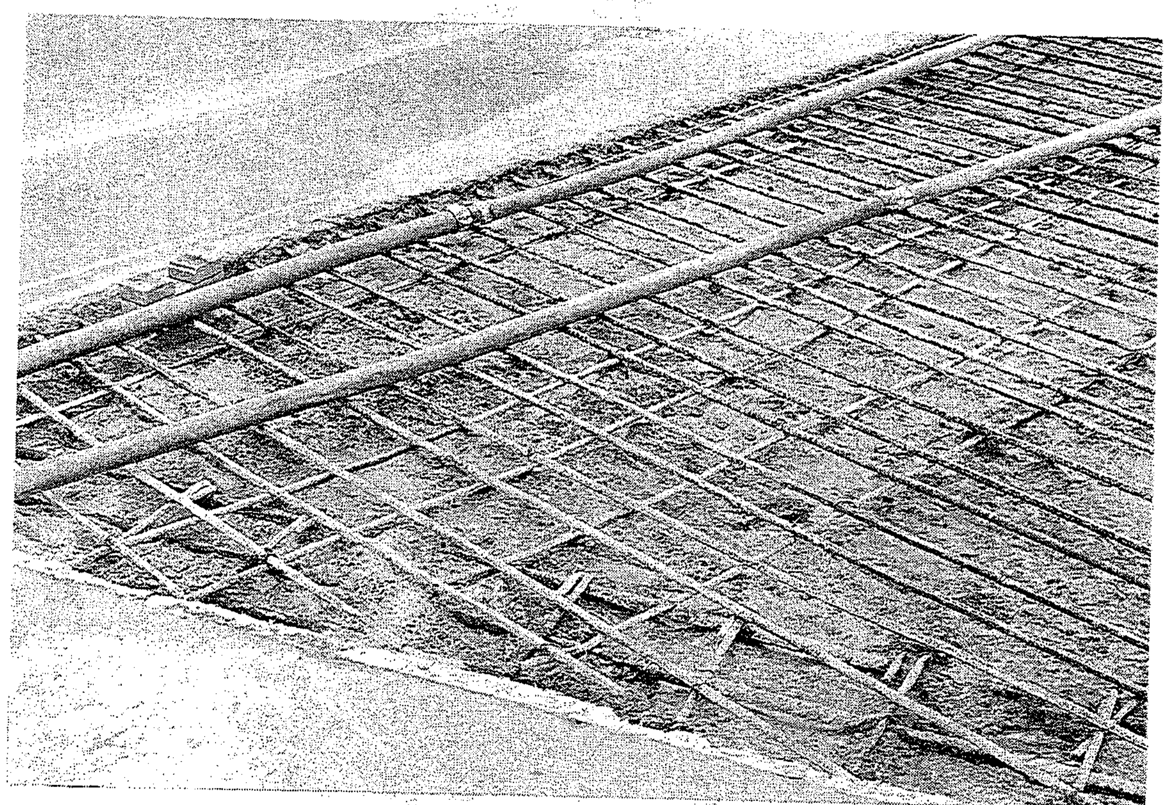


Fig. 6.24. Areas Marked by Georgia DOT Inspector for Further Concrete Removal or Rebar Repair

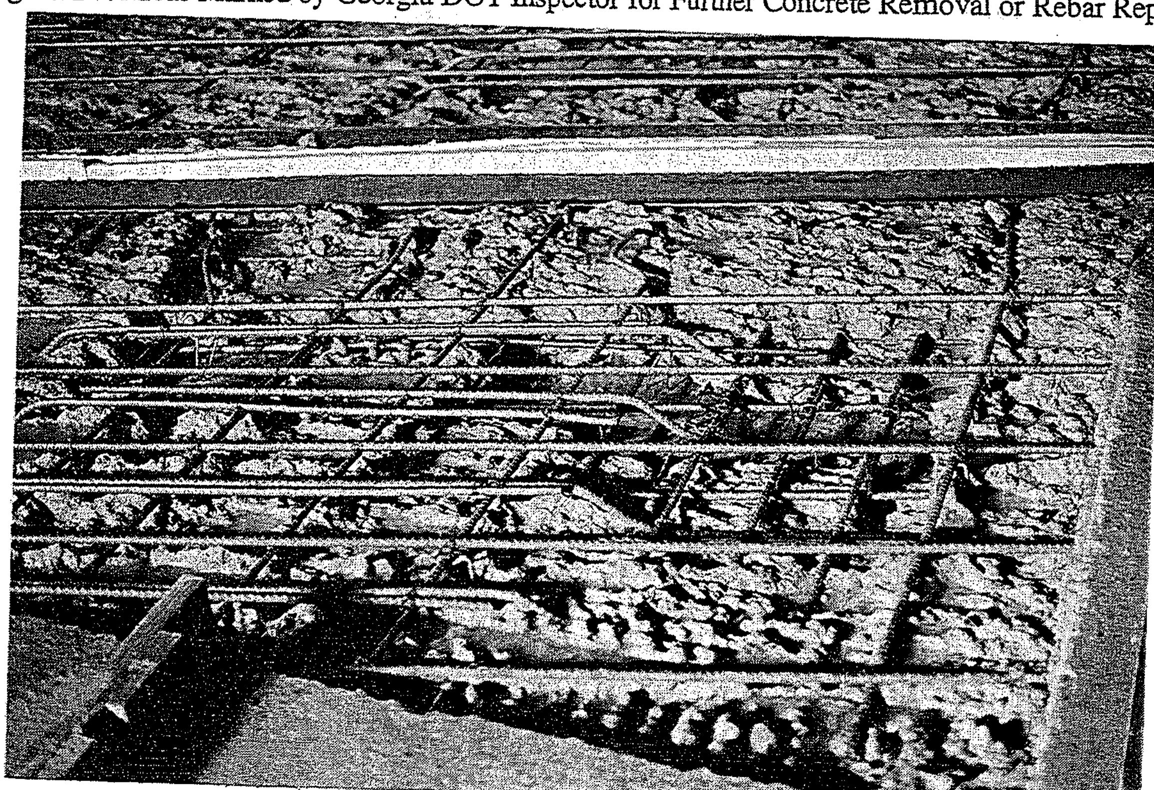


Fig 6.25. Area Near End of Strip Ready for Overlay Placement

Fig. 6.26. Preparation of Screed Rails, Vibratory Screed and Wet Burlap for Curing

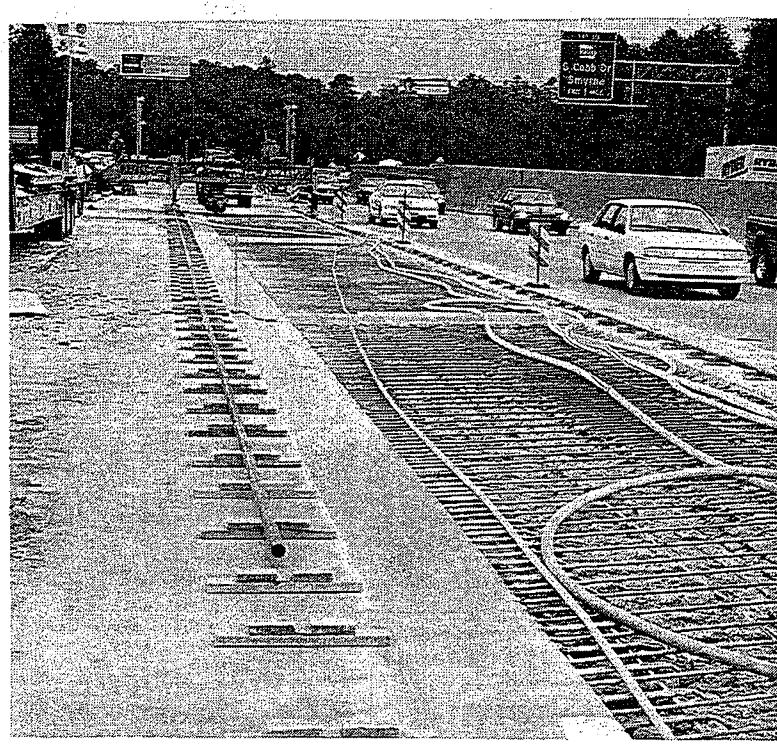


Fig. 6.27. Screed Rails Being Set and Vibratory Screed in Background Being Prepared.

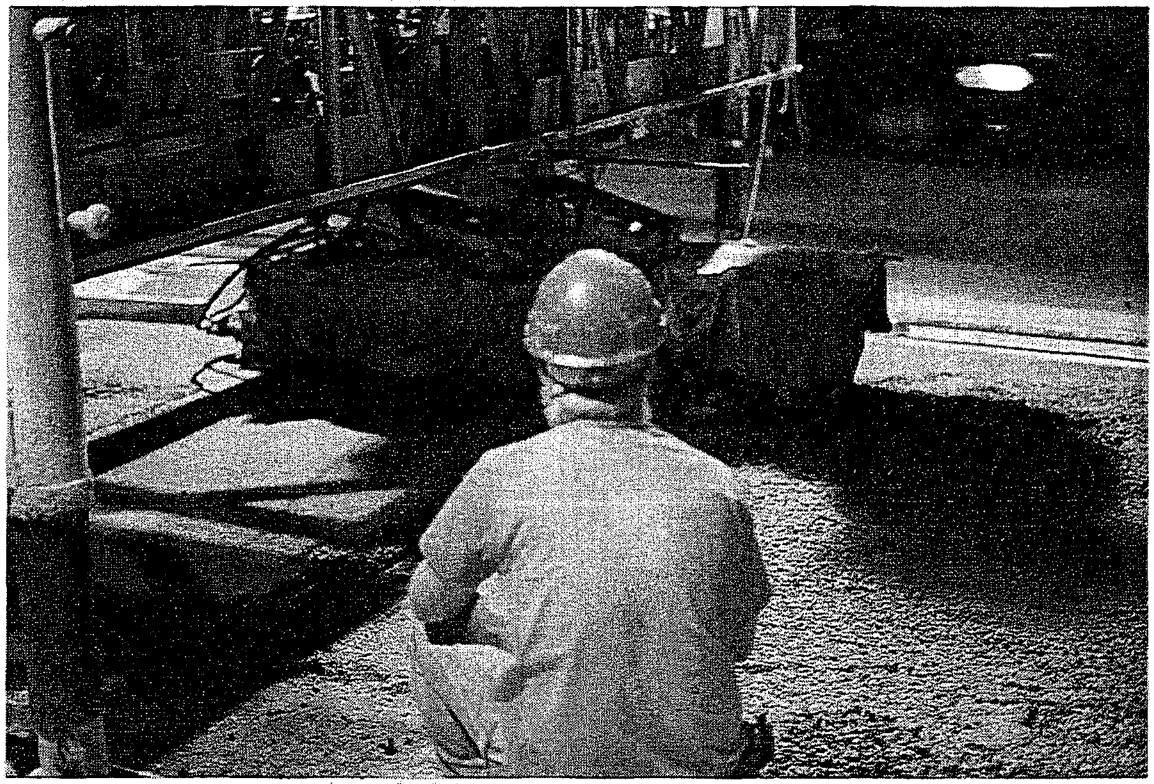


Fig. 6.28. Overlay Being Placed, Vibrated and Screeded

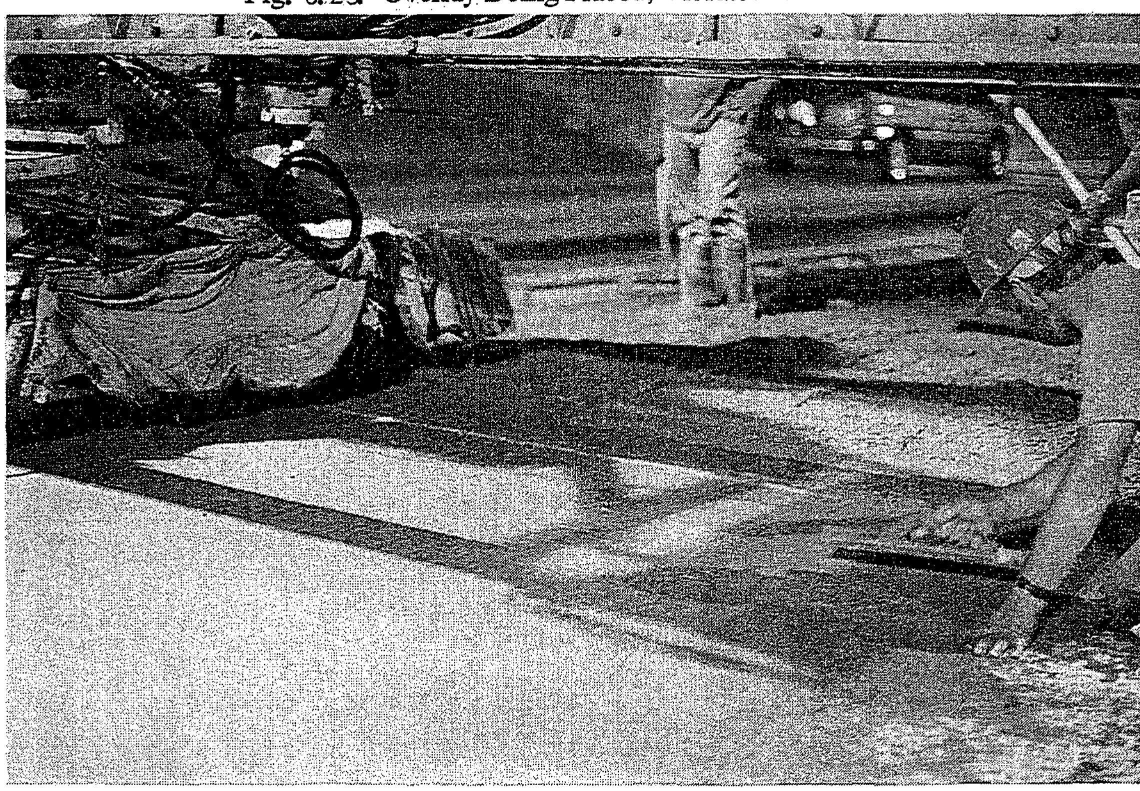


Fig. 6.29. Overlay Screeding and Finishing

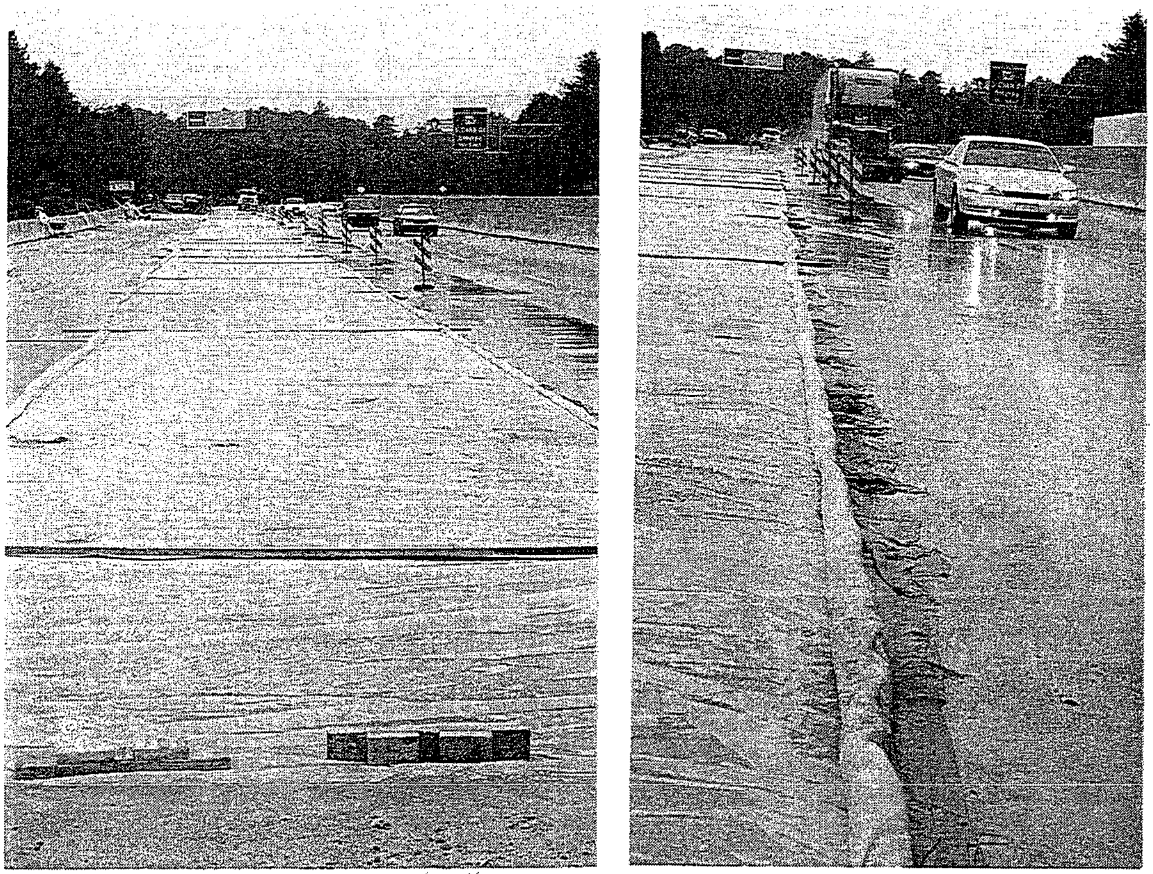


Fig. 6.30. Overlay Wet Burlap and Polyethylene Curing System in Place

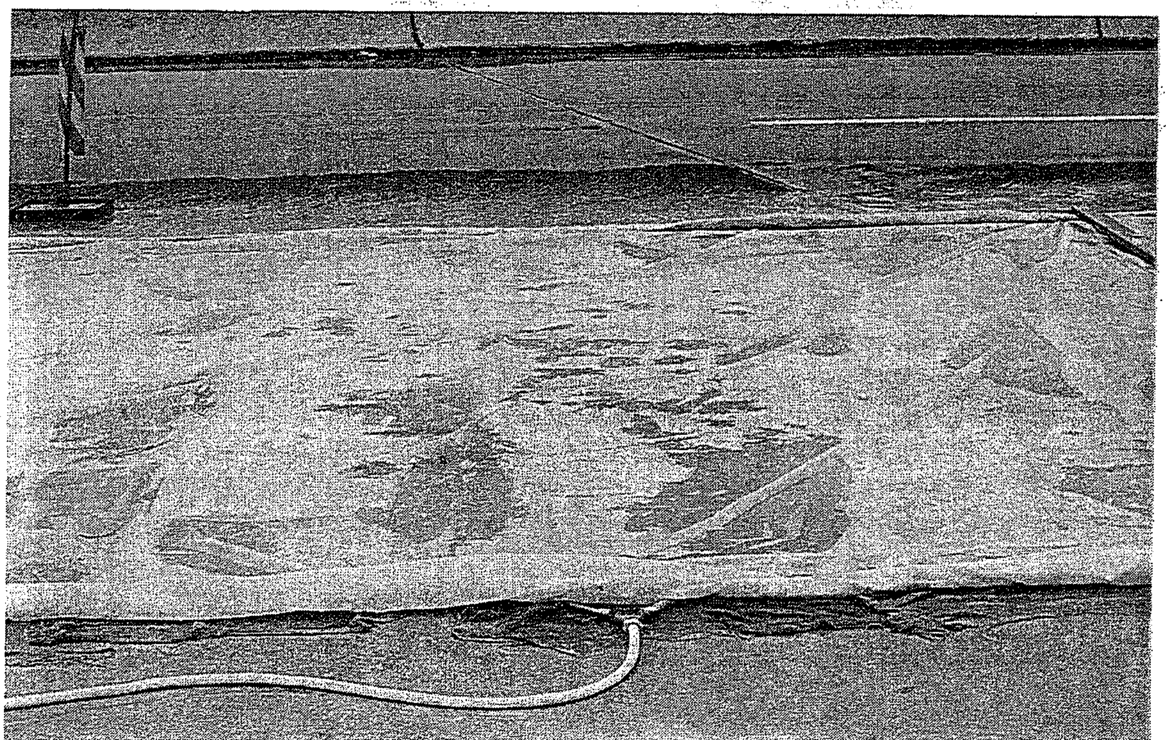
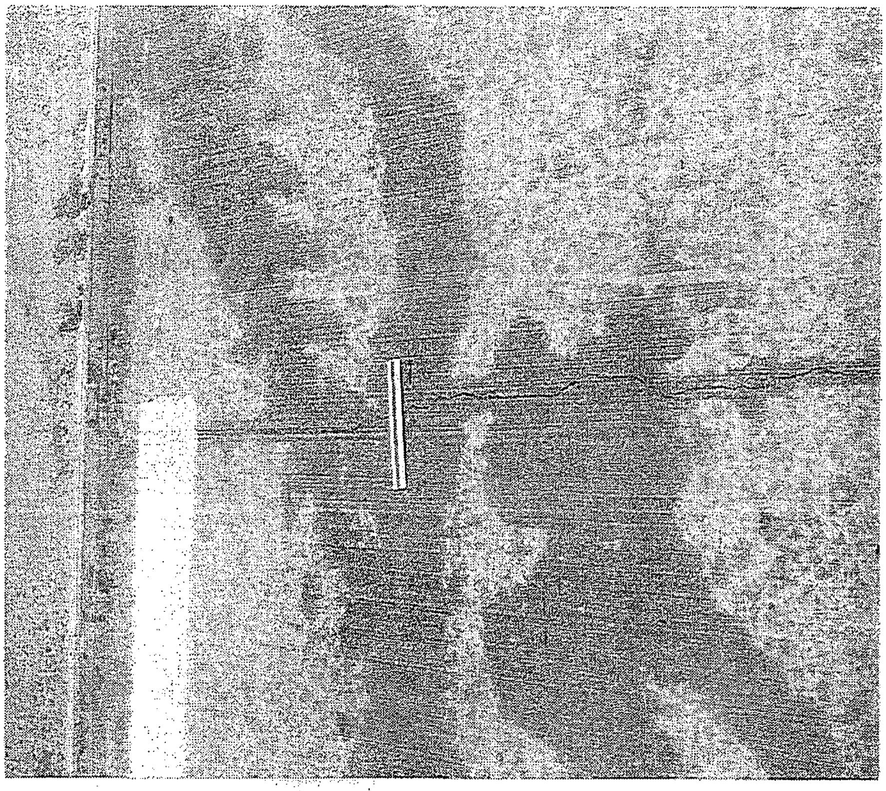


Fig. 6.31. Soaker Hose Under Polyethylene Covering



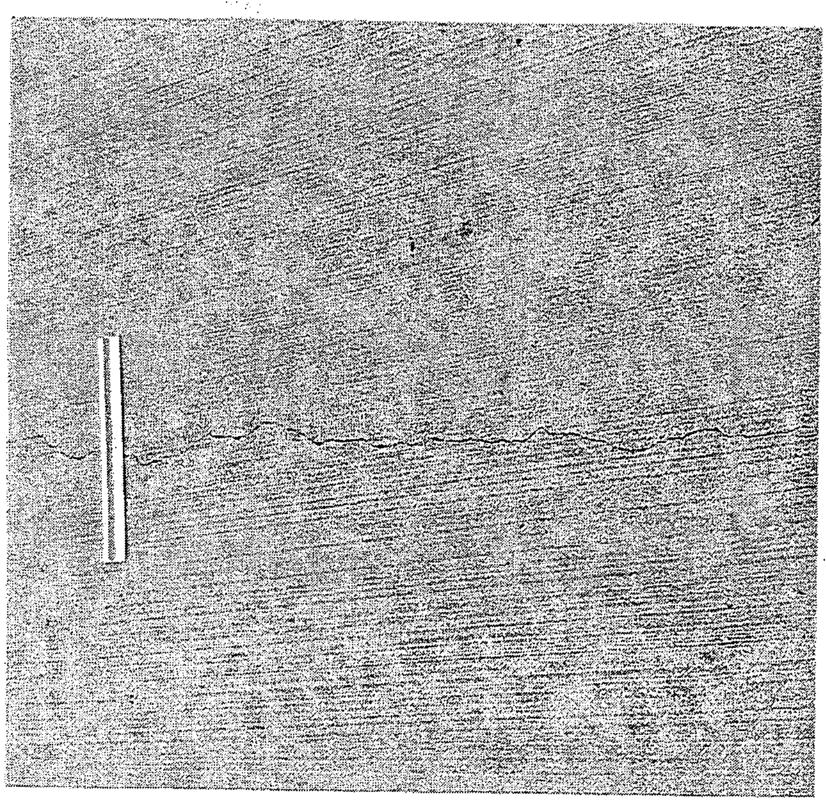


Fig. 6.32. Transverse Cracks in Previously Placed Overlay

7. KENTUCKY DOT RSLMC BRIDGE DECK OVERLAY CONSTRUCTION SEQUENCE

7.1 General

On the weekend of September 19-21, 1997, Dr. Ramey visited the job site of two bridge deck overlay projects in Louisville, Kentucky as a guest of the Kentucky Department of Transportation (DOT). The bridges are located just off of I-65 at the interchange of I-65 with I64/I71 ("Spaghetti Junction"). The primary descriptive parameters for the bridges are shown in Table 7.1. As evident from Table 7.1, the original concrete bridge decks had service lives of around 21 years before requiring an overlay, and the 1¼ " thick LMC overlays had service lives of around 12 years before being replaced with 1½" thick RSLMC overlays. It should be noted that the overlays on some of the bridge ramps and lanes in "Spaghetti Junction" were still in good shape and would have provided a number of additional years of service life. However, several had reached the point of requiring replacement, and a decision was made to replace all of the overlays a one time (over a 3 month period) rather than replacing them as they wore out.

It appeared that the overlays on the low lanes where water, snow and deicing salts would tend to accumulate, and the ones in the most highly trafficked lanes were the ones exhibiting the greatest deterioration. This deterioration was primarily in the form of "patches" of overlay delamination which were partly caused by entry of water and freezing of entrapped water. Because the bridges in "Spaghetti Junction" are mostly interchange ramps and thus are quite curvy with rather steep slopes, the authorities in Louisville make heavy utilization of deicing salts on these bridges. This is one of the primary causes of reduced deck and overlay service life for these bridges. The Kentucky DOT's approach in maintaining these bridges appears to be to mill off and replace 1¼"-1½" overlay every 12 - 15 years and replace it with a new overlay. There are several construction firms in Kentucky which specialize in placing bridge deck overlays, and they seem to be quite knowledgeable and competent in placing deck overlays.

A bonded overlay of rapid setting latex modified concrete (RSLMC) was placed on both of the bridges in Table 7.1 after removing the existing 1¼" LMC overlay by milling. After the milling was completed, the decks were sounded by drag chains to locate delaminations and other concrete deteriorations which in turn were removed by jack hammers. The concrete was then cleaned by sand blasting in preparation for the RSLMC overlay. The Dow Chemical Company provided the latex (Dow-Modifier A Latex) and early advising and assistance. Later the Kentucky DOT hired Dow as a technical consultant on the project until they became comfortable with the use of the RSLMC.

Table 7.1. Bridge General Descriptive Parameter Values.

Parameter	Values	
Structure No.	B177	B178
Location	I-65 NB To I64 EB & WB	I64 EB & WB To I-65 SB
Year Built	1965[a]	1963[b]
Age	32 years[c]	34 years[c]
Length	424'	320'
Number of Spans	6	5
Span Lengths	-	-
Width	35.5'	≈ 53' (tapers)
Traffic	1 way	1 way
Number Traffic Lanes	2	3 → 2
Right Shoulder Width	≈ 6'	≈ 6'
Left Shoulder Width	≈ 2'	≈ 2'
Structure Type	Steel	Steel
Deck-Girder Design	Composite	Composite
Girder Spacing	≈ 8'	≈ 8'
Deck Thickness	≈ 6" - 8"	≈ 6" - 8"
ADT	123,000[d] (1995)	-
% Trucks	-	-

[a]Overlayed with LMC (1¼") in August, 1986.
[b]Overlayed with LMC (1¼") around 1985.
[c]As of 1997
[d]Appears to be too large.

Highway Structures, Inc., from Louisville,KY was the contractor for the entire project and Mid American Bridge Co. from Lexington, KY was the specialty RSLMC overlay subcontractor. Highway Structures was contracted by the Kentucky DOT to overlay 12 bridges in Spaghetti Junction in Louisville with a total deck surface area of 14,820 square yards for a total cost of $2.07 million. The cost was all inclusive from traffic control to final overlay striping, and translated into a unit cost of $140 per square yard or $15.50 per square foot. Highway Structures, Inc. bid a unit price of $770/yd^3 for the cost of the concrete in place ($400/yd^3 for the material supplier and $370/ yd^3 for placement and finishing). The project included 845 yd^3 of RSLMC for a total project cost of $650,650 for the RSLMC in place. The remainder of the project cost was in old concrete removed, deck preparation, lane striping, traffic control, etc. The time frame for the work was August 1 - November 3, 1997.

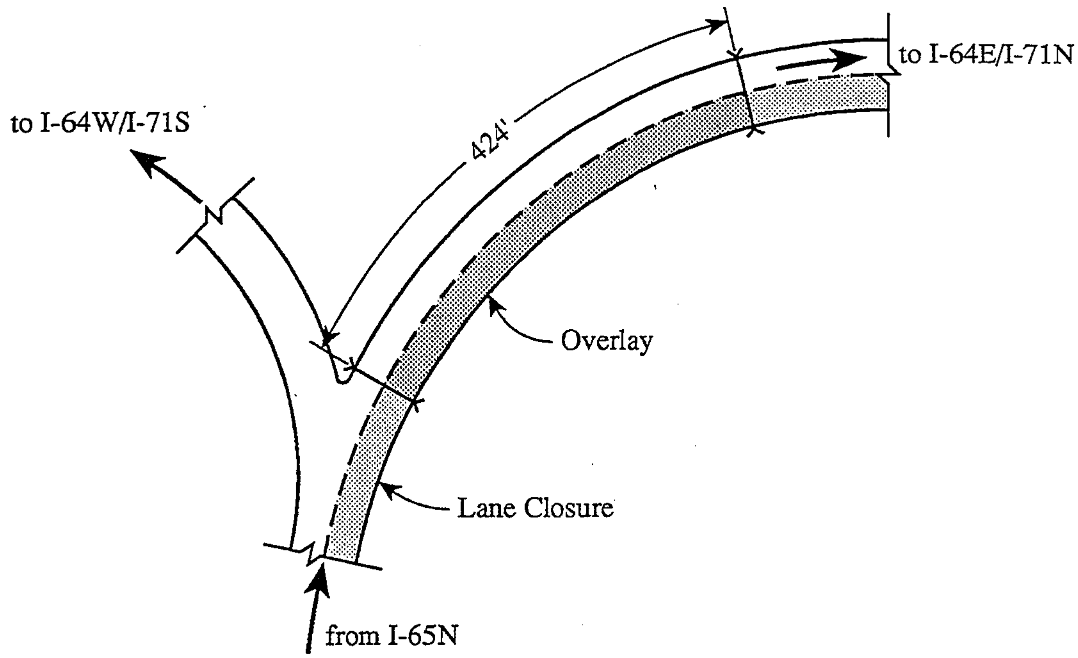
Because of the importance of the I-65-I64/I71 interchange and the heavy traffic load that it carries, the work had to be done in stages under concurrent traffic conditions. Basically this consisted of restricting the overlay work to weekends only, i.e., from 9:00 p.m. on Fridays until 5:00 a.m. the following Monday. Although the contractor had until 5:00 a.m. Monday mornings, the bridges were typically reopened to traffic on Sunday afternoon or evening.

The bridges in the "Spaghetti Junction" Project are typically 2 lane and the construction staging consists of closing and overlaying 1 lane each weekend (280' - 430' of lane length). The contractor worked simultaneously on 2 bridges each weekend (with the same lane closure/traffic control set-up being used for both bridges wherever possible). The staged construction sequence and lane closures to accomplish the September 19 - 21 overlaying are shown in Figures 7.2 and 7.3.

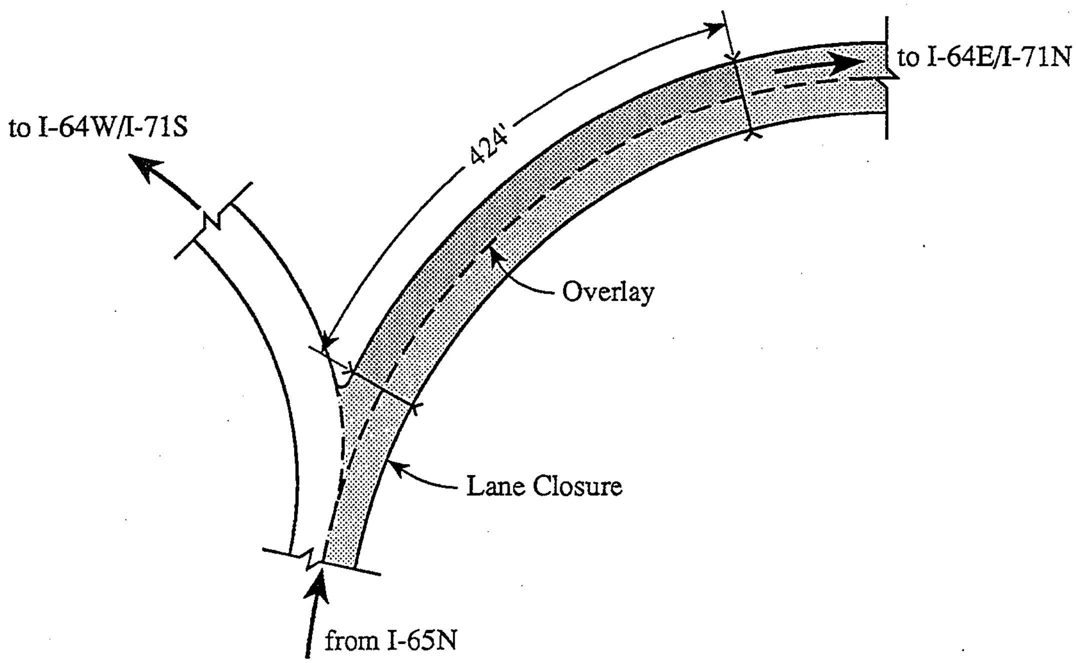
Traffic control consisted of appropriate signage, orange traffic control cones placed well "up-traffic" from the work site and at the work site, and a police car at the front of the work zone for the duration of the period of lane closure.

The primary construction sequence and tasks for each weekend during the overlaying work were as follows:

1. Place the traffic control system, move supplies and equipment onto bridge, and set-up lighting.
2. Remove top 1¼" of deck concrete (old overlay) by rotamilling.
3. Sound deck after rotamilling with drag chains to locate delaminations and deteriorated concrete and mark with paint.
4. Remove concrete at delaminations and other areas of deterioration via a Galion (a grader mounted rotamill) and jack hammers.

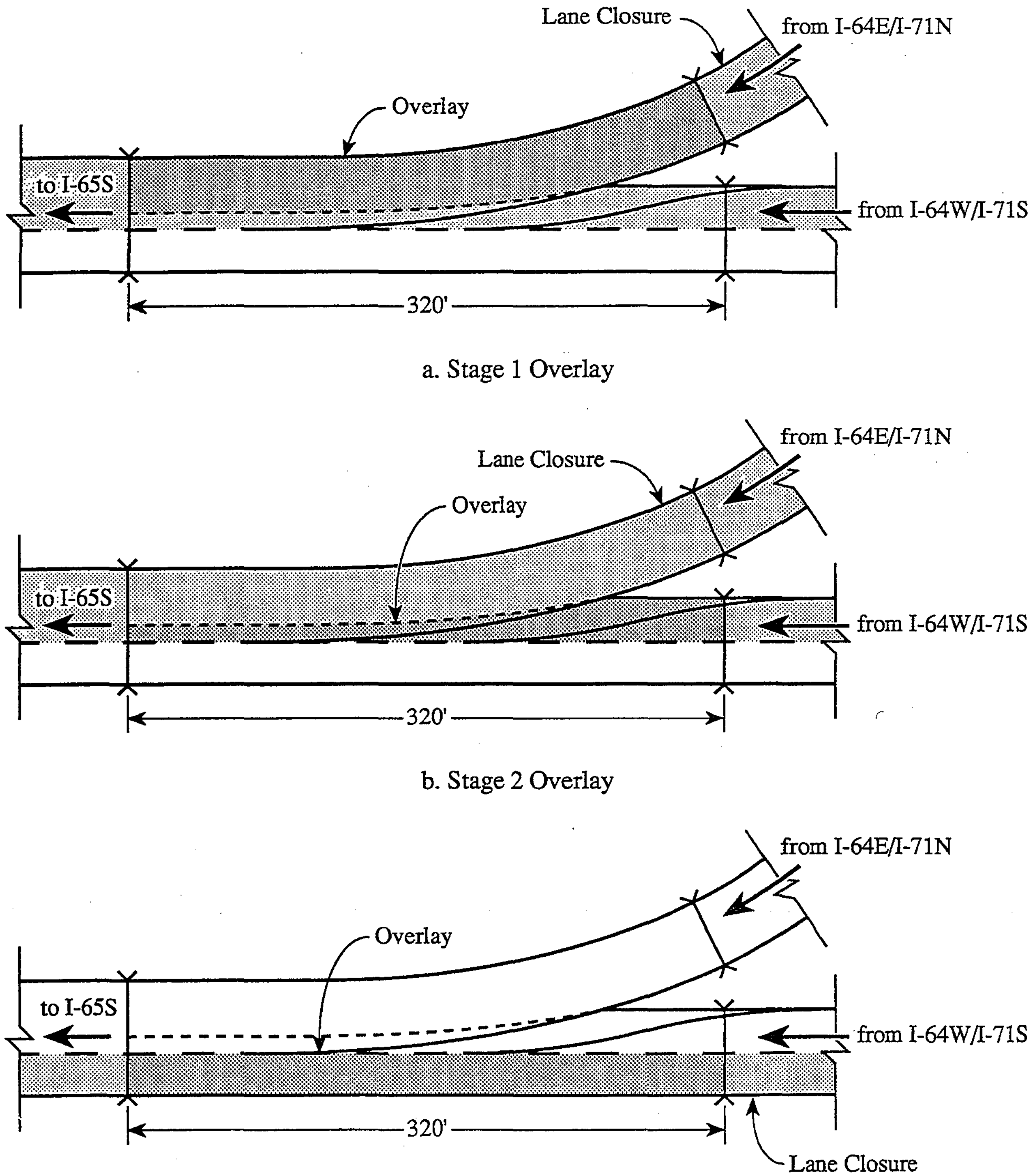


a. Stage 1 Overlay



a. Stage 2 Overlay

Fig. 7.1. Staged Construction for B177 Bridge Deck Overlaying.



a. Stage 1 Overlay

b. Stage 2 Overlay

c. Stage 3 Overlay

Fig. 7.2. Staged Construction for B178 Bridge Deck Overlaying.

5. Check marked areas after jack hammering for delaminations and remark and re-jack hammer as necessary.
6. Clean deck surface to be overlayed via sandblasting and then blowing clean with air.
7. Set rails for and prepare self-driving Bidwell vibratory screed by making a dry run.
8. Overlay deck with a 1½" rapid setting latex modified concrete (RSLMC) mixture using a mobile concrete mixer.
9. Place quality 12-hour wet curing system.
10. Put away materials and equipment and clean the deck.
11. Remove deck curing system (after approximately 12 hours) and finish clearing and cleaning the deck.
12. Place deck lane striping.
13. Remove traffic control system and reopen all lanes.

Photographs showing the overlay work in progress during Stages 1 and 2 on the B177 and B178 bridges respectively are presented and discussed in the following section.

7.2 Photographic Display and Discussion of Deck Overlay Work

A photographic display of the state of deck deterioration prior to overlaying, the overlaying process in progress, and the resulting bonded RSLMC overlay are shown in Figures 7.4 - 7.26. Photographs were taken at both bridge sites (B177 and B178), and the best photos are presented below without attempting to distinguish which bridge. The down side of this is that if one compares some of the "shots," it appears that they are of different bridges which they are. The up and dominating side is that they provide a fuller and more complete photographic presentation of the RSLMC overlaying process, which is the subject matter of interest.

Typical deck damages are shown in Figures 7.3 and 7.4 and are caused by overlay delaminations which in turn are caused by the entry and later freezing of water. Once delaminations occur, the pounding of traffic results in the damage shown in Figures 7.3 and 7.4. Figures 7.3 and 7.4 represent the extreme in damage just prior to overlaying, but there were numerous patches that reflected this same type of damage in previous months or years. Interestingly, the overlays on many of the lanes were in excellent condition as can be seen in the right lane (in the photograph) in Figure 7.5 and a close-up of the same in Figure 7.6.

Figures 7.7 and 7.8 show the milling off of the old 1¼" LMC overlay which is begun immediately after setting up the traffic control and lighting for the bridge work area. The machine removed a strip approximately 3' wide as it moved longitudinally along the bridge and simultaneously deposited the removed concrete in a trailing dump truck. The removal was quite rapid as small sections of the old overlay material would tend to debond under the severe loading of the scarifying/milling drum and teeth. The old overlay was removed from an area of approximately 18' x 424' in about two hours.

Figure 7.9 shows jack hammer removal of concrete at corners, regions adjacent to expansion joints, etc. concurrently with the milling removal (note the far right strip of the old overlay in Figure 7.10 has not yet been milled off). Figure 7.10 shows a deck sweeping and vacuum truck cleaning the deck after finishing of the milling work. After milling and cleaning, the deck was completely sounded by drag chains to detect and mark delaminations. This work is shown in progress in Figure 7.11. It should be noted that all jack hammering work was stopped during the sounding so that the inspectors could hear good and properly locate all delaminations.

After the delaminations were marked, the contractor began removing the delaminated concrete via use of a Galion (a motorized road grader with a milling attachment) and jack hammers. The Galion was used on large areas and areas out in the open, and jack hammers were used on small areas, adjacent to joints and curbs, and in finishing some areas where the Galion was used. Figures 7.12 - 7.14 show the results of local delamination removals. It should be noted that KYDOT requires where rebars are exposed to one-half the depth of the bar, that the old concrete be removed from around the bar to a depth sufficient to get new concrete completely around the bar. In many instances the contractor violated this requirement.

After removal of local delaminations, KYDOT inspectors resounded the deteriorated areas to assure removal of all delaminated concrete. The contractor then removed any further identified concrete. After removal of all deteriorated concrete the deck was sandblasted and then air blasted to remove small laiances and small particles as shown in Figures 7.15 and 7.16.

Next, the Bidwell screed and rails are set-up as indicated in Figures 7.17 and 7.18, and a dry run is made to assure proper transverse alignment and vertical elevations will be achieved. Note in Figure 7.18 the vertical saw cut left edge of the lane prepared for overlaying. KYDOT requires a 3" width of the new overlay be saw cut out when the adjacent overlay is placed to assure getting a quality vertical construction joint at the interface of the two overlay placements. KYDOT also requires that the overlays be placed at night (7:00 p.m. is the earliest allowed beginning time) when ambient temperatures are low (and relative humidities are high). One of the bridge overlays (B178) was placed from 10:00 p.m. Saturday - 1:00 a.m. Sunday and the other overlay (B177) was placed from 1:30 a.m. - 5:30 a.m. Sunday. After the screed has made its dry run, the prepared deck is hosed down with water and covered with a layer of polyethylene. The polyethylene serves to prevent the water from evaporating and to prevent "drippings" from the mobile concrete mixer truck from getting in the prepared deck surface.

Figures 7.19 and 7.20 show the beginning of placement of the first RSLMC overlay (B178). Note in Figure 7.20 that a slurry of the liquid components of the RSLMC is being broomed into the old deck surface immediately in front of the placement of the RSLMC by two "broomers" (the Bidwell screed is just to the right of the concrete shown in the photo). Figure 7.21 shows the work from the other side (from Figure 7.20) and shows the "broomers" along with the mobile mixing truck and the polyethylene being rolled up behind the rear wheels of the mixing truck as it moves forward.

Figure 7.21 also shows the surface finishing which consisted of simply hand troweling along the side edges (after the wet burlap drag of the Bidwell screed), and then tining of the surface. Figure 7.22 shows the placement of the wet burlap following closely behind the tining, and Figure 7.23 shows the covering with polyethylene close behind the wet burlap. Thus, as can be seen in Figures 7.21 - 7.23, the placement, screeding, finishing, tining, covering with wet burlap and polyethylene are proceeding very close to each other with little elapsed time in between. Given that the overlay placement proceeded at a rate of approximately 100 feet per hour, the various task above were proceeding in the order of minutes, perhaps 5 minutes, between each other. Figure 7.24 shows the newly placed overlay with curing system in place. KYDOT specs call for the curing system to remain in place for 24 hours; however, because they are achieving very early high strength (approximately 4000 psi in 6 hours) they have apparently relaxed this as the contractor removed the curing system after only 8 hours.

Figure 7.25 shows the new overlay (middle lane) the next day immediately after removal of the curing system. The surface looked excellent. In walking and closely examining the overlay, no cracking whatsoever could be identified. Also, in walking the overlay placed the week before (the left lane in Figure 7.25 and the ramp shown in Figure 7.26), not a single crack could be located. It should be noted that the discolorations in the left lane in Figure 7.25 are simply wet concrete dust and dirt pushed onto that lane while working on the middle lane. It should also be noted that there appeared to be very little cracking in the old overlays (other than at delamination locations), and very little cracking in the original deck based on visual inspection of the prepared deck surface prior to placing the new overlay.

As a side note, KYDOT's overlay mixture is a cement rich mixture (about 7 bags per yd^3) using maximum size crushed limestone of around ¼" and attaining compressive strengths of around 4000 psi in 6 hours. KYDOT Resident Engineer, Rob Harris, monitored the RSLMC surface temperature (with a Ray-Tech infrared temperature instrument) for the first couple of hours. The results are shown in Figure 7.27. The increase of surface temperature to a maximum of about 93° F (a $\Delta T_{max} \approx 22$°F) occurred about 1 hour after placement at the approximate time of final set of the concrete.

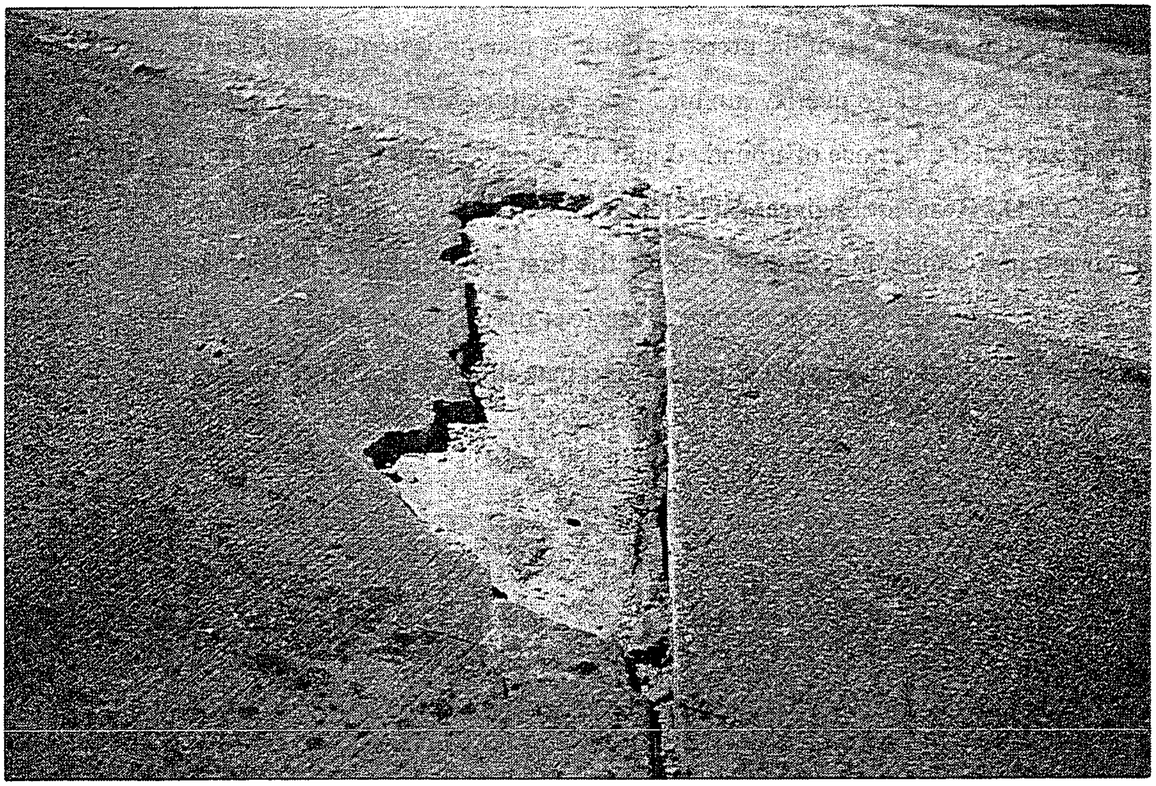


Fig. 7.3. Typical Deck Damage at Joints

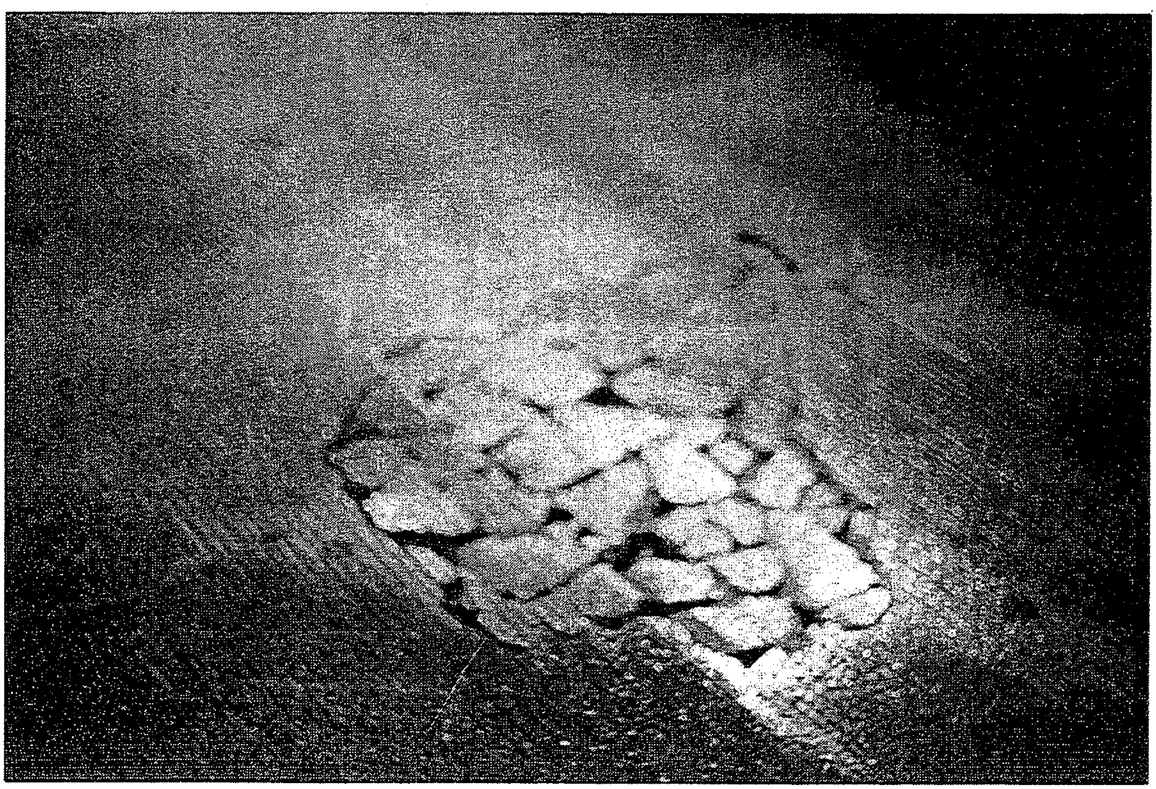


Fig. 7.4. Typical Deck Damage Away from Joints

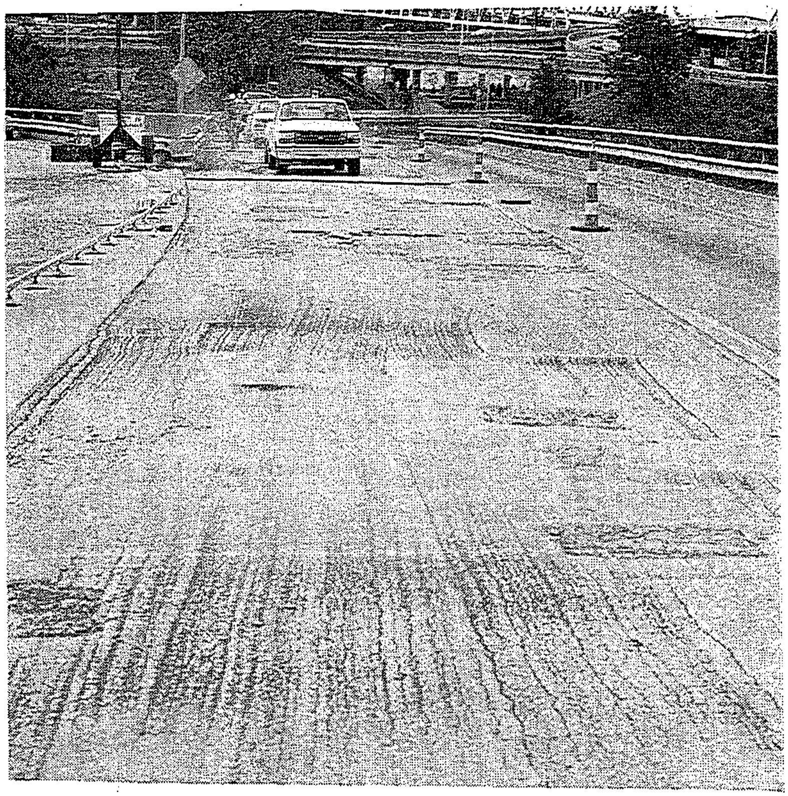


Fig. 7.5. Nondamaged Old Deck Overlay in Right Lane

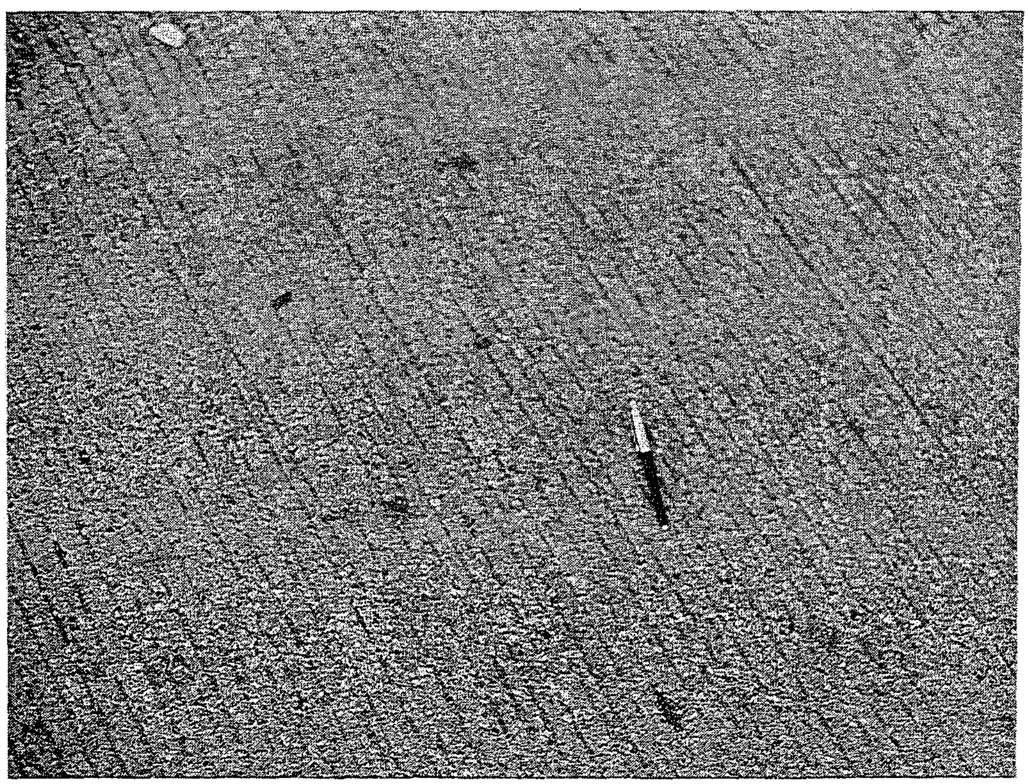


Fig. 7.6. Close-up of Nondamaged Old Deck Overlay

Fig. 7.7. Removal of Old Overlay with Milling Machine

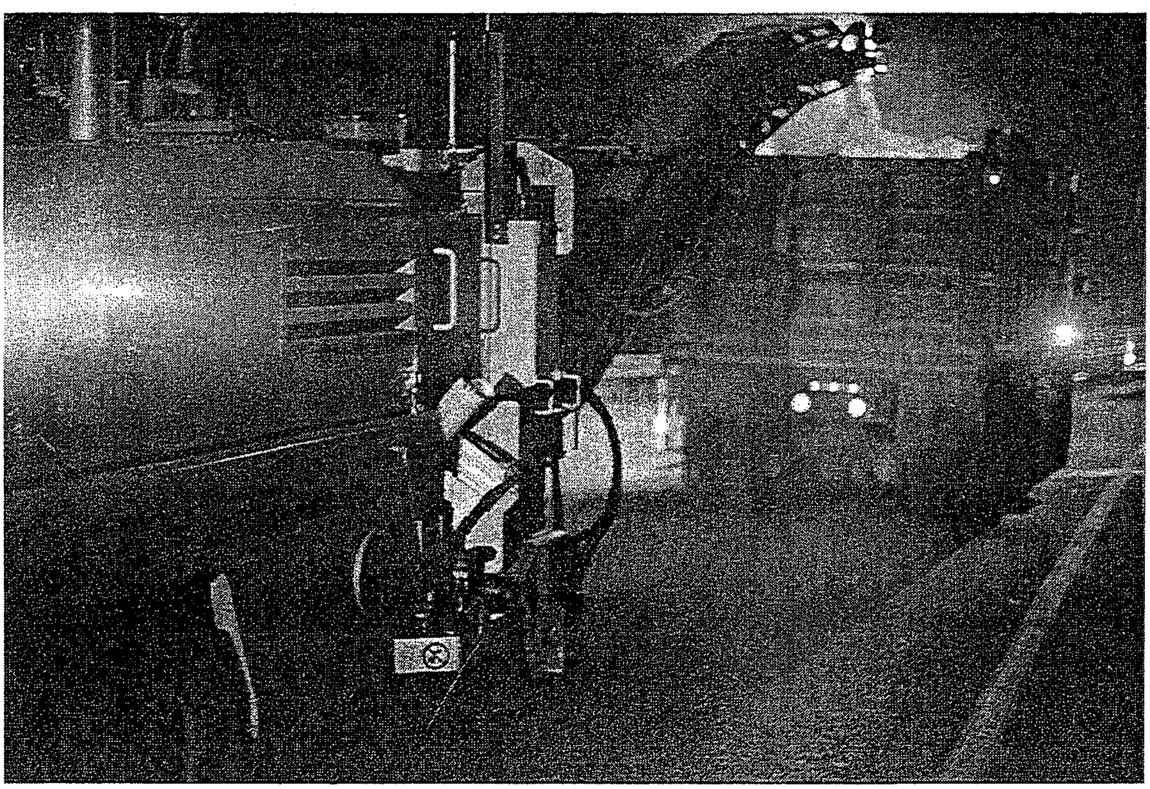


Fig. 7.8. Milling Machine Loading Dump Truck as it Mills

Fig. 7.9. Jack-hammering Proceeding Simultaneously with Milling

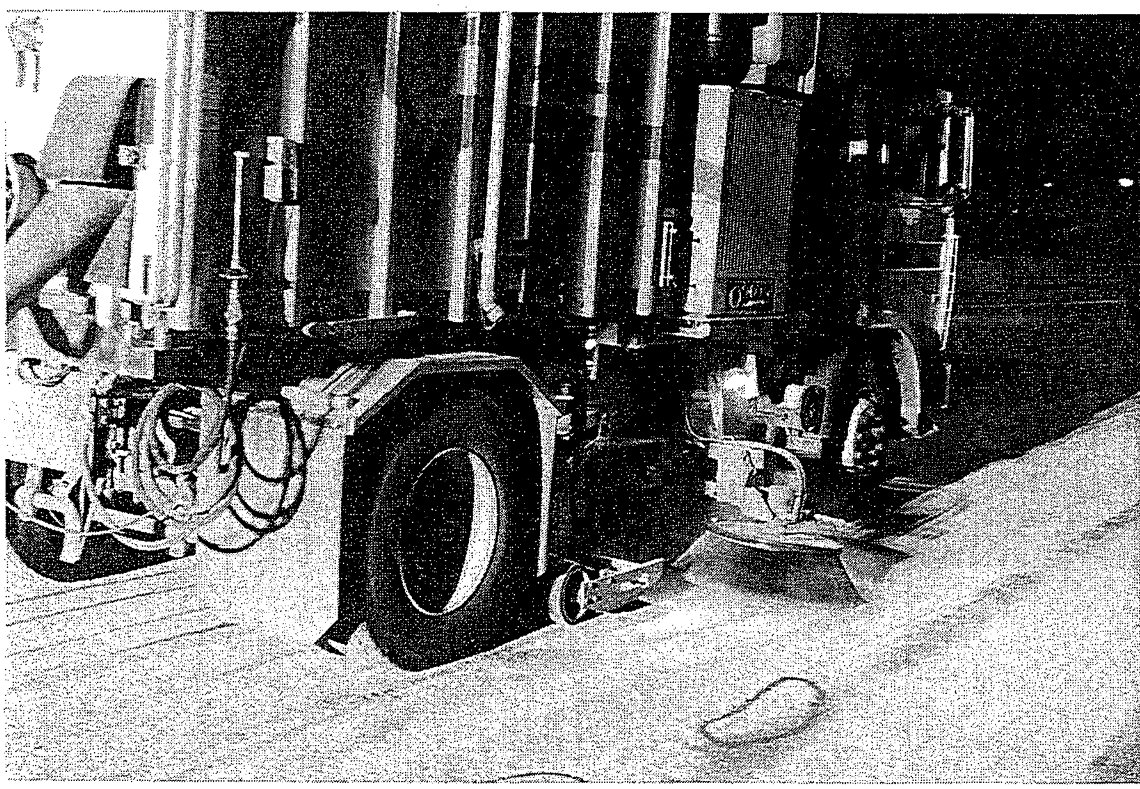


Fig. 7.10. Deck Sweeper and Vacuum Truck Cleaning Deck

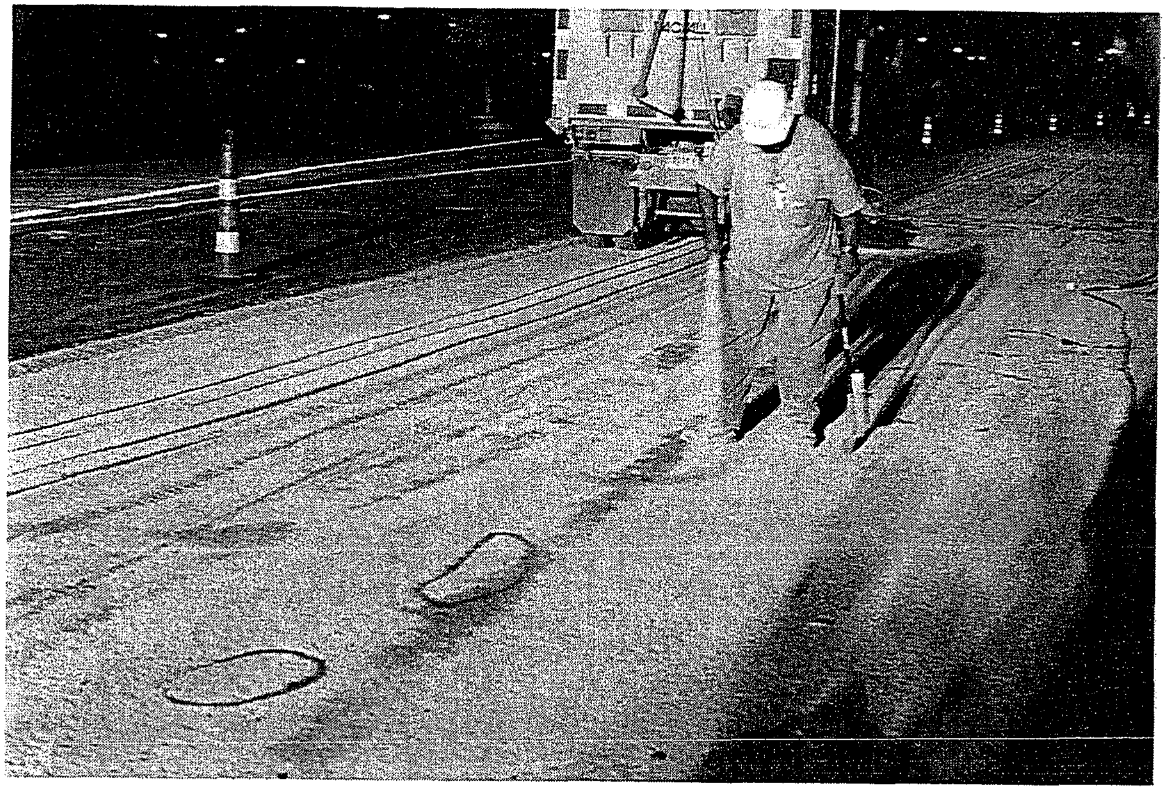


Fig. 7.11. KYDOT Inspector Locating and Marking Delamination Areas

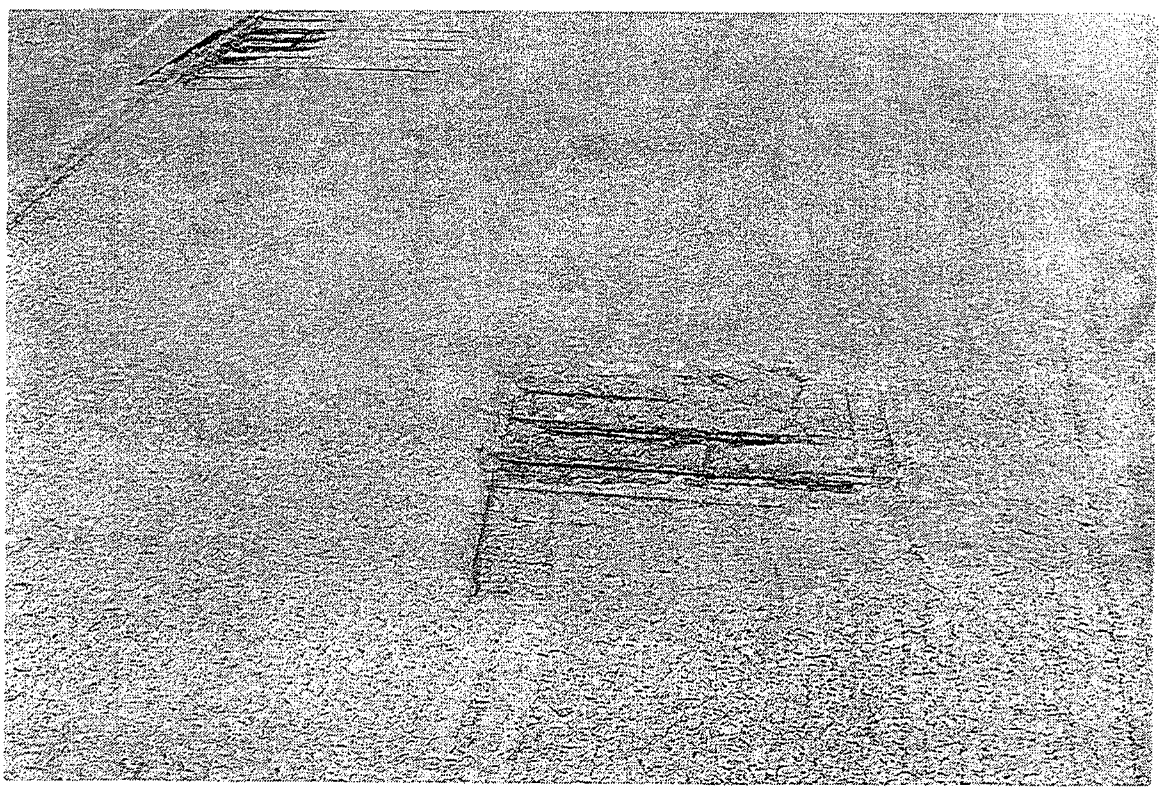


Fig. 7.12. Deck After Milling and Spot Delamination Removals

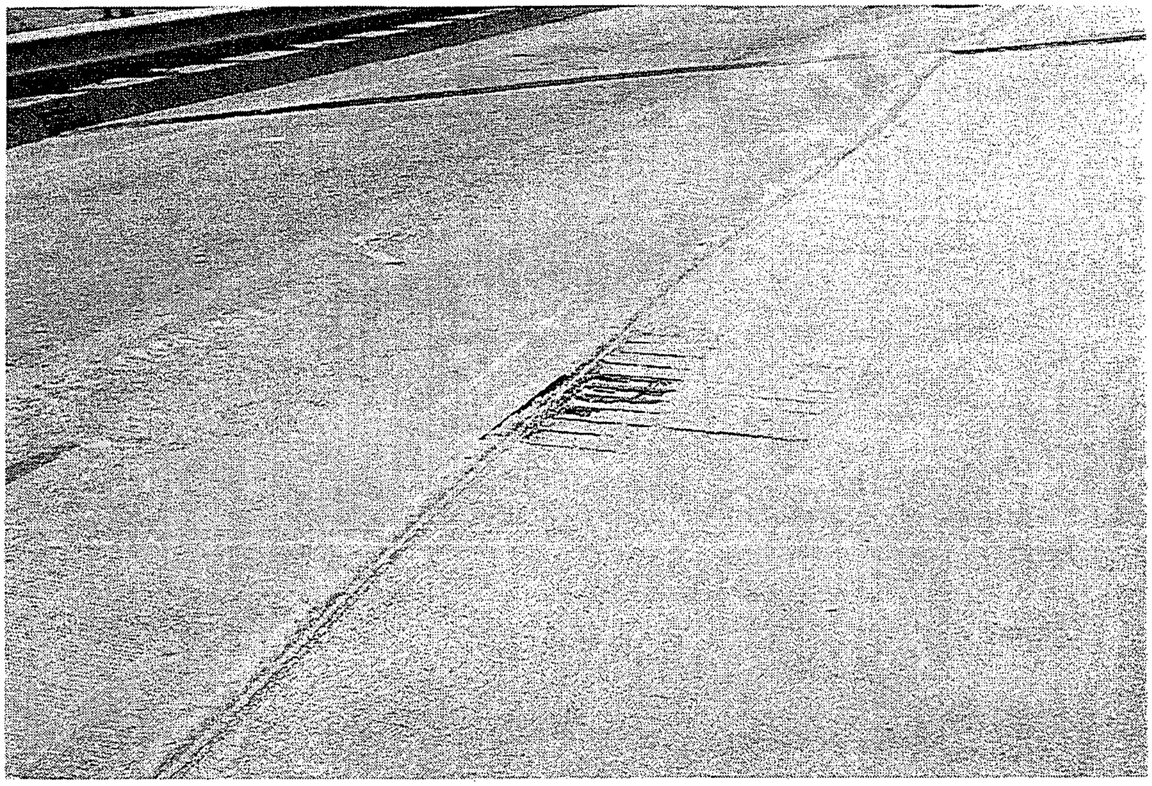


Fig. 7.13. Old Overlay (Left) and Deck Prepared for New Overlay (Right)

Fig. 7.14. Local Delamination Prepared Area

Fig. 7.15. Sandblast Cleaning of Prepared Deck

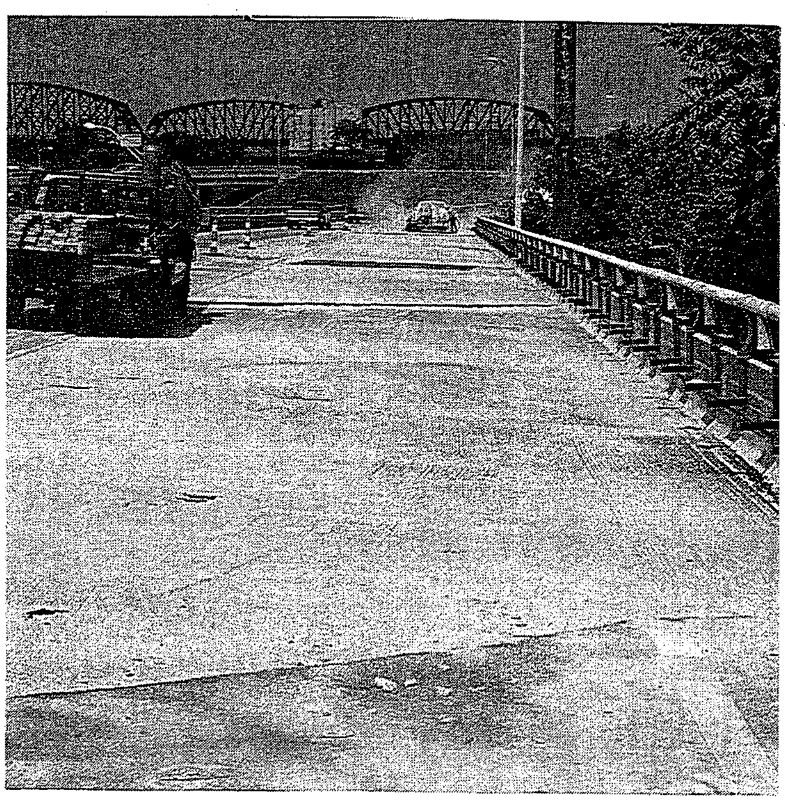


Fig. 7.16. Deck About Ready for New Overlay

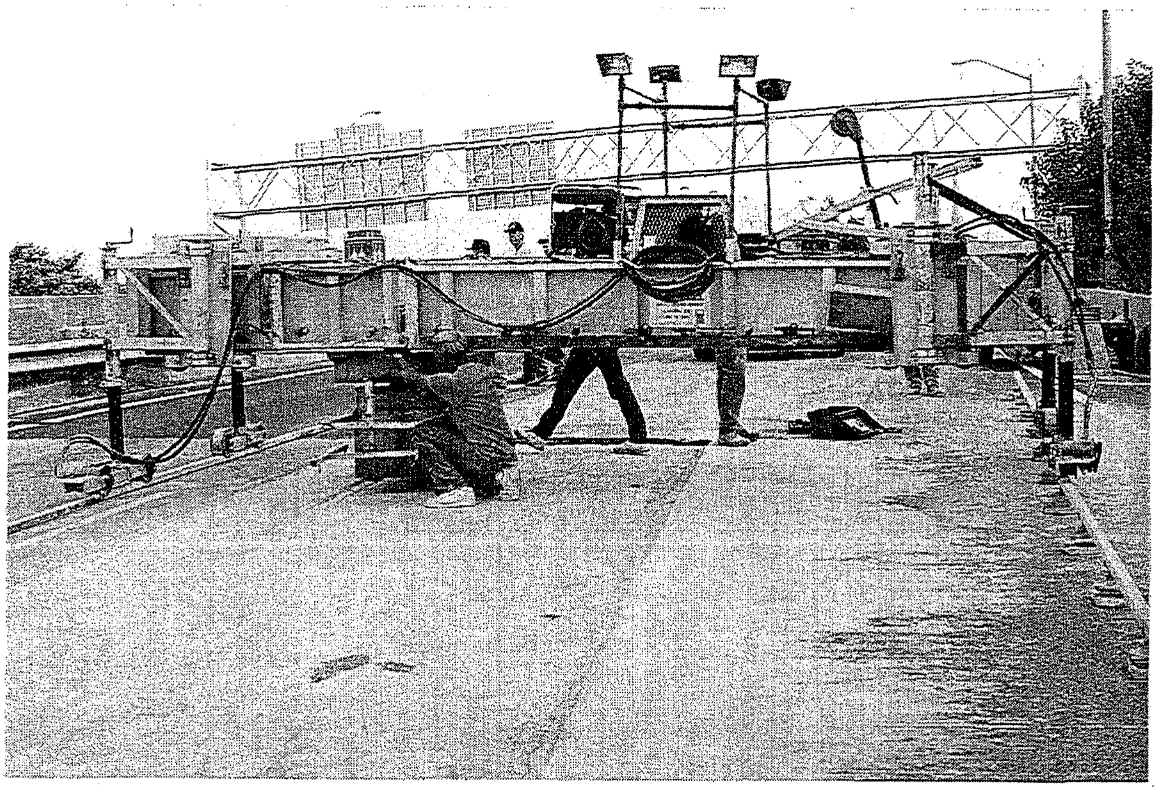


Fig. 7.17. Bidwell Screed Being Set-up

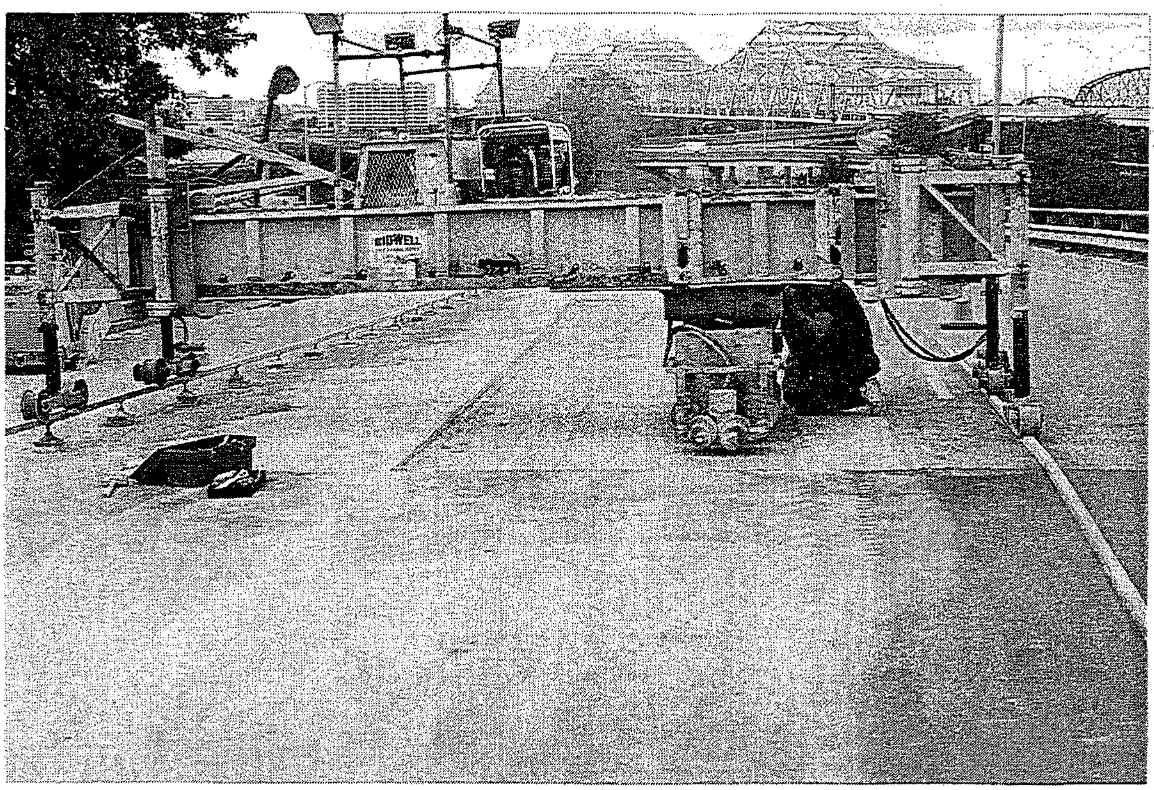


Fig. 7.18. Bidwell Screed Being Prepared for Dry Run

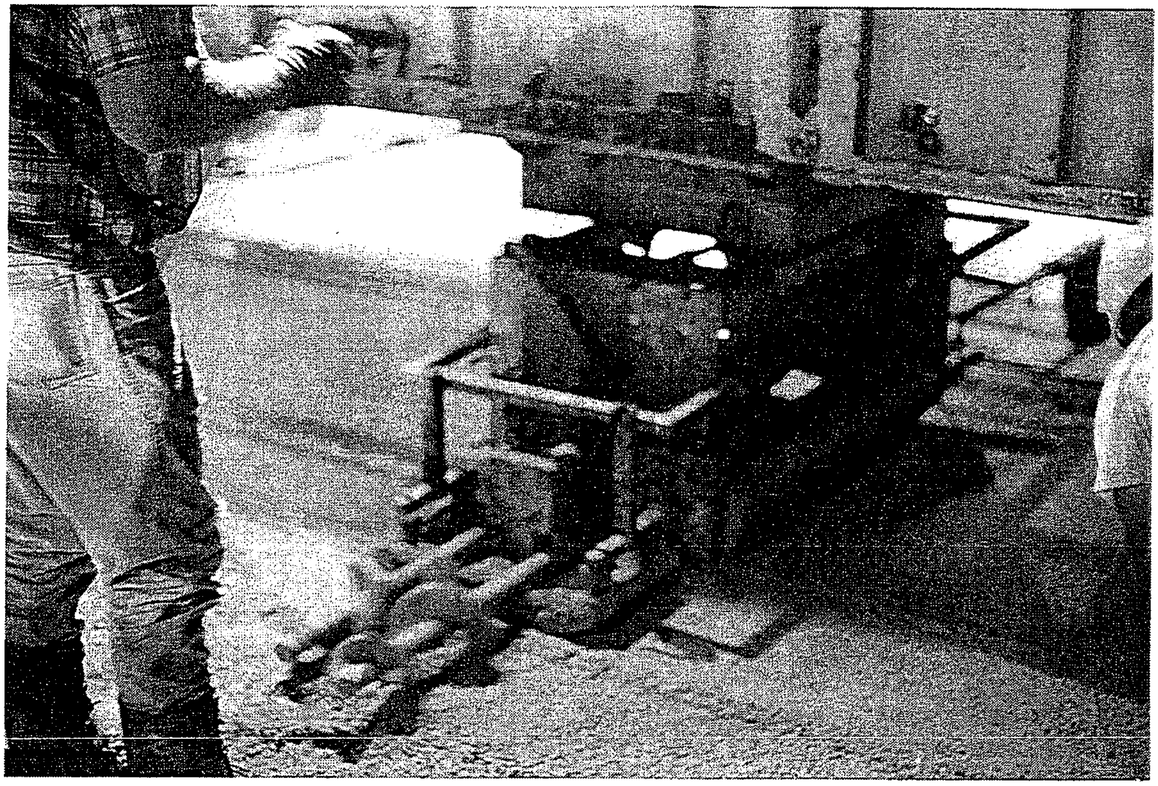


Fig. 7.19. Beginning of Placement of Overlay

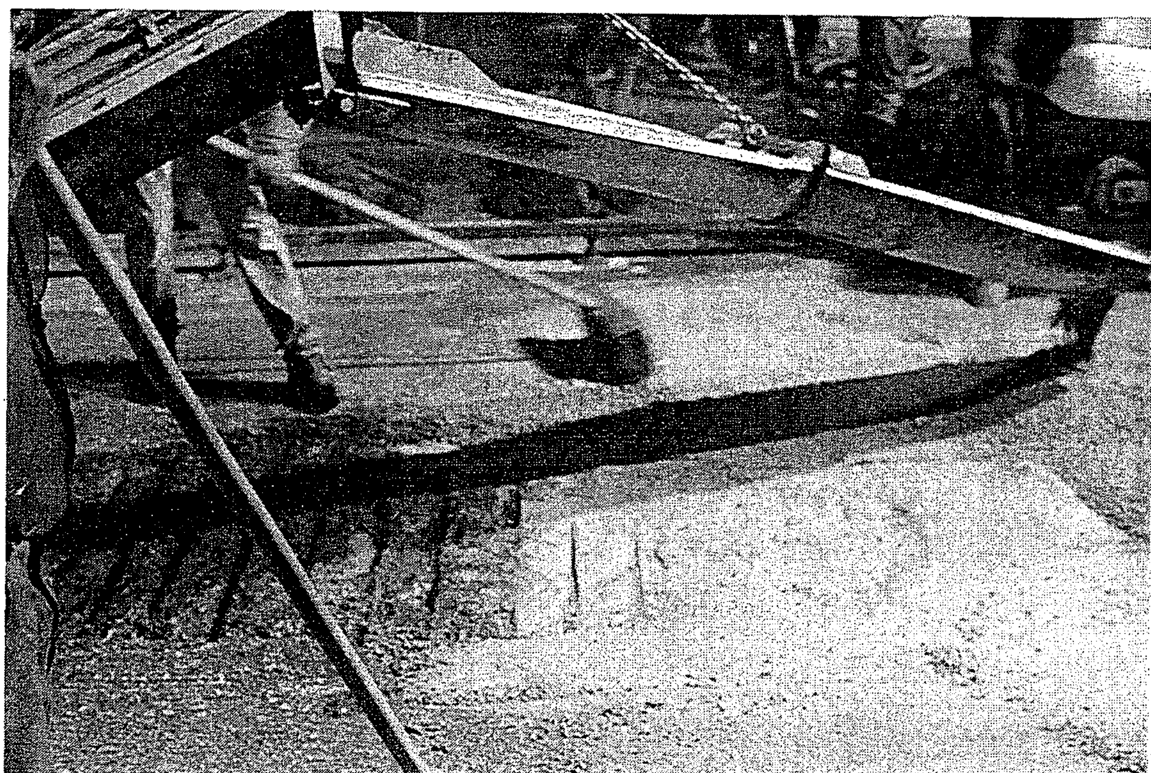


Fig. 7.20. Placement of RSLMC

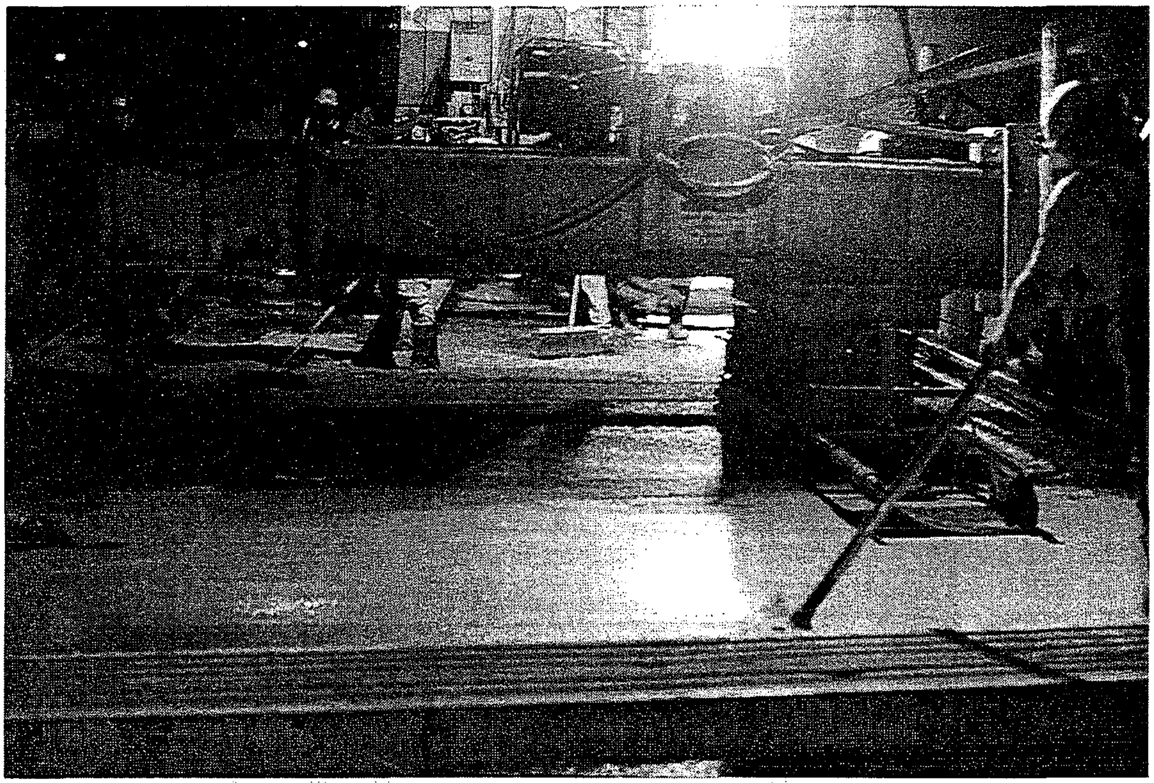


Fig. 7.21. Placing, Screeding and Tining of Overlay

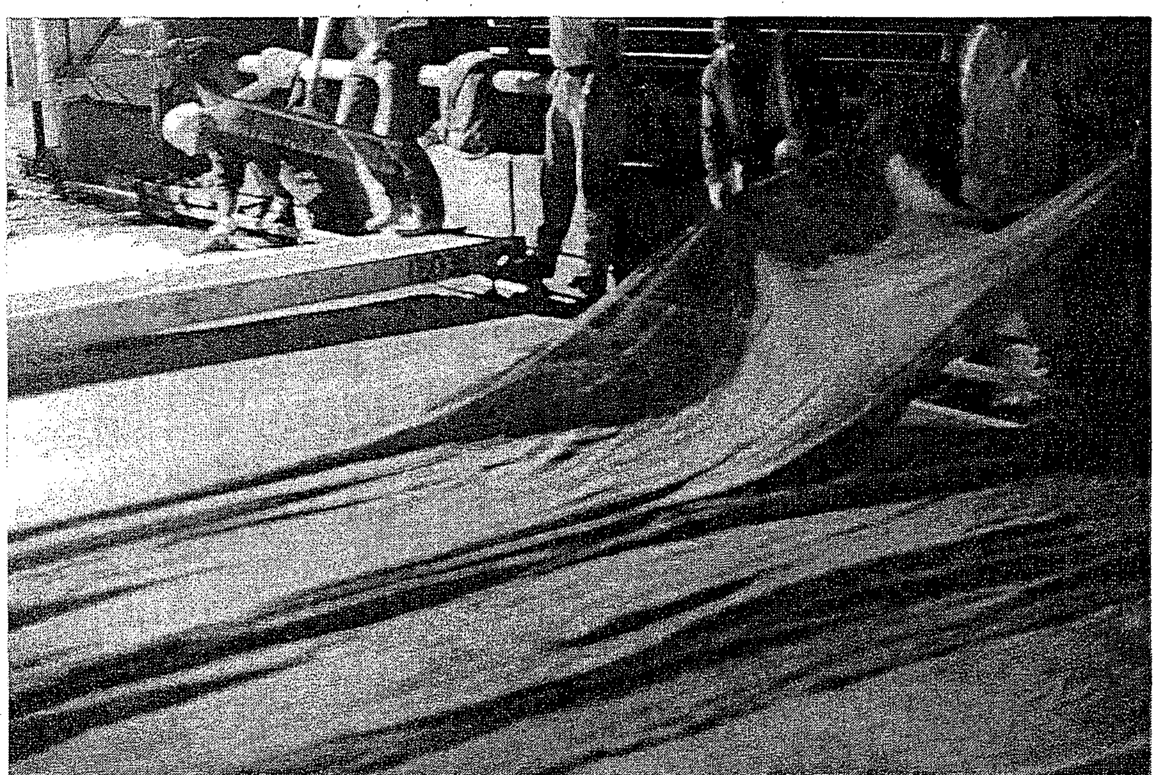


Fig. 7.22. Covering of Overlay with Wet Burlap

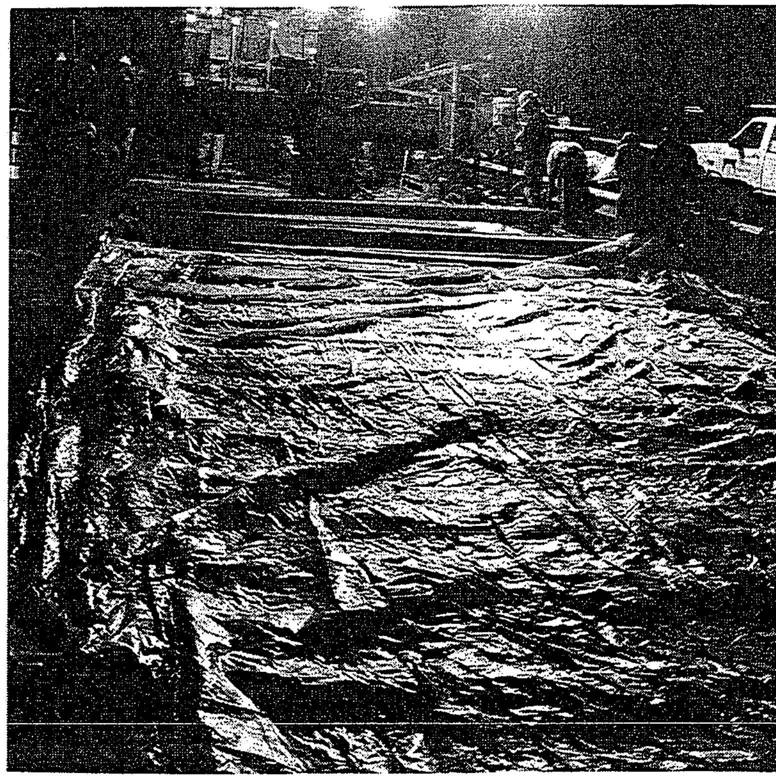


Fig. 7.23. Covering of Wet Burlap with Polyethylene

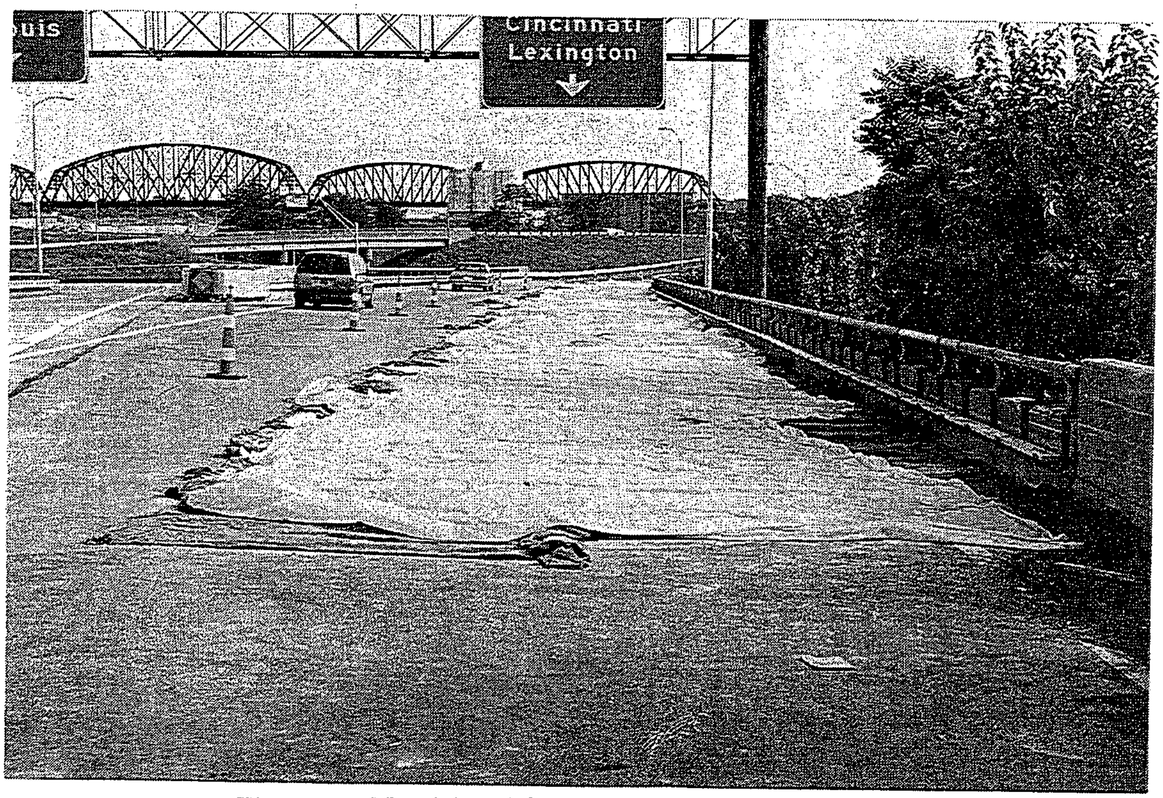


Fig. 7.24. New Placed Overlay During Curing Process

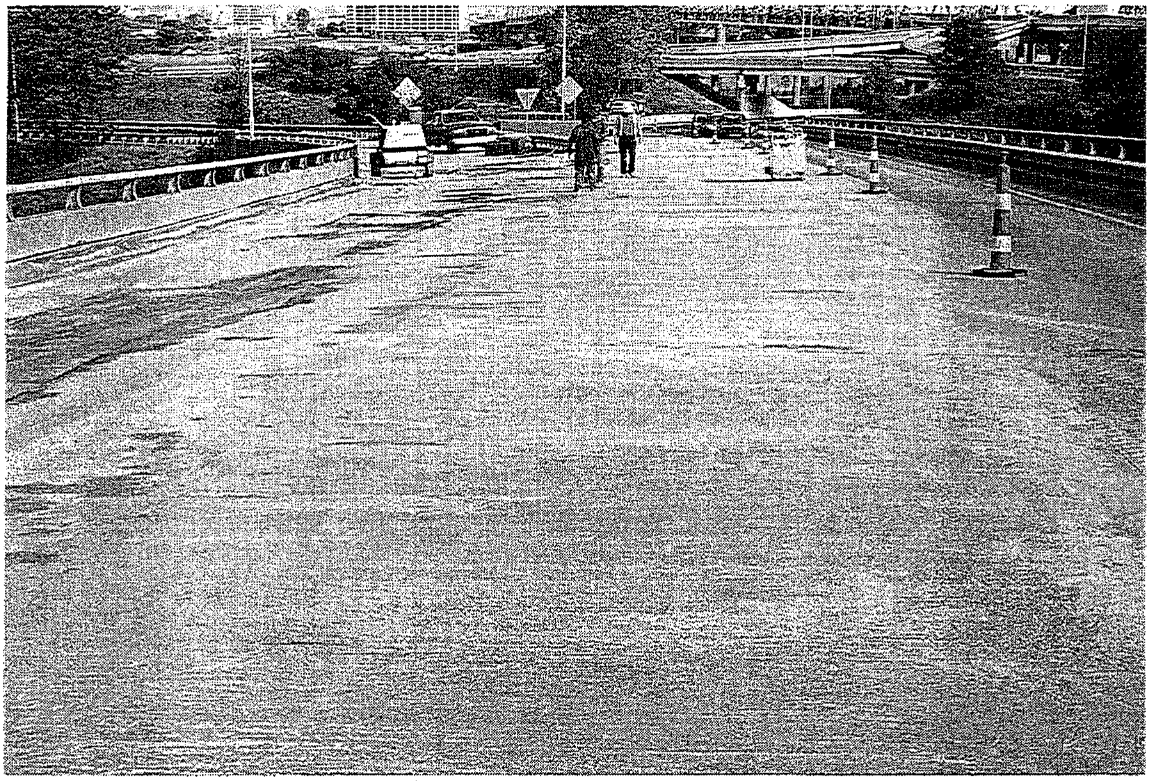


Fig. 7.25. New Overlay (Middle Lane) After Removal of Curing Covering

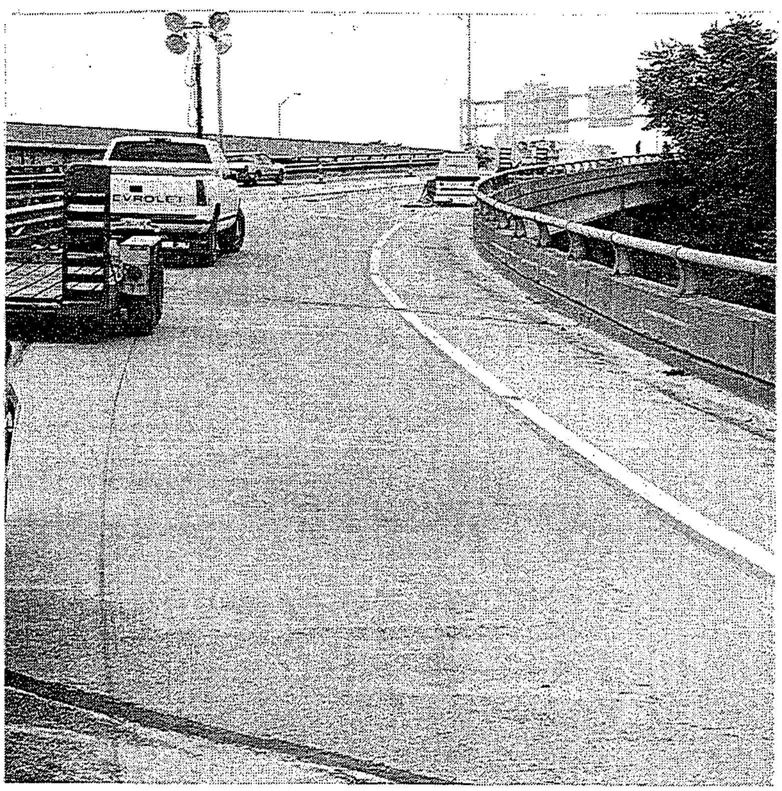


Fig. 7.26. Overlay 1-2 Weeks After Placement

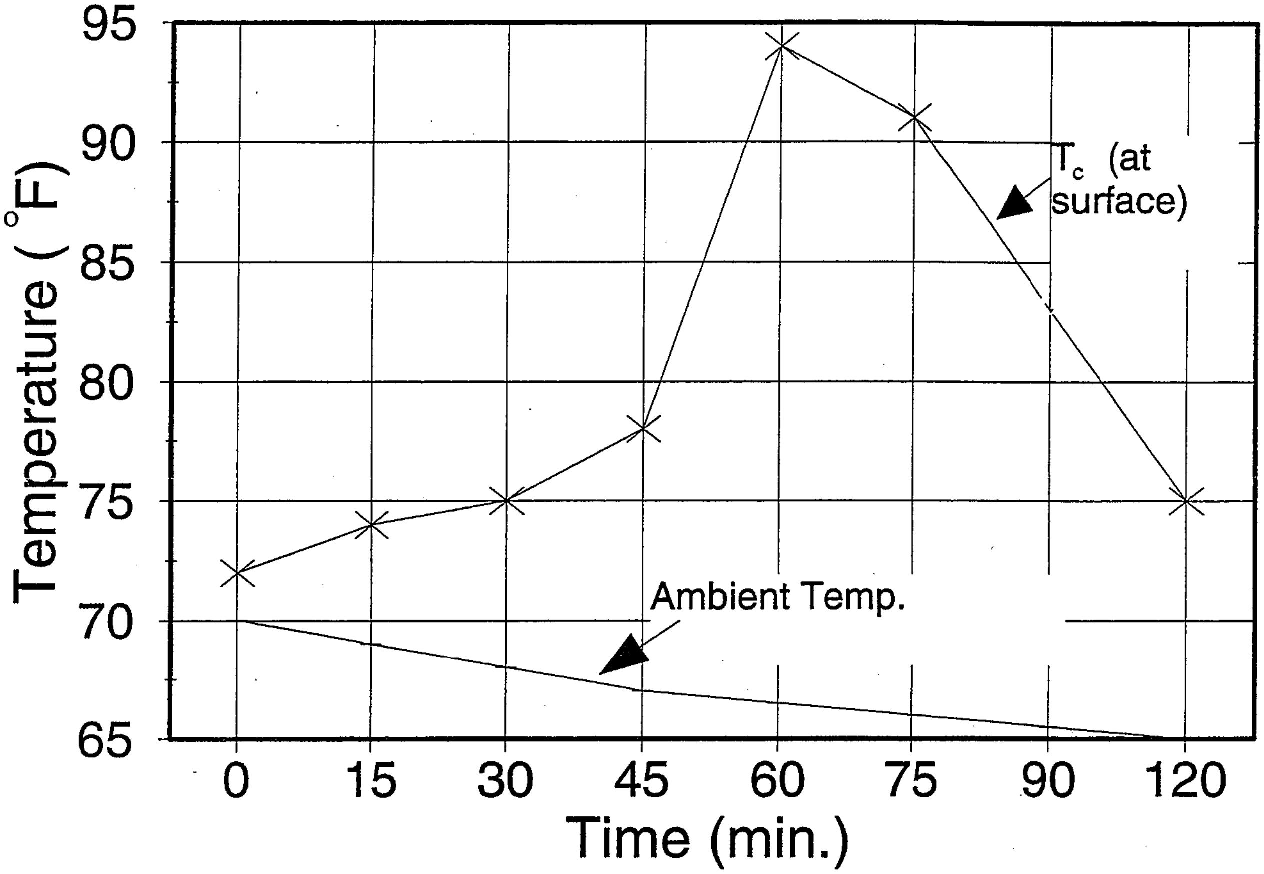


Fig. 7.27. RSLMC Early Surface Temperature Variations

8. CALIFORNIA DOT POLYMER CONCRETE BRIDGE DECK OVERLAY CONSTRUCTION SEQUENCE

8.1 General

On the weekend of October 3-5, 1997, Dr. Ramey flew to San Francisco, California to visit the job site of two bridge deck overlay projects on I-80 near Oakland, California as a guest of the California Department of Transportation (Caltran) and Atlas Construction Supply, Inc. The primary descriptive parameters for the bridges are shown in Table 8.1. As evident from Table 8.1, the original concrete bridge decks had service lives of 37 and 42 years before requiring an overlay.

Romero Construction, a California overlay speciality contractor, was the construction contractor for the deck overlaying project, and Atlas Construction Supply, Inc., was the materials' supplier for the polyester polymer concrete (PPC) overlay materials. Atlas indicated that the PPC materials cost to Caltran was \$30 - \$35/ft^3, which for a ¾" thick overlay translated to \$2.00 - \$2.25/ft.^{2} of overlay. Romero was capable of placing as much as 20,000 square feet (1666 linear feet of 12' wide lane) of overlay a night; however, they normally placed 2 lanes of overlay a night, with a total square footage less than 20,000 ft^2.

Unfortunately, early in the morning of October 3, a DUI driver penetrated the orange traffic control cones and killed one of Romero's deck preparation workers at the job site. As a result, Romero suspended overlay work until the following week. Ramey could not be notified of the fatality and construction plan change until he arrived on Friday night. Though unable to observe an overlay placement, he was able to visit with personnel involved with the project and gather information on PPC overlays, Caltran's requirements, and the planned overlay construction sequence. Also, Ramey was able to arrange for appropriate photographs of the overlaying process to be sent to him.

Because of the importance of the I-80 route and the heavy traffic load that it carries, the work had to be done in stages under concurrent traffic conditions. Basically this consisted of restricting the overlay work to nights only, i.e., from 8:00 p.m. until 5:00 a.m. the following day. The contractor on the project visited did his overlay work on Tuesday, Wednesday, Friday, and Saturday nights. Typically, two lanes were overlayed each night of construction, and each lane of overlaying is considered a construction stage as traffic control systems needed to be changed. For example, the contractor may execute Stages 1 and 2 (see Fig. 8.1) during the Friday night work, and Stages 3 and 4 during the Saturday night work. Also, the contractor would work simultaneously on 2 bridges each night (with the

Table 8.1. Bridge General Descriptive Parameter Values

Parameter	Value	
Structure No.	33-0127	33-0051L
Location	04-Ala-80-6.62-Berkeley (I-80 near Berkeley)	04-Ala-80-7.20-Albany (I-80 near Albany)
Year Built	1955	1960
Age[1]	42 years	37 years
Length	310'	2112'
Number of Spans	5	31
Span Lengths	2 @60', 1 @ 65', 2 @ 60'	Various: 126' max, 17' min.
Width	119.8'	Mainline 46' min., Ramp 33' min.
Traffic	2-way	1-way
Number Traffic Lanes	8	3 mainline, 2 ramp
Right Shoulder Width	3'	don't know
Left Shoulder Width	2'	don't know
Structure Type	RC Box	Combination RC "T" girders and welded steel composite girders
Deck-Girder Design	Composite	Composite
Girder Spacing	6' - 10½"	8' - 8" RC T's; 14' - 0" steel
Deck Thickness	6½"	8¾" RC T's; 11" steel
ADT	239,000	250,000
% Trucks	7%	7%

[1]As of 1997

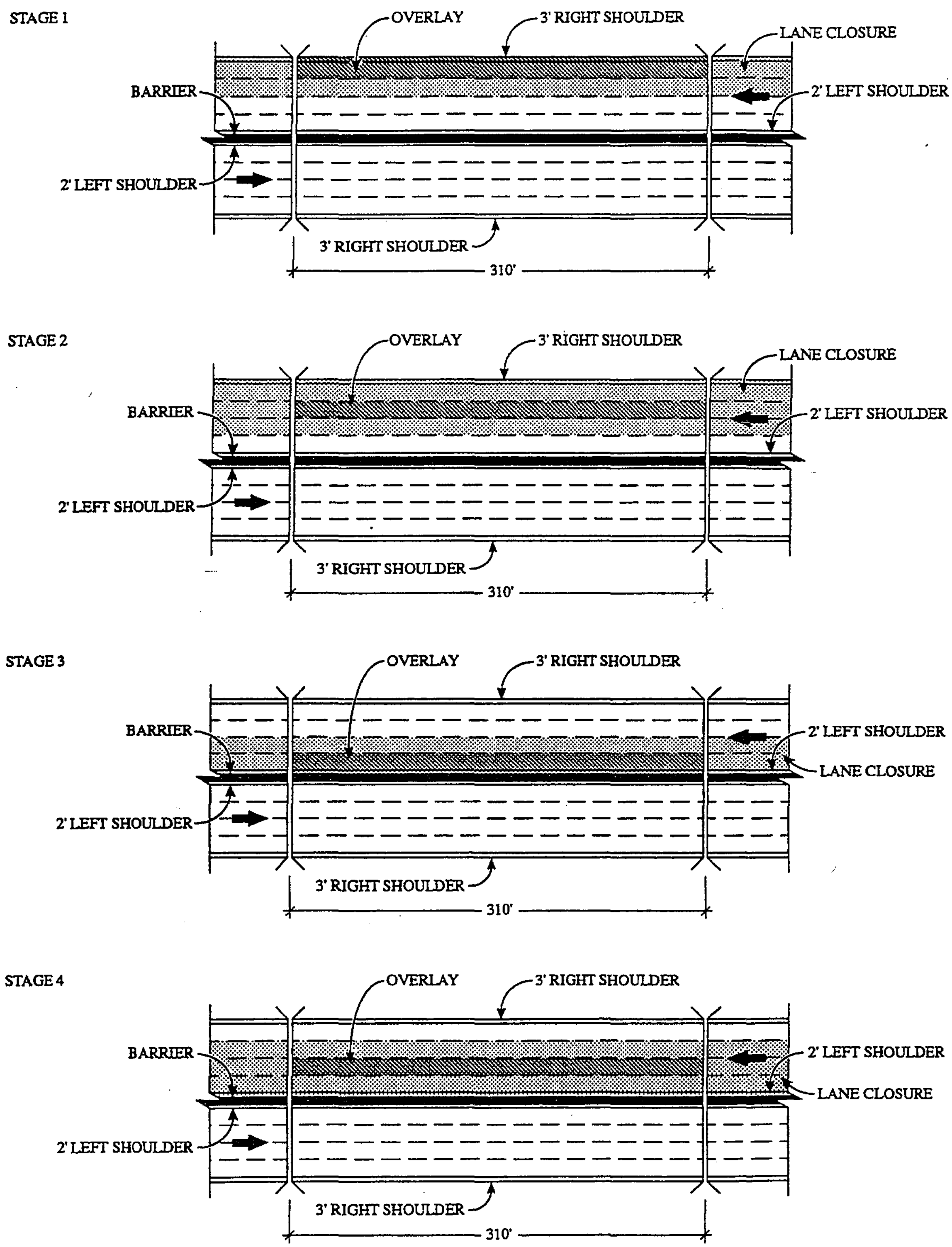


Figure 8.1. Staged Overlay Construction Sequence for Caltran I-80 Bridge Near Berkeley.

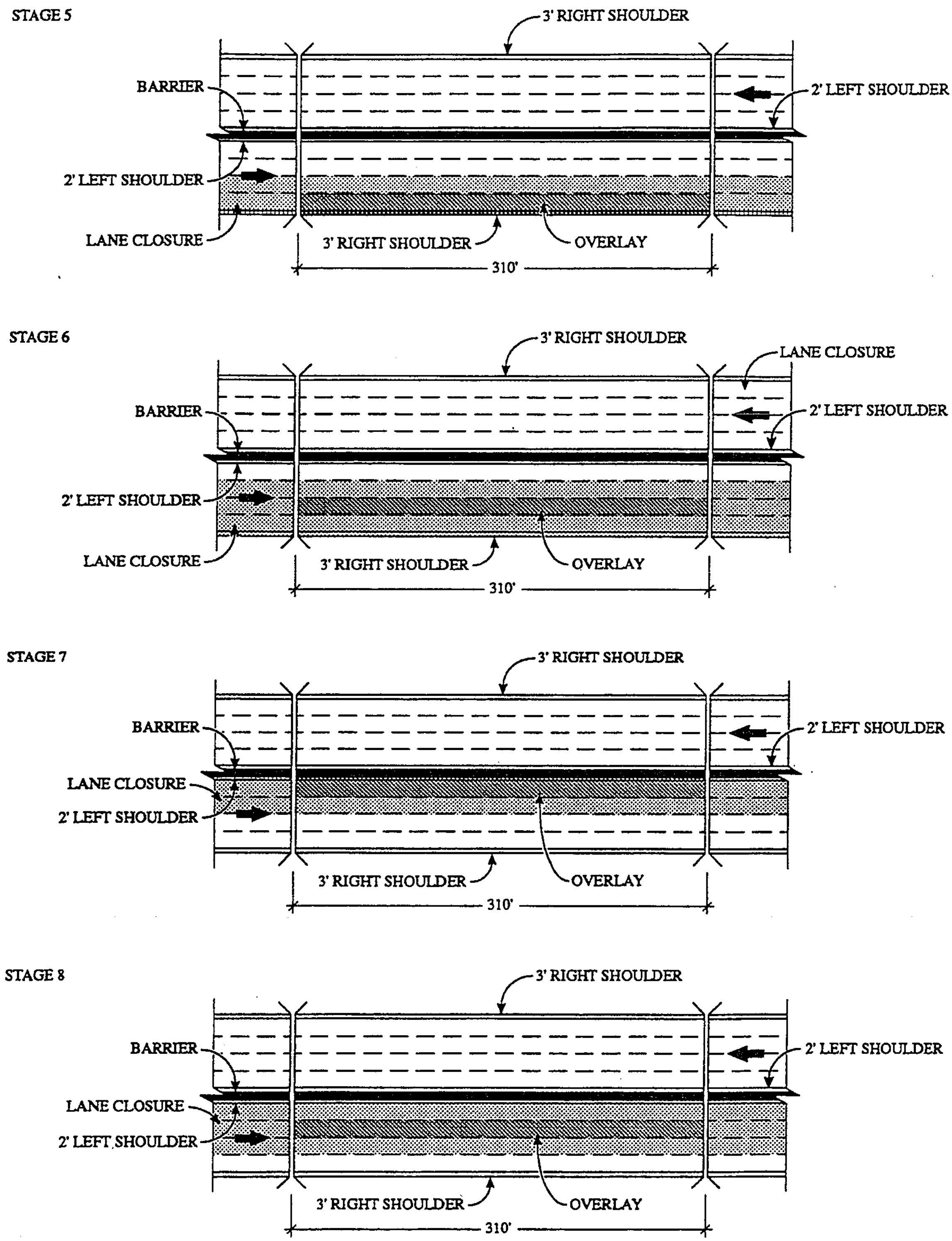


Figure 8.1. Staged Overlay Construction Sequence for Caltran I-80 Bridge Near Berkeley (Continued).

same lane closure/traffic control set-up being used for both bridges) wherever possible. The planned staged construction sequence and lane closures to accomplish the overlay work on the I-80 bridge near Berkeley are shown in Figure 8.1.

Discussions with Caltran engineers indicated that Caltran has extensively researched overlays in general and polymer concrete overlays in particular. They have looked closely at polyester, epoxy, and acrylic based (resin) polymer concrete and chose polyester polymer concrete (PPC). They made this choice primarily because PPC's lower viscosity (relative to epoxy) allows greater penetration of the base concrete and superior bonding, and it allows a higher charge of aggregate to be used and thus a lower cost overlay concrete. The Caltran polyester polymer concrete employs styrene (to decrease viscosity) as part of the polymer and this is the cheapest of all resins/binders. The polyester they use has superior tensile elongation properties relative to the acrylics and superior lower viscosity relative to the epoxies. Caltran engineers indicated that their overlay aggregate is essentially a ⅜" pea gravel, and because of this they chose to use a ¾" overlay thickness in order to achieve aggregate on top of aggregate in the overlay.

Caltran indicated that they probably have placed 100-200 deck or highway slab overlays over the past 12 years, and have had only 2 failures. One was a highway slab project which exhibited some debonding at joints and corners after 10 years of service life. The other was a bridge deck overlay which had a raveling problem after only 1 year. A post mortem analysis revealed that the overlay contained only 4% resin binder rather than the 10-12% recommended, i.e., the failure was the result of a construction problem. Thus, Caltran's PPC overlay performances have been excellent.

Caltran indicated that proper deck preparation is very important in placing a bonded overlay. They have found the sandblasting is not sufficient to establish an excellent bond surface. Thus, they require shotblasting to establish a clean (while concrete) surface as a minimum. In some instances, they require diamond grinding or scarification to remove the top ¼" (typically) of concrete, and then shotblasting. To help assure an excellent bonding of the overlay, they also require the use of a methyl methacrylate as a surface primer prior to placing the overlay.

Caltran provided the following average cost data for the 1994 year for PPC overlays. They pay by the cubic yard to supply PPC and by the square yard to place PPC.

To supply PPC: $1375/yd^3$ or $50.90/ft^3$

To place PPC: $37.62/yd^2$ or $4.18/ft^2$

For a ¾" thick overlay, this yields,

To supply PPC:	$28.65/yd^2
To place PPC:	$37.62/yd^2
Total:	$66.27/yd^2 (1994)

Assuming a 3% inflation rate per year would result in a 1997 cost of

Total: $72.42/yd^2 (1997)

The primary construction tasks and sequence during Caltran's deck preparation work are as follows:

1. Place traffic control system and lighting to inspect bridge deck and identify the state of concrete deterioration and mark areas requiring concrete removal.
2. Remove local deteriorated concrete as necessary and patch with rapid setting material.
3. Prepare deck surface for overlays by either shotblasting, or grinding with diamond blade saws or scarifying the deck.
4. Reopen prepared lane(s) to traffic.

The primary construction sequence and tasks for each night of closure during the actual overlaying work are as follows:

1. Place the traffic control system, move supplies and equipment onto bridge, and set-up lighting.
2. Shotblast deck surface to remove all laitance and surface contaminants.
3. Set edge forms (¾" plywood or other) for overlay lane as necessary.
4. Set-up small pan bottomed vibratory screed.
5. Prime deck surface with high molecular weight methyl methacrylate to seal/heal deck cracks and to enhance bonding of the overlay. Allow minimum of 15 minutes and maximum of 2 hours between placing primer and placing polymer concrete overlay.
6. Overlay one lane of deck with a ¾" polyester polymer concrete (PPC) mixture using a mobile concrete mixer. For small deck projects, portable mortar mixers may be used in lieu of a mobile concrete mixer.
7. Approximately 15 minutes after screeding with a pan vibratory screed, broadcast sand on surface of overlay.
8. Allow overlay to chemically/self cure for a minimum of 4 hours.
9. Move traffic control set-up to close the next adjacent lane in preparation for overlaying that lane.
10. Repeat Tasks 3 - 8 above for the second lane.
11. Sweep/vacuum, or blow away excess sand from the deck.
12. Clear and clean the deck surface.
13. Place temporary deck lane stripping.
14. Remove lighting and traffic control system and reopen all lanes.

As indicated above, Caltran uses ¾" thick PPC overlays almost exclusively at this time. For typical deck shotblasting preparation (as apposed to ¼" scarification), this results in an increase in deck dead load of approximately 8 psf. It also results in temporary longitudinal joints with a ¾" drop/set-up between a pair of adjacent lanes during the staged construction process. Also, during the construction process, adjacent lane overlays are placed directly against the previous placed overlay without requiring removal of any of the previously placed overlay.

The most common type of failure for bonded overlays is a bond failure. Thus, special attention to enhancing bond of the overlay to the base concrete is most appropriate. Actions to minimize overlay bond failure include,

1. Use overlay material with excellent bond strength to portland cement concrete.
2. Use bond enhancement material.
3. Use material with zero to low drying shrinkage.
4. Use material with thermal expansion characteristics similar to portland cement concrete
5. Use material with stiffness properties similar to base concrete and with good tensile strength and tensile elongation capacity.
6. Use material with low permeability to prevent water from reaching the bond line.
7. Properly mix, place, consolidate and finish the overlay material.
8. Properly cure the overlay before opening to traffic.

With the exception of item 4 above, i.e, similar thermal coefficient of expansion, polymer concrete probably meets the above requirements better than any material available.

It should be noted that the Ohio DOT believes that hydrodemolition of the deck surface is the best method of deck preparation. They believe the scarification of the surface causes micro cracking of the substrate. They have experienced overlay failures of delamination just below the bond line which they feel were probably caused by damage to the substrate during deck scarification procedures. They also like the fact that the hydrodemolition will remove any areas of poor quality concrete to a greater depth, and it will provide a rough and very clean deck bonding surface.

Mr. Floyd Dimmick of Thermal Chem Corporation emphasizes that proper surface preparation is essential for any bonded overlay. He recommends shot blasting the surface with a large shot blasting machine (6 foot cleaning width) using angular shaped steel shot. He indicated that rounded steel shot tend to polish the substrate aggregate, whereas angular shot remove the surface concrete to slightly different depths which enhances bonding of an overlay. He also indicated that using the angular shaped steel shot cost a small amount more (approximately 1 cent per square foot), but is well worth the slight cost increase.

Due to cancellation of the weekend work, the authors were not able to photograph actual overlay work in progress. However, arrangements were made with the polyester polymer concrete supplier, Atlas Construction Supply, Inc., to secure such photographs, and these are presented and discussed in the following section.

8.2 Photographic Display and Discussion of Deck Overlay Work

A photographic display of Caltran's deck overlaying process in progress, and the resulting bonded PPC overlay are shown in Figures 8.2 - 8.23. It should be that the photographs are not of one bridge project, but rather are from various recent projects in which the PPC supplier was involved. They are presented however, in the time sequence of the overlaying process. Also, rather than having a narrative description and discussion of each photograph here and referring to the appropriate figure as was done for the Georgia and Kentucky site visits, descriptions and discussions of the photographs are provided as part of the figure titles.

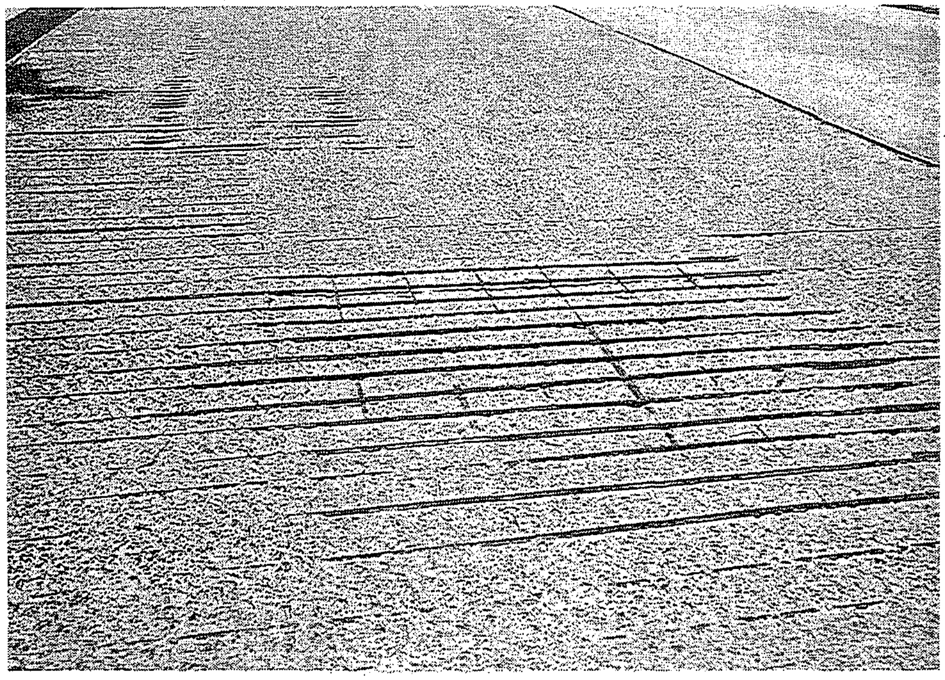


Fig. 8.2. Surface Preparation. One method of bridge deck surface preparation uses high pressure water jet hydrodemolition equipment at a minimum of 30,000 PSI.

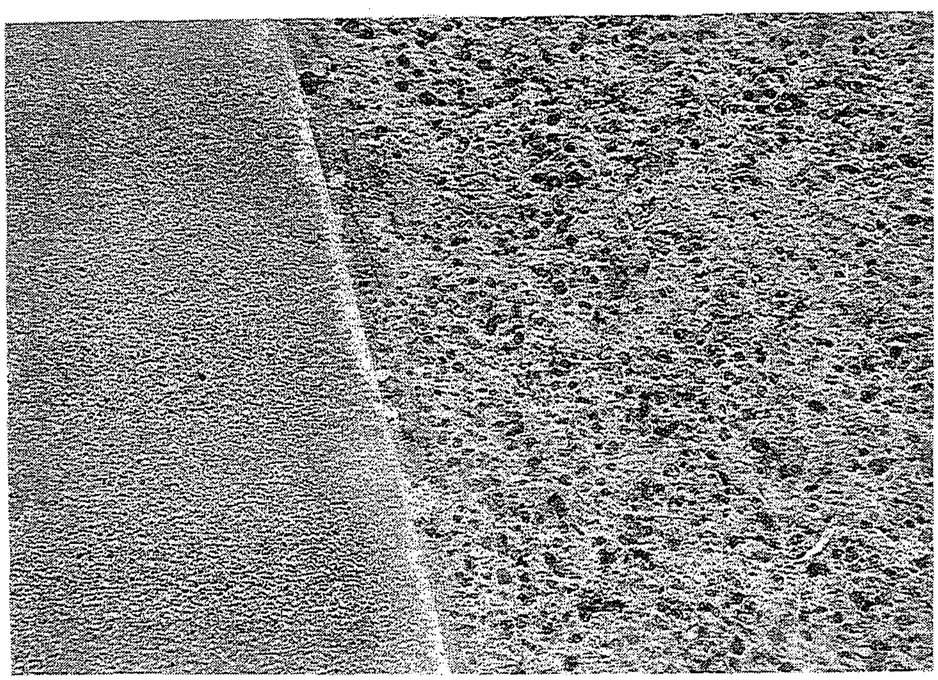


Fig. 8.3. Surface Preparation. Right side shows deck surface preparation using steel shot blasting equipment, and left side shows finished polyester concrete overlay in adjacent lane.

Fig. 8.4. Surface Spalls/Delams. Delaminated areas on bridge deck that have unsound concrete removed, but before being primed and filled with polyester concrete prior to starting the overlay process.

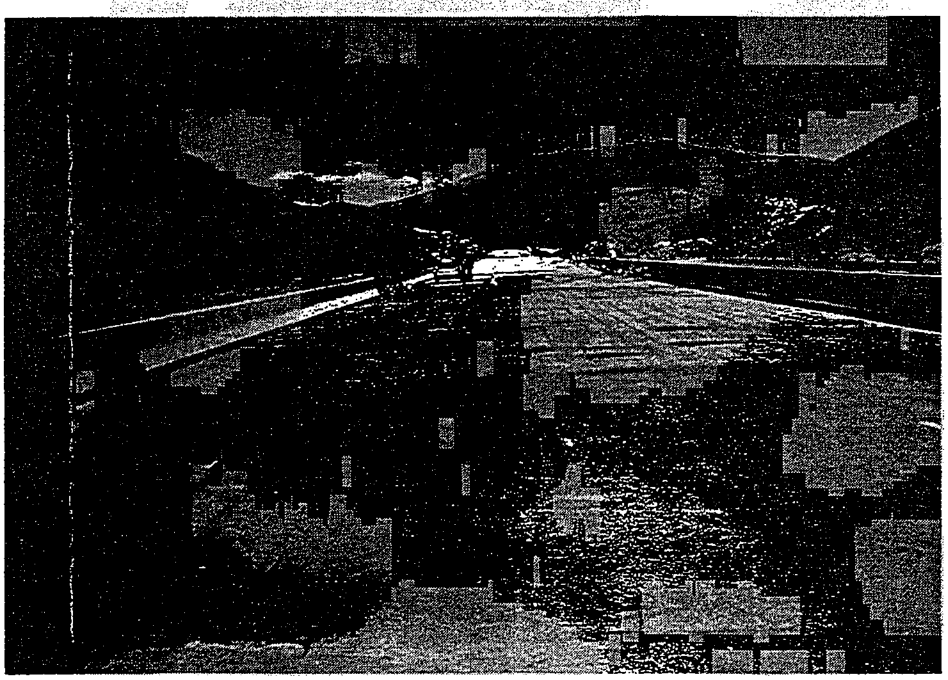


Fig. 8.5. Surface Patches. Bridge deck after "delam" areas have been filled with polyester concrete.

Fig. 8.6. Deck Primer Being Mixed. The prepared surface shall receive a wax-free, low odor (maximum 30% VOC) high molecular weight methacrylate prime coat.

Fig. 8.7. Application of Deck Primer. Primer being applied to prepared deck surface.

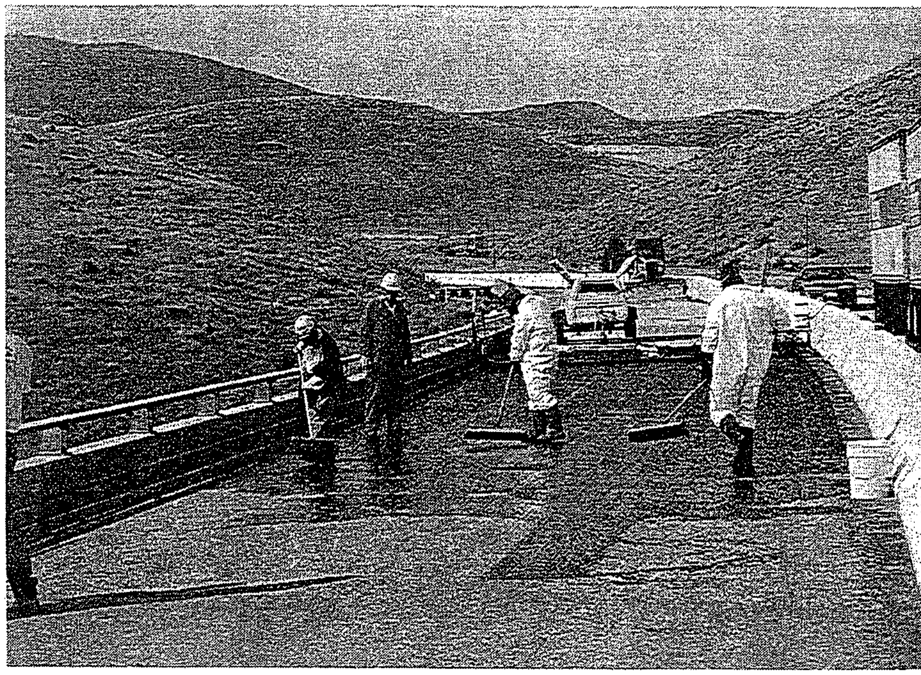


Fig. 8.8. Applying Deck Primer. Primer applied to prepared surface and spread by broom and squeegee at suggested rates of 55 square feet per gallon if some deck surface has been removed, or at 100 square feet per gallon where no deck surface has been removed.

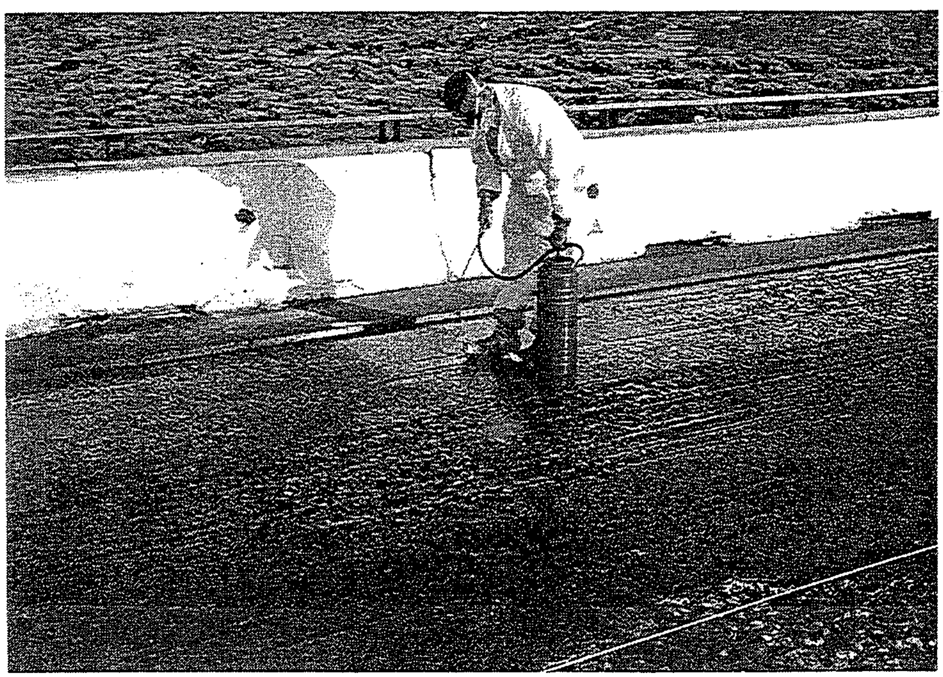


Fig. 8.9. Alternate Application of Deck Primer. An alternate method of applying primer using spray can. Largest spray tip should be used and care must be taken to get material out of spray can and on to bridge deck before it begins to gel.

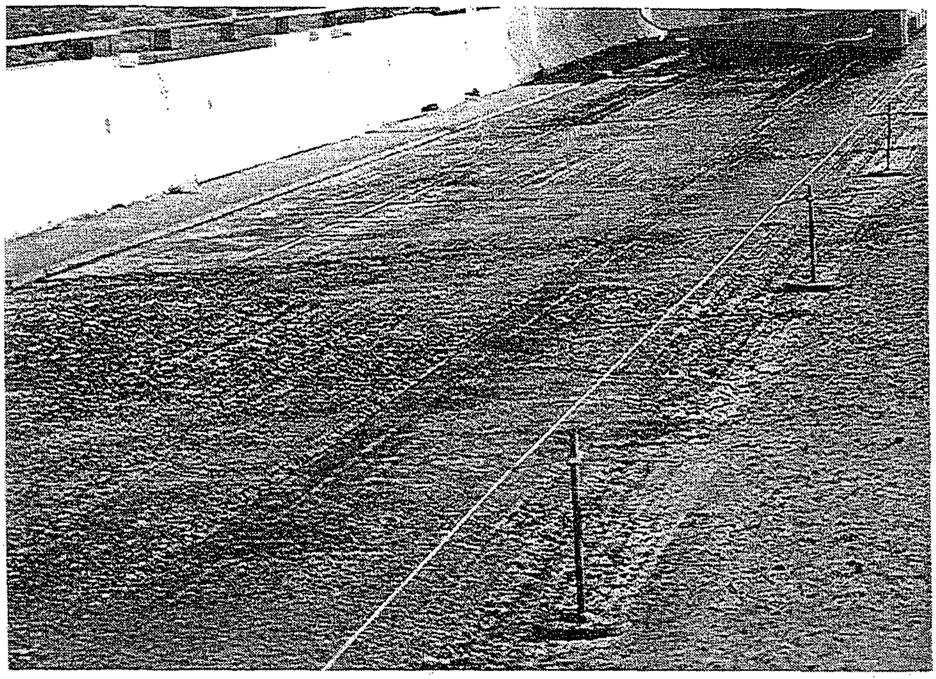


Fig. 8.10. Deck Primer In-Place. High molecular weight methacrylate primer penetrates micro cracks, structurally heals cracks and provides a superior bond between substrate and overlay.

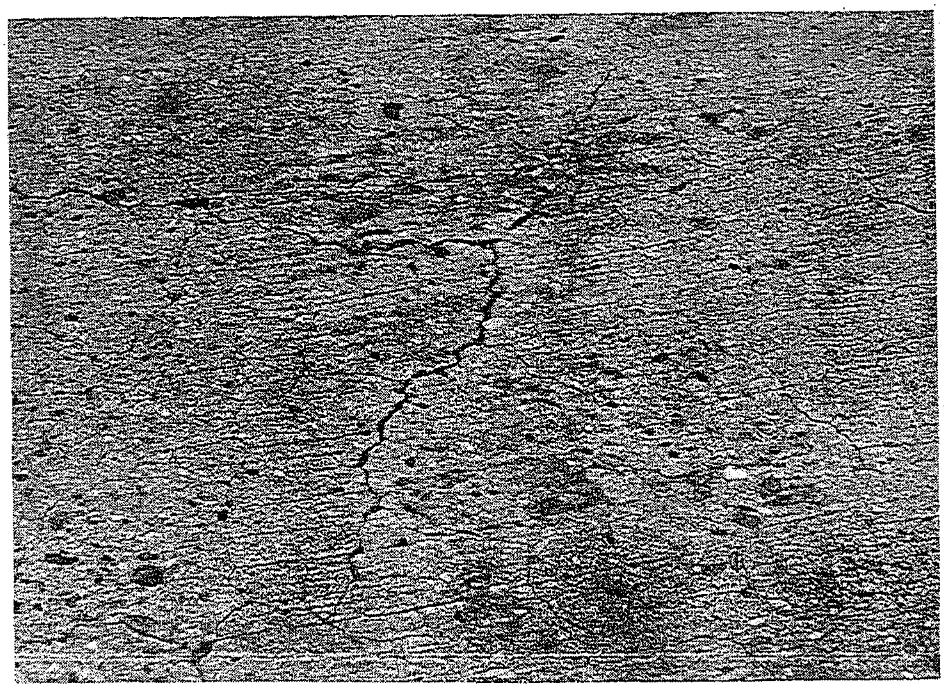


Fig. 8.11. Close-Up of Primed Deck with Cracks. Primer is allowed to cure a minimum of fifteen minutes before placing polyester concrete overlay. This allows time for primer to penetrate cracks in bridge decks.

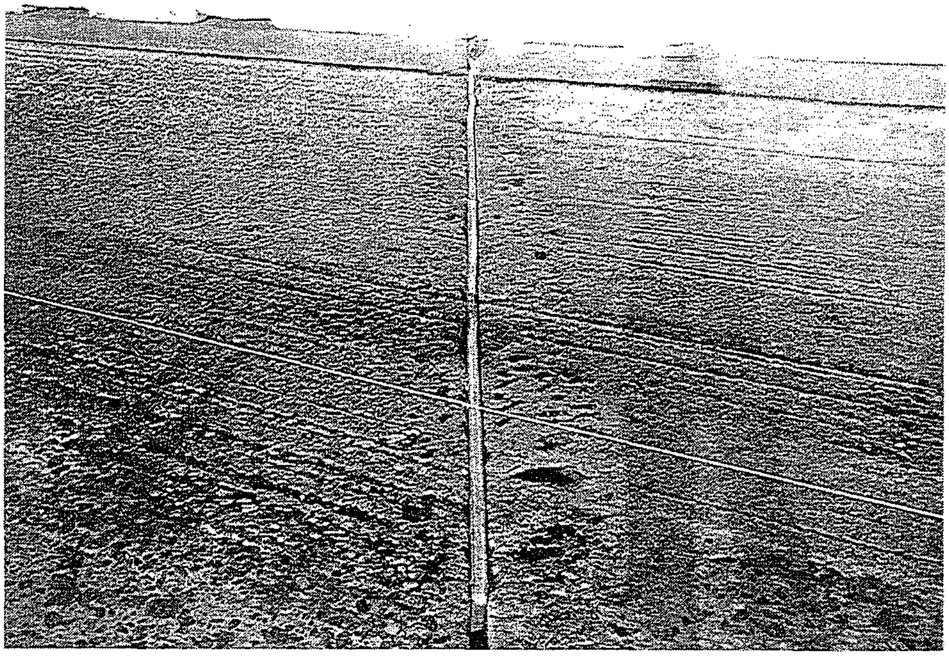


Fig. 8.12. Handling of Deck Expansion Joints. Bridge deck expansion joints shall be adequately isolated prior to overlaying or may be sawed within four hours after overlay placement.

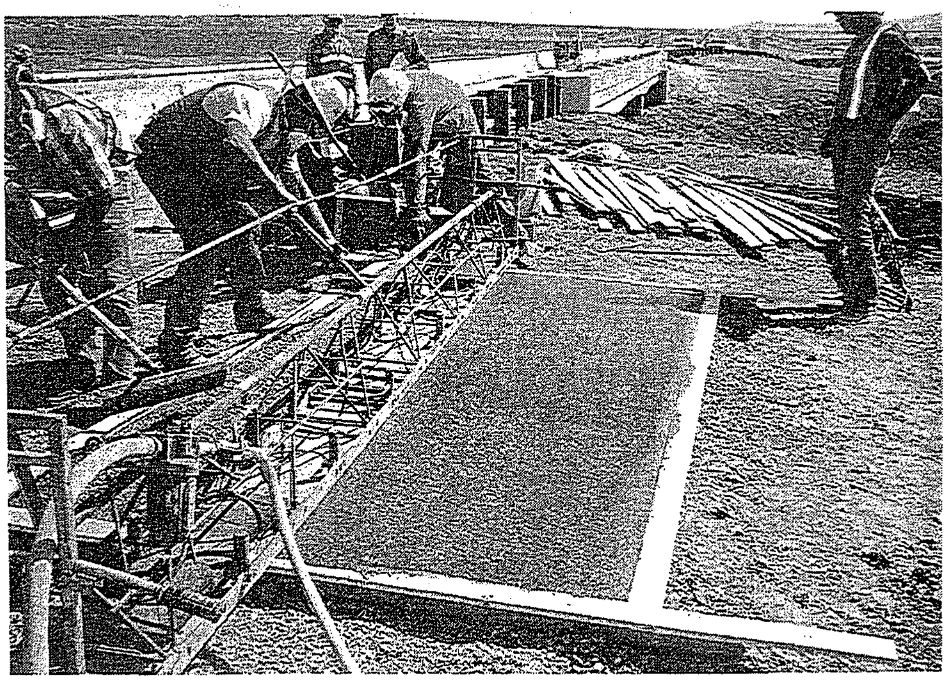


Fig. 8.13. Trial Overlay Placement. Trial overlay placed on a previously constructed 12' wide by 6' long concrete base, at the same thickness as the overlay to be constructed, under conditions similar to those expected for construction.

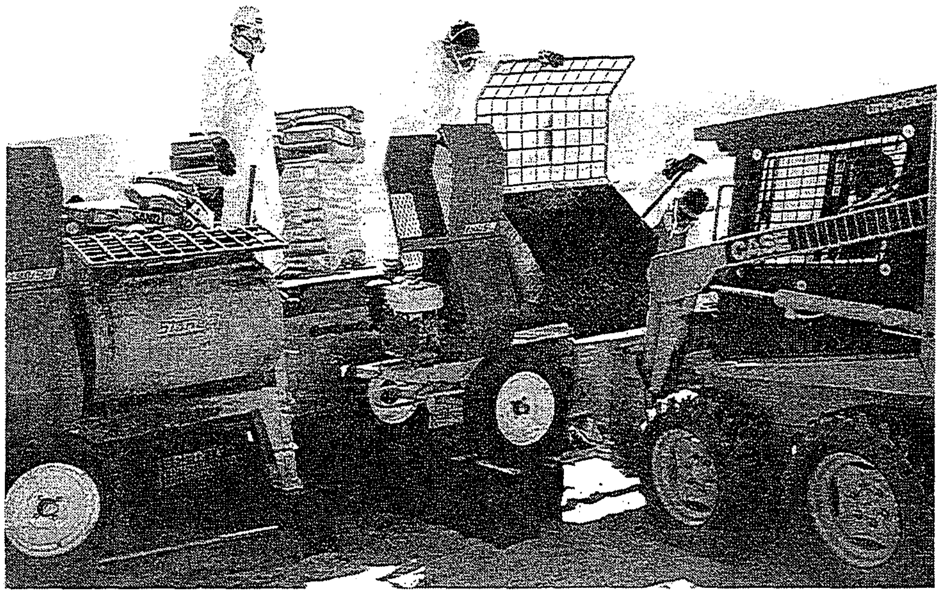


Fig. 8.14. Mixing of PPC. Various methods of mixing polyester resin, catalyst and special aggregate are practical. Use of multiple mortar mixers is shown above.

Fig. 8.15. Mixing Set-Up at Job Site. Preparing to mix polyester concrete using mortar mixers.

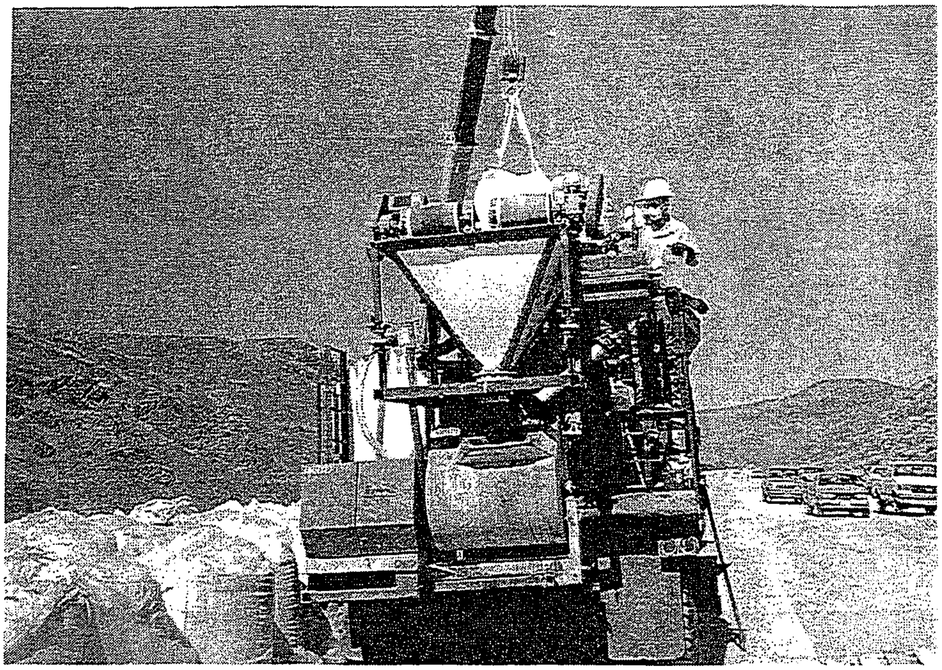


Fig. 8.16. Alternate Mixing of PPC. Mixing of polyester resin, catalyst and aggregates using a contractor built rig. Conveyor belts send aggregates to small front hopper. Load sensors on hopper feet shut down conveyor belts after proper amount of aggregate has been delivered to hopper.

Fig. 8.17. Mobile Mixer for PPC. More sophisticated continuous mixer with auger screw/chute device.

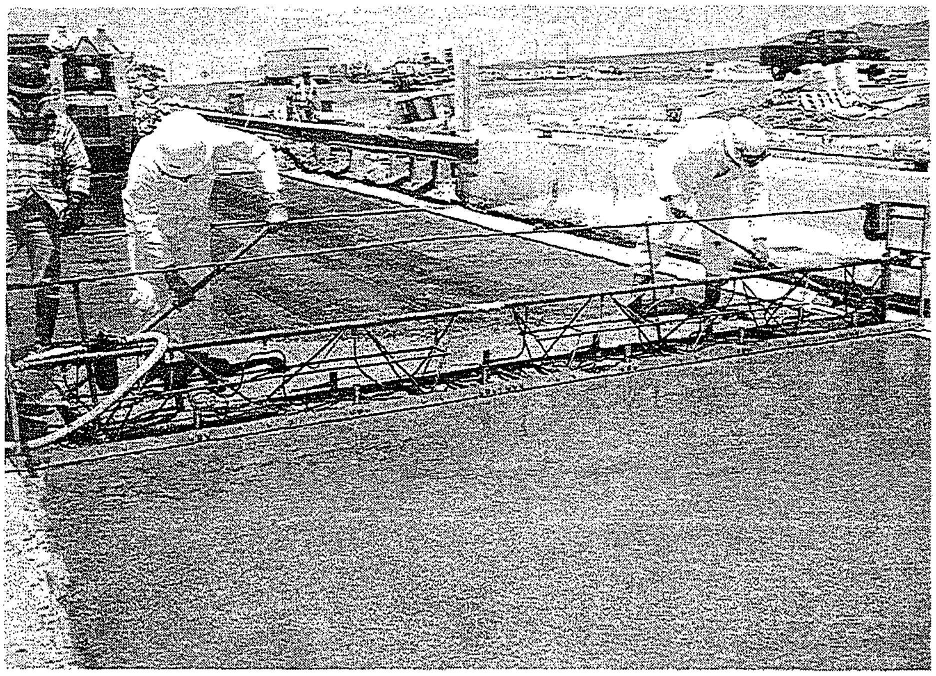


Fig. 8.18. Screeding PPC. Polymer screed has full stainless steel pan under the base, allowing vibration to be spread throughout the underside of the screed. Air vibrations "hit" harder than gasoline engine driven vibration and are much preferred. Notice glossy surface directly behind the screed. This indicates good compaction. The gloss will disappear in a few minutes as the resign drops below the aggregate and the mix cures.

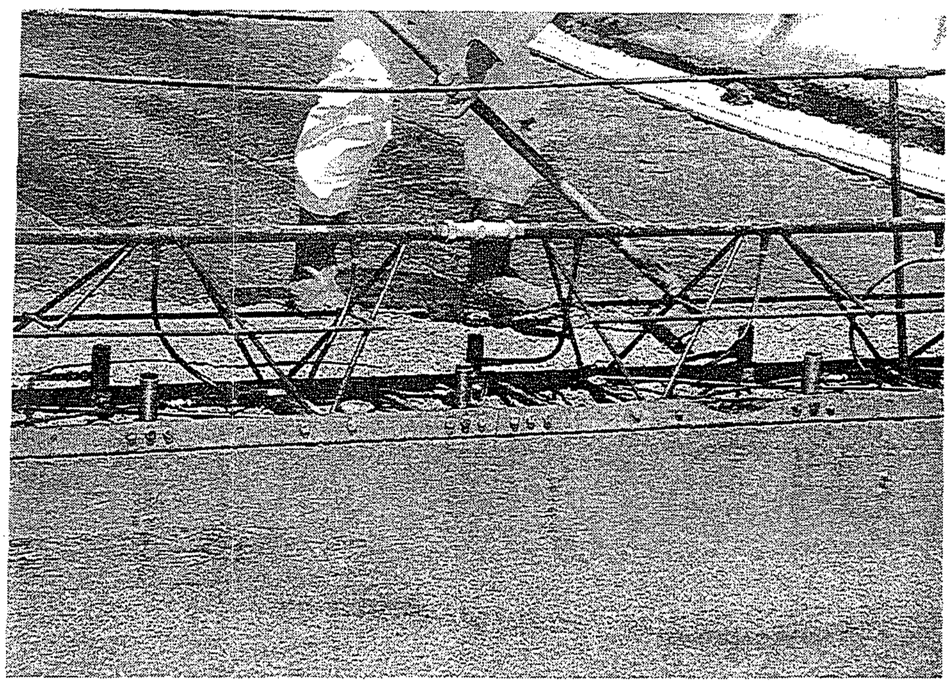
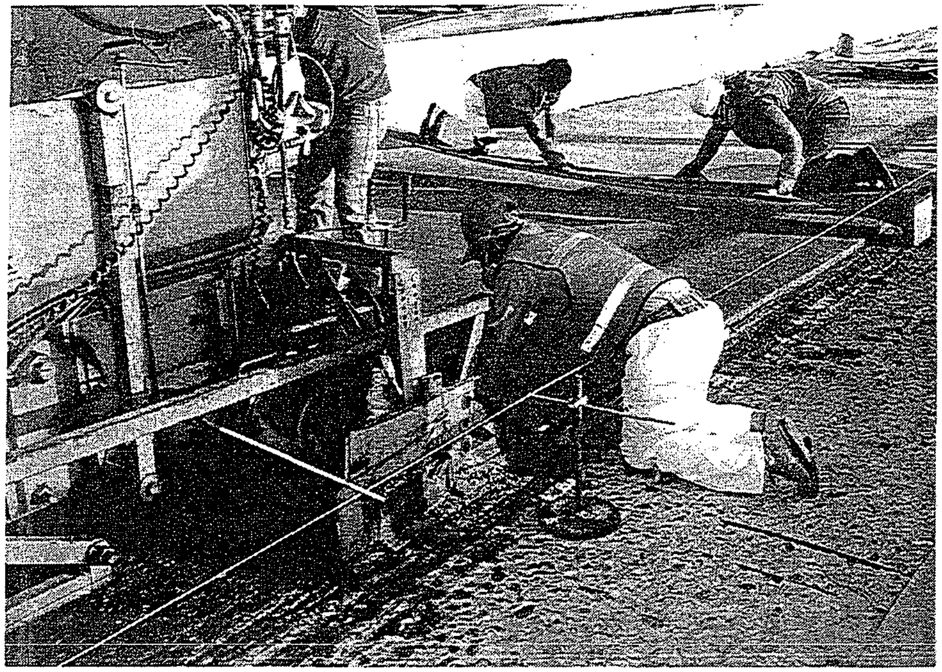


Fig. 8.19. Close-Up of Screeding PPC. Vibrations brings the resin to the surface immediately after the screed passes over the surface of the freshly placed overlay material. As the rein begins to polymerize, it will pull back below the surface of the aggregate. A sand finish is applied immediately after overlay strike-off.



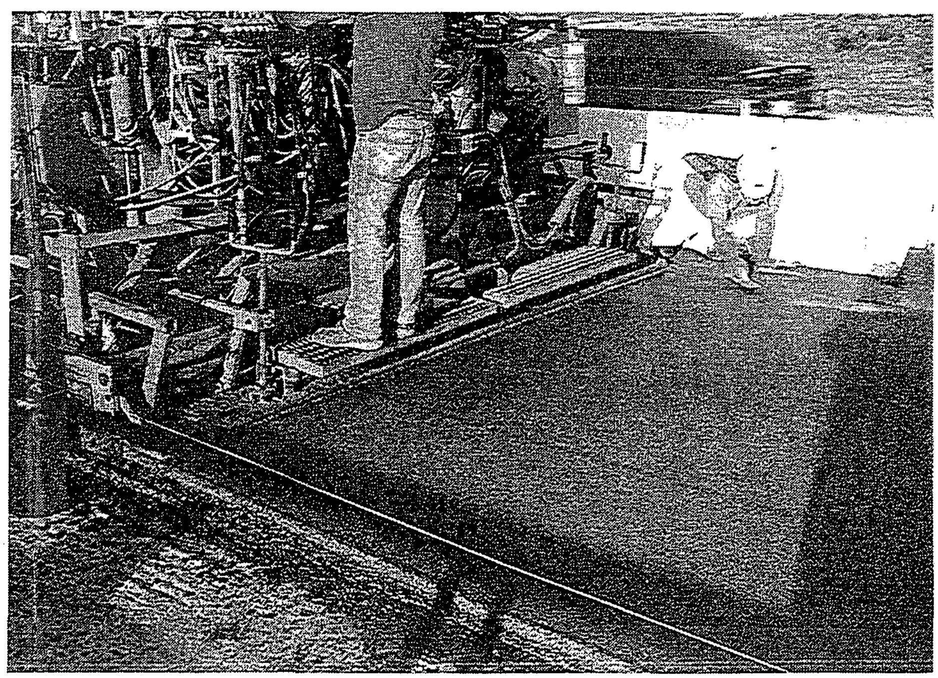


Fig. 8.20. Placing PPC Overlay with Polymer Paver. Several sophisticated types of equipment have been manufactured for placing PPC overlays such as the Polymer Paver shown above.

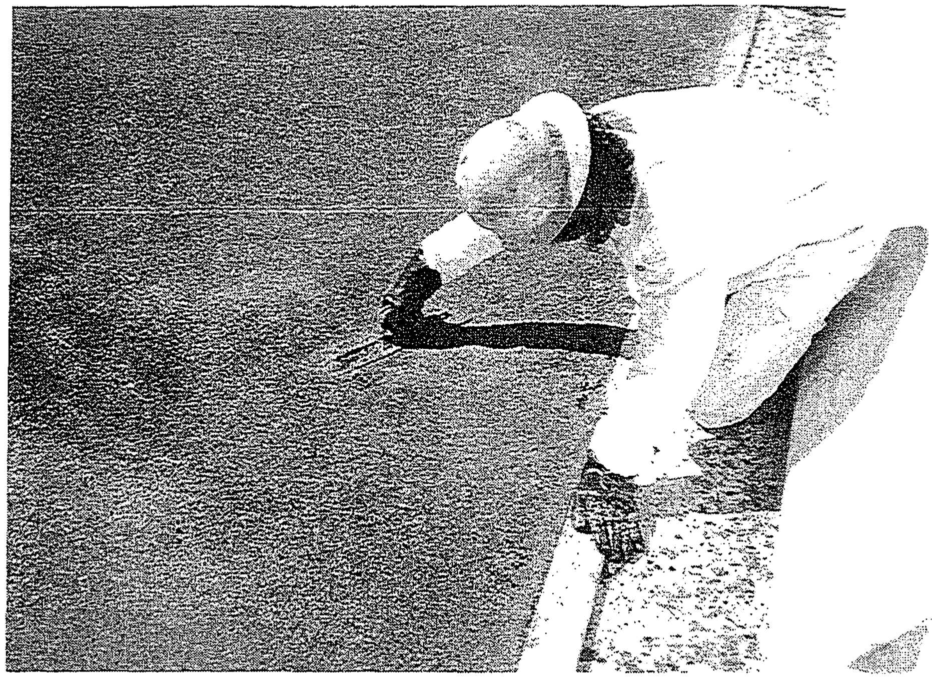
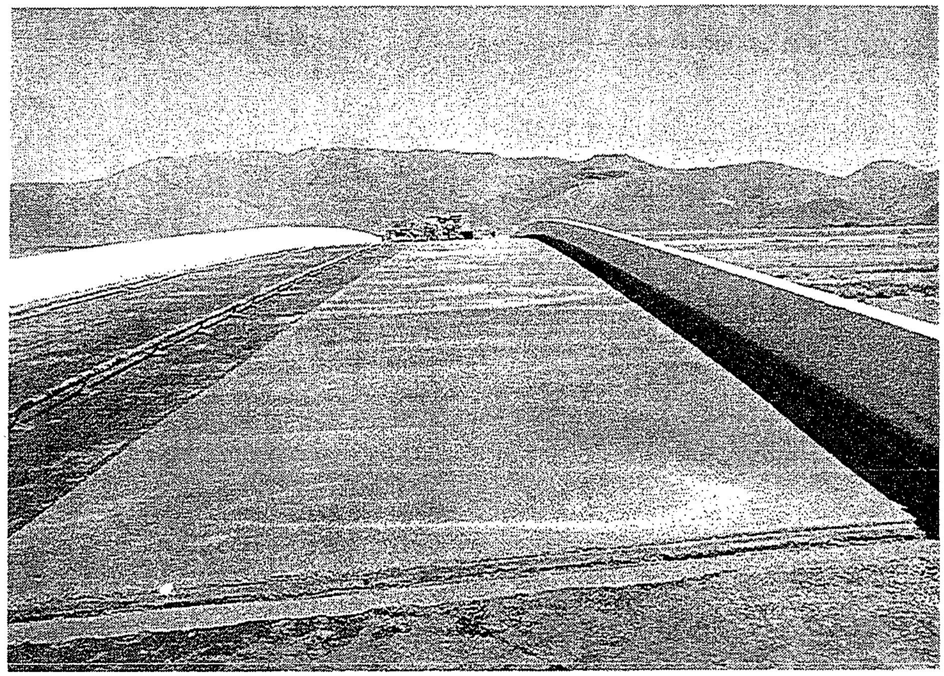


Fig 8.21. Hand Touch-Up Finishing. Dressing up the surface can be done, but should be done promptly. Disturbing the polyester concrete while it is setting permanently destroys the "cross-linking" of the resin.

Fig. 8.22. Trimming Edge of a Multiline Overlay Placement. Finisher trimming up the edge of the newly placed polyester concrete overlay.



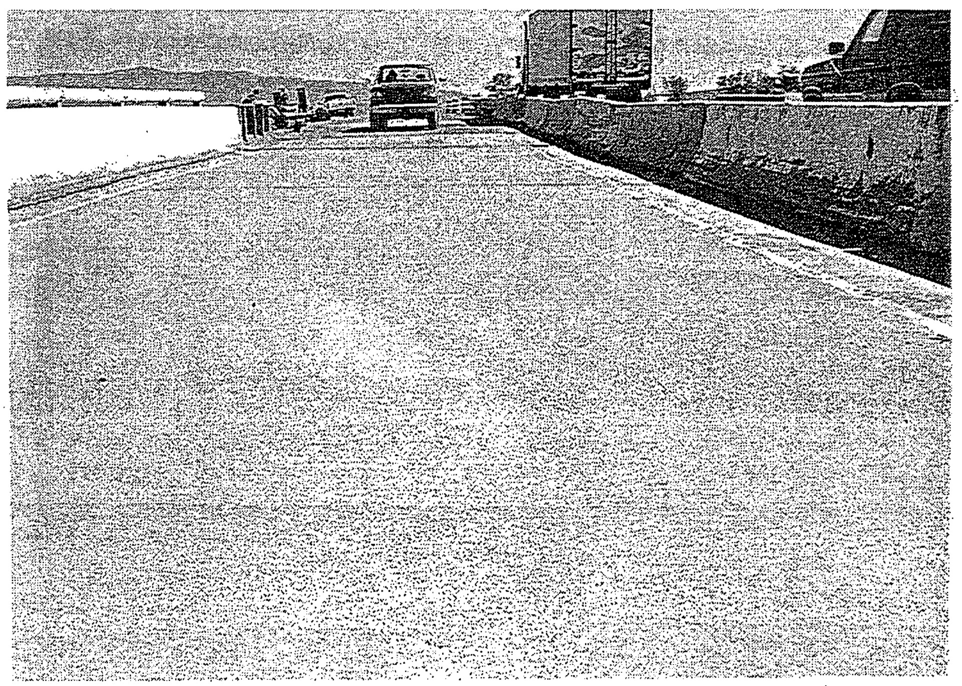


Fig. 8. 23. Completed Full Lane Width PPC Overlay. Once the overlay has set-up traffic can be put back on the lane in about four hours. Then work can begin on adjacent lanes.

9. NYTA'S RAPID DECK REPLACEMENT ON THE TAPPAN ZEE BRIDGE

9.1 General

On September 15, 1998, the project PI, Dr. Ed Ramey, and project graduate research assistant, Mr. Russell Oliver, traveled to Tarrytown, NY to visit the job site of the rapid bridge deck replacement under concurrent traffic conditions of the Tappan Zee Bridge. The bridge is owned and operated by the New York Thruway Authority (NYTA), and carries the I-87 route across the Hudson River (see Fig. 9.1). It is 7 lanes wide (4 lanes SB and 3 lanes NB in mornings and the reverse in the afternoons) and over 3 miles long. The deck replacement project consisted of replacing 258,500 s.f. of the east deck truss spans of the bridge. Precast Exodermic deck panels of 12' width by 18' or 24' length are being used for the new deck. The panels are 7" thick and will be topped with a thin (approximately 1/4") flexogrid overlay once the entire bridge deck has been replaced. The Exodermic design was chosen by the contractor over a half-filled grid alternate.

Exodermic Grid fabrication and panel precasting is being provided by American Grid, a wholly owned subsidiary of American Bridge, Pittsburgh, Pennsylvania. American Grid is one of three licensed suppliers of Exodermic decks; the other two are L. B. Foster and IKG Greulich, both of which are also located in the Pittsburgh area. The panel concrete specified is a high performance mixture and incorporates microsilica and fly ash in the design to provide a dense concrete resistant to chloride intrusion.

Deck replacement was scheduled to begin in April, 1998 but was delayed for various reasons until August 1998. At the time of our visit, Stage I work was being performed, and approximately 30 panels had been placed to date. The contractor was placing 3 panels of 12' width each night, and planned to bring in a second crane and crew and begin placing 6 panels a night in the near future. The work is being done from 8 p.m. - 6 a.m. each weekday with a severe penalty of $1,300 a minute for each minute beyond 6 a.m. that the bridge is not fully open to traffic.

A local New York contractor, Arben Corporation is the prime contractor for the project, and another local New York company, Cutting Technologies, is serving as a subcontractor to perform cutting-up of the old existing deck for removal. The Li Ro Group has been hired by the NYTA to serve as construction inspectors for the deck replacement project.

The deck replacement project was bid in December 1997 with furnishing of deck panels, installation of panels, and demolition of the existing deck being bid as separate items. The Arben Corporation was the low bidding contractor with itemized bids of:

Exodermic panels	-	$26.10/s.f.
Installation of panels	-	$9.38/s.f.
Demolition of deck	-	$10.31/s.f.
Diamond grinding of deck	-	$1.76/s.f.
Placement of thin Flexogrid overlay	-	$6.58/s.f.
Total:		$54.13/s.f.

The Exodermic panel price includes complete precast panel delivery to the job site. It is an average price for the entire deck area, and includes complex joint panels at the ends of each deck truss span. The prices reflect a short work window, New York City union labor, and a $1300/minute penalty for not being completely off the bridge by 6 a.m. Unit bid prices for the Tappan Zee Bridge deck replacement project are shown in Table 9.1, and reflect a wide range of prices for each bid item.

Mr. Robert Bettigole, President of Exodermic Bridge Deck, Inc. was of great help to us in providing pre-trip information and guidance and in serving as host to us during our visit. Mr. Viorel Chirila, assistant resident engineer with the Li Ro Group, and Mr. Mark Ronnow, project superintendent for the Arben Corporation also shared their knowledge of the project and were of great service to us during our visit to the job site.

Because of the importance of the I-87 route and the heavy traffic load that it carries, the deck replacement work had to be done in stages (Stages I, II, III, IV) under concurrent traffic conditions. Basically this consisted of restricting the work to nights only, i.e., from 8:00 p.m. until 6:00 a.m. the following day. The work is being executed in 4 stages as indicated in Figs. 9.2 - 9.3 and will require approximately 2 years to complete the 258,500 square feet of deck replacement.

The primary construction tasks and sequence for a typical night's work (8 p.m. - 6 a.m.) are as follows:

1. At 8 p.m. relocate the moveable barrier to the a.m. traffic pattern (3 lanes northbound & 4 lanes southbound).
2. Place cones closing off the 2 right southbound driving lanes (for Stage I work).
3. Mobilize equipment on the closed lanes.
4. At 11 p.m. install temporary concrete barrier closing third southbound lane.
5. Saw-cut and remove existing deck.
6. Clean top flange of steel stringers.
7. Flame cut and remove existing finger joint locations when installing joint panels.
8. Install support steel at joint locations when installing joint panels.
9. Install sheet metal haunch forms on top of steel stringers.
10. Install grid panels and level to surveyed elevation.

11. Install shear studs on steel stringers.
12. Connect grid panels and place approved rapid set grout mix.
13. At flange locations where existing deck meets new grid deck place a temporary material to top roadway elevation.
14. Once rapid-set concrete has set, place cones down where picking up temporary barrier continuing to leave one lane southbound.
15. Remove equipment from bridge.
16. Pick up cones.
17. Traffic reopened to a.m. traffic pattern by 6 a.m.

The construction tasks/sequence is slightly different for each stage of construction (Stages I, II, III, IV); however it is essentially as shown above. On Friday nights the work period is 9 p.m. - 7 a.m.

Photographs showing the deck replacement work in progress during Stage I are presented and discussed in the following section.

9.2 Photographic Display and Discussion of Deck Replacement Work

A photographic display of a typical night's work of placing 3 new 12' wide Exodermic deck panels is shown in Figs. 9.4 - 9.29.

Figs. 9.4 and 9.5 show three new 12' x 24' Exodermic panels ready for placement in the next night's work. The panels are shown in the off-bridge staging area which is located approximately 1/4 mile from the bridge. Note that the lifting spreader assembly is in place on one of the panels, and the panel tops have been tined for skid resistance. The panel transverse joints (parallel to the truck bed) have shear keys which have been sand-blasted to enhance bond. The panel longitudinal seams (transverse to truck bed) have tapped-thread couplers welded on the ends of rebar to allow continuity of the transverse deck rebar between adjacent panels. Note that the 12' panel width constitutes a wide load on the trucks which came unescorted via interstate highways from Pittsburgh, PA where the panels were fabricated.

Fig. 9.6 shows a short section (approximately 60 ft) of temporary concrete barrier which is placed at 11:00 p.m. (and closes down an additional traffic lane) in order to protect traffic from the "open hole" when the old deck is removed. Fig. 9.7 shows a typical transverse joint between a new Exodermic panel and the existing deck. This is how the joint is left each night and how it appears at the beginning of the next night's work.

Figs. 9.8 and 9.9 show jack hammering work at locations of shear lugs on the existing deck. The section of existing deck which the workers are removing are precast panels which were placed as test panels 4 years earlier, and have 2 shear lugs placed in block-out/filled holes 4' apart in the longitudinal direction. Note the locating of the shear stud/block-out locations in Fig. 9.8 and the longitudinal saw cuts on each side of the block-out/shear studs in Fig. 9.9. Fig. 9.10 shows the removing of concrete which was jack hammered from the shear stud/block-out locations, light jack hammering to remove any remaining concrete on the studs, and burning off of the shear stud.

Figs. 9.11 - 9.13 show the preparation of the next 36' longitudinally along the deck in preparation for the next night's work. Fig. 9.11 shows deck coring work to allow attachment of nylon webbing to lift out sections of the existing deck. Figs. 9.12 and 9.13 show transverse deck saw-cutting for the next night's deck removal. Note in Fig. 9.13 the wet vacuuming which is taking place concurrently with the sawing in order that the deck is clean of wet or dry cutting residue when the bridge is opened at 6:00 a.m. the next morning. In addition to the transverse cutting, short longitudinal cuts are made at the shear stud locations (4' apart). All other longitudinal cutting of the existing deck must be done on the night that section of the existing deck is removed and replaced with Exodermic panels.

Figs. 9.14 and 9.15 show removal of the first section (approximately 12' x 12') of the existing deck. The sections are loaded on the same low-boy trailers which brought on the temporary concrete barriers and taken to a disposal site. The first deck section removed is the hardest. After that, a sea-clamp like bracket (see bracket lying on deck in Fig. 9.16) is attached to the free edge of the next section which is then lifted slightly to break it free (saw cuts do not go full depth at stinger locations), and then nylon webbing is attached to lift it out. Fig. 9.17 shows attaching the webbing, at the previously cored locations, for removal of a section of the existing deck.

Fig. 9.18 shows the opening (approximately 24' transversely by 36' longitudinally) in the deck after removal of the existing deck sections. Also shown in Fig. 9.18 are the sheet metal haunch forms which are connected across the top of the stringers with metal straps which in-turn are kinked to tighten each side of the metal form up against the edges of the stringer top flange. It should be noted that prior to placing the sheet metal haunch forms, the tops of the stringers were cleaned of any concrete, and were then cleaned of loose rust via a hand-held 3-prong electric scabber and a hand-held power wire brush, and then blown clean with compressed air. Then, the metal haunch forms shown in Fig. 9.18 were placed.

Figs. 9.19 - 9.22 show the placement of the three 12' x 24' new Exodermic panels. Note in the background of Figs. 9.21 and 9.22 placement of foam backer-rods at the bottom of the transverse joints, and the pulling up and securing of the longitudinal sheet metal haunch forms. Note in the foreground of Figs. 9.21 and 9.22 the core holes, saw cuts, and clean top surface of the existing deck which has been prepared for the next night's work.

Fig. 9.23 shows the temporary concrete barrier being removed after the deck opening has been closed up with the new panels. The barrier segments are placed on the same low-boy trailer that brought on the last Exodermic panels.

Fig. 9.24 shows workers pulling up the sheet metal haunch forms and securing (preventing them from dropping back down) with a sheet metal screw which goes through the metal form and under the Exodermic panel. Fig. 9.25 shows the installation of 3/4" shear studs at 8" o.c. with a shear stud gun.

Fig. 9.26 shows a worker welding a connection strap at a transverse joint connecting two of the Exodermic panels together. There were three such straps at each transverse joint in addition to a continuous field placed concrete shear key. These can best be seen in the photos of the panels in the staging area in Figs. 9.4 and 9.5. A close-up of the block-out welded strap connection is shown in Fig 9.27. It should be noted that Exodermic panels on most other deck applications employ only the field placed concrete shear key to connect adjacent panels in the longitudinal direction. Note the unplaced foam backer rod to be installed at the bottom of the transverse shear key in the foreground in Fig. 9.26.

Fig. 9.28 shows the placement of a proprietary rapid-set concrete mixture above each longitudinal stringer (and around the shear studs), and at each transverse joint. Note in Fig. 9.28 the mobile concrete mixer which allows the contractor to better control mixture water and admixtures in order to achieve 2000 psi concrete in 1-hour. All joints were wetted with water from the mobile mixer before placing the rapid-set concrete. The concrete was consolidated with a 1" diameter probe vibrator and left with a rough top surface finish. It was noted that the transverse gap between adjacent panels was a little too narrow in some instances (should have been 1 1/2") and that consolidation of the transverse joint concrete left something to be desired. Figure 9.29 shows placement of wet burlap for curing the field placed concrete. The contractor was slow in implementing the curing system. Assuming that concrete cylinders cast at the time of placement of the joint concrete reached their specified strength of 2000 psi in 1-hour, the wet burlap covering was removed, deck clean-up and removal of construction equipment completed, traffic control cones removed and the complete deck reopened to traffic by 6:00 a.m.

Table 9.1 Tappan Zee Bridge Deck Replacement Unit Bid Prices

Contractor	Bid Price ($/s.f.)					
	Exodermic Panels	**Installation of Panels**	**Demolition of Existing Deck**	**Diamond Grinding Deck**	**Thin Flexogrid Overlay**	**Total**
Arben Corporation	**26.10**	**9.38**	**10.31**	**1.76**	**6.58**	**54.13**
Defoe	29.00	18.00	15.00	0.72	6.00	68.72
Modern Continental	24.00	16.10	25.00	0.67	4.44	70.21
Ecco III	35.00	18.60	16.00	1.39	5.26	76.25
Perini	42.69	15.00	14.50	1.11	5.00	77.30
Grow Tunneling	35.00	25.00	10.00	1.44	8.33	79.77
Schiavone	32.75	22.00	22.00	1.11	5.00	82.86
Karl Koch	28.00	35.50	30.00	2.00	5.56	101.06
Nab	35.00	35.00	25.00	1.11	6.67	102.78
Halmar	40.00	39.52	15.00	1.67	6.67	102.86
Kiska	30.00	37.17	29.00	1.67	6.67	104.51
D'Annunzio	41.50	32.00	30.00	1.67	6.67	111.84
Yonkers	33.00	27.00	47.81	1.67	6.11	115.59
Anselmi & De Cicco	35.00	70.00	45.00	0.44	5.00	155.44
Felix Equities	48.00	55.00	40.00	1.67	12.22	156.89

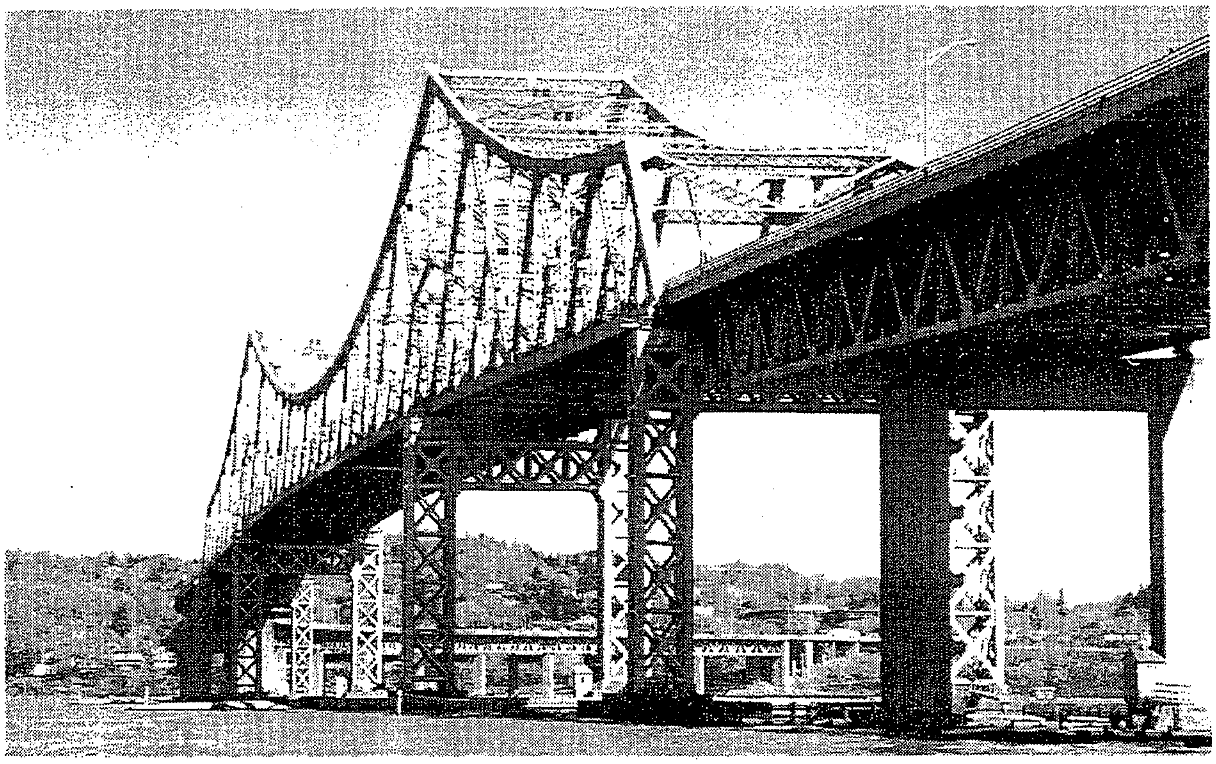


Fig. 9.1 NYTA's Tappan Zee Bridge

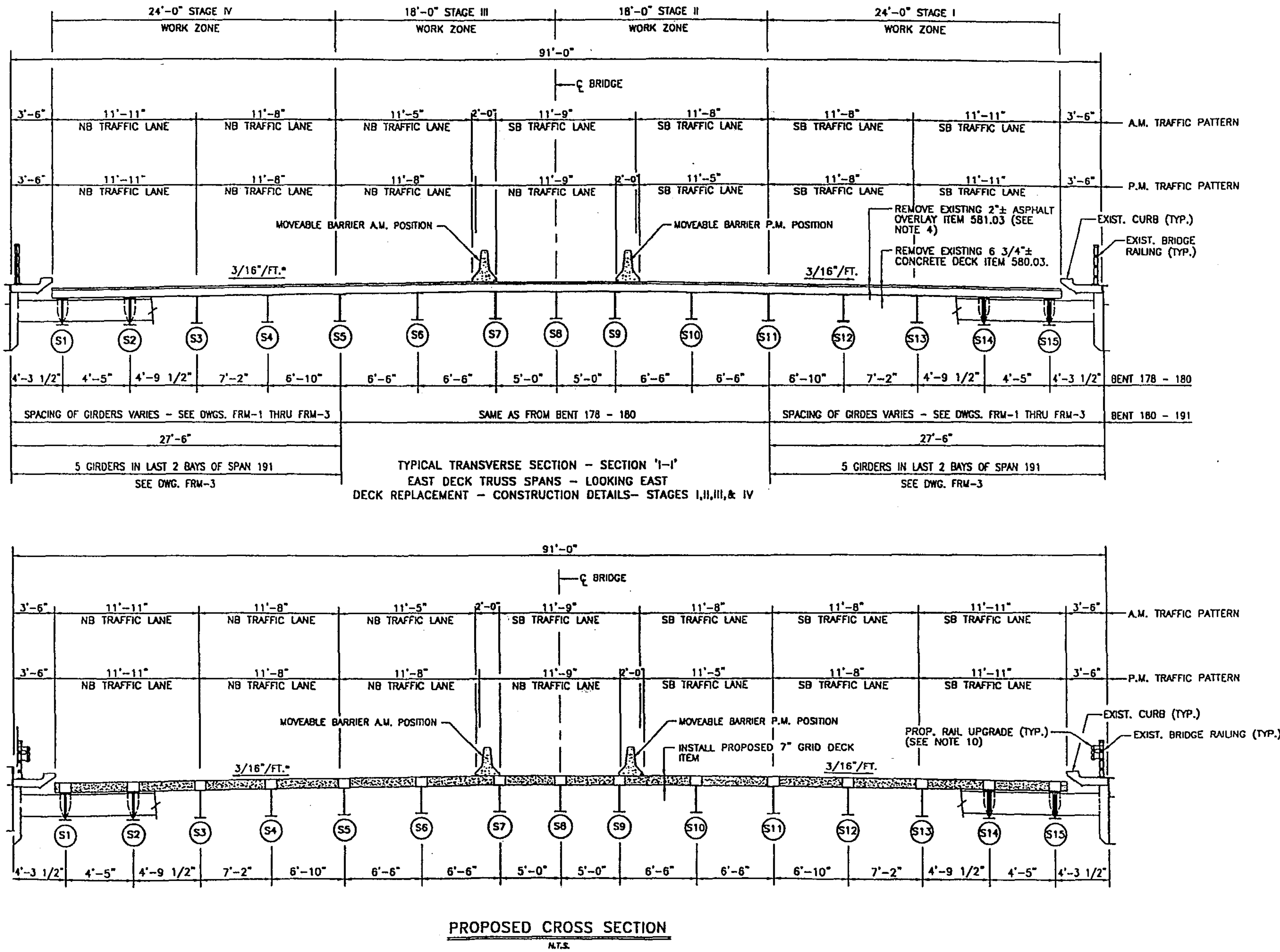
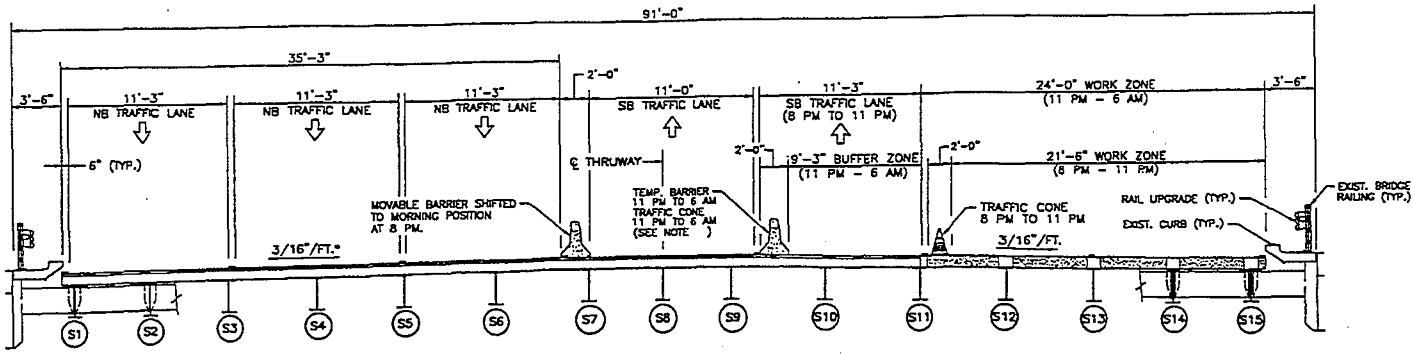


Fig. 9.2 Deck Replacement Construction Staging



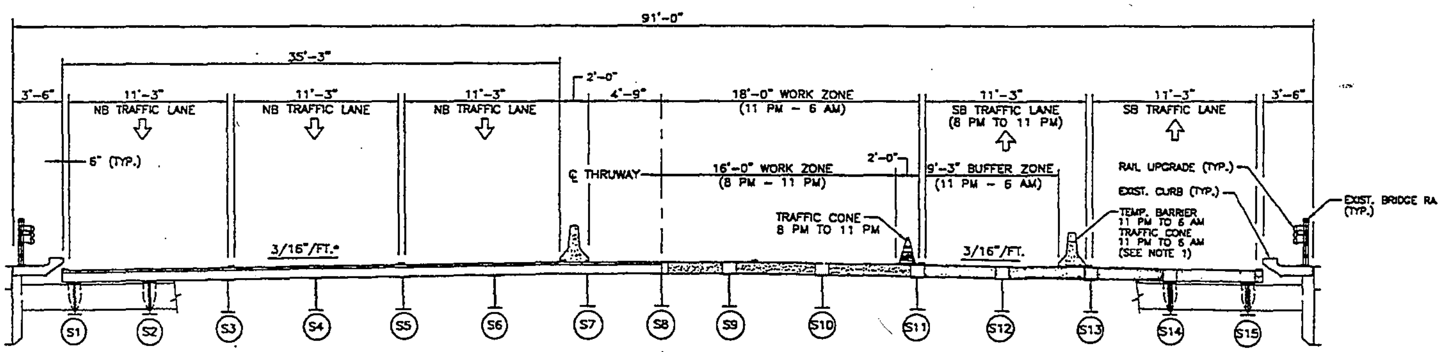
LANE CONFIGURATION
STAGE I
N.T.S.

STAGE I

1. Commencing at 8 p.m. every work night (9 p.m. on Friday night), the moveable barrier system shall be shifted to the A.M. traffic pattern (3 northbound & 4 southbound lanes). the contractor shall then close the two (2) right southbound lanes and commence deck removal as ordered by the Engineer.

2. Commencing at 11 p.m. every work night (12 a.m. on Friday night), a temporary barrier system shall be placed on the existing deck closing the right lane of the two (2) southbound lanes leaving one (1) southbound lane open from 11 p.m. to 6 a.m..

3. The contractor SHALL have all lanes opened to traffic by 6 A.M.. Penalties, as described in this contract, shall be imposed on the contractor for failure to comply with this requirement.



* FOR SUPERELEVATION TRANSISTION SEE DWG. XSCTN-3

LANE CONFIGURATION
STAGE II
N.T.S.

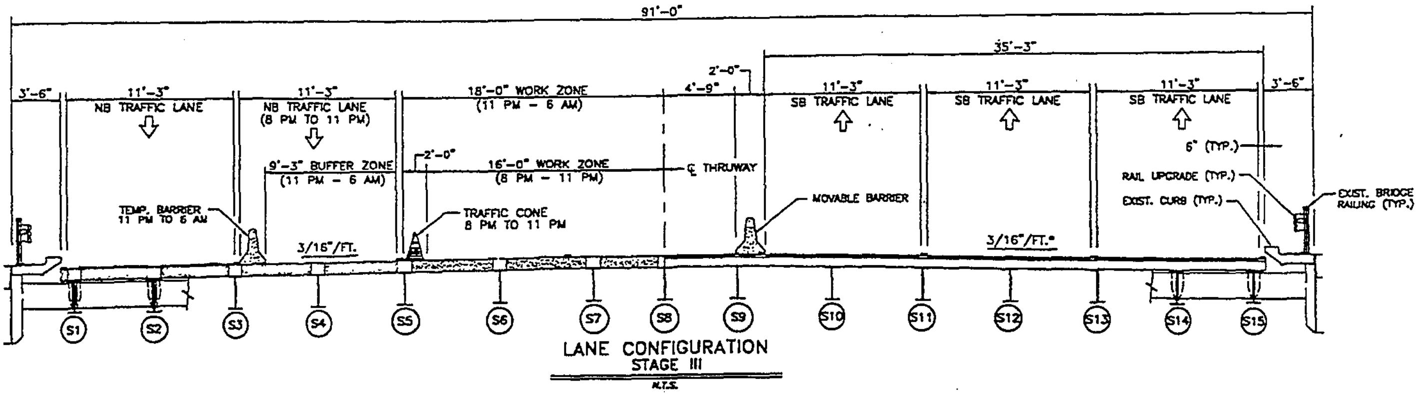
STAGE II

1. Commencing at 8 p.m. every work night (9 p.m. on Friday night), the moveable barrier system shall be shifted to the A.M. traffic pattern (3 northbound & 4 southbound lanes). the contractor shall then close the two (2) right southbound lanes and commence deck removal as ordered by the Engineer.

2. Commencing at 11 p.m. every work night (12 a.m. on Friday night), a temporary barrier system shall be placed on the existing deck closing the right lane of the two (2) southbound lanes leaving one (1) southbound lane open from 11 p.m. to 6 a.m..

3. The contractor SHALL have all lanes opened to traffic by 6 A.M.. Penalties, as described in this contract, shall be imposed on the contractor for failure to comply with this requirement.

Fig. 9.3a Construction Staging and Traffic Control (Stages I and II)



LANE CONFIGURATION
STAGE III
N.T.S.

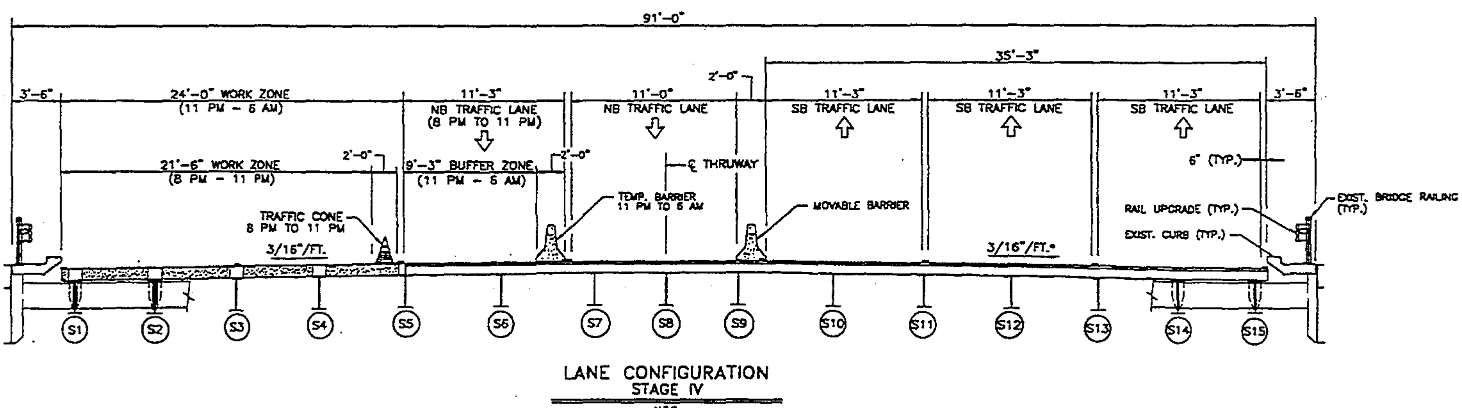
STAGE III

1. Commencing at 8 p.m. every work night (9 p.m. on Friday night), the moveable barrier system shall be shifted to the A.M. traffic pattern (3 northbound & 4 southbound lanes). the contractor shall then close the two (2) right southbound lanes and commence deck removal as ordered by the Engineer.

2. Commencing at 11 p.m. every work night (12 a.m. on Friday night), a temporary barrier system shall be placed on the existing deck closing the right lane of the two (2) southbound lanes leaving one (1) southbound lane open from 11 p.m. to 6 a.m..

3. The contractor shall have nightly work completed in time for the moveable barrier shift to the A.M. traffic pattern (4 southbound & 3 northbound lanes).

4. The contractor SHALL have all lanes opened to traffic by 6 A.M. Penalties, as described in this contract, shall be imposed on the contractor for failure to comply with this requirement.



LANE CONFIGURATION
STAGE IV
N.T.S.

STAGE IV

1. Commencing at 8 p.m. every work night (9 p.m. on Friday night), the moveable barrier system shall be shifted to the A.M. traffic pattern (3 northbound & 4 southbound lanes). the contractor shall then close the two (2) right southbound lanes and commence deck removal as ordered by the Engineer.

2. Commencing at 11 p.m. every work night (12 a.m. on Friday night), a temporary barrier system shall be placed on the existing deck closing the right lane of the two (2) southbound lanes leaving one (1) southbound lane open from 11 p.m. to 6 a.m..

3. The contractor shall have nightly work completed in time for the moveable barrier shift to the A.M. traffic pattern (4 southbound & 3 northbound lanes).

4. The contractor SHALL have all lanes opened to traffic by 6 A.M.. Penalties, as described in this contract, shall be imposed on the contractor for failure to comply with this requirement.

Fig. 9.3b Construction Staging and Traffic Control (Stages III and IV)

Fig. 9.4 Exodermic Panels in Off-Bridge Staging Area

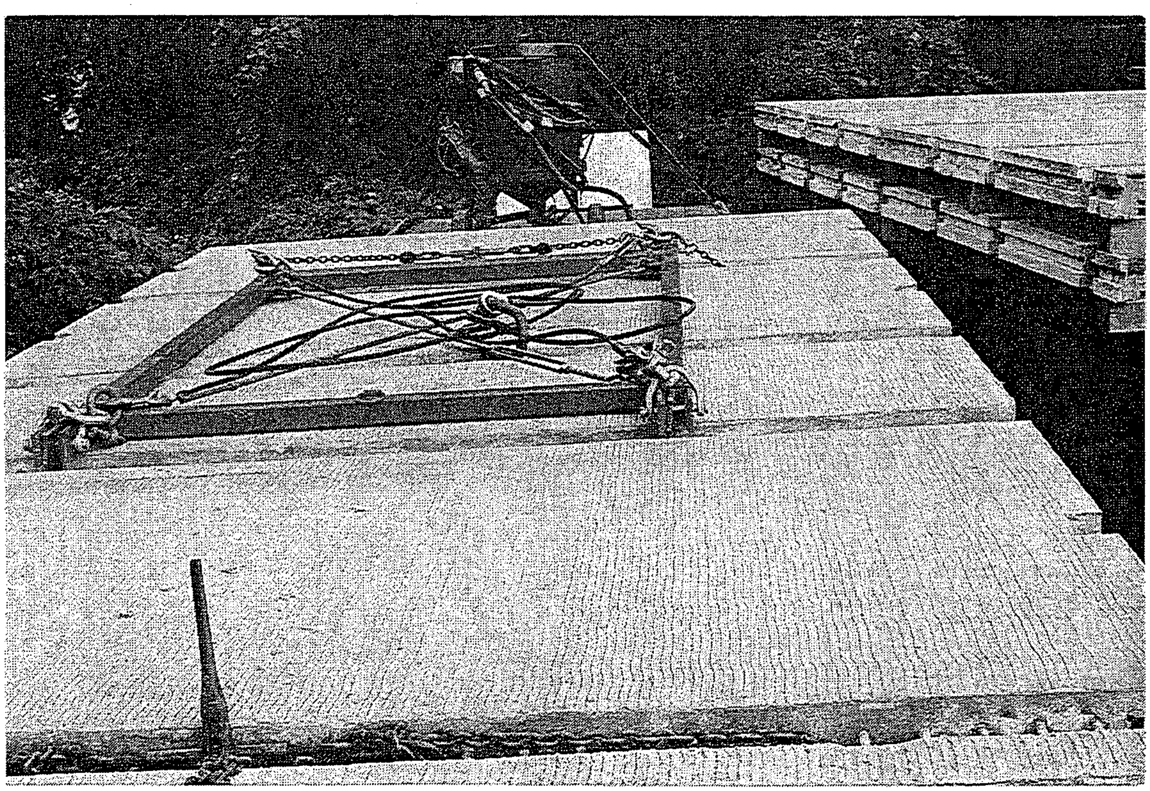


Fig. 9.5 Exodermic Panel with Lift Spreader

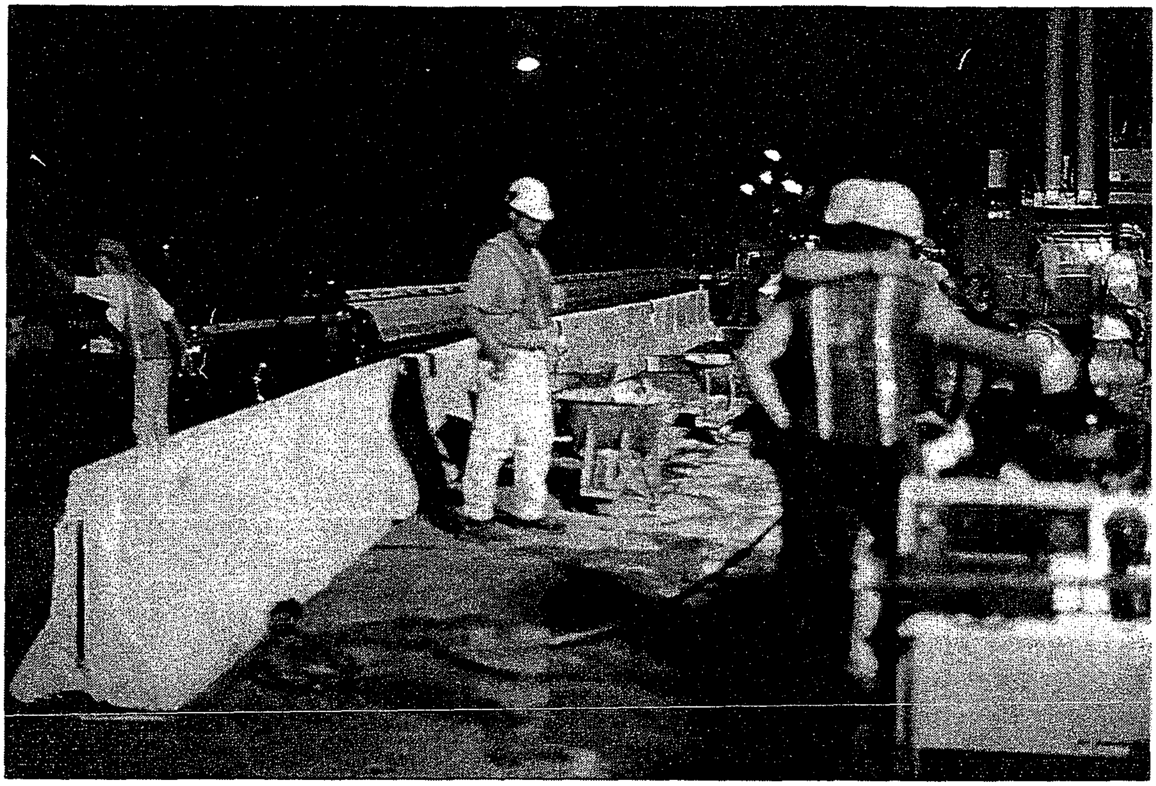


Fig. 9.6 Work Site with Short Length of Temporary Concrete Barrier

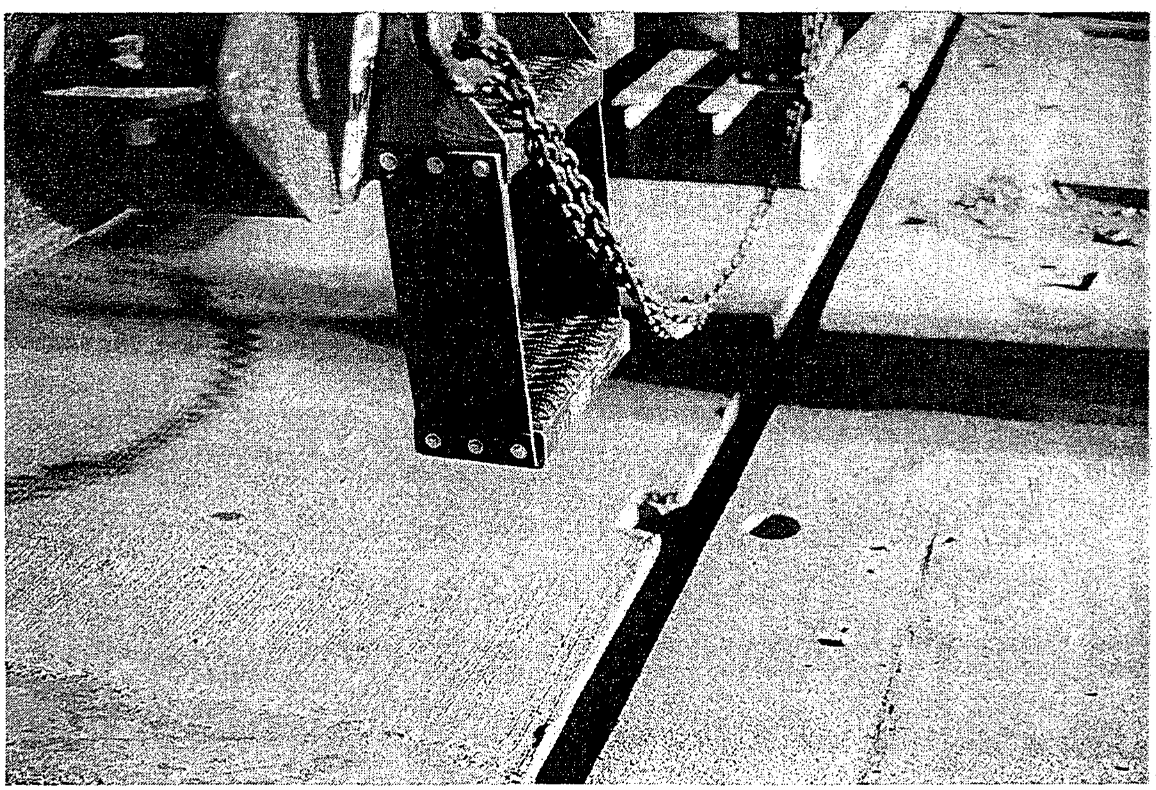


Fig. 9.7 New Panel - Existing Deck Transverse Joint

Fig. 9.8 Jack-Hammering at Locations of Shear Lugs

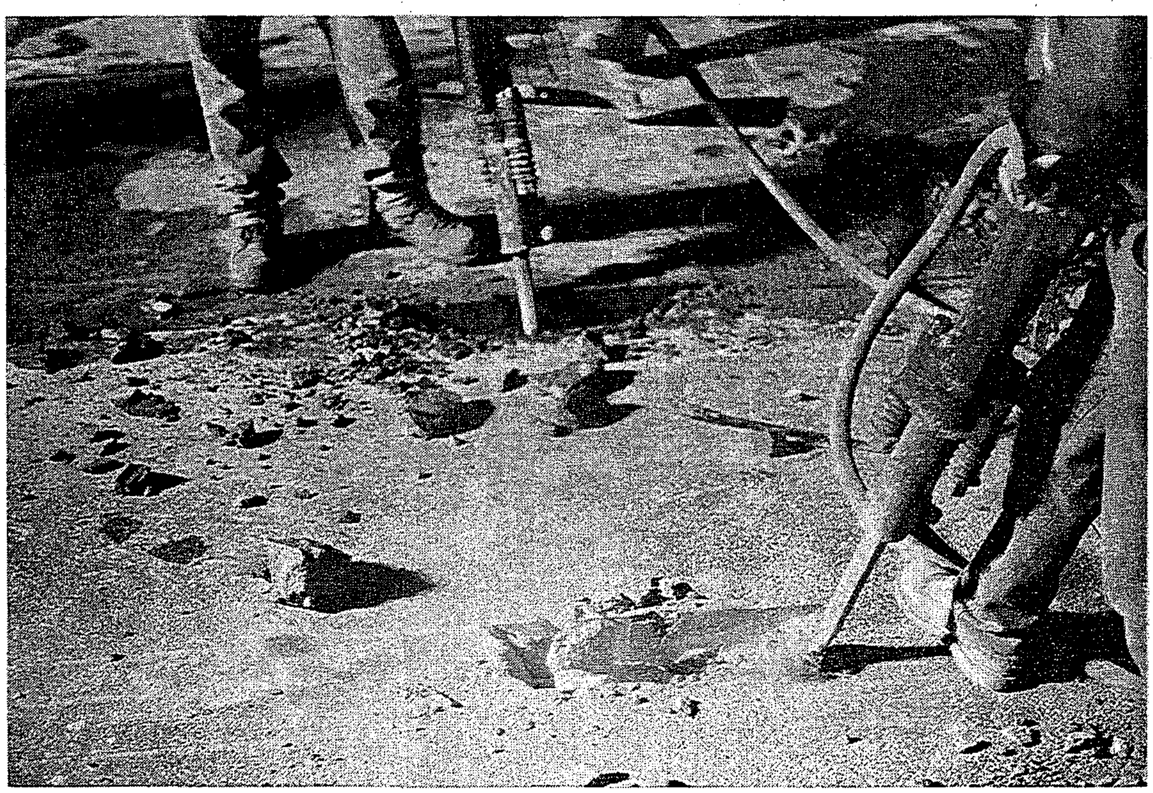


Fig. 9.9 Close-Up of Jack-Hammering Operation

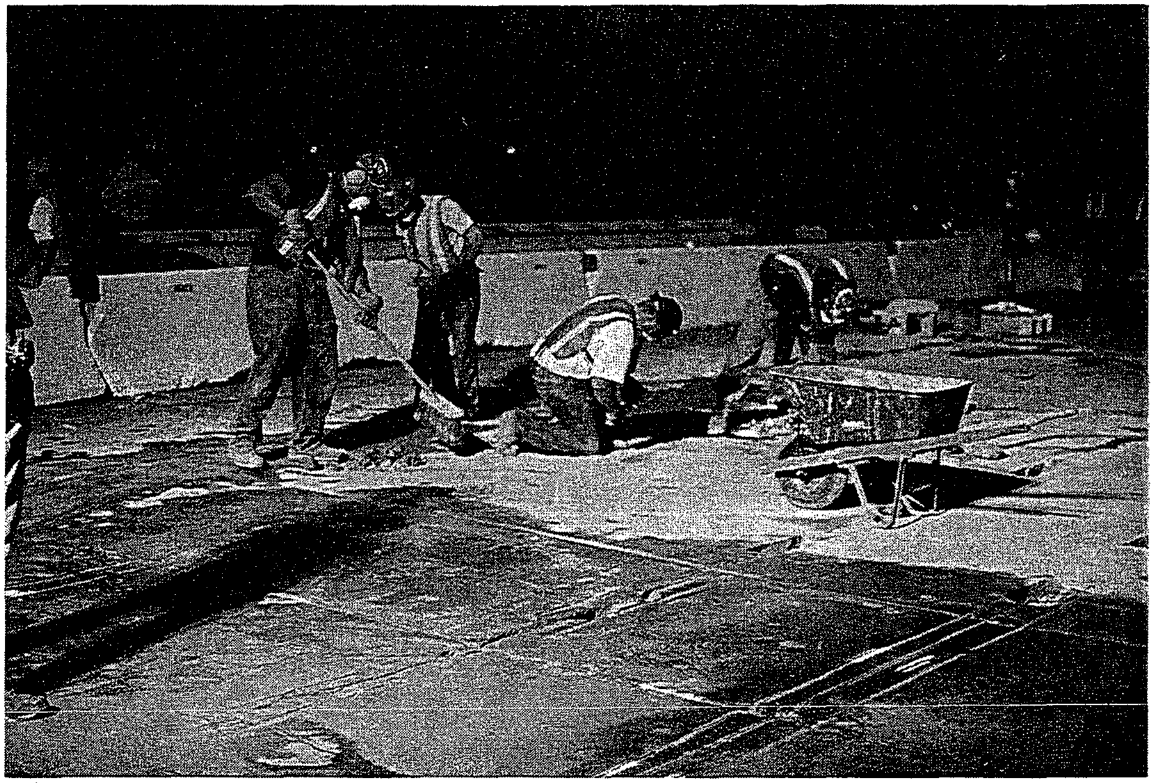


Fig. 9.10 Removing Concrete and Burning-off of Shear Lugs

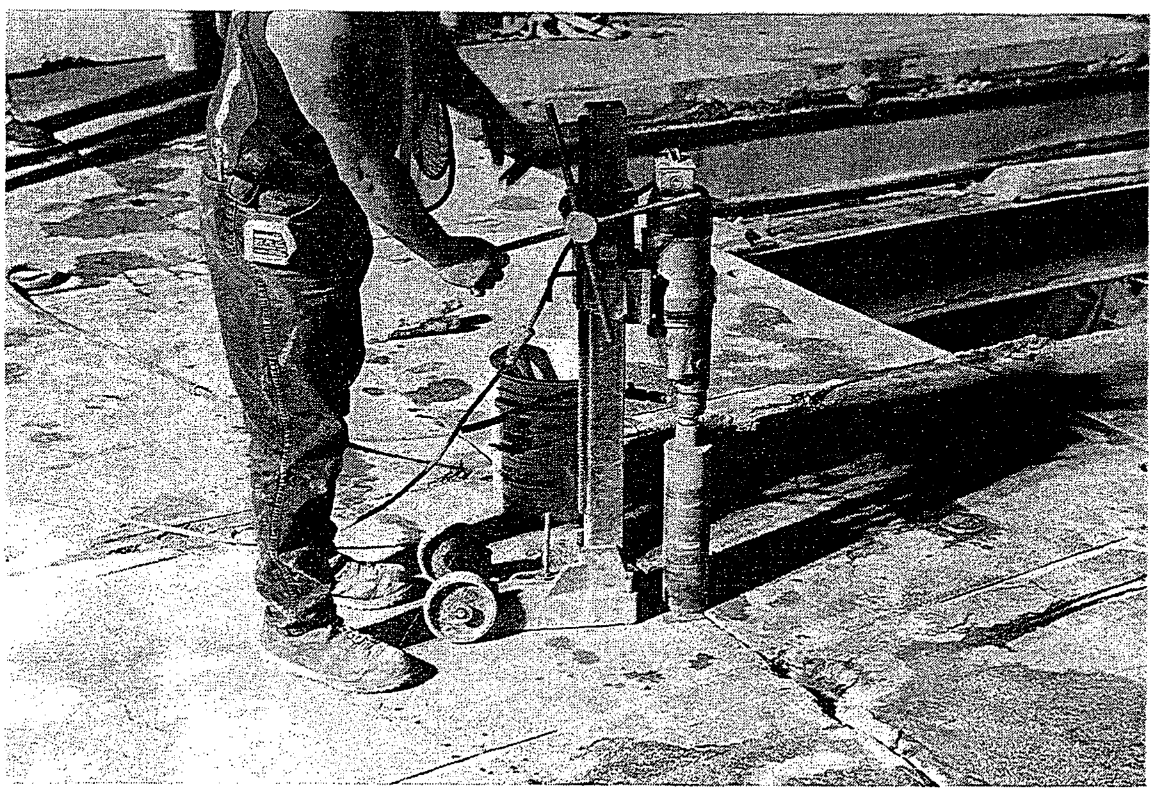


Fig. 9.11 Core Drilling for Deck Removal for Next Night

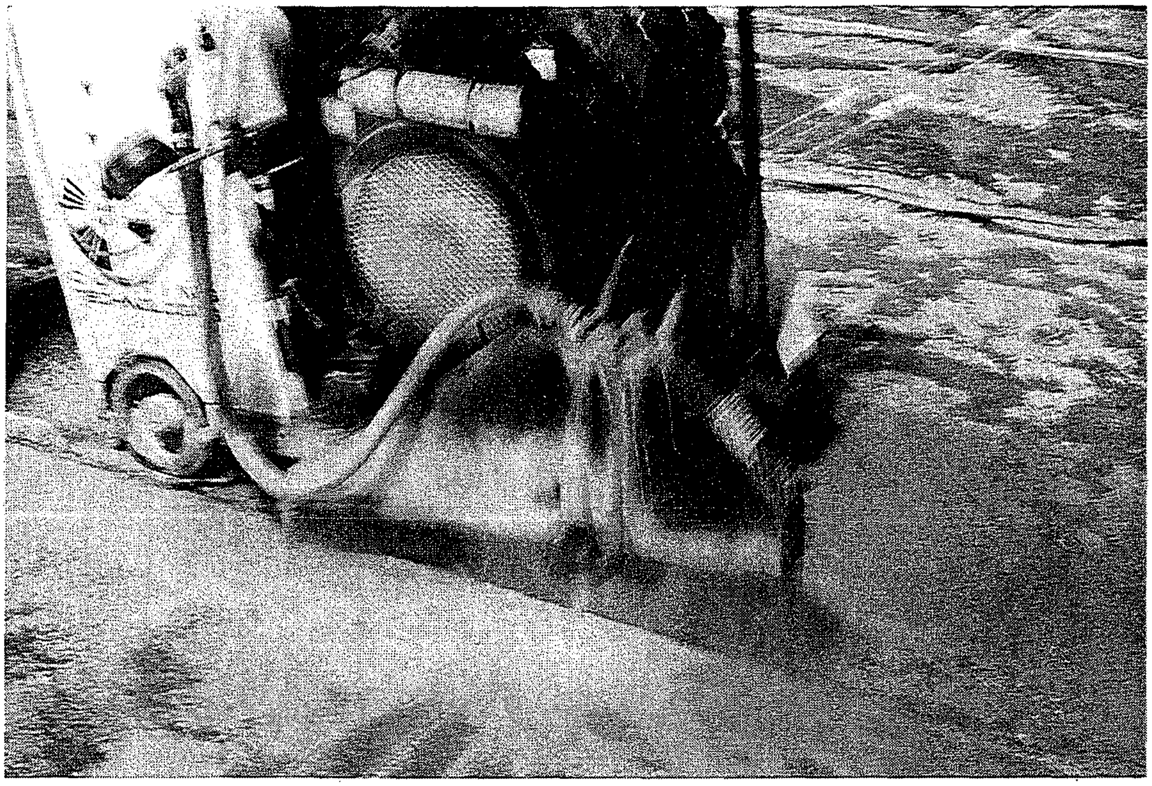


Fig. 9.12 Saw - Cutting Concrete for Deck Removal Next Night

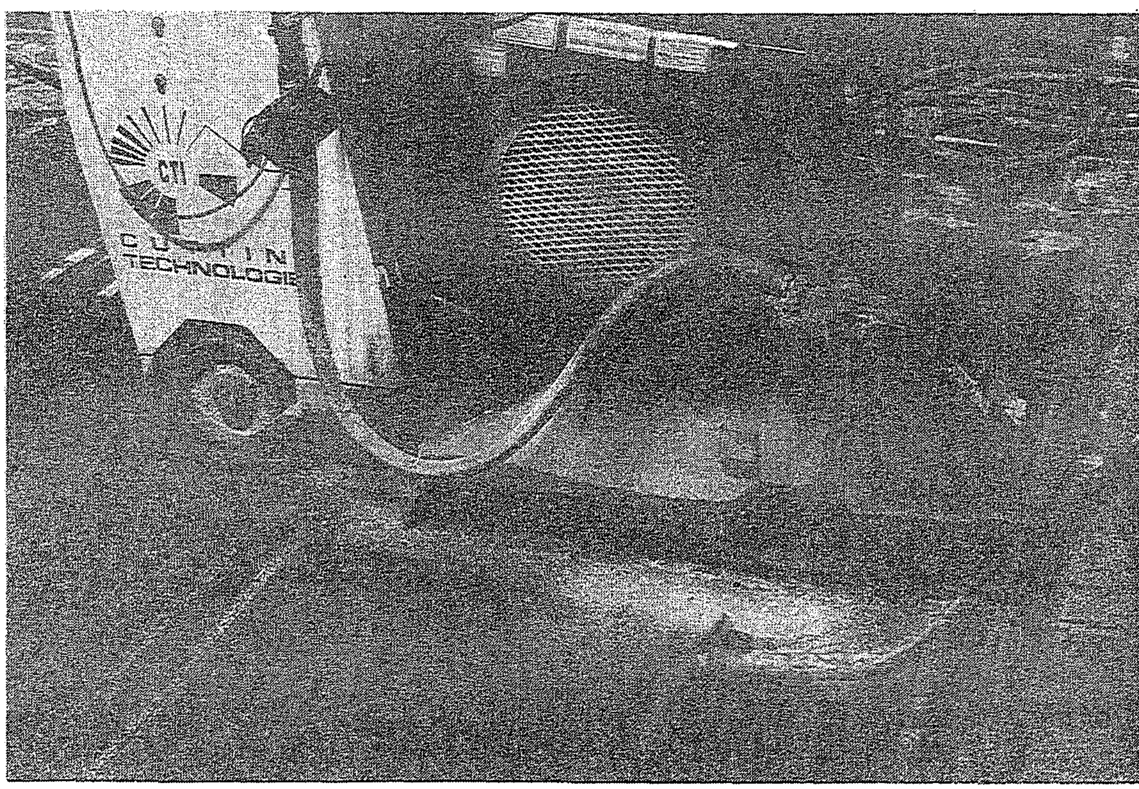


Fig. 9.13 Saw - Cutting Concrete and Vacuuming Residue

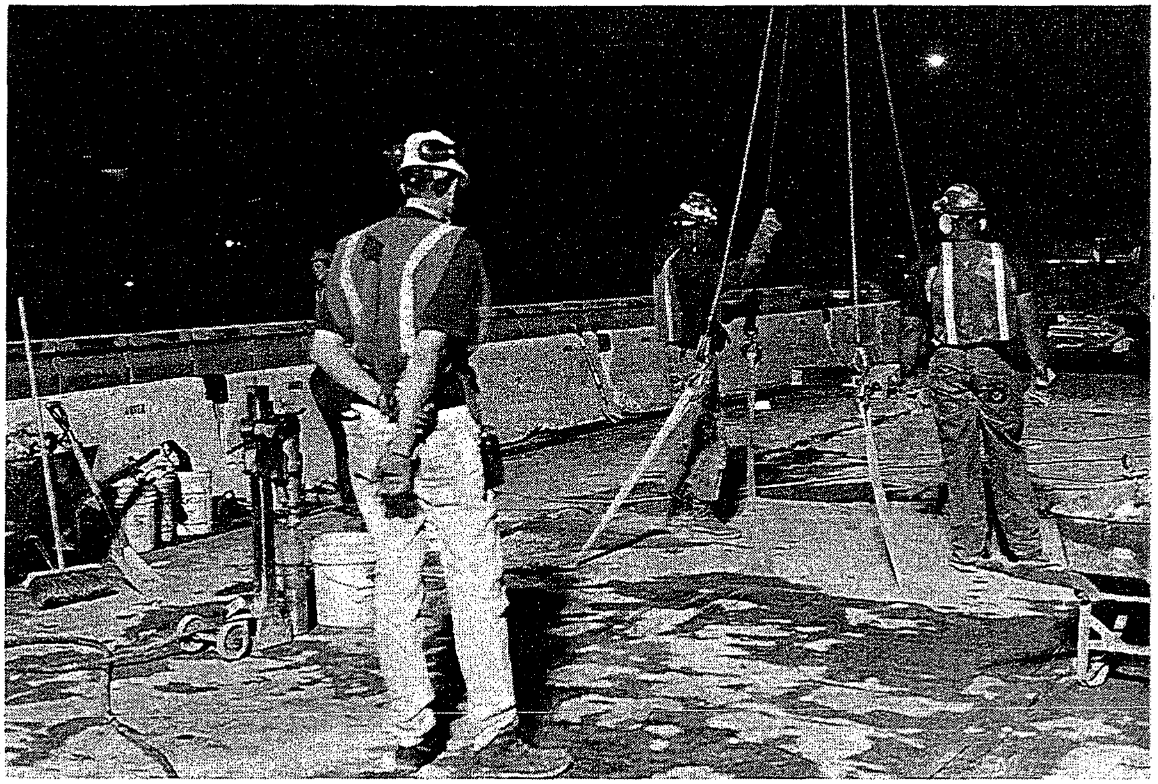


Fig. 9.14 Preparing to Lift Out First Section of Existing Deck

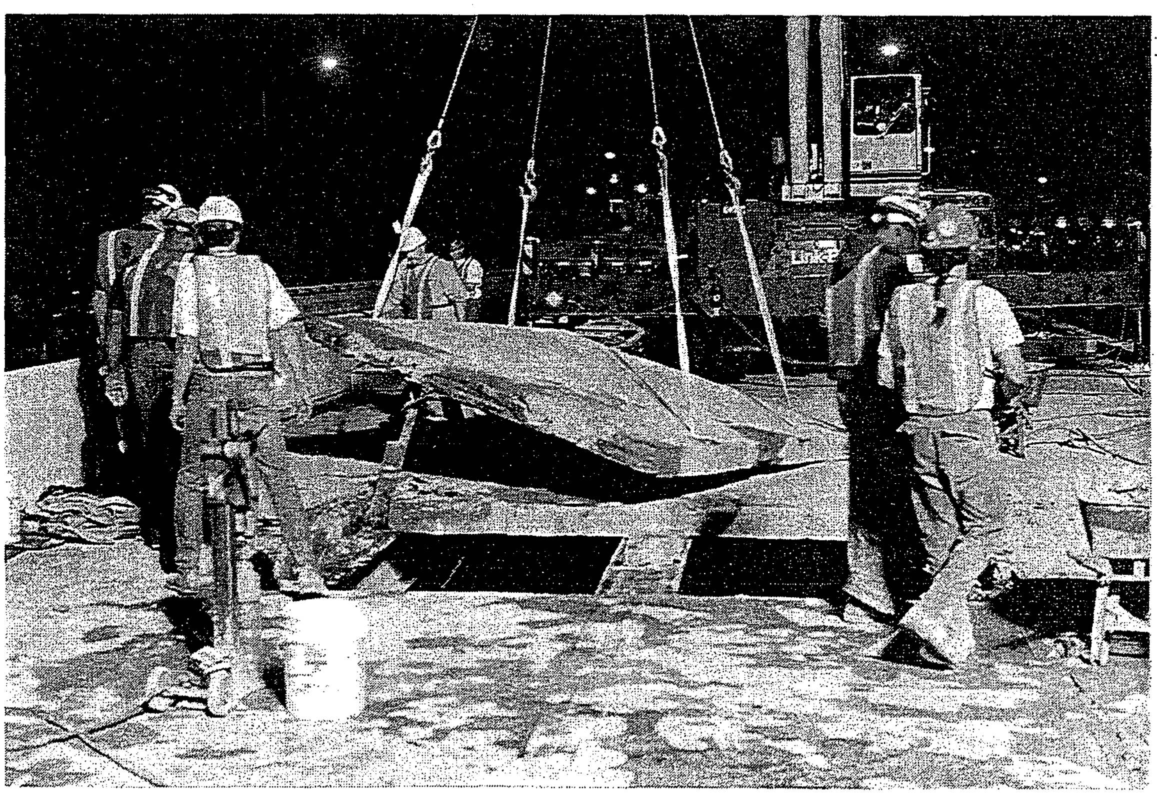


Fig. 9.15 First Section of Existing Deck Removed

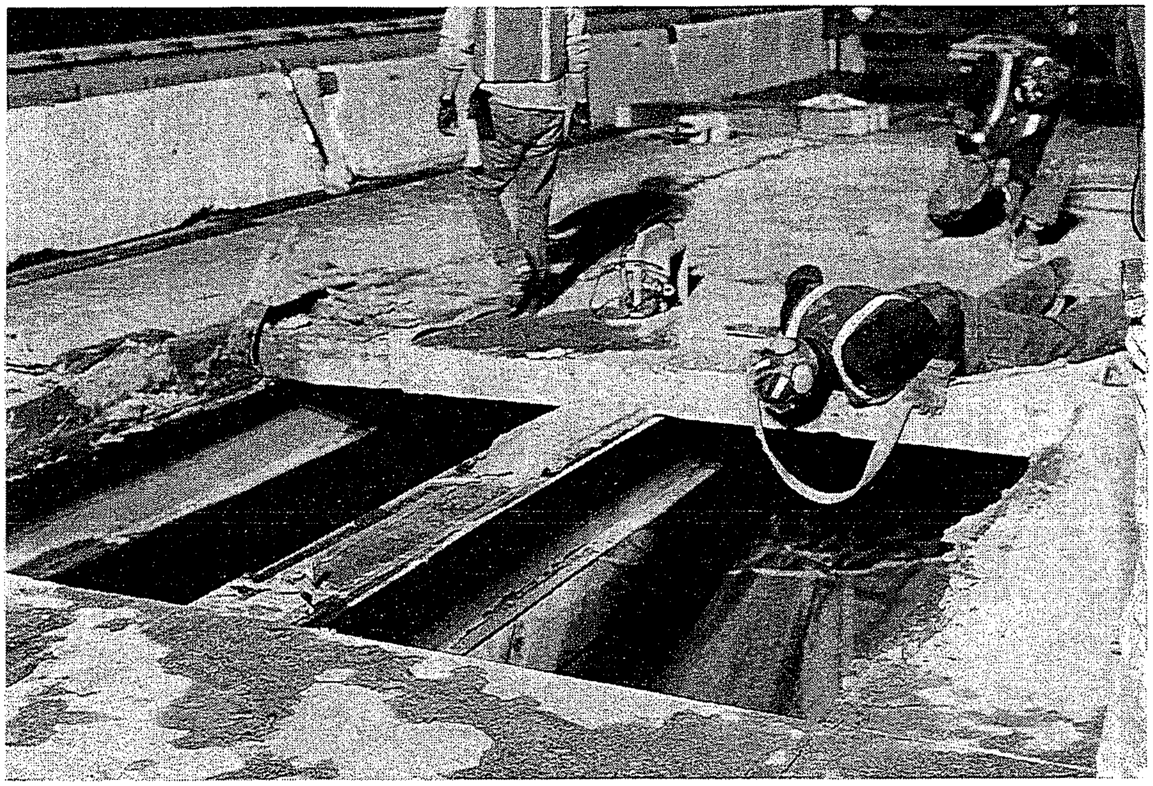


Fig. 9.16 Attaching Lift Webbing for Removal of Second Section

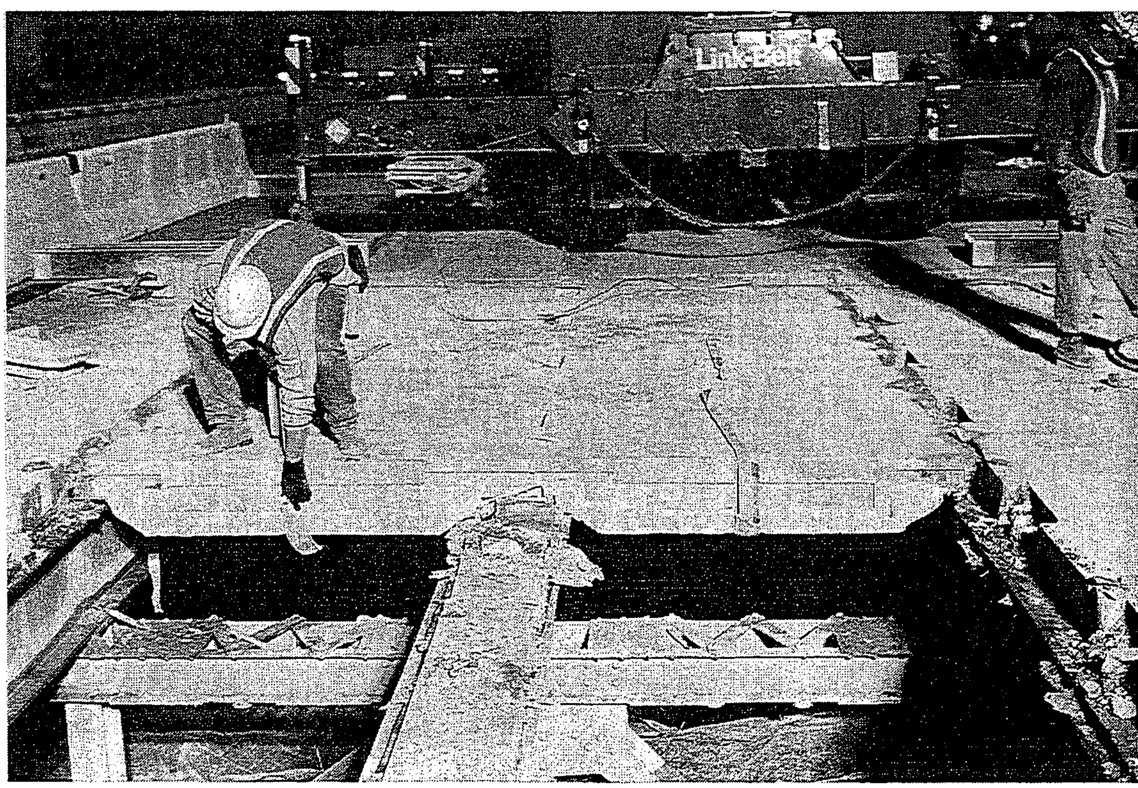


Fig. 9.17 Attaching Lift Webbing for Removal of Third Section

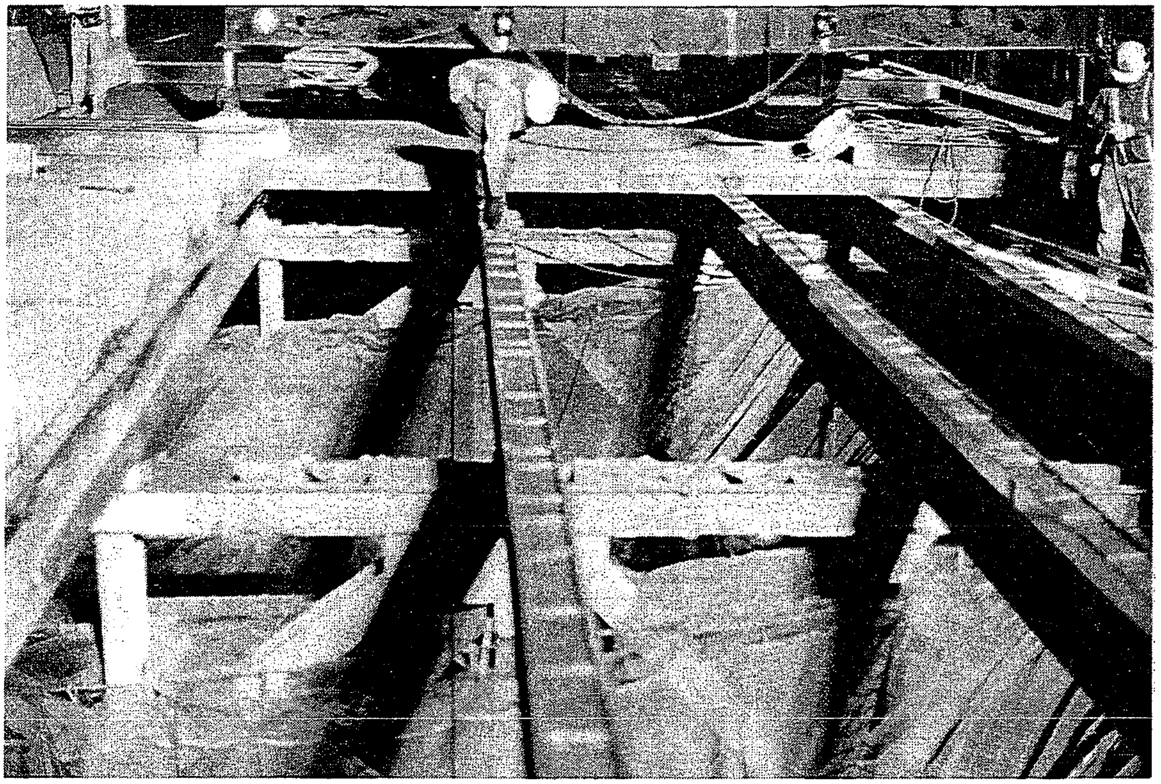


Fig. 9.18 Existing Deck Removed and Setting of Sheet Metal Haunch Forms

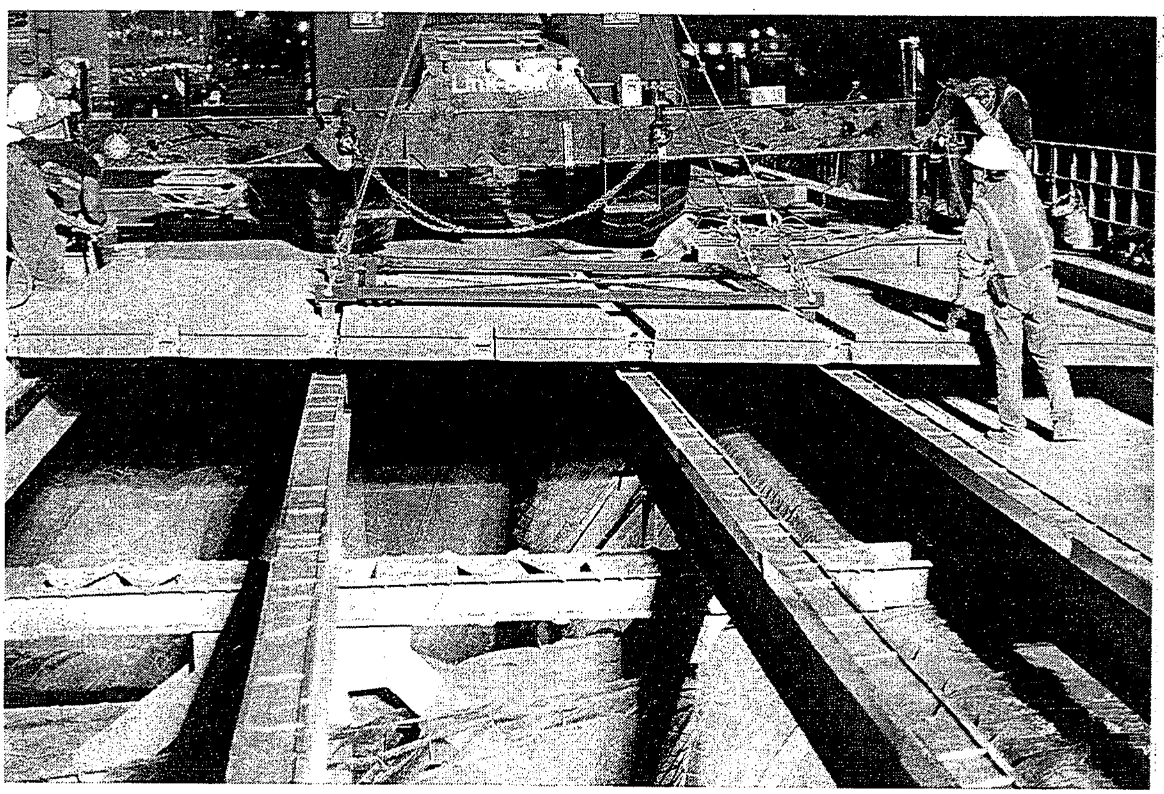


Fig. 9.19 Setting of First Exodermic Panel

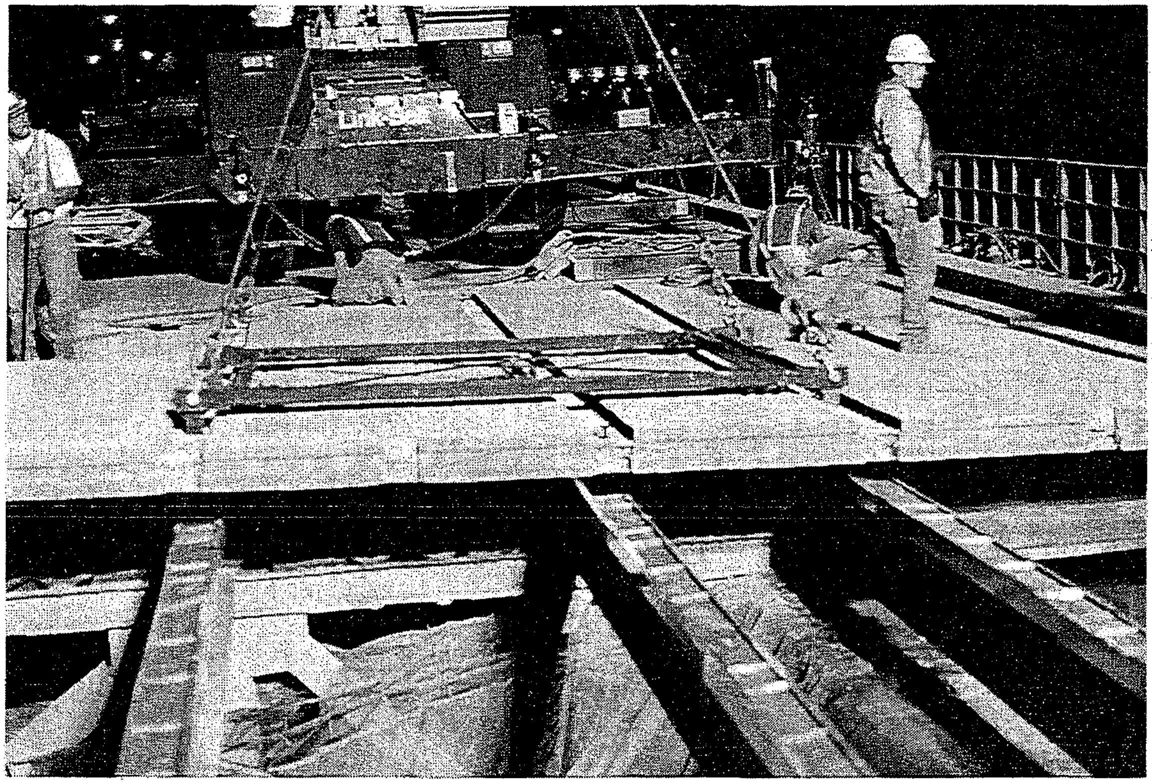


Fig. 9.20 Setting of Second Exodermic Panel

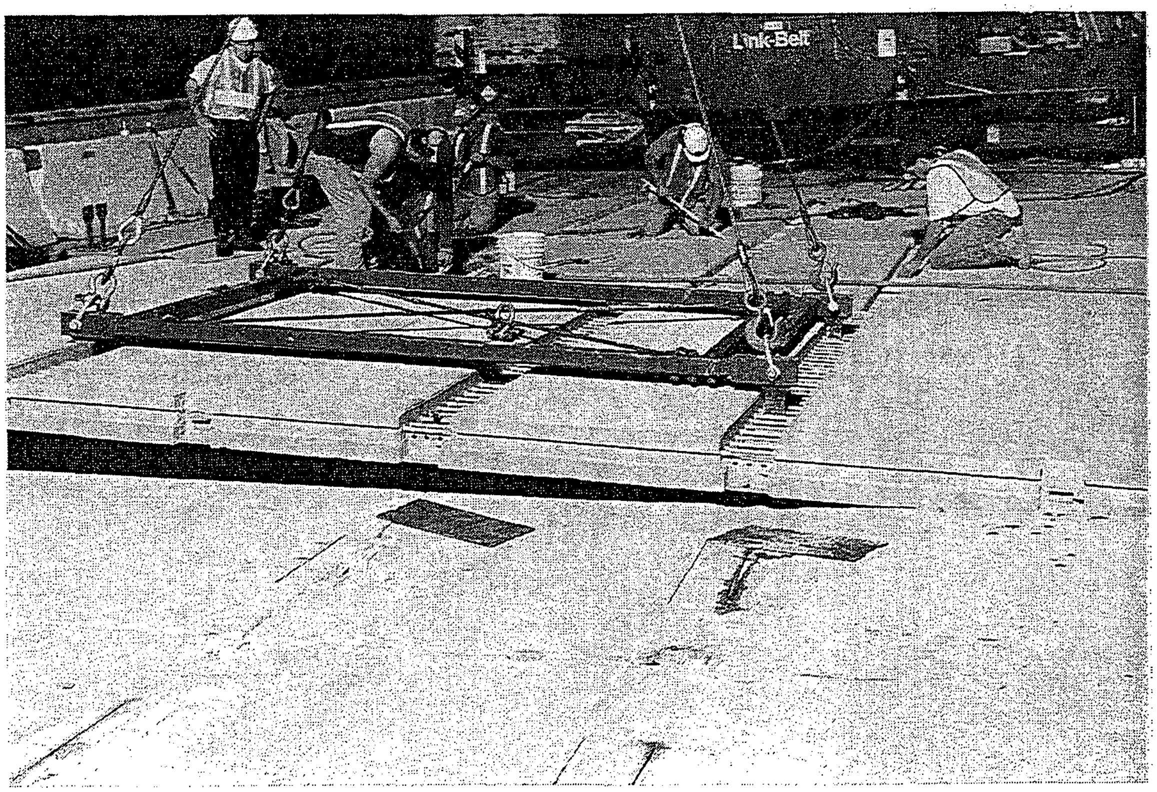


Fig. 9.21 Setting of Third Exodermic Panel

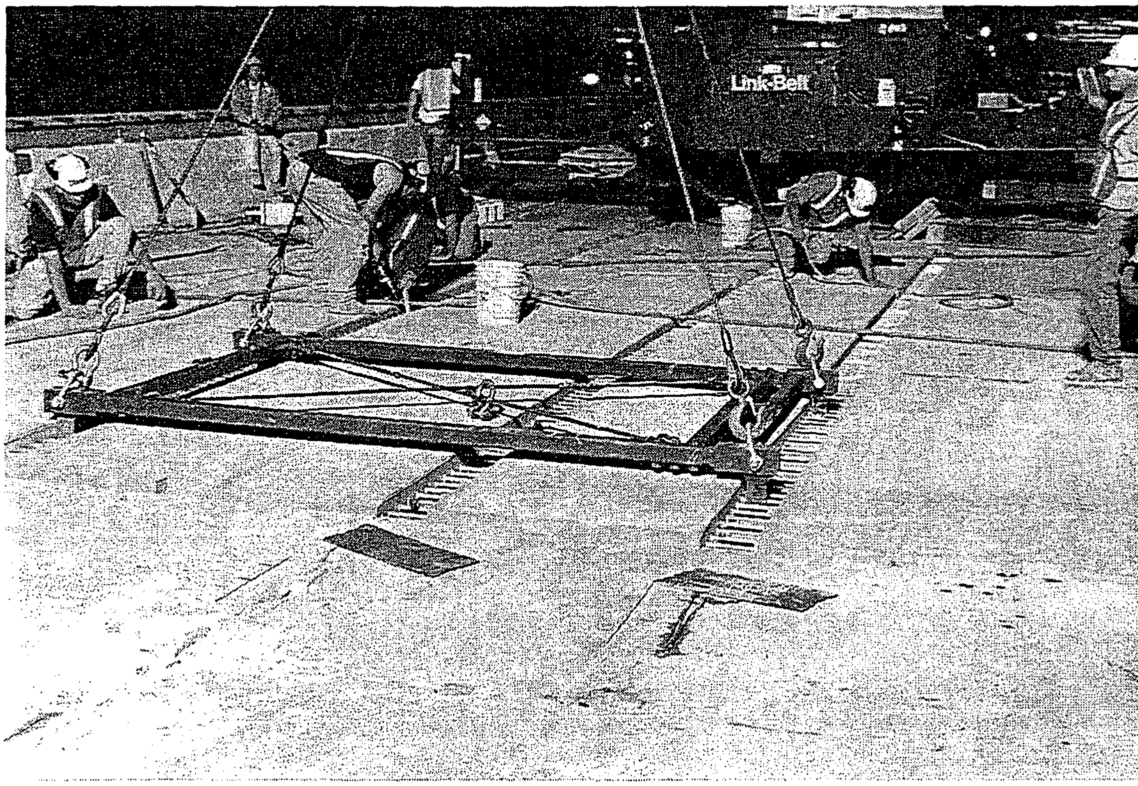


Fig. 9.22 Third Exodermic Panel In-Place

Fig. 9.23 Removal of Temporary Concrete Barrier

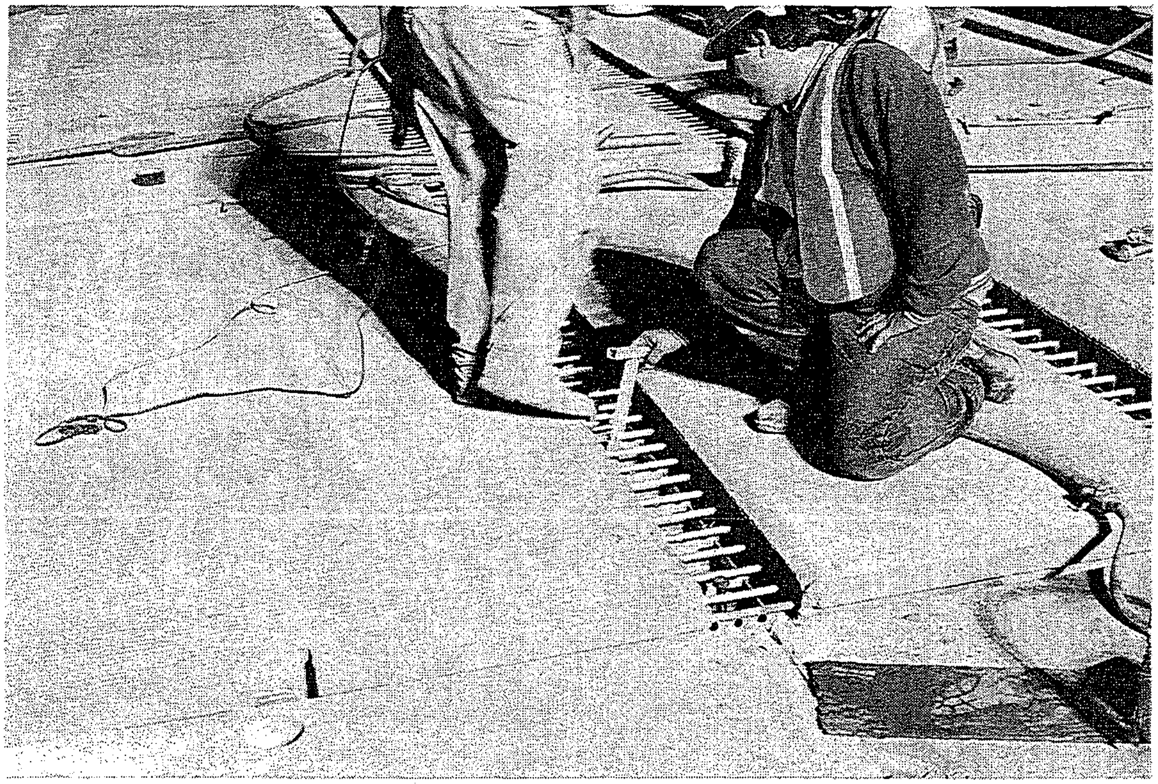


Fig. 9.24 Pulling Up and Securing Sheet Metal Haunch Forms

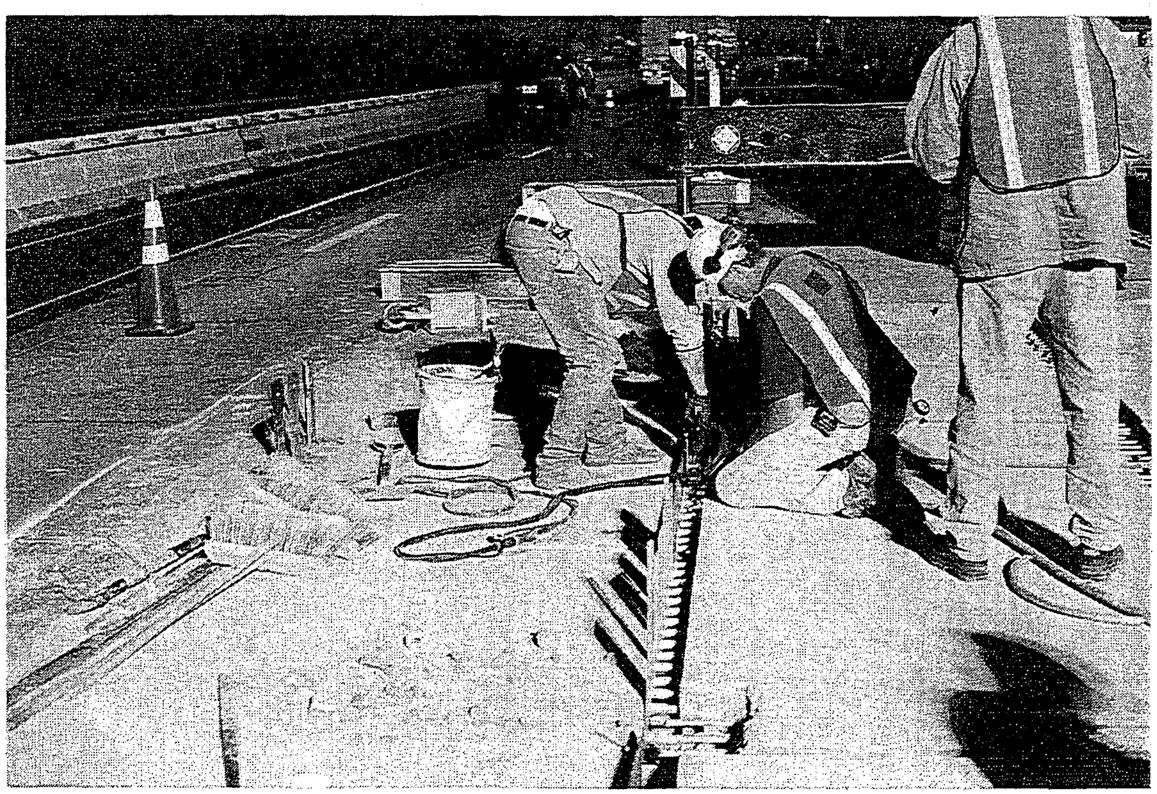


Fig. 9.25 Placing Shear Studs with Stud Gun

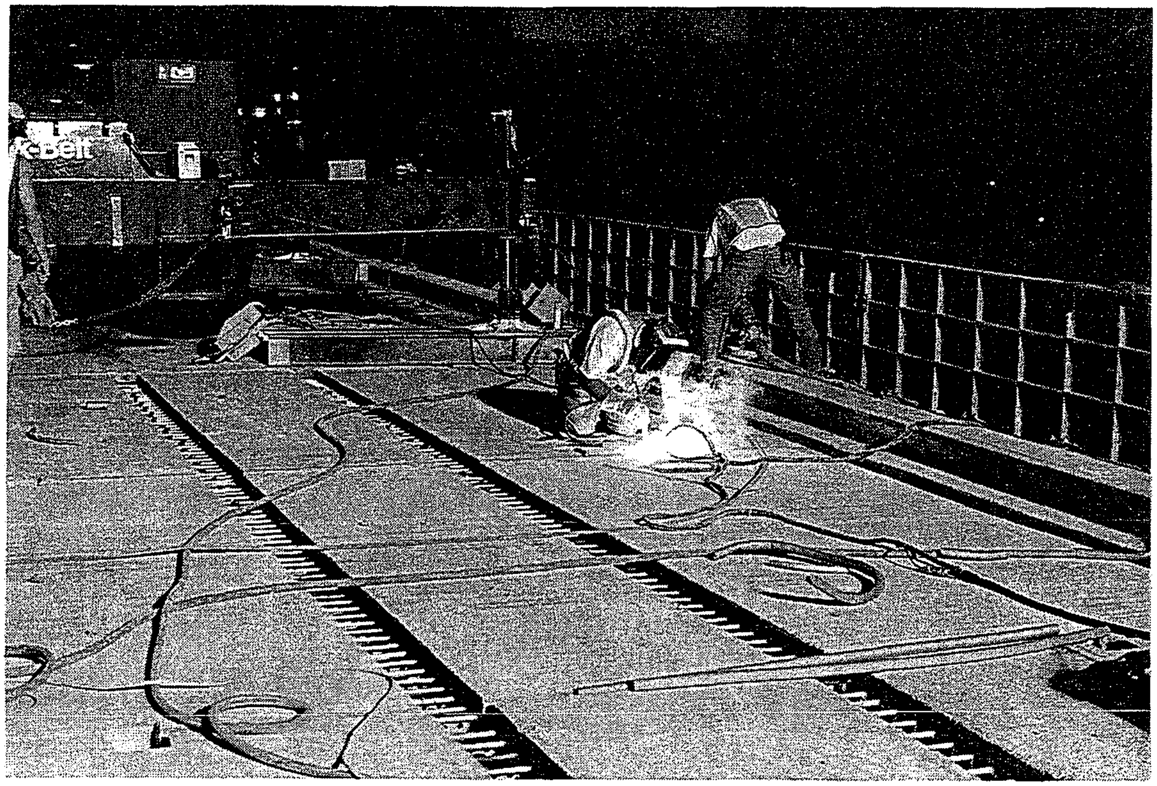


Fig. 9.26 Welding Transverse Joint Connection Strap

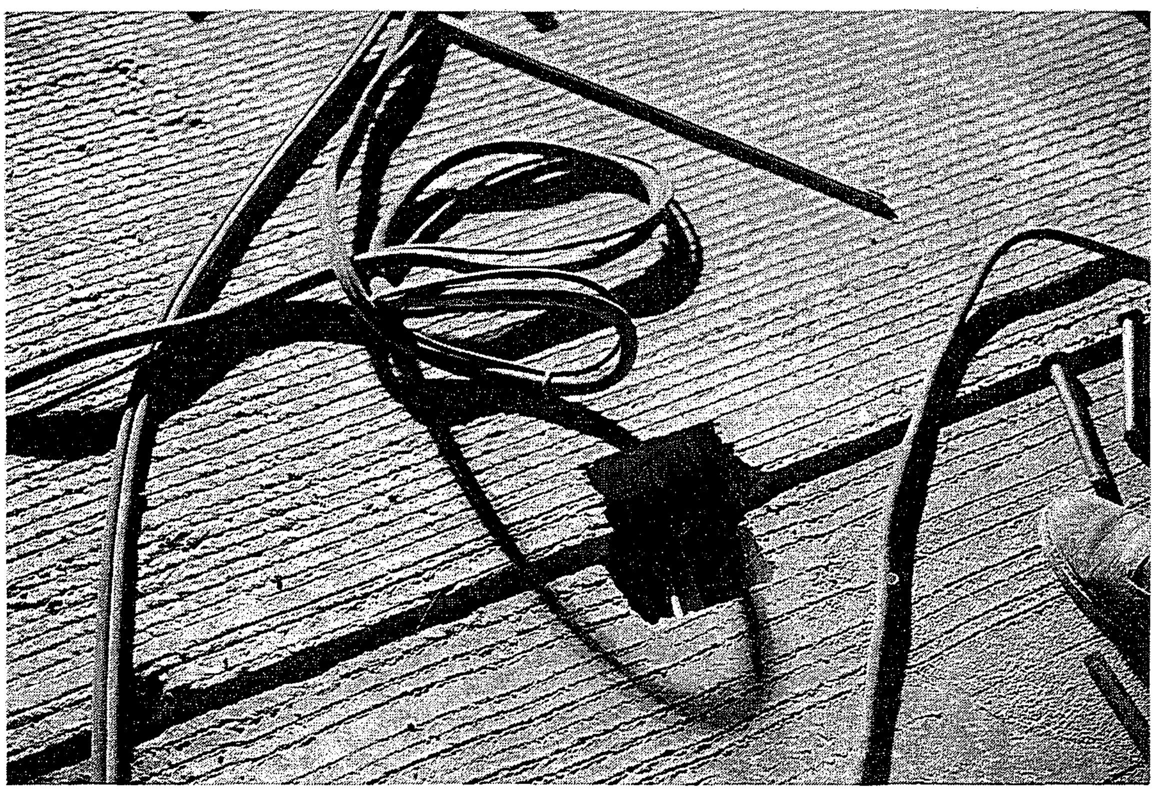


Fig. 9.27 Transverse Joint Connection Strap

Fig. 9.28 Placement of Rapid-Setting Concrete at Stringers and Joints

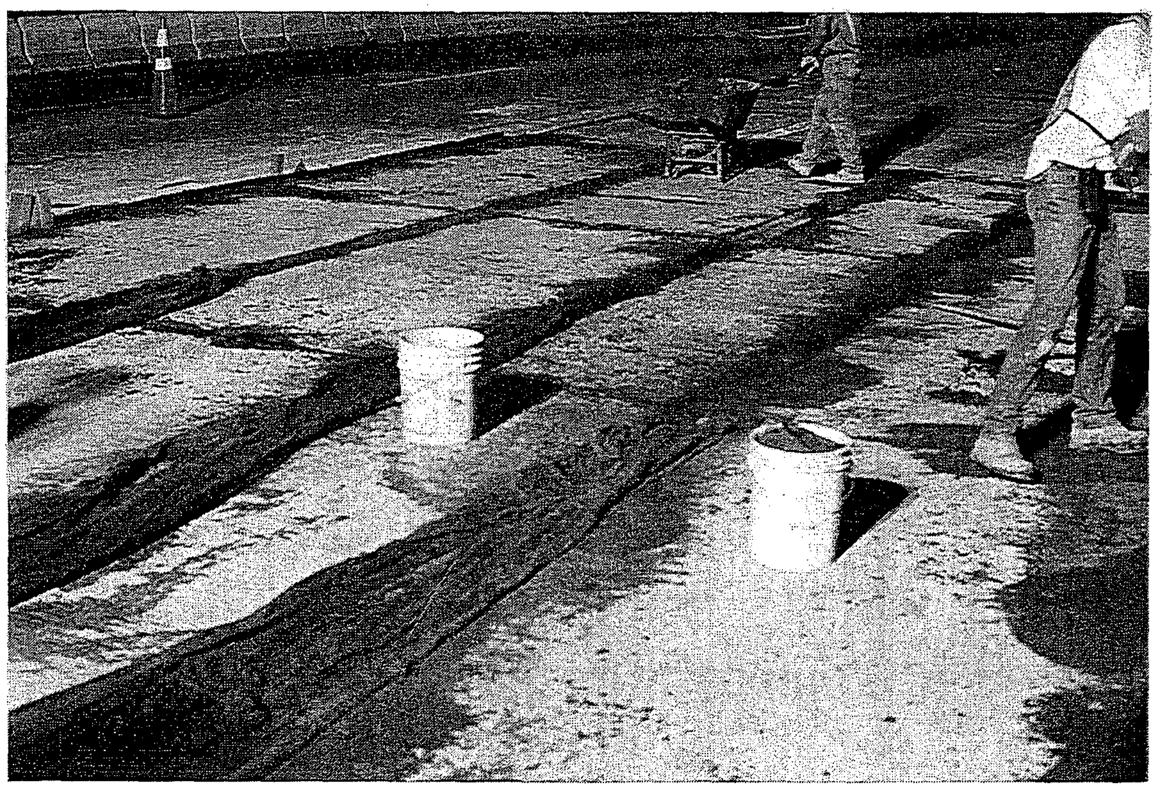


Fig. 9.29 Curing of Rapid-Setting Concrete at Stringers and Joints

10. PRELIMINARY DESIGN FOR NUDECK PRESTRESSED CONCRETE PANELS/CIP DECK

10.1 General

As mentioned earlier in the report, a good candidate for rapid deck replacement is the continuous precast and prestressed SIP panel system (NUDECK) developed by researchers at the University of Nebraska. It is recommended that a test section of this type of bridge deck replacement be installed by ALDOT. This will help quantify construction friendliness, costs, traffic problems, performance, etc., for this system. A 3-span continuous segment of the southbound I-65 bridge in Birmingham, AL, over the railroad tracks between Bents 12 and 15 (see Fig. 10.1) was selected for developing a test section preliminary design. This design was performed by Sherman Prestressed Concrete in Pelham, AL and the authors. Note that this bridge is adjacent to the bridge in Chapter 11 of this report which is recommended for an Exodermic deck replacement test section. Figure 10.2 shows a possible construction-staging scenario for the NUDECK test section project.

10.2 Main Components

This system consists of a 4.5" thick precast continuous panel with a 4.0" thick CIP concrete topping. Figure 10.3 shows a plan view of the panel layout for this system. As mentioned before, this system is continuous in both the longitudinal and transverse directions, which eliminates the reflective cracks that occur when using discontinuous precast panels. The portion of the panels over the girder line is kept open to accommodate shear studs, which can be seen by examining Figures 10.3 and 10.4. This system utilizes transverse joints with reinforced pockets for longitudinal reinforcement. Detail B of Figure 10.5 shows the reinforced pocket. Also, the leveling device used to level the panel is detailed in Figure 10.5, along with two alternatives for shear key connectors. These shear key connectors are located along the transverse joints, in between the reinforced pockets (see Figure 10.3). Detail A and Section D-D of Figure 10.5 shows the reinforced scheme for the SIP panels. For the CIP topping, welded wire fabric may be used instead of conventional reinforcing steel to speed up construction. Figures 10.3 through 10.5 showing the NUDECK preliminary design were prepared by Sherman Prestressed Concrete in Pelham, AL.

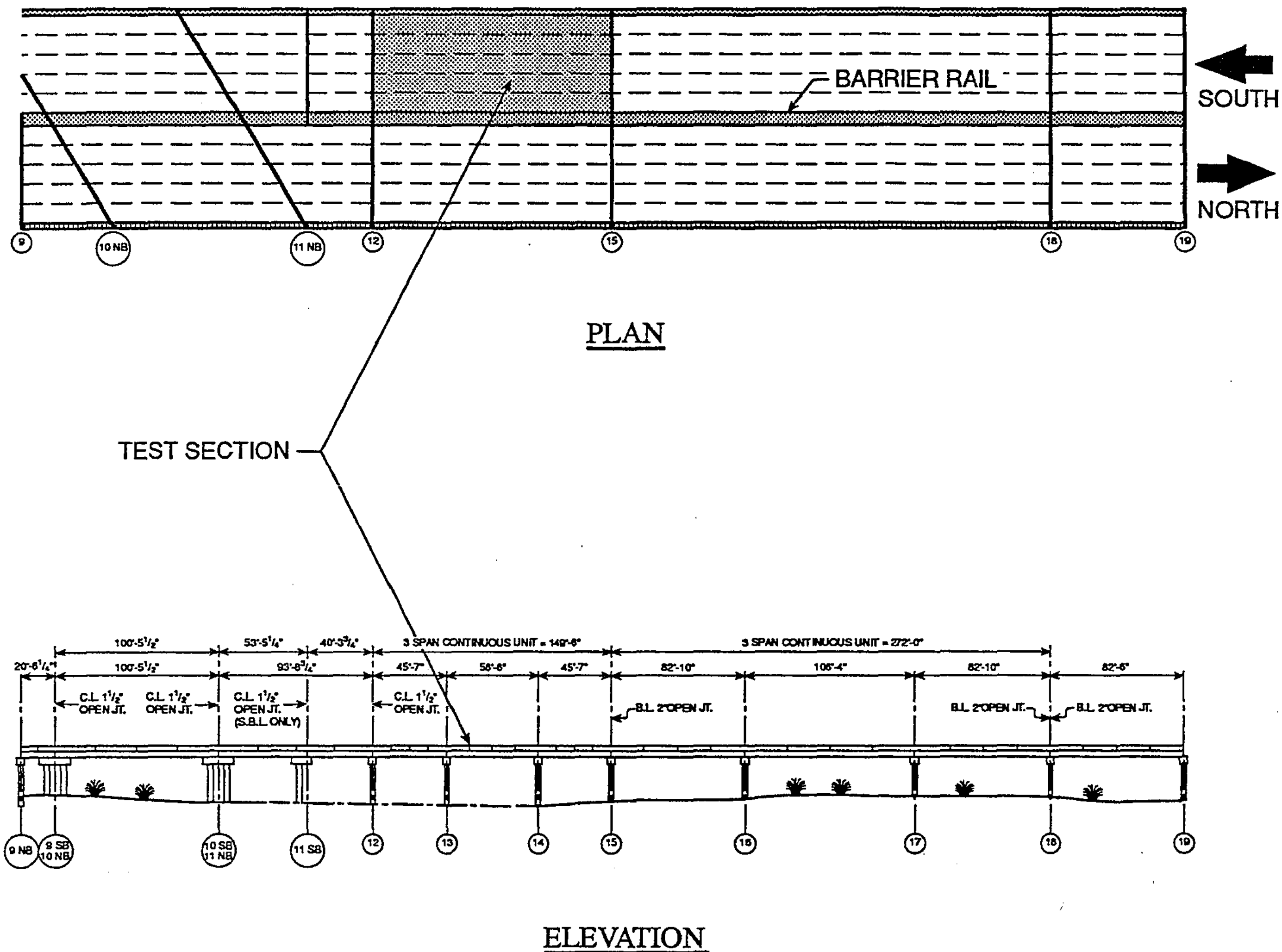


Figure 10.1. I-65 Bridge Bents 9 and 19.

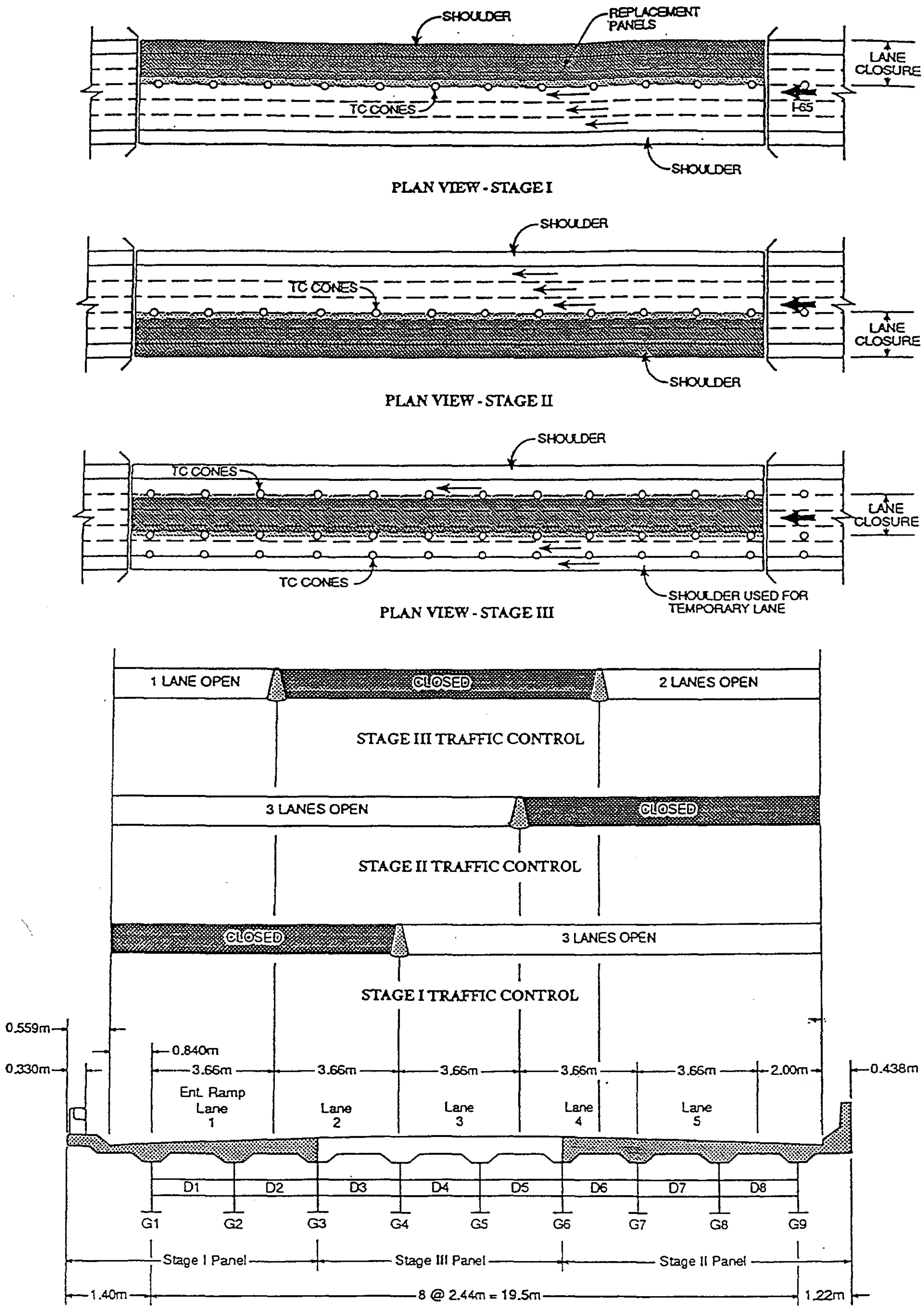


Fig. 10.2. Deck Replacement Staged Construction for the I65 Bridge (Looking into Traffic).

10.3 Preliminary Design

A preliminary design of a NUDECK continuous SIP precast prestressed deck panel system with a CIP concrete topping is presented in Figs. 10.3 - 10.5. Continuous SIP panels with CIP concrete topping were selected over full-depth precast panels because of the need to either overlay or grind the top surface of full-depth precast panel decks. To speed up the field construction process, a Varigrid (or equivalent) mat will be used for reinforcement for the CIP topping layer which will be a rapid-setting high performance concrete. Also, prefabricated Varigrid reinforcing will be cast into the precast panels for the guard rails which will be slip-form casts in the field as indicated in Fig. 10.4.

10.4 Construction Sequence

University of Nebraska researchers estimated the construction time for the NUDECK system to be 20% faster than a conventional CIP system. This would result in the cost estimates for the system being very comparable to those for a CIP deck system. The sequence of construction for Stage I is given in Figure 10.4. However, a generic summary of the construction steps for this system is as follows (68).

- Remove existing slab.
- Clean the girder surface by grit blasting or other acceptable means.
- Glue the grout barrier to the edges of the top surface of the girders (or place grading angles and straps along length of girders.
- Install the precast panels, insert the leveling device, and adjust the panels with the leveling devices.
- Install a backer rod between adjacent panels to prevent leakage during the casting of the CIP topping slab.
- Install shear connectors at required locations.
- Fill the 8" gaps over the girders with a flowable mortar mix or a rapid set, non-shrink grout. Allow to cure to specified strength.
- Install the #4 bar splices in the pockets and adjust the spiral bars into position.
- Install a Varigrid or equivalent reinforcing mat for the CIP topping slab.
- Place the CIP topping concrete and cure. Shear keys and pockets will be cast and cured at the same time as when the CIP topping concrete is cast.

Sherman Prestressed Concrete Co. in Pelham, AL also provided cost estimates for the prestressed panels delivered to the job site. These and other cost estimates related to using the NUDECK panels as a deck replacement option are given in Chapter 13.

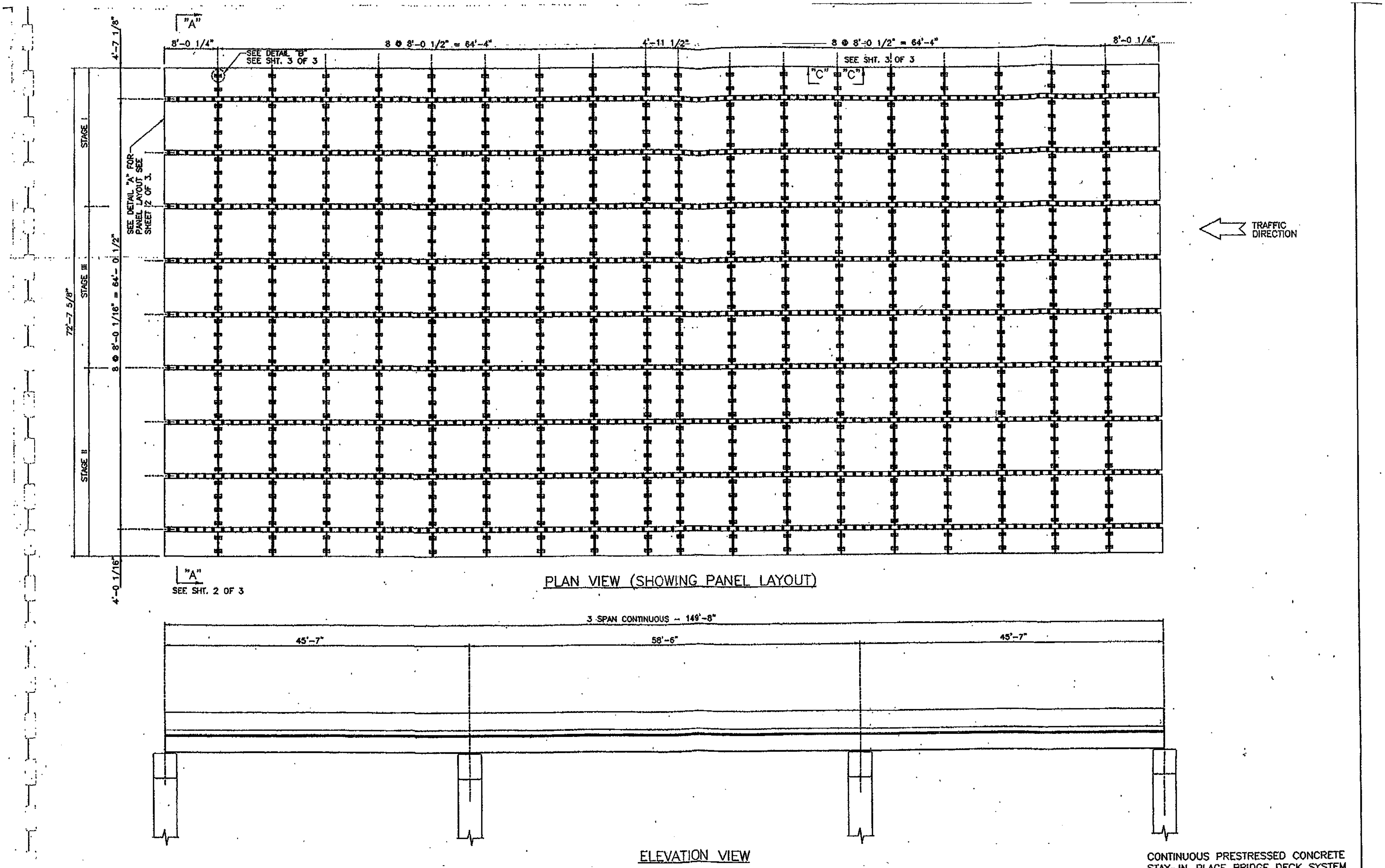
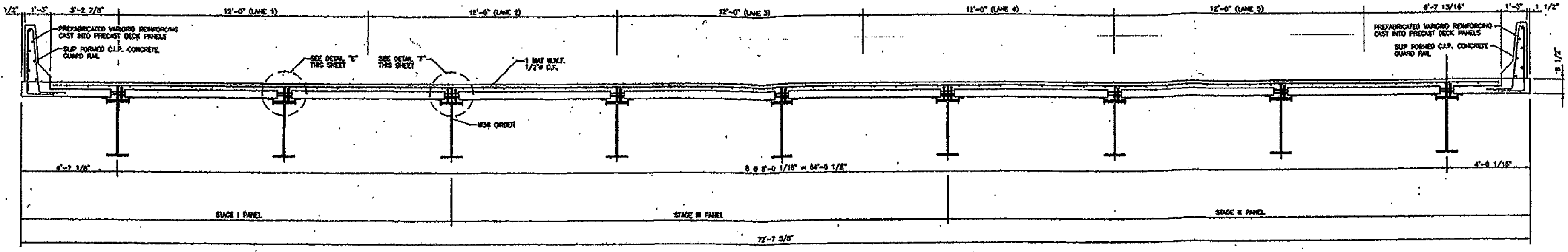
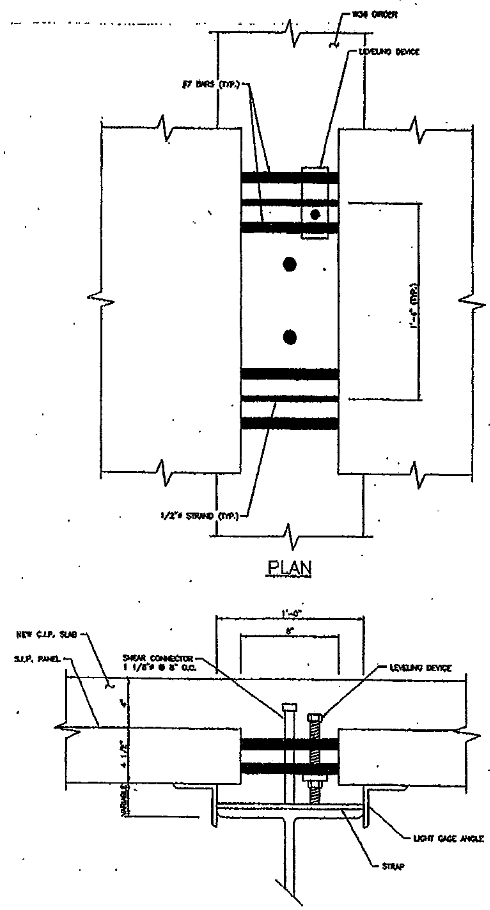
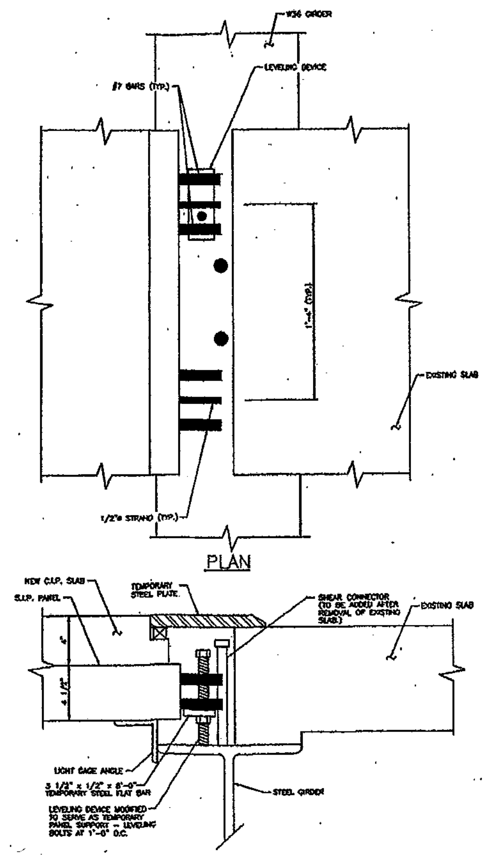


Figure 10.3 Plan and Elevation of Panel Layout.







SEQUENCE OF CONSTRUCTION – STAGE I

(1.) CLOSE LANES 1 & 2 – INSTALL TRAFFIC CONTROL STAGE I

(2.) REMOVE EXISTING SLAB. INSIDE LOCATION IS OVER THE CENTER LINE OF THE GIRDER

(3.) PLACE GRADING ANGLES AND STRAPS ALONG LENGTH OF GIRDERS.

(4.) INSTALL PANELS – SET ALL FOR STAGE I CONSTRUCTION. LEVEL PANELS WITH LEVELING DEVICE AS REQUIRED.

(5.) INSTALL SHEAR CONNECTORS AT REQUIRED LOCATIONS.

(6.) INSTALL BACKER ROD AT TRANSVERSE JOINT LOCATIONS. THEN MAKE CONNECTIONS BETWEEN ADJACENT PANELS (INSTALL SPIRALS AND #4 SPLICE BARS).

(7.) PULL GROUT BARRIER STRAPS AND ANGLES UP SNUG AGAINST THE BOTTOM OF THE PANELS.

(8.) GROUT ALL LONGITUDINAL AND TRANSVERS JOINTS. ALLOW GROUT TOO REACH 4,000 P.S.I. COMPRESSIVE STRENGTH.

(9.) INSTALL W.W.F. REINFORCEMENT FOR 4" THICK C.I.P. CONCRETE TOPPING.

(10.) POUR AND FINISH C.I.P. CONCRETE TOPPING.

(11.) CAST NEW BARRIER RAIL.

(12.) COVER CONNECTION BLOCK–OUTS AT EDGE OF STAGE WITH STEEL PLATE.

(13.) RE–OPEN STRUCTURE TO TRAFFIC UNTIL STAGE II CONSTRUCTION CAN BEGIN.

(14.) ALL GROUT AND C.I.P. CONCRETE TO BE HPC WHICH REACHES DESIRED STRENGTH IN 1 DAY.

Figure 10.4 Transverse Cross Section and Details for NUDECK System.

CONTINUOUS PRESTRESSED CONCRETE
STAY–IN–PLACE BRIDGE DECK SYSTEM
SHEET 2 OF 3

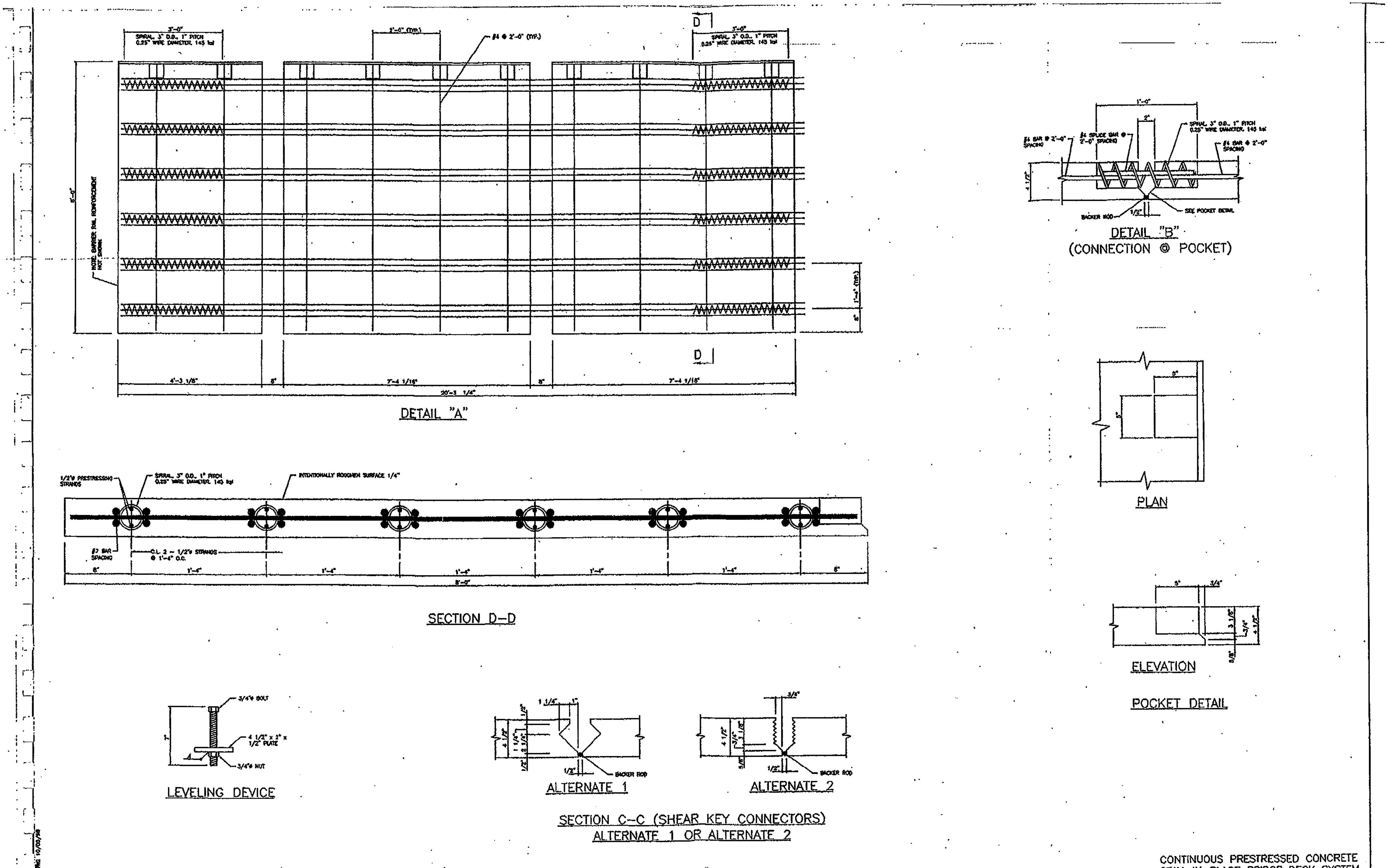


Figure 10.5 Reinforcement Scheme and Other Details for the NUDECK system.

11. PRELIMINARY DESIGN FOR CIP EXODERMIC BRIDGE PANELS/DECK

11.1 General

As mentioned in Section 3.2, an Exodermic deck is a good candidate for rapid replacement of bridge decks. This type of deck, which is a composite of concrete and an unfilled steel grid (see Figure 11.1), maximizes the use of the compressive strength of concrete and the tensile strength of steel as indicated in Figure 11.2.

In the following pages, a preliminary design for an Exodermic bridge deck replacement on the Birmingham bridges will be presented. This particular design is intended for illustrative purposes only, and should not be implemented.

The bridge chosen for the preliminary design is a 3-span continuous segment of the northbound I-65 bridge over the railroad tracks between Bents 12 and 15 as indicated in Figure 11.3. The deck will be replaced in a rapid and staged construction manner in order to maintain and minimize disruptions to traffic. Three construction stages are planned as indicated in Figure 11.4. The parameters for this particular bridge which will be used in the design are; a replacement deck thickness of 8.25", a live-load rating of HS-25, and a girder spacing of 8'.

The *Exodermic Bridge Deck Handbook* (20), which is published by the Exodermic Bridge Deck Institute (EBDI), was used extensively for the creation of this preliminary design. The available Exodermic Handbook at this time (August 1998) features the earlier Exodermic design and therefore the preliminary design in this report is of the earlier type. It is believed that this will be sufficient for the preliminary design, since the earlier and revised designs are very comparable. But, it should be noted here, as in Section 3.2, that the revised design has been estimated to save the owner over $2 per square foot verses the earlier design.

In designing an Exodermic bridge deck panel, several choices must be made by the designer in response to project specific considerations. These considerations include choosing a precast or cast-in-place panel, maintenance of traffic, desired deck thickness, required concrete cover, desired live-load rating, weight limitations, deck layout (main bearing bars parallel or transverse to traffic), and strength of the superstructure. With these considered, the designer chooses main bearing bar type and spacing, rebar size and spacing, and concrete type.

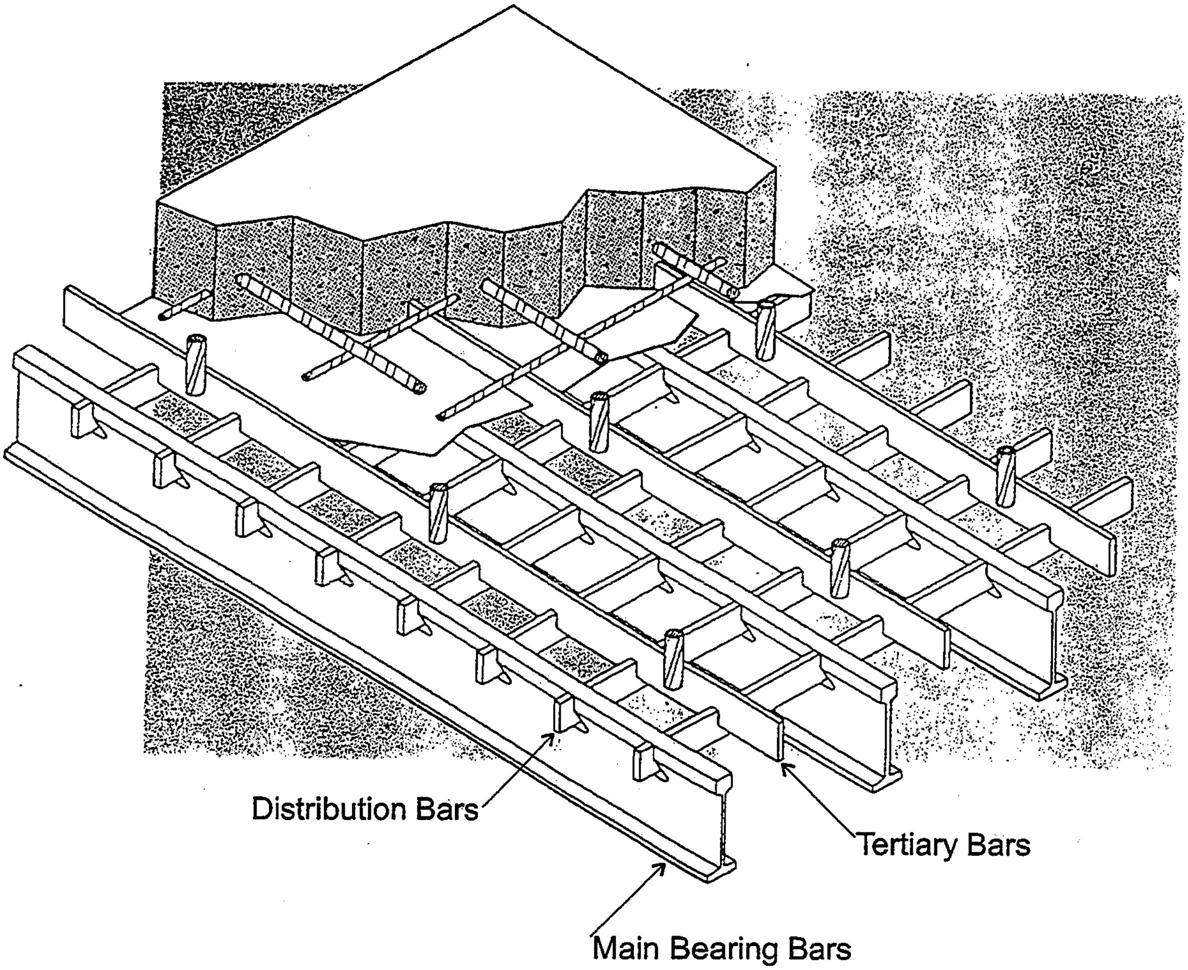
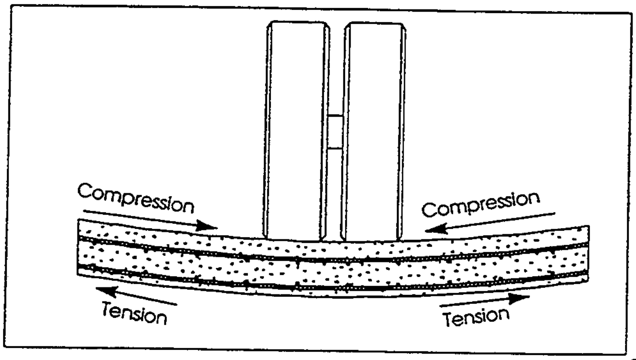


Fig. 11.1. Cut Away View of Portion of Exodermic Panel (26).

POSITIVE FORCES

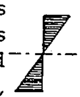


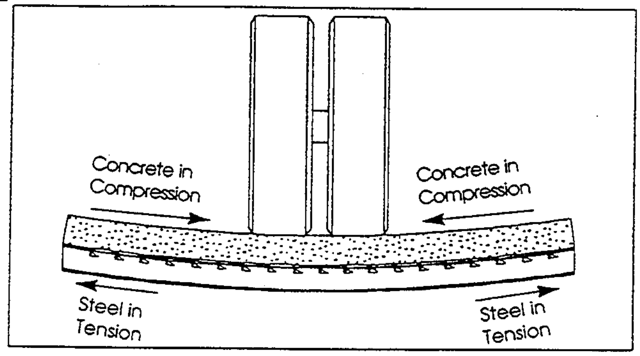
Standard Reinforced Concrete Deck

In a standard reinforced concrete deck, in positive bending, the concrete at the bottom of the deck is considered 'cracked' and provides no practical benefit. Thus, the effective depth of the slab is reduced, and the entire bridge, superstructure and substructure, has to carry the dead load of this 'cracked' concrete.

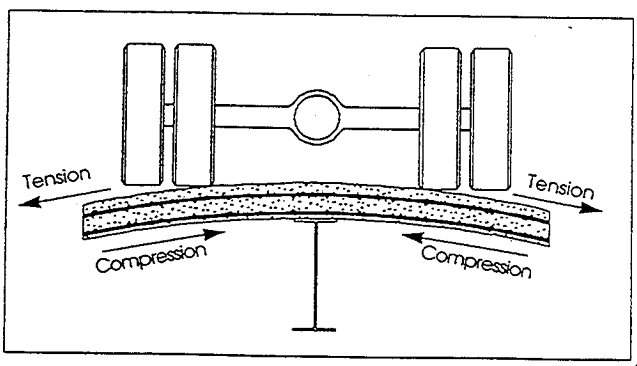
Exodermic Deck

In an Exodermic deck, in positive bending, essentially all of the concrete is in compression and contributes fully to the section. The main bearing bars of the grid handle the tensile forces at the bottom of the deck. Because the materials (steel and concrete) in an Exodermic deck are used more efficiently than in a reinforced concrete slab, an Exodermic design can be substantialy lighter without sacrificing stiffness or strength.



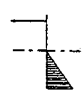


Negative Forces



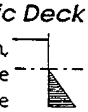
Standard Reinforced Concrete Deck

In negative bending, a standard reinforced concrete deck handles tensile forces with the top rebar; concrete handles the compressive force at the bottom of the deck.



Exodermic Deck

Similarly, in an Exodermic design, the rebar in the top portion of the deck handles the tensile forces, while the compressive force is borne by the grid main bearing bars and the full depth concrete placed over all stringers and floorbeams.



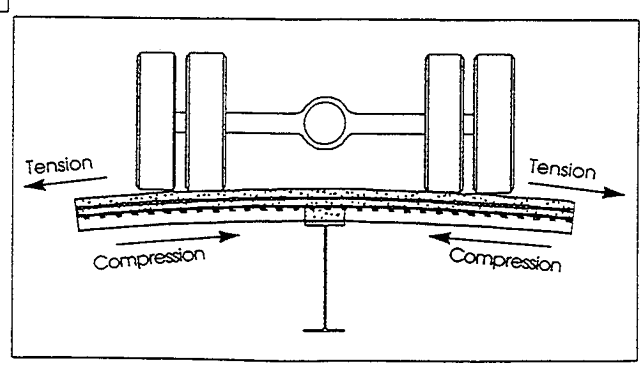


Figure 11.2 Exodermic Deck-Load Transmittal and Stresses (26).

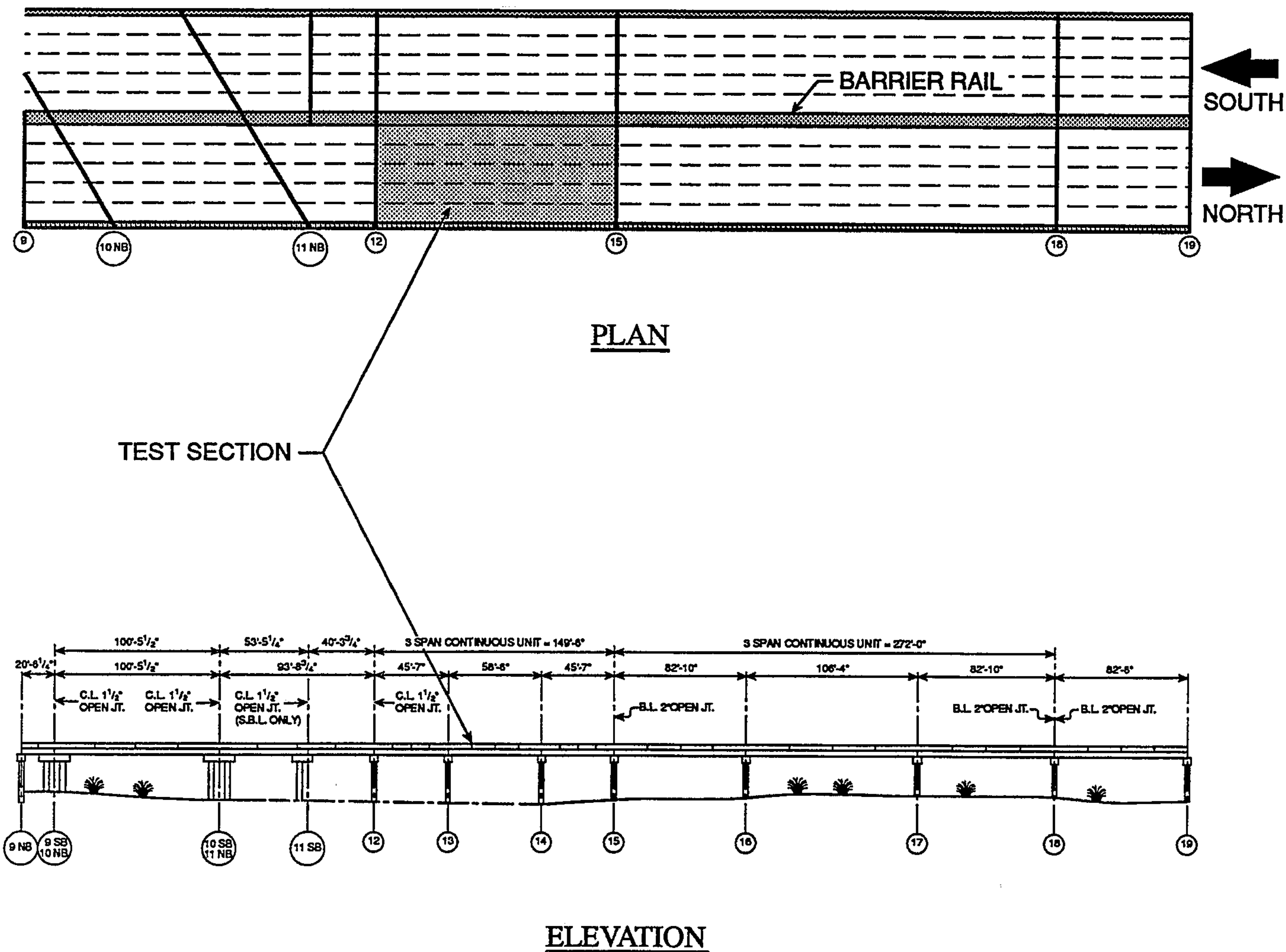


Figure 11.3. I-65 Bridge Bents 9 and 19.

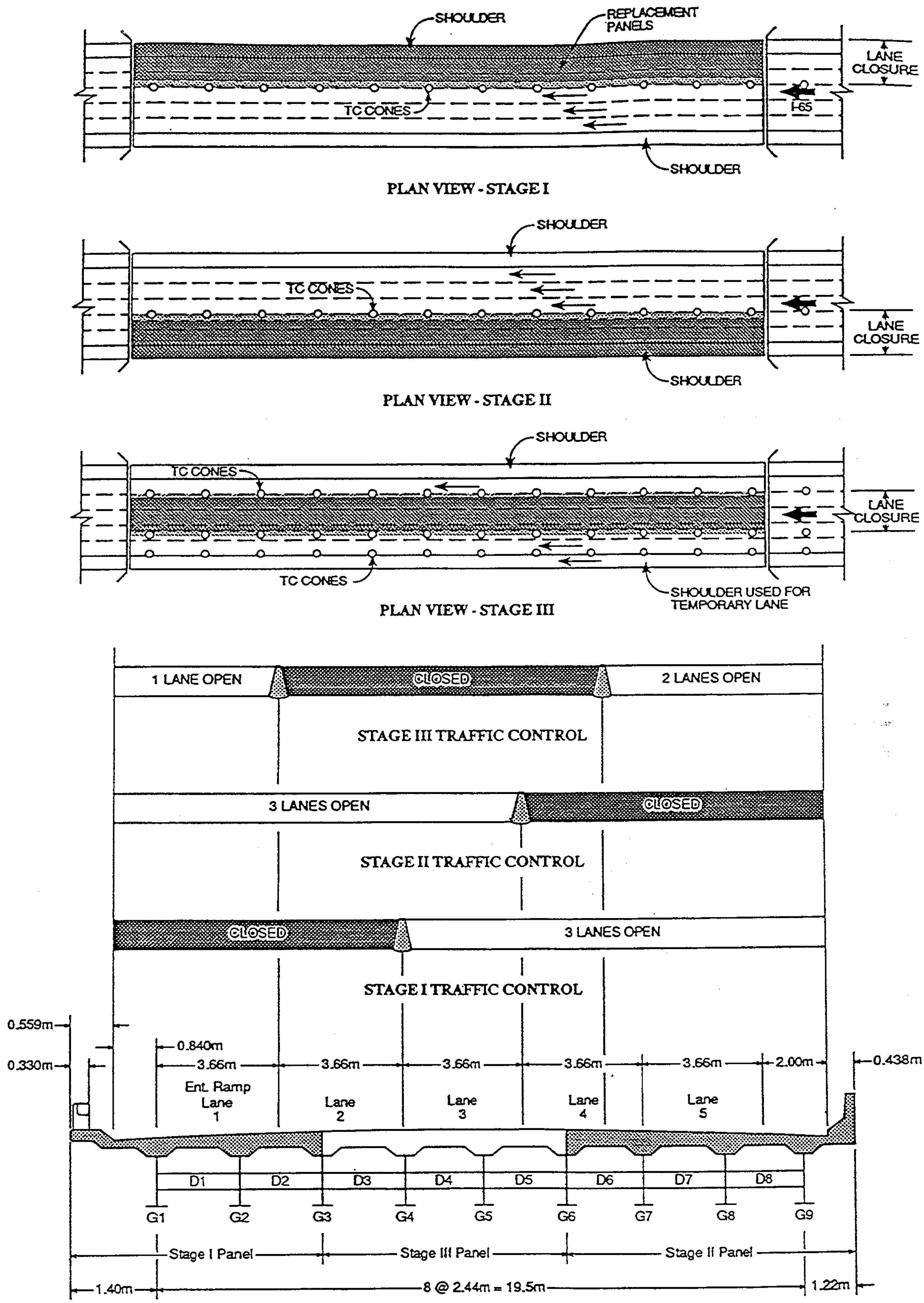


Figure 11.4 Deck Replacement Staged Construction for the I65 Bridge (Looking into Traffic).

11.2 Main Components

Main Bearing Bars - Even though any steel grid can be used to construct an Exodermic deck, EBDI recommends the use of industry standard grid configurations where possible to avoid costs associated with new tooling. The types of main bearing bars (see Fig. 11.1) employed in the standard grids are: 3" structural T, 4-1/4" special section rolled I beam, and 5-3/16" special section rolled I beam. The choice of which bearing bar to use may depend on desired deck thickness, or can depend on dead-load limitations. The grid for and Exodermic Deck, as for all steel grids, should be protected from the elements. Three methods that have been used with Exodermic Decks are painting and epoxy coating, galvanizing, and the use of weathering steel. However, galvanizing has proven to provide the longest lasting protection. Galvanizing is the best choice for grid protection from a life cycle cost point of view. It has been used much more extensively than the other options. Distribution and tertiary bars, while not considered main components, are important components of the grid. The distribution bars serve to distribute loads across the several main bearing bars. The tertiary bars provide horizontal shear transfer, assuring full composite action between the grid and the concrete, in addition to adding to the overall strength of the deck.

Rebar - Rebar size and spacing is chosen based on the negative moments to which the deck will be subjected. Rebar can be selected to permit overhangs and decks that are continuous over stringers of floorbeams spaced well apart. The temperature/distribution rebar is typically #3 spaced at 4" to 6", while the main rebar are typically #5 bars. The main (top) rebar is generally spaced at 50%, 75%, or 100% of the main bearing bar spacing.

Concrete - Weight constraints, deck thickness limitations, and desired cover over rebar all must be considered when selecting the thickness of the concrete component of the Exodermic deck. The total concrete thickness usually ranges from 3-1/2" to 4-1/2", providing a top cover from 1-1/2" to 2-1/2". Since this concrete is very thin, it will be susceptible to plastic shrinkage cracking. This would be detrimental to the important riding surface of the deck. Therefore, it is imperative that proper curing techniques be used while placing the topping on an Exodermic deck.

11.3 Preliminary Design

Before the preliminary design was started, some design criteria and assumptions were set forth. These criteria and assumptions were used in the design examples in the Exodermic Bridge Deck Handbook.

11.3.1 Criteria and Assumptions. The design code used in this preliminary design (as those in the Exodermic Bridge Deck Handbook) is the AASHTO 1992 15^{th} Edition as amended, using the transformed area method (3.27.2.2) service load (working stress) design. Numbers shown in parentheses give the operative sections in the AASHTO code.

Materials

The material specifications for the preliminary design are as follows:

- A36 steel $0.55f_y$=20ksi (10.32.1)
- A588 (or A572 Gr 50) $0.55f_y$=27ksi (10.32.1)
- steel weight = 490lb/ft^3
- concrete weight = 145 lb/ft^3 (10.38.13)
- f_c'=4000psi; n=8 (10.38.1.3)(8.15.3.4)
- concrete not considered in tension regions

Loads and Moments

- impact = 30% (3.8.2.1)
- continuity factor; 0.8 for dead and live load (3.24.3.1)
- dead load moment, cast-in-place: grid only
- live load: composite positive and negative moment
- span length, S=distance between edges of stringer flanges +1/2 flange width (3.24.1.2.b)
- main bearing bars transverse to traffic:

  $M_{DL}=WS^2/8$ (x 0.8 if continuous)

  $M_{LL}=[(S+2)/32]xP$ (3.24.3.1)

  for HS-25, P=20x1.3x0.8, where 1.3 is the factor for impact, and 0.8 for continuity
- distribution factor S/5.5 (3.23.2.2)
- deflection limited to l/800 (10.6.2)

11.3.2 Design Tables. The Exodermic Bridge Deck Institute provides several design tables in the Exodermic Bridge Deck Handbook. These tables are given in Appendix E (Tables E.1 through E.4). The tables are based on using an 8 digit configuration code made up of main bearing bar type, main bar spacing, concrete thickness, top rebar size, and top rebar spacing. The tables were intended to aid the designer in deciding what deck configurations should be looked at in more detail. They were generated using a Microsoft Excel for Windows spreadsheet. Table E.1 provides a starting point for design where deck thickness or deck weight are critical. Table E.2 suggests possible deck configurations as a function of span, where deck thickness is not constrained by project conditions. Tables E.3 and E.4 show certain quantities and design properties for given deck configurations.

11.3.3 Analysis Steps. The following steps were used in selecting and verifying the adequacy of the deck components.

1. Select the candidate deck type using the various tables.
2. Calculate deck weight.
3. Derive section properties.
4. Calculate stresses in the deck under design load, and compare to allowable.
5. Check deck deflection.
6. Check deck cantilever if required.
7. Derive the section properties of the deck/girder.
8. Check girder stress level and deflection under design load.

11.3.4 Results of the Preliminary Design Process. From Tables E.1, E.2, and E.3, the Exodermic Type 40640506 is recommended for the I-65 bridge. This design configuration code translates into 4.25" main bearing bars spaced at 6" c-c, 4" thick concrete component, and #5 top rebars for a total deck thickness of 8.25". These are reiterated below, along with the other components

Grid:

- main bearing bars: 4.25" I special rolled at 6" c-c
- distribution bars: 1.25" x 0.25" at 4" c-c
- tertiary bars: 1.5" x 0.25" at 6" c-c, extending 1" into the slab

Rebar:

- bottom rebar (distribution/temperature steel): #3 bars at 4" c-c
- top rebar: #5 bars at 6" c-c

Concrete:

- concrete: 4" normal weight concrete, 145 lb/cu.ft., 4000 psi compressive strength, cast-in-place; there will be 2" concrete cover over rebar
- EBDI recommends 3/8" maximum aggregate size to ensure filling of all crevices and voids

Deck panels are to be 8' wide with a variable length (Stage I, II or III panels - see Figure 11.4). See Appendix E for design calculations for deck weight, section properties, and stresses and deflection under design load. Figure 11.5 shows typical Exodermic panels for the Birmingham bridge, and typical connections between panels and for panels which are adjacent to the existing deck (temporary). Note that Figure 11.5 shows both the option of adhesive backed foam and the option of using sheet-metal forms for forming haunches. Section D-D shows the use of adhesive backed foam and Section B-B shows the use of sheet metal forms. Figure 11.6 shows three different types of barrier rails that could possibly be used. Cross-section A shows a precast concrete barrier section taken from the Exodermic Bridge Deck Handbook. Cross-section B shows a particular type of bolted concrete barrier rail. This may be the best choice for the Exodermic Deck, since the concrete layer is only 4" thick. Cross-section C shows ALDOT's standard cross section for concrete barrier rail. Cost estimates for employing Exodermic panels as a deck replacement option are give in Chapter 13.

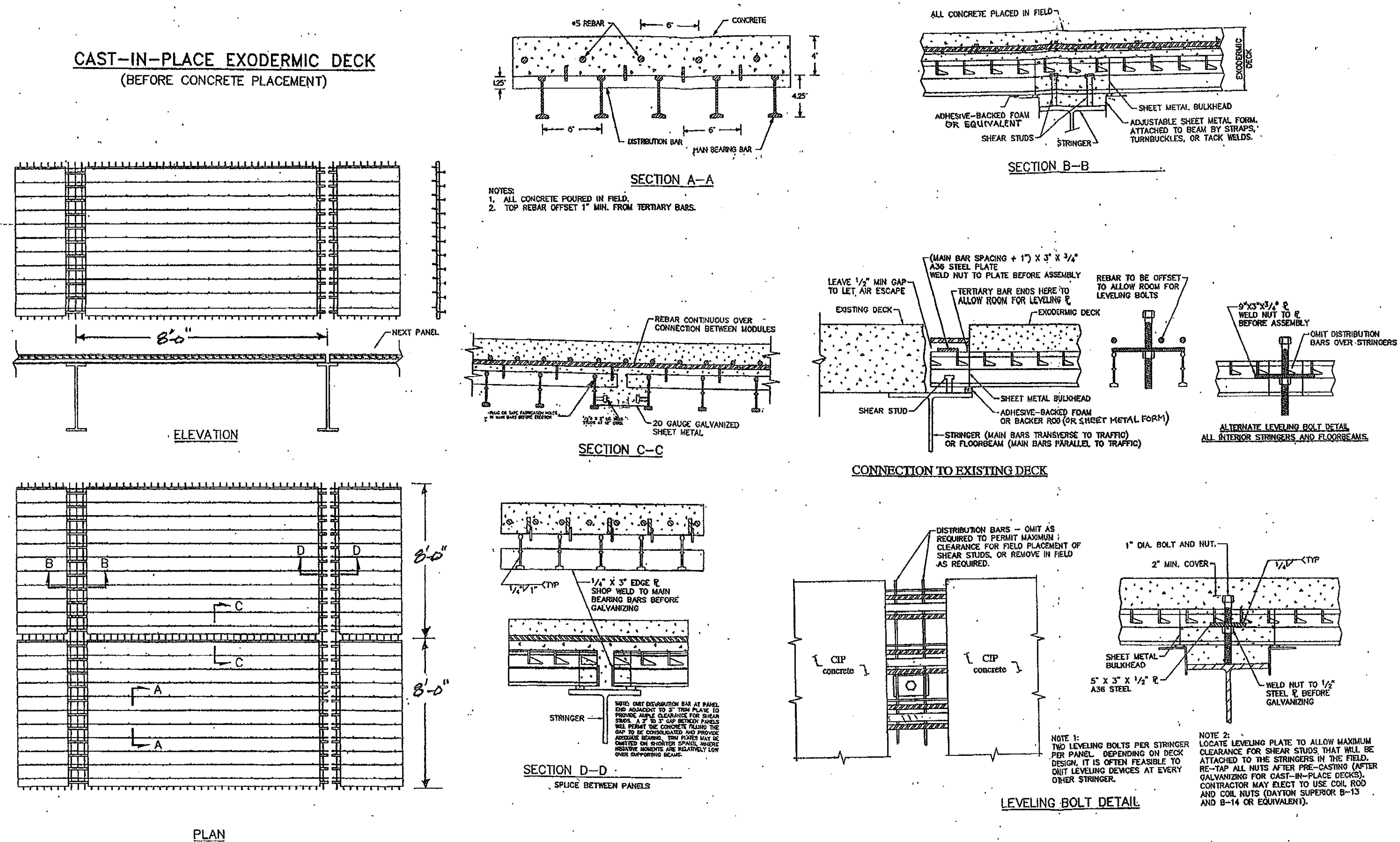
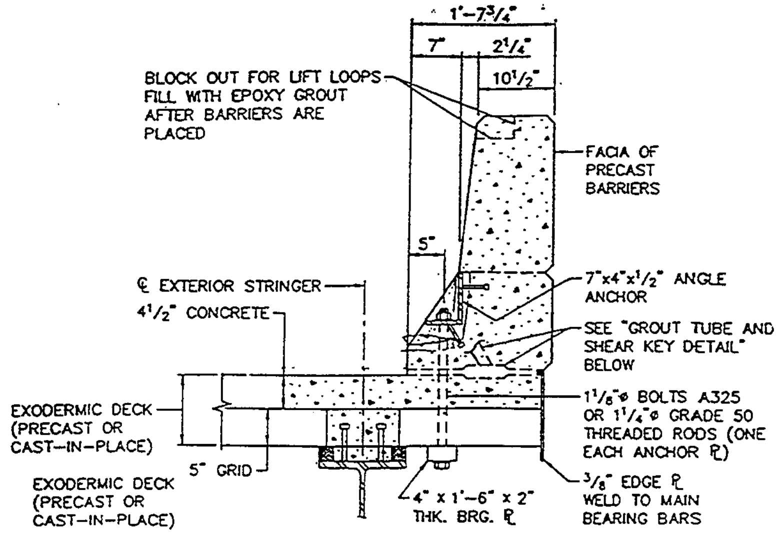
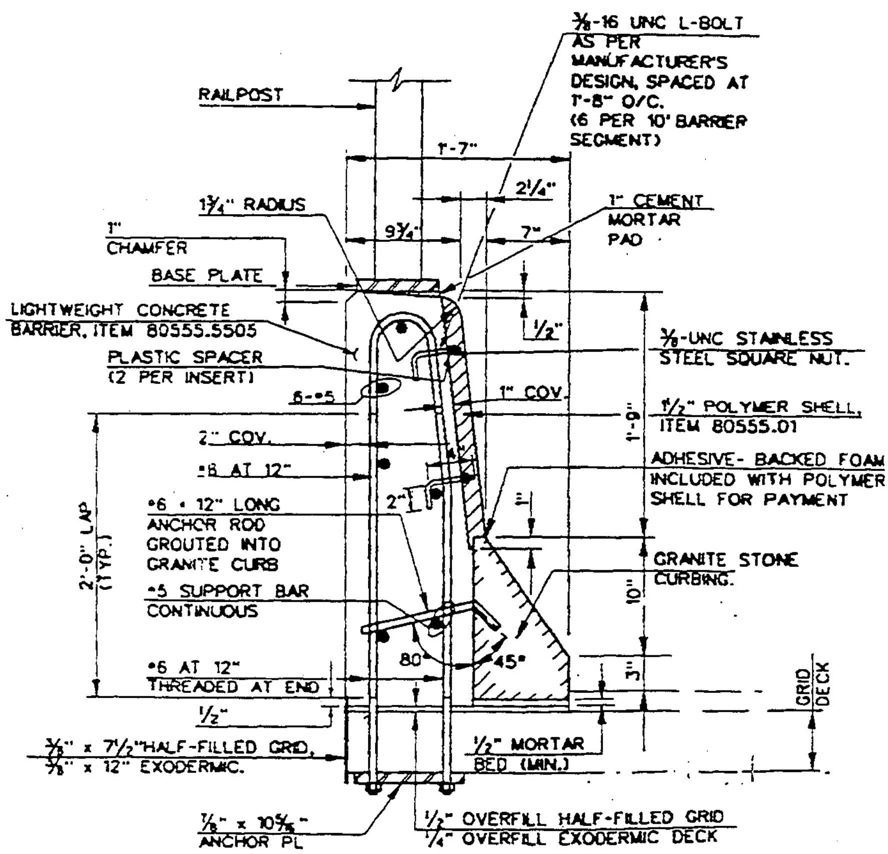


Figure 11.5 Details for Cast-in-Place Exodermic Deck (26).

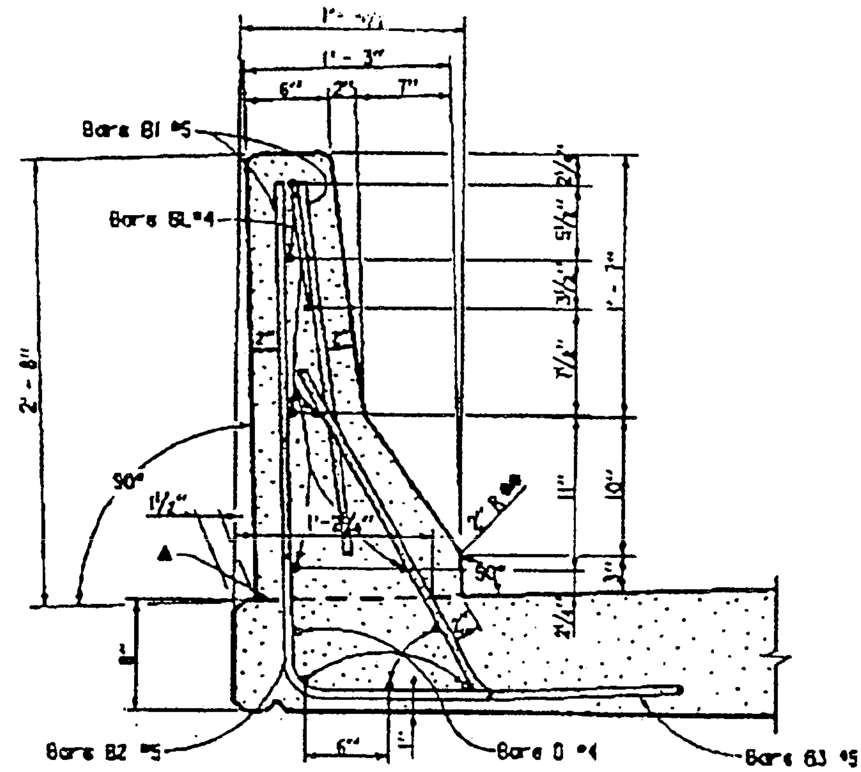


A. PRECAST BARRIER SECTION SHOWN IN EXODERMIC HANDBOOK



B. BOLTED CONCRETE BARRIER RAIL

Figure 11.6. Barrier Details



C. TYPICAL BARRIER SECTION-
ALDOT STANDARD

Figure 11.6 Barrier Details (contin.)

12. SUPPLEMENTAL GIRDERS AND CIP CONCRETE DECK REPLACEMENT OPTIONS

12.1. General

In addition to the deck replacement options described in Chapters 10 and 11, i.e., using NUDECK prestressed concrete panels and Exodermic panels respectively, use of CIP concrete with SIP metal forms is also viewed as being a viable deck replacement option. Additionally, adding a supplemental support girder between each of the existing girders and leaving the existing deck in place is viewed as being a viable rehabilitation option. Each of these two options is briefly discussed below.

12.2. Rapid Deck Replacement via CIP Concrete with SIP Metal Forms

Rapid replacement of existing bridge decks in a staged construction manner with concurrent traffic can be achieved by conventional practices for CIP with SIP metal forms by making a few modifications to speed up the construction process. The primary modifications are to use Varigrid rebar mats (top and bottom mats) that are precut to proper dimensions and set in place by crane, and to use rapid-setting concrete for the deck material. Rapid-setting concrete employing Type III cement similar to that used by the GADOT in their deck structural overlays would be workable and rather inexpensive. It is felt that this approach may be slightly more time consuming than other rapid deck replacement methods, but that it could be done within acceptable time constraints. Also, it is expected that this approach will be the least expensive of the deck replacement options.

The construction steps and sequence for this deck replacement procedure would be as follows:

1. Close work zone to traffic.
2. Make longitudinal saw-cuts (transverse cuts made the previous work stage), remove and dispose of the existing deck.
3. Sandblast/clean the tops of the exposed steel girders.
4. Install shear lugs (2 - 7/8" lugs @ 12" o.c.) on girder top flanges in regions of + moment.
5. Place SIP metal forms.
6. Place Varigrid (or equivalent) deck reinforcement mats and Varigrid guard rail reinforcement as appropriate.
7. Place 8" thick (8" - 10" rib/nonrib) rapid-set concrete deck.
8. Place rapid-set concrete guard rail via slip forming where appropriate.

9. Cure rapid-set concrete for 24 hours.
10. Make transverse saw-cuts (while deck is curing) in section to be removed the next work stage.
11. Reopen to traffic.

Work will proceed on this staged construction manner each weekend. The approximate costs for replacing the 149' - 8" "test section" on I-65 shown in Fig. 12.1 in this staged construction manner are given in Chapter 13.

12.3. Adding Supplemental Girders to Existing Superstructure

An inadequate bridge deck and superstructure can be strengthened and stiffened by placing additional girders between the existing girders as indicated in Fig. 12.2. The new girders can be installed from below the structure by crane or hoisting in place using the existing girders and then jacking into final position. The gap between the top flange of the beam and the underside of the deck slab can be filled by drilling holes through the deck slab and pressure grouting or by pressure grouting from below.

Existing girders on the I-65 and I59/120 interstate system appear to range in size from W 36 x 150 to W 36 x 245 depending on the bridge span with some having flange cover plates in the regions of high moment. The older/original portions of the deck have haunches at the girders on the underside as indicated in Fig. 12.2. For these portions, addition of W 36 x 150 to W 36 x 210 (these have 12" wide flanges) may be possible and this should be determined. It should be noted that in the "new" portions of the deck (where lanes were added around 1986), SIP metal forms were used and no haunch is available. In these portions, W 33 series girders will be the deepest that can be added.

Rehabilitation of a "test section" of the northbound I-65 bridge between Bents 12 - 15, as indicated in Fig. 12.1, by the addition of girders would require the following construction sequence:

1. Remove diaphragms from between the girders.
2. Sandblast strips of underside of deck where girders are being added.
3. Lift new girders into place, set on bearings, splice for continuity as required, and adjust to proper elevation.
4. Place new diaphragms at select locations in the negative moment regions of the girders.
5. Seal edges of girders and pressure grout gap between top of new girder and underside of deck.
6. Apply a methacrylate sealer/healer to the deck top surface.

It is assumed in the above sequence that the bents and bent caps have been determined to be adequate for the addition of the new girders. If this is not the case, the bent caps must be strengthened before adding the new girders. Also, it should be noted that traffic will need to be diverted from the lanes being worked on below for some of the rehabilitation work. However, these diversions and disruptions of traffic will be minimal.

Approximate costs associated with the construction tasks identified above for the 149' - 8" "test section" on I-65 shown in Fig. 12.1 are given in Chapter 13.

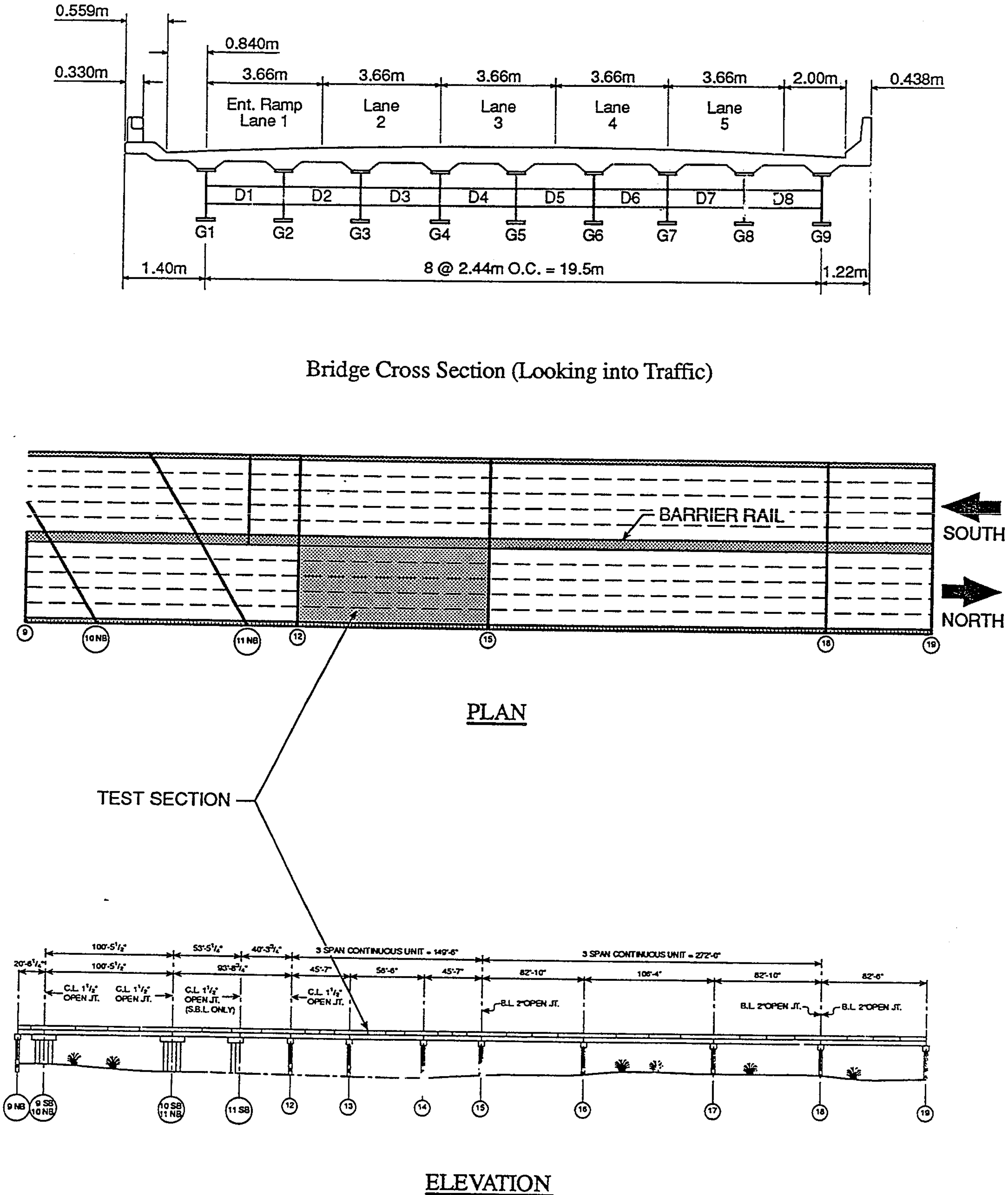


Fig. 12-1. I-65 Bridge Bents 9 and 19.

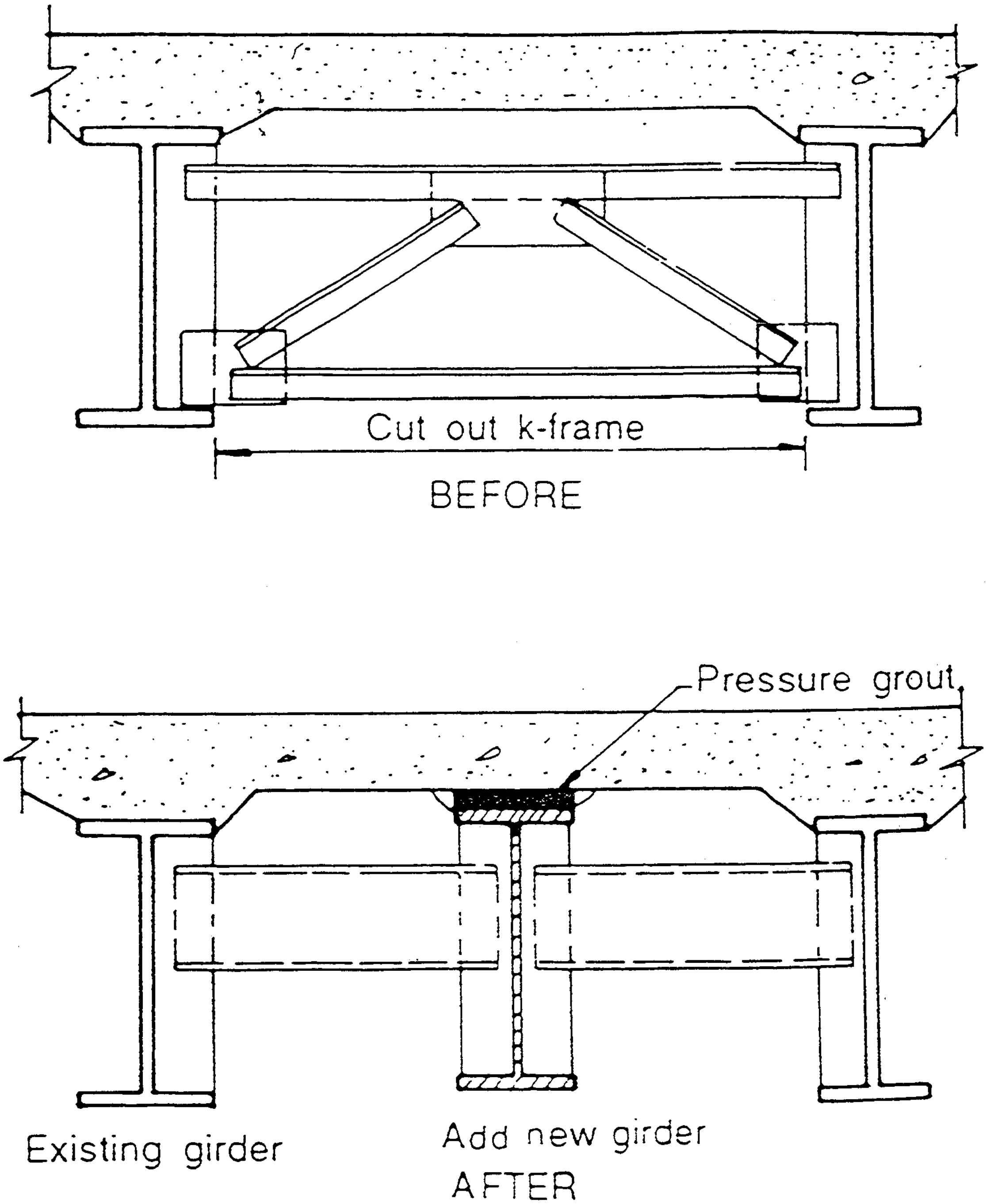


Fig. 12.2. Adding Additional Girder

13. COSTS OF BRIDGE DECK OVERLAY AND REPLACEMENT SYSTEMS

13.1 General

In the literature review section of the report, little attention was paid to the cost of the various deck rehabilitation and replacement (R/R) options. Recognizing that cost and relative cost of the various R/R options plays a significant role in the decision making process, this chapter was added. Cost of various R/R options as found in the literature are presented first for deck overlays in Section 13.2 , and then for deck replacements in Section 13.3. Additionally, to attain a more accurate estimate of the cost of the four deck replacement options that the authors feel are most appropriate for the Birmingham interstate system, comparison cost estimates for each option are presented in Section 13.4 for the same "test section" bridge on I-65.

13.2 Bridge Deck Overlay Costs

Michael Sprinkel (66) reported on the costs of polymer concrete overlays based on data collected in the SHRP Project C103 and afterwards from the Virginia DOT. A condensation of his comments regarding costs follows.

> The cost of an overlay is a function of the cost of materials, surface preparation, labor, equipment, overhead, and traffic control. Table 13.1 gives typical costs in 1991 in Virginia of the three most frequently used overlays constructed with mixture proportions shown in Table 13.2. The estimates are based on the following material costs:
>
> - Polyester: \$2.20/kg (\$1.00/lb)
> - Epoxy: \$3.30/kg (\$1.50/lb)
> - Methacrylate: \$7.72/kg (\$3.50/lb)
> - Aggregate: \$0.07/kg (\$0.03/lb)
>
> Multiple layer epoxy overlays are typically constructed in Virginia at a cost of \$30 to \$42/m^2 (\$25 to \$35/yd^2). The premix polyester overlay that is frequently used in California costs slightly more (but is twice as thick). The methacrylate slurry has been used less than polyester and epoxy because of the higher cost. A polyester/methacrylate hybrid

binder has been substituted for the methacrylate to provide a slurry overlay at a cost that is competitive with multiple layer epoxy and premix polyester overlays. A recent review of FY1994 and 1995 bid tabulations in Virginia for 27 projects specifying multiple layer epoxy overlays and 52 projects specifying latex or silica fume modified concrete overlays provided the results shown Table 13.3. The epoxy overlays cost 25% of the hydraulic cement concrete overlays based on total initial cost and 36% based on life-cycle cost assuming a 15 year life for epoxy and a 30 year life for hydraulic cement concrete. The differences in cost were attributed to the overlay materials, miscellaneous construction activities, and traffic control. The initial treatment cost of the epoxy overlay was 39% of that of the hydraulic cement concrete. Miscellaneous costs for the hydraulic cement concrete included building up the approach slabs and backwalls and replacing joints. Traffic control costs were higher for the hydraulic cement concrete overlays because of the requirements for concrete barricades, removing and installing permanent and temporary pavement markings and longer construction time. Epoxy overlays are installed in a short time using cones for delineation and without the need to replace pavement markings or joints and without raising approach slabs or backwalls.

The successful use of polymer overlays in California, Ohio, New York, Virginia, Washington, and other states demonstrate that polymer concrete bridge overlays are an economical alternative for extending the life of bridge decks.

Based on conversations with state DOT managers, construction contractors, and material manufacturers via telephone and during site visits to deck overlay projects in Georgia, Kentucky, and California, deck overlay cost data (1997 cost data) obtained by the authors is as shown in Table 13.4.

Table 13.1. Typical Costs of Polymer Overlays in 1991 in Virginia (66)

Overlay	Multiple-Layer Epoxy ($/m²)	Methacrylate Slurry ($/m²)	Premixed Polyester ($/m²)
Materials	12	30	18
Surface preparation	4	4	4
Labor, equipment, overhead	18	18	18
Traffic Control	1	1	1
Total	35	53	41

$\$1/m^2 = \$0.8/yd^2$

Table 13.2 - Typical Polymer Concrete Thickness and Application Rates (66)

Overlay	Multiple Layer Epoxy (kg/m²)	Methacrylate Slurry (kg/m²)	Premixed Polyester (kg/m²)
Thickness, mm	6.4	7.6	19.1
Prime Coat	--	0.41±0.14	0.41±.14
Layer 1 resin	1.1±0.14	2.7±0.27	5.29±0.41
Layer 1 aggregate	5.4±0.54	6.5±0.54	38.6±0.54
Layer 2 resin	2.2±0.14	--	--
Layer 2 aggregate	7.6±0.54	7.6±2.7	--
Seal Coat resin	--	0.68±0.14	--
Approx. resin content, %	25	24	13

Table 13.3. Cost of Bridge Deck Protective Treatments (66)

Treatment	Epoxy Overlay $/m² ($/yd²)	Concrete Overlay $/m² ($/yd²)
Treatment	28 (24)	73 (61)
Misc.	0 (0)	27 (23)
Traffic	10 (8)	55 (46)
Total	38 (32)	155 (130)
Life Cycle	56 (47)	155 (133)
Life (years)	15	30

Table 13.4. Deck Overlay Cost Data

SOURCE	OVERLAY TYPE/THICKNESS	UNIT COST					COMMENTS
		DECK PREPARATION		MATERIAL ($\$/YD^3$)[1]	PLACING & FINISHING ($\$/YD^2$)[1]	TOTAL ($\$/YD^2$)	
		SAND/SHOT BLASTING ($\$/FT^2$)	HYDRO DEMOLITION ($\$/FT^2$)				
Caltran	3/4" PPC Bonded Overlay	1.00	65.00	810-945	--	72.42	$72.42/yd² is the average cost of Caltran's 3/4" PPC overlays for the 1994 year adjusted to 1997 dollar value.
GA DOT	4 1/2" Type III Cement Structural Overlay			≈100	--	298	Based on total overlay project cost. Hydrodemolition of old deck required.
KY DOT	1 1/4" RSLMC Bonded Overlay			400	$370/yd²	43.90 140	Based on $770/yd³ for new material, placement and finishing. Based on total overlay project cost.
Dow Chemical	1 1/4" RSLMC Bonded Overlay					34.72	Based on $1000/yd³ for new material, placing, finishing, and assuming t=1 1/4" throughout the deck.
Thermal-Chem	1/4" - 3/8" EPC Bonded Overlay	1.00				60-70	Assuming only shot blasting of surface for preparation
Caltran	Thin Methacrylate Bonded Overlay	1.00		$13.50/yd²		22.50	Assuming only shot blasting of surface for preparation
WA DOT	2" Asphalt with Membrane Overlay					17-27	$7-$12/yd² for fabric membrane in place. $10-15/yd² for asphalt concrete in place.

[1]Caltran pays by the yd³ for overlay material, and by the yd² for placement and finishing overlays.

13.3. Bridge Deck Replacement Costs.

The most attractive deck replacement systems for the Birmingham bridges to the authors are

- precast and prestressed concrete system
- exodermic deck system
- CIP rapid-set concrete system

because of their friendliness to rapid and staged construction under concurrent traffic conditions. The least expensive (if one excludes highway user costs) replacement system is probably a CIP concrete deck. Estimated costs of all three systems are provided below for comparison purposes.

13.3.1. Precast/Prestressed Concrete Systems. University of Nebraska researchers (68), in their NCHRP Project 12-41, performed cost analysis and comparisons on the following four bridge deck replacement systems.

1. Full Depth 9" (22.86 cm) Cast-in-Place Bridge Deck with conventional epoxy coated reinforcement
2. Precast Prestressed Panel with 6" (17.15 cm) Cast-in-Place Concrete Deck utilizing epoxy coated welded wire fabric reinforcement
3. Precast Prestressed Continuous Panel with 4.5 (11.43 cm) Cast-in-Place Concrete Deck utilizing epoxy coated welded wire fabric reinforcement
4. Full Depth Precast Prestressed Deck System with longitudinal post tensioning

Each system was evaluated for schedule, construction cost, field productivity, single vs. multiple shift construction, and the bridge out-of-service duration/road use cost. The evaluation for each system was based on the removal and replacement of an existing 250' - 0" x 44' - 0" (76.2m x 13.4lm) bridge deck and traffic barrier/railing with the bridge closed to traffic (not a staged construction/concurrent traffic construction condition). The total area of the bridge deck was 11,000 sq. ft. (1,022m^2). The construction estimates (wages and material costs) baseline was Kansas City, Missouri, June 1996 (ENR INDEX=5597).

All four systems were evaluated for the following three work schedules and road user costs:

8 hours/day single shift, 5 days/week (40 hours work/week)

16 hours/day double shift, 6 days/week (96 hours work/week)

24 hours/day triple shift, 7 days/week (168 hours work/week)

\$0/day (road user cost)

\$20,000/day (road user cost)

\$40,000/day (road user cost)

The results of their cost analysis are presented in Table 13.5

Table 13.5. Cost Analysis

NCHRP RAPID DECK REPLACEMENT

AWARD BASIS (A + B) = BASE BID (A) + USER COST COMPARISON (B)

(CONTRACT VALUE IS BASE BID)

				ROAD USER COST = $0/CAL DAY			ROAD USER COST = $20,000/CAL DAY			ROAD USER COST = $40,000/CAL DAY		
			A		B	A + B		B	A + B		B	A + B
ITEM #	DESCRIPTION	WORK HOURS	Base Bid	Out of Serv Dur	Road User Cost Days x 0	Award Basis	Out of Serv Dur	Road User Cost Days x 20,000	Award Basis	Out of Serv Dur	Road User Cost Days x 40,000	Award Basis
1	CONVENTIONAL 9" CAST-IN-PLACE DECK	5 DAYS/WK x 8 HR /DAY	$337,281	52	$0	$337,281	52	$1,040,000	$1,377,281	52	$2,080,000	$2,417,281
10	3" P/C P/S PANEL w/ 6" CAST-IN-PLACE DECK	5 DAYS/WK x 8 HR /DAY	$315,351	47	$0	$315,351	47	$940,000	$1,255,351	47	$1,880,000	$2,195,351
20	4.5 " P/C P/S FULL WIDTH PANEL w/ 4.5 " CIP DECK	5 DAYS/WK x 8 HR /DAY	$345,583	40	$0	$345,583	40	$800,000	$1,145,583	40	$1,600,000	$1,945,583
30	9" P/C WAFFLE PANEL - LONG. POST TENSIONED	5 DAYS/WK x 8 HR /DAY	$359,322	37	$0	$359,322	37	$740,000	$1,099,322	37	$1,480,000	$1,839,322

				ROAD USER COST = $0/CAL DAY			ROAD USER COST = $20,000/CAL DAY			ROAD USER COST = $40,000/CAL DAY		
			A		B	A + B		B	A + B		B	A + B
ITEM #	DESCRIPTION	WORK HOURS	Base Bid	Out of Serv Dur	Road User Cost Days x 0	Award Basis	Out of Serv Dur	Road User Cost Days x 20,000	Award Basis	Out of Serv Dur	Road User Cost Days x 40,000	Award Basis
2	CONVENTIONAL 9" CAST-IN-PLACE DECK	6 DAYS/WK x 16 HR /DAY	$409,196	30	$0	$409,196	30	$600,000	$1,009,196	30	$1,200,000	$1,609,196
11	3" P/C P/S PANEL w/ 6" CAST-IN-PLACE DECK	6 DAYS/WK x 16 HR /DAY	$377,600	30	$0	$377,600	30	$600,000	$977,600	30	$1,200,000	$1,577,600
21	4.5 " P/C P/S FULL WIDTH PANEL w/ 4.5 " CIP DECK	6 DAYS/WK x 16 HR /DAY	$403,982	26	$0	$403,982	26	$520,000	$923,982	26	$1,040,000	$1,443,982
31	9" P/C WAFFLE PANEL - LONG. POST TENSIONED	6 DAYS/WK x 16 HR /DAY	$409,876	20	$0	$409,876	20	$400,000	$809,876	20	$800,000	$1,209,876

				ROAD USER COST = $0/CAL DAY			ROAD USER COST = $20,000/CAL DAY			ROAD USER COST = $40,000/CAL DAY		
			A		B	A + B		B	A + B		B	A + B
ITEM #	DESCRIPTION	WORK HOURS	Base Bid	Out of Serv Dur	Road User Cost Days x 0	Award Basis	Out of Serv Dur	Road User Cost Days x 20,000	Award Basis	Out of Serv Dur	Road User Cost Days x 40,000	Award Basis
3	CONVENTIONAL 9" CAST-IN-PLACE DECK	7 DAYS/WK x 24 HR /DAY	$521,718	24	$0	$521,718	24	$480,000	$1,001,718	24	$960,000	$1,481,718
12	3" P/C P/S PANEL w/ 6" CAST-IN-PLACE DECK	7 DAYS/WK x 24 HR /DAY	$473,505	22	$0	$473,505	22	$440,000	$913,505	22	$880,000	$1,353,505
22	4.5 " P/C P/S FULL WIDTH PANEL w/ 4.5 " CIP DECK	7 DAYS/WK x 24 HR /DAY	$497,398	20	$0	$497,398	20	$400,000	$897,398	20	$800,000	$1,297,398
32	9" P/C WAFFLE PANEL - LONG. POST TENSIONED	7 DAYS/WK x 24 HR /DAY	$497,032	13	$0	$497,032	13	$260,000	$757,032	13	$520,000	$1,017,032

NOTES:

	WORK	PAY
1. 5 DAYS/WK x 8 HR/DAY	40 HR	40 HR
2. 6 DAYS/WK x 16 HR/DAY	96 HR	104 HR
3. 7 DAYS/WK x 24 HR/DAY	168 HR	192 HR

4. NO INCIDENTAL OVERTIME.
5. NO MAINTENANCE OF TRAFFIC
6. NO BEARING REPAIR OR REPLACEMENT.
7. NO APPROACH SLAB REMOVAL OR REPLACEMENT.
8. NO GRADING, DRAINAGE OR ROADWAY PAVING.
9. NO STRUCTURAL STEEL REPAIRS.
10. OUT OF SERV DUR = BRIDGE OUT OF SERVICE DURATION.
11. ALL ESTIMATES BASED ON 250'-0" x 44'-0" (76.20 m x 13.41 m) = 11,000 sf (1,022 sm) BRIDGE.
12. ALL DAYS ARE CALENDAR DAYS.
13. A + B IS FOR AWARD BASIS ONLY; CONTRACT VALUE IS BASE BID
14. THE CONTRACT WILL ALSO HAVE AN INCENTIVE/DISINCENTIVE EQUAL TO THE ROAD USE COST WHICH IS EXCLUDED FROM THE ABOVE ANALYSIS.

13.3.2. Exodermic Deck System. Cost information and data on Exodermic decks were obtained from the Exodermic Bridge Deck, Inc., and the NYSDOT which has to date been the largest user of exodermic decks. This information and data are given below.

Exodermic Bridge Deck, Inc. Cost Information. The in-place cost of an Exodermic deck is dependent on many factors:

- Whether the concrete component is to be pre-cast or cast-in-place.
- Deck design including grid type and configuration, rebar size and spacing, and concrete thickness
- Concrete type--lightweight concrete, and the addition of microsilica or other materials add cost.
- Staging of the work--is all work to be done at night, with the full bridge open to traffic in the morning? Are a number of stages required in order to keep traffic moving in a reduced number of lanes while the bridge is re-decked?
- What details are specified with the grid. The deck price of an Exodermic deck often includes scuppers, joints or joint supports, curb channels, edge forms, and railing or barrier anchorages--items that would be bid separately for a standard reinforced concrete deck.
- Panel shape--skews, curves, and cut-outs (such as for truss members) add cost.
- Construction environment--limited crane and equipment access can add cost. For example, a low clearance thru truss bridge may require the use of small crane with a limited reach--the crane will have to be moved more often, limiting the amount of work that can be done in a short work window.
- Maintenance and protection of traffic require additional effort and cost. For example, emergency deck repairs using Exodermic panels on the Tappan Zee bridge in 1994, took an hour to set up cones and an hour to pick them up, cutting 2 hours from the time available to work each night.
- Area of the country; union or non-union.
- Whether contractors are busy or not.

Some recent Exodermic deck unit costs are shown in Table 13.6, and reflect a rather wide range of unit costs. This is due to the considerable variations in working conditions. For example, a recent pre-cast project completed was the re-decking of a 47,500 s.f. bridge over the Hudson River in Albany, NY. Work was done at night, with the bridge fully open to traffic by 6 a.m. each morning. The in-place cost of the Exodermic deck including galvanized grid, pre-cast concrete, haunch forming materials, rapid setting grout, and erection was $33/s.f. This cost also reflects some unusual aspects of the job. Lightweight concrete with 9% microsilica was specified by NYSDOT - a more expensive mix than with standard concrete. The limited overhead clearance due to the shallow thru-truss spans required an

Table 13.6 - Recent Exodermic Deck Cost History[1,2]

Project	Date	Deck Size (s.f.)	Cast-in-Place (C) or Pre-Cast (P)	Low bid-deck ($/s.f.)	Deck price bid winning bidder ($/s.f)	Other bids ($/s.f.)	Comments
Main Street Bascule Bridge Green Bay, WI	Oct-96	11,873	C	$23.23	$23.23	$32.98, $51.10	New, double leaf bascule. Exodermic deck spans over 13' floorbeam to floorbeam, without stringers. Normal wt. concrete.
Milton-Madison bridge over the Ohio River, Kentucky-Indiana	Apr-96	72,700	P	$37.00	$37.00	$39.75, $40, $46, $49.85, $52	Bridge is largely narrow thru-truss spans, complicating demolition and deck placement. Small panels (typically 11' long by 7'10" wide) and close stringer spacing (3'8") added to cost. Curb precast with deck panels. Scuppers included in deck cost. 3200' of #5 couplers at centerline.
Troy-Mendands (Route 378) Bridge over the Hudson River, Albany, NY	Mar-95	47,500	P	$33.00	$33.00		Exodermic option was bid against a newly proposed proprietary design. Concrete component of the exodermic deck is lightweight concrete with 9% microsilica. 18' thru truss clearance limited crane size and use.
Ithaca, NY--Two new bridges	Feb-95	12,500	C	$32.00	$32.00		Exodermic design allowed use of shallower beams, maximizing underclearance of new structures. (Item bid price by other bidders being checked.)
Route 9W Popolopen Creek, NY	Mar-91	32,550	C	$33.31	N.A.	N.A.	Price is contractor's price for exodermic, including grid, rebar, concrete, and installation. Value engineering change from original design which called for filled grid.
St. Johnsville, NY	May-92	18,402	C	$31.75	$34.00	$34	Contract had incentive/disincentive clause. Contractor said that rapid deck erection "saved the job" for them--they collected the full $100,000 incentive.
Western Boulevard Viaduct, Chicago	Sep-92	8,704	P	$36.07	$40.21	$42.91, $48.52, $50.61, $56.63	FHWA demonstrated project. Panel layout not most cost efficient.
Pitman Creek, Pulaski County, NY	Dec-93	24,582	P	$33.00	$42.00	$39, $55	Work to be done only at night, in one lane at a time, maintaining traffic in the adjacent lane. Both lanes to be fully open to traffic during the day.
Lake Shore Road, Putnam County, NY	Feb-93	2,170	C	$22.00	$23.50	$30, $31, $34, $33.50, $34, $37	Project was bid but not awarded, due to budgetary problems.

[1]Deck prices are from the Exodermic Bridge Deck, Inc.
[2]Deck prices represent in-place cost including grid, rebar, and concrete, and attachment details for scuppers, railings, joints, etc. are generally also included.

almost flat boom lift, limiting the number of panels that could be placed without moving the crane. And, of course, replacing a small portion of the deck each night is more expensive than doing a large area of a bridge at one time. The redecking of this bridge was accomplished in three months. After a few week "learning curve", the crew was able to place 924 s.f. per night (6 panels).

It is reasonably straightforward to estimate what bid prices *should* be. On a large project, galvanized steel grid delivered to the bridge or precasting site generally costs the contractor between $12 and $19 depending on configuration. Little cost is incurred in placing the panels. Additional cost for shooting headed studs, placing rebar, and pouring concrete (typically 4" to 4 1/2") should be comparable to costs for standard reinforced concrete decks, adjusted for the fact that an Exodermic deck replaces stay-in-place forms or other formwork, uses half as much rebar, and half as much concrete. For a large pre-cast project, a price of $28/s.f., or better, should be obtainable. Where conditions permit the concrete component of the Exodermic deck to be cast-in-place, prices of $23 to $25 should be possible. A revised Exodermic design, now in the planning stage, promises to provide a significant reduction in the cost of the grid to the contractor, without any sacrifice in performance or structural integrity.

Because Exodermic is often new to contractors, the better informed contractors are when bidding, the less "cushion" they feel they need to build into their prices. For a number of projects, this has been aided by the inclusions in the P, S, & E package of information on Exodermic geared to contractors.

NYSDOT Cost Information. The NYSDOT has used exodermic panels in several instances due to its relatively light weight and, in some instances, its ability to be prefabricated for quick deck replacements. Cast in place installations have performed satisfactory to date. These have been used principally to reduce dead load when rehabilitating older structures although some designers are specifying the exodermic in lieu of cast-in-place concrete. There is usually a specific basis for this. There is an installation savings in form work with the decking.

Two major installations utilized prefabricated panel. Both sites required night work with the decks reopened to traffic the next morning. Studs were attached to stringers of floor beams but not for composite action, for these installations. Cast in leveling bolts were used and open areas were filled in with a commercial cement rapid setting concrete (CTS products). This product worked well but quantities were limited to a maximum 8" wide by about 12" deep in fills over the supporting beams.

It is difficult to compare costs as exodermic installations to date have been with some form of unique needs that required premium prices and lack of familiarity on contractor's part. Based on our bid price records, the following were the project prices:

- Standard Cast-in-Place Deck-(Structural Slab Concrete, HP, with re-bar included): \$21.32/SF based on 7/96 to 6/97 avg. Bid prices, statewide
- Composite Exodermic cast-in-place-(3 installations, 1 bid in '92, 2 bid in '95): approximately \$32/SF
- Non-Composite, overnight construction, prefabricated panels: \$33/SF actual ('95)

Tappan Zee Deck Replacement. The Tappan Zee deck replacement project was bid in December 1997 with furnishing of deck panels, installation of panels, and demolition of the existing deck being bid as separate items. The Arben Corporation of New York was the low bidding contractor with itemized bides of:

Exodermic panels	-	\$26.10/s.f.
Installation of panels	-	\$9.38/s.f.
Demolition of deck	-	\$10.31/s.f.
Diamond grinding of deck	-	\$1.76/s.f.
Placement of thin Flexogrid overlay	-	\$6.58/s.f.
Total:	-	\$54.13/s.f.

The exodermic panel price includes complete precast delivery to the job site. It is an average price for the entire deck area, and includes complex joint panels at the ends of each deck truss span. The prices reflect a short work window (8:00 p.m. - 6:00 a.m. nightly), New York City union labor, and a \$1,300/minute penalty for not being complete off the bridge by 6 a.m.

It should be noted that 4 years prior tot he December 1997 bidding, test sections of the Tappan Zee bridge were rehabilitated via deck replacements using Exodermic panels, prestress concrete panels, and half-filled steel grid panels. The NYTA indicated that all three of these replacement methods were economically competitive from an initial cost standpoint.

13.3.3. CIP Rapid-Set Reinforced Concrete System. A CIP deck using a high performance concrete mixture (rapid development of strength) and prefabricated weld rebar deck mats is viewed as being a viable option for rapid deck replacement. Mr. Fred Conway, ALDOT Bridge Engineer, estimated the cost of CIP reinforced decks for different construction conditions to be as indicated in Table 13.7. Table 13.8 shows the approximate cost for replacing a "test section" of deck on a I-65 bridge in Birmingham with a rapid-set CIP concrete deck on SIP metal forms.

13.4. Supplemental Girder Rehabilitation Costs

Cost estimates for the supplemental girder addition strategy described in Ch. 12 were determined with the help of Carolina Steel Fabrication, Inc., McInnis Corporation, and the ALDOT. These estimates for the 149'-8" x 72'-5" "test section" described earlier are given in Table 13.9.

Table 13.7. Estimated In-Place Cost of New CIP Bridge Decks[1]

Construction Condition	Unit Costs Using ALDOT Standard Deck Concrete		
		For 8" Deck Thickness	
	\$/yd^3	\$/yd^2	\$/ft^2
New Bridge/Deck (no traffic and regular work schedule)	250-350 ↓ 300	\$66.70	\$7.40
Bridge/Deck Widening (concurrent traffic conditions)	400-500 ↓ 450	\$100	\$11.10
Bridge/Deck Widening (concurrent traffic conditions) Using High Performance Concrete[2]	450-550 ↓ 500	\$111.10	\$12.40

[1]Does not include any old deck removal costs
[2]Estimated by Ramey

Table 13.8. Approximate Cost of Deck Replacement Via SIP Metal Forms with Rapid-Set CIP Concrete on I65 Test Section

Work/Material Item	Quantity	Unit Cost	Test Section Cost
Saw-cut, remove, dispose of existing deck	10,870 ft^2	\$9.50/ft^2	\$103,265
Sandblast/clean tops of exposed steel girders	1197 ft^2	\$15/ft^2	\$17,955
Install shear lugs (2 - 7/8" at 12" o.c.) on girder top flanges in regions of + Moment	1836 Shear Lugs	\$1/lug	\$1,836
Purchase and placement of metal SIP forms	10,870 ft^2	\$2.00/ft^2	\$21,740
Purchase and placement of Varigrid (or equivalent) deck reinforcement mats	10,870 ft^2 @ 6.87 lb/ft^2	\$0.50/lb or \$3.44/ft^2	\$37,393
Purchase and placement of Varigrid guard rail reinforcement	300 lin. ft. @ 19.57 lb/ft	\$0.50/lb or \$9.78/ft^2	\$2,934
Purchase and placement of 8" thick rapid-set concrete deck	10,870 ft^2 or 269 yd^3	\$350/yd^3	\$94,150
Purchase and placement of rapid-set concrete guard rail via slip forming (≈ 9" x 2' - 8")	300 lin. ft. or 23 yd^3	\$350/yd^3	\$8,050
		Total:	\$287,323

Table 13.9 Approximate Cost of Deck Rehabilitation Via the Adding of New Girders Between Existing Girders on I65 Test Section

Work/Material Item	Quantity	Unit Cost	Test Section Cost
Remove existing diaphragms	56	\$250 ea.	\$14,000
Sandblast bottom of deck at girder locations	1' - 4" x 1197' 1595 ft^2	\$15/ft^2	\$23,925
New girders (W 36 x 150)	1197 lin. ft.	\$0.78/lb	\$140,049
New bearings	32	\$1,200 ea.	\$38,400
Install new bearings on bent caps	32	\$150 ea.	\$4,800
Lift and set girders in place	24	\$4,500 ea.	\$108,000
New Diaphragms (C15 x 33.9 x 3' - 9")	64	\$0.78/lb	\$6,346
Place new diaphragms at select locations	64	\$100 ea.	\$9,576 \$130,000
Form edges of new girders and inject epoxy or nonshrink grout	2,394 lin. ft. edge ∢ 108 ft^3 of epoxy or nonshrink grout	\$4/ft \$160/gal	\$9,576 \$130,000
Apply methacrylate sealer/healer to deck top	10,870 ft^2	\$1,25/ft^2	\$13,588
		Total:	\$495,084

13.5. Cost Comparisons for Four Test Sections

As will be seen in the final two chapters of this report, the authors feel that the most viable deck rehabilitation strategies for the Birmingham interstate bridge decks are as follows:

- Deck replacement with NUDECK prestressed concrete SIP continuous panels with CIP rapid-set concrete topping.
- Deck replacement with Exodermic bridge deck panels with CIP rapid-set concrete topping.
- Deck replacement with SIP metal forms and CIP rapid-set concrete.
- Deck rehabilitation via the addition of a supplemental girder between each existing girder (from the underside) and application of a low viscosity methacrylate sealer/healer to the deck top surface.

Cost information from the literature, from manufacturers, and from similar projects in other states has been given in previous sections for these rehabilitation strategies. However, in each case the extent of the work and other project variables were different, as were the locations of the work, etc., and this makes a relative cost comparison between the strategies difficult. Thus, in order to attain a better cost comparison, the authors, with the help of manufacturers, steel fabricators, bridge contractors and the ALDOT, estimated the cost of rehabilitating a 149'-8" x 72'-5" "test section" on the 1,667' long I-65 bridge over the railroad tracks and 1st Avenue in Birmingham (see Fig 10.1). Traffic control and temporary and permanent lane striping costs are not included in estimates and are viewed as common costs to all of the deck replacement strategies (except the supplemental girder deck rehabilitation).

The Exodermic panel price bid for the ongoing Tappan Zee bridge deck replacement was \$26.10/s.f. However, this price was for full depth panels, whereas the Exodermic panels recommended in this report call for steel Exodermic grids with CIP rapid-set concrete. For bare steel grids, costs usually run between \$12 and \$19 per s.f. The panels needed for the Birmingham "test section" will need to be relatively stout due to the large cantilever overhang on the I-65 bridge. Based on discussions with the Exodermic Bridge Deck Institute, estimated unit costs for the Birmingham "test section" Exodermic panels was \$18/s.f delivered to the site. Other estimated costs for the Exodermic replacement strategy for the "test section" are as shown in Table 13.10.

Sherman Prestressed Concrete quoted a jobsite delivered (but not put in place) price of the precast prestressed SIP panels shown in Figs. 10.3 - 10.5 to be \$8.25/s.f. for 10,870 s.f. of panels with leveling devices included. Ivy Steel & Wire of Jacksonville, FL, manufacturer of Varigrid reinforcement, indicated that their welded grids are higher in cost than normal rebar, but the savings in manpower in installation offsets their higher product cost and results in approximately the same cost as a conventional rebar mat in place. The attractive feature of the Varigrid mat is the shorter installation time and thus shorter down time before reopening the deck to traffic. Other estimated costs for the NUDECK prestress concrete replacement strategy for the "test section" are shown in Table 13.11.

Estimated costs for the CIP rapid-set concrete on SIP metal forms and the addition of supplemental girders for the "test section" were given previously in Tables 13.8 and 13.9 respectively. Total cost estimates (except for traffic control and lane paint striping) for the four rehabilitation strategies for the I-65 "test section" were extracted from Tables 13.8 - 13.11, and are shown in Table 13.12 for convenience in making relative cost comparisons.

Table 13.10. Approximate Cost of Deck Replacement Via Exodermic Panels with Rapid-Set CIP Concrete on I65 Test Section

Work/Material Item	Quantity	Unit Cost	Test Section Cost
Saw-cut, remove, dispose of existing deck	10,870 ft^2	\$9.50/ft^2	\$103,265
Sandblast/clean tops of exposed steel girders	1197 ft^2	\$15/ft^2	\$17,955
Install shear lugs (2 - 7/8" at 12" o.c.) on girder top flanges in regions of + Moment	1836 Shear Lugs	\$1/Lug	\$1,836
Exodermic Panels	10,870 ft^2	\$18/ft^2	\$195,660
Placement of Exodermic Panels	10,870 ft^2	\$3.75/ft^2	\$40,763
Purchase and placement of Varigrid (or equivalent) deck reinforcement mat	10,870 ft^2 @ 3.44 lb/ft^2	\$0.50/lb or \$1.72/ft^2	\$18,696
Purchase and placement of Varigrid guard rail reinforcement	300 lin. ft. @ 19.57 lb/ft	\$0.50/lb or \$9.78/ft	\$2,934
Purchase and placement of 4" thick rapid-set concrete deck	10,870 ft^2 or 135 yd^3	\$350/yd^3	\$47,250
Purchase and placement of rapid-set concrete guard rail via slip forming (≈ 9" x 2' - 8")	300 lin. ft. or 23 yd^3	\$350/yd^3	\$8,050
		Total:	\$ 436,409

Table 13.11. Approximate Cost of Deck Replacement Via NUDECK Panels with Rapid-Set CIP Concrete on I65 Test Section

Work/Material Item	Quantity	Unit Cost	Test Section Cost
Saw-cut, remove, dispose of existing deck	10,870 ft^2	$9.50/$ft^2$	$103,265
Sandblast/clean tops of exposed steel girders	1197 ft^2	$15/$ft^2$	$17,955
Install shear lugs (2 - 7/8" at 12" o.c.) on girder top flanges in regions of + Moment	1836 Shear Lugs	$1/Lug	$1,836
NUDECK Panels	10,870 ft^2	$8.25/$ft^2$	$89,678
Placement of NUDECK Panels	10,870 ft^2	$4.00/$ft^2$	$43,480
Purchase and placement of Varigrid (or equivalent) deck reinforcement mat	10,870 ft^2 @ 3.44 lb/ft^2	$0.50/lb or $1.72/$ft^2$	$18,696
Purchase and placement of Varigrid guard rail reinforcement	300 lin. ft. @ 19.57 lb/ft	$0.50/lb or $9.78/ft	$2,934
Purchase and placement of 4" thick rapid-set concrete deck	10,870 ft^2 or 135 yd^3	$350/$yd^3$	$47,270
Purchase and placement of rapid-set concrete guard rail via slip forming (≈ 9" x 2' - 8")	300 lin. ft. or 23 yd^3	$350/$yd^3$	$8,050
		Total:	$333,164

Table 13.12. Approximate Cost for Four Rehabilitation Strategies for I65 Test Section[1]

Rehabilitation Strategy	Total Unit Cost ($/s.f.)	Total Cost[1]
Deck Replacement Using Exodermic Panels with CIP Concrete Toppings	40.15	436,409
Deck Replacement Using NUDECK Panels with CIP Concrete Topping	30.65	333,164
Deck Replacement Using SIP Metal Forms with CIP Rapid-Set Concrete	26.43	287,323
Deck Rehabilitation Via Adding Supplemental Girders and Placing Methacrylate Sealer/Healer	45.55	495,084

[1] I65 Test Section is 149'-8" x 72'-5" or 10,870 s.f.

14. Rehabilitation Options for Birmingham Interstate Bridges

14. General

As indicated earlier in this report, based on discussions with ALDOT traffic, bridge and maintenance engineers, and bridge inspectors, it appear that ALDOT's primary concerns about the Birmingham I-65 and I59/20 bridges and bridge decks are as follows:

1. Inadequate traffic lanes and traffic capacity (on I-65 and on I59/20 from the I59/20 juncture to the I-65 interchange in particular).

2. Significant levels of live load deflections and out-of-plane movement of the deck superstructure system. In turn, these are probably the major contributor to the distresses indicated in (3) - (5) below.

3. Significant level and rate of increase of deck cracking and deterioration which is requiring ever increasing maintenance attendance in the form of surface spalls and potholes (which generally require full-depth patches), is probably reducing the bending stiffness in both the longitudinal and transverse directions and leading to greater deflections and cracking, and will eventually lead to deck punching shear failures.

4. Extensive state of fine cracking on the deck undersides with a concern for future underside spalling problems which would create a safety hazard, and additional maintenance requirements.

5. Past history of fatigue problems with diaphragms, diaphragm-to-girder connections, and support girders (at locations of transverse diaphragms) and a concern that the girders may be approaching their fatigue limit/life and need to be replaced.

The number of rehabilitation options for the Birmingham bridges is obviously very large and before one or more options is selected, due consideration should be given to

- the current and near-future traffic demands and capacities of the bridges and highway systems (functional adequacy of the bridges/highway systems)
- the current state of soundness/deterioration of the existing bridges
- cost competitive rehabilitation alternatives which can be implemented in rapid and staged construction manner under concurrent traffic conditions.

Each of these major considerations are examined and discussed below.

14.2 Options to Address Heavy Traffic Demands

Mr. George Ray, Chief of ALDOT's Transportation Planning Bureau, has stated that the interstate system (I-65 and I-59/20) through Birmingham is about to or over capacity at the present time, and additional lanes are needed now. He indicated that even if the existing bridge decks on the interstate systems were in mint condition, a lane addition rehabilitation would be needed in the near future because of heavy traffic conditions.

If widening of interstate arteries through Birmingham to accommodate high traffic volumes is anticipated or planned in the foreseeable future (0-20 years), then widening the system (and bridges) to increase traffic capacity before rehabilitating the existing bridge decks would be less disruption to traffic and obviously the way to go. An additional lane in each direction would allow more traffic lanes to remain open when later rehabilitation of the existing deck begins.

The addition of an additional lane in each direction on I-65 on the outside of the existing lanes appears to be very feasible. However, addition of an additional lane in each direction on the I-59/20 system through the Central Business District (CBD) of Birmingham will be a challenge. It appears that it can be done, but will require closing up the center ramps and accesses in some locations as well as adding a lane to the outside in some locations. Schematic sketches of how lane additions might be accomplished at select locations on the I-65 and I-59/20 systems are shown in Figs. 14.1 and 14.2.

As an alternate to adding laneage to the I-65 and I-59/20 systems, alternate routes through and/or around Birmingham could possibly be identified and improved or developed as indicated in Fig. 14.3. Also, as an alternate to widening the I-59/20 system in the tight quarters of the CBD, a new section (approximately 1-1½ miles) of I-59/20 or I-59S/20W could be constructed which routes traffic around the north side of the Birmingham Convention Center as indicated in Fig. 14.4. Once this new section is in place, the old section of I-59/20 could be rehabilitated if it is to be used as I-59N/20E or removed to allow other usage of the land if the new section is to be I59/20. The layouts shown in Figs 14.3 and 14.4 are qualitative only and do not pretend to identify the exact locations of such additions.

Thus, near future traffic volume demands in and through Birmingham need to be closely examined, and decisions made on how to accommodate these demands, before deciding on the best means of rehabilitating the existing I-65 and I-59/20 bridge decks in Birmingham. It is the authors' understanding that the alternatives above have been discussed and explored to some extent by the ALDOT. However, because of the significant impact that these decisions have on the question of rehabilitation of the existing bridge decks, these explorations should now be carried through to the final stage and appropriate decisions made.

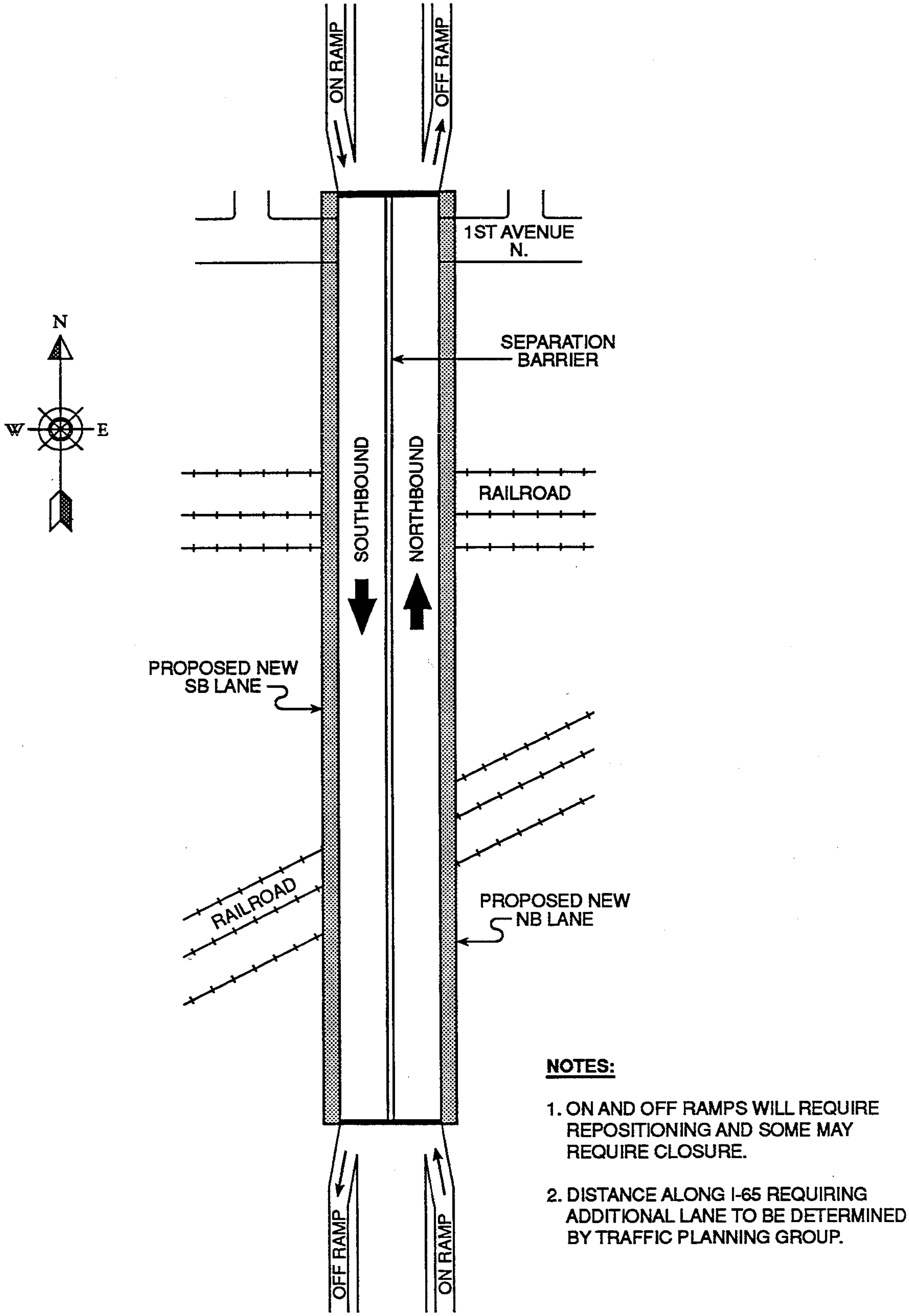


Fig. 14.1. Schematic of I-65 Lane Additions at Long Bridge Over Railroad

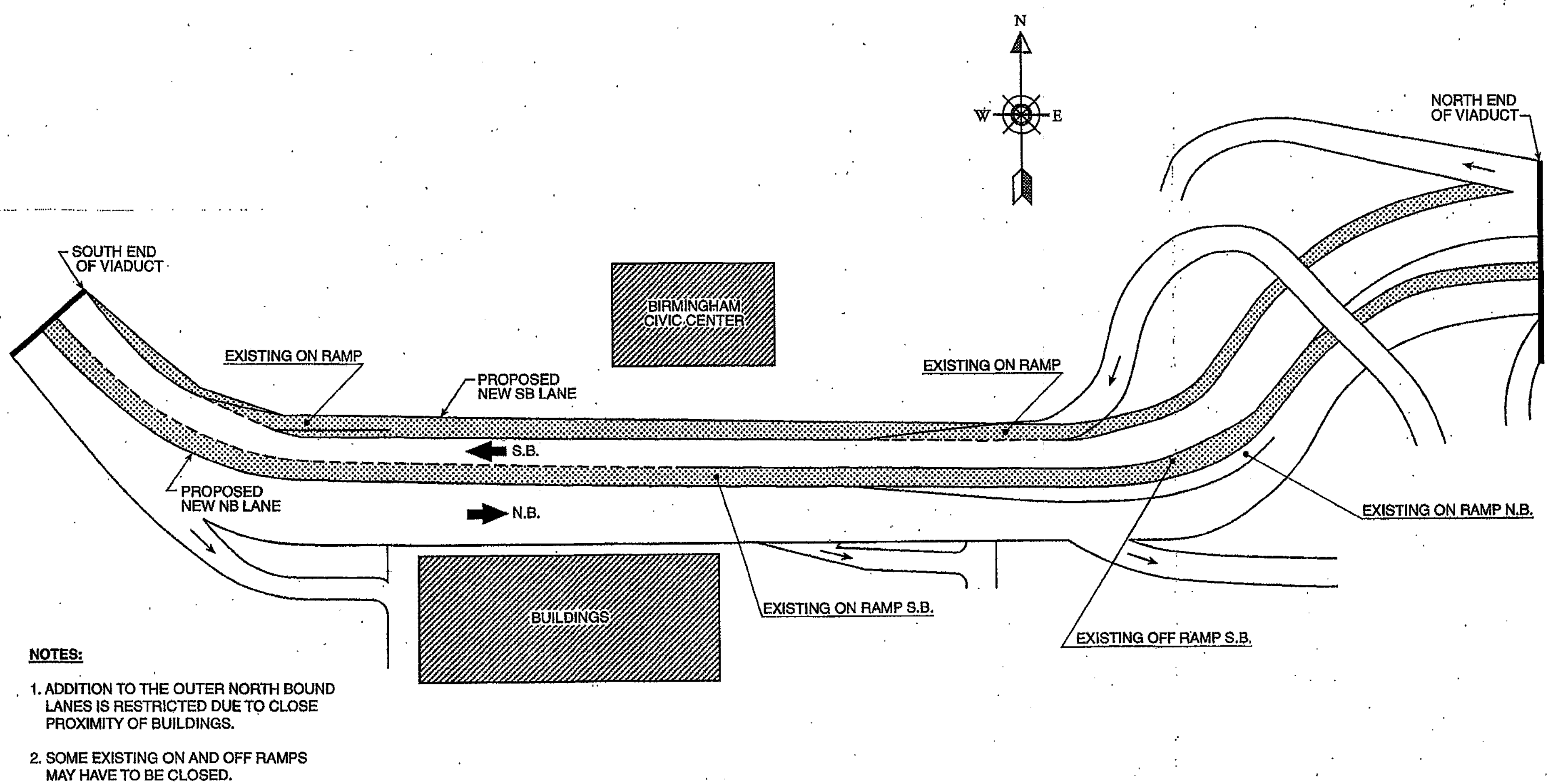


Fig. 14.2. Schematic of I-59/I-20 Lane Additions at Viaduct

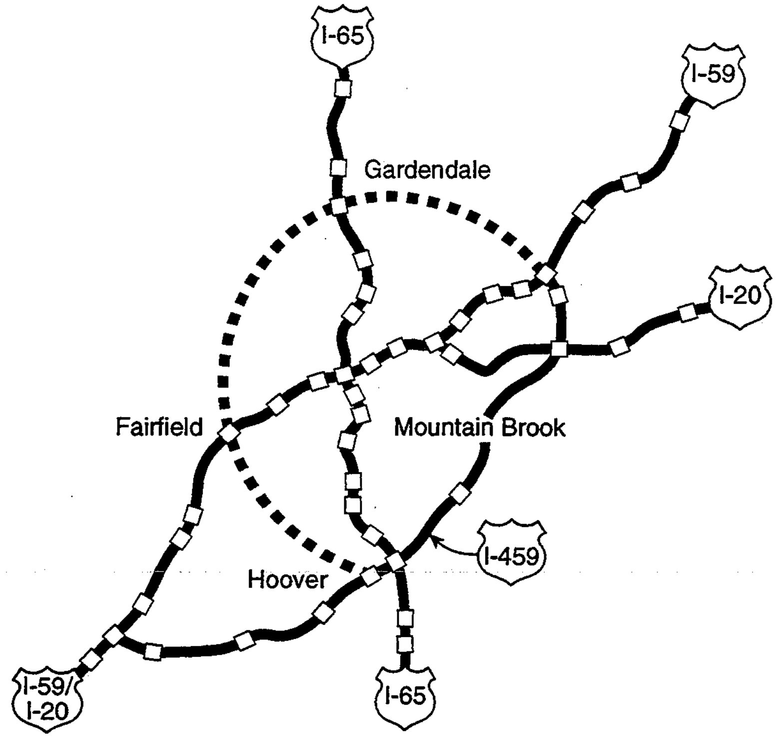


Fig. 14.3. Completion of I-459 Loop Around Birmingham

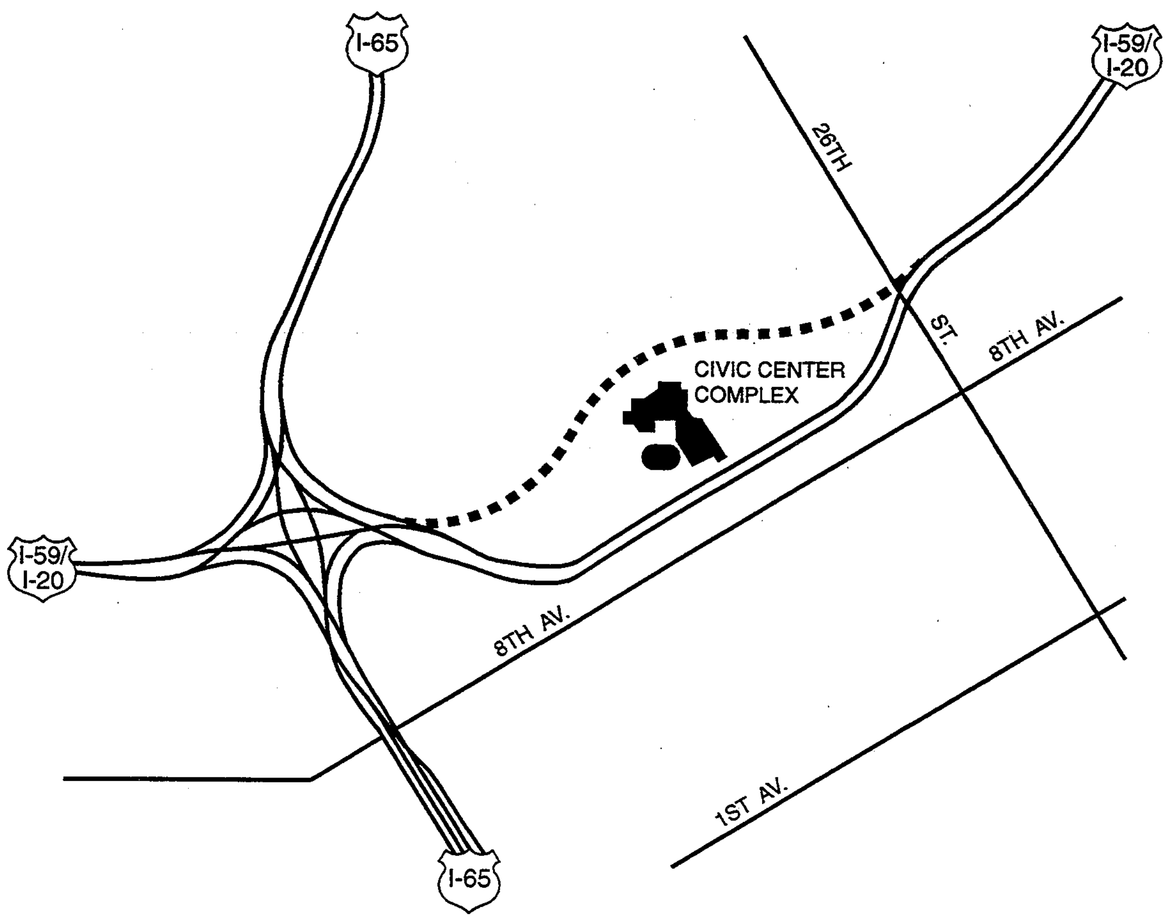


Fig. 14.4. Possible New Section of I-59/20 or I-59S/I20W Through the Birmingham CBD

14.3 Assessment of Structural Condition of Existing Birmingham Bridges

Before any decisions can be made on the best course of action to take for the deteriorating Birmingham interstate bridges, all of the bridges must be examined closely to assess their structural condition. Obviously, it would be foolish to rehab the bridge decks via overlays (which are rather expensive) only to find that the decks begin to have major delaminations and spalling on the underside four years later. Or, to perform a major deck replacement or rehabilitation only to find that the support girders/superstructure have exhausted their service life 10-years later. Or, to replace the superstructure only to find, in a few years, that the substructure has deteriorated to the point of requiring total replacement of the bridge.

Major questions which must be addressed and answered for the Birmingham bridges are as follows:

1. Are the badly cracked (top and bottom) decks structurally sound, i.e., do they have adequate strength or load capacity now and in the foreseeable future?
2. What is the approximate remaining service life of the bridge decks before they must be rehabilitated or replaced due to excessive delaminations and spalling of either the top or bottom surface?
3. What is the approximate remaining service life of the bridge support girders which have a history of fatigue damage?
4. What is the approximate remaining service life of the bridge substructures?

Regarding **Question One**, since punching shear is the structural failure mode for bridge deck, punching shear load testing should periodically be conducted at select locations on the I-65 and I59/20 decks. A bridge on AL79 just north of Birmingham of similar design to the I-65 and I59/20 bridges is scheduled to be taken out of service in 1999 and affords ALDOT the opportunity to

- develop a punching shear load testing capability within its Bridge Load Testing Section
- perform punching shear load testing to failure on the bridge deck at locations of extensive cracking and at locations of minimal cracking to determine failure loads and to determine deck P-Δ characteristics (see Figs. 2.6 and 2.7 in Chapter 2).
- use the equipment, procedures and experience from testing on the AL79 bridge (after it is taken out of service) to perform punching shear proof load testing to 3 times the maximum anticipated truck wheel load at select locations on the Birmingham I-65 and I59/20 decks.

Regarding **Question Two**, the quantification of remaining life of the decks before delaminations on the top or bottom surfaces becomes excessive will be difficult. However, estimates based on rational considerations can and should be made. Delaminations and spalling of the deck top surfaces are beginning to become more prevalent (see Fig. 4.30). These damages typically occur at locations where significant longitudinal cracking has begun and these cracks cross the ever present transverse cracks. These conditions, combined with the flexing of the decks from truck traffic and deck impact loadings, lead to surface spalling. Maintenance personnel and bridge inspectors responsible fro the Birmingham bridges can provide good guidance on the elapsed time between significant longitudinal cracking and the beginning of deck spalling.

Since the deck bottom surfaces of the Birmingham bridges are full of hairline cracks (see Figs. 4.19 - 4.23 in Chapter 4), it is critical that the undersides be examined to assess the seriousness of this cracking regarding providing an access for water to reach the lower rebar mat and the potential for rebar corrosion and bottom surface delaminations and spalling. Sounding of representative regions of the deck bottom surface must be done to determine if delaminations exist at the level of the bottom mat in the decks. If it does, the deck should be removed. It should be noted that representative regions of the deck top surface should also be sounded (drag chained) to identify any significant delamination problems.

The ALDOT is beginning a research project in Fall 1998 to address **Question Three**. However, it should be noted that the bridge on AL79 that is similar to the I-65 and I59/20 bridges, and is scheduled to be taken out of service in 1999, also affords ALDOT an opportunity to improve its assessment of the remaining fatigue life of the bridge steel support girders. Two of the AL79 steel girders should be removed (when it is taken out of service) and fatigue load tested in a laboratory to experimentally assess their remaining fatigue life. Additionally, the tested girders should be strain-gaged int he same manner as those in ALDOT's field project to allow comparison with and possible refinement of the analytical values to be predicted in the ALDOT project.

It appears at this time that **Question Four** is not an area of significant concern. However, it should be assessed before making any decisions regarding major deck or superstructure rehabilitation or replacements.

An additional question arises for the I-65 bridges, i.e., what is the condition of the new portion of the bridge added on the inside when the bridge was widened about 12 years ago? This portion should be examined closely to determine its structural soundness and state, and whether it should be left in place or removed. It appears that these "new" portions are in very good condition and could be left in place. If so, this would significantly reduce the time, cost, and disruption to traffic when the deck is replaced.

14.4 Rehabilitation Options for Birmingham Bridges

Once the decision on how to address the heavy traffic volume problem is made and results of the structural condition of the existing bridges are available, appropriate decisions can be made for rehabilitating the existing bridges. Whatever traffic improvement options are decided on (see Table 14.1), these improvements should be made prior to beginning the major job of rehabilitating or replacing portions of the deteriorating existing bridges if at all possible. Obviously, rehabilitation/replacement work is going to be a source of significant traffic interference for a lengthy period of time, thus any and all traffic flow improvements prior to beginning the work should be made.

Independent of the traffic flow improvements identified in Table 14.1, in the near future (5 - 15 years), some portions of the existing Birmingham interstate bridges are
going to have to be rehabilitated or replaced. The authors feel that the most likely results of the structural condition assessment discussed in the previous section will be as follows:

Bridge Substructures:	50^+ years estimated remaining service life
Bridge Superstructure/ Support Girders:	15 - 50^+ years estimated remaining service life
Bridge Decks:	8 - 24 years estimated remaining service life

It should be noted that remaining life of a bridge component (or any object or component) is not a fixed period of time but is strongly related to the level of maintenance attention it receives. The life ranges speculated on above are the estimate lives <u>before</u> major rehabilitation expenditures are required.

Assuming the substructure remaining life is 50^+ years as indicated above, we can identify "best" rehabilitation strategies for various combinations of estimated remaining life of the bridge support girders and decks. These are shown in Table 14.2. Obviously, other combinations of remaining service life of the bridge components may occur and may be considered; however, those shown in Table 14.2 indicate the need to identify the "best"

- "temporary" or short life overlay (8 years service life) ➜ thin epoxy polymer concrete overlay, or asphalt with membrane overlay
- long life overlay (15 year service life) ➜ epoxy or polyester polymer concrete (Caltran mixture), rapid set latex modified concrete, micro silica modified concrete overlay or 2 applications of asphalt with membrane overlay

Table 14.1 Options to Address Heavy Traffic Demands In and Through Birmingham

Option	Comment
1. Interchange and On/Off ramp improvements.	This option should be taken if interstate traffic laneage is viewed as adequate, but traffic "bottlenecks" need to be eliminated. Interchange, on/off ramp, and acceleration/deceleration lane improvements can significantly enhance peak traffic flow. ALDOT is in the process of exercising this option for the I-65-I59/20 interchange. There may be other such improvements that can be made.
2. Adding one or more additional lanes in each direction on I-65 and I59/20.	This option should be implemented if it is viewed that Option 1 will not solve the heavy traffic demand problem. It should be exercised after or concurrent with Option 1. If more than 1 lane in each direction is needed, this option may necessitate the rerouting of the 1-mile I59/20 viaduct around the north side of the Birmingham Civic Center (see Fig. 14.4).
3. Add alternative interstate routes through or around Birmingham.	This option should be explored as an alternative to Option 2 or as a supplement to Option 2 as the traffic load demands may dictate. The completion of a I-459 loop around Birmingham should be a viable option to explore and is probably an option "whose time has come". It may be advisable to modify the west side of such a loop to make it more circular and thus improve its use by traffic traveling around the loop in any direction (see Fig. 14.3).

Table 14.2. Rehabilitation Strategies for Various Combinations of Estimated Remaining Life for Support Girders and Deck[1]

Deck Estimated Remaining Life	Support Girders Estimated Remaining Life		
	15 years	30 years	50^+ years
8 years	Overlay in 7 years	Replace deck in 7 years	Replace deck in 7 years
16 years	Replace super-structure in 14 years	Overlay in 15 years	Replace deck in 15 years
24 years	Replace super-structure in 14 years	Replace super-structure in 23 years	Replace deck in 23 years

[1]An alternate strategy to those indicated in this table is to add an additional support girder between each existing girder. This is felt to be a viable alternate option for situations where the estimated remaining life of the girders and the deck are 15 years or greater.

- deck replacement scheme → precast/prestress concrete panels with rapid-set CIP toppings, Exodermic panels with rapid-set CIP toppings, or SIP metal forms with CIP topping
- superstructure replacement scheme → various combinations of precast deck and girder combinations to be looked into later.

Each of the above rehabilitation procedures needs to be implemented in a rapid and staged construction manner under concurrent traffic conditions.

Another rehabilitation option which the authors feel is a viable candidate for serious consideration is the addition of a new steel support girder between each of the existing support girders. These girders could be hoisted into place via crane or by attaching lift equipment to the existing girders. An attractive feature of this rehabilitation scheme is that the work can be done from the underside of the bridges with little or no disruption of traffic. This scheme is indicated by footnote in Table 14.2. If it is implemented it should significantly increase the service life of the support girders as well as the deck. Obviously this is another attractive aspect of this rehabilitation scheme.

It is possible that adding supplemental girders would introduce the requirement of increasing the capacity of the bent caps. However, after reviewing the original plans showing the 3-column piers, Randall Mullins of ALDOT's Bridge Design Bureau feels that strengthening the bent caps will probably not be necessary.

As indicated in the last two paragraphs, various materials and procedures within each basic rehabilitation strategy are viable candidates for consideration, e.g.,

- Overlay
  - -Thin epoxy polymer concrete
  - -Asphalt with membrane
  - -Latex modified concrete
  - -Micro silica modified concrete
  - -Polyester polymer concrete (Caltran design)
- Deck replacement
  - -Precast prestressed concrete panels
  - -Exodermic panels
  - -SIP metal forms with CIP concrete

Because of the large volume of rehabilitation work to be implemented, the necessity to minimize traffic disruptions during this work, and the desire for the bridges to have a maximum service life once they are rehabilitated, it would be wise to place some test sections of each of the viable candidates in the field. This would allow an accurate evaluation of the construction friendliness, time of construction/traffic disruptions, costs, early performances (to allow projection of long-term performances and service life), etc. of the viable candidates. In turn, this should provide a good degree of confidence in selecting the "best" rehabilitation actions for the Birmingham situation.

It should be noted that a test section of adding additional support girders to an existing bridge would allow a measurement of reduced girder and deck stresses under traffic which in turn could be used to provide an estimate of the increased service life resulting from this rehabilitation strategy.

15. CONCLUSIONS AND RECOMMENDATIONS

15.1 General

The ALDOT has over 4,830 m (3 miles) of major interstate bridges (3 to 5 lanes wide with approximately 55,740 m^2 (600,000 ft^2) of deck) near downtown Birmingham with significant levels of deck cracking and deterioration. The bridges are part of the I-65 and I59/20 interstate highway system through the city, and are approximately 27 years old. The rehabilitation of these bridges is obviously a matter of great concern because of the enormous cost and potential for disruptions of traffic. Specific primary concerns about the Birmingham I-65 and I-59/20 bridges are as follows:

1. Inadequate traffic lanes and traffic capacity (on I-65 and on I-59/20 from the I-59/20 juncture to the I-65 interchange in particular).
2. Significant levels of live load deflections and out-of-plane movement of the deck superstructure system.
3. Significant level and rate of increase of deck cracking and deterioration which is requiring ever increasing maintenance attendance in the form of surface spalls and potholes (which generally require full-depth patches). In turn, this is reducing the bending stiffness in both the longitudinal and transverse directions and leading to greater deflections and cracking, and will eventually lead to deck punching shear failures.
4. Extensive state of fine cracking on the deck undersides with a concern for future underside spalling problems which would create a safety hazard, and additional maintenance requirements.
5. Past history of fatigue problems with diaphragms, diaphragm-to-girder connections, and support girders (at the locations of transverse diaphragms) and a concern that the girders may be approaching their fatigue limit/life and need to be replaced.

The objective of this research work was to identify the most viable rapid bridge deck rehabilitation or replacement (R/R) options which can be implemented under staged construction/concurrent traffic conditions. This being the case, only bridge deck R/R were considered in detail. However, some consideration was given to deck support girder rehabilitation/replacement, addition of traffic lanes, alternate routing, etc. because these actions are integrally related to actions which should be taken on the bridge decks.

15.2 Conclusions

Based on a review of the literature, a mail questionnaire survey to all State DOTs in the U.S., telephone discussions with DOT bridge and maintenance engineers in over half the states in the U. S., in-person meetings with select personnel of the ALDOT from hands-on bridge maintenance and inspection personnel to bureau chiefs of the primary player bureaus, site visits to the Birmingham bridges, discussions and meetings with bridge deck product industry representatives and bridge construction contractors, and site visits to four states to observe and discuss their rapid deck rehabilitation practices, the following conclusion and recommendations were drawn and are herein offered.

- ALDOT traffic planning personnel indicate that the interstate system through Birmingham is already at or exceeding its design capacity. Decisions on how the system will be "upgraded" to handle additional traffic demands need to be made before making decisions on how to rehabilitate the deteriorating I-65 and I59/20 bridge decks if at all possible. As a first step in deciding the best course of action for the Birmingham bridge decks, a survey questionnaire was prepared and sent to all 50 states in the U.S. to determine how other states handle their deck deterioration problems. The questionnaire focuses on information performing to (1) rapid bridge deck replacement, (2) rapid overlaying of bridge decks, and (3) overlaying of severely cracked decks. Forty-one (41) states participated in the survey for an 82 percent response.

  Survey results indicated that approximately 54% of the participating states have executed rapid deck replacement in an urban setting with staged construction and concurrent traffic. All of these states have used CIP concrete as the new decking material, and almost all use either CIP concrete or precast concrete panels with a CIP topping.
  A high percentage (80%) of the responding states have executed deck overlaying in urban settings with staged construction and concurrent traffic. Low slump dense concrete (Iowa or similar mixture) and latex modified concrete are the most widely used overlay materials, and they have provided mean service lives of approximately 22 years and 17 years respectively. Micro silica modified concrete appears to becoming the overlay material of choice for many states; however, overlays of this material have not been in place long enough to determine its service life. Polymer concrete is a very popular and highly regarded overlay material in California. Caltran makes extensive use (placing approximately 25 overlays a year) of a 1.90 cm (0.75 in) thick polyester polymer concrete overlay that is providing excellent performance. Interestingly, 21% of the states employing overlays make use of asphalt with membrane, and these overlays have provided a very respectable mean service life of approximately 14 years. Also, 56% of the responding states have employed overlays on badly cracked decks.

- A bridge on AL79 near Birmingham which is of similar design as the I-65 and I59/120 bridges is scheduled to be taken out of service in 1999. This bridge should be used (after it is taken out of service) to help assess the punching shear load capacity of the I-65 and I59/20 decks and to assess the remaining fatigue life of the support girder for these same bridges.

- Punching shear failures have occurred on some ALDOT bridge decks in the past, e.g., on some I59 decks near Gadsden, AL, on an AL79 bridge deck near Birmingham, and when 40 ton rolls of steel fall off of trucks. These have been the only deck structural failures in Alabama to the authors' knowledge. Punching shear is a localized failure mode and, while obviously not desirable, it does not typically lead to catastrophic accidents and is relatively easy to repair. It has been observed in Alabama deck punching shear failures, that they occur in regions of relatively new and growing longitudinal deck cracking (transverse deck cracking is typically quite prevalent at almost all locations), and at locations about midway between support girders. Thus inspecting for significant longitudinal deck cracking and identifying effective and efficient repair schemes for such cracks is believed to be a key factor in avoiding deck punching shear failures. Recent analytical and experimental analyses by Canadian bridge engineers and researchers, and others, as well as ALDOT's own experiences indicate that concrete bridge decks designed according to AASHTO specifications fail in a punching shear mode. This being the case, it would be helpful in assessing the state of the current Birmingham decks to conduct limited punching shear field testing. This testing could be limited to 2 or 3 times service loads levels, or carried to failure.

- A deflection comparison between experimental and theoretical results was made in this study for two typical Birmingham, AL interstate bridges. The experimental results were obtained in the summer of 1994 as part of a study by Dr. M. Stallings on diaphragm behavior and performance in multigirder steel bridges for the ALDOT. A special truck was used to apply loads to the bridges while deflectometers were used to determine the deflections. The theoretical results for deflections were obtained by ALDOT's Bridge Bureau using computer programs. The reasons for making the comparisons were twofold, i.e.,

    1. to establish benchmark values of load-deflection responses for future comparisons to assess structural degradation of the bridge girder-deck superstructure system.
    2. to determine how effective it would be to add shear lugs to the girders to attain composite action with the deck and thus enhance the stiffness and strength of the superstructure via a deck replacement.

Results of maximum girder deflection load testing indicate that both the 24.5 m simple span bridge and the 76 m three span continuous bridge are behaving in a composite manner. This is as it should be for the simple span bridge since it was constructed with shear lugs to achieve composite behavior. However, the continuous span bridge does not have shear lugs, but is apparently achieving composite behavior by friction and adhesive forces. This suggests that as decks are replaced on the continuous span bridge, and shear lugs are added to the girders for composite action, no significant stiffening of the superstructure and reduction in deflections will occur since the superstructure is currently acting in a composite manner. However, if the decks are replaced, shear lugs should still be placed to assure composite behavior in a positive manner.

- Several questions which are essential to deciding on the best rehabilitation strategy for the Birmingham bridges remain unanswered. These are:

    1. What is the present state of structural adequacy of the bridge decks (present factor of safety against structural failure)?
    2. What is the remaining service life of the bridge superstructure girders?
    3. What are the construction friendliness, required traffic disruptions, and costs of the most viable deck replacement schemes identified in this work?

Addressing these questions in order to arrive at accurate or best estimate answers is needed before final decisions can be made on the best course of action for the Birmingham bridges. It should be noted that ALDOT will begin a research project (with Dr. M. Stallings) to address the question of remaining service life of the Birmingham bridge superstructure girders beginning in Fall 1998. However, it should also be noted that the girders on a soon to be removed bridge on AL79 afford the opportunity to laboratory test some girders to verify and/or calibrate the remaining fatigue life prediction model, and perhaps to assess the effectiveness of girder retrofit strengthening procedures to increase fatigue life.

- The most viable rehabilitation options at his time appear to be
    - rehabilitate the bridge decks via the use of overlays (for 10 - 20 year life extension)
    - replace the decks
    - add additional longitudinal girders to strengthen and stiffen the existing deck and superstructure
    - replace the bridge superstructures

  Which of the rehabilitation strategies would be the most cost-effective for the Birmingham bridges depends on knowing the structural adequacy of the existing concrete decks and their estimated remaining service life, along with the remaining service life of the bridge girders.

- Cost comparisons of various rehabilitation strategies in Chapter 13 reflect that deck overlaying is definitely the least costly strategy in terms of lowest initial cost. However, the service life is lower and the life-cycle cost may not be the lowest. Additionally, thin deck overlays would do little to stiffen and strengthen the existing deck/superstructures, and thin overlays appear to require greater construction expertise and quality control than is typical in bridge construction in order to attain a quality overlay. Thicker overlays would stiffen and strengthen the bridges, however with the exception of the Caltran polyester polymer concrete overlay, they tend to be more brittle. The high level of existing deck cracks and superstructure flexibility of the Birmingham bridge could cause a cracking or debonding problem with these overlays.

- Cost comparisons of various deck replacement strategies in Chapter 13 reflect that a CIP rapid-set concrete on SIP metal forms is the least expensive deck replacement. However, it would probably also take a little longer to implement and would disrupt the traffic a little more. The addition of supplemental girders under the existing deck strategy gave the highest cost in Table 13.12. However traffic control costs for this strategy should be less as would interference with traffic. Also, if a nonshrink grout rather than epoxy is used as a filler/bonder between the top of the new girders and the underside of the deck, then the cost of this strategy would be about the same as the Exodermic panels. It should be noted that the NYTA found that deck replacements employing precast Exodermic panels, prestressed concrete panels, and filled steel grid panels were economically competitive.

15.3 Recommendations

Based on the work performed and knowledge gained in performing this research work, the following recommendations pertaining to the Birmingham interstate bridges and decks are made.

1. Since traffic capacities of the I-65 and I59/20 systems through Birmingham appear to be at or exceed capacity at this time, a study should be initiated to decide on the best means of increasing the interstate traffic capacity through the city. This study should be initiated immediately since it strongly impacts decisions on the best course of action in rehabilitating the existing I-65 and I59/20 bridge decks which have a limited remaining service life (approximately 10 years).

2. Perform field and laboratory testing as necessary to determine the remaining fatigue/service life of the bridge support girders. This should include laboratory testing of two girders extracted from the AL79 bridge near Birmingham when it is taken out of service in 1999 in addition to the planned in-situ testing of some of the Birmingham interstate bridges.

3. If the results of the girder remaining fatigue life study indicates a remaining life of 15 - 25 years, then perform the structural condition assessment plan discussed in Chapter 14 to determine if the deck is sufficiently sound to rehabilitate via overlay. The remaining elements of this plan (after the assessment in (2) above) are:

    - Spot drill holes in the underside of the Test Section to be rehabilitated via overlay to assess the depth of penetration of the underside cracking grid.
    - Drag chain (or equivalent) the top and bottom surfaces of the Test Section to be rehabilitated via overlay to assess horizontal delaminations.
    - Test deck to be overlayed to a load level 3 times the maximum probable wheel load to assess punching shear adequacy.

4. Use the AL79 bridge near Birmingham that is due to be taken out of service in 1999 as a resource to help decide the state of and best course of action for the Birmingham interstate bridges. More specifically, this bridge should be used (after it is closed to traffic) as follows:

    - Strain gage at select locations and field test to evaluate remaining fatigue life in the same manner as will be done by Dr. Mike Stallings on the I-65 and I59/20 bridges.
    - Remove two support girders from bridge and send these to Auburn for fatigue load testing to experimentally evaluate remaining fatigue life. This data can be used to calibrate the field data from the I-65 and I59/20 testing.
    - Perform deck punching shear load testing to failure and develop P-Δ curves for such testing at locations of severe and minor deck cracking.
    - Implement deck repair scheme(s) and load test in punching shear to identify improved repair and preventative maintenance schemes.

5. Place and monitor the performances of four candidate deck replacement/ rehabilitation "test sections" on a Birmingham bridge(s) of similar construction to the I-65 and I59/20 bridges. The four "test sections" should be:

    - Exodermic grid with a rapid-set CIP concrete topping (full depth Exodermic panels typically require either a wearing topping, overlay, or surface grinding, neither of which is an attractive option).
    - Precast and prestresses concrete SIP continuous panels (NU panels) with a rapid-set CIP concrete topping (full depth prestressed panels require the same top surface treatment as the full depth Exodermic panels).

- CIP concrete with steel SIP forms and Varigrid reinforcing steel mats and a rapid-setting Type III concrete (perhaps the GADOT structural overlay mixture).
- Addition of W36 or W33 steel girders between each of the existing W36 girders. The bridge bent caps can probably take these new girders as they are. If not, they can be strengthened to take them. This system will significantly stiffen and strengthen the bridge superstructure, extend the fatigue life of the old girders, and all of the work can be done from the underside with little or no disruption of traffic.

A CIP concrete topping is recommended for both of the deck replacement panels (Exodermic and NUDECK), rather than use of full-depth precast panels. This recommendation was essentially based on the need to use a thin bonded overlay or surface grinding to provide a smooth riding surface when using full-depth panels. The cast-in-place topping will provide the same smooth and uniform riding surface as when using conventional cast-in-place decks. Also, with the rapid setting concrete that can be produced today, it is believed that construction time will not be exceedingly slow when compared to setting panels and then placing a bonded overlay.

The bridge(s) selected for placement of the "test sections" should probably be one with a badly deteriorated deck that currently requires major rehabilitation. It should have its deck replaced/rehabilitated as "test sections" in a rapid and staged construction manner even though the traffic conditions at the location may not dictate this manner of replacement. In turn, this will allow an evaluation of most of the parameters of interest in determining the best strategy for rapid replacement/rehabilitation under concurrent traffic for the Birmingham interstate bridge decks. Each "test section" should extend from expansion joint to expansion joint longitudinally, and should extend full width of the bridge to fully assess panel joining, construction staging, and traffic control issues.

For deck replacement (first three candidates above), the new deck should be made composite with the girders via the welding of shear lugs to the girders. Additionally, the new decks should be made 8" thick with the deck being thickened on the bottom side (eliminate 2" haunch at the girders) rather than the top. However, the new deck top surface could probably be 3/4" higher than the old deck top surface without incurring significant problems if needed.

Each of the systems should be placed for one span of the bridge in a staged construction manner in order that restricted but concurrent traffic may be maintained. Field placed reinforcing steel should be Varigrid mats to reduce construction time, and the CIP concrete topping should be a rapid-setting Type III Portland cement concrete mixture similar to that used by the GADOT in their structural deck overlays. Each of the systems should be monitored to document its:

- construction "friendliness"
- required lane closure time
- costs
- first year performance

6. If results of the Structural Condition Assessment Plan indicate remaining girder and deck service lives that point to deck rehabilitation via overlay as indicated in Table 13.1, then overlay "test sections" should be placed and monitored. Candidate overlay "test sections" should be:

- Thin (approximate 3/8") bonded epoxy polymer concrete (EPC) as proposed by Thermal-Chem
- Asphalt (approximately 1 1/2") with membrane overlay
- 1 1/4" rapid-set latex modified concrete
- 1 1/2" micro silica modified concrete
- 3/4" polyester polymer concrete (Caltran design)

These are temporary rehabilitations, but the first two should provide 8 - 12 years of service life, and the last three around 12 - 20 years of service. The same criteria for the test bridge selection and overlay placement size, location, procedures, and monitoring as for the deck replacements in (5) above are recommended.

7. Expand the scope of this research project to allow a continuing moving forward in determining the best course of action in rehabilitating the Birmingham interstate bridge decks. This expansion of scope should be done immediately to allow maintaining the momentum of the work, and because time is limited as the bridge decks continue to deteriorate at an accelerated rate. This would allow the PI to use the knowledge gained and contacts made during this investigation, to move forward and work with the ALDOT in implementing the recommendations made above. In turn this will provide the ALDOT with the information needed to make an informed and good decision on the best course of action for the deteriorating Birmingham bridge decks.

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APPENDIX A

Survey Questionnaire

Auburn University

Auburn University, Alabama 36849-5337

College of Engineering

Highway Research Center
Department of Civil Engineering
238 Harbert Engineering Center

Telephone: (334) 844-5250
FAX: (334) 844-6290

August 7, 1997

Mr. Steve Bradford
Chief Bridge Engineer
Alaska Department of Transportation
and Public Facilities
3132 Channel Drive
Juneau, Alaska 99811

Dear Mr. Bradford:

I have prepared the enclosed questionnaire in hopes of learning of your experiences in the area of "Rapid Bridge Deck Replacement or Overlaying Under Concurrent Traffic Conditions."

We have little experience with rapid bridge deck replacements or overlays in Alabama, and unfortunately many of our bridge decks in highly trafficked urban areas are beginning to show signs of significant distress (cracking). Thus, I am working with the Alabama Department of Transportation (ALDOT) in identifying how other state DOTs are handling this problem.

Your assistance via completing the enclosed questionnaire would be most helpful to us. If another section in your agency would be more appropriate in responding to all or part of the questionnaire, I would appreciate your coordinating a complete response from your state. We hope to assemble and analyze the survey data by the end of September. Therefore, receipt of your response by September 15, 1997 will be appreciated.

Please contact me at the number on this letterhead or on the last page of the questionnaire if you have any questions. Thank you in advance for your help.

Sincerely,

G. E. Ramey
Feagin Professor of Civil Engineering

GER/gl
Enclosure

Survey Questionnaire
on
Rapid Bridge Deck Replacement or Overlaying
Under Concurrent Traffic Conditions

Instructions

The questions below pertain to situations where bridge superstructures and substructures are in good/ sound condition, and deck replacements or overlays are expected to provide good service for many years.

Questions

1. Have you executed bridge deck replacements in urban settings under heavy traffic requiring staged and rapid construction with concurrent traffic (1 or 2 lanes closed and 1,2 or more lanes open)?

   ☐ Yes

   ☐ No

   If No, go to next question.
   If Yes, please indicate:

   a. Type of replacement deck employed.

   b. Everything considered, i.e., design, construction, maintenance, cost, etc., are you pleased with this type of deck replacement? ☐ Yes ☐ No

   c. What is the longest period of satisfactory performance of this type of deck replacement in your state?

   d. Other types of deck replacement you have employed and their years of satisfacatory service.

   e. Time schedule for replacement work (e.g. weekends, nights only, etc).

   f. Are you pleased with this work time schedule? ☐ Yes ☐ No

   g. Other work time schedules you have employed.

   h. Traffic control/construction work zone lane closure procedure employed.

   i. Are you pleased with this traffic control/ work zone procedure? ☐ Yes ☐ No

   j. Other traffic control/construction work zone lane closure procedures you have employed.

2. Have you executed bridge deck overlaying in urban settings under heavy traffic requiring staged and rapid construction, with concurrent traffic (1 or 2 lanes closed and 1,2 or more lanes left open)?

☐ Yes

☐ No

If No, go to next question.
If Yes, please indicate:

a. Type and thickness of deck overlay employed.

b. Everything considered, i.e., design, construction, maintenance, cost, etc., are you pleased with this type of deck overlay? ☐ Yes ☐ No

c What is the longest period of satisfactory performance of this type of deck overlay in your state?

d. Other types and thicknesses of deck overlays you have employed, and their years of satisfactory service.

e. Time schedule for overlay work (e.g. weekends, nights only, etc).

f. Are you pleased with this work time schedule? ☐ Yes ☐ No

g Other work time schedules you have employed.

h. Traffic control/construction work zone lane closure procedure employed.

i. Are you pleased with this traffic control/work zone procedure? ☐ Yes ☐ No

j. Other traffic control/construction work zone lane closure procedures you have employed.

3. Whether under rapid and staged construction conditions <u>or</u> under normal time and bridge closure conditions, have you employed overlays on bridges with badly cracked decks (see attached Figs).

   ☐ Yes

   ☐ No

   If No, go to next question
   If Yes, please indicate:

   a. Type and thickness of overlay employed.

   b. Everything considered, i.e. design, construction, maintenance, cost, etc., are you pleased with this type of overlay? ☐ Yes ☐ No

   c. What is the longest period of satisfactory performance of this type of deck overlay in your state?

   d. Other types and thicknesses of overlays you have employed for this type of deck deterioration, and their years of satisfactory service.

4. Contact person in the event that additional questions arise (if your answers to Questions 1-3 above were all <u>No</u>, just give your <u>organization</u>).

   Name: ______________________________

   Organization: __________________________

   Address: ______________________________

   ____

   ____

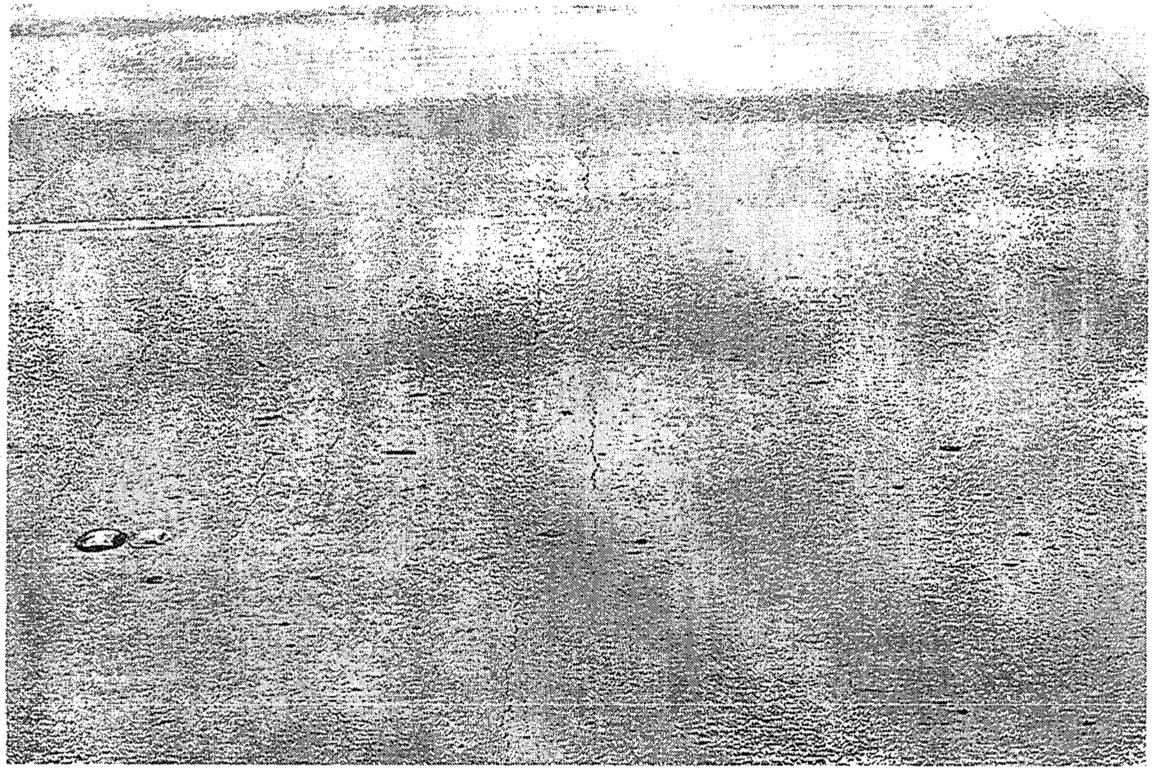
   Phone: ______________________________

Thank you for taking the time to share your expertise and experience with us!

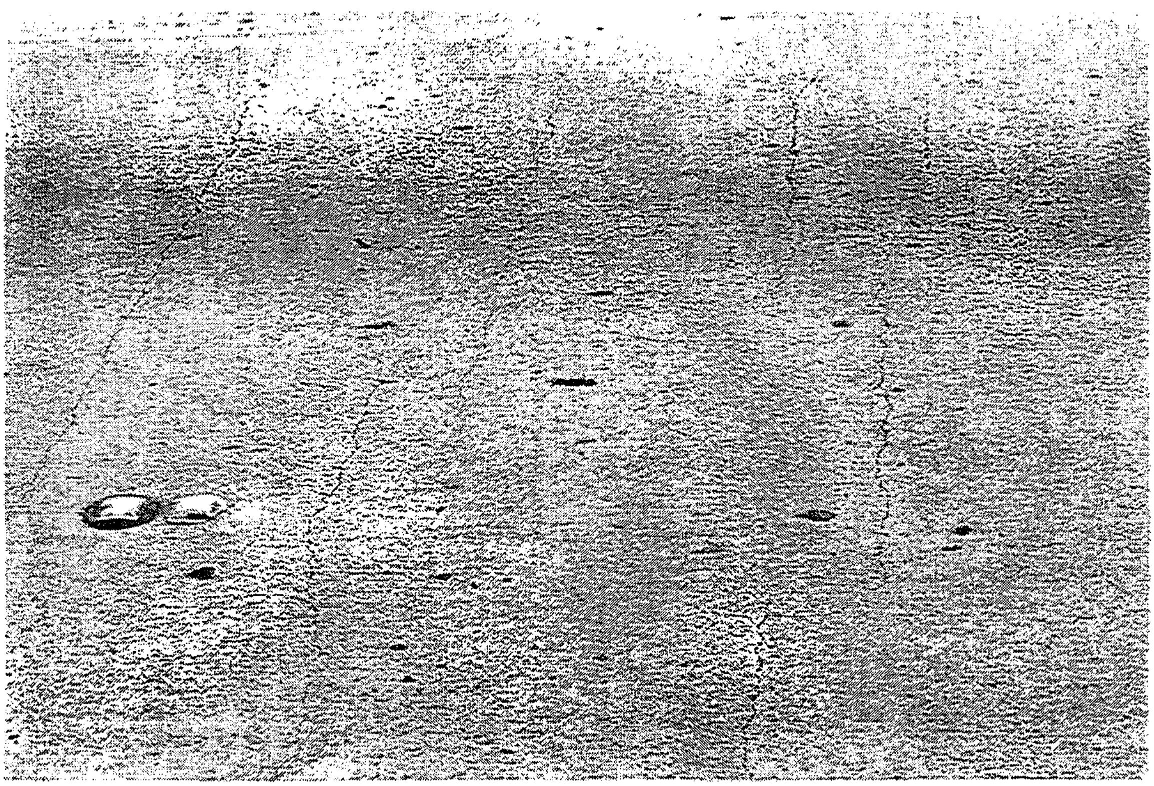
Please return this questionnaire (first 3 pages) in the self-addressed and stamped envelope provided. In the event that the envelope is lost, please return to:

Dr. G. Ed Ramey
Department of Civil Engineering
Harbert Engineering Center
Auburn University, AL 36849

Telephone: (334) 844-6292
Fax: (334) 844-6290
E-Mail: geramey@eng.auburn.edu

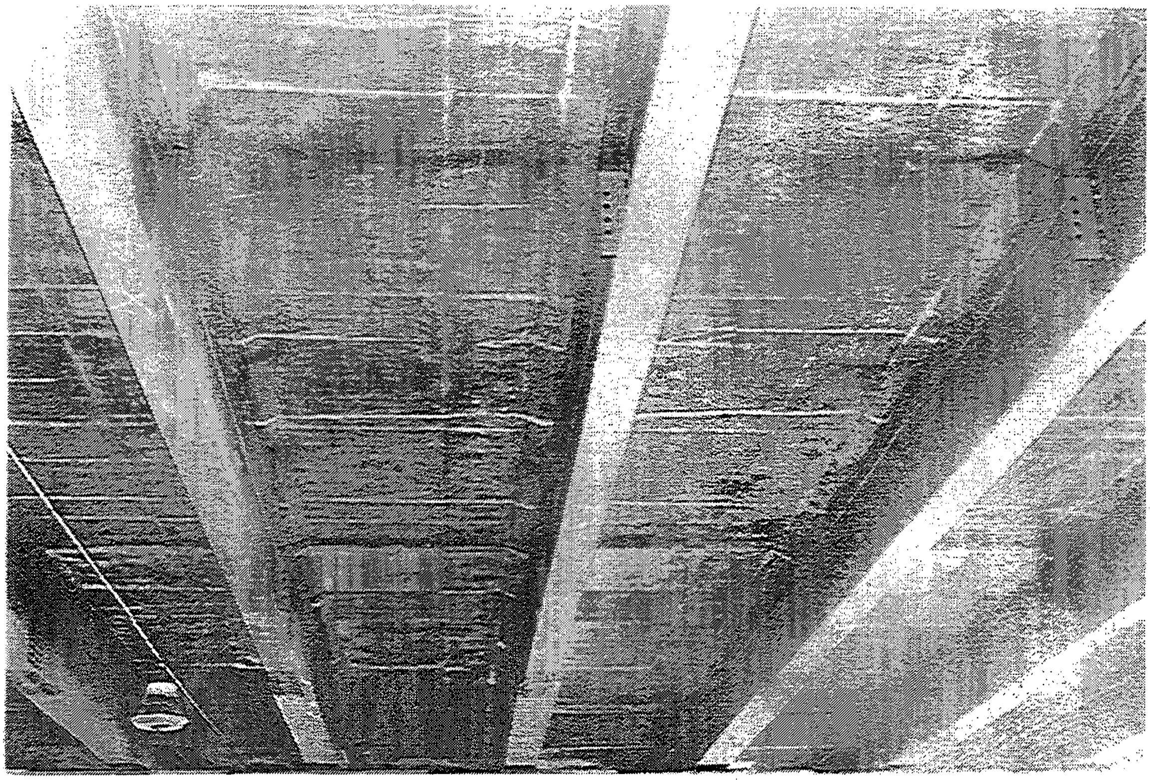


Extensive Transverse Cracking on I-65 Bridge

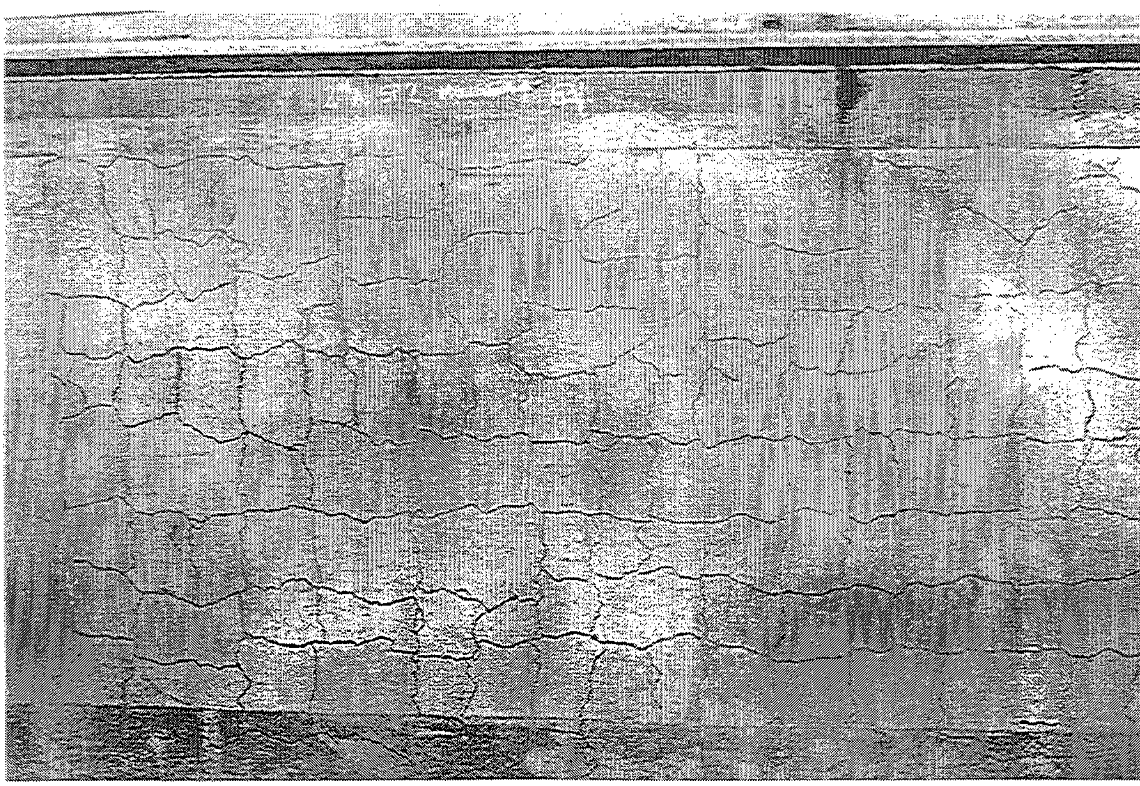


Close-up of Transverse Cracking on I-65 Bridge

FIGURE 1. DECK CRACKING - TOP



Underside of I-65 Bridge



Close-up of Underside of I-65 Bridge at Midspan with Hairline Cracks Highlighted

FIGURE 2. DECK CRACKING - BOTTOM

APPENDIX B

Bridge Deck Rehabilitation/Overlay
Product Summary Sheet

BRIDGE DECK REHABILITATION/OVERLAY
PRODUCT SUMMARY SHEET

Name: GEORGIA DOT FAST TRACK BRIDGE DECK OVERLAY CONCRETE

Concrete Mix Requirements:

Type III Cement Minimum (lbs/cu. yd.)	750
Coarse Aggregate Size No.	7
Maximum Water/Cement Ratio	.40
Slump Limits (Jobsite) Maximum	7 inches
Air Acceptance Limits (Jobsite)	3.5 to 7.5%
Admixture (Required)	Type F or G
Minimum Compressive Strength (Jobsite) @ 24 hours	3000 psi

Temperature Limitations:

Concrete Placement Temperature (Range)	70° - 90°F
Air Temperature (Range)	50° - 90°F

Weather Limitations:

Placement of the overlay shall be performed during favorable weather conditions. When the atmospheric temperature is expected to exceed 90 degrees F during daylight hours, the Contractor shall schedule work hours that insure complete placement by 10:30 a.m. Placement shall not be scheduled when wind velocity is expected to exceed 20 mph or when rain is expected. The minimum acceptable temperature of concrete at the point of delivery shall be 70 degrees F. Overlay concrete shall be kept at a temperature above 70 degrees F for at least 24 hours after placement.

Typical Mix Design:

The contractor shall submit a mix design for approval. This submittal shall include the actual quantity of each ingredient and laboratory designs which demonstrates the ability of this design to attain a compressive strength of 3500 psi at 24 hours. Laboratory acceptance strengths shall be determined by at least eight compressive test specimens prepared and cured in accordance with AASHTO T-126. The specimens shall be made from two or more separate batches with an equal number of cylinders made from each batch.

A typical mix design used by the contractor (provided 4000 psi concrete in 24 hours) is given in Table 1.

Table 1. Typical GADOT's Rapid-Set Structural Concrete Overlay Mixture*

Ingredient	Weight or Volume/yd^3	Comment
Type III Cement	750 lb	Cement rich and "hot" mix
Crushed Aggregate #7 (½" max size)	1700-1750 lb	Can use limestone
Sand (#10)	1100 lb	Can not use limestone because of polishing characteristics
Water	35 gal (292 lb)	w/c = 0.39
AEA	4%	Sufficient AEA to achieve a 4% air content
HRWR	Manufacturer's recommended dosage	Add at job site to achieve a max slump of 7"
Slump	3" - 4" at Batching Plant	Max of 7" at job site (after adding HRWR at job site)

*Cement rich, Type III cement, HRWR increase the cost of the mixture approximately $20/$yd^3$

BRIDGE DECK REHABILITATION/OVERLAY
PRODUCT SUMMARY SHEET

Name: "RSLMC" Rapid Set Latex Modified Concrete

Manufacturer: CTS Cement Mfg. Co. & Dow Chemical Co.

Description: CTS's Rapid Set Cement, sand, stone, water, and Dow Modifier A Latex

Typical Applications: Bridge Deck Overlays, Parking Garage, Industrial Floors

Advantages: Good bond, tensile strength, durability, low permeability, and flexibility

Limitations: See ACI 306 Recommended Practice For Cold Weather Concreting and ACI 305 Recommended Practice For Hot Weather Concreting

Surface Preparation Requirements: Removal of all loose contaminants detremental to achieving bond

Typical Overlay Thickness: 1 1/4" Minimum

Typical Application Procedure: RSLMC produced by on-site mobile mixers and placed by self propelled finishing machine

Curing Requirements: 2 hours wet minimum

Weather Condition Constrains: * See Limitations

Special/Safety Handling Requirements: See attached Material Safety Data Sheet

Estimated Construction Time: Depends on accessibility to the different areas of the structure

Expected Service Life: 20+ Years

Estimated In-Place Initial Cost: $800 - 1200 per cubic yd. again depending ~~upon contractor accessibility~~

Basis of Payment: 30 day estimates for the contractor RSLMC bid item is; furnish & mix by the cubic yd, place finish & cure by sq. yd

PHYSICAL PROPERTIES

Design Properties:

- Compressive Strength @ 6 hrs - 3300 - 4100 psi
  @ 24 hrs - 4400 psi
  @ Ultimate - 5000 psi at 28 days
- Tensile Strength @ ~~6 hrs~~ 1 day - 1150 psi
  3 days @ ~~24 hrs~~ - 1300 psi
  @ Ultimate - ______ psi
- Tensile Bond Strength @ ~~x6~~ 12 hrs - 200 psi ACI 503
  @ 24 hrs - 300 psi " "
- Shear Bond Strength @ 6 hrs - ______ psi not known
  @ 24 hrs - ______ psi
- Modulus of Elasticity - ______ psi not known
- Tensile Elongation - ______ % not known
- Thermal Coefficient of Expansion - ______ in/in/°F " "
- Shrinkage Strain @ 28 days - wet +.0006% dry -.02% in/in
  @ Ultimate - ______ in/in
- Properties Significantly Affected by Ultraviolet Light? ☐ Yes ☒ No

Construction Properties:

- Shelf Life - One Construction Season
- Pot Life - One Construction Season
- Initial Set Time - ______
- Approximate Elapsed Time to Open to Traffic - 2 - 4 hours after placement
- Workability - Slump to help determine amount of Citric Acid Retarder

Other Pertinent Properties: High Quality Rapid Repair Product. Needs to be done by experienced Bridge Contractors.

BRIDGE DECK REHABILITATION/OVERLAY PRODUCT SUMMARY SHEET

Name: THERMAL-CHEM ARMORFLEX T, PRODUCT NO.107
A MIRCO-THIN LOW-MODULUS EPOXY POLYMER
CONCRETE OVERLAY - BROADCAST METHOD

Manufacturer: Thermal-Chem Corporation
2550 Edgington Street, Franklin Park, IL 60131 USA
800/635-3773

Description: A low-modulus Epoxy Polymer Concrete applied as a coating with a select gradation of aggregate broadcasted into the polymer. Two or three lifts are applied to create the final thickness of 6 to 9mm (1/4 in. to 3/8 in.).

Typical Applications: Bridge decks, ramps and slab-on-grade concrete pavements for vehicular traffic uses.

Advantages:
1. Prevents infiltration of chloride ions and water.
2. Improves skid resistance and ride quality.
3. Quietness of ride.
4. Excellent wear capabilities.
5. Useable within hours after placement.
6. Extends concrete life-cycle.
7. An initial cost effective, corrective and protective system.
8. Low life-cycle cost.
9. Minimal traffic interruptions during placement.
10. Factory technicians available to assist during installation.

Compliance: * ACI 548 Epoxy Polymer Concrete Broadcast Overlays
* AASHTO-AGC-ARTBA, Task Force 34, Epoxy Polymer Concrete Overlays

Limitations: Minimum temperature during application and cure is 5°C (40°F).
Do not apply to wet surface.

Surface Preparation Requirements: Clean sound concrete . Typical methods of cleaning include sand blasting and shot blasting.

Typical Overlay Thickness: 6 to 9 mm (1/4 in. to 3/8 in.).

Typical Application Procedure: Spray or squeegee and roll base epoxy coat evenly over the concrete surface and immediately broadcast aggregate onto the surface until a dry layer develops. When the epoxy becomes tack-free the excess aggregate is removed and a second coat of epoxy is applied over the surface. Immediately broadcast aggregate onto the wet epoxy until a even dry layer of aggregate has covered the entire surface. The excess aggregate is removed when the epoxy has become tack-free. Two lifts typically measure 6 mm (1/4 in) thickness. An additional lift can be placed for thicker pavement requirements.

Curing Requirements: Approximately four hours in the 70 degree (20) temperature range after the final lift. Traffic may then be allowed to use the new epoxy polymer concrete pavement. ACI 503 R Tensile Pull Test can be used to determine adhesion to the concrete and compressive strength test can be used to determine the time necessary before traffic can travel over the pavement when temperatures are lower.

Weather Condition Constrains: The cleaned concrete surface should be dry for best results, however, a moist surface (no free standing water) is O.K.

Special/Safety Handling Requirements: The epoxy is non-flammable and does not require any special handling during mixing, placement or curing. Follow the manufacturer's standard written instructions for handling.

Estimated Construction Time: To be determined on each project.

Expected Service Life: 10 to 15 years

Estimated In-Place Initial Cost: \$60.00 to \$70.00 per Yd.^{2} (\$71.70 to \$83.70 M^2)

Basis of Payment: Normal contracting method

PHYSICAL PROPERTIES

Design Properties:

• Compressive Strength	@ 4 hrs.	min. (1,000 psi)	6.8 MPa
ASTM C579	@ 24 hrs.	min. (4,000 psi)	27.5 MPa
	Ultimate	min. (5,000 psi)	35.0 MPa
• Tensile Strength	@ 6 hrs.	- ? psi	
ASTM D638	@ 24 hrs.	- ? psi	
	@ Ultimate, min.	(2,000 psi)	14 MPa
• Tensile Bond Strength	@ 6 hrs.	- ? psi	
	@ 24 hrs.	- ? psi	

- **Shear Bond Strength ASTM C882, min.**

@ 6 hrs.	- ?	psi
@ 24 hrs.	- ?	psi
@ 7 days	(1,450 psi)	7 MPa

- **Modulus of Elasticity ASTM D638** — 4-8 x 10^2 MPa

- **Tensile Elongation ASTM D638** — 25-40%

- **Thermal Coefficient of Expansion ASTM D696** — 5-9 x 10^5 mm/mm/^{0}C

- **Shrinkage Strain**

@ 28 days	?	in/in
@ Ultimate	?	in/in

- **Properties Significantly Affected by Ultraviolet Light?** Yes __ No XX

Construction Properties:

- **Shelf Life :** One (1) Year

- **Pot Life :** 20 to 30 Minutes

- **Initial Set Time :** 4 Hours @ 77^{0}F

- **Approximately Elapsed Time to Open to Traffic:** 4 Hours or if colder temperatures during cure when the epoxy polymer concrete compressive strength exceed 6.8 MPa (1,000 psi).

- **Workability :** The system is easily applied by trained contractors or DOT maintenance crews.

Other Pertinent Properties: This system is used when decks or pavements have large quantity of cracking that are not going to be epoxy injected. Consult Thermal-Chem for specifications and crack treatment.

BRIDGE DECK REHABILITATION/OVERLAY PRODUCT SUMMARY SHEET

Name: THERMAL-CHEM MORTAR RESIN, PRODUCT NO. 3
A MICRO-THIN, 1/4 IN. TO 1 1/4 IN. THICK, MEDIUM MODULUS EPOXY POLYMER CONCRETE OVERLAY - PREMIXED/BROADCAST METHOD

Manufacturer:
Thermal-Chem Corporation
2550 Edgington Street
Franklin Park, IL 60131 USA
800/635-3773

Description: A medium modulus Epoxy Polymer Concrete (EPIC) Overlay applied in one lift as a premixed (thicker consistency than a slurry) EPIC. The premixed EPIC is then broadcast with a select gradation of aggregate to cause the premix to grow in height. Typical height growth is from 50 to 100 percent. The aggregate selected determines the final color and surface profile.

Typical Applications: Bridge decks, ramps, tollbooth lanes and slab-on-grade concrete pavements for vehicular traffic uses.

Advantages:

1. 21 years of exceptional service life on bridge decks and slab-on-grade concrete pavements. (See attached 15 Year Tracking Study Published By American Concrete Institute)
2. No offensive odors during placement
3. Safe, 100 % solids EPC mixture, Non-flammable
4. Prevents infiltration of chloride ions and water
5. Class A placeable and curable down to -18°C (0°F)
6. Improves skid resistance and ride quality
7. Quietness of ride
8. Minimal traffic interruptions during placement
9. Protects and extends concrete life-cycle
10. An initial cost effective, corrective and protective system
11. Lowest life-cycle cost available
12. Factory technicians available to assist during placement.

Compliance:
* ACI 548 Epoxy Polymer Concrete Premixed Overlays
* AASHTO-AGC-ARTBA, Task Force 34, Epoxy Polymer Concrete Overlays
* ASTM C881 Type III, Grade 1, Classes A, B and C

Limitations: Minimum temperature during application and cure for Classes B and C is 5° C (40° F) and Class A is -18°C (0°F). Do not apply to surfaces that are wet or have ice crystals.

Surface Preparation Requirements: Clean sound concrete. Typical methods of cleaning include sand blasting and shot blasting.

Typical Overlay Thickness: 6 to 31 mm (1/4 to 1 1/4 in.)

Typical Application Procedure: Placement is typically one lane wide per pass at the full desired thickness. A Static Screed is used to spread and establish the thickness of the EPC overlay. Immediately the aggregate is broadcast into the surface until only a dry layer of aggregate is on the surface. The excess aggregate is typically removed before traffic use.

Curing Requirements: The EPC overlay cures without the aid of any assistance. Typical cures rates are from 2 to 6 hours after placement depending on cure selection and temperature exposure during cure.

Weather Condition Constrains: No cover is required for curing. If rain or snow starts before tack-free cure stage develops cover overlay with plastic cover until tack-free stage is achieved.

Special/Safety Handling Requirements: None are required. The epoxy system is non-flammable and contains 100% solids. Very low odor when mixing and normally not detectable during placement.

Estimated Construction Time: To be determined on each project.
Expected Service Life: 20 to 30 years
Estimated In-Place Initial Cost: \$60.00 to \$70.00 per $yd.^2$ (\$71.70 to \$83.70 M^2)
Basis of Payment: Normal contracting methods

PHYSICAL PROPERTIES

Design Properties:

•	**Compressive Strength**	@ 3 hrs.	(2,000 psi)	13.8 MPa
	ASTM C579	@ 24 hrs.	(8,000 psi)	55.2 MPa
		Ultimate	(14,000 psi)	96.5 MPa
•	**Tensile Strength**	@ 6 hrs.	? psi	
	ASTM D638	@ 24 hrs.	? psi	
		@ 7 Days	(7,000 psi)	48.3 MPa
•	**Tensile Bond Strength**	@ 6 hrs.	? psi	
		@ 24 hrs.	? psi	

- **Shear Bond Strength** @ 6 hrs. ? /* -30 ^{0} C (5,000 psi) 34.5 MPa
  ASTM C 882 @ 24 hrs. ? /* + 30^0 C (4,800 psi) 33.0 MPa
  * 7 Day Cure

- **Modulus of Elasticity** 1.393×10^6 psi
  ASTM C 469

- **Tensile Elongation** 5 - 10 %
  ASTM D 638

- **Thermal Coefficient of Expansion** 8.879×10^{-6} in/in/^{0}F
  ASTM D 696

- **Shrinkage Strain** @ 28 days ? in/in
  @ Ultimate ? in/in

- Properties Significantly Affected by Ultraviolet Light? ? Yes ? No

Construction Properties:

- **Shelf Life:** 2 Years

- **Pot Life:** 20 to 30 Minutes

- **Initial Set Time:** Typical 3 Hours

- **Approximately Elapsed Time to Open to Traffic:** 3 to 4 Hours

- **Workability:** User friendly. Factory training available

Other Pertinent Properties: This system has been used on highways for more than 20 years. Its track record is excellent and provides the ultimate in wear surface for interstate traffic. Thermal-Chem works very close with the owners, engineers and contractor to make the project run as smooth as possible.

BRIDGE DECK REHABILITATION/OVERLAY
PRODUCT SUMMARY SHEET

Name: T-17X MMA POLYMER CONCRETE

Manufacturer: CASTEK INC.

Description: TOUGH, ELASTIC, WEAR SURFACE FOR SEALING AND RESTORING BRIDGE DECKS AND OTHER CONCRETE PAVEMENTS.

Typical Applications: USED AS WEAR SURFACES ON BRIDGE DECKS, RAMPS, SIDEWALKS, PARKING DECKS, LOADING DOCKS, WAREHOUSE FLOORING.

Advantages: WIDE TEMPERTURE APPLICATION RANGE, RAPID CURE, LIGHWEIGHT, HIGH SKID RESISTANCE, WATERPROOF, EXCELLENT BOND, HIGH UV AND WEAR RESISTANCE.

Limitations:

Surface Preparation Requirements: SURFACE CLEAN AND DRY OF DIRT, DUST, ETC., SHOTBLASTED, AND PRIMED.

Typical Overlay Thickness: 1/4" - 1/2"

Typical Application Procedure: SEE INFORMATION ATTACHED

Curing Requirements:

Weather Condition Constrains: CANNOT BE APPLIED IN THE RAIN OR ON WET SURFACES.

Special/Safety Handling Requirements: SEE M.S.D.S. ATTACHED

Estimated Construction Time:

Expected Service Life:

Estimated In-Place Initial Cost: DEPENDENT ON LABOR FOR PREPARATION.

Basis of Payment: SQUARE FT.

PHYSICAL PROPERTIES

Design Properties:

- Compressive Strength @ 6 hrs - 3,500 psi
  @ 24 hrs - 4,500 psi
  @ Ultimate - 4,500 - 5,500 psi
- Tensile Strength @ 6 hrs - ______ psi
  @ 24 hrs - ______ psi
  @ Ultimate - 500 psi
- Tensile Bond Strength @ 6 hrs - ______ psi
  @ 24 hrs - ______ psi
- Shear Bond Strength @ 6 hrs - ______ psi
  @ 24 hrs - ______ psi
- FLEXURAL Modulus of Elasticity - 250,000 - 550,000 psi
- Tensile Elongation - 100 %
- Thermal Coefficient of Expansion - $5.94X10^{-5}$ in/in/°F
- Shrinkage Strain @ 28 days - ______ in/in
  @ Ultimate - ______ in/in
- Properties Significantly Affected by Ultraviolet Light? ☐ Yes ☒ No

Construction Properties:

- Shelf Life - AT LEAST 12 MONTHS
- Pot Life - 24 MIN. @ 70 F
- TACK FREE ~~Initial Set~~ Time - 1 HOUR @ 70 F
- Approximate Elapsed Time to Open to Traffic - 1 1/2 - 2 HOURS DEPENDING ON TEMPERTURE.
- Workability - FLOWABLE WORKING TIME OF 15 - 20 MIN. DEPENDING ON TEMPERTURE.

Other Pertinent Properties: ______

BRIDGE DECK REHABILITATION/OVERLAY
PRODUCT SUMMARY SHEET

Name: T 48 EPOXY POLYMER CONCRETE

Manufacturer: CASTEK INC.

Description: PREMIUM QUALITY, TWO COMPONENT, EPOXY BASED. LOW MODULUS, ELASTIC WEAR SURFACE USED FOR RESTORING BRIDGE DECKS AND CONCRETE PAVEMENTS.

Typical Applications: USED AS WEAR SURFACES ON BRIDGE DECKS, RAMPS, SIDEWALKS, PARKING DECKS, LOADING DOCKS, AND WAREHOUSE FLOORING.

Advantages: HIGH FLEXIBILITY, MOISTURE INSENSITIVE, STRESS RELIEVING, EXCELLENT BONDING, HIGH EARLY STRENGTHS, EXTREMELY TOUGH, WATERPROOF, ADDED RESILENCE.

Limitations: 50 F MINIMUM APPLICATION TEMPERTURE

Surface Preparation Requirements: SURFACES MUST BE CLEAN AND DRY OF DIRT, DUST ETC. ALL DAMAGED & DETERIORATED SURFACES MUST BE REMOVED, SHOTBLASTED, PRIMED.

Typical Overlay Thickness: 1/4" - 1/2"

Typical Application Procedure: SEE INFORMATION ATTACHED

Curing Requirements:

Weather Condition Constrains: CANNOT BE APPLIED IN THE RAIN OR ON WET SURFACES.

Special/Safety Handling Requirements: SEE MS.D.S. ATTACHED

Estimated Construction Time:

Expected Service Life:

Estimated In-Place Initial Cost: DEPENDENT ON LABOR FOR PREPARATION

Basis of Payment: SQUARE FT.

PHYSICAL PROPERTIES

Design Properties:

- Compressive Strength @ 6 hrs - ________ psi
  @ 24 hrs - ________ psi
  @ Ultimate - 5,000 psi (MIN.)
- Tensile Strength @ 6 hrs - ________ psi
  @ 24 hrs - ________ psi
  @ Ultimate - 1,500 psi
- Tensile Bond Strength @ 6 hrs - ________ psi
  @ 24 hrs - ________ psi
- Shear Bond Strength @ 6 hrs - ________ psi
  @ 24 hrs - ________ psi
- FLEXURAL Modulus of Elasticity - 500,000 - 600, 000 psi
- Tensile Elongation - 50- 70 %
- Thermal Coefficient of Expansion - ________ in/in/°F
- Shrinkage Strain @ 28 days - ________ in/in
  @ Ultimate - ________ in/in
- Properties Significantly Affected by Ultraviolet Light? ☐ Yes ☒ No

Construction Properties:

- Shelf Life - 1 YEAR
- Pot Life - 15 - 30 MIN. @ 70 F
- Initial Set Time - ________
- Approximate Elapsed Time to Open to Traffic - 3 HRS @ 70 - 100F, DEPENDS ON TEMPERTURE.
- Workability - ________

____

Other Pertinent Properties: ________

____

APPENDIX C

Thermal-Chem Thin Polymer Concrete Overlay Proposal

Since 1976

thermal-chem ®

June 4, 1998

Dr. G.E. Ramey
Feagin Professor of Civil Engineering
Auburn University
Highway Research Center
Department of Civil Engineering
238 Harbert Engineering Center
Auburn University, Alabama 36849-5337

Dear Dr. Ramey:

The proposal for the State of Alabama Highway Department, I-65 Bridge Deck Repair is attached. After you read it, please call me if you have any questions.

I would like to recommend a meeting be set up with the Highway officials, so that we can plan the first installation. Our company has given this project considerable time and has spent a large sum of money on testing, to verify our belief that the bridge deck problems can be solved and extend the structure life cycle at a considerable cost savings. The constructions time will also be greatly reduced, so as not to inconvenience travelers on this vital bridge link on the Interstate Highway System in Birmingham.

Sincerely,
THERMAL-CHEM CORPORATION

F Dimmick

Floyd Dimmick, Sr.
Director of Technical Sales

Attachments: Figure No. 1
Figure No. 2
Bridge Deck Strengthening and Wear Surface Proposal

BRIDGE DECK REPAIR, STRENGTHENING AND WEAR SURFACE EPOXY POLYMER CONCRETE OVERLAYMENT PROPOSAL

Project: State of Alabama Highway Department, I-65 Bridge Deck over R.R. Viaduct Between 2nd Avenue South and 1st Avenue North, Birmingham, (Jefferson County) Alabama.

General Background

At the request of G. Ed Ramey, Feagin Professor of Civil Engineering, Auburn University, Alabama; a meeting with William F. Conway, P.E. Bridge Engineer, Alabama Department of Transportation, and eleven other department engineers and supervising personnel; and two visual project surveys of the structures, one from the under side of the deck and two from the top size of the deck were made; a review of a previous report; review of the deck plans; laboratory testing and evaluation of cores taken from the deck; and a overall evaluation of the deck condition and driving surface has resulted in the following proposal, which includes repair recommendations and analysis of possible repair options.

The project study was directed by Floyd Dimmick Sr., Director of Technical Sales. John Carlin, Technical Director, and William Dimmick, Senior Technical Field Technician, also with the Thermal-Chem Corporation, located in Franklin Park, Illinois, whom each assisted in testing of the concrete cores and testing and placement of the proposed systems. Floyd Dimmick Sr. may be reached by telephone at 800/635-3773.

Description of the Structure

The bridge structure is approximately 28 years old and the original wear surface is being used as of the date of this report. The length of the deck is 1,666 feet with a maximum span of 105 feet and deck width of 148 feet. Presently, two-way traffic is using the structure with 10 lanes and 2 shoulders being divided by a concrete wall in the center. The concrete deck and continuous multi-steel girder structure is a non-composite design. The average deck thickness as measured from the cores, removed from the deck for testing is six and one half (6.5) inches. The reinforcement steel used as part of the concrete deck design is located in the lower portion of the deck, from the bottom of the deck 25% upward to the 50% (half-way) area in the concrete.

Bridge Deck Condition

General: The overall condition of the deck is considered to be rapidly deteriorating, as explained herein.

Cracks: At different locations on the upper surface of the deck and on the under side of the deck cracks are visible in map patterns. Transverse cracks are randomly spaced and longitudinal cracks follow steel beams in some areas on the upper surface. The visual survey of the deck and testing of the removed concrete cores from the deck supports the report from Auburn University, stating that the cracks are growing in width annually. The cracks present in the cores taken from the deck also exhibit crack growth by being wider at the driving surface area and narrowing as they penetrate deeper into the concrete. Only 2 cracks penetrated the full depth of the concrete cores, and the balance of the cracks stopped at the reinforcement steel depth. Two of the five cores had separation of the cement paste around the reinforcement steel. No notable amount of steel deterioration was observed from the cores. The cores exhibited cracks that could have been caused from shrinkage at the time of originally curing the concrete.

However, the field survey showed more than the normal quantity of cracks on the upper surface (wear surface area) of the concrete. The majority of these cracks does not seem to penetrate the full depth of the deck.

Conclusion About The Cracks: The cracks are growing in width and depth, because of concrete corrosion within the crack void and the flexural movements of the deck. The upper surface of most of the crack is sealed with normal dirt and debris caused by the vehicular traffic using the deck and normal environmental conditions. The non-waterproofing seal (dirt) seems to be deposited in the upper portion of the crack about 1/8 inch in depth on the cores tested. The broken cores showed the cement is being dissolved and some release of sand particles on the crack walls. This action of concrete corrosion is caused by the water penetrating into the crack void. Chlorides can also be carried by the water and they will speed up the deterioration process when the crack void is kept in a moist or wet state. These conditions and excessive flexing of the deck account for the enlargement of the crack width. Typically, when crack width develops, crack depth will also develop. The field survey clearly showed that when heavy loaded vehicles, such as trucks and buses, drive over the surface the flexure of the deck could be observed by looking at the under side of the deck movement. A few of the cracks observed from the under side of the deck exhibited uneven movement of the adjacent concrete on each side of the crack void. This could account for the reinforcement steel being to close to the bottom of the concrete mass or a delamination within the deck closer to the underside of the deck. This uneven crack movement is not the typical condition observed during the field survey. These cracks and possible delaminations left untreated will continue to grow and cause further deck deterioration.

Wear Surface of Deck

The wearing surface profile of the deck is polished and worn down approximately 1/4 inch in the driving track areas of each lane and some spalling has developed that will require some partial depth patching. The entire deck area on the upper surface and under surface on a continuous span, including the concrete next to the expansion joints needs to be sounded, to determine where patching is needed and delaminations are located. At the time of the inspection, very few areas were noticed that needed to be patched in the inspected area.

Concrete Core Testing

A study was conducted on six of the seven concrete cores submitted to the Thermal-Chem Laboratory. One core was broken and taped together and was not testable.

Core No. 1 One surface crack penetrated full depth of core. Delamination void above the reinforcement steel at 50% depth of concrete. One mat of steel. Concrete depth 7 1/4 inches.

Core No. 2 Two cracks on surface of core. One crack penetrated full depth of core. One crack penetrated 1 inch deep and stopped. Delamination void just starting at 50% depth of concrete, same as core No. 1. Two reinforcement steel bars at center area of the concrete and one bar located at the bottom of the core with one inch cover. No delamination at lower bars. One vertical crack from the underside of the deck is growing upward to a depth of 2 1/4 inches. Concrete depth 7 1/4 inches.

Core No. 3 This core was received broken and no test were conducted on the core. It had two cracks on the upper surface area that were both filled with surface dirt. It seems that only one crack has penetrated the full depth of the core. The depth of dirt has penetrated 1 1/4 inches into the crack. There is no corrosion on the steel, therefore, the crack at the steel level was most likely caused during the coring process. The concrete has two rows of steel. The upper mat has 2 1/8 inches of cover and the lower mat 1/2 inch. Concrete depth 6 inches.

Core No. 4 Two hair line cracks on the surface. One penetrates 7/8 inch deep and the second crack full depth. There is no steel in this core. Concrete depth 6 1/2 inches.

Core No. 5 It seems that surface cracks are just starting or the concrete was damaged during the coring process. A delamination crack is present running horizontal 1 1/2 to 2 inches above the lower concrete surface. The core was broke away from a mat of steel at the same level; therefore, I question if this is really a delamination or damaged during the coring process. There is no steel in this core. Concrete depth 6 1/2 inches.

Core No. 6 This core has no cracks or reinforcement steel. Concrete depth 6 inches.

Core No. 7 This core has no cracks or reinforcement steel. Concrete depth 6 inches.

Compressive Strength: The testing averaged 3,600 psi. The breaks on cores 2, 6 and 7 showed typical yielding with no noticeable irregular fracturing. The test method used was ASTM C39.

Adhesion Testing: Two types of adhesion testing were used. A Shear Method and a Tensile Pulling Method (ACI 503 R).

Core Number	**Test Method**	**Results**
2	Shear	100% Concrete Failure
7	Shear	100% Concrete Failure
4	Tensile	100% Concrete Failure, 230 psi *
5	Tensile	100% Concrete Failure, 240 psi *

* **Note:** Field values are typically higher, because of more control of working with a larger mass of concrete.

Testing Conclusions

1. The cleaning of the wear surface of the concrete is possible with standard industry abrasive blasting methods. The depth of surface preparation removal using the cores as a guide will range from 1/16 to 1/8 inch.

2. The cracks will accept a two-component epoxy adhesive and the bonding ability of the polymer will equal the concrete strength value and provide future waterproofing of the welded crack area.

3. The compressive strength of the concrete exceeds the original design of 3,000 psi. By welding the cracks together, the original monolithic concrete design will be achieved and possibility exceeded.

4. The adhesion strength of the Epoxy Polymer Concrete Overlay and Epoxy Crack Welding Polymers, will create a monolithic bond to the concrete.

5. The concrete deck can be repaired, preserved and strengthen to handle the increased amounts of future vehicular traffic volumes, as understood at the time of writing this report.

Bridge Deck Repair and Overlayment Recommendations

General: We believe the bridge decks can be repaired, strengthened and overlayed with minimal downtime at a reasonable repair cost and that the extended life cycle of the decks will warrant the repair costs. Our philosophy is based on proven systems that are being combined together to satisfy the repair and strengthening requirements of these structures. The performance of the individual systems is well established. The only unknown factor, is, can the underside epoxy injection work be eliminated? Therefore, we recommend one span with the complete repair recommendations and another span without the underside epoxy injection process.

The recommendations herein are all to be combined together to make one complete system. The system is broken into parts to further explain why each approach has been selected. To properly evaluate the system, it must be applied over the entire length of one continuous span of deck and by its full width. We also recommend on the first project that the contractor selected be a qualified experienced contractor in the placement of the polymer concrete's and epoxy injection.

The consideration of deck downtime has been carefully examined because of the heavy traffic volumes over this structure and has been factored into the overall repair plan. If the plan is accepted, Thermal-Chem will assist by writing the work specification and have a technician on the job to assist the owner and contractor.

Surface Preparation of Upper Concrete Deck Surface

The surface will be abrasive blasted with a dust free steel shot method to remove all necessary contaminate concrete and debris to open all crack voids and to provide a clean surface profile that will provide 100% concrete failure when tested according to ACI 503 R Tensile Pull Method.

Crack and Delamination Repair

General: The crack and delamination repairs will be divided into two types of product placement to weld the cracks and voids together. The combination of methods should provide between 90 and 100% filling of the cracks and delaminations. This combination of crack and delamination repairs will reduce the overall cost of welding the deck into a monolithic unit as originally designed. Depending on the actual area of deck that is used for the first repair method, it may be necessary to saw cut control joints in the deck that will be filled with an epoxy flexible system that has load bearing capability. It was noted during the visual survey that some areas on the upper deck surface had longitudinal cracks and were apparently running parallel to the steel beams. If this is the result of the overall design of the structure, it is advisable to allow these cracks to work as control joints and keep them properly sealed from water and chlorides. The control joints should be sealed with Thermal-Chem FlexGard J, Product No. 108.

Low Pressure Injection Method

The cracks that travel vertical through the concrete deck and the delamination voids that travel horizontal to the lower concrete surface should be welded together with a low viscosity epoxy injection resin, Thermal-Chem Injection Resin, Product No. 2 from the under side of the concrete deck. The surface sealing of the open cracks and entry ports on the lower surface of the concrete deck will be with Thermal-Chem Bonder, Product No. 4, a non-sag epoxy adhesive. Holes will be drilled into the concrete with a vacuum drilling process to intersect the crack that has debris in the surface area of the crack void or to intersect the delamination void. On clean cracks with no debris in the void, a surface tee may be used as the entry port. The epoxy resin should be pumped into the concrete from only the lower side of the deck at 14 psi to avoid longer traffic interruptions. The epoxy resin will travel upward in the crack void filling any fissures that are connected to the main crack void. (See Figures 1 and 2.)

Gravity Filling

After the upper surface of the concrete deck has been cleaned, prepared and the crack voids are opened, a super low viscosity, 100% solid, epoxy shall be poured onto the clean surface and manually worked into the crack voids to assist in its penetration. After the cracks cannot accept anymore epoxy, the excess uncured epoxy shall be squeegeed from the deck surface. In cracks that continue to allow penetration of more epoxy, they shall be retreated with epoxy until rejection occurs. The gravity sealing epoxy shall be Thermal-Chem Hairline Crack Sealer, Product No. 207.

Concrete Deck Overlayment

The overlay strengthening composite design has been carefully analyzed to address possible future cracking or cracks that did not accept full repair and the existing and potential traffic volume increases. The overlay composite will water and chemical proof the wear surface area of the concrete deck. It will provide a new driving surface that will provide bald tire skid numbers between 50 and 70 depending on the actual aggregate selected by the owner.

The overlay composite is available in two designs:

System No. One: System No. One is for vehicular traffic volumes up to 25,000 vehicles per day and System Two is for higher traffic volumes. System No. One is also used on larger traffic volume decks with the understanding that it can extend the life of a deck until future expansion plans can be implemented.

System One consists of a single lift of a low modulus Epoxy Polymer Concrete with an epoxy glass fiber grid embedded next to the concrete surface. (See Figure 1.)

System No. Two: System No. Two consists of adding an additional medium modulus Epoxy Polymer Concrete over System No. 1 to provide longer service and wear for the higher traffic volumes. (See Figure 2.)

These systems have been designed for these individual projects with excessive cracking, high flexure movements and their existing wear surface conditions by using existing products in the Thermal-Chem Product Line. The idea of reinforcing the epoxy polymer concrete is not new. It has been used on vibrating industrial floors and elevated decks for over three decades. Asphalt reinforced concrete systems have been used since the early 1960's. The initial forms of reinforcing were with wire mesh and expanded metal. These forms of reinforcement had difficulties in laying the mesh flat and were unreliable in overall performance, because of the flattening problems But, the overall important fact learned was that a reinforcement could provide additional resistance to reflection cracks and add strengths in thin mass. Glass reinforcement has replaced metals with superior performance.

The introduction of Epoxy Polymer Concrete with a fiber/glass grid solved most crack reflection problems for industry uses. This same technology can be used for the transportation industry. Our recommendations are based on many months of trail combinations to accommodate traffic volumes of approximately 70,000 vehicles per day on this bridge deck. The recommended Epoxy Polymer Concrete's are our existing products that we have been selling to the transportation industry for over two decades. The only difference is the change of the epoxy fiber/glass grid selection. We chose an appropriate one inch grid opening that complimented the overlay requirements for this project. Our study involved the testing of the epoxy and glass/fiber grid in different combinations so as not to stiffen the overlay beyond the adhesion compatibility of the Epoxy Polymer Concrete to the portland cement concrete when heavy loads are driven over the bridge deck. Another test was to control crack propagation from the portland cement concrete into the Epoxy Polymer Concrete. If the crack should develop, the mode of failure is designed to stop the vertical crack growth at the epoxy glass/fiber area. This portion of the overlay composite will prevent normal cracks from reflecting through the entire thickness of the overlay. This is accomplished by the higher tensile elongation of the epoxy that embeds the epoxy glass/grid and controls the epoxy glass/grid tensile strength. The low modulus epoxy binder takes up adhesion stresses at the bond line to the portland cement concrete surface. It's not only a crack protection material, but also acts as a stress relief material when quick thermal changes take place such as rain or a freezing condition that suddenly drops the temperature by 15^0 F or more in a short period of time. We believe we have designed an Epoxy Polymer Concrete Glass/Fiber Grid Composite that is less stiffer than used in other industries that will increase the life of the overlay in resisting reflection cracking and provide a safe driving surface for Interstate traffic. The Thermal-Chem Epoxy Polymer Concrete Glass/Fiber Grid Composite Overlay also provides reinforcement as if a mat of number 6 rebar, spaced at 8 inches on center was added to the deck with proper concrete cover. Its purpose is to strengthen the concrete element by reducing stress in the primary reinforcement and improving live load capability. We also believe that any non-reinforced overlay on this structure will have reflective cracking and result in a sufficient shorter life cycle than expected.

The products used in the Epoxy Polymer Concrete Glass/Fiber Grid Overlay Composites are:

System No. 1

Base and Wear Course: Thermal-Chem FlexGard T, Product No. 309 is the Epoxy Polymer Concrete binder.
Selected silica sands and larger aggregates are the concrete fillers.
Thermal-Chem Epoxy Glass/Fiber Grid, Product No. 8600 is the concrete reinforcement.
Approximate overlay thickness is 3/8 inch.
The surface color is the same as the largest aggregate selected.

System No. 2

Wear Surface: Thermal-Chem Mortar Resin, Product No. .3 is the Epoxy Polymer Concrete binder.
Selected silica sands and larger aggregate are the concrete fillers.
Approximately 1/4 inch thick.
Surface color is the same as the selected largest aggregate.

Base Course: Thermal-Chem FlexGard T, Product No. 309 is the Epoxy Polymer Concrete binder.
Selected silica sands and larger aggregate are the concrete fillers.
Epoxy Glass/Fiber Grid No. 8600 is the concrete reinforcement.
Approximate 3/8 inch thick.

System Thickness: Approximate total overlay thickness is 1/2 inch plus. The 1/4 inch wear surface covers the Base Course and fills in between the raised aggregate of the Base Course.

Summary

We believe that the above repair program is a suitable strengthening reinforcement system and overlay for this concrete bridge deck. From the tests completed so far, it appears that the durability and fatigue behaviors are the characteristics that will protect this bridge deck and extend the useable life of the existing deck with minimum traffic interruption's during rehabilitation.

We believe the next step in this repair program is to proceed promptly with an engineering meeting with all persons involved in the decision making process for this bridge repair program. Thermal-Chem will travel to Alabama for this meeting and present additional data from the testing program. If our system is acceptable to the Alabama DOT, we will assist immediately in writing the material and placement specification, so that the first project could be placed this year before Winter. We will also make available a factory trained technician to assist the engineering staff and contractor during placement of the project. Winter evaluation is important, because of the large quantity of cracks and delamination in this bridge deck.

Why Use This Method?

1. Ease of Application.

2. Reduces Traffic Interruptions.

3. Requires Sufficiently Less Construction Time.

4. Increases Strength:

- Flexural
- Shear
- Compressive
- Fatigue Enhancement
- High Strength to Thickness Ratio

5. Structural Upgrade by Enhancing the Performance of Under-Reinforced Concrete Elements.

6. Improves Load Capability.

7. Lightweight.

8. Control of Crack Propagation.

9. Proven High Performance Track Record on Surface Wear Ability.

10. Greater Durability.

11. Non-Corrosive Environment.

12. Aesthetically Appealing.

13. Ease of Maintenance.

14. Cost Savings Today and in the Future.

Cost Considerations

1. Estimates are based on approximately 15,000 ft^2 of surface area in one location.

2. Estimates do not include traffic control costs.

3. Estimates are believed to be higher, because of contractors considering the project as a demo type application. Larger quantities and second time application experiences with the grid reinforcement, typically reduce overall future costs.

4. Saw-cutting and control joint filling with an epoxy flexible sealant are not included, because the deck area of placement is unknown.

5. Patching holes on the deck surface with an epoxy polymer concrete are not included, because the deck area of placement is unknown and has not been sounded.

6. Repainting of traffic lines or other surface markings has not been included.

7. The underside low pressure (14 psi) epoxy crack injection cost, based on 200 L. Ft. or greater quantity should range between $30.00 and $45.00 per L. Ft. The wide range of unit costs will depend on the type of lift required for the repair area selected.

8. The underside low pressure (14 psi) epoxy delamination injection cost, based on 500 ft^2 or greater quantity should range between $3.75 and $5.25 per square foot. If a crack runs through the delamination area, no additional cost is charged as a crack repair. The wide range of unit costs will depend on the type of lift required for the repair area selected.

9. **System No. 1** Thermal-Chem FlexGard T, Product No. 309, an Epoxy Polymer Concrete Overlay with Glass/Fiber Grid and Hairline Surface Crack treatment, as shown on Figure 1 includes surface preparation material and placement costs.

   Estimated Range $10.00 to $13.00 per square foot.

10. **System No. 2** System No. 1 as stated above with an additional wear course of Thermal-Chem Mortar Resin, Product No. 3, an Epoxy Polymer Concrete.

    Estimated Range $15.00 to $18.00 per square foot.

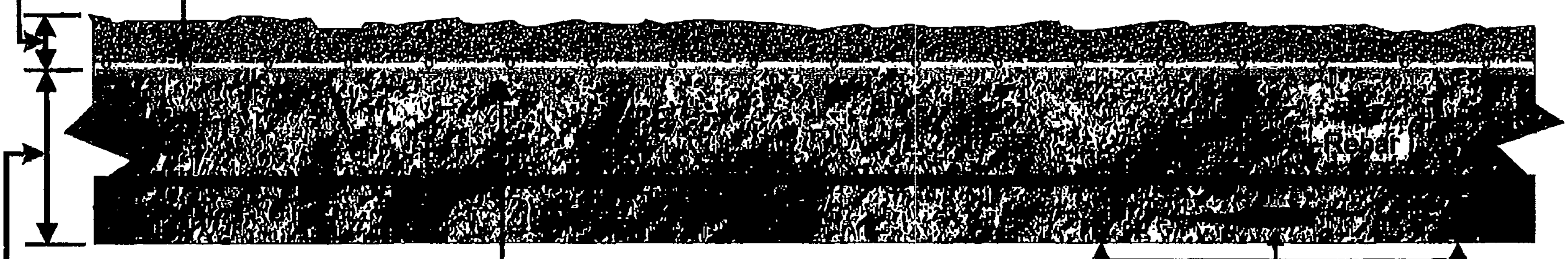
11. These cost estimates assume that all work items will be performed in one project under the direction of a single contractor.

Thermal-Chem Epoxy Polymer Concrete Composite Overlay Designed for Alabama Highway Department Bridges On Interstate I-65 and I-59, Birmingham, Alabama

System No. 1 For Less Than 25,000 Vehicles / Day

FlexGard T Epoxy Polymer Concrete, Product No. 309

Strengthening Epoxy / Glass Grid



Concrete Deck
6 to 7 inches Thick

Partial Depth cracks on deck surface welded together with Thermal-Chem Hairline Crack Sealer, Product No. 207

Cracks and Delaminations Welded Together from Underside of Deck with Thermal-Chem Injection Resin, Product No. 2 Low Pressure System (14 psi)

Typical Overlay Thickness is 3/8 in. plus

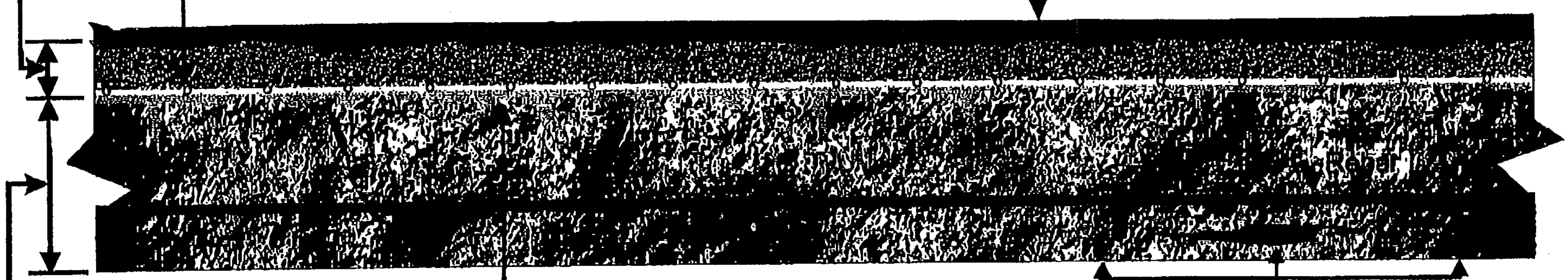
thermal-chem.		Thermal-Chem Corporation 2550 Edgington Street Franklin Park, IL 60131 (800)635-3773
Title: ADOT Bridge System #1		
Date: 05/29/98	By: FED/ljh	DRWG No. SYS01

Figure 1

No Scale

Thermal-Chem Epoxy Polymer Concrete Composite Overlay Designed for Alabama Highway Department Bridges On Interstate I-65 and I-59, Birmingham, Alabama

System No. 2 For Greater Than 25,000 Vehicles / Day



Typical Overlay Thickness Is 1/2 In. plus

Figure 2

NO SCALE

thermal-chem.

Thermal-Chem Corporaton
2550 Edgington Street
Franklin Park, IL 60131
(800)635-3773

Title: ADOT Bridge System #2

Date:	By:	DRWG No.
05/29/98	FED/ljh	SYS02

APPENDIX D

GADOT Select Special Contract Provisions
for
Staged and Rapid Construction of Bridge Deck
Structural Overlays

STAGED AND RAPID CONSTRUCTION OF STRUCTURAL OVERLAYS ON I-285 AND I-85 BRIDGE DECKS NEAR DOWNTOWN ATLANTA, GEORGIA UNDER CONCURRENT TRAFFIC CONDITIONS

Georgia Department of Transportation

Select Special Contract Provisions

SECTION 108 - PROSECUTION AND PROGRESS

108.07 DETERMINATION OF CONTRACT TIME: Retain as written except as follows:

For this Contract, a Completion Date of December 31, 1997 is established for the overall completion of the work.

In order to minimize the disruption of normal traffic flow, a separate completion time is specified for those portions of the work that require closing of lanes as specified in the Special Provision - Sequence of Operations under Section V Special Conditions.

Failure to reopen the lanes by the times specified in Section V.- J. of the Sequence of Operations for each bridge site will result in the assessment of liquidated damages at a rate of $24,000 per day. Said damages will be assessed on an hourly basis of $1,000 per hour or portion of an hour thereof.

This rate is in addition to liquidated damages which may be assessed in accordance with Sub-Section 108.08 for failure to complete the overall project.

CONCRETE REMOVAL (HYDRODEMOLITION METHOD)

I. GENERAL:

A. This section specifies hydrodemolishing equipment for the removal of concrete bridge areas as indicated in the plans. This equipment shall be capable of removing deteriorated or non-deteriorated concrete and cleaning the existing reinforcing steel of all rust and corrosion products by use of high-velocity water jets acting under continuous automatic control.

The concrete shall be removed as detailed on the Plans and to a depth such that the concrete overlay minimum thickness of 2 inches is achieved. High areas shall be removed to acceptable tolerances before paving operations begin and shall be removed at no additional expense to the Department.

II. EQUIPMENT:

A. The hydrodemolishing equipment shall consist of filtering and pumping units operating in conjunction with a remote-controlled robotics device.

B. The equipment must operate at a noise level of less than 90 decibels at a distance of 50 feet from either the powerpac unit or the remote robot.

C. The equipment must be capable of working 24 hours per day.

D. The equipment must be capable of using river, stream or lake water when it is practical to avoid the waste of potable water.

III. CONSTRUCTION:

A. All deteriorated concrete shall be removed, and all exposed reinforcing steel shall be cleaned of all rust and corrosive products including oil, dirt, concrete fragments, laitance, loose scale and other coating of any character that would destroy or inhibit the bond with the new concrete.

B. The Contractor shall take all steps necessary to prevent cutting or otherwise damaging reinforcing steel, including any vertical stirrups, and/or structural steel including welded shear connectors projecting into the slab and designated to remain in place. Any such bars or shear connectors damaged by the Contractor's operations shall be repaired or replaced at the Contractor's expense.

C. Areas of the deck not accessible or otherwise convenient to hydrodemolition operations shall be treated with conventional (jack-hammer) removal methods. Such removal may be performed by power chipping or hand tools, except that pneumatic hammers heavier than 15 lb. class (nominal) (30 lb. maximum) will not be permitted. Pneumatic hammers and chipping tools shall not be operated at an angle exceeding 60 degrees relative to the surface of the deck slab. Such tools may be started in the vertical position but must be immediately tilted to a 60 degree operation angle. Pneumatic hammers heavier than 15 lb. class (nominal) will not be permitted for removal in areas directly below the top longitudinal reinforcing steel or around primary girder reinforcing steel.

IV. OPERATION OF THE HYDRODEMOLITION EQUIPMENT:

A. Once the operating parameters of the Hydrodemolisher are defined by programming and calibration, they shall not be changed as the machine progresses across the bridge deck or deck unit, in order to prevent the unnecessary removal of sound concrete below the required minimum removal depth. The Contractor shall exercise care to avoid removal of sound concrete below the required depth.

B. Operation of the Hydrodemolition equipment shall be performed by and supervised by qualified personnel.

C. The Contractor shall submit to the Engineer, for approval, a plan to contain and remove the water and residue caused by the Hydrodemolition operation. If satisfactory containment and removal of the runoff water or residue is not being accomplished, the Contractor shall discontinue operations and submit a revised plan for review and approval by the Engineer. Work shall not resume until a plan has been approved.

The Contractor shall provide for the disposal of runoff water and residue generated by the Hydrodemolition operation. The Contractor shall obtain any required permits and shall comply with applicable regulations concerning such water disposal and residue disposal.

The Contractor shall make provision for the safe handling of runoff water insofar as it may constitute a hazard on the adjacent or underlying traveled roadway surface. If any damage occurs to existing berm slopes from scouring water jet or runoff water, all damages shall be repaired at no additional expense to the Department.

D. The Contractor shall provide adequate lighting as required to allow for the safe conduct of night time removal operations, and shall obtain the Engineer's approval for same, exercising care to avoid any hazardous glare in the direction of oncoming traffic.

E. The Contractor shall maintain, on the job site, an inventory of common wear parts and replacement accessories for the equipment adequate to assure that routine maintenance tasks can be performed readily without undue project delay.

F. Removal of concrete debris shall be accomplished by hand or by mechanical means, and shall be accomplished directly following the Hydrodemolition process, to prevent the debris from re-setting or re-adhering to the surface or remaining sound concrete. Any removal debris which is allowed to re-settle or to re-adhere to the surface of sound concrete shall be carefully removed by the Contractor, and the Contractor shall exercise care to avoid any damage to the remaining sound concrete.

G. The Contractor shall take all necessary precautions to prevent any blow through of the bridge deck from occurring. In the event that this occurs, all removal or cleaning operations shall cease until the Contractor's procedures and/or the bridge deck has been corrected to the satisfaction of the Engineer.

H. The Contractor shall provide protective platforms over areas of vehicular traffic when under portions of existing bridges where hydrodemolition takes place. See the Plans and Specifications for additional requirements.

V. MEASUREMENT AND PAYMENT:

A. Payments for deck removal shall be included in Item No. 519 Concrete Overlay.

B. The above payments shall be full compensation for all work, equipment, materials, and incidentals required to complete each item including the furnishing and the appropriate handling of water, jack hammer work required along parapet curbs and other locations, and the required cleanup work, all as required to complete the items.

SECTION 519- FAST TRACK BRIDGE DECK CONVENTIONAL CONCRETE OVERLAYS

519.01 DESCRIPTION: This work consists of the construction of a Portland Cement concrete overlay of an existing concrete bridge deck and shall include the furnishing of all material, labor and equipment necessary to prepare and finish the work in accordance with these Specifications and Plan details.

519.02 MATERIALS: All materials shall meet the requirements of the Standard Specifications.

Concrete Mix Requirements

Type III Cement Minimum (lbs/cu. yd.)	750
Coarse Aggregate Size No.	7
Maximum Water/Cement Ratio	.40
Slump Limits (Jobsite) Maximum	7 inches
Air Acceptance Limits (Jobsite)	3.5 to 7.5%
Admixture (Required)	Type F or G
Minimum Compressive Strength (Jobsite) @ 24 hours	3000 psi

Temperature Limitations

Concrete Placement Temperature (Range)	70° - 90°F
Air Temperature (Range)	50° - 90°F

The contractor shall submit a mix design for approval. This submittal shall include the actual quantity of each ingredient and laboratory designs which demonstrates the ability of this design to attain a compressive strength of 3500 psi at 24 hours. Laboratory acceptance strengths shall be determined by at least eight compressive test specimens prepared and cured in accordance with AASHTO T-126. The specimens shall be made from two or more separate batches with an equal number of cylinders made from each batch.

519.03 CONSTRUCTION: The construction procedures for the overlay shall be as follows:

A. Concrete Manufacturing: Concrete for Portland cement concrete overlays shall be manufactured in accordance with section 500.05 of the Standard Specifications.

B. The existing deck to be overlaid shall be given a machine preparation consisting of removal of the concrete to a minimum depth of 0.5 inch below the top mat of reinforcement or as shown in the Plans. This operation shall be accomplished by use of high pressure water blasting equipment designed specifically for this purpose.

C. After removal of the concrete, the entire area of the deck surface shall be blast cleaned to a bright, clean appearance which is free from dust, dirt, oil, grease and all foreign matter. The blast cleaning will be performed by high pressure water blasting. All debris of any type, including dirty water, resulting from the blast cleaning operation shall be immediately and thoroughly cleaned from the blast cleaned surface. The blast cleaned areas shall be protected, as necessary, against contamination prior to placement of the overlay. Contaminated areas shall be blast cleaned again as directed by the Engineer, at the Contractor's expense. All areas of standing water shall be removed and the surface reasonably dry before the concreting operation begins.

D. The minimum overlay thickness is to be two inches with a minimum of 1½ inches of concrete cover over the top reinforcement mat or as specified in the plans.

   The finishing machine or approved screeding device shall be passed over the existing deck prior to placing the concrete overlay in order that measurements can be made to insure that proper overlay thickness and steel cover will be achieved. Screeds shall be equipped with surface vibrators sufficient to thoroughly consolidate the overlay full depth, unless other methods are approved by the Engineer. Consolidation using hand-held vibrators shall be required when placing the mixture around steel reinforcement or structural members.

   The overlay shall satisfy the surface tolerances as found in Article 500.11.C.6.e of the Standard Specifications except as noted on the Plans. After finishing, the surface shall be textured in accordance with the requirements of Section 500.11.C.6.c(l) or (2) or as required by the Plans and Proposal. Surface grooving shall not begin until the curing period specified herein has expired.

The formation of longitudinal joints and transverse construction joints shall be held to the minimum number necessary, and both type of joints shall be thoroughly blast cleaned and coated with an approved bonding agent before fresh concrete is placed against the hardened sides of the joints. When longitudinal construction joints are necessary, they shall be formed by use of a header secured to the deck. After removal of the longitudinal header and transition (p.5, sequence of Operations Special Provision), the overlay shall be sawed three inches or more inside the construction joints and the overlay outside the saw cut removed before the adjacent overlay is placed. The volume of the overlay removed will not be included in the volume measured for payment.

E. Curing of the overlay shall begin immediately after the water sheen disappears and the surface finish is applied. A film of water shall be kept on the surface by fogging until covering materials are in place. Curing covers shall be applied as soon as the concrete has set sufficiently to prevent marring of the surface. Curing material shall consist of two layers of wet burlap and at least one layer of plastic sheeting conforming to the requirements of AASHTO M-171. Adjacent sheets of curing covers shall be lapped a minimum of six inches. Sheeting materials that become torn, broken or damaged shall be immediately replaced. Provisions shall be made for additional applications of water under the plastic sheeting. This may be accomplished with soaker hoses or other methods approved by the Engineer. In any event, the overlay surface and burlap material shall remain wet throughout the curing period. Curing shall continue for a minimum of 24 hours.

Weather Limitations:

Placement of the overlay shall be performed during favorable weather conditions. When the atmospheric temperature is expected to exceed 90 degrees F. during daylight hours, the Contractor shall schedule work hours that insure complete placement by 10:30 a.m. Placement shall not be scheduled when wind velocity is expected to exceed 20 mph or when rain is expected. The minimum acceptable temperature of concrete at the point of delivery shall be 70 degrees F. Overlay concrete shall be kept at a temperature above 70 degrees F. for at least 24 hours after placement.

519.04 MEASUREMENT: The concrete overlay shall be measured for payment by the square yard of existing deck surface to be overlayed, complete in place and accepted, and shall include all materials and labor to remove the existing concrete, clean and prepare the deck surface, place and finish the overlay.

519.05 PAYMENT: The concrete overlay measured as specified above will be paid for at the Contract Unit price bid per square yard. Such payment shall be full compensation for furnishing all equipment and materials and performing the work in accordance with the Plans and Specifications.

Payment will be made under:

Item No. 519. Concrete Overlay Variable
Thickness.. per Square Yard

SEQUENCE OF OPERATIONS

I. <u>General</u>

A. The purpose of this Special Provision is to provide a Sequence of Operations for the construction of this project. This Special Provision also provides specific procedures that will permit vehicular and pedestrian traffic to pass through and around the project area safely with a minimum of inconvenience.

B. This Special Provision sets forth specific procedures and does not relieve the Contractor of any responsibilities required by the Specification Section 150, other Specifications, Plans, or the MUTCD.

C. Planned off-site detours are not provided for this Project, except as shown under **V. <u>SPECIAL CONDITIONS</u>**.

D. Where traffic is permitted through the work area under stage construction, the Contractor may choose to construct, at no additional expense to the Department, temporary on-site bypasses or detours in order to expedite the Work. Plans for such temporary bypasses or detours shall be submitted to the Engineer for approval 30 calendar days prior to proposed construction. Such bypasses or detours shall be removed promptly when in the opinion of the Engineer, they are no longer necessary for the satisfactory progress of the work.

E. The Contractor's trucks and other vehicles shall travel in the direction of normal roadway traffic unless separated from the through-traffic by positive construction barriers approved by the Engineer. On Interstates or other divided highways, the Contractor's vehicles shall not cross the medians and shall enter and exit at the existing interchanges.

F. When construction operations necessitate an existing traffic signal to be out of service, the Contractor shall furnish off-duty police officers to regulate and maintain traffic control at the site.

G. There shall be no reduction in the total number of available traffic lanes except as specifically allowed by the Contract and as approved by the Engineer.

H. The Contractor shall schedule and arrange the Work to ensure the least inconvenience and the utmost in safety to the traveling public and to the Contractor's and the Department's forces.

I. All outfall ditches, special ditches, critical storm drain structures, erosion control structures, retention basins, etc. shall be constructed, where possible, prior to the beginning of grading operations so that the best possible drainage and erosion control will be in effect during the grading operations, thereby keeping the roadway areas as dry as possible.

J. In the prosecution of the Work, if it becomes necessary to remove any existing signs, markers, guardrail, etc. not covered by a specific pay item, they shall be removed, stored, and reinstalled, when directed by the Engineer, to line and grade, and in the same condition as when removed.

K. The Sequence of Operations provided for in this Contract, in conjunction with any staging details which may be shown in the Plans, is a suggested sequence for performing the Work. It is intended as a general staging plan for the orderly execution of the Work while minimizing the impact on the mainline, cross-streets, and side-streets. The Contractor shall develop detailed staging and traffic control plans for performing specific portions of the Work, including but not limited to traffic shifts, detours, bridge widenings, paces, lane closures or other activities that disrupt traffic flow. These plans shall be submitted for approval at least two weeks prior to the scheduled date of the activity.

L. As an alternative to the Sequence of Operations described herein, the Contractor may submit a Sequence of Operations for approval. A twenty calendar day lead time for the Department's review shall be given this submission so that a decision on its acceptability can be made and presented at the Preconstruction Conference. Insufficient lead time or no submission by the Contractor shall be construed as acceptance of the procedures outlined herein and the willingness to execute same.

The Department will not pay, or in any way reimburse the Contractor for claims using from the Contractor's inability to perform the Work in accordance with the Sequence of Operations provided in this Special Provision or the Plans or from an approved Contractor alternate.

II. ORDER OF WORK

A. Interstate

The Contractor shall not simultaneously perform work on both the inside shoulder and outside shoulder on either direction of traffic flow when the Work is within 12'-0" of the traveled way, unless such areas are separated by at least one-half mile of distance.

B. Non-Interstate Divided Highways

The Contractor shall not simultaneously perform work on both the inside shoulder and outside shoulder on either direction of traffic flow when the Work is within 12'-0" of the traveled way, unless such areas are separated by at least one-half mile of distance in rural areas or at least 500 feet of distance in urban areas.

C. Non-Divided Highways

1. The Contractor shall not simultaneously perform work on opposite sides of the roadway when the Work is within 12'-0" of the traveled way, unless such areas are separated by at least one-half mile of distance in rural areas or at least 500 feet of distance in urban areas.

2. Pilot vehicles will be required during placement of bituminous surface treatment or asphaltic concrete on two-lane roadways unless otherwise specified.

3. On two-lane projects where full width sections of the existing subgrade, base or surfacing are to be removed, and new base, subgrade, or surfacing are to be constructed, the Contractor shall maintain one-lane traffic through the construction areas by removing and replacing the undesirable material for half the width of the existing roadway at a time. Replacement shall be made such that paving is completed to the level of the existing pavement in the adjacent lane by the end of the work day.

D. Excavation

1. All areas within the limits of the project which are determined by the Engineer to be damaged, due either directly or indirectly to the process of construction, shall be cleaned up, redressed and grassed. All surplus materials shall be removed and disposed of as required. Materials to be wasted shall be disposed of in accordance with Sub-Section 201.02.E.3 of the current Specifications.

2. When trenching is required for minor roadway or shoulder widening, all operations at one site shall be completed to the level of the existing pavement in the same work day.

III. ENFORCEMENT

In the event that compliance with the objectives stated herein, or contained in the Contractor's approved alternate Sequence of Operations is not achieved, the Engineer may close down all operations being performed except Traffic Control and Erosion Control. The Engineer may also withhold any payments due until all the requirements herein have been met.

IV. MEASUREMENT AND PAYMENT

There will be no separate measurement or payment for the Work described herein, and all costs, direct or indirect, of complying with the requirements of this Special Provision shall be included in the overall bid submitted, as shown in Section 150, Traffic Control.

V. SPECIAL CONDITIONS

The Special Conditions contained herein are project specific. When there is a conflict between the General Provisions of Sections I through IV of this Specification and the Special Conditions, the Special Conditions will govern the Work.

A. The Department reserves the right to restrict construction operations when, in the opinion of the Engineer, the continuance of the work would seriously hinder traffic flow on days immediately before and during days on which major sporting events occur within the area.

B. The Contractor shall provide four (4) variable message boards and sequential flashing arrow signs for use as required and as needed. See Georgia Standards 9106 and 9107.

C. Two lane, one way bridge:
The Contractor shall be allowed to close one shoulder lane at a time. The lane shall be closed using portable sequential or flashing arrow panels meeting the requirements of Table IV-3 of the MUTCD and in accordance with Sub-Section 150.03 of the current Georgia Specifications.

D. The Contractor shall request, in writing, the Engineer's approval for any shifts, change, or restrictions to traffic through the project at least 14 days prior to the anticipated traffic control work. Upon approval, the Engineer shall notify the Public Affairs Office of the GA DOT at least five (5) days prior to each shift, change or restriction to traffic. Also, the Engineer shall notify the DOT Permits and Enforcement Office (404-635-8529) 7 days prior to lane closures to enable them to detour overwidth loads. Permits and Enforcement shall also be notified when work is interrupted or completed. Notification of beginning and ending work is mandatory to keep overwidth load detours to a minimum.

E. If in the opinion of the Engineer unsatisfactory work progress is being accomplished, the Engineer may instruct the Contractor to reopen the closed roadway section. If this occurs, the Contractor will not be allowed to close any roadway lane until sufficient manpower is provided by the Contractor to complete the work as quickly as possible.

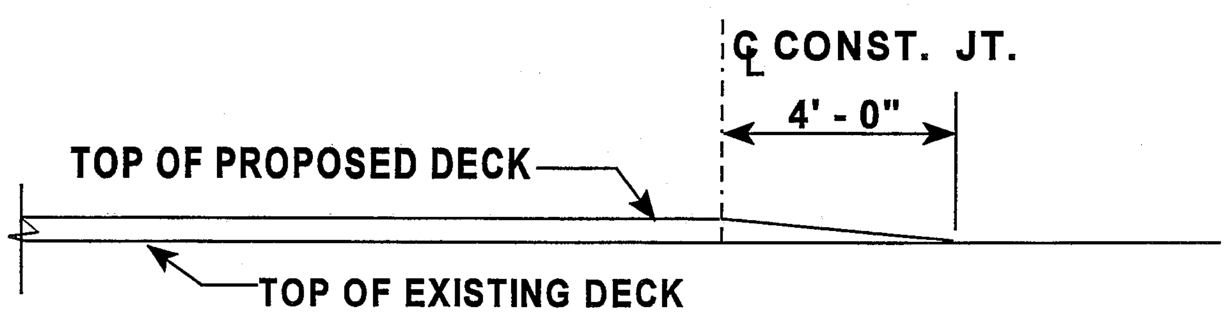
F. CONSTRUCTION STAGING

The Contractor shall submit a work plan for each bridge that shows how the work will be accomplished within the specified work time. Special attention shall be placed on the deck removal operation because of the slow production rate. Standby blasting and finishing equipment shall be provided to insure completion of work by specified time.

Individual through lanes shall be completed before the adjacent lane is started unless the Contractor provides adequate equipment to accomplish multi-lane construction and this work plan is approved by the Engineer prior to any construction.

If at any time a problem in the removal operation occurs, a pour may be terminated at a construction joint plus four feet (see detail below). This four feet shall be provided to taper down from the new construction to the existing deck. If weather related problems occur the removed portion of the deck shall be replaced within the specified closure time. The Engineer shall determine whether to accept or remove and replace this affected deck area.

It is expected that the Contractor shall make whatever plans necessary to complete the designated weekend work in the specified time restrictions. If an emergency occurs where the Contractor cannot complete the work in the specified time, the Contractor shall continue traffic control and maintain an adequate force on standby until such time as the Engineer directs that work be resumed. Striping in the bridge area shall be replaced before the lane closure is removed.



TRANSITION DETAIL

NO SCALE

G. Long term storage of materials or equipment will not be permitted. Equipment and material required for the construction operation in progress may remain in an area protected by temporary concrete barrier. Temporary guardrail or median barrier installed by the Contractor for his convenience of storing materials or equipment shall be placed and removed at his expense. Approval shall be obtained from the Engineer prior to installation and shall meet Georgia Standard Details for installation.

H. Cleaning Up and Finishing: At the end of the weekend work period, the Contractor will be required to remove all debris, stockpiled materials, equipment, tools and any other hazards and store them in such a manner as not to be vulnerable to run-off-the-road vehicle impact, or placed behind a positive barrier protection system.

All areas within the limits of the project which are determined by the Engineer to be unnecessarily damaged, due either directly or indirectly to the process of construction, shall be cleaned up, redressed and grassed. All surplus materials shall be removed and disposed of as required. This is not a payment item and shall be done without additional compensation.

I. The Contractor shall be aware that other Georgia DOT projects may be underway during the duration of this project. See Subsection 105.07 of the Specifications concerning cooperation between contractors.

J. The following are the Special Conditions for each bridge:

Work or equipment and material movement that interferes with traffic will be restricted as follows:

The concrete removal (hydrodemolition method) and concrete deck overlay work must be performed and completed between the hours of 9:00 p.m. Friday and 5:00 a.m. the following Monday. Equipment or materials moved across the traveled way at other times shall be done in a manner as not to unduly interfere with traffic.

The Contractor shall submit a work schedule for the work required to erect and remove the protective platforms. Single and double lane closures will be permitted on 1-285 for protective platform work: single lane closures will be permitted Monday through Friday from 9:00 a.m. to 3:30 p.m. or all day on weekends; double lane closures will be permitted Sunday through Thursday from 9:00 p.m. to 5:00 a.m. the following day, and Friday and Saturday from 10:00 p.m. to 9:00 a.m. the following day. For protective platform work on non--interstate routes, single lane closures will be permitted on Cleveland Avenue between the hours of 9:00 a.m. and 4:00 p.m. Monday through Friday or all day on weekends. Single lane closures with flagging and pilot car will be permitted on Collier Drive between 9:00 a.m. and 4:00 p.m. Monday through Friday or all day on weekends. The flagging and pilot car shall be provided and meet the requirements of the MUTCD, Georgia Specifications 150.07, and Georgia Standard 9102. Prior to any protective platform work, the Contractor shall submit a traffic control plan to the Engineer for approval as provided for in the Georgia Specifications, Section 150. The flagging and pilot car shall be included in the overall bid price submitted for Item 150 - Traffic Control.

A traffic control plan for each stage of construction shall be submitted to the Engineer for approval at least fourteen (14) days prior to beginning work. See Georgia Standard 9106.

APPENDIX E

Design Calculations for Exodermic Deck Preliminary Design

Design Calculations: Exodermic Bridge Deck Preliminary Design

Following the Analysis Steps given in Section 11.3.3:

1. **Select a Candidate Deck Type**

From Figure 11.4, showing the proposed construction staging of the I65 bridge, the largest panel dimensions are:

length=28 ft.
width=8 ft.

From Table E.1, choose 4.25" I for main bearing bar
From Table E.3, choose: main bearing bar spacing = 6in
concrete thickness = 4in
total thickness = 8.25in
top rebar size = #5
top rebar spacing = 6in

2. **Calculate Deck Weight**

From Table E.3: Deck weight = 67 lb/s.f.

3. **Derive Section Properties**

From Tables E.3 and E.4:
composite moment of inertia (positive bending) = 51.1 in^4
composite moment of inertia (negative bending) = 17.4 in^4
moment of inertia for grid only = 11.11 in^4
section modulus for composite deck:
positive bending; top of concrete = 124.05 in^3
positive bending; bottom of main bar = -10.31 in^3
negative bending; rebar = -5.90 in^3
negative bending; top of main bar = -13.78 in^3
negative bending; bottom of main bar = 5.83 in^3
section modulus for steel grid only:
top of grid = 6.23 in^3
bottom of grid = 4.51 in^3

4. **Check Stresses Under Design Load**

HS-25 live load, 8.25" exodermic deck

Composite Section Properties (in ^{3}/ft)

	Positive		Negative			Exodermic
	Top of Concrete Slab	Bottom of Steel Grid Main Bar	Embedded Steel Rebar	Top of Grid Main Bearing bar	Bottom of Steel Grid Main Bar	Deck Weight (lbs/s.f.)
Composite	124.05	-10.31	-5.9	-13.78	5.83	67
Grid Only	0	-4.51	0	-6.23	4.51	

(negative sign indicates tension)

Dead-Load Moment = 0.8 $WS^2/8$ where, W = weight/s.f.
S = girder spacing, c-c, minus ½ flange width

Live-Load Moment = [(S+2)/32]x(20x1.3x0.8) (for HS-25)

Dead-Load Stress = dead-load moment x 12/section modulus (grid only)
Live-Load Stress = live-load moment x 12/section modulus (composite deck)
Total Stress = DL Stress + LL Stress

S = {(8ft x 12in/ft)-(12in/2)}/12 in/ft = 7.5ft
Impact = 30%

DL moment = 377 ft-lb
LL moment = 6175 ft-lb

<u>Stress at top of concrete, positive bending</u>

LL Stress = (6175 ft-lb x 12in/ft)/124.05in^3 = 597psi (compression) **O.K.**

<u>Stress at bottom of steel grid main bar, positive bending</u>

DL Stress = (377ft-lb x 12in/ft)/-4.51in^3 = 1003 psi (tension)
LL Stress = (6175ft-lb x 12in/ft)/-10.31in^3 = <u>7187 psi (tension)</u>
total = 8190 psi (tension) **O.K.**

<u>Stress in embedded steel reinforcing bar, negative bending</u>

DL Stress = N/A
LL Stress = (6175ft-lb x 12in/ft)/-5.90in^3 = 12559 psi (tension) **O.K.**

<u>Stress in top of grid main bearing bar, negative bending</u>

DL Stress = (377ft-lb x 12in/ft)/-6.28 in^3 = 726 psi (tension)
LL Stress = (6175ft-lb x 12in/ft)/-13.78 in^3 = <u>5377 psi (tension)</u>
total = 6103 psi (tension) **O.K.**

<u>Stress in bottom of steel grid main bar, negative bending</u>

DL Stress = (377ft-lb x 12in/ft)/4.51in^3 = 1003 psi (compression)
LL Stress = (6175ft-lb x 12in/ft)/5.83in^3 = 12710 psi (compression)
total = 13713 psi (compression) **O.K.**

	Stresses (psi)				
	Positive		Negative		
	Top of Concrete Slab	Bottom of Steel Grid Main Bar	Embedded Steel Rebar	Top of Grid Main Bearing bar	Bottom of Steel Grid Main Bar
Dead-Load	0	1003	0	726	1003
Live-Load	597	7187	12559	5377	12710
Total	597	8190	12559	6103	13713

5. **Check Deck Deflection**

bare grid with wet concrete

defl. = 5wl^4/384EI
w = uniform load per foot = 67 lbs/s.f. = 5.58 lbs/in
l = beam spacing = 8 ft = 96 in
I = the moment of inertia = 11.11in^4 (bare steel grid)
E = modulus of elasticity for steel = 29E+6 psi

defl. = 5(5.58lb/in)(96in)^4/384(29E6psi)(11.11in^4) = 0.019 in **O.K.**

composite deck (completed and unshored)

defl. due to live-load and impact = 0.015Pl^3/EI
P= 26 kips (HS-25 wheel load + 30% impact)
l = 8ft = 96in
I (per ft) = 51.1 in^4
effective width = {4 + 0.06(S)}; (AASHTO 3.24.3.2)
= 4 + 0.06 x 8 = 4.48ft.
I_{eff} = 4.48 x 51.1 = 228.9in^4

defl. max = 0.015(26kips)(96in)^3/(228.9in^4)(29000ksi)
=0.052 < 0.12 = l/800 **O.K.**

6. **Check Deck Cantilever**

distance from centerline of fascia stringer to edge of deck = 4 ft.
barrier height (assuming same dimensions as EBDI example) = 34 in.
width of barrier at base = 1.5 ft.
distance from centerline of fascia stringer to curb or edge of barrier = 2.5 ft.
distance from centerline of fascia stringer to vertical centroid of barrier = 3.41 ft.
deck weight = 67 lb/s.f.
barrier weight = 390 lb/s.f.
live-load (HS-25 + 30%LL) = 26 kips
impact force per AASHTO figure 2.7.4B =10 kips
barrier width over which impact force is distributed = 5 ft.
section modulus at top of rebar in negative bending = -5.9 in^3

section modulus at bottom of main bar in negative bending = $5.83in^3$

dead-load moment:

deck cantilever moment = (width of cantilever/2) x (deck weight per sq.ft. x width of cantilever)
= 0.536 ft-kips per foot

barrier cantilever moment = (barrier weight per ft.) x (distance to vertical centroid of barrier)
= 1.326 ft-kips per foot

total dead-load moment = 1.862 ft-kips per foot

live-load moment:

distribution width given by AASHTO (3.24.5.1.1) is E = 0.8X + 3.75 where X is the distance from the load to the point of support. AASHTO 3.24.2 says that the wheel load is 1 ft. from the curb (or barrier)

X = 2.5 - 1 = 1.5ft.

E = 0.8(1.5) +3.75 = 4.95

live-load moment = P x distance to wheel/E
= 7.88 ft-kips per foot

total moment = 9.742 ft-kips per foot

Stresses under design load

stress = total moment x 12/section modulus

rebar stress = 19.8 ksi **O.K.**
bottom of main bearing bar stress = 20 ksi **O.K.**

6(b). **Cantilever Stresses Due to Impact Moment**

barrier impact moment = 10 kips x 34in/12in per ft. = 28.333 ft-kips
AASHTO figure 2.7.4B; increase load by factor C = 1 + (h-33)/18
C = 1.05556; adjusted moment = 29.907 ft-kips

AASHTO 2.7.1.3.7 says impact distance over 5 ft. of barrier. The length of deck that will resist moment = 5 + (0.8x + 3.75), where x = distance from centerline of fascia stringer to centroid of barrier)

L = 11.478

moment per foot = 29.907 ft-kips/11.478 = 2.6056 ft-kips per foot of deck
add dead-load moment: 2.6056 + 1.862 = 4.47 ft-kips per foot of deck

Stress due to barrier impact and dead load:

rebar = (4.47 ft-kips x 12 in/ft)/-5.90in^3 = 9.1 ksi (tension) **O.K.**
main bearing bar = 4.47 ft-kips x 12 in/ft)/5.83in^3 = 9.2 ksi (tension) **O.K.**

THE EXODERMIC DECK IS SUFFICIENT TO SUPPORT THE CANTILEVER

7. **Derive Section Properties for Composite Deck/Girder (positive bending)**

DECK:

main bar type	= 4.25 in I
main bar height	= 4.25 in
distribution bar height	= 1.25 in
distribution bar thickness	= .25 in
distribution bar spacing	= 4 in
lower rebar size	= #3
lower rebar spacing	= 4 in
concrete thickness	= 4 in
concrete strength (28 day)	= 4000 psi
modular ratio	= 8
haunch height	= 4.75 in
haunch width	=12 in

GIRDER:

type	= W36 x 150
depth	= 35.85 in
area	= 44.2 in^2
centroid	= 17.925 in
moment of inertia	= 9040 in^4
spacing	= 8 ft.
length	= 58.5 ft.

determine effective flange width (10.38.3.1):

l/4 = 14.6 ft
stringer spacing = 8ft
12t = 8.25 ft
use smallest = 8.25 ft.

determine concrete slab area:

concrete thickness x width = 48 in^2 (transformed)

determine concrete haunch area:

height x width = 7.125 (transformed)

determine distribution bar area:

area = area per bar x # of bars = 7.5 in^2

determine rebar area:

area = area per bar x # of bars x (n-1)/n, to adjust for displace concrete
= 2.31 in^2

calculate neutral axis:

Element	A	d	Ad
concrete slab	48	42.6	2044.8

rebar	2.31	41.79	96.5
haunch	7.125	38.23	272.4
distribution bars	7.5	39.98	299.9
girder	44.2	17.93	792.5
sum =	109.14		3506.07

location of center of gravity = 3506.07/109.14 = 32.13 in

calculate moment of inertia:

Element	d'	A(d')^2	Io	Ig
slab	10.47	5261.8	64	5325.8
rebar	9.66	215.56	0	215.56
haunch	6.1	265.12	13.40	278.52
distribution bars	7.85	462.17	0.98	463.15
stringer	-14.2	8912.49	9040	17952.49
			sum =	24235.52

moment of inertia = 24235.52 in^4

Section Moduli; Positive Bending (in^3):

point of interest	distance from centroid to point of interest	effective section modulus
top of concrete	12.47	15548.05
reinforcing in slab	9.66	2508.85
top of distribution bar	8.47	2861.34
top of girder	3.72	6514.9
bottom of girder	-32.13	754.3

7(b). **Derive Section Properties for Composite Deck/Girder (negative bending)**

rebar area (without considering concrete) = 2.64 in^2

calculate neutral axis:

Element	A	d	Ad
rebar	2.64	41.79	110.33
distribution bars	7.5	39.98	299.85
stringer	44.2	17.93	792.506
sum =	54.34		1202.68

center of gravity = 22.13 in

calculate moment of inertia:

Element	d'	A(d')^2	Io	Ig
rebar	19.66	1020.40	0	1020.40
distribution bars	17.85	2389.67	0.98	2390.65

stringer	-4.2	779.69	9040	9819.69
			sum =	13230.74

moment of inertia = 13230.74 in^4

Section Moduli; negative bending (in^3):

point of interest	distance from centroid to point of interest	effective section modulus
reinforcing bars	19.66	672.98
top of distribution bar	18.47	716.34
top of girder	13.72	964.34
bottom of girder	-22.13	597.86

8. **Check Girder Stress Levels and Deflections**

note: stress due to additional loads from barriers, sidewalks, etc. not included

effective width of deck = 8 ft.
deck weight = 67 psf
+6.5 psf (due to haunch and shear key concrete)
73.5 psf

deck dead-load = 63 psf x 8 ft = 588 lb/ft.
beam dead-load = 150 lb/ft.
total = 738 lb/ft.
span = 58.5 ft.
beam spacing = 8ft.
impact = 29% (AASHTO 3.8.2.1)

dead-load moment = $wL^2/8$ = (738lb/ft)$(58.5ft)^2/8$ = 315.7 ft-kips
live-load moment: from AASHTO loading table, p. 643, 15th edition:
779.7 ft-kips/ft/lane (x 1.25 for HS-25)
= 974.7 ft-kips/ft.
this is for axle loads, divide by 2 = 487.3 ft-kips
AASHTO 3.23.2.2 determines fraction of moment to apply to composite girder
= S/5.5 = 8.0/5.5 = 1.45
live-load moment = 911.5 ft-kips (including 29% impact)

Check stress at bottom of girder (positive bending)

dead-load stress = dead-load moment/SM (steel beam only) (for construction)
= (315.7 ft-kips x 12 in/ft.)/504 = 7.52
but the girders are continuous, so multiply by 0.8 = 6 ksi **O.K.**

for total stress;
dead-load = 315.7 x 12/754.3 = 5.02 ksi
live-load = 911.5 x 12/754.3 = 14.5 ksi
multiply by 0.8 for total stress = 15.62 ksi **O.K.**

Check stress at top of concrete (positive bending)

live-load stress = 911.5 x 12 / 15548.05 = 0.7 ksi
x 0.8 = 0.56 ksi **O.K.**

Estimated Deflection

deflection due to live-load and impact for a simple span = $PL^3/48EI$
derive P from AASHTO LL moment, using M=PL/4

974.7 ft-kips = P(58.5ft)/4; P = 66.6 kips/lane
divide by 2 since this is for axle loads:
= 33.3 kips/lane
use same factor as before: 1.45
33.3 x 1.45 = 48.3 x 1.29 (impact) = 62.33 kips

P = 62.33 kips
l = 58.5 ft. = 702 in
I = 24235.52 in^4
E = 29.0E+06 psi

max. deflection (live-load and impact) = $62.33kips(702\ in)^3/48(29E3ksi)(24235.52in^4)$
= 0.639 in = l/1126 < l/800 **O.K.**
*since this is for a simple span, the deflections will be much less than this

Table E.1. Exodermic Bridge Deck Design Starting Point (26)

Use this table as a starting point where there are weight or thickness constraints.

			W E I G H T S (lbs/sf)				Maximum Span (ft.)			
			Normal		Lightweight		Continuous over multiple spans		Simple Spans	
Main Bar Type	Concrete Thickness (in.)	Total Deck Thickness (in.)	Min	Max	Min	Max	HS-20	HS-25	HS-20	HS-25
3"T	3.5	6.50	52	58	44	50	10	9	9	8
	4.0	7.00	59	64	49	55	11	10	9	8
	4.5	7.50	65	70	54	59	12	11	10	9
4.25" I	3.5	7.75	55	64	46	56	13	11	11	9
	4.0	8.25	61	70	51	61	14	12	12	10
	4.5	8.75	67	76	56	66	15	13	13	11
5" I	3.5	8.69	57	67	49	59	16	14	13	11
	4.0	9.19	63	74	54	64	17	15	14	12
	4.5	9.69	69	80	58	69	18	16	15	13

After choosing grid type, concrete type, and thickness, complete the deck selection by choosing main bearing bar spacing and rebar size and spacing, using the tables that follow, and then verify with appropriate calculations as shown in the design examples.

NOTE: These numbers have been prepared using generally accepted engineering principles, and assumptions given elsewhere in this document. It is the responsibility of the design engineer to verify these and any other numbers found herein by independent calculation. Neither the Exodermic Bridge Deck Institute, Inc. nor any of its officers, directors, employees or consultants shall be liable in any way for any omissions or errors that may be contained herein.

NOTE 2: EBDI urges the use of conservative design. Please refer to the design philiosophy section elsewere in this document.

NOTE 3: Maximum spans checked for deflection (l/800) for n=8, normal weight concrete.

Table E.2. Exodermic Bridge Deck Configuration Codes (26)

Use this table as a starting point where there are no thickness constraints. Deck configurations shown include 4.5" of concrete, which gives 2.5" of cover over top rebar. Where weight is critical, cover can be reduced, and/or lightweight concrete specified, although reduction of concrete thickness may require a different configuration of grid and rebar. The configurations listed are only examples; others may be equally satisfactory.

DECK CONFIGURATION CODES*

	Main Bearing Bars Transverse to Traffic		Main Bearing Bars Parallel to Traffic	
SPAN (ft.)	HS-20	HS-25	HS-20	HS-25
3	31045405	31045405	31045510	31045505
4	31045405	31045505	31045505	31045505
5	31045505 51045410	31045505 51045510	31045505 51045510	30845504 51045405
6	31045505 51045510	30845504 51045510	30845504 51045505	30845604 50845508
7	30845504 51045510	30845504 51045505	50845508	50845504
8	30845504 51045505	30845504 50845504	50845504	50845504
9	51045505	50845504	50845504	50645504
10	50845508	50845504	50845504	50645504
11	50845504	50845504	50845604	50645604
12	50845504	50645504	50645606	50645604
13	50645506	50645606	50645604	50645604 **
14	50645504	50645604	50645604	50645604 **
15	50645504	50645604	50645604 **	
16	50645504	50645603	50645604 **	
17	50645504			

* FOR DECK CONFIGURATION CODES, SEE P. 37
** INDICATES A588 STEEL REQUIRED.

EBDI recommends using the indicated deck configuration as a starting point, refining the design by selecting concrete type and thickness, using the tables that follow, and then verify with appropriate calculations as shown in the design examples. Some changes in configuration may be required where the concrete component is pre-cast versus cast-in-place.

NOTE 1: These numbers have been prepared using generally accepted engineering principles, and assumptions given elsewhere in this document. It is the responsibility of the design engineer to verify these and any other numbers found herein by independent calculation. Neither the Exodermic Bridge Deck Institute, Inc. nor any of its officers, directors, employees or consultants shall be liable in any way for any omissions or errors that may be contained herein.
NOTE 2: EBDI urges the use of conservative design. Please refer to the design philiosophy section elsewere in this document.
NOTE 3: Maximum spans checked for deflection (l/800) for n=8, normal weight concrete.

Table E.3. 4 ½" Main Bearing Bar Information (26)

Deck Configuration Code (See Page 37)	Main Bar Type	Main Bar Spacing, in.	Concrete Thickness, in.	Total Thickness, in.	Top rebar size	Top rebar spacing, in.	WEIGHT: lbs/sf (normal wt concrete @ 145 pcf)	WEIGHT: lbs/sf (light wt concrete @ 115 pcf)	MOMENT OF INERTIA: Positive Bending, (N=8) in⁴	MOMENT OF INERTIA: Negative Bending, (N=8) in⁴	SECTION MODULUS, Positive Bending: SM Top of Concrete, in³	SECTION MODULUS, Positive Bending: SM Bottom of Main Bar, in³	SECTION MODULUS, Negative Bending: SM Rebar, in³	SECTION MODULUS, Negative Bending: SM Top of Main Bar, in³	SECTION MODULUS, Negative Bending: SM Bottom of Main Bar, in³	MAX. SPAN, ft. (See Note), Main Beaing Bars Transverse to Traffic, Cast-in-place: HS-20	Transverse, Cast-in-place: HS-25	Transverse, Pre-cast: HS-20	Transverse, Pre-cast: HS-25	Main Beaing Bars Parallel to Traffic, Cast-in-place: HS-20	Parallel, Cast-in-place: HS-25	Parallel, Pre-cast: HS-20	Parallel, Pre-cast: HS-25
40635603	4.25" I	6	3.5	7.75	6	3	63	55	45.1	25.8	123.54	-9.35	-11.01	-43.41	7.06	13	11	13	11	11	9	11	9
40635604	4.25" I	6	3.5	7.75	6	4	62	54	44.6	23.1	119.96	-9.34	-9.04	-28.58	6.73	13	11	13	11	11	9	11	9
40635606	4.25" I	6	3.5	7.75	6	6	61	53	44.0	19.9	116.23	-9.32	-7.08	-18.66	6.27	13	11	13	11	10	8	10	8
40635503	4.25" I	6	3.5	7.75	5	3	62	54	44.3	22.1	118.42	-9.31	-8.60	-25.10	6.55	13	11	13	11	10	9	11	9
40635504	4.25" I	6	3.5	7.75	5	4	61	53	43.9	19.9	116.00	-9.30	-7.25	-18.81	6.24	13	11	13	11	10	8	10	8
40635506	4.25" I	6	3.5	7.75	5	6	61	52	43.5	17.4	113.49	-9.29	-5.90	-13.78	5.83	12	11	12	11	9	8	10	8
40635406	4.2[illegible]	[illegible]	[illegible]	7.75	4	6	60	52	43.1	15.2	111.40	-9.27	-4 97	-10.58	5.41	12	10	12	10	9	7	8	7
40640603	4.25" I	6	4	8.25	6	3	69	60	52.2	25.8	130.98	-10.30	-11.01	-43.41	7.06	14	12	14	12	11	9	11	9
40640604	4.25" I	6	4	8.25	6	4	68	59	51.8	23.1	128.55	-10.31	-9.04	-28.58	6.73	14	12	14	12	11	9	11	9
40640606	4.25" I	6	4	8.25	6	6	67	58	51.4	19.9	125.99	-10.31	-7.08	-18.66	6.27	14	12	14	12	10	8	10	8
40640503	4.25" I	6	4	8.25	5	3	68	59	51.6	22.1	127.40	-10.30	-8.60	-25.10	6.55	14	12	14	12	10	8	11	9
40640504	4.25" I	6	4	8.25	5	4	67	58	51.4	19.9	125.76	-10.31	-7.25	-18.81	6.24	14	12	14	12	10	8	10	8
40640506	4.25" I	6	4	8.25	5	6	67	57	51.1	17.4	124.05	-10.31	-5.90	-13.78	5.83	13	11	14	11	9	8	10	8
40640406	4 25" I	6	4	8 25	4	[illegible]	[illegible]	[illegible]	50.9	15.2	122 59	-10 32	-4 97	-10 58	5.41	13	10	12	10	9	7	8	7
[illegible]	[illegible]	[illegible]	[illegible]	8.75	[illegible]	[illegible]	75	65	60.4	25.8	[illegible]	-11.37	-11.01	-43.41	7.06	15	13	15	13	11	9	11	9
[illegible]5604	4.25 I	6	4.5	8.75	6	4	74	64	60.2	23.1	139.02	-11.39	-9.04	-28.58	6.73	15	12	15	13	10	9	11	9
40645606	4.25" I	6	4.5	8.75	6	6	73	62	60.0	19.9	137.39	-11.41	-7.08	-18.66	6.27	14	12	15	12	10	8	10	8
40645503	4.25" I	6	4.5	8.75	5	3	74	63	60.1	22.1	138.17	-11.39	-8.60	-25.10	6.55	14	12	15	13	10	8	11	9
40645504	4.25" I	6	4.5	8.75	5	4	73	63	59.9	19.9	137.15	-11.41	-7.25	-18.81	6.24	14	12	15	12	10	8	10	8
40645506	4.25" I	6	4.5	8.75	5	6	73	62	59.8	17.4	136.09	-11.43	-5.90	-13.78	5.83	13	11	14	11	9	8	9	8
40645406	4.25" I	6	4.5	8.75	4	6	72	61	59.7	15.2	135.14	-11.44	-4.97	-10.58	5.41	12	10	12	9	9	7	8	7
40835604	4.25" I	8	3.5	7.75	6	4	59	51	36.7	19.4	107.70	-7.29	-8.26	-32.55	5.30	11	10	11	10	8	7	9	7
40835608	4.25" I	8	3.5	7.75	6	8	58	49	36.1	15.0	103.05	-7.29	-5.31	-13.99	4.70	11	9	11	9	8	6	8	6
40835504	4.25" I	8	3.5	7.75	5	4	58	50	36.2	16.6	104.37	-7.28	-6.45	-18.82	4.91	11	9	11	10	8	6	8	7
40835508	4.25" I	8	3.5	7.75	5	8	57	49	35.8	13.1	101.27	-7.28	-4.43	-10.33	4.37	10	8	11	8	7	6	7	6
40840604	4.25" I	8	4	8.25	6	4	65	56	42.7	19.4	115.54	-8.07	-8.26	-32.55	5.30	12	10	12	10	8	7	9	7
40840608	4.25" I	8	4	8.25	6	8	64	54	42.4	15.0	112.65	-8.10	-5.31	-13.99	4.70	11	9	11	9	8	6	8	6
40840504	4.25" I	8	4	8.25	5	4	64	55	42.5	16.6	113.38	-8.08	-6.45	-18.82	4.91	11	9	12	9	8	6	8	7
[illegible]	[illegible]	8	4	8.25	5	8	63	53	42.3	13.1	111.49	-8.10	-4.43	-10.33	4.37	10	8	10	8	7	6	7	6
40845604	4.25" I	8	4.5	8.75	6	4	71	61	49.8	19.4	125.16	-8.95	-8.26	-32.55	5.30	12	10	13	10	8	7	9	7
40845608	4.25" I	8	4.5	8.75	6	8	70	59	49.7	15.0	123.64	-8.98	-5.31	-13.99	4.70	11	9	11	9	8	6	8	6
40845504	4.25" I	8	4.5	8.75	5	4	70	59	49.7	16.6	123.91	-8.97	-6.45	-18.82	4.91	11	9	12	9	8	6	8	7
40845508	4.25" I	8	4.5	8.75	5	8	69	58	49.6	13.1	122.97	-8.99	-4.43	-10.33	4.37	10	8	10	8	7	6	7	6
41035605	4.25" I	10	3.5	7.75	6	5	57	48	31.2	15.5	97.26	-6.01	-6.61	-26.04	4.24	10	8	10	8	7	6	7	6
41035610	4.25" I	10	3.5	7.75	6	10	56	47	30.8	12.0	94.09	-6.02	-4.25	-11.20	3.76	9	7	9	7	6	5	6	5
41035505	4.25" I	10	3.5	7.75	5	5	56	48	30.9	13.2	94.93	-6.01	-5.16	-15.06	3.93	9	7	9	7	6	5	7	5
41035510	4.25" I	10	3.5	7.75	5	10	55	47	30.7	10.4	92.84	-6.01	-3.54	-8.27	3.50	8	6	8	6	6	5	6	5
41040605	4.25" I	10	4	8.25	6	5	63	53	36.5	15.5	105.17	-6.68	-6.61	-26.04	4.24	10	8	10	8	7	6	7	6
41040610	4.25" I	10	4	8.25	6	10	62	52	36.4	12.0	103.42	-6.70	-4.25	-11.20	3.76	9	7	9	7	6	5	6	5
41040505	4.25" I	10	4	8.25	5	5	62	52	36.4	13.2	103.79	-6.69	-5.16	-15.06	3.93	9	7	9	7	6	5	6	5
41040510	4.25" I	10	4	8.25	5	10	61	51	36.3	10.4	102.68	-6.70	-3.54	-8.27	3.50	8	6	8	6	6	5	6	5
41045605	4.25" I	10	4.5	8.75	6	5	69	58	42.8	15.5	114.64	-7.42	-6.61	-26.04	4.24	9	8	10	8	7	6	7	6
41045610	4.25" I	10	4.5	8.75	6	10	68	57	42.7	12.0	113.98	-7.43	-4.25	-11.20	3.76	9	7	9	7	6	5	6	5
41045505	4.25" I	10	4.5	8.75	5	5	68	57	42.7	13.2	113.99	-7.43	-5.16	-15.06	3.93	9	7	9	7	6	5	6	5
41045510	4.25" I	10	4.5	8.75	5	10	67	56	42.7	10.4	113.62	-7.44	-3.54	-8.27	3.50	8	6	8	6	6	5	6	5
41235606	4.25" I	12	3.5	7.75	6	6	55	47	27.3	12.9	89.69	-5.13	-5.50	-21.70	3.53	8	6	8	6	6	5	6	5
41235506	4.25" I	12	3.5	7.75	5	6	55	46	27.1	11.0	87.98	-5.13	-4.30	-12.55	3.28	7	6	8	6	5	4	5	4
41235406	4.25" I	12	3.5	7.75	4	6	54	45	27.0	9.3	86.72	-5.14	-3.34	-8.03	3.00	7	5	7	5	5	4	5	4
41240606	4.25" I	12	4	8.25	6	6	61	52	32.1	12.9	97.54	-5.71	-5.50	-21.70	3.53	8	6	8	6	6	5	6	5
41240506	4.25" I	12	4	8.25	5	6	61	51	32.0	11.0	96.63	-5.72	-4.30	-12.55	3.28	7	6	8	6	5	4	5	4
41240406	4.25" I	12	4	8.25	4	6	60	50	32.0	9.3	96.00	-5.73	-3.34	-8.03	3.00	7	5	7	5	5	4	5	4
41245606	4.25" I	12	4.5	8.75	6	6	67	56	37.7	12.9	106.79	-6.36	-5.50	-21.70	3.53	8	6	8	6	6	5	6	5
41245506	4.25" I	12	4.5	8.75	5	6	67	56	37.7	11.0	106.48	-6.36	-4.30	-12.55	3.28	7	6	8	6	5	4	5	4
41245406	4.25" I	12	4.5	8.75	4	6	66	55	37.7	9.3	106.34	-6.37	-3.34	-8.03	3.00	7	5	7	5	5	4	5	4

Table E.4. Additional Information (26)

Deck Configuration Code (See Page 37)	Positive Bending n=24		BARE STEEL GRID		
	SM Top of Concrete, in^3	SM Bottom of Main Bar, in^3	Moment of Inertia, I, Bare Grid, in^4	SM Top of Grid, in^3	SM Bottom of Grid, in^3
30835604	166.04	-7.29	4.37	2.43	3.80
30835608	144.17	-7.22	4.37	2.43	3.80
30835504	151.22	-7.22	4.37	2.43	3.80
30835508	136.02	-7.16	4.37	2.43	3.80
30840604	170.57	-7.85	4.37	2.43	3.80
30840608	153.67	-7.86	4.37	2.43	3.80
30840504	159.02	-7.83	4.37	2.43	3.80
30840508	147.24	-7.85	4.37	2.43	3.80
30845604	178.44	-8.51	4.37	2.43	3.80
30845608	165.51	-8.58	4.37	2.43	3.80
30845504	169.50	-8.54	4.37	2.43	3.80
30845508	160.48	-8.60	4.37	2.43	3.80
31035605	146.46	-5.97	3.50	1.94	3.04
31035610	130.16	-5.93	3.50	1.94	3.04
31035505	135.33	-5.93	3.50	1.94	3.04
31035510	124.08	-5.90	3.50	1.94	3.04
31040605	152.40	-6.47	3.50	1.94	3.04
31040610	140.16	-6.49	3.50	1.94	3.04
31040505	143.94	-6.47	3.50	1.94	3.04
31040510	135.47	-6.49	3.50	1.94	3.04
31045605	161.22	-7.04	3.50	1.94	3.04
31045610	152.21	-7.11	3.50	1.94	3.04
31045505	154.89	-7.08	3.50	1.94	3.04
31045510	148.67	-7.13	3.50	1.94	3.04
31240606	139.68	-5.52	2.91	1.62	2.53
31240506	133.20	-5.53	2.91	1.62	2.53
31240406	128.08	-5.54	2.91	1.62	2.53
31245606	149.02	-6.04	2.91	1.62	2.53
31245506	144.34	-6.07	2.91	1.62	2.53
31245406	140.66	-6.10	2.91	1.62	2.53

Deck Configuration Code (See Page 37)	Positive Bending n=24		BARE STEEL GRID		
	SM Top of Concrete, in^3	SM Bottom of Main Bar, in^3	Moment of Inertia, I, Bare Grid, in^4	SM Top of Grid, in^3	SM Bottom of Grid, in^3
40635603	234.90	-8.18	11.11	6.23	4.51
40635604	218.06	-8.06	11.11	6.23	4.51
40635606	200.13	-7.92	11.11	6.23	4.51
40635503	211.69	-7.99	11.11	6.23	4.51
40635504	199.79	-7.89	11.11	6.23	4.51
40635506	187.33	-7.79	11.11	6.23	4.51
40635406	177.20	-7.69	11.11	6.23	4.51
40640603	233.95	-8.72	11.11	6.23	4.51
40640604	220.58	-8.65	11.11	6.23	4.51
40640606	206.26	-8.56	11.11	6.23	4.51
40640503	215.36	-8.59	11.11	6.23	4.51
40640504	205.88	-8.53	11.11	6.23	4.51
40640506	195.89	-8.47	11.11	6.23	4.51
40640406	187.67	-8.40	11.11	6.23	4.51
40645603	237.66	-9.37	11.11	6.23	4.51
40645604	227.00	-9.34	11.11	6.23	4.51
40645606	215.52	-9.29	11.11	6.23	4.51
40645503	222.69	-9.30	11.11	6.23	4.51
40645504	215.12	-9.27	11.11	6.23	4.51
40645506	207.12	-9.25	11.11	6.23	4.51
40645406	200.48	-9.21	11.11	6.23	4.51
40835604	195.57	-6.33	8.33	4.67	3.38
40835608	171.39	-6.19	8.33	4.67	3.38
40835504	179.28	-6.22	8.33	4.67	3.38
40835508	162.46	-6.11	8.33	4.67	3.38
40840604	198.01	-6.81	8.33	4.67	3.38
40840608	179.18	-6.74	8.33	4.67	3.38
40840504	185.22	-6.75	8.33	4.67	3.38
40840508	172.12	-6.70	8.33	4.67	3.38
40845604	204.24	-7.38	8.33	4.67	3.38
40845608	189.66	-7.37	8.33	4.67	3.38
40845504	194.24	-7.36	8.33	4.67	3.38
40845508	184.08	-7.35	8.33	4.67	3.38
41035605	171.24	-5.20	6.67	3.74	2.70
41035610	153.16	-5.12	6.67	3.74	2.70
41035505	158.95	-5.13	6.67	3.74	2.70
41035510	146.49	-5.07	6.67	3.74	2.70
41040605	175.55	-5.64	6.67	3.74	2.70
41040610	161.82	-5.60	6.67	3.74	2.70
41040505	166.13	-5.60	6.67	3.74	2.70
41040510	156.63	-5.58	6.67	3.74	2.70
41045605	183.12	-6.14	6.67	3.74	2.70
41045610	172.85	-6.15	6.67	3.74	2.70
41045505	175.97	-6.14	6.67	3.74	2.70
41045510	168.87	-6.15	6.67	3.74	2.70
41235606	154.49	-4.44	5.56	3.12	2.25
41235506	144.81	-4.39	5.56	3.12	2.25
41235406	137.14	-4.36	5.56	3.12	2.25
41240606	159.94	-4.83	5.56	3.12	2.25
41240506	152.68	-4.81	5.56	3.12	2.25
41240406	146.93	-4.80	5.56	3.12	2.25
41245606	168.28	-5.29	5.56	3.12	2.25
41245506	162.94	-5.29	5.56	3.12	2.25
41245406	158.73	-5.30	5.56	3.12	2.25

Deck Configuration Code (See Page 37)	Positive Bending n=24		BARE STEEL GRID		
	SM Top of Concrete, in^3	SM Bottom of Main Bar, in^3	Moment of Inertia, I, Bare Grid, in^4	SM Top of Grid, in^3	SM Bottom of Grid, in^3
50635603	305.61	-10.59	17.14	7.17	6.13
50635604	282.78	-10.43	17.14	7.17	6.13
50635606	258.73	-10.24	17.14	7.17	6.13
50635503	274.82	-10.34	17.14	7.17	6.13
50635504	258.73	-10.22	17.14	7.17	6.13
50635506	242.01	-10.07	17.14	7.17	6.13
50635406	228.69	-9.93	17.14	7.17	6.13
50640603	303.68	-11.16	17.14	7.17	6.13
50640604	285.06	-11.05	17.14	7.17	6.13
50640606	265.28	-10.92	17.14	7.17	6.13
50640503	278.39	-10.98	17.14	7.17	6.13
50640504	265.17	-10.89	17.14	7.17	6.13
50640506	251.36	-10.80	17.14	7.17	6.13
50640406	240.27	-10.70	17.14	7.17	6.13
50645603	306.49	-11.84	17.14	7.17	6.13
50645604	291.25	-11.77	17.14	7.17	6.13
50645606	274.96	-11.69	17.14	7.17	6.13
50645503	285.63	-11.72	17.14	7.17	6.13
50645504	274.78	-11.67	17.14	7.17	6.13
50645506	263.38	-11.61	17.14	7.17	6.13
50645406	254.15	-11.55	17.14	7.17	6.13
50835604	254.23	-8.17	12.86	5.38	4.60
50835608	221.32	-7.97	12.86	5.38	4.60
50835504	232.48	-8.02	12.86	5.38	4.60
50835508	209.54	-7.87	12.86	5.38	4.60
50840604	256.20	-8.68	12.86	5.38	4.60
50840608	229.84	-8.55	12.86	5.38	4.60
50840504	238.69	-8.58	12.86	5.38	4.60
50840508	220.25	-8.49	12.86	5.38	4.60
50845604	261.98	-9.27	12.86	5.38	4.60
50845608	240.90	-9.21	12.86	5.38	4.60
50845504	247.88	-9.22	12.86	5.38	4.60
50845508	233.12	-9.18	12.86	5.38	4.60
51035605	222.41	-6.69	10.28	4.30	3.68
51035610	197.70	-6.57	10.28	4.30	3.68
51035505	205.98	-6.60	10.28	4.30	3.68
51035510	188.83	-6.50	10.28	4.30	3.68
51040605	226.48	-7.15	10.28	4.30	3.68
51040610	207.12	-7.08	10.28	4.30	3.68
51040505	213.51	-7.09	10.28	4.30	3.68
51040510	200.05	-7.05	10.28	4.30	3.68
51045605	233.89	-7.68	10.28	4.30	3.68
51045610	218.72	-7.66	10.28	4.30	3.68
51045505	223.62	-7.66	10.28	4.30	3.68
51045510	213.11	-7.65	10.28	4.30	3.68
51235606	200.44	-5.69	8.57	3.58	3.07
51235506	187.49	-5.63	8.57	3.58	3.07
51235406	177.10	-5.57	8.57	3.58	3.07
51240606	205.83	-6.11	8.57	3.58	3.07
51240506	195.73	-6.08	8.57	3.58	3.07
51240406	187.67	-6.05	8.57	3.58	3.07
51245606	214.21	-6.59	8.57	3.58	3.07
51245506	206.39	-6.58	8.57	3.58	3.07
51245406	200.15	-6.58	8.57	3.58	3.07