AUBURN

SAMUEL GINN
COLLEGE OF ENGINEERING

Final Report 1

RAPID REHABILITATION/REPLACEMENT OF BRIDGE DECKS – PHASE III

PART I REPORT

DESIGN AND CONSTRUCTION PLANNING FOR RAPID DECK REPLACEMENT OF I-59 SISTER BRIDGES AT COLLINSVILLE, AL

Submitted to

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The objective of the research reported on herein was to develop the design and construction sequences and procedures for each of the four rapid-deck replacement systems identified above for the Collinsville, AL "test bridges."

In later work, each of the four deck systems will be placed on two adjacent simple spans of the sister I-59 bridges at Collinsville, AL and the design friendliness, construction friendliness, construction time, product and construction costs, structural performances, and projected long-term durability of each of the systems will be evaluated and compared. In turn, this will allow the ALDOT to make an informed and good decision on the best approach and systems to use on the required major rapid-deck replacement work coming up in Birmingham.

List of Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
AL	The State of Alabama
ALDOT	Alabama Department of Transportation
BIN	Bridge Identification Number
CIP	Cast-in-Place
CT	Vehicular Collision Force
DC	Dead Load of Structural Components and Nonstructural Attachments
FHWA	Federal Highway Administration
IM	Vehicular Dynamic Load Allowance
LL	Vehicular Live Load
LRFD	Load and Resistance Factor Design
NCHRP	National Cooperative Highway Research Program
NBR	Northbound Roadway
NU	Nebraska University
PC	Precast
SBR	Southbound Roadway
WL	Wind on Live Load

rehabilitation strategy for rapid replacement under concurrent traffic for the Birmingham bridge decks.

1.2 Research Objectives

The overall objectives of this Part 1 research were to develop effective and efficient design and construction procedures for four viable rapid bridge deck replacement systems which were identified by Auburn University researchers in an earlier phase of the investigation. These systems are:

1. Exodermic steel grid panels with CIP concrete topping
2. Standard steel grid panels with CIP concrete topping
3. Prefabricate deck panels made of Exodermic steel grids and precast concrete
4. Modified NCHRP precast concrete deck panels

Each of these four deck systems will be later placed on two adjacent simple spans of sister I-59 bridges over SR68 at Collinsville, AL (Part 2 research) and the design friendliness, construction friendliness, construction time, product and construction costs, structural performances, and projected long-term durability of each of the systems will be evaluated and compared.

The focus and objective of the research reported on herein is the accomplishment of the Part 1 work identified in the first paragraph above, i.e., to develop the designs and construction sequences and procedures for the four rapid deck replacement systems identified above for the Collinsville, AL “test bridges.”

deflection behavior of each of the four deck replacement systems with each other and with the currently existing deck system.

1.4 Scope

The report focuses on the design of each of the rapid deck replacement systems and their behavior under service loads. Each of the systems was designed based on a proposed sequence of construction which was developed in this report. The proposed sequence of construction was developed to better understand the appropriate bridge loadings occurring throughout the redecking construction process. Once the design loads were determined, the existing deck systems could be analytically checked to assure adequacy to resist these design loads. After the final designs for the deck systems were determined, the analytically evaluated behavior of each of the four deck systems were compared with each other and with the behavior of the existing deck system. No actual bridge deck replacement or load testing was conducted during this phase of the investigation.

1.5 Organization

In this report, Chapter 2 presents some background information on rapid replacement of bridge decks and reviews the literature on rapid strength gain concrete mixtures used by others or rapid deck replacement projects. Each of the four deck systems identified in Section 1.2 that are to be tested on the two Collinsville “test bridges” are described in Chapter 3. Chapter 4 discusses the proposed medications that will be implemented on each of the “test bridges”. Chapter 5 presents a proposed sequence of construction for each of the bridges. Each of the systems will need to be modified to carry the loads specific to Collinsville “test bridges”. These modifications

2. Background and Literature Review

2.1 Background

Rapid replacement of the decks on the Collinsville, AL "Test Bridges" will provide the ALDOT with much of the specific information that they need to make informed and good decisions about rapid deck replacement of the more than three linear miles of badly deteriorating interstate bridge decks in Birmingham, AL. Because of the high volume of traffic and the large square footage of deck replacement required in Birmingham, new redecking materials and construction sequences and procedures should be investigated and identified. Rapid redecking of the Collinsville, AL "Test Bridges" is a major part of this investigative work. Different prefabricated deck panel systems will be used in Collinsville, and in turn require different lane closure work periods for deck replacement, and in turn require different deck concrete mixtures designs in order to attain adequate concrete strength before reopening the closed lanes to traffic.

The four rapid deck replacement systems to be employed at Collinsville were identified in Chapter 1 and have been discussed in detail in earlier reports to ALDOT [3,4,5,6]. These same systems will be discussed in more detail in later chapters of this report. The recommended construction sequence and procedure for each system will also be discussed in detail in later chapters, as will the lane closure periods required to execute a reasonable portion of deck replacement for each of the deck systems for each lane closure period, and the times required to execute the main work tasks for each deck replacement system.

Type III Concrete	380 kg (640 lb)
Type C fly ash	42 kg (70lb)
Air content	6 ½%
Water reducer	xx
Water-to-cementing matls.	0.40

Strength data for this mixture is shown in Table 2.1.

Table 2.1 Strength Data for Fast-Track Bonded Overlay [1]

Age	Compressive strength, MPa (psi)	Flexural strength, MPa (psi)	Bond strength, MPa (psi)
4 hours	1.7 (252)	0.9 (126)	0.9 (120)
6 hours	7.0 (1020)	2.0 (287)	1.1 (160)
8 hours	13.0 (1883)	2.7 (393)	1.4 (200)
12 hours	17.6 (2546)	3.4 (494)	1.6 (225)
18 hours	20.1 (2920)	4.0 (574)	1.7 (250)
24 hours	23.9 (3467)	4.2 (604)	2.1 (302)
7 days	34.2 (4960)	5.0 (722)	2.1 (309)
14 days	36.5 (5295)	5.7 (825)	2.3 (328)
28 days	40.7 (5900)	5.7 (830)	2.5 (359)

In 1994 the New York State Department of Transportation (NYSDOT) appointed a committee to identify changes needed to improve the durability of their bridge decks [2]. The committee looked into three areas: design, materials, and construction. In the area of materials, they developed a high-performance concrete for bridge decks. The purpose of the development was not to attain higher strength concrete (although they did due to reducing the $^{w}/_{c}$ ratio), but rather to improve the concrete resistance to cracking, reduce permeability, and improve free-thaw durability and scaling resistance.

NYSDOT Class E concrete is the standard bridge deck concrete, with Class H concrete an allowable substitution in pumping placement applications. In developing their improved mixture, their current NYSDOT Class H Concrete was used as the

These are obviously very significant improvements; however, the 14-day wet cure requirements is a major contributor to these improvements and is not something that is consistent with rapid bridge deck replacement work.

Table 2.2. Concrete Mixtures for Laboratory Testing [2].

Material	NYSDOT Class H (Control)	NYSDOT Class HP (Mod. Class H)
Water (kg/m³)	167.2	160.1
Cement (kg/m³)	400.3	296.5
Fine Aggregate (kg/m³)	690.8	680.2
Coarse Aggregate[a] (kg/m³)	1038.9	1038.3
Class F Fly Ash (kg/m³)	0	80.1
Microsilica (kg/m³)	0	23.7
W/C Ratio	0.42	0.40
Total Cementitious Materials (kg/m³)	400.3	400.3
Air-Entraining Agent (mL/m³)	261	456
Set-Retarding Water Reducer (mL/m³)	1305	1305

[a]Laboratory mixtures used a coarse aggregate split of 50-percent No. 1 stone and 50-percent No. 2 stone.

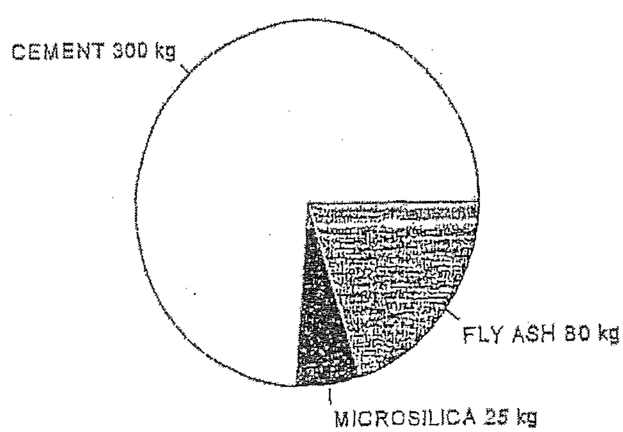


Figure 2.1. Cementitious constituents of NYSDOT Class HP concrete per m^3 [2].

Table 2.4 Performance of Concrete Mixtures in Laboratory Testing - Compressive Strength Results [2].

Mixture Name	Compressive Strength (MPa)					
	1 day	3 day	4 day	7 day	14 day	28 day
NYSDOT Class H	11.5		29.4	35.2	40.1	42.4
(Control)	11.8	-	30.9	35.9	40.1	42.4
NYSDOT Class HP	8.1	20.9		30.8	35.9	43.7
(Mod. Class H)	7.8	22.0	-	29.9	36.7	44.1

Table 2.5 Performance of Concrete Mixtures in Laboratory Testing - Permeability, Chloride, and Cracking Results [2].

Mixture Name	Rapid Chloride Permeability (Coulombs)	90-Day Chloride Ponding (ppm at depth)	Total Length of Slab Cracks[a]	Length per Area Conversion[b]
NYSDOT Class H (Control)	4349 4339	90 ppm @ 25 mm 0 ppm @ 50 mm 54 ppm @ 75 mm	1.150 m	5.800 m/m^2
NYSDOT Class HP (Mod. Class H)	1291 1249	36 ppm @ 25 mm 18 ppm @ 50 mm 36 ppm @ 75 mm	0.025 m	0.130 m/m^2

[a]No cracking formed after 4 ½ hrs; cracking reported is from measurements taken at 24 hrs; no further cracking developed.
[b]Bridge decks measured for cracking in the field average approximately 3 m/m^2 (98.75 in./yd^2).

This mix achieved the following early strengths:

1300 psi in 3hrs.
2050 psi in 4 hrs.
3080 psi in 5 hrs.

The NYSDOT recommended to premeasured and record all ingredients at the plant in a concrete truck approved for on-site dispensing of admixtures (pressurized pony tank), and if necessary additional HRWR may be added on site if needed to maintain a workable mix. It should be noted that it appears that the NYSDOT requires a minimum deck concrete strength of 3000 psi for allowing reopening to traffic.

The NYSDOT recently (2002) made a rapid bridge deck replacement employing Exodermic steel grid panels with a CIP concrete topping that they classify as a Class DP concrete. The proportions for this mixture are shown in Table 2.7. Note that this mixture employs smaller stone relative to their standard HP mixture (CA1 rather than CA2 or No. 67 rather than No. 57 stone). Also note that the mixture still uses 20% fly ash and 6% microsilica, but the total cementitious material is slightly more. This is probably due to using the smaller aggregate.

Table 2.7 Class DP Concrete for Exodermic Bridge Deck [2].

Material	Quantity
Cement (kg/m^3)	318
Fly ash (kg/m^3)	86
Microsilica (kg/m^3)	26
Total cementitious material (kg/m^3)	430
Sand - % of total aggregate (solid volume)	45.8
Maximum water/cementitious material ratio	0.40
Desired air (%)	7.5
Allowable air (%)	6.0 - 9.0
Desired slump (mm)	100
Allowable slump (mm)	75 - 125
Aggregate gradation (≈ No. 67 stone)	CA 1

NOTE: Criteria given for design information is based on a fine aggregate fineness modulus of 2.80. Determine the mixture proportions by using actual fineness modulus and bulk specific gravities (saturated surface dry for aggregate). Compute proportions according to Department written instructions.

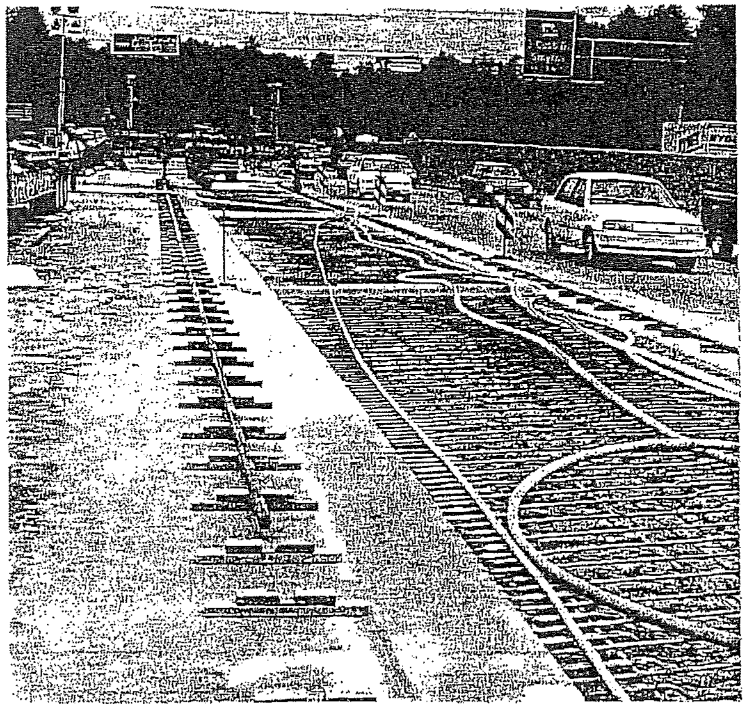


Fig. 2.3 Screed Rails Being Set and Vibratory Screed in Background Being Prepared [3].

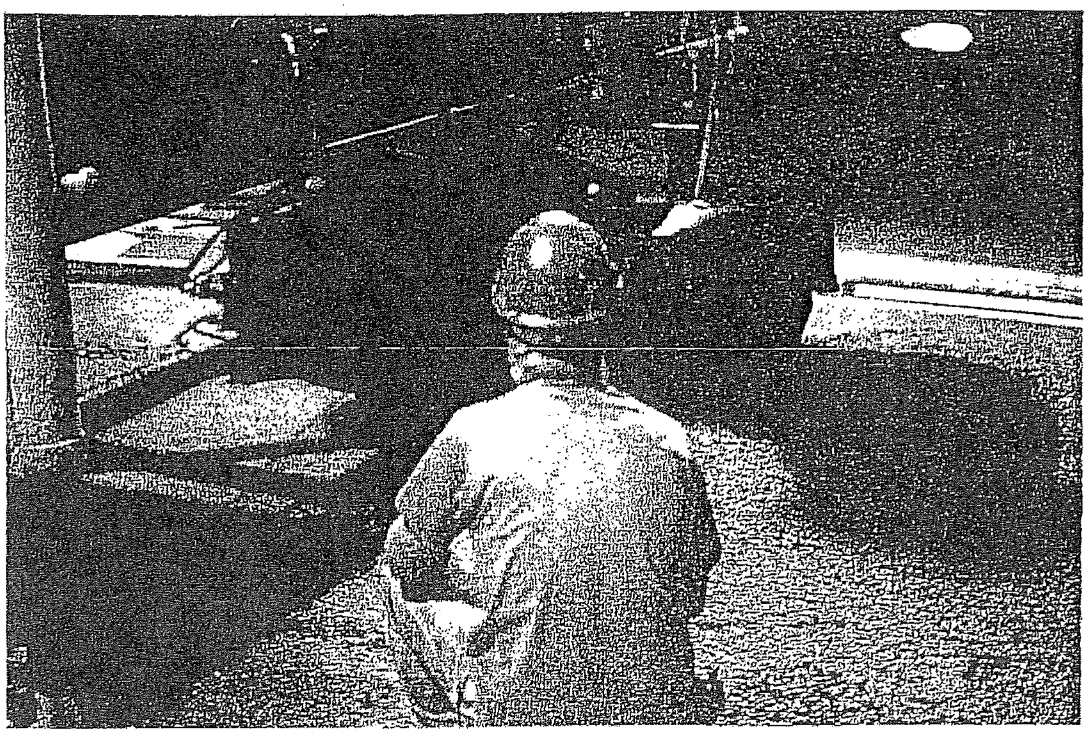


Fig. 2.4 Overlay Being Placed, Vibrated and Screeded [3].

Table 2.8 **Concrete Test Mixture Proportions [4].**

Component	Concrete Mixture			
	GA	GA-PF	GA-SRA	GA-IM
Cement Type III (lb/yd^3	750	750	750	680
Coarse Aggregate (#57 Limestone) (lb/yd^3)	1725	1725	1725	1874
Fine Aggregate (lb/yd^3)	1100	1100	1100	1152
HRWR[1] (ml/yd^3)	1350	1350	1350	1350
AEA[2] (ml/yd^3)	67.5	108	900	200
SRA[3] (ml/yd^3)	---	---	5670	---
Polypropylene Fibers (lb/yd^3)	---	3	---	---
Water (lb/yd^3)	292	292	280*	265
$^{w}/_{c}$ Ratio	0.39	0.39	0.39**	0.39

1. Glenium 3030 NS manufactured by Master Builders Technologies
2. MB AE-90 manufactured by Masters Builders Technologies
3. Tetraguard manufactured by Master Builders Technologies

* The amount of water was reduced to compensate for the addition of SRA

** The $^{w}/_{c}$ ratio due to the addition of the STA is also 0.39

Table 2.9 Fresh Concrete Property Testing Results* [4].

Concrete Property/Parameter	Concrete Mixture			
	GA	GA-PF	GA-SRA	GA-IM
Slump ASTM C143 (inches)	6.2	5.5	7.5	7.25
Unit Weight ASTM C138 (pcf)	155	155	155	151
Air Content ASTM C231 (%)	2.5	3.5	2.0	5.3
Concrete Temperature ASTM C1064 (ºF)	83	80	83	81
Initial Set-Time ASTM C191 (minutes)	81	85	100	77
Ambient Laboratory Temperature (ºF)	78	79	83	77

*Results shown are the average of two replica tests

A hardened property testing program was also conducted to examine certain characteristics of the concrete mixtures: compressive strength, split-tensile strength, unrestrained bar drying shrinkage, freeze-thaw durability, and rapid chloride ion penetration. Results from that testing are presented in Table 2.10 where the results shown are an average of two replica tests [4].

Table 2.11a. GDOT Fast Track Bridge Deck Overlay Concrete Summary Sheet

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 expected. The minimum acceptable temperature of concrete tat 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 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.

On the Tappan Zee Bridge in New York, the New York Throughway Authority replaced the existing deck in 1998 with precast Exodermic deck panels in a staged construction manner while working each night from 9:00 p.m. - 6:00 a.m. [3]. Figure 2.5 shows a construction crew setting the second of three Exodermic precast panels that they initially placed each night. Note in the figure the light sheet metal angles used to form the haunches at each girder lying on the girders. Figure 2.6 shows the workers pulling up and securing the light metal haunch forms. Note in this figure, one can see the existing deck in the foreground.

Figure 2.7 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. 2.7 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 ½") and that consolidation of the transverse joint concrete left something to be desired. Figure 2.8 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. the next day.

Fig. 2.7. Placement of Rapid-Setting Concrete at Stringers and Transverse Joints [3]

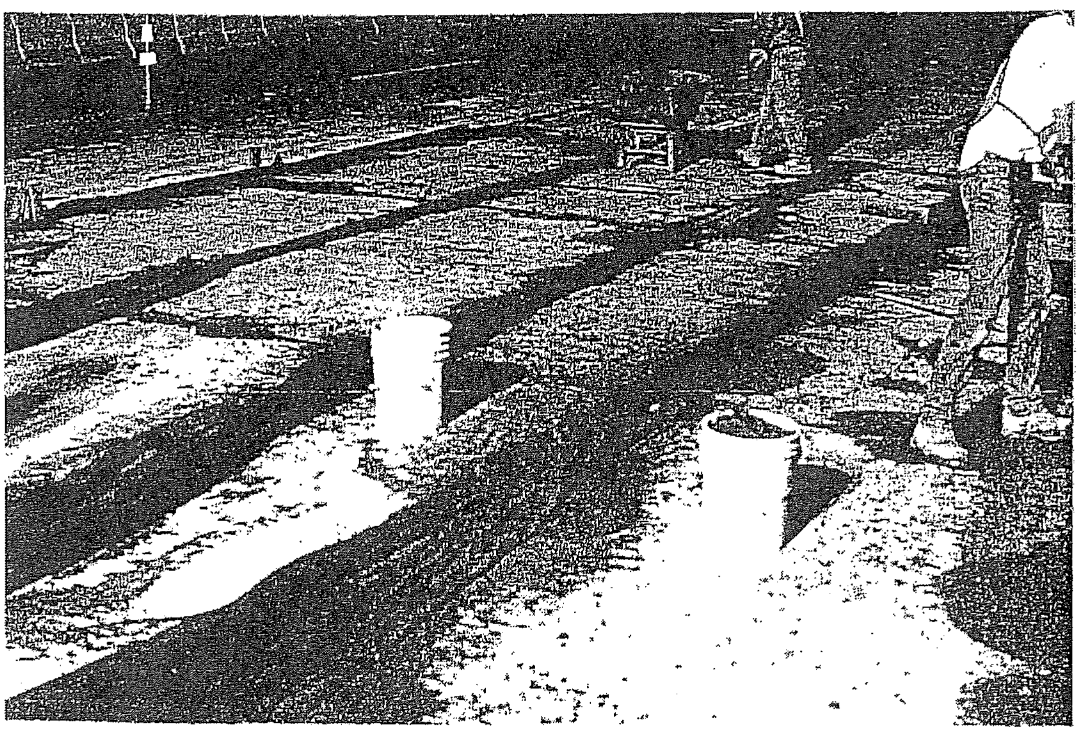


Fig. 2.8. Curing of Rapid-Setting Concrete at Stringers and Transverse Joints [3]

concrete mix proportions for the I-285 closure pours are shown in Table 2.12. The proportions are given in the first column of Table 2.12 yielded a batch size of 0.96 cubic yards, therefore a scaled proportion yielding 1.0 cubic yards was specified in the second column. The fresh concrete properties for the Portland Cement Concrete (PCC) mixture are shown in Table 2.13. Table 2.14 shows the compressive strength (psi) of the concrete relative to the concrete age. The concrete mixture design for the I-285 closure pours, which was designed by RMC-CEMEX, was furnished by GDOT.

During the period August 2005 - September 2005 the L.C. Whitford Co. replaced the original portions, i.e., two lanes in one direction and three lanes in the other direction, on sister I-285 bridges over Buford Highway in Atlanta, GA. The same precast Exodermic deck panels as used on the I-285 bridges over US 41 were employed as well as the same closure pours concrete and week-end construction procedure. However for the bridges over Buford Highway, the contractor had gained valuable experience with the bridges over US 41 and even though the bridges over Buford Highway had a little more square footage to replace (a total of 13,176 ft^2), the work was accomplished in only five weekend closures. Figures 2.9 - 2.13 show some of the concreting equipment and work in progress on the bridges over Buford Highway. Note in Fig. 2.9 that ready-mix concrete trucks were used for delivery of the closure pour concrete; however, the accelerator and corrosion inhibitor (DCI) was placed in the concrete mixture upon each trucks arrival at the bridge site. Figures 2.9 and 2.10 show the equipment used to pump the admixture into the concrete mixture. The amount of admixture added was four to six gallons for every yard of concrete. Figures 2.11 and 2.12 show placement of the closure pour concrete via both ready-mix truck and Georgia

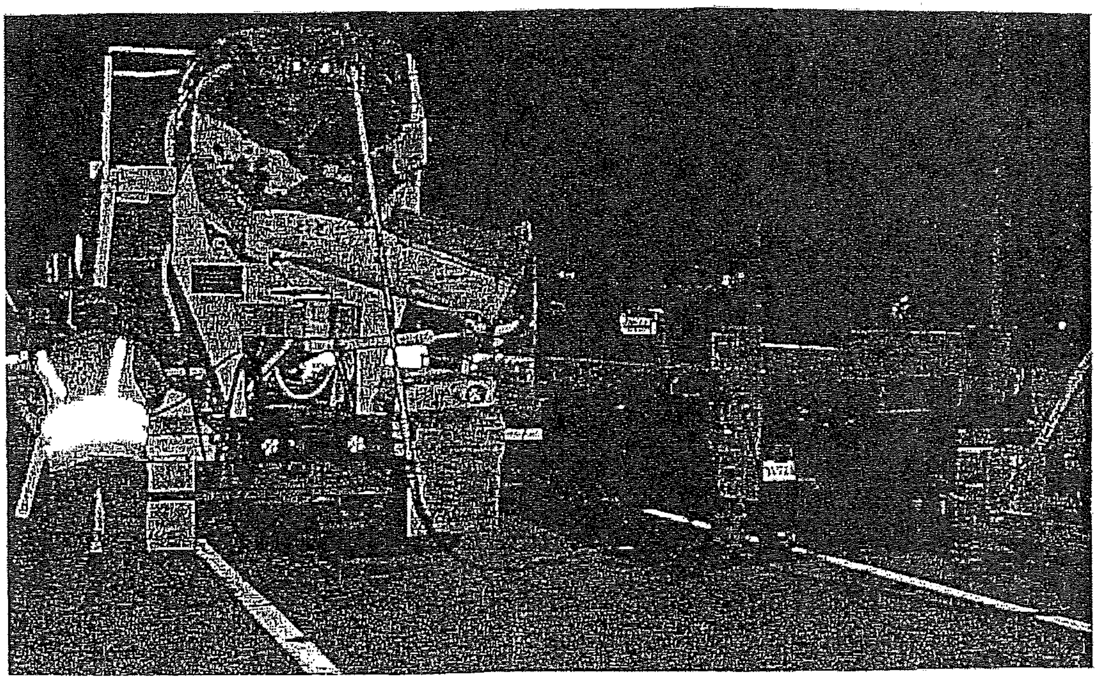


Fig. 2.9. Placement of the DCI Corrosion Inhibitor and Accelerator Admixture [5]

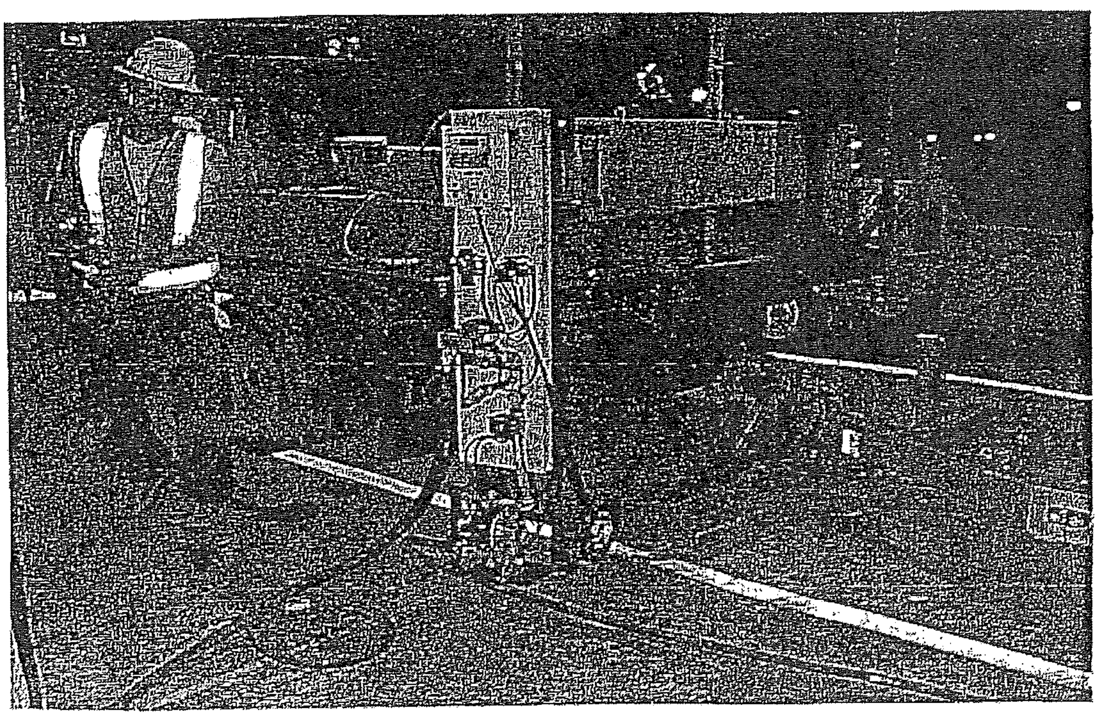


Fig. 2.10. Pumping Equipment Used for Admixture [5]

buggy. Figure 2.13 shows an overview photo of a newly replaced portion of the I-285 bridge over Buford Highway shortly after placement of the rapid-setting closure pour concrete.

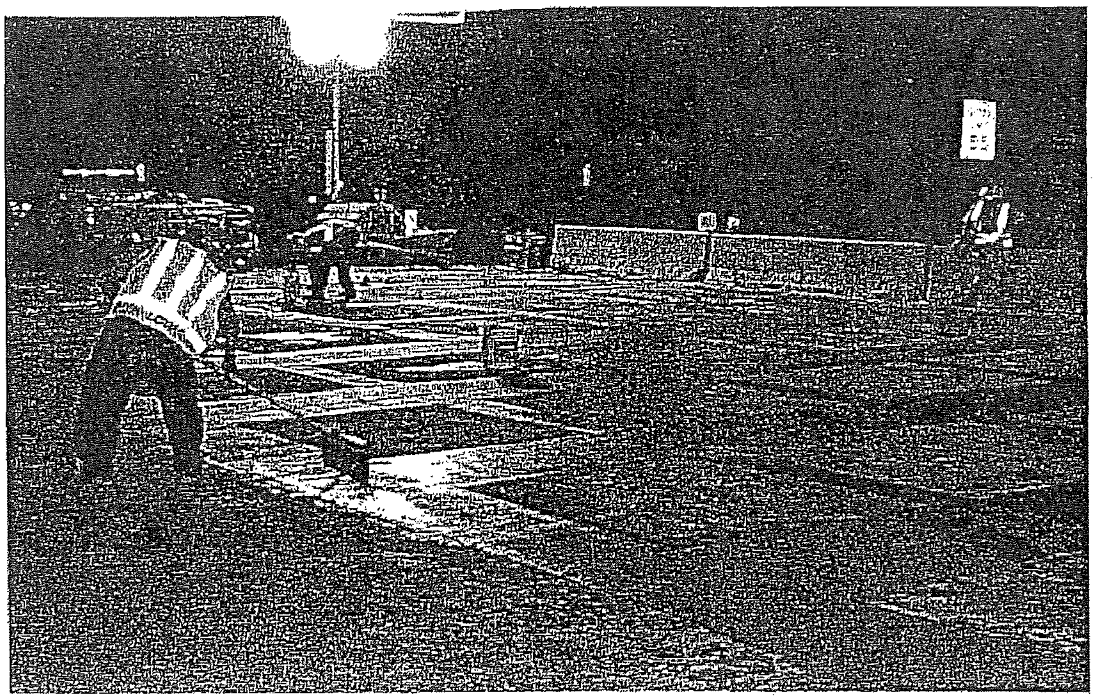


Fig. 2.13. Overview Shot of Longitudinal and Transverse Closure Pours [5]

3. DESCRIPTION OF COLLINSVILLE BRIDGES

3.1 General

Identifying a test bridge for the rapid deck replacement work proved much more difficult than anticipated. After considerable searching and discussions with ALDOT personnel, sister bridges on I-59 over SR68 at Collinsville, AL had been selected with the deck replacement work scheduled for FY2011. Collinsville is located approximately 20 miles northeast of Gadsden, AL. Pertinent information about the bridges is summarized below.

Bridge Location:	I-59 at Collinsville, AL (over SR 68)
BIN:	007536 (SBR) and 007537 (NBR)
Year Built:	1962
Design Load:	HS20-16
Number of Lanes:	2
Width:	28′ curb-to-curb
Deck:	6 ¼″ thick reinforced concrete (pea gravel concrete)
Support Girders:	Steel 36WF@150# (36-ksi yield strength) with Cover Plate
Girder Spacing:	8′ - 0″
Span Length(s):	4 simple spans (56′-10″/ 56′/56′/56′-10″)
Shear Studs:	¾″ dia. X 4″ Nelson studs
Diaphragms:	15 [33.9# spaced at 18′-4″

The I-59 sister bridges over SR68 at Collinsville, AL are steel girder bridges carrying interstate traffic. The bridges have girders acting compositely with the concrete deck. They are similar to most of the Birmingham, AL bridges that need rapid deck replacement, except the Birmingham bridges are much wider with somewhat longer span lengths.

The sister bridges are on a curve, but the curve is so small that it is considered negligible for the deck design. The panels will be designed as rectangles but will be placed as skewed as necessary to handle the slight curve. The cast-in-place portion of the decking will then shape the small curve. The horizontal alignment details for each of

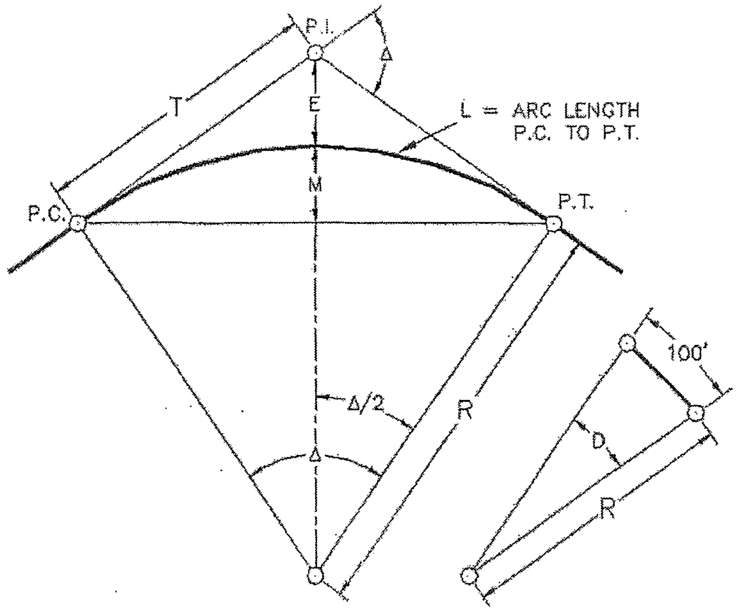


Figure 3.2 Horizontal Alignment Variables

Although they aren't shown in the figure, bearings transfer loads from the girders to the abutments and bents. The girders rest on top of the bearings. The existing girders have a yield strength of 36-ksi and are 36WF@150lb/ft steel sections. This section is similar to today's W36x150. The girders have a 28-ft cover plate welded at mid span to the bottom flange. A detail of this plate is shown in Figure 3.4. The girders are connected by 15C@33.9 lb/ft diaphragms. This section is similar to today's C15x33.9. Diaphragms transmit lateral loads, provide bracing for lateral torsional buckling, aid in gravity load distribution, and provide stability during construction. The diaphragms are spaced at 18′-4″ in each of the four spans.

The bridge decking is the next component of the bridge superstructure. The bridge decking is made composite with the steel girders through ¾" dia. x 4" Nelson shear studs as shown in Figure 3.5. A 6.25-in thick reinforced concrete was used for the decking of the Collinsville bridges. A typical cross-section showing the reinforcement bars is shown in Figure 3.6 and Table 3.1 shows the reinforcing bar details, Figure 3.6 also shows the existing superstructure diaphragms and their connection to the girders as well as the deck end beams and their support.

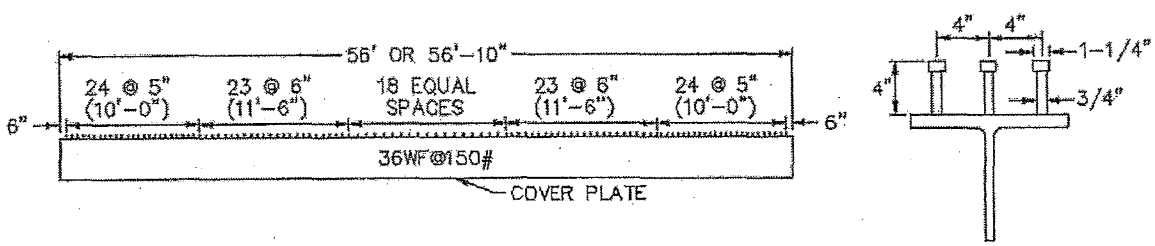


Figure 3.5 Shear Studs on Typical Bridge Girder

Table 3.1 Reinforcing Bar Details

BAR	SIZE	END SPAN LENGTH	INT. SPAN LENGTH	SHAPE
A	#5	31'-5.75"	31'-5.75"	
B	#5	30'-8.75"	30'-8.75"	
C	#5	28'-8.00"	28'-8.00"	
D	#4	36'-2.75"	35'-9.50"	
E	#5	36'-4.25"	35'-11.00"	
J	#5	5'-10.50"	35'-11.00"	
K	#6	36'-5.25"	36'-0.00"	

A plan view of the bridges is shown in Figure 3.7a. The primary importance of this figure is to show abutment, bent, and span numbers, deck drainage directions, and the general orientation of the bridges.

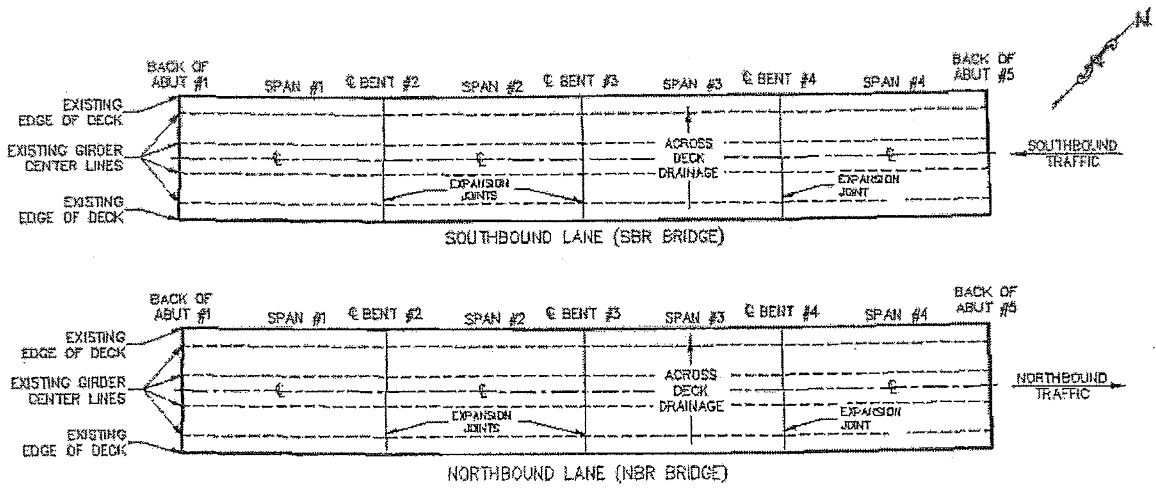


Figure 3.7a Plan View of Existing Bridges

The edge of the deck consists of a curb and a barrier rail as shown in Figure 3.6. A typical elevation view and section of the existing barrier rail is shown in Figure 3.7b. The test rating of the existing rail is unknown and is unnecessary for this investigation. The average weight of the existing barrier rail was estimated at 118 pounds per linear foot.

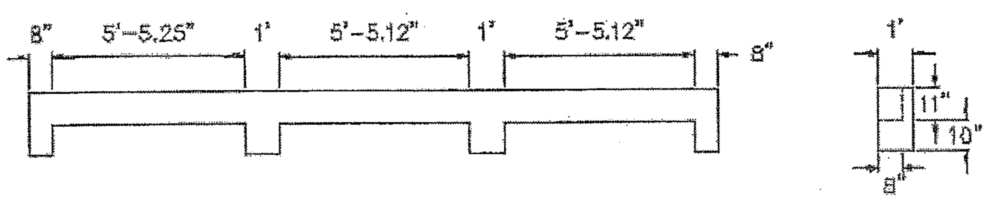


Figure 3.7b Typical Elevation View and Section of Existing Rail

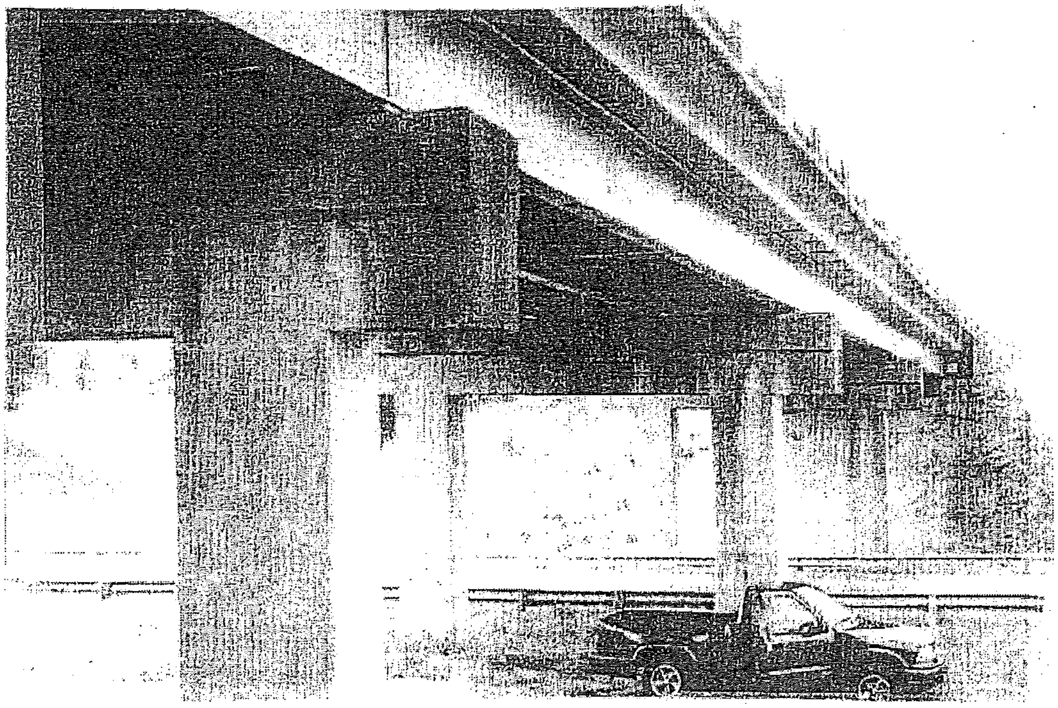


Figure 3.9 I-59 (SBR) Bridge Over SR68 at Collinsville

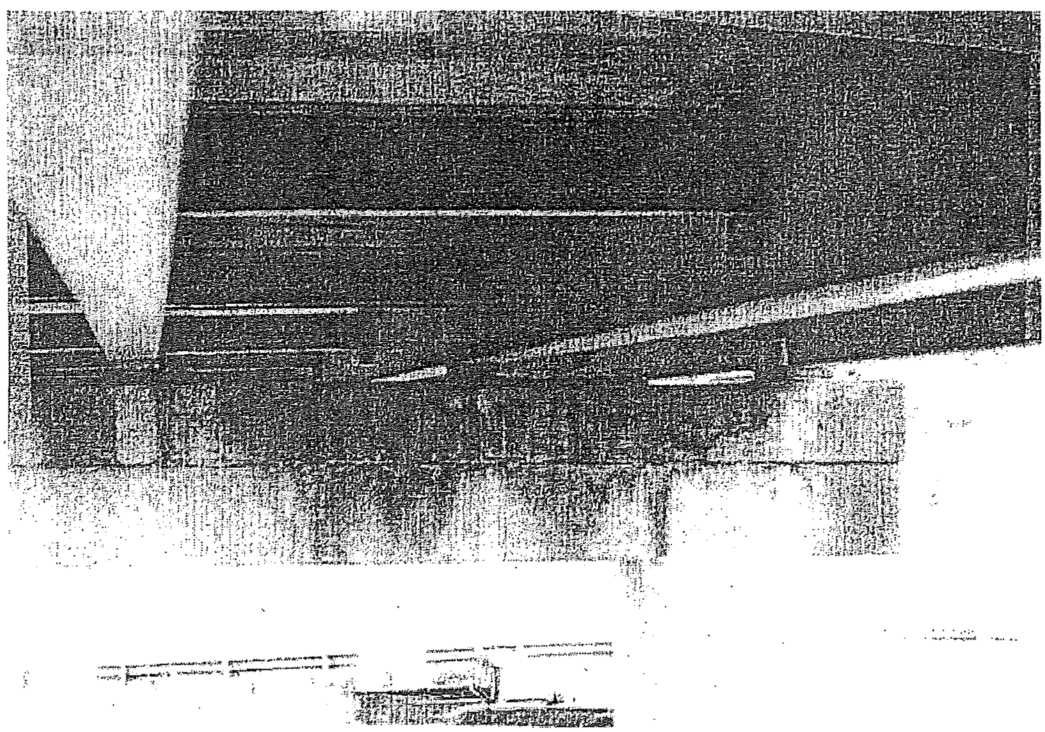


Figure 3.10 Underside View of I-59 (SBR) Bridge Over SR68

4. Description of Planned Rehabilitations of I-59 Sister Bridges at Collinsville, AL

4.1 Planned Bridge Widening

As evident in the figures and photos in Chapter 3, the existing Collinsville sister bridges are narrow (28′ curb-to-curb) two-lane bridges. In order to make the renovated sister bridges safer and to better replicate the condition of the Birmingham, AL bridges, ALDOT engineers, and the Auburn researchers agreed that it would be better to widen and add Jersey Barrier Rails to the Collinsville bridges while in the rapid deck replacement processes. Figure 4.1 shows cross-section views of existing and renovated I-59 sister bridges at Collinsville, AL. For the bridges/decks to be widened and new Jersey barriers added, then the abutments and support bents should be widened/prepared before beginning the deck replacement work. This substructure work can be done with minimum interference with I-59 or SR 68 traffic. When this work is completed, the I-59 bridge deck replacement work can begin and can be performed in two construction stages for each bridge, wherein one lane of traffic is maintained at all times and the CIP (NBR Bridge) or prefabricated (SBR) deck panels are field spliced near the bridge centerline as indicated in Fig. 4.2.

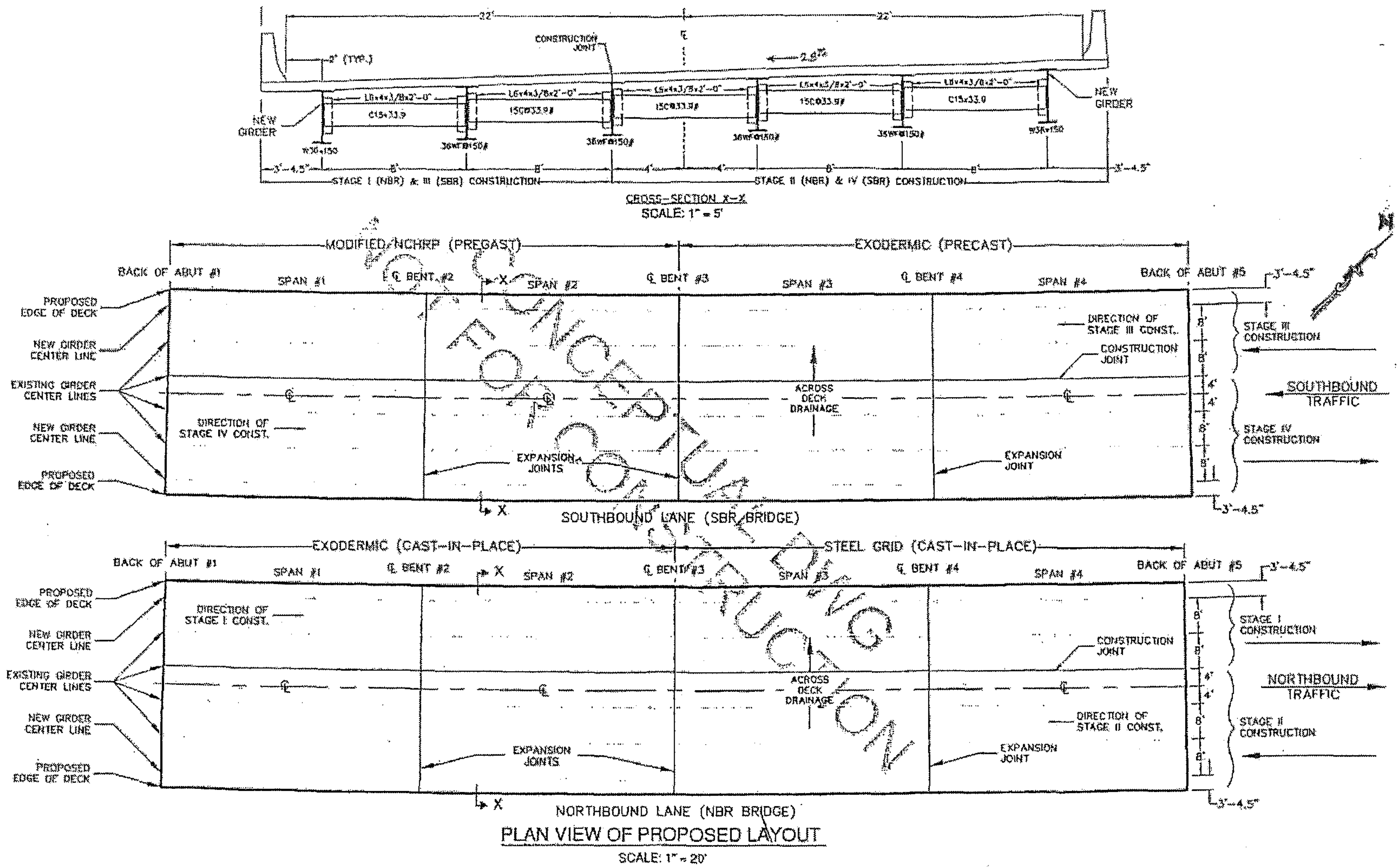


Fig 4.2 Planned Deck Rehabilitation and Construction Sequence

4.2.2 Exodermic Steel Panel with CIP or Precast Concrete Topping System.

Lightweight and modular, Exodermic bridge deck system have been used since 1984, and are increasingly being used to re-deck existing structures with minimal interruption to traffic.

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. The composite action is accomplished by the holes drilled into the upper portion of the main bearing bars. A isometric cut-away view of an Exodermic deck panels is shown in Fig. 4.3. The concrete portion of an Exodermic deck can either be cast-in-place (the steel grid panels act as the form work), or precast, where rapid construction is critical. On the Collinsville, AL “test bridges” we will employ both the CIP and the precast Exodermic deck panels to compare their construction friendliness. Photos showing workers installing Exodermic panels for CIP decks are shown in Fig. 4.4 and 4.5, and installing Exodermic precast panels are shown in Figs 4.6 and 4.7.

As with any precast concrete bridge units “the devil is in the detail” of the connections of the units with each other and with other components/units in the bridge. Figures 4.8 - 4.11 show drawings of the primary connections of the precast Exodermic deck panels with each other and with the supporting bridge girders.

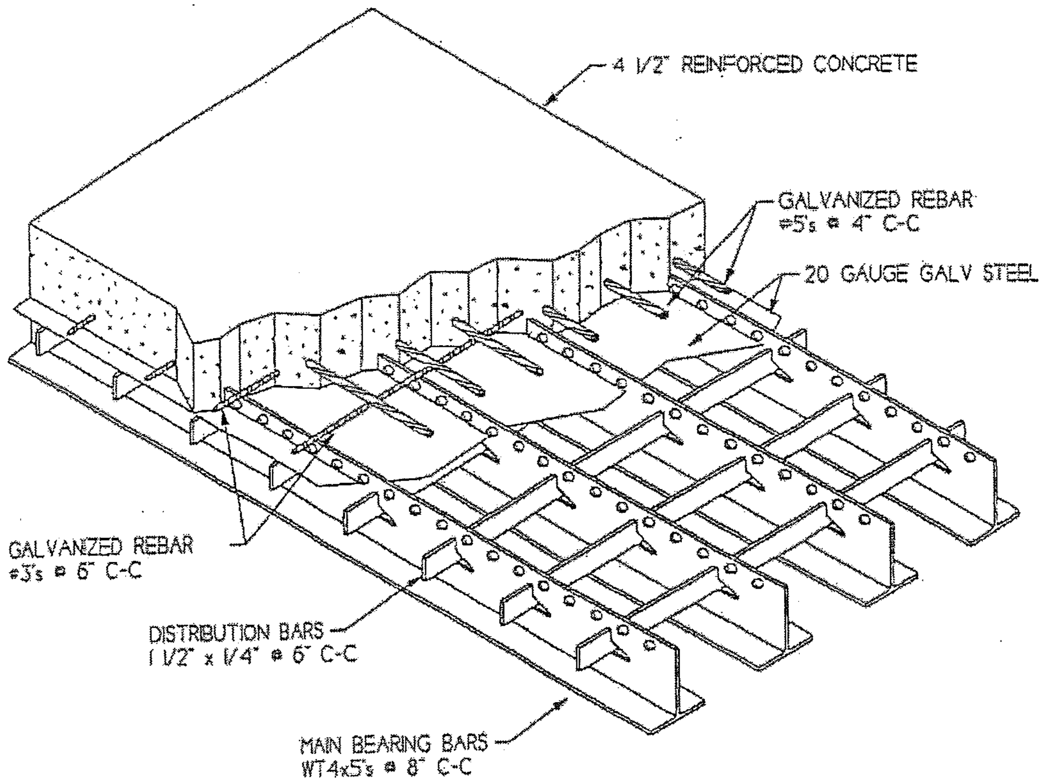


Figure 4.3 Isometric Cut-Away View of Exodermic Deck Panel (3)

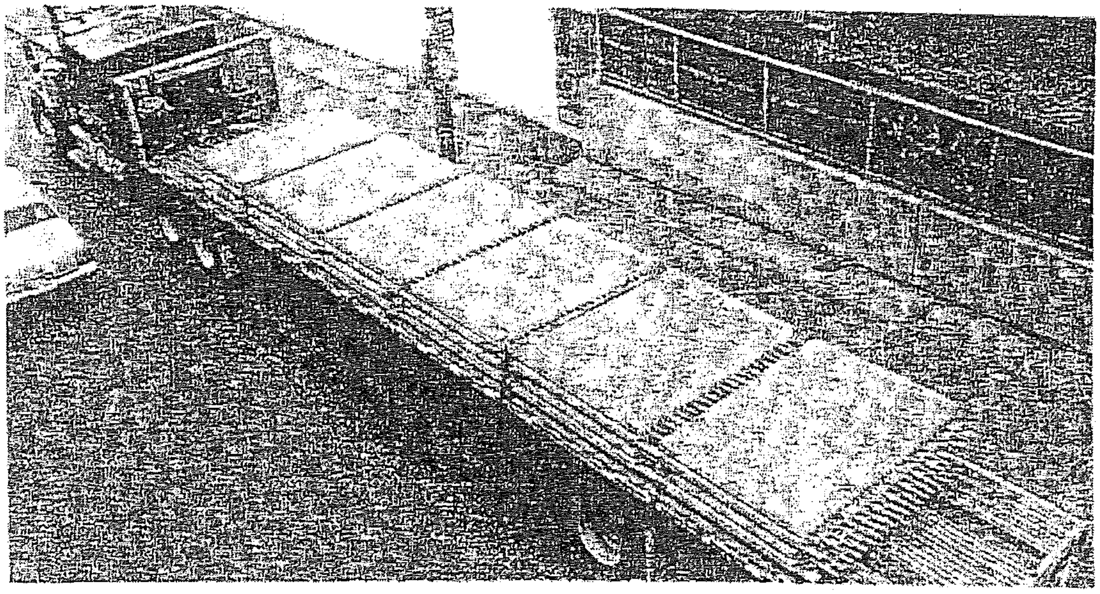


Fig. 4.6 38 ft x 8 ft Precast Exodermic Panels Being Delivered to Job Site (4)

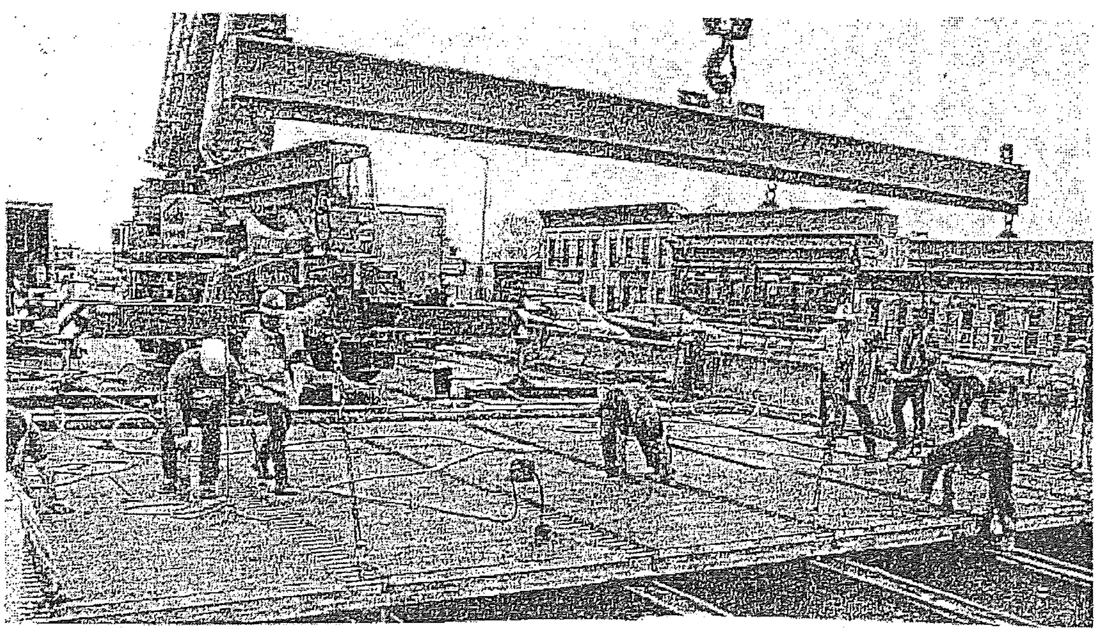


Fig. 4.7 Partial Weekend Closure Placement of 38 ft x 8 ft
Precast Exodermic Panels (4)

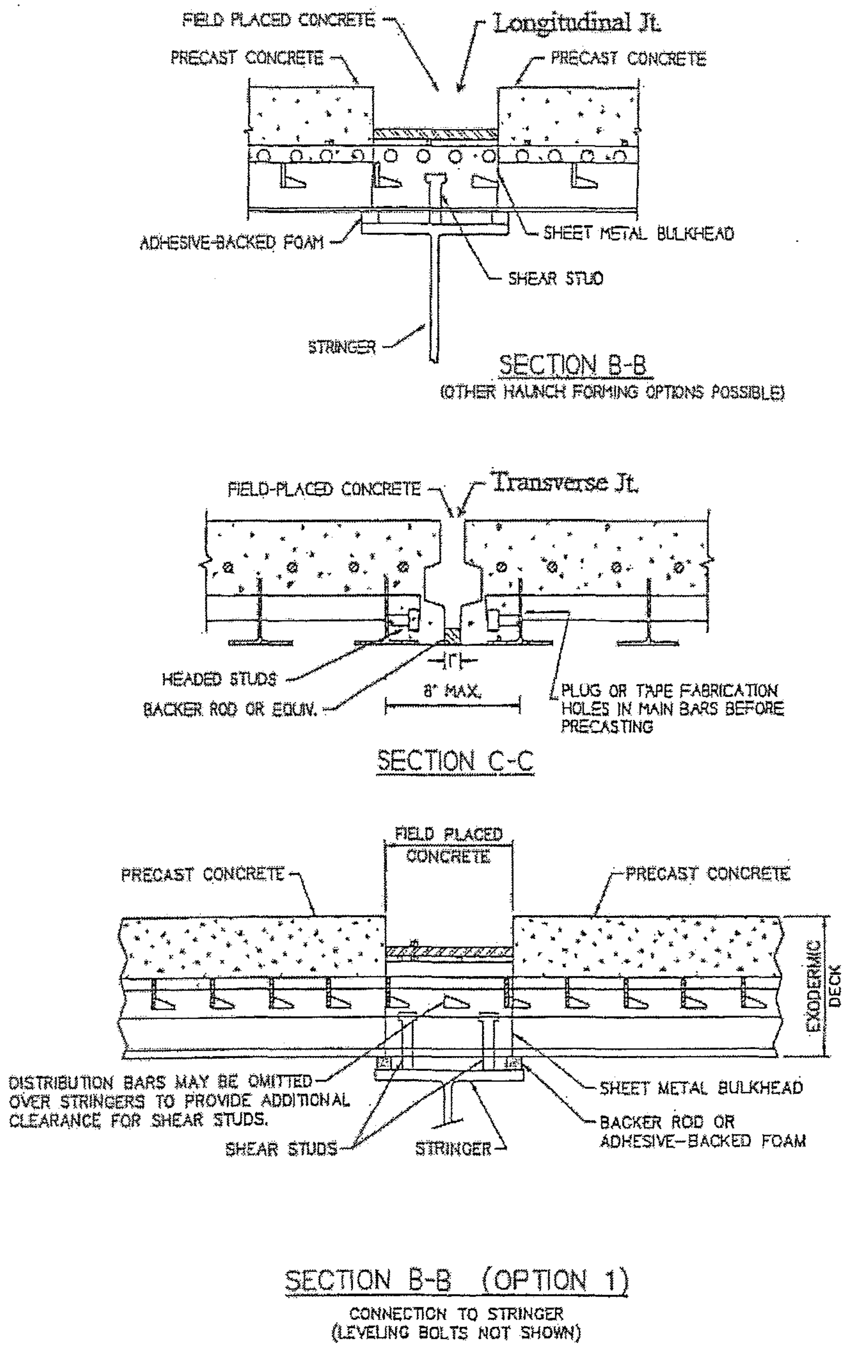


Fig. 4.9 Sections B-B (Longitudinal Joint) and C-C (Transverse Joint) for Precast Exodermic Panels (3)

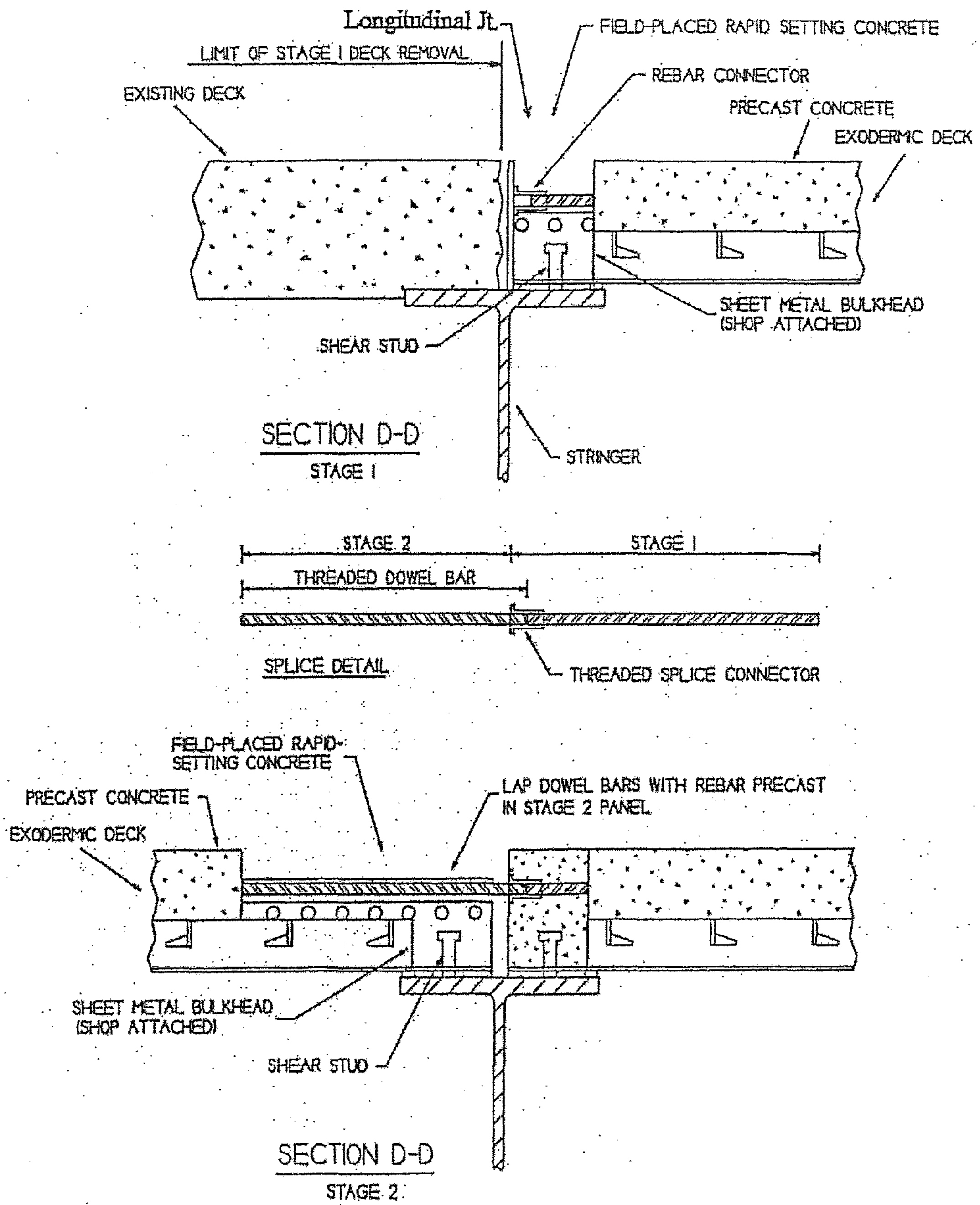


Fig. 4.11 Longitudinal Deck Splice Detail for Staged Construction with Precast Exodermic Panels

- 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

Deck panel connection details for the CIP steel grid system are shown in Appendix E.

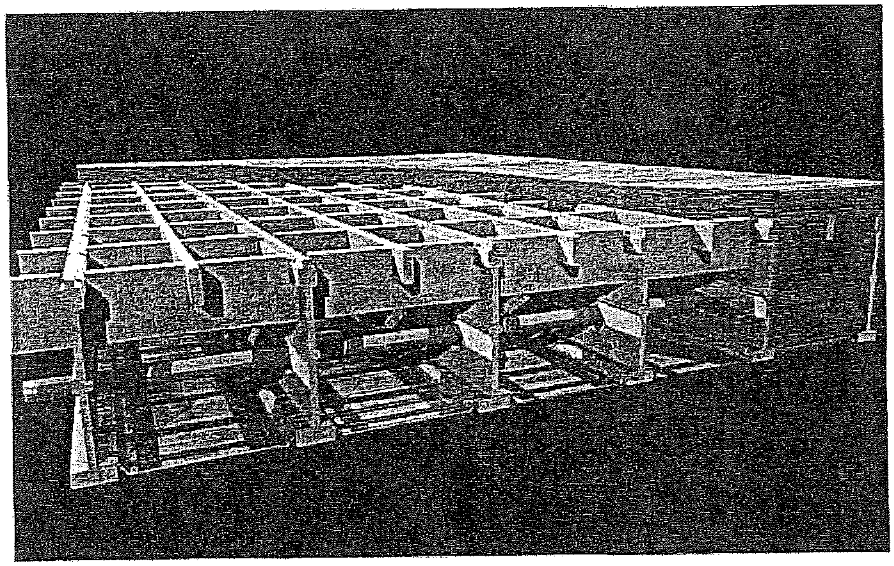


Fig 4.14 Steel Grid Panel – Full Depth Fill with Overfill

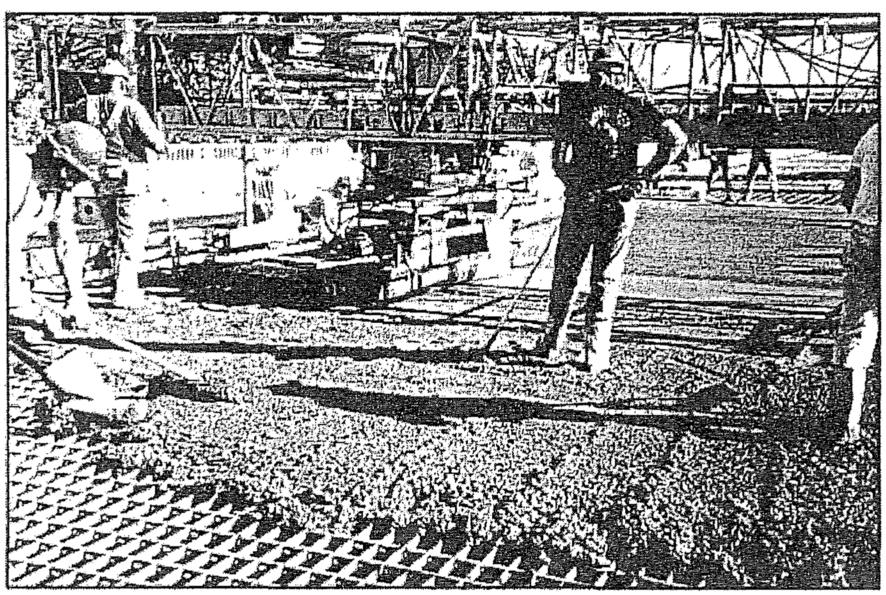


Fig. 4.15 Steel Grid Panel Deck Being Overfilled in Place

The primary advantages and disadvantages of fully precast concrete deck panels are given below.

Advantages:

- One-way prestressing of panel results in controlled deck cracking in the direction perpendicular to the prestressing and this results in little or no deck surface delamination which in turn results in reduced deck maintenance problems
- Slightly lighter weight than conventional CIP concrete decks
- Greatly reduces field construction time relative to that of conventional CIP concrete decks
- Estimated cost is probably slightly less than that of the other rapid deck replacement systems

Disadvantages:

- This deck system is new and unfamiliar to bridge designers, precast concrete manufactures and construction contractors
- Deck panels and system are approximately 30% heavier than the Exodermic precast system
- Due to their heavier weight, setting of deck panels may require a larger crane and/or more frequent movement of the crane which would cause somewhat greater construction time
- Will probably require at thin bonded overlay or grinding of deck top to achieve a good riding surface

Connection details for the modified NCHRP precast deck system are shown in Appendix E.

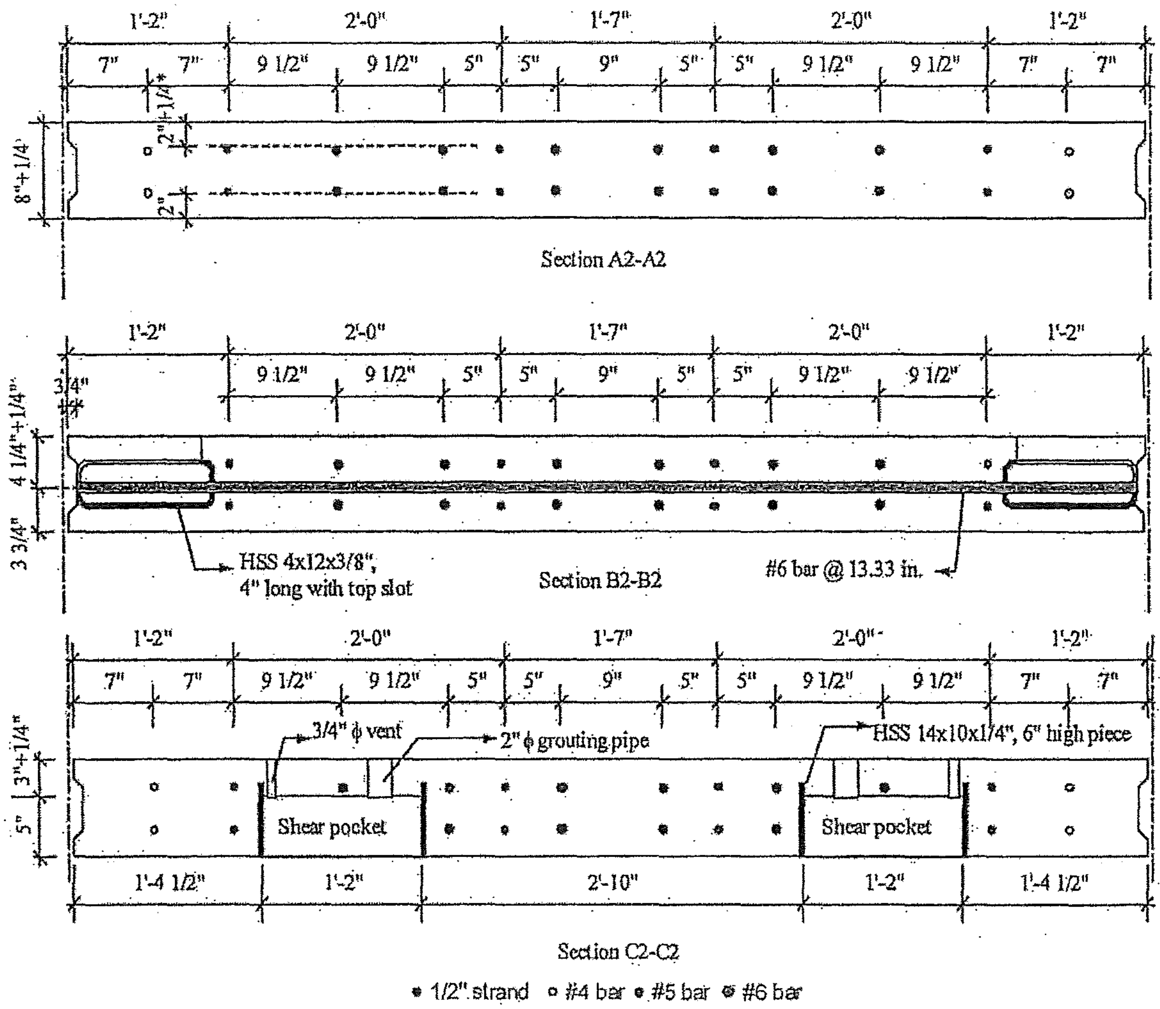


Fig. 4.17 CD-1B, Sections A2-A2, B2-B2 and C2-C2

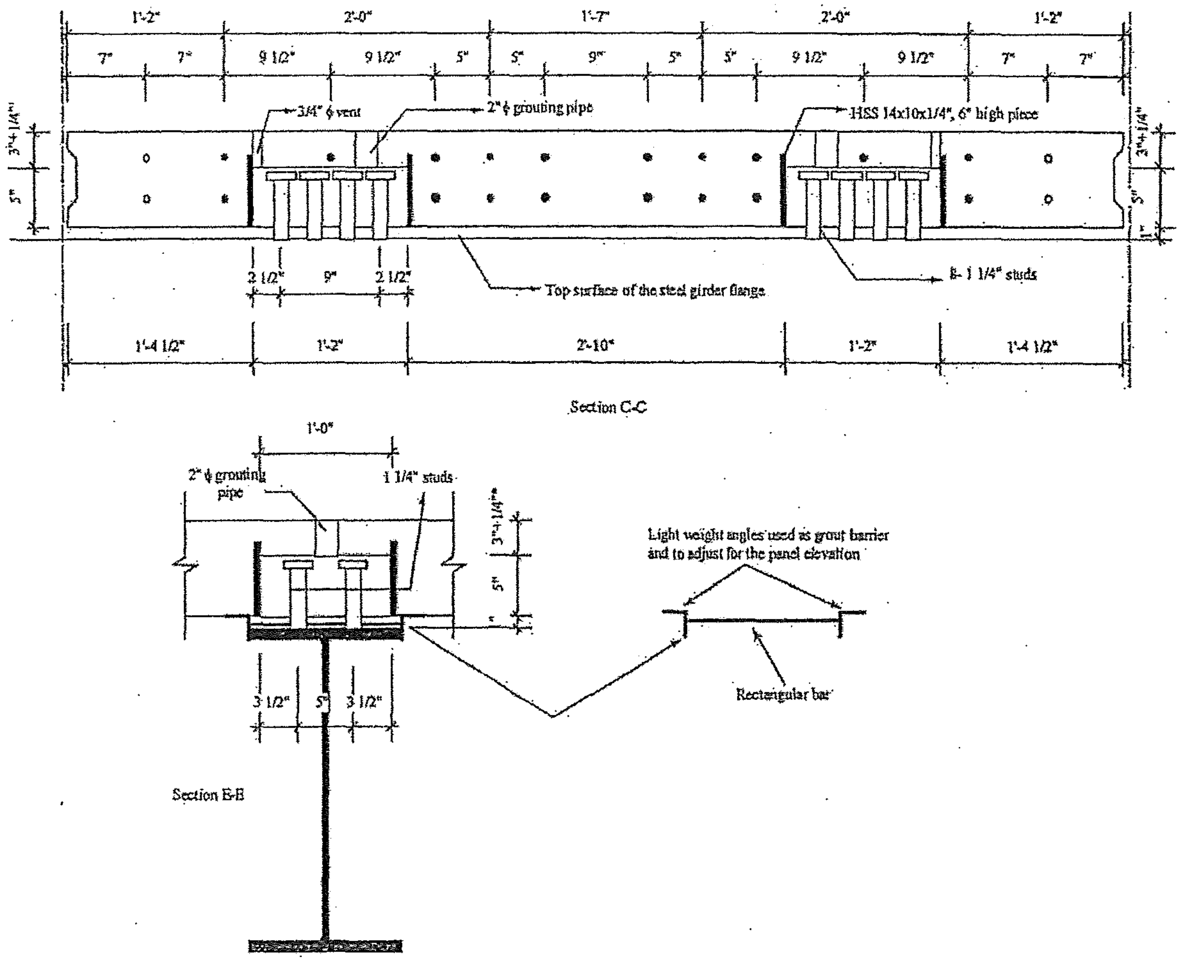


Fig. 4.19 Sections C-C & E-E for Steel Girders

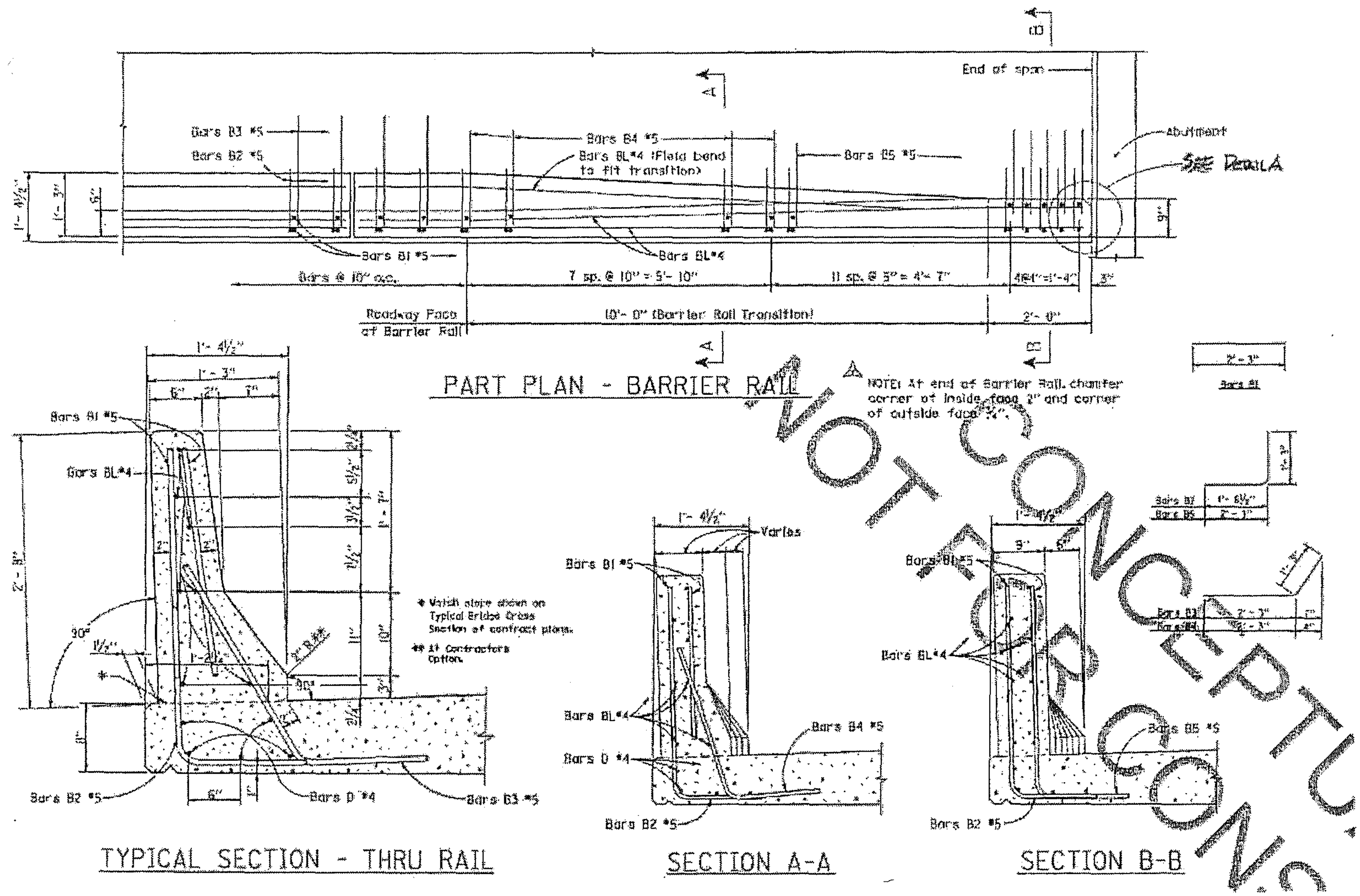


Fig. 4.20 CIP BARRIER RAIL

5. PROPOSED SEQUENCE OF CONSTRUCTION AND WORK-TIME SCHEDULES

5.1 General

Understanding the construction sequence is important for load calculations. Different loads apply to different cross-sections at different stages during the construction process. The deck systems must be design to withstand the loads carried throughout the construction process as well as the loads on the final cross-section.

The construction sequence for each of the bridges varies. The northbound bridge (NBR Bridge) will contain both of the cast-in-place systems while the southbound bridge (SBR Bridge) will contain both of the precast systems (see Figure 5.1). Each deck system will be used over two spans. The two cast-in-place systems are to be installed in Stages I and II on the NBR Bridge, while the two precast systems are to be installed in Stages III and IV on the SBR Bridge. Both of the bridges will also be widened during the construction process.

In describing Stage I through Stage IV construction sequences in the following sections, terminology such as outside lane, inside lane, direction of traffic, and other bridge/traffic conditions are referred to. These are explained below.

1. Outside lane is the lane away from the median strip, i.e., the right lane when looking in the direction of traffic
2. Inside lane is the lane adjacent to the median strip, i.e., the left lane when looking in the direction of traffic
3. Work shall progress in the direction of traffic for Stage I & Stage III construction (replacing the inside lane deck)

The first task is to install new girder lines on the north side of the bridge for all four spans as shown in Figure 5.2. Traffic is maintained in both lanes during this task. Task 2 is to place temporary barriers for the full length of the bridge and close down the inside lane of traffic as shown in Figure 5.3. Once traffic is closed to the inside lane, the inside lane portion of the existing deck shall be demolished for the full length of the bridge as shown in Figure 5.4. This is Task 3 of Stage I. The new Exodermic deck panels can now be installed in Span 1. This is Task 4 of Stage I and is accomplished by executing the following for the full length of Span 1. Working in the direction of traffic, place the unfilled Exodermic steel deck panels, place deck top reinforcement mat, place span end steel plates, and then place rapid-setting concrete on the deck panels. This is depicted in Figure 5.5. This same process is then repeated for Span 2 as shown in Figure 5.6 for Task 5. The next task, Task, 6, is to install the barrier rails for Spans 1 and 2. Working in the direction of traffic, place the barrier rail reinforcing steel, place the barrier slip forms, and cast the barrier rail concrete. This is shown in Figure 5.7. Tasks 4 through 6 are then repeated for Spans 3 and 4 using standard steel grid panels rather than Exodermic panels as shown in Figure 5.8. This is Task 7 of Stage I and concludes the Stage I construction.

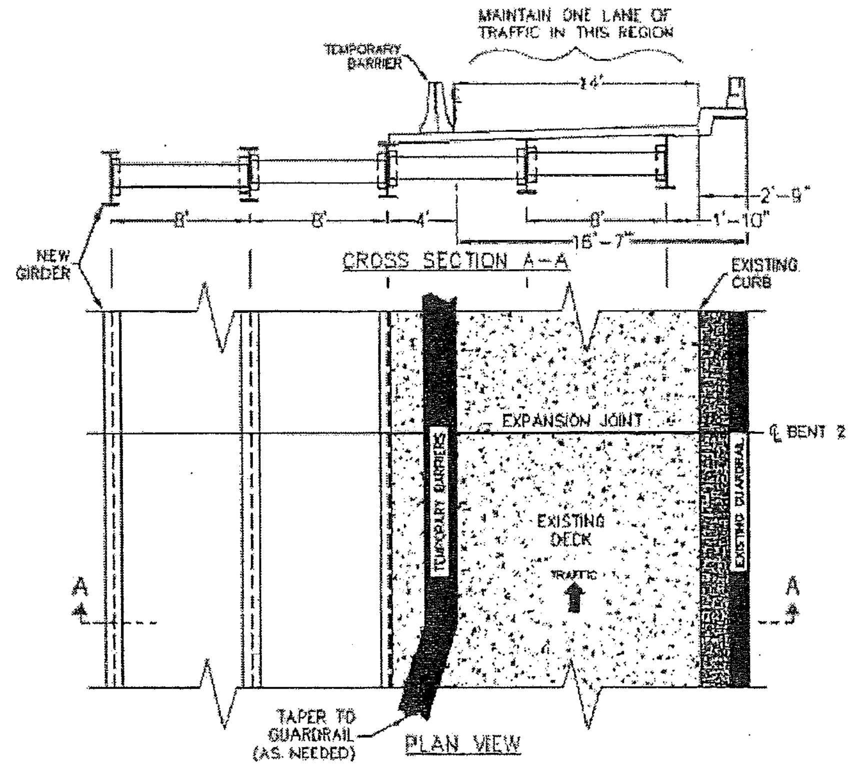


Figure 5.4 Construction Sequence, Stage I - Task 3

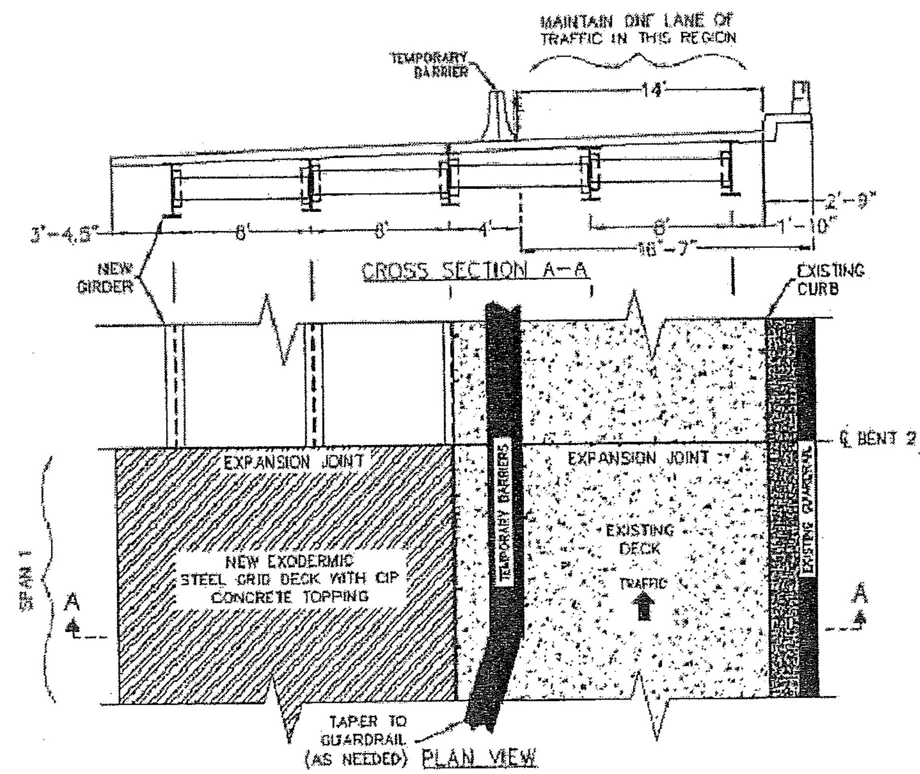


Figure 5.5 Construction Sequence, Stage I - Task 4

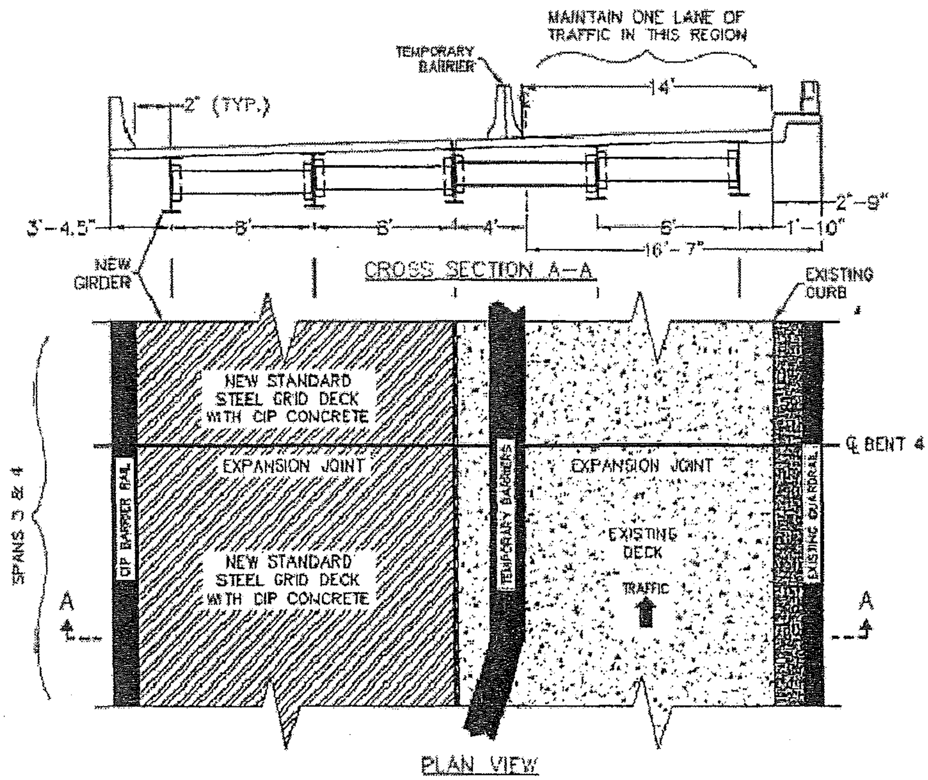


Figure 5.8 Construction Sequence, Stage I - Task 7

and 4 once the new decking is in place. This is Task 8 and is accomplished by placing the barrier rail reinforcing steel, placing the barrier slip forms, and casting the barrier rail concrete. This is shown in Figure 5.15. Tasks 2 through 8 are then repeated for Spans 1 and 2 using steel Exodermic deck panels rather than the standard steel grid panels. This is Task 9 and is shown in Figure 5.16. After Task 9 is completed, all temporary barriers and equipment is to be removed and traffic is to be opened to two lanes. This is Task 10 and concludes the Stage II construction on the NBR Bridge. The final view of the northbound bridge is shown in Figure 5.17. The construction on the northbound bridge is now complete.

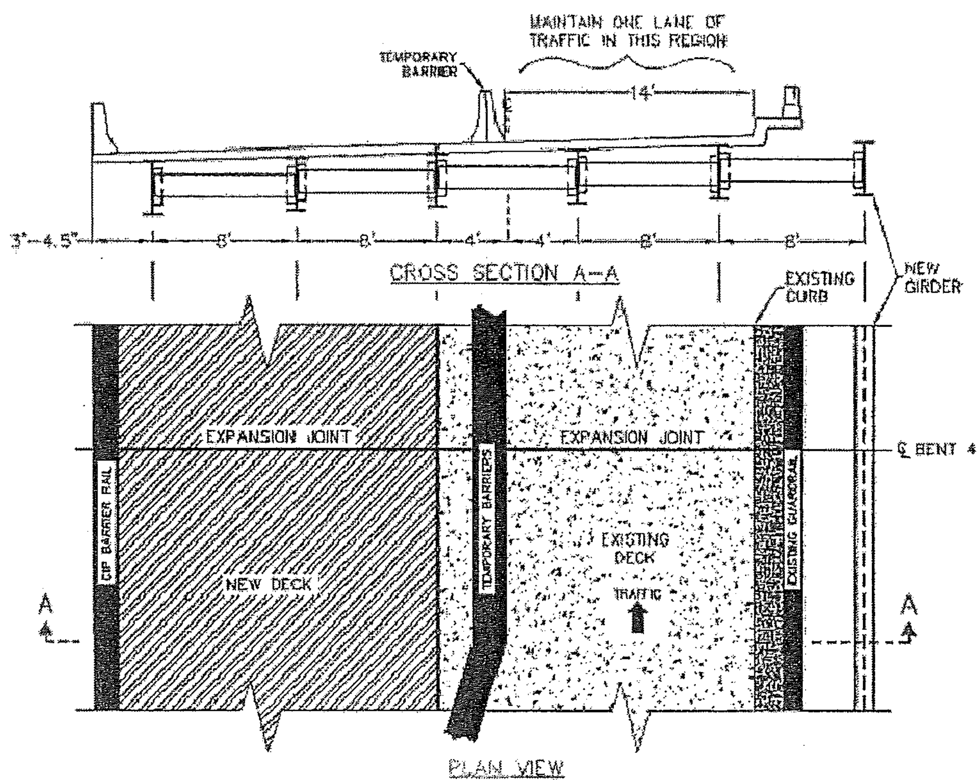


Figure 5.9 Construction Sequence, Stage II - Task 1

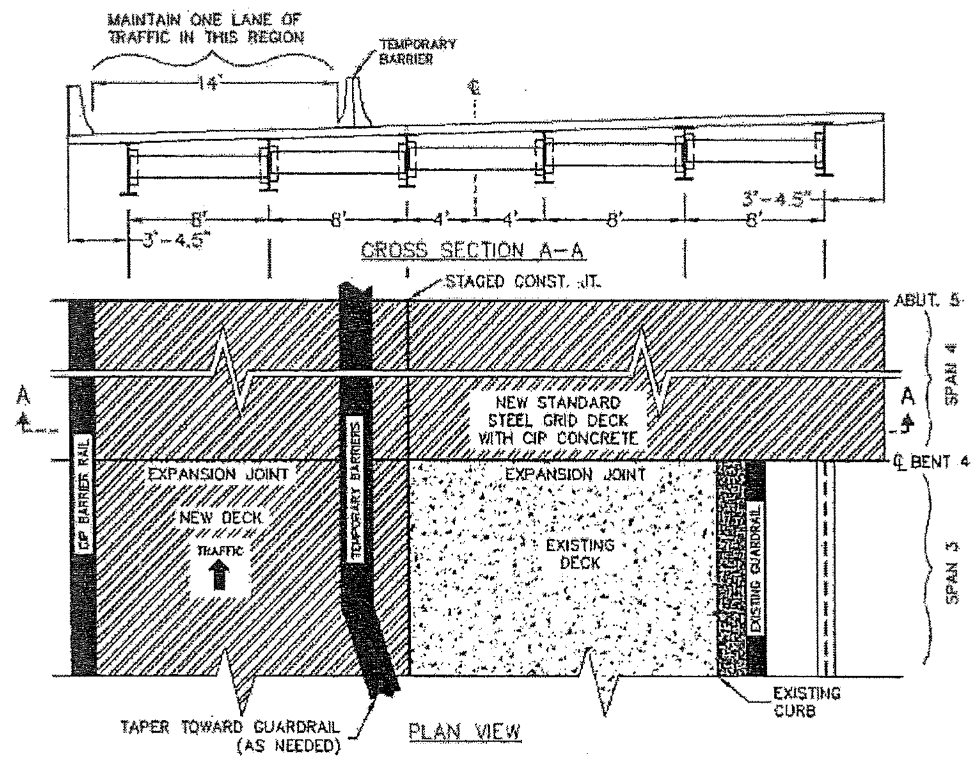


Figure 5.12 Construction Sequence, Stage II - Task 5

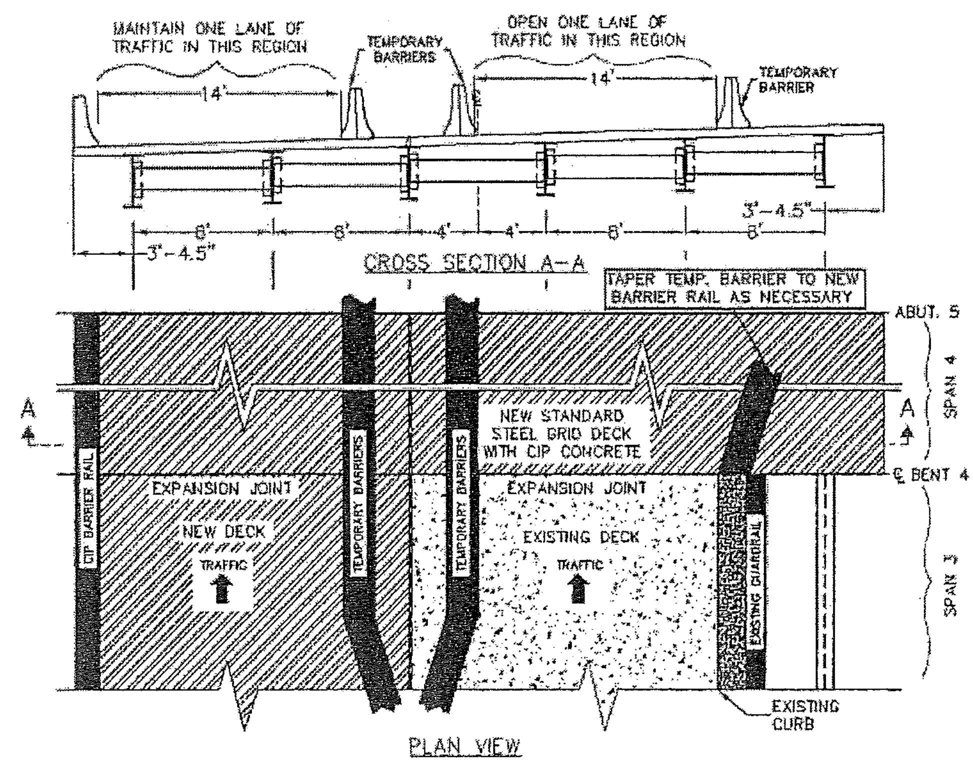


Figure 5.13 Construction Sequence, Stage II - Task 6

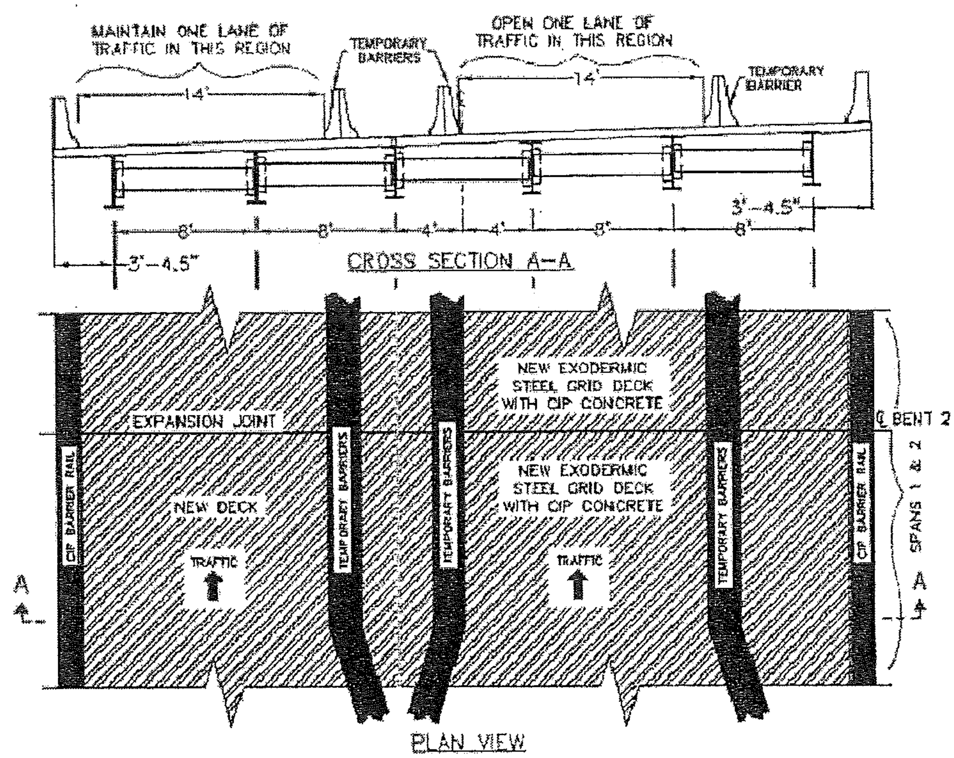


Figure 5.16 Construction Sequence, Stage II - Task 9

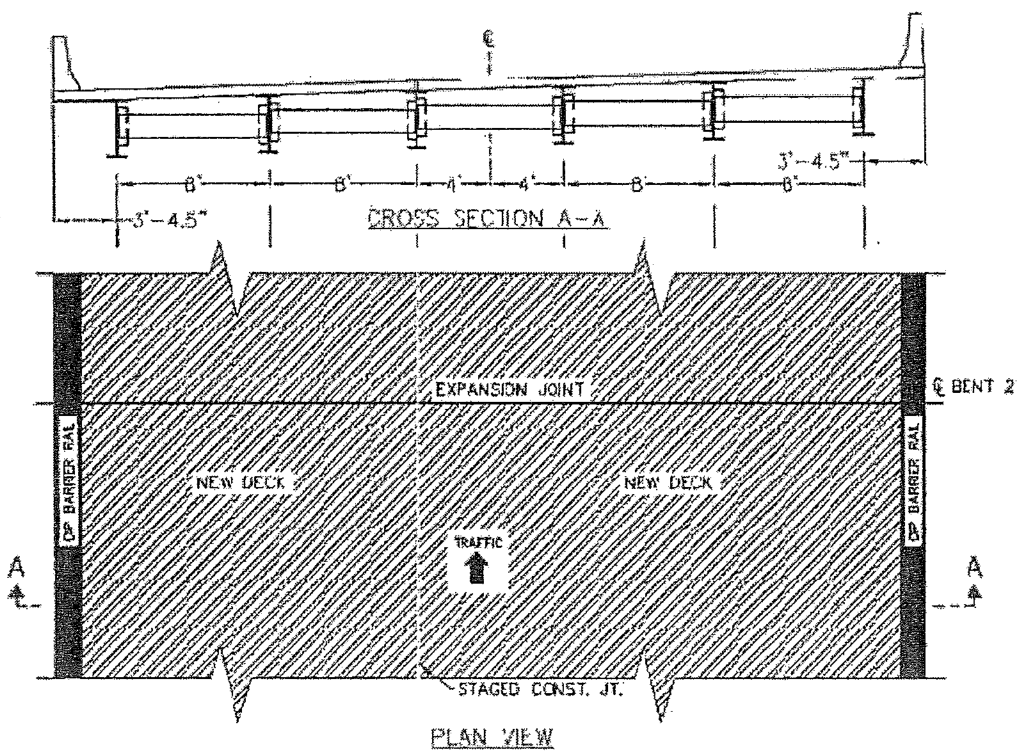


Figure 5.17 Construction Sequence, Stage II - Task 10

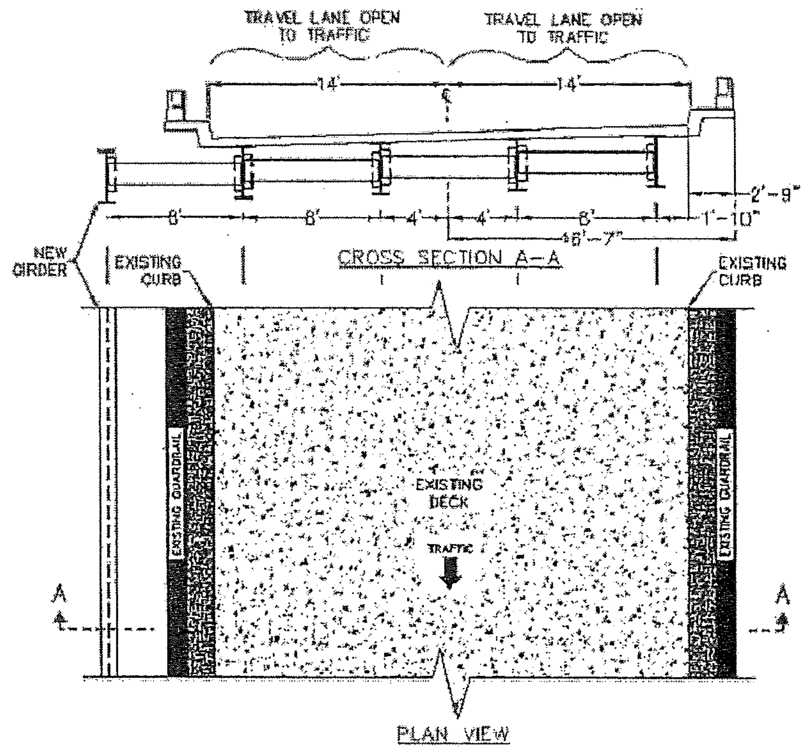


Figure 5.18 Construction Sequence, Stage III - Task 1

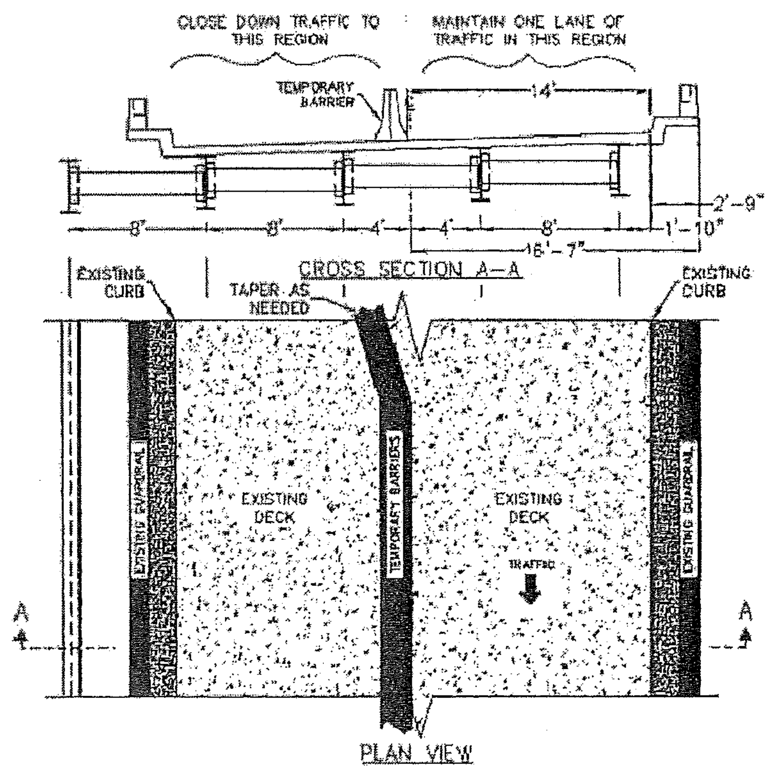


Figure 5.19 Construction Sequence, Stage III - Task 2 & 3

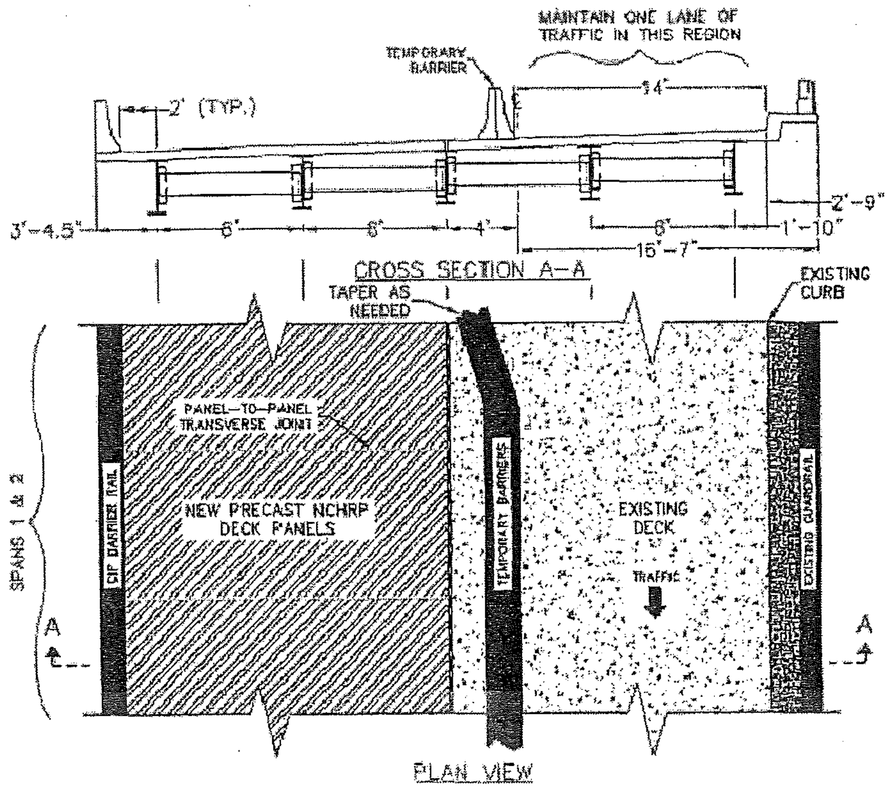


Figure 5.22 Construction Sequence, Stage III - Task 6

5.5 Stage IV Construction

The existing deck of the inside lane portion of the southbound bridge is replaced in Stage IV. Figures 5.23 through 5.36 show the different tasks to be completed during the Stage IV construction. Work for this stage shall progress in the direction opposing traffic. A description of these tasks and work are as follows.

The first task of Stage IV is to install a new girder line on the south side of the SBR bridge for all four spans as shown in Figure 5.23. Tasks 2 and 3 are to relocate and add temporary barriers to redirect traffic from the inside lane to the outside lane. This will close down traffic to the inside lane so the existing deck can be replaced. This is shown in Figure 5.24. The next task, Task 4, is to demolish as much of the existing deck of the outside lane as can be replaced during the next work period as illustrated in southbound bridge is shown in Figure 5.36. The construction on the southbound bridge is now complete.

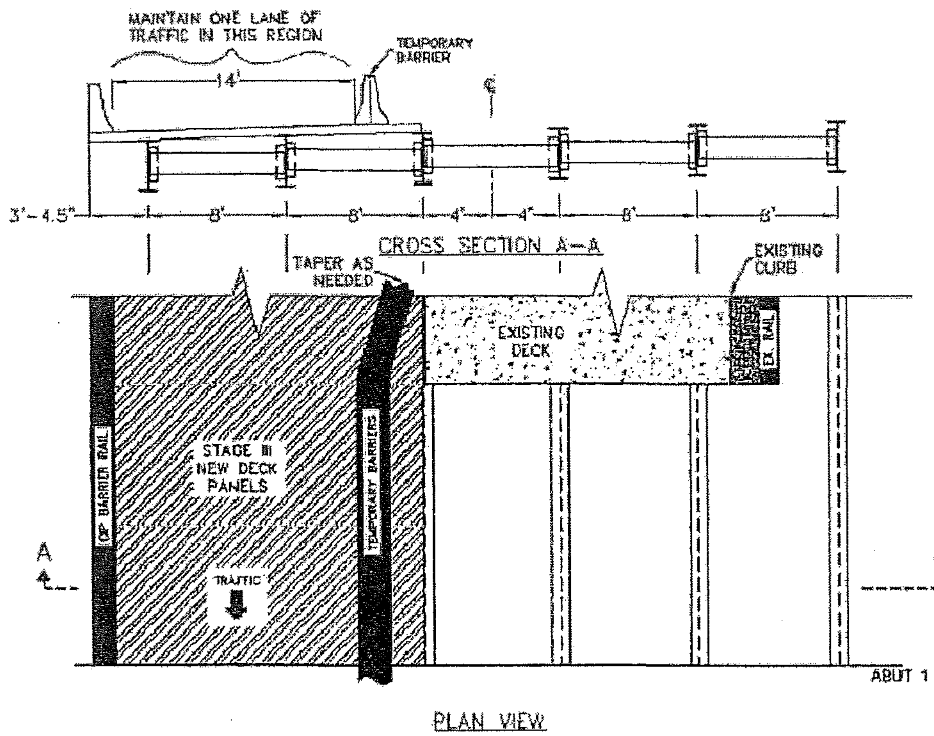


Figure 5.25 Construction Sequence, Stage IV - Task 4

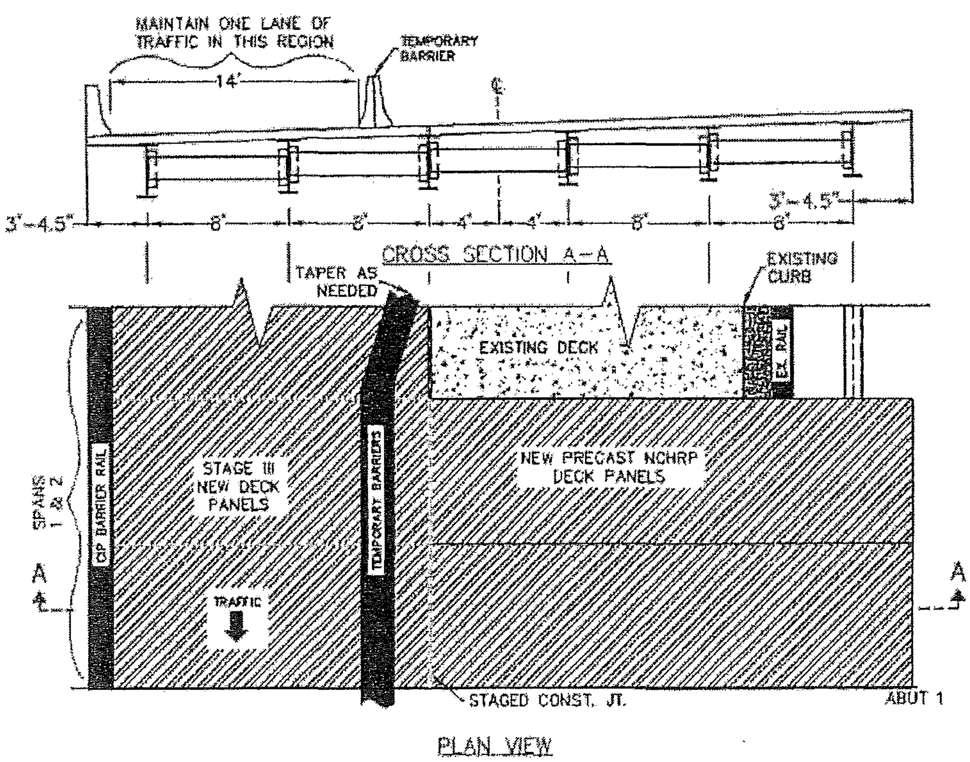


Figure 5.26 Construction Sequence, Stage IV - Task 5

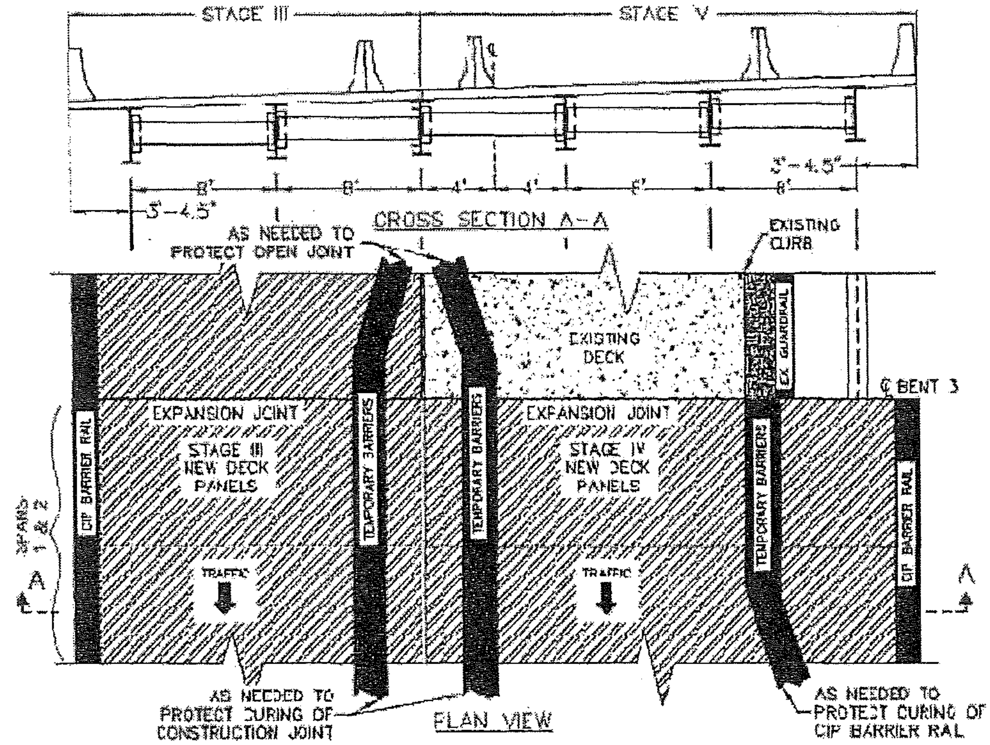


Figure 5.29 Construction Sequence, Stage IV - Task 8

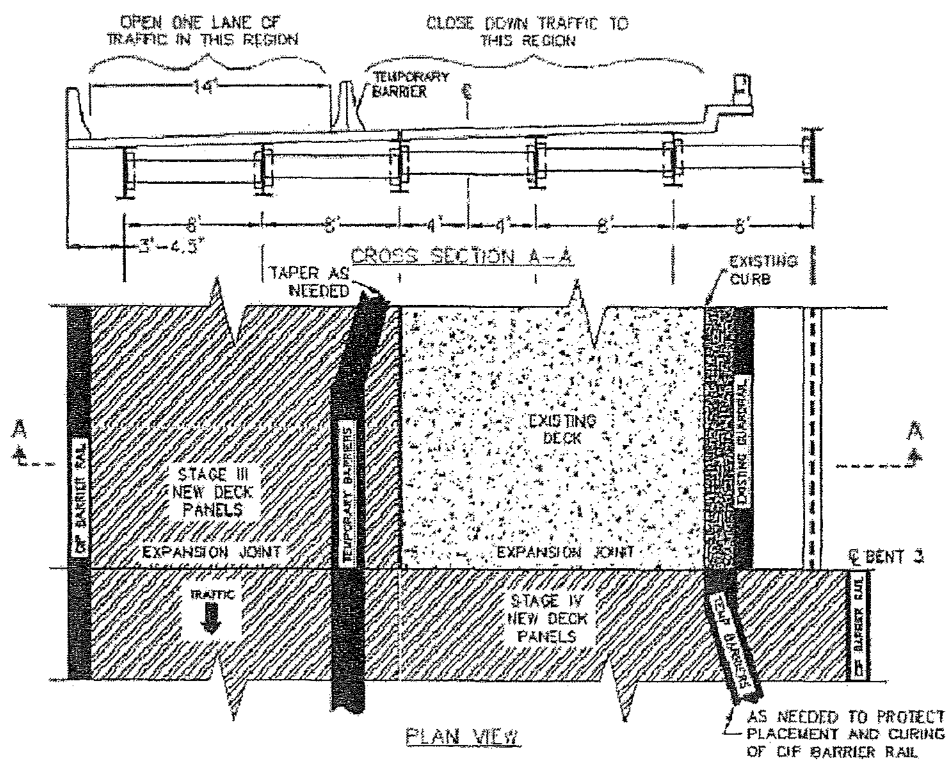


Figure 5.30 Construction Sequence, Stage IV - Task 9 & 10

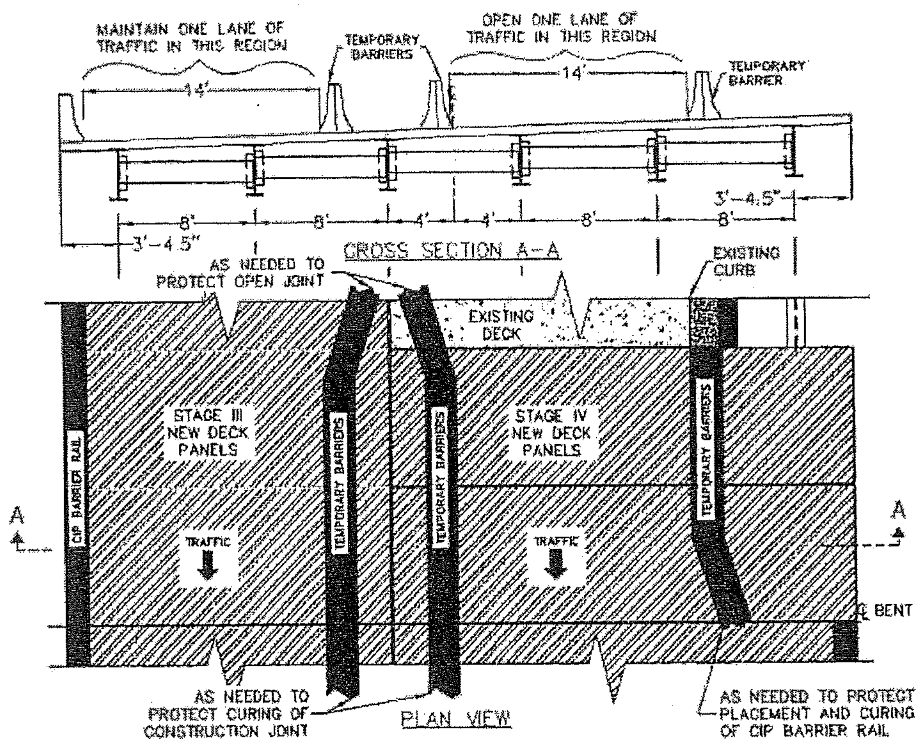


Figure 5.33 Construction Sequence, Stage IV - Task 13

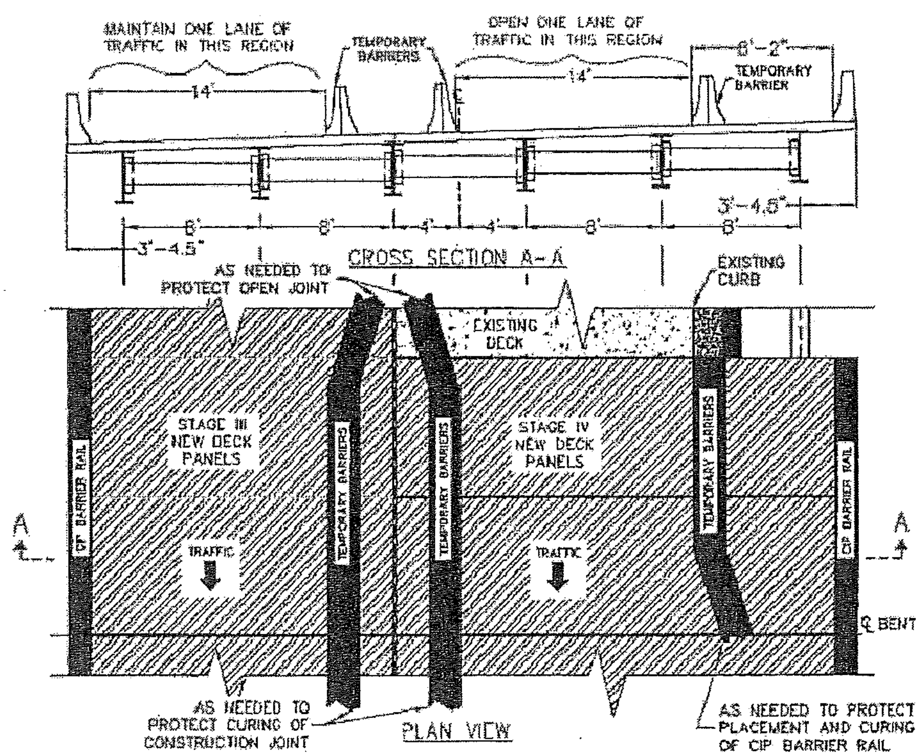


Figure 5.34 Construction Sequence, Stage IV - Task 14

5.6 Deck Replacement Work-Time Schedules and Associated Concrete Mixture Performance Demands

The four rapid deck replacement systems planned for the Collinsville, AL "Test Bridges," i.e.,

NBR Bridge:

- Exodermic Steel Deck Panels with CIP Concrete
- Standard Steel Grid Panels with CIP Concrete

SBR Bridge:

- Precast Exodermic Deck Panels
- Precast Modified NCHRP Deck Panels

will require week-end closure/work periods for the CIP deck systems (the NBR Bridge) and overnight closure/work periods for the precast deck systems (the SBR Bridge). Figure 5.37 shows typical weekend and nightly work-time schedules for the Collinsville Bridges.

The rapid deck replacement construction staging and sequence as well as the work periods-tasks and more detailed estimated time schedules for the CIP deck systems are shown in Fig. 5.38 and Table 5.1, and those for the precast deck systems are shown in Fig. 5.39 and Table 5.2 respectively. Table 5.3 provides a summary of where the new rapid-setting concrete mixtures identified in Figs. 5.38 and 5.39 are to be used as well as the critical required properties of each of the mixtures.

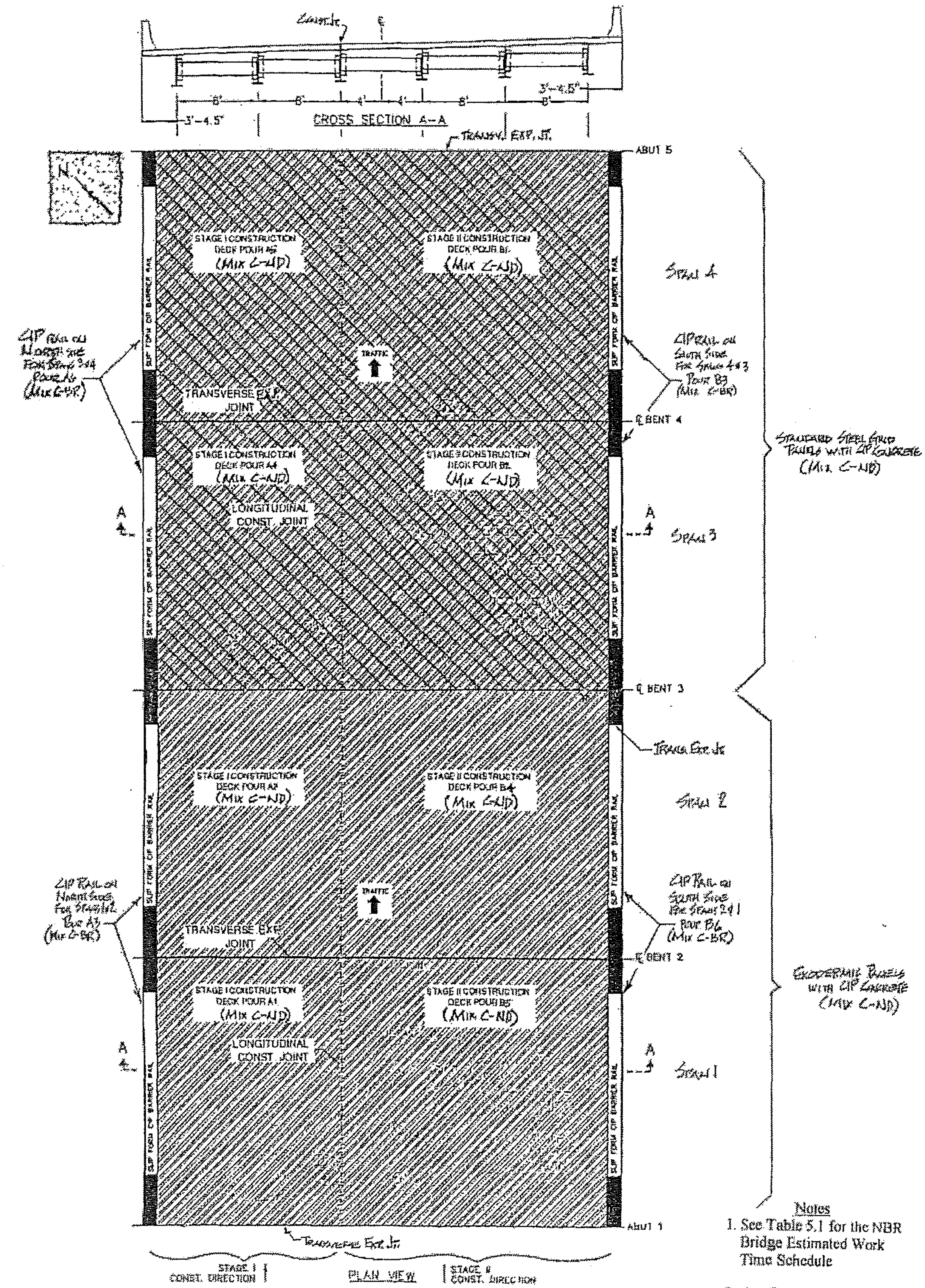


Fig.5.38 NBR Bridge CIP Deck Replacement

Notes

1. See Table 5.1 for the NBR Bridge Estimated Work Time Schedule
2. See Table 5.3 for a Listing and Properties of the Concrete Mixtures Identified Above

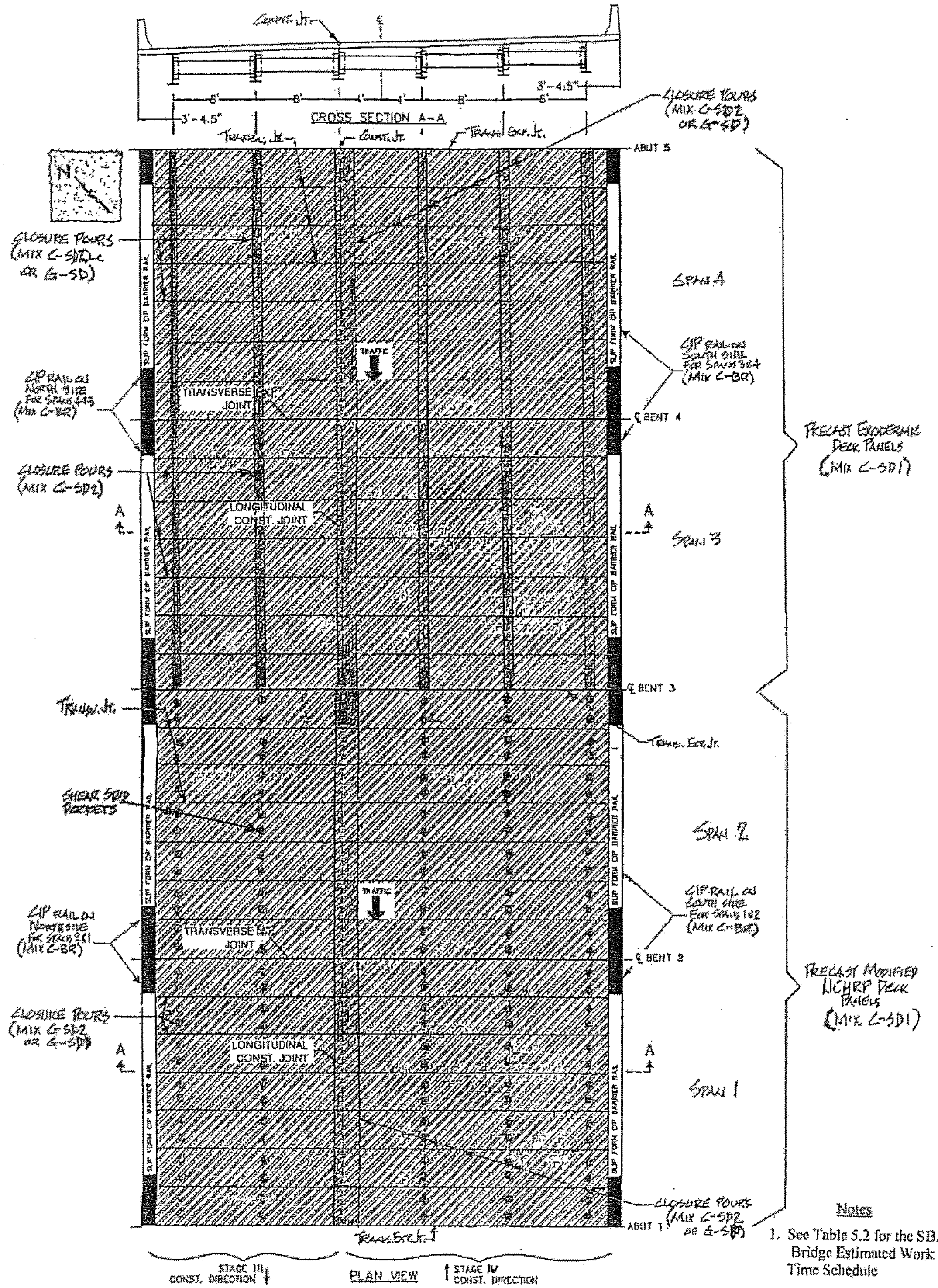


Fig. 5.39. SBR Bridge Precast Panel Deck Replacement

Notes

1. See Table 5.2 for the SBR Bridge Estimated Work Time Schedule
2. See Table 5.3 for a Listing and Properties of the Concrete Mixtures Identified Above.

Table 5.3

Collinsville, AL Rapid Deck Replacement Concrete Mixture Locations and Required Properties

Mixture ID	Location Where Used	Properties
C-BR (Barrier rail concrete)	CIP Barrier Rails for both NBR & SBR Bridges	High quality SCC with compressive strength greater than 3000 psi in 24-hours after placement and $f'_c > 4000$ psi or, C-ND concrete (see below)
C-ND (NBR deck concrete)	NBR deck concrete to be CIP in Exodermic and Standard Grid Steel Panels	High quality concrete with low shrinkage, low-permeability, and a compressive strength greater than 3000 psi in 24-hours after placement and $f'_c > 4000$ psi
C-SD1 (SBR deck concrete)	SBR deck concrete to be precast in the Exodermic and Modified NCHRP Panels	High quality concrete with low shrinkage, low permeability, and $f'_c > 4000$ psi
C-SD2 (SBR deck closure pour concrete)	SBR deck closure pour concrete to be field placed in deck longitudinal joints and/or in panel shear pockets over girders and in deck transverse joints between precast panels wherever possible	High quality concrete with low shrinkage, low permeability, and a compressive strength greater than 3000 psi in 2-hours after placement and $f'_c > 4000$ psi
G-SD (SBR grout)	SBR grout for use in transverse joints between precast deck panels and for injecting in NCHRP panel shear pockets over bridge girders if needed in lieu of using C-SD2 mixture	High quality grout with low shrinkage, low permeability, and a compressive strength greater than 3000 psi in 2-hours after placement and $f'_c > 4000$ psi

traffic patterns, work was chosen to be completed in the direction of traffic to minimize the moving of work equipment. This is beneficial because all of the equipment will be at the correct end of the bridges at the end of the first construction passes.

It should be noted that the Alabama Department of Transportation (ALDOT) has now finalized a sequence of construction that is very similar to the initial proposed sequence of construction in this report. ALDOT's construction sequence differs slightly by incorporating additional steps and by using a different traffic pattern for the southbound bridge. The temporary barrier rails that are to be used are also different.

A step was added at the beginning of the construction sequence for Stages I and III. This step diverts traffic into one lane in the center of each bridge to provide room for the expansions of the bridge end slabs and substructures. The expansion of the end slabs and substructures is necessary for the widening of the bridge deck which requires the addition of a girder line on both sides of each bridge. This step should not affect the results from this study. Also, ALDOT changed the traffic pattern for the southbound bridge to mirror that of the traffic pattern for the proposed sequence of construction. That is, Stage III of the proposed sequence of construction is Stage IV of ALDOT's sequence of construction and Stage IV of the proposed sequence of construction is Stage III of ALDOT's sequence of construction. This has no impact on the results from this report.

It should be noted that some special provisions must be made to the Alabama Standard Construction Specifications in order to implement the rapid deck replacement on the sister I-59 bridges at Collinsville, AL described above. A first draft of the Special Provisions required are given in Appendix C.

correspond to each of the load cases indicated in Tables 6.1 and 6.2, respectively. These construction steps are displayed in the construction sequence drawings found in Chapter 5. Although the final load case will generally be the worst, there are a few locations along the cross-section where other load cases control. The maximum values at specific locations will be needed to design certain components of the decking systems. Some of the loads listen in Tables 6.1 and 6.2 will only act on parts of the cross-section due to lack of continuity at the longitudinal construction joint. Figures 6.3 and 6.4 illustrate the regions in which the loads act for the cast-in-place and precast concrete systems, respectively.

Table 6.1 Cast-in Place Deck System Load Cases

Load Case #	Stage	Load Case	Loads		
			DL	LL, WL	Barriers
1	I.4	Only Case	Stage I		
2	I.6	Only Case	Stage I		B1
3	II.2&3	Before I L.L.	Stage I		B1, B2
4	II.2&3	After I L.L.	Stage I	Stage I	B1, B2
5	II.5	Only Case	Stages I, II	Stage I	B1, B2
6	II.6	Only Case	Stages I, II	Stage I	B1, B2, B3, B4
7	II.7	Only Case	Stages I, II	Stages I, II	B1, B2, B3, B4
8	II.8	Only Case	Stages I, II	Stages I, II	B1, B2, B3, B4, B5
9	Final	Only Case	Stages I, II	Final	B1, B5

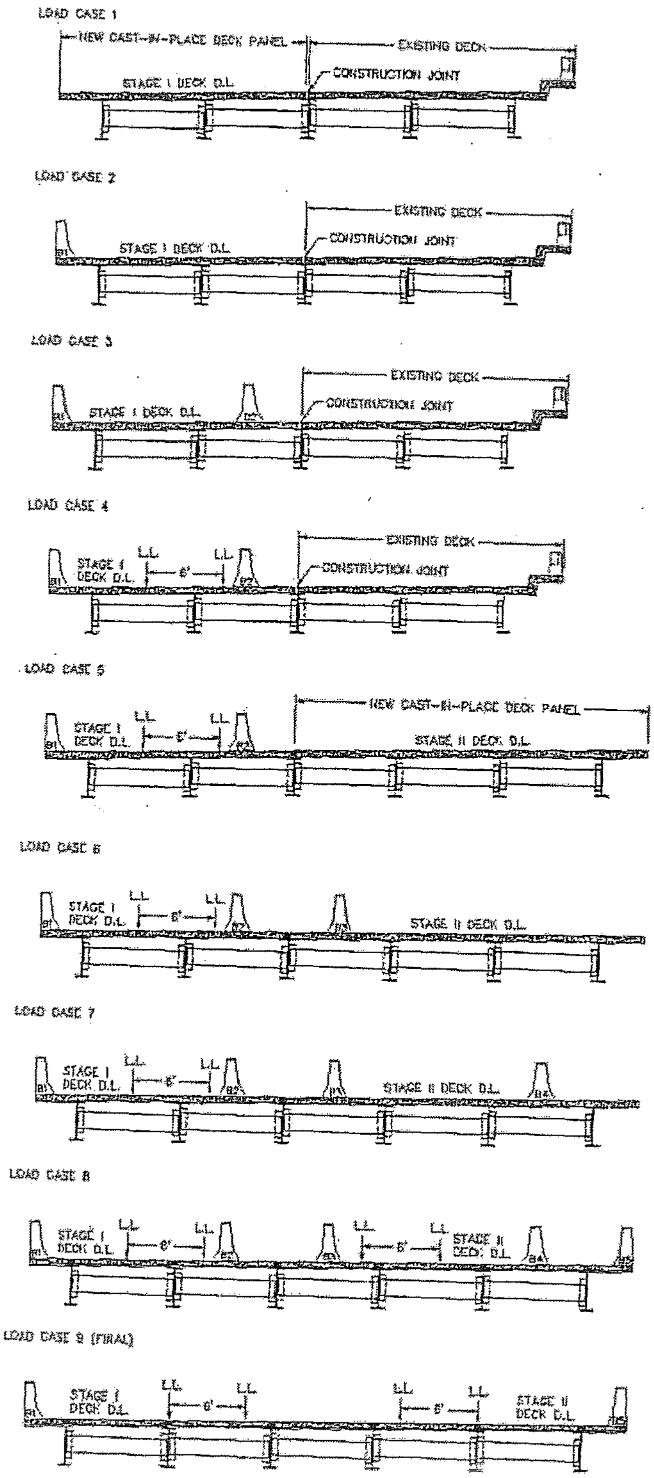


Figure 6.1 Cast-In-Place Deck System Load Cases

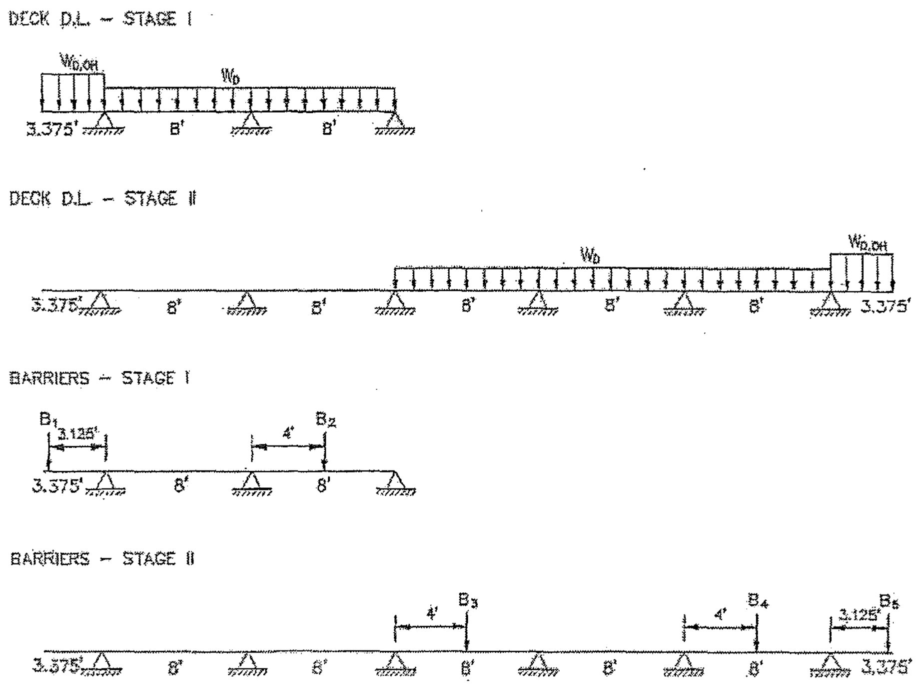


Figure 6.3 Cast-in-Place Deck System Loading Regions

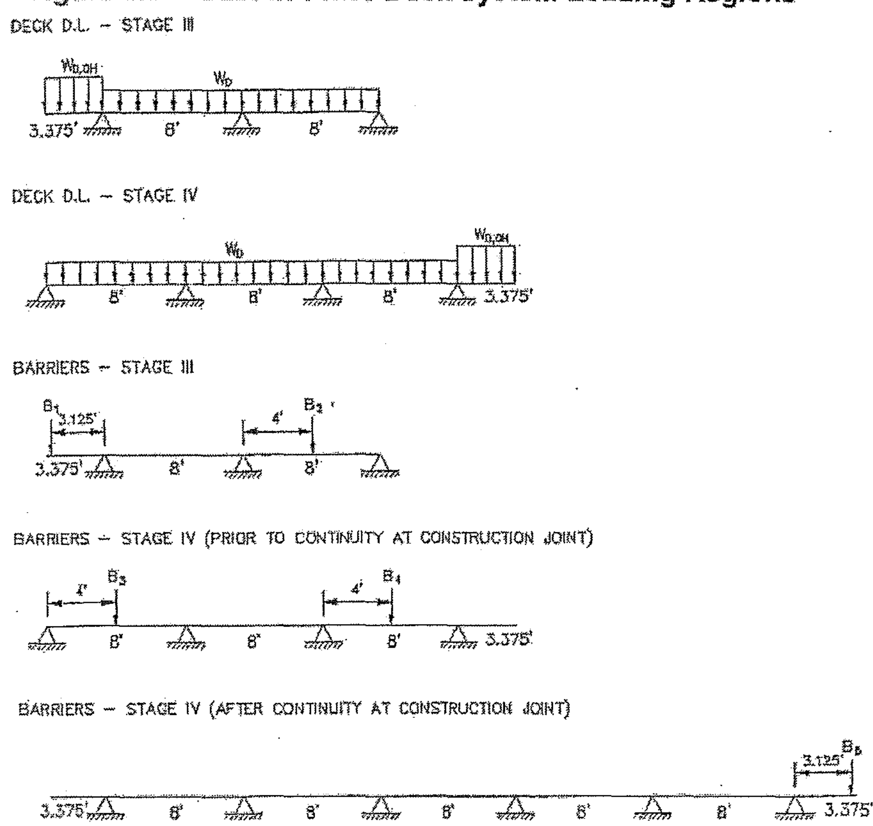


Figure 6.4 Precast Deck System Loading Regions

calculations. Appendix D contains the verification of the procedure taken for the determination of the vehicular live load values as well as an outline of the procedure itself.

The centerline of the truck wheels were positioned at least 2 feet from the edge of each design lane as specified in the AASHTO Bridge Specifications Article 3.6.1.3.1 (AASHTO 2008). The design lane for Stage IV lane 2 was taken from the temporary barrier bear the construction joint to the new barrier rail. The lane was modeled this way because traffic may be on this area of the bridge depending on how the temporary rail is tapered.

The raw live load data obtained from SAP2000 is not per foot of deck. The values obtained must be distributed over an equivalent width. The lengths of these equivalent widths are determined by Table 4.6.2.1.3-1 of the AASHTO Specifications (AASHTO 2008). This table is shown in Table 6.3 and S in the table is the spacing of supporting components and is 8-ft for the Collinsville bridges. Substituting this value into the equation yields an effective width of 6.57 - ft for negative moments and 6-ft for positive moments. The values obtained from SAP2000 are divided by these effective width values to yield a value per foot of decking. The live load values must also be multiplied by a multiple presence factor that accommodates for the probability of multiple design vehicles being on the bridge at the same time. This factor is determined by Table 3.6.1.1.2-1 of the AASHTO Bridge Specifications (AASHTO 2008). A replica of this table is shown in Table 6.4. This factor is not used for the fatigue limit state.

Table 6.4 Multiple Presence Factor, m

Number of Loaded Lanes	Multiple Presence Factors *m*
1	1.20
2	1.00
3	0.85
>3	0.65

The static vehicular live loads previously found must be amplified to accommodate for dynamic effects. This is done by using a dynamic load allowance factor (IM). This amplification factor is found in Table 3.6.2.1-1 of the AASHTO Bridge Specifications (AASHTO 2008). This value varies depending on the component and limit state being considered. Table 6.5 shows these values and is a direct copy of Table 3.6.2.1-1 of the AASHTO Bridge Specifications (AASHTO 2008). The static vehicular live load is increased by the appropriate percentage from Table 6.5.

Table 6.5 Dynamic Load Allowance, IM

Component	IM
Deck Joints—All Limit States	75%
All Other Components • Fatigue and Fracture Limit State	15%
• All Other Limit States	33%

The deck overhangs must be designed to withstand a vehicle collision with barrier rail. The vehicle collision load (CT) shall be determined by Section 13 of the AASHTO Bridge Specifications (AASHTO 2008). The deck overhang design requirements are outlined in Section A13.4 of the specifications.

After all of the load components were determined, they were factored in the load combination limit states of interest. The Strength I, Service I, Fatigue, and Extreme Event II load combination limit states are of interest for the deck design. These limit states are defined in 3.4.1 of the AASHTO Bridge Specifications (AASHTO 2008). The Strength I limit state is the "basic load combination relating to the normal vehicular use of the bridge without wind." The Service I limit state is the "load combination relating to the normal operational use of the bridge with a 55 mph wind and all loads taken at their normal values." This limit state is "also related to control crack width in reinforced concrete structures." The Fatigue limit state is the "load combination relating to repetitive gravitational vehicle live load and dynamic responses under a single design truck." The Extreme Event II load combination limit state is also of interest for the design of the deck overhang to accommodate for barrier rail collisions. Table 6.6 is a replica of Table 3.4.1-1 of the AASHTO Bridge Specifications (2007). This table shows the load combinations and their load factors. The permanent load factor, Υ_{p}, is determined from Table 6.7. Table 6.7 is a copy of Table 3.4.1-2 of the AASHTO Bridge Specifications (AASHTO 2008).

Equation 6.1 must be met for all components and connections for each limit state. This is equation 1.3.2.1-1 of the AASHTO Bridge Specifications (AASHTO 2008).

$$\Sigma \eta_i \gamma_i Q_i \leq \phi R_n \tag{6.1}$$

$$\eta_i = \eta_D \eta_R \eta_I \geq 0.95 \tag{6.2}$$

$$\eta_i = \frac{1}{\eta_D \eta_R \eta_I} \leq 1.0 \tag{6.3}$$

Where η_i is a load modifier relating to ductility, redundancy, and operation importance, γ_i is the load factor determined from Table 6.6, Q_i is the force effect, ϕ is the resistance factor, and R_n is the nominal resistance. Definition for the load modifier, η_i, is shown in Equations 6.2 and 6.3. Equation 6.2 is used when maximum values of η_i is appropriate and Equation 6.3 is used when minimum values are appropriate. The parameter η_D is the load factor relating to ductility, η_R is the load factor relating to redundancy, and η_I is the load factor relating to operational importance. See Section 1.3.2 of the AASHTO Bridge Specifications for further details (AASHTO 2008).

The load modifier, η_i, is taken as 1.0 for non-strength limit states as specified by Article C1.3.2.1 of the AASHTO Bridge Specifications (AASHTO 2008). A value of 1.0 was chosen for the load factor relating to ductility, η_D, because of compliance with AASHTO Bridge Specifications (AASHTO 2008). A value of 1.05 was chosen for the load factor relating to redundancy, η_R, for the deck overhang regions. The Article 1.3.4 of the AASHTO Bridge Specifications requires a value at least this much for non-redundant members such as the overhang region (AASHTO 2008). This arbitrary value satisfies the specifications requirement. The redundancy load factor was taken as 1.0 for the rest of the deck since the deck has more than two supports. The importance load factor was taken as 1.0 since this is a typical bridge and is not of high importance.

loads and load effects for the NCHRP deck system were calculated and the results from these calculations are given below.

The live load moments are shown in Figure 6.6 for the final load case. There are three different live load cases; only lane 1 loaded, only lane 2 loaded, and both lanes loaded simultaneously. Each case controls at different locations along the cross-section. The wind on live load moment values are shown in Figure 6.7 for the final load case. Wind in both directions was considered. The live load and wind on live load results shown in Figs. 6.6 and 6.7 are applicable to all four decks systems.

The different dead load components were all graphed on the same chart for the final load case. This provides a comparison of the load effects imposed by each. The chart is shown in Figure 6.8 for moment load effects. The load effects for each of the loads, except for the collision load, are shown in Figure 6.9 for the final load case. Strength I, Service I, and Fatigue load combination limit states are plotted in Figure 6.10.

The same process was repeated for all of the other load cases of Table 6.1. However, the Fatigue limit state was only considered for the final load case. The worst case for each cross section was then used to generate a worse case chart for each of the limit states. This envelope is shown in Figure 6.11. Comparing Figure 6.11 to Figure 6.10, it is recognizable that the final load case will typically be the worst load case, but there are a few locations where other load cases control. Since the strip method will be used, the extreme positive moment will be used in all positive regions for the design of the deck as required by AASHTO Bridge Specifications Article 4.6.2.1.1 (AASHTO 2008). Similarly, the extreme negative moment will be used at all negative sections for the deck design. The Strength I, Service I, and Fatigue Limit State extreme positive and negative values are summarize in Table 6.8.

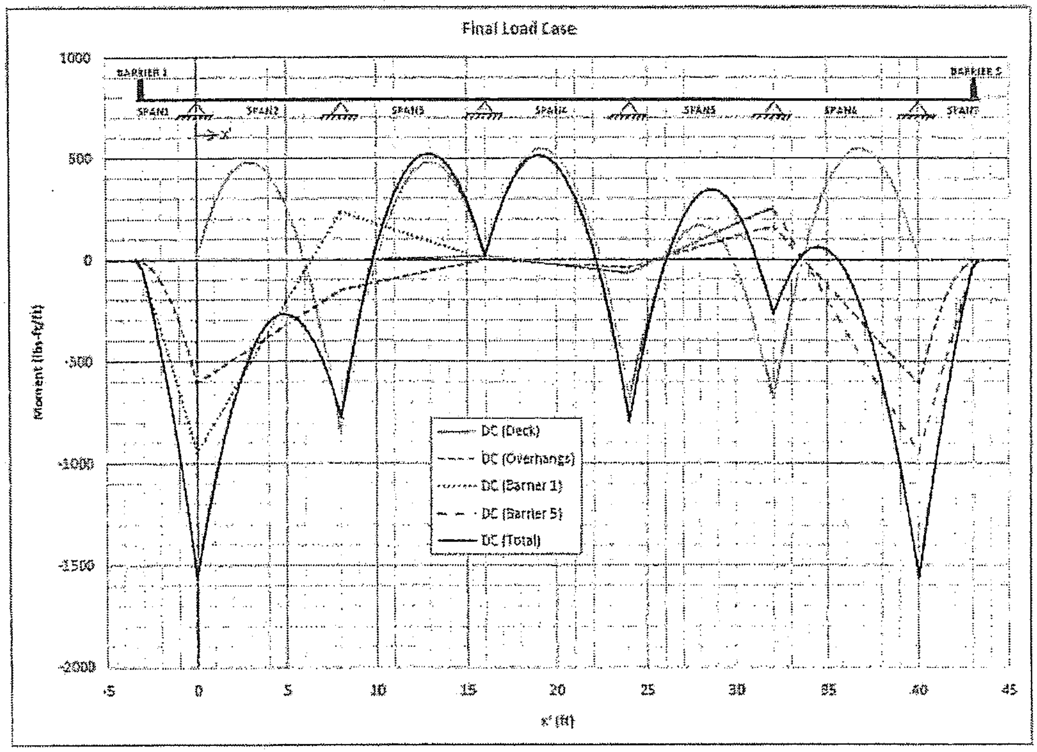


Figure 6.8 Dead Load Moments for Final Load Case

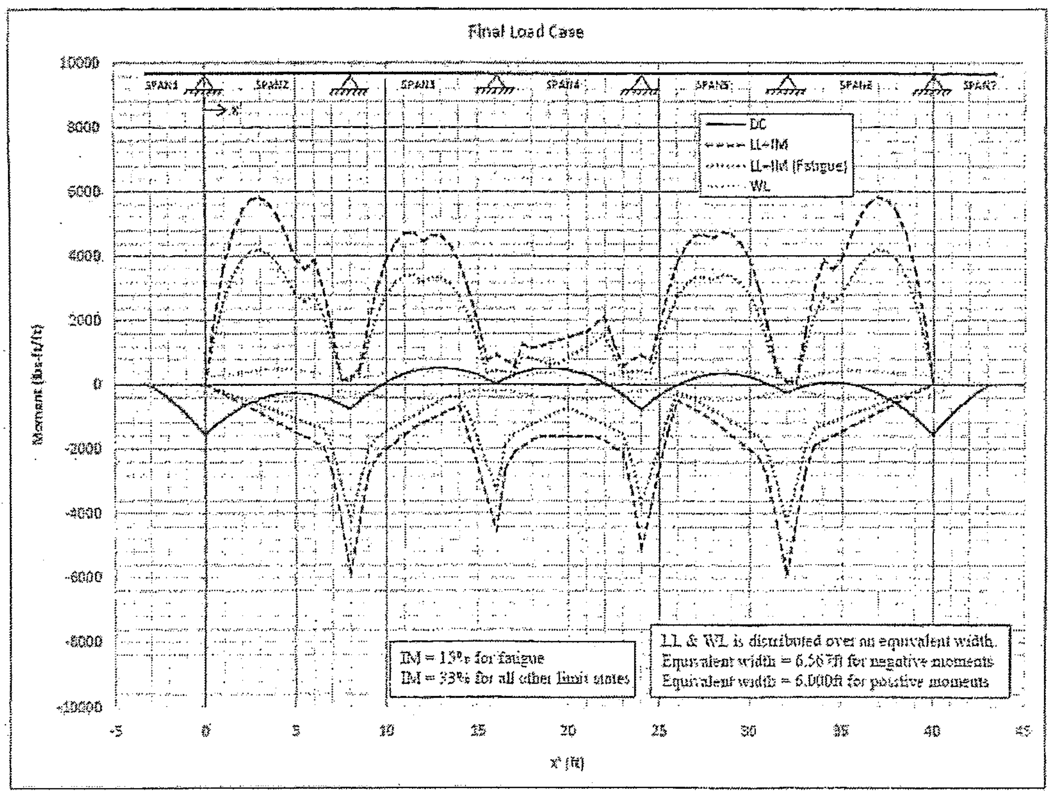


Figure 6.9 Moments for Final Load Case

Table 6.8 NCHRP Precast Deck System Extreme Moment Values

Moment (lbs-ft/ft)	Extreme Positive	Extreme Negative
Strength I	10441	-11321
Service I	6251	-7024

6.2.2. Deck Design

The NCHRP full-depth deck system was designed for a girder spacing of 12 ft. This design could be refined for our application, but using the existing design is conservative for this project. The design of this system was verified against the Strength I and Service I limit states. The Service I limit state includes the control of crack width and deflection limits.

Strength I limit state calculations were performed and the factored negative and positive moment capacities determined from these calculations were -20.4kip-ft/ft and 20.4 kip-ft/ft. The controlling Strength I load effect minimum and maximum moments on this deck system are -11.3 kip-ft/ft and 10.4 kip-ft/ft respectively. This deck system design is adequate for the Strength I limit state since the resistance is greater than the load effect.

The spacing of the mild steel reinforcement in the layer closest to the tension face is limited by AASHTO Bridge Specifications Article 5.7.3.4 to control cracking (AASHTO 2008). The maximum spacing was calculated to be 165 inches for the bottom mild steel reinforcement and 95 inches for the top steel. Since the maximum spacing of the mild steel reinforcement for the NCHRP deck panel is 9.5 inches, which is much less than the maximum allowable spacing, the design is adequate for the placed as close to the top of the deck surface as possible while satisfying concrete cover requirements.

The reinforcement that is used to resist the effects of a vehicle collision must adequately develop from the critical section shown in Figure 6.12. The available development length is equal to 12.37 inches, as shown in the figure, minus the required cover. This leaves an available development length of 9.87 inches. This values was determined using a cover of 2.5 inches to take advantage of the 0.7 factor for adequate cover as described in Article 5.11.2.4.2 of the AASHTO Bridge Specifications (AASHTO 2008).

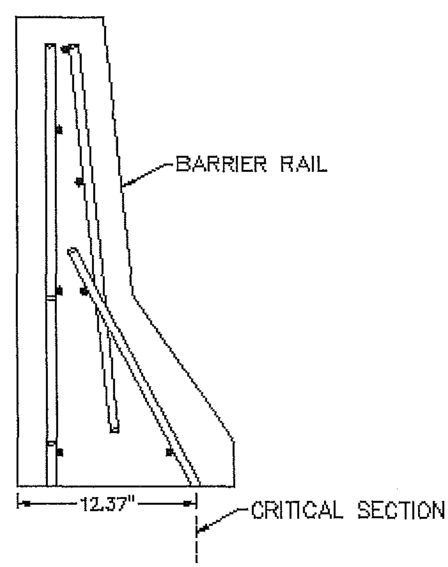


Figure 6.12 Critical Section Location of Barrier Rail for Vehicle Collision

The required development length for the #5 reinforcing bars is determined from Equation 5.11.2.4.1.1 of the AASHTO Bridge Specification. Using a modification factor of 0.7 for adequate cover, the development length required for a standard hook is 8.3

6.3. Exodermic Precast Deck System

6.3.1. Load Calculations

The loads for this deck system were determined as outline in 6.1.1. The extreme positive and negative moments are summarized in Table 6.9.

Table 6.9 Exodermic Precast Deck System Extreme Moment Values

Moment (lbs-ft/ft)	Extreme Positive	Extreme Negative
Strength I	12877	-14389
Service I	9992	-10605
Service II	9992	-10605
Fatigue	3617	-3617
Extreme Event	N/A	-18324

6.3.2. Deck Design

The Exodermic precast deck system was designed for the Strength I, Service I, Service II, and Fatigue limit states. The Service I limit state is used to limit the deflection of the deck system. The Service II limit state is used to control yielding of the steel.

Strength I limit state calculations were performed, and the factored negative and positive flexural capacity determined from these calculations were -21.4 kip-ft/ft and 37.0 kip-ft/ft respectively. The deck system was analyzed using plastic section properties for these calculations as permitted by AASHTO Bridge Specifications Article 9.5.4 (AASHTO 2008). The controlling Strength I load effect minimum and maximum moments on this deck system are -14.4 kip-ft/ft and 12.9 kip-ft/ft respectively. The load resistance is greater than the load effect, thus the design of this deck system is adequate for the Strength I limit state.

The design of the Exodermic precast deck system meets all of the AASHTO Bridge Specifications criteria except the fatigue requirement for negative flexure. The fatigue requirement is disregarded per Appendix B as previously discussed. The design of this deck system is deemed to be acceptable for use on the I-59 bridges.

The total depth of this deck system was chosen to be close enough to the total depth of the NCHRP deck system to provide similar deck surface elevations. This will provide a smooth transition for traffic between the two deck systems without requiring a large haunch. This deck system is somewhat over designed to accommodate for this depth requirement.

6.3.3. Deck Overhang Design

A minimum area of steel of 0.788-in^2 per foot of deck is required at the top of the deck for the overhang region. The Exodermic deck system has #5 reinforcing bars spaced at 5 inches center-to-center. This reinforcement provides an area of steel equal to 0.744-in^2 per foot of deck. So an additional 0.044-in^2 per foot of deck is required in the top portion of the deck overhang to meet the minimum area of steel. This is rather small and is disregarded. The deck system is acceptable as is for the overhang region.

The reinforcing bars are required to be developed to resist the loads associated with a barrier rail collision. The development requirements are the same as those determined in Section 6.2.3. Refer to Figure 6.13 for required reinforcing bar dimensions.

The deflection check for this deck system was made, and the computed deflection was much less than the deflection limit. Thus, the design is adequate for the Service I deflection limit state.

The Service II limit state load calculations for the control of yielding of steel are shown found the negative and positive moments that correspond to first yielding of steel to be -12.2 kip-ft/ft and 38.4 kip-ft/ft respectively. The deck system was analyzed as fully elastic for these calculations as required by AASHTO Bridge Specifications Article 9.5.2, thus transformed cracked section properties were used (AASHTO 2008). These values were compared to the minimum and maximum Service II limit state load effects of -10.6 kip-ft/ft and 10.0 kip-ft/ft respectively. The design of this system is adequate for this limit state since the moment that first causes yielding of the steel is less than the moments applied to the deck system.

The welds on the distribution bars must be checked for fatigue. The results for this system are the same as those for the Exodermic precast deck system. The fatigue stress at the weld at the bottom end of the distribution bar is okay for positive flexure, but the fatigue stress at the weld at the top end of the distribution bar does not pass the fatigue requirement for negative flexure. As previously mentioned for the Exodermic precast deck system, the negative flexure requirement is dismissed due to the performance of several similar in-service grid decks as described in Appendix B.

The design of the Exodermic cast-in-place deck system meets all of the AASHTO Bridge Specifications Criteria except the fatigue requirement for negative flexure. The fatigue requirement is disregarded per Appendix B as previously discussed. The design of this deck system is deemed to be acceptable for use on the I-59 bridges.

6.5.2. Deck Design

The steel grid cast-in-place deck system was designed for the Strength I, Service I, Service II, and Fatigue limit states. The Service I limit state is used to limit the deflection of the deck system. The Service II limit state is used to control yielding of the steel.

The factored negative and positive flexural capacity values for the Strength I limit state were determined to be -20.4 kip-ft/ft and 37.8 kip-ft/ft respectively. The deck system was analyzed using plastic section properties for these calculations per AASHTO Bridge Specifications Article 9.5.4 (AASHTO 2008). The controlling Strength I load effect minimum and maximum moments on the deck system are -14.9 kip-ft/ft and 13.3 kip-ft/ft. The flexural capacity is greater than the load effect for both positive and negative flexure, thus the design of this deck system is adequate for the Strength I limit state.

The deflection check for this deck system found the computed deflection to be much less than the deflection limit. Thus, the design is adequate for the Service I deflection limit state.

The Service II limit state load calculations that pertain to the control of yielding of steel found the negative and positive moments corresponding to yielding of the steel to be -20.2 kip-ft/ft and 43.4 kip-ft/ft. Transformed cracked section properties were used since the deck system was analyzed as fully elastic for these calculations per AASHTO Bridge Specifications Article 9.5.2 (AASHTO 2008). These values are compared to the minimum and maximum Service II limit state load effects -11.0 kip-ft/ft and 10.2 kip-ft/ft respectively. The design of this system is adequate for this limit state since the moment provide adequate resistance. These bars shall be placed as close to the top of the deck surface as possible without violating concrete cover requirements. The reinforcing bars are required to be developed to resist the loads associated with a barrier rail collision. The development requirements are the same as those determined in Section 6.2.3. Refer to Figure 6.13 for required reinforcing bar dimensions.

6.6. Barrier Rails

The Alabama Department of Transportation (ALDOT) requires the bridge railings for the I-59 Collinsville bridges to be rated at least Test Level Four (TL-4). TL-4 is "generally acceptable for the majority of applications on high speed highways, freeways, expressways, and interstate highways with a mixture of trucks and heavy vehicles" as described in Article 13.7.2 of the AASHTO Bridge Specifications (AASHTO 2008). It is recommended for ALDOT's standard barrier rail, shown in Figure 4.8, to be used for the test bridges. It is lightweight in comparison to other TL-4 rate barrier rails as stated in Chapter 4. Moreover, the barrier rail is a previously tested crashworthy system so additional cash testing is not required as long as minor modifications don't change the performance of the railing system as permitted by Article A13.7.3.1.1 of the AASHTO Bridge Specifications (AASHTO 2008). Although this barrier is a previously tested crashworthy system, a verification of the crash test rating as well as minimum reinforcement and development requirements of the reinforcement can be found in Appendix A.

It should be noted that the design of the approach railing system is not considered in this study. The "approach railing system should include a transition guardrail system to the rigid bridge railing system that is capable of providing lateral

7. DESIGN ADEQUACY OF THE FOUR SUPERSTRUCTURE SYSTEMS

7.1. General

The adequacy and performance of each of the deck systems acting compositely with their supporting components are investigated in this chapter. Design ratings are determined for limit states of interest at critical locations along each of the spans. Design ratings greater than one are considered acceptable. The design ratings are determined by modifying Equation 6.1 as shown in Equation 7.1.

$$Design\ Ratio = \frac{\phi R_n}{\sum \eta_i \gamma_i Q_i} \qquad (7.1)$$

7.2 Load Calculations

Load calculations are required to determine the design ratings for each of the four deck systems. The longitudinal load effect calculations for each of the sections of interest were performed and a summary of these load effects are shown in Tables 7.1 and 7.2 for the interior and exterior girders respectively. Dead loads and live loads were considered and a general outline of the calculation procedure and assumptions that were made are as follows.

The loads due to dead weight are different for the systems due to the variation in deck structure and material weights. Unit dead weight of each of the four deck systems were calculated and are shown in Table 7.3. As can be seen in the table, the NCHRP deck system is the heaviest by a considerable margin and the standard steel grid and Exodermic system weights are about the same. Note in Table 7.3 that the unit weight of the deck overhang is considerably heavier for the Exodermic and standard steel grid

Table 7.2 Load Effect Summary for Exterior Girders

Deck System	Select Internal Force Resultants	Exterior Girders						
		mg*	LL+IM	mg x (LL+IM)	DC	Strength I	Service II	Fatigue
Exodermic Cast-in-Place	M_{max} (kip-ft)	0.769	1219	938	324	2148	1543	N/A
	V_{max} (kip)	0.769	98	75	22	168	120	N/A
	M_{cvrplt} (kip-ft)	0.769	1033	794	242	1778	1275	N/A
	$Mfat_{cvrplt}$ (kip-ft)	0.769	497	382	N/A	N/A	N/A	287
	$Mfat_{diaph}$ (kip-ft)	0.769	591	455	N/A	N/A	N/A	341
	$Vfat_{studs,end}$ (kip)	0.769	56	43	N/A	N/A	N/A	32
	$Vfat_{studs,mid}$ (kip)	0.769	18	14	N/A	N/A	N/A	11
Steel Grid Cast-in-Place	M_{max} (kip-ft)	0.769	1219	938	350	2182	1569	N/A
	V_{max} (kip)	0.769	98	75	24	170	122	N/A
	M_{cvrplt} (kip-ft)	0.769	1033	794	262	1803	1295	N/A
	$Mfat_{cvrplt}$ (kip-ft)	0.769	497	382	N/A	N/A	N/A	287
	$Mfat_{diaph}$ (kip-ft)	0.769	591	455	N/A	N/A	N/A	341
	$Vfat_{studs,end}$ (kip)	0.769	56	43	N/A	N/A	N/A	32
	$Vfat_{studs,mid}$ (kip)	0.769	18	14	N/A	N/A	N/A	11
Exodermic Precast	M_{max} (kip-ft)	0.769	1219	938	330	2155	1548	N/A
	V_{max} (kip)	0.769	98	75	23	168	121	N/A
	M_{cvrplt} (kip-ft)	0.769	1033	794	247	1783	1279	N/A
	$Mfat_{cvrplt}$ (kip-ft)	0.769	497	382	N/A	N/A	N/A	287
	$Mfat_{diaph}$ (kip-ft)	0.769	591	455	N/A	N/A	N/A	341
	$Vfat_{studs,end}$ (kip)	0.769	56	43	N/A	N/A	N/A	32
	$Vfat_{studs,mid}$ (kip)	0.769	18	14	N/A	N/A	N/A	11
NCHRP Precast	M_{max} (kip-ft)	0.769	1219	938	373	2213	1592	N/A
	V_{max} (kip)	0.769	98	75	26	172	124	N/A
	M_{cvrplt} (kip-ft)	0.769	1033	794	279	1826	1312	N/A
	$Mfat_{cvrplt}$ (kip-ft)	0.769	497	382	N/A	N/A	N/A	287
	$Mfat_{diaph}$ (kip-ft)	0.769	591	455	N/A	N/A	N/A	341
	$Vfat_{studs,end}$ (kip)	0.769	56	43	N/A	N/A	N/A	32
	$Vfat_{studs,mid}$ (kip)	0.769	18	14	N/A	N/A	N/A	11

*mg is the Distribution Factor

Each girder carries a fraction of the live load. The amount carried by each is obtained by applying distribution factors to the live load force effect. The distribution factors, mg, are determined in accordance with AASHTO Article 4.6.2.2.2 and are shown in Tables 7.1 and 7.2 (AASHTO 2008). Note in Table 7.2 that the distribution factor for the Exterior girders are all the same. This is due to the lever rule controlling. Similarly, the distribution factors for shear are the same for all of the systems due to the level rule controlling.

A longitudinal stiffness parameter is required to calculate the moment distribution factors. This stiffness parameter is dependent upon the modulus of elasticity of the material that makes up the deck. The Exodermic and steel grid deck systems have decks that are made up of steel and concrete and are therefore not homogenous. A modulus of elasticity that will closely represent the stiffness of these deck systems can be estimated by taking a volume fraction of the concrete and the steel and factoring it with the modulus of elasticity for each of the materials. However, there isn't much area of steel in comparison to the concrete deck so the modulus of elasticity of the concrete was conservatively used. The concrete used for the design of these deck systems has a 28 day compressive strength of 4 ksi which results in a modulus of elasticity of 3600 ksi. The NCHRP deck system is designed for a 28 day compressive strength of 6 ksi, but the modulus of elasticity used to calculate the longitudinal stiffness parameter was based on a compressive strength of 4 ksi. This should be acceptable because it produces a conservative value for the longitudinal stiffness parameter.

Table 7.4 Summary of Nominal Moment and Shear Resistance of Deck Systems

Deck System	With Cover Plate			No Cover Plate		
	Moment Interior Girder (ft-lbs)	Moment Exterior Girder (ft-lbs)	Shear (lbs)	Moment Interior Girder (ft-lbs)	Moment Exterior Girder (ft-lbs)	Shear (lbs)
Exodermic CIP	4892	4732	383	4247	4090	383
Steel Grid	5024	4922	383	4375	4216	383
Exodermic PC	4892	4732	383	4247	4247	383
N CHRP	5456	5372	383	4566	4566	383

The elastic properties are used to determine the design ratings for the Strength I Limit State, Service II Limit State, and the Fatigue Limit State. They are also used in calculating the deflections.

There are several sections along the span that need to be investigated for load resistance. The maximum moment occurs at mid-span and verification of the adequacy of that cross-section must be checked against Strength I and Service II Limit States. The cross-section at the termination of the cover plate was checked due to the change in cross-sectional properties. This section was checked for moment resistance for the Strength I and the Service II Limit States. This cross-section was also checked for shear resistance for the maximum shear at the span ends for the same limit states. Fatigue checks were conducted at the toe of the weld for the cover plate and at the weld on the diaphragm connection plates. The ductility of critical cross-sections was also verified. The design rating for each case are presented in Table 7.5. A design rating greater than 1.0 is acceptable. All of the deck systems are ductile and have acceptable design rating excluding the fatigue resistance of the weld at the cover plate termination. Further investigation of this fatigue detail should be conducted.

Table 7.5 Design Rating for the Four Deck-Girder Systems

Limit State	Deck System / Girder / Location	Exodermic CIP		Steel Grid		Exodermic PC		NCHRP	
		Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior
Str. 1	Maximum Moment	2.604	2.203	2.608	2.56	2.6	2.196	2.724	2.428
	Moment at Termination of Cover Plate	3.155	2.662	3.168	2.73	3.149	2.654	3.319	2.942
	Maximum Shear	2.312	2.284	2.244	2.257	2.298	2.284	2.18	2.231
Srv II	Maximum Moment	2.292	2.111	2.279	2.132	2.286	2.101	2.299	2.231
	Moment at Termination of Cover Plate	2.082	1.192	2.106	1.963	2.075	1.097	2.17	2.096
Fat.	Toe of Cover Plate Weld	0.515	0.462	0.542	0.482	0.515	0.462	0.592	0.528
	Weld of Diaphragm Connection Plate	4.209	3.941	4.122	3.795	4.209	3.941	3.856	3.515

Table 7.6 Summary of Deck Deflection Due to Live Load

Deck System	Exodermic CIP		Steel Grid		Exodermic PC		NCHRP	
Girder	Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior
Deflection (in)	0.82	0.84	0.75	0.77	0.82	0.84	0.63	0.64
Limit (in)	0.84	0.84	0.84	0.84	0.84	0.84	0.84	0.84

7.4 Shear Stud Requirements

Shear studs are required to provide composite action between the deck systems and their supporting components. The spacing of the shear studs is based on fatigue

Table 7.8 Recommended Shear Stud Configuration

Deck System	Girder	From Span End to 5 ft		From 5 ft to 10 ft		From 10 ft to 15 ft		From 15 ft to Midspan		Total Number Of Studs per Girder
		Pitch (in)	No. Studs	Pitch (in)	No. Studs	Pitch (in)	No. Studs	Pitch (in)	No. Studs	
Exodermic* CIP	Interior	4.5	28	5.5	22	6.5	18	8	40	216
	Exterior	4	32	5	24	6	20	7.5	42	236
Steel Grid*	Interior	4.5	28	5.5	22	6.5	18	8	40	216
	Exterior	4.5	28	5.5	22	6.5	18	8	40	216
Exodermic* PC	Interior	4.5	28	5.5	22	6.5	18	8	40	216
	Exterior	4.5	28	5.5	22	6.5	18	8	40	216
NCHRP †	Interior	48	16	48	8	48	8	48	32	128
	Exterior	48	16	48	8	48	8	48	32	128

* Stud configuration for $^7/_8$" studs
†Stud configuration for 1 ¼" studs

The spacing, or pitch, of the shear studs is dependent upon the number of studs across the girder, the fatigue resistance of each stud, and the fatigue load that is applied. The fatigue load varies along the span with the maximum being at the span ends and the minimum occurring at mid-span. For simplicity, it was assumed that the fatigue load varies linearly from the span ends to mid-span.

The NCHRP deck system is designed to have the shear studs placed in clusters. The shear stud clusters are spaced at 48 inches and consists of eight 1¼" diameter shear studs. This spacing is larger than the spacing allowed per AASHTO LRFD Bridge Specification. However, validation of the spacing of the shear stud clusters has been done by previous full-scale beam testing as outlined in the NCHRP Report 584. It was concluded that it is okay to use AASHTO Equation 6.10.10.2-1 to "determine the fatigue the four systems, a normalized value for the moment resistance and the deflection per deck weight was evaluated and is shown in Table 7.9. Viewing these values, it is apparent that the NCHRP deck exhibits the best deflection but the worst nominal moment resistance. The Exodermic cast-in-place and the Exodermic precast deck systems have the worst deflection, but have the best nominal moment resistance. The steel grid deck system falls between the NCHRP and the Exodermic Systems.

Table 7.9 Normalized Comparison of the Deck-Girder Systems

Deck System / Parameter	Exodermic CIP	Steel Grid	Exodermic PC	NCHRP
Weight	0.68	0.82	0.69	1.00
Moment Resistance	0.90	0.92	0.90	1.00
Deflection	1.00	0.92	1.00	0.77
Deflection/Weight	1.00	0.76	0.99	0.52
MomentResistance/Weight	1.00	0.85	0.99	0.76

The precast deck systems were designed considering a quarter inch, non-structural, sacrificial layer. This additional thickness provides for surface texturing. Texturing is done by machine grinding and results in a quality-riding surface. Surface texturing of the finished deck is recommended since an overlay will not be used.

The barrier rails for all four deck systems will be the ALDOT's standard Jersey barrier that they currently use on new interstate highway bridges. The rails are qualified as TL-4 which is the test rating that is generally used for freeway bridges and will be CIP for all four deck systems. The deck overhangs are designed to resist the loads from a vehicle collision with a TL-4 test rated barrier rail.

Composite action between the deck systems and their supporting components is obtained through the use of shear studs. The NCHRP full-depth deck system has shear studs that are grouped in clusters that are spaced at 48 inches. The spacing of the clusters is qualified through previous testing by Badie, et al. as described in Chapter 7. The number of studs and their maximum spacing is shown in Table 7.8 for each of the deck systems.

A requirement to maintain at least one of the existing two traffic lanes open at all times during the deck replacement work required that both sister bridges, i.e., northbound roadway (NBR) and southbound roadway (SBR) bridges be widened during the deck replacement work. The bridge widening requires work to be completed on the substructure as outlined in Chapter 4. Performing the work on the substructure simultaneously with the deck replacement work could hinder the comparability of the deck systems in regards to the time required to perform the deck replacement work.

Each of the four rapid deck replacement panels and elements were designed to act compositely with the supporting girders and were checked for structural adequacy in Chapters 6 and 7 and were found to be satisfactory.

Fatigue resistance was checked for the welds attaching the diaphragm connection plates and the welds attaching the cover plates to the girders. All of the fatigue checks for the welds attaching the diaphragm connection plates are satisfactory. The welds of the cover plates failed the fatigue checks. Further investigation of this fatigue detail should be conducted. It is recommended to not use cover plates on the girders that are to be installed. The load resistance and behavior of the composite cross-sections are adequate for the W36x150 girders without the use of a cover plate. This was determined by expanding on the calculations of Chapter 7.

8.3 Recommendations

The portion of the rapid bridge deck replacement research reported on in this report was focused on the design and constructability of four candidate rapid deck replacement systems as evaluated on paper via analytical and critical analyses. To gain a more complete understanding of the relative merits and demerits of each of the four candidate deck replacement systems, it is recommended that the following actions be taken.

1. Develop concrete/mortar mixture designs for the four rapid deck replacement options with the primary objective of developing rapid setting high performance mixture designs for topping concrete for the CIP deck systems and for closure pour concrete and/or mortar for the precast deck systems.
2. Prepare final construction documents for the Collinsville, AL sister bridge rapid

- Design “friendliness” of replacement system
- Construction “friendliness” of replacement system
- First year performance
- The relative ease/difficultly in making deck panel splices in the transverse direction of the bridge (joint/connection between deck panels and between panels and existing deck during staged construction)
- The relative ease/difficulty in making deck panel splices in the longitudinal direction of the bridge (joint connection between deck panels and existing deck during staged construction)
- The relative ease/difficulty in handling skewed bridges
- Any unique positive and negative features associated with the replacement system

6. Have post construction meeting with the deck replacement contractor to get his feedback and evaluation of the construction merits and demerits of the four test deck systems.
7. Input the information gathered from executing actions cited in numbers 3, 5 and 6 into the developed construction evaluation document cited in number 4 above to analyze and synthesize the results to draw tentative conclusions and recommendations on the best rapid deck replacement system for the Birmingham bridge decks.

APPENDIX A

Proposed Barrier Rail Adequacy Check

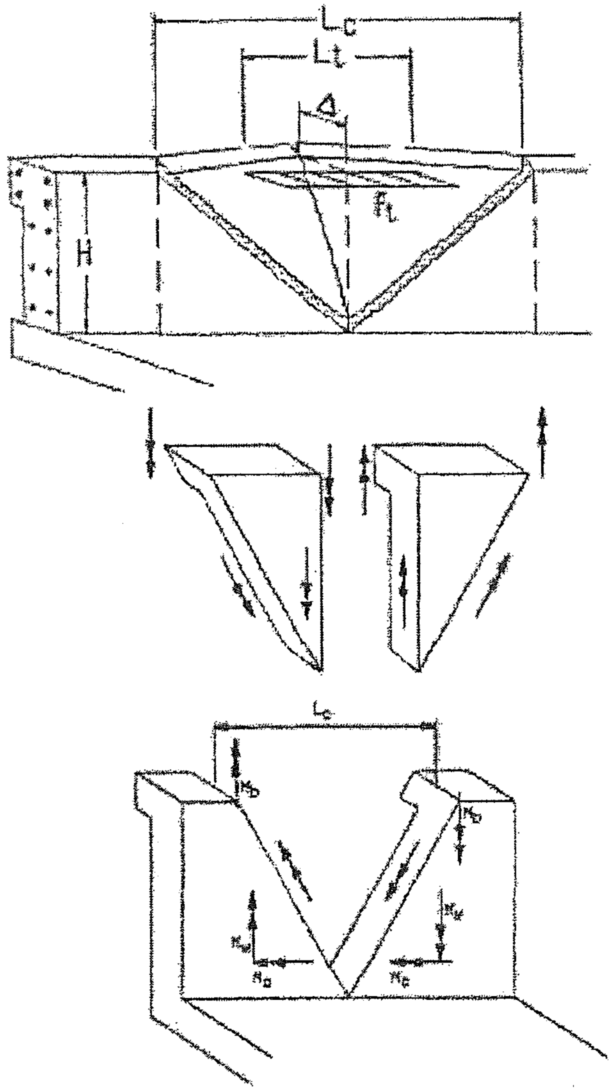


Figure CA13.3.1-1 Yield Line Analysis of Concrete Parapet Walls for Impact within Wall Segment
(From AASHTO Bridge Speicification 2007)

$As_{neg} := 3 \cdot 0.2 \cdot in^2 = 0.6 \cdot in^2$ 3 #4 Bars

$$davg_{neg.I} := \frac{3.35 \cdot in + 4.29 \cdot in + 4.6 \cdot in}{3} = 4.08 \cdot in$$

$$a := \frac{As_{neg} \cdot fy}{0.85 \cdot fprimec \cdot b} = 0.557 \cdot in$$

$$Mn_{neg} := As_{neg} \cdot fy \cdot \left(davg_{neg.I} - \frac{a}{2} \right) = 11.404 \cdot kip \cdot ft$$

$$Mn_{I} := \frac{Mn_{pos} + Mn_{neg}}{2} = 10.036 \cdot kip \cdot ft$$

Segment II:

Neglecting contribution of compressive reinforcement

$b := 13 \cdot in$

$As_{pos} := 0.2 \cdot in^2$ 1 #4 Bar

$d_{pos.II} := 12.13 \cdot in$

$$a := \frac{As_{pos} \cdot fy}{0.85 \cdot fprimec \cdot b} = 0.271 \cdot in$$

$$Mn_{pos} := As_{pos} \cdot fy \cdot \left(d_{pos.II} - \frac{a}{2} \right) = 11.994 \cdot kip \cdot ft$$

$As_{neg} := 0.2 \cdot in^2$

$d_{neg.II} := 10.52 \cdot in$

$$a := \frac{As_{neg} \cdot fy}{0.85 \cdot fprimec \cdot b} = 0.271 \cdot in$$

$$Mn_{neg} := As_{neg} \cdot fy \cdot \left(d_{neg.II} - \frac{a}{2} \right) = 10.384 \cdot kip \cdot ft$$

$$Mn_{II} := \frac{Mn_{pos} + Mn_{neg}}{2} = 11.189 \cdot kip \cdot ft$$

$$Mw := \phi \cdot Mn_{I} + \phi \cdot Mn_{II} = 21.226 \cdot kip \cdot ft$$

Segment II

$$As := 0.372 \cdot \frac{in^2}{ft}$$ #5 Bars @ 10" C-C

Since the depth of the vertical reinforcement varies, an average value will be used.

$$d_{avg.II} := \frac{5.37 \cdot in + 12.37 \cdot in}{2} = 8.87 \cdot in$$

$$a := \frac{As \cdot fy}{0.85 \cdot fprimec} = 0.547 \cdot in$$

$$Mc_{II} := As \cdot fy \cdot \left(d_{avg.II} - \frac{a}{2} \right) = 15.989 \cdot kip \cdot \frac{ft}{ft}$$

Mc is determined from a weighted average of Mc_I and Mc_{II}

$$Mc := \frac{Mc_I \cdot 17 \cdot in + Mc_{II} \cdot 13 \cdot in}{17 \cdot in + 13 \cdot in} = 11.684 \cdot kip \cdot \frac{ft}{ft}$$

3. Critical Length of Yield Line Failure Patter, Lc

$$Lc = \frac{Lt}{2} + \sqrt{\left(\frac{Lt}{2}\right)^2 + \frac{8 \cdot H \cdot (Mb + Mw)}{Mc}}$$ (AASHTO A13.3.1-2)

$Lt := 3.5 \cdot ft$ (AASHTO Table A13.2-1, TL-4)

$H := 2.67 \cdot ft$ (overall height of barrier rail)

$Mb := 0$ (additional flexural resistance of beam)

$$Lc := \frac{Lt}{2} + \sqrt{\left(\frac{Lt}{2}\right)^2 + \frac{8 \cdot H \cdot (Mb + Mw)}{Mc}} = 8.22 \cdot ft$$

4. Nominal Resistance to Transverse Load, Rw

$Ft := 54.0 \cdot kip$ (transverse design force, TL-4 - AASHTO Table A13.2-1)

$$Rw := \left(\frac{2}{2 \cdot Lc - Lt}\right) \cdot \left(8 \cdot Mb + 8 \cdot Mw + \frac{Mc \cdot Lc^2}{H}\right) = 71.945 \cdot kip$$

$Rw = 71.945 \cdot kip$ > $Ft = 54 \cdot kip$ Okay

7. Minimum Area of Steel AASHTO 5.8.4.4

$$Avfmin = \frac{0.05Acv}{fy}$$

$$Avfmin := \frac{0.05 \cdot Acv}{\frac{fy}{ksi}} = 0.15 \cdot \frac{in^2}{ft}$$

$$Avf = 0.744 \cdot \frac{in^2}{ft} \quad > \quad Avfmin = 0.15 \cdot \frac{in^2}{ft}$$ Okay

8. Development Length AASHTO 5.11.2.4

$$l_{hb} = \frac{38 \cdot db}{\sqrt{fprimec}} \qquad l_{hb} := \frac{38 \cdot 0.625 \cdot in}{\sqrt{\frac{fprimec}{ksi}}} = 11.875 \cdot in$$

$$l_{dh} := \max(l_{hb}, 8 \cdot 0.625 \cdot in, 6 \cdot in) = 11.875 \cdot in$$

$l_{dh} < 7 \cdot in$ for 8" thick deck with 1" cover (See AASHTO Figure C5.11.2.4-1)

l_{dh} is larger than 7", however, a factor of As,req/As,provided can be multiplied to l_{hb}

Must determine if excess steel is provided to obtain an $l_{hb} < 7 \cdot in$

$$l_{dh} \cdot \left(\frac{As_{req}}{As_{prov}}\right) = l_{dh.req} \qquad \text{or} \qquad As_{req} = As_{prov} \cdot \left(\frac{l_{dh.req}}{l_{dh}}\right)$$

$$As_{prov} := 0.372 \cdot \frac{in^2}{ft}$$

$$As_{req} := As_{prov} \cdot \frac{7 \cdot in}{11.875 \cdot in} = 0.219 \cdot \frac{in^2}{ft}$$

Okay if As,req provides enough strength such that $Rw > Ft$

$$a := \frac{As_{req} \cdot fy}{0.85 \cdot fprimec} = 0.322 \cdot in$$

$$Mc_{I.check} := As_{req} \cdot fy \cdot \left(d_{avg.I} - \frac{a}{2}\right) = 5.07 \cdot kip \cdot \frac{ft}{ft} \qquad d_{avg.I} = 4.785 \cdot in$$

$$Mc_{II.check} := As_{req} \cdot fy \cdot \left(d_{avg.II} - \frac{a}{2}\right) = 9.548 \cdot kip \cdot \frac{ft}{ft} \qquad d_{avg.II} = 8.87 \cdot in$$

$$Mc_{check} := \frac{Mc_{I.check} \cdot 17 \cdot in + Mc_{II.check} \cdot 13 \cdot in}{17 \cdot in + 13 \cdot in} = 7.01 \cdot kip \cdot \frac{ft}{ft}$$

APPENDIX B

Proposal to AASHTO to Revise Fatigue Requirements for Concrete Filled Steel Grid Decks

Item #3

Insert the following paragraph into the Commentary for Article 9.8.2.3.3.

Fully-filled and partially-filled steel grid decks must be checked for fatigue in only the positive moment region (mid span). However the fatigue moment should be calculated for a simple span configuration (C=1.0) regardless of the actual span configuration.

BACKGROUND:

The first edition of the AASHTO LRFD Bridge Design Specification and all revisions up to 2003 did not require fatigue design checks on internal connections (within the concrete fill) of fully-filled and partially-filled steel grid decks. However in the 2003 Interims, modifications were made which required these internal connections to be designed for fatigue which results in maximum design span lengths much lower than historical limits.

To investigate this discrepancy a calibration study was performed using 26 in-service decks (16 full depth grid and 10 partial depth grid) with at least 10 years of service history [Higgins, 2009]. The study concluded that live load moments based on orthotropic plate theory from Article 4.6.2.1.8 of the AASHTO code are consistent with conventional concrete deck live load demands and therefore no change is required to the force effects. While strength resistance, deflection resistance, and fatigue resistance in positive moment regions on the grid deck yield allowable span lengths consistent with the 26 projects studied, fatigue resistance in the negative moment region greatly reduce allowable span lengths. An example of these span reductions are illustrated below.

Bridge	Mackinac	I-70 over MO River	Upper Buckeye
Deck Type	Full Depth	Partial Depth	Partial Depth
LRFD Span	1.6'	0.6'	1.3'
Actual Span	5.0'	7.1'	8.25'
Years in Service	52	30	15

ANTICIPATED EFFECT ON BRIDGES:

Implementation of the revised specification will allow fully-filled and partially-filled steel grid decks to be designed for span lengths consistent with historical limits.

REFERENCES:

"Calibration of AASHTO-LRFD Section 4.6.2.1.8 with Historical Performance of Filled, Partially Filled, Unfilled and Composite Grid Decks – Final Report", Christopher Higgins, O. Tugrul Turan, School of Civil and Construction Engineering, Oregon State University, Corvallis, OR, 97331, June 2009

OTHER:

None

The following is a composite of,

1. Special Provisions first draft prepared by the ALDOT.

2. Added material, changes and comments to No.1 above (so indicated by | ADD or | CHANGE or BOXED ☐ comments) prepared by Auburn Researchers.

3. Select Special Provisions extracted from GDOT Special Provisions for rapid deck replacement of two 1-285 bridges in Atlanta, GA.

4. Other Special Provisions deemed appropriate by the AU research team.

Table 1

Collinsville, AL Rapid Deck Replacement Concrete Mixture Locations and Required Properties

Mixture ID	Location Where Used	Properties
C-BR (Barrier rail concrete)	CIP Barrier Rails for both NBR & SBR Bridges	High quality see with compressive strength greater than 3000 psi in 24-hours after placement and fie> 4000 psi or, CNO concrete (see below)
C-ND (NBR deck concrete)	NBR deck concrete to be CIP in Exodermic and Standard Grid Steel Panels	High quality concrete with low shrinkage, low-permeability, and a compressive strength greater than 3000 psi in 24-hours after placement and f ie > 4000 psi
C-SD1 (SBR deck concrete)	SBR deck concrete to be precast in the Exodermic and Modified NCHRP Panels	High quality concrete with low shrinkage, low permeability, and fie> 4000 psi
C-SD2 (SBR deck closure pour concrete)	SBR deck closure pour concrete to be field placed in deck longitudinal joints and/or in panel shear pockets over girders and in deck transverse joints between precast panels wherever possible	High quality concrete with low shrinkage, low permeability, and a compressive strength greater than 3000 psi in 2-hours after placement and fie> 4000 psi
G-SO (SBR grout)	SBR grout for use in transverse joints between precast deck panels and for injecting in NCHRP panel shear pockets over bridge girders if needed in lieu of using C-S02 mixture	High quality grout with low shrinkage, low permeability, and a compressive strength greater than 3000 psi in 2-hours after placement and fie> 4000 psi

ADD

(c) SUBMITTAL OF DESIGNS, SPECIFICATIONS, DETAILS AND INSTALLATION PROCEDURES.

1. PURPOSE OF SUBMITTAL.

The Contractor shall submit the designs, specifications, details and installation procedures for each of the bridge deck systems. This submittal shall be a supplement to the Department's plans and specifications to provide all of the information that is required for the construction of bridge decks that will meet the required loading capacity and geometric requirements. The designs, specifications, details, and installation procedures shall be complete to the point that all required materials are identified, all structural features are adequately described and dimensioned, and all installation procedures are thoroughly described. .

2. DESIGN REQUIRMENTS.

The bridge deck systems shall be designed using the Service Load Design Method to carry HS20 loading in accordance with the current edition of the AASHTO Standard Specifications for Highway Bridges.

~~Design computation shall be submitted in duplicate. Each set of design computations shall be prepared, stamped, dated and signed by a Professional Engineer registered in the State of Alabama, and not employed by the State of Alabama. The design computations shall address all members, connections, and anchorage. The designer shall note on the design computations where the design of each member meets the design requirements and reflection tolerances.~~

~~Design computations, shall include all formulas used and a coy of all calculations and computer printouts that cover all members, connections, and details necessary for complete bridge deck structure designs for all bridge deck systems. Where computer generated designs are used, the printouts shall consist of the applied loadings structure geometry, component sizes, moments, shears, reactions, component forces, component stresses, allowable component stresses, combined stress ratios, and deflections. The method of solution used by the computer program, including all formulas used, shall be submitted.~~

DRAFT

3. SPECIFICATIONS.

References shall be given to other Sections of these Standard Specifications where they are applicable to the material, fabrication and installation requirements for the bridge deck systems. The Contractor shall submit all other specifications that are required to completely identify the material, fabrication and installation requirements for the bridge deck systems.

4. DETAILS.

Details of the bridge deck systems shall be submitted on 22" x 34" size plan sheets. Plan, elevation and cross sectional views shall be shown on the drawings to completely identify the sizes, placement and connection details of all components of the bridge deck systems.

5. INSTALLATION PROCEDURES.

The installation procedures shall be submitted for all sequences of installation, formwork, and falsework for the bridge deck systems.

6. INITIAL SUBMITTAL.

The Contractor shall submit 3 sets of the designs, specifications, details and installation procedures for the bridge deck systems to the Construction Engineer as an initial submittal. This submittal shall be made within 30 calendar days after the date of the "Notice to Proceed".

If clarifications are required, one set will be returned to the Contractor with comments for clarification. The initial submittal will be returned to the Contractor for clarification within 14 calendar days after the receipt of the initial submittal.

The Contractor will be allowed 7 calendar days for each clarification of the initial submittal and the Construction Engineer will be allowed 7 calendar days to determine if further clarification is necessary.

The Materials and Tests Engineer will not approve the submittal of design calculations and ~~Wall~~ Construction Drawings but will review the submittal for completeness.

The initial submittal of the design calculations and ~~Wall~~ Construction Drawings shall be

Bridge

Bridge

Special provisions need to be established to put time provisions on construction of each deck system in Collinsville when experimenting with accelerated construction efforts. This will help ensure the schedules associated with each deck system are comparable so a proper evaluation can be performed. On a typical accelerated construction effort, contracts would have penalties for extended lane closures or lane rental fees.

Also, we may need to address provisions for allowing pauses in construction for researcher inspection without penalizing the contractor.

DRAFT

e. Deck replacement work shall be completed linearly from Span #1 to Span #4 using a 7 -day work week as needed during Stage I construction.

f. Stage I construction which includes placing the new CIP barrier rail on the north side of the NBR bridge shall be completed and ready for traffic before beginning Stage II construction.

g. Only minimal lane closures will be allowed on SR68 (under the I-59 bridges) as viewed necessary by the State Bridge Engineer.

h. Stage II construction shall begin immediately upon completion of the Stage I work and shall progress in a linear manner in the direction opposing traffic as indicated in Fig. 1.

i. Deck replacement and edge beam work during Stage II is allowed from 9:00 PM Friday until 6:00 AM Monday. Saw-cutting for deck removal, grinding, joint sealing", permanent striping, temporary striping for traffic shifts, bearing repair and painting will be allowed from 9:00 PM to 6:00 AM, Monday through Sunday. Deck replacement should be for 1-span each weekend during the Stage II work.

j. The CIP barrier rails on the south side of the NBR bridge can be placed (behind the temporary barrier rails) during regular work hours as each spans CIP concrete deck gains sufficient strength.

k. The temporary barrier rails should be removed as soon as the deck has been replaced on all four spans, and the new CIP barrier rails have gained sufficient strength.

l. All work on the NBR bridge shall be completed using a maximum of six (6) weekend partial-closures on the interstate.

m. Work on this bridge shall be completed and both lanes opened to traffic prior to beginning deck replacement work on the SBR bridge.

2. SBR Bridge (See Fig. 1 above)

a. Work shall progress in the direction of traffic for Stage III construction (replacing the outside lane of deck)

b. Work shall progress in the direction opposing traffic for Stage IV construction (replacing the inside lane deck)

c. The SBR Bridge will be a 1-traffic lane bridge throughout Stage III construction.

d. Steel diaphragm work shall be completed prior to beginning deck replacement and edge beam work.

e. Deck replacement work shall be completed linearly from Span #4 to Span #1 using a 7 -day work week as needed during Stage III construction.

7. The Contractor shall provide extra lighting for night-time work (lighting beyond the minimum required) on both the NBR and SBR bridges in order to achieve good quality night work results and to achieve good quality Camcord recordings of the night-time work. Cost for the extra lighting will be paid for under Pay Item XXXX.

D. The Contractor shall request, in writing, the Engineers 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 ALDOT at least five (5) days prior to each shift, change, or restriction of traffic. Also, the Engineer shall notify the DOT Permits Office fourteen (14) days prior to the lane closures to enable them to detour overwidth loads. Permits 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. The Contractor shall provide six (6) changeable message signs for use as needed. Cost for changeable message signs shall be paid for under Pay Item 632-0003 Variable Message Sign, Portable, Type 3. The Contractor shall also provide as dictated by field conditions, sequential flashing arrow signs in accordance with Special Provision 150 as included in the contract, applicable Alabama Standards and MUTCD.

SECTION 505 - COMPOSITE STEEL GRID DECK WIT.H PRECAST CONCRETE SLAB

505.1 General Description

This work consists of furnishing and installing of pre-fabricated structural steel grid deck composite with a reinforced concrete deck slab. Furnish all materials, tools, equipment, transportation, necessary storage, access, labor and supervision required for the proper installation of the deck system",

505.1.03 Submittal Requirements

Submit for review by the Engineer, complete shop drawings, design calculations, and product data that includes the information listed below. Submit seven (7) complete sets of information. Five (5) sets are to be set to the Office of Bridge Maintenance and the remaining two (2) sets to the Area Engineer.

C. Product data information for the proposed including the ·following:

1. Certification from the system manufacturer of the material and section properties of the deck system.
2. Written verification from the system manufacturer that the installers have received the required certifications and training to install the deck system.

Approval of the above by the Engineer does not relieve the Manufacturer and Contractor of responsibility for the
performance of the deck system used.

505.1.04 Contractor Qualifications

Submit to the Engineer for approval, written verification from the Manufacturer proving past experience or training in the installation of the manufacturer's system. Provide quality control procedures in compliance with the Manufacturer's installation requirements for the deck system. Employ or retain for the duration of the contract a full-time on-site Project Manager with technical knowledge of deck system that has successfully completed three (3) installations on similar deck replacement projects. The Project Manager is to be on-site at all times during installation of the panels. The Project Manager is to attend the Pre-Construction Conference and provide a demonstration or presentation on the deck system to the Department. Submit the following information to the Engineer for review and approval 30 days prior to beginning work on the project:

A. Proof of certification by the Manufacturer.
B. Proof of five (5) year Manufacturer's warranty.
C. Resume of proposed Project Manager.
D. A list of three (3) deck replacement projects completed by the Project Manager, including the dates of work ,type, description arid amount of work performed, and the name and telephone number of a contact person at the agency or company for which the work was completed.

The engineer of record reserves the right to approve or reject the personnel qualifications as submitted. The engineer may suspend the work if the contractor substitutes unauthorized personnel for authorized personnel during construction

505.2.03 Precast Concrete

Provide deck with Class AAA concrete in accordance with Section 500 of the Standard Specifications. Furnish concrete deck with a minimum 28-day compressive strength of 4000 psi, design slump between 2" and 4", maximum water cement ratio of 0.40 and a maximum coarse aggregate size of 0.375". Provide deck with a minimum of ¼ " additional concrete overfill to be milled off after all panels are installed. Cure precast panels using we(cure methods a minimum of one week. Do not remove panels from the forms until the concrete has reached 75% of the design strength. After curing, remove all form release material and any other forming materials adhering to the shear key and block out concrete. Sandblast shear key faces , with care taken to avoid damage to ~~the~~ galvanized coating. any

Furnish deck reinforcing steel in the accordance with Section 511 of the Standard Specifications. Place rebar with due consideration to the location of the leveling bolts, providing sufficient clearance for adjustment of the leveling bolts from above using a socket wrench, as well as for clearances required for field placement of headed shear studs. Place main (top) rebar, which runs in the same direction as the main bearing bars of the steel grid, a minimum of 1" from the web of the main bearing bars. Provide a minimum cover between rebar and exposed surfaces of precast concrete of 1" unless otherwise indicated on the plans.

Install sheet metal forms in such a manner as to minimize leakage during concrete placement. Support forms to prevent displacement during precasting operations and to obtain the proper concrete thickness. Use casting beds and forms with provisions for holding the steel grid panels flat and square priOr to placing concrete. Check the steel grid panels for conformity with the required dimensions as to cross slope.

Furnish completed precast panels with dimensional tolerances between adjacent panels of ± 0.25 inches horizontally and ± 0.125 inches vertically.

Texture the surface in accordance with the requirements of Section 500.3 .05.T.9.A and Special Provision 519 to accommodate the co-polymer overlay.

505.2.04 Field Placed Concrete

For the closure pours and shear keys, provide 24 hour accelerated strength concrete in accordance with Section 504 of the Standard Specifications, except as follows:

1. Furnish accelerated strength concrete designed to produce a compressive strength of 3,500 psi (24 MPa) within 24 hours and 4,000 psi (27 MPa) after 3 days, as well as a minimum bond strength of 200 psi after 24 hours.
2. Furnish concrete with a maximum coarse" aggregate size of 0.375".
3. Furnish field placed concrete with a color which closely matches the color of the precast deck panel concrete. Texture the surface in: accordance with the requirements of Section 500.3.05.T.9.A.
4. Do not allow traffic on the deck panels until the concrete has reached a minimum compressive strength of 3,500 psi.

505.3 Construction Requirements

505.3.01 Personnel

Provide a Project Manager meeting the requirements specified in 505.1.04.

APPENDIX D

Verification of SAP2000 Live Load Calculation Procedure

2. HAND CALCULATIONS

The moment envelope for a single point load is shown below in Figure 2.1. Calculations were done by hand through statics and this data was input into Microsoft Excel. It is apparent from this figure that the maximum moment will occur at mid-span and the minimum moment will occur at both of the supports (for a single point load). The influence lines for these sections are needed to determine the expected maximum and minimum moments for the live load case.

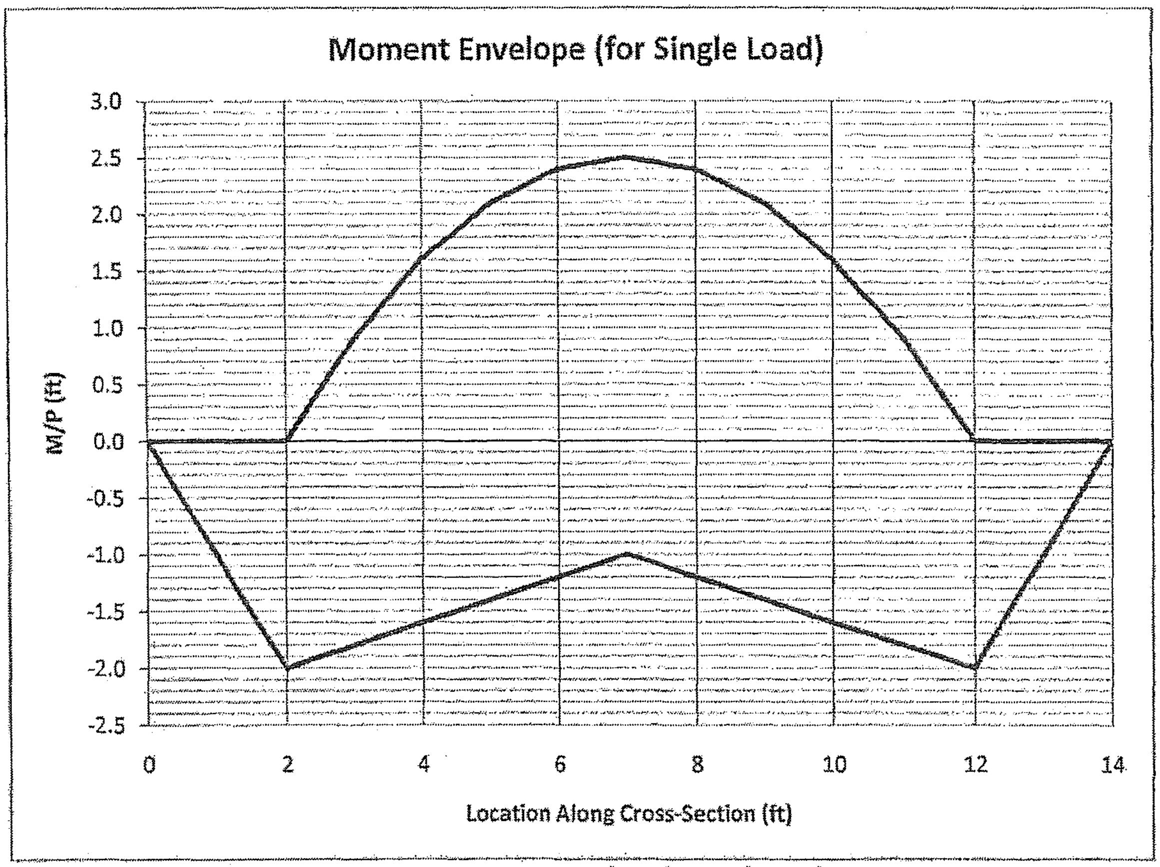


Figure 2.1. Moment Envelope for Single Load

Figures 2.2, 2.3, & 2.4 below show the moment influence lines for the mid-span and support sections (110/200, 205, and 210/300) and the minimum/maximum moment that corresponds to each. As seen from these figures, the moment is expected to be -16 (kip-ft) at both of the supports and 32 (kip-ft) at mid-span.

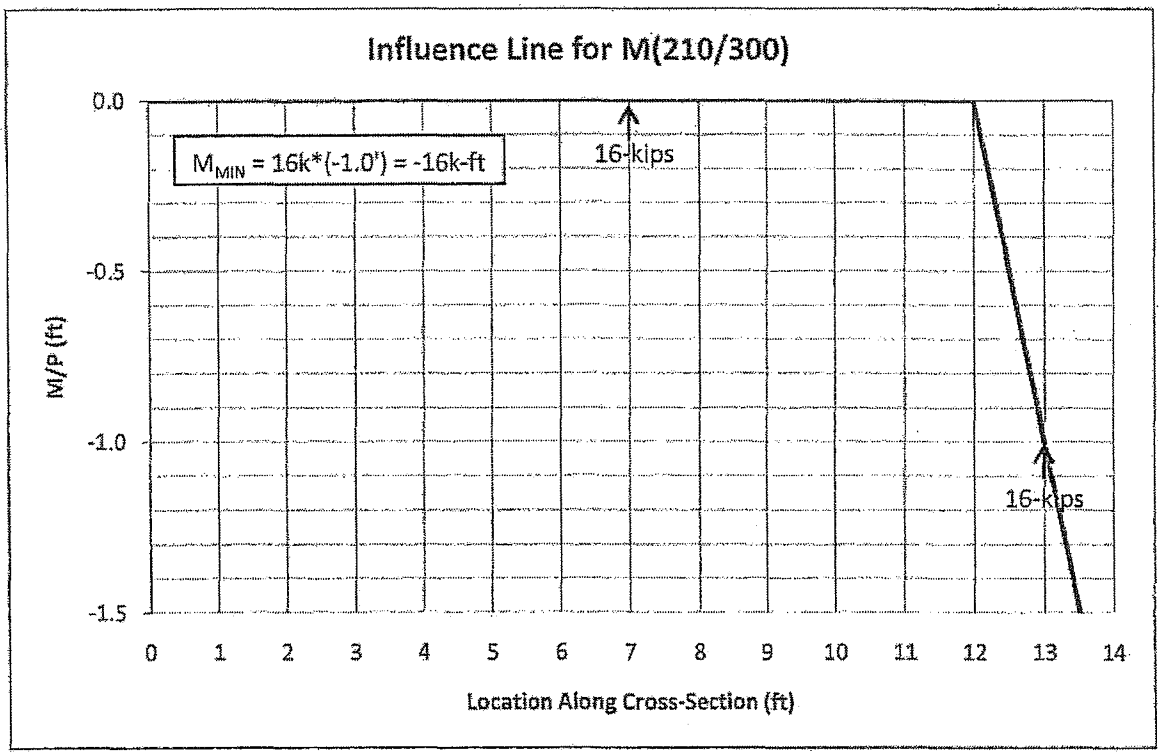


Figure 2.4. Moment Influence Line for Section 210/300

After following the procedure discussed in the next section (Section 3), it is apparent that the maximum moment for the live load case will occur at Section 204. The influence line for this section as well as the maximum moment for the live load case is shown below in Figure 2.5. The calculated maximum moment is 38.4 (kip-ft).

3. SAP2000 PROCEDURE (SAP2000 V14)

This procedure was done using SAP2000 V14. Deviation from this procedure may be necessary for different versions of the software.

The test-cross section was modeled in SAP2000 with frames and joints. Section properties were ignored since the live load is the only load being considered and only shear and moment values are desired.

Initial Loading Procedure

An initial attempt to use the moving load capabilities of the software was taken. To model the live loads placement across the cross-section, a lane had to be defined. The lane was defined by the frames were the live load is allowed to be placed. This procedure is as follows.

- First we need to display the frame numbers
    - View > Set Display Options (Ctrl-E)
    - Check "Labels" under "Frames/Cables/Tendons"
- Define > Bridge Loads > Lanes
- Click "Add New Lane Defined From Frames"
- Input values shown below in Figure 3.1 and click "OK"
- Click "OK"

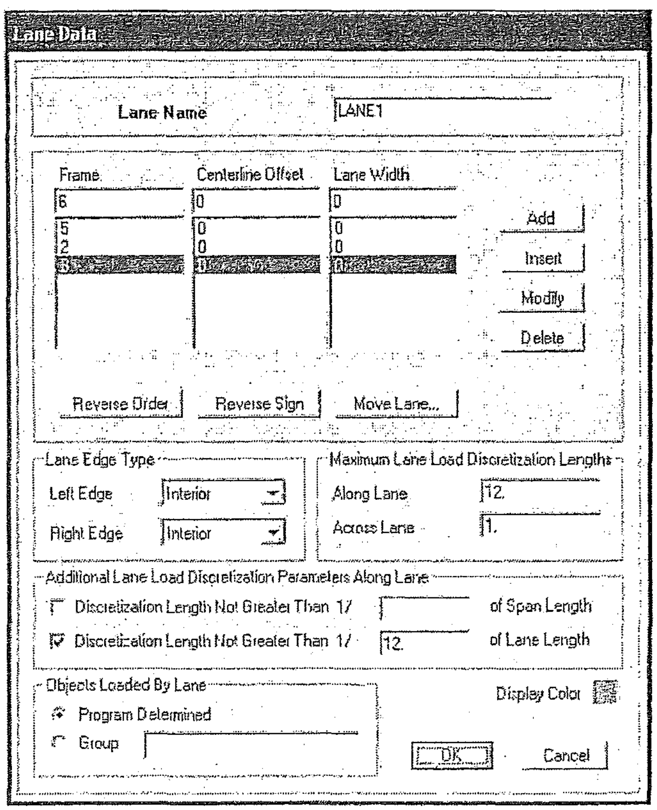


Figure 3.1. Lane Data

- Input the values shown below in Figure 3.3 and press "OK"
- Press "OK"

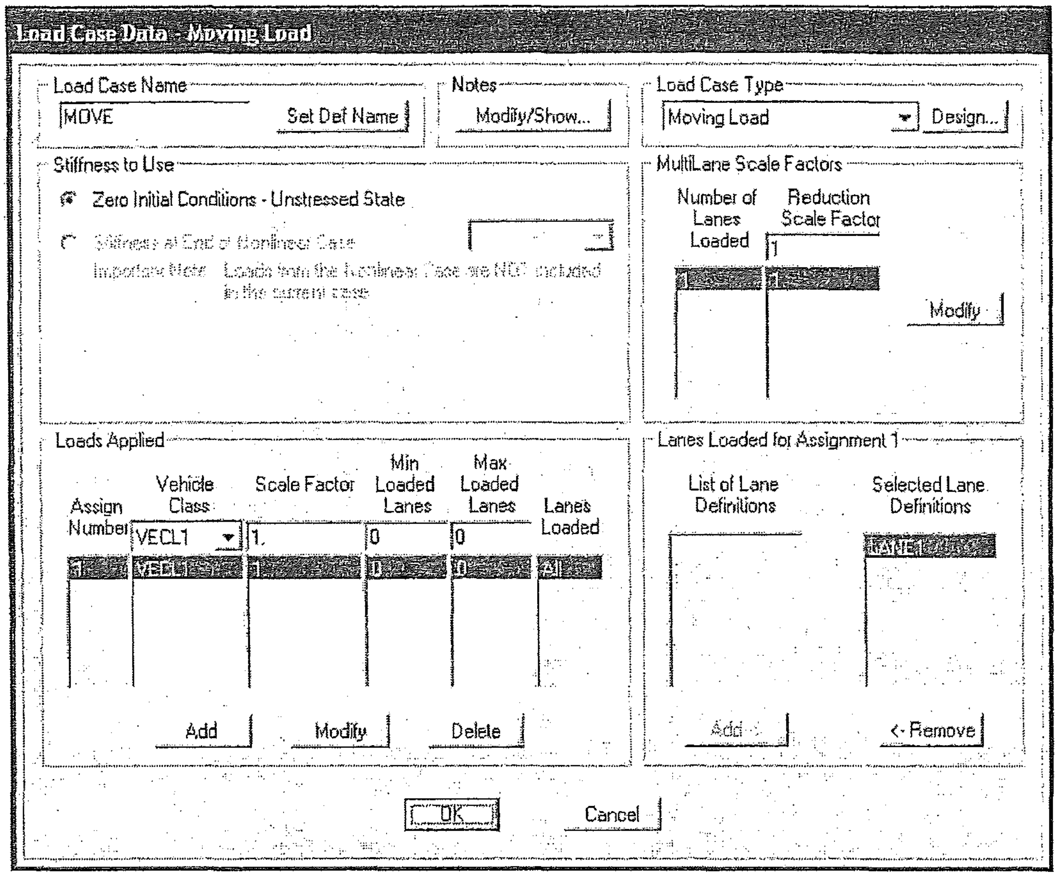


Figure 3.3. Load Case Data

This concludes the live load case definition. From here the "MOVE" load case was analyzed. The results are shown below in Figure 3.4.

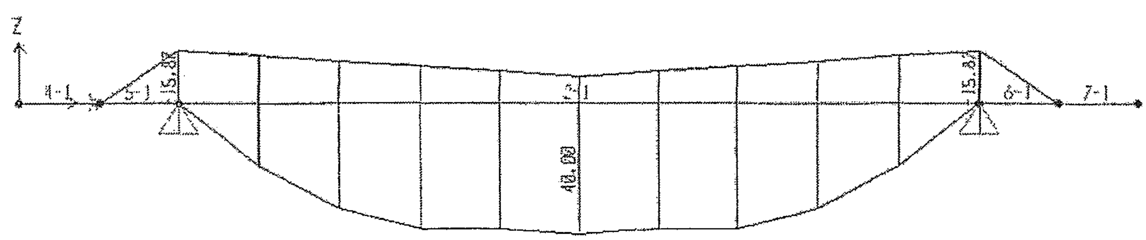


Figure 3.4. SAP2000 Moment Envelope for Live Load

The value at mid-span is much larger than expected. According to the hand calculations, this should be 32 (kip-ft). Therefore, a conclusion was made that the "Vehicle Remains Fully In Lane (In Lane Longitudinal Direction)" option in the "General Vehicle Data" window doesn't work as expected. This option is supposed to force all axle loads to remain in the lanes longitudinal direction. This is necessary

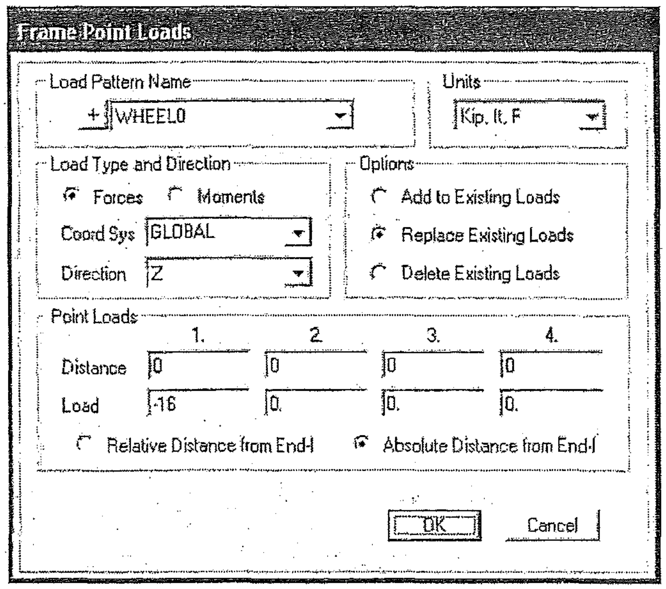


Figure 3.6. Frame Point Loads for WHEEL0, Frame 5

- Select the next frame to be loaded (Frame 2).
- Assign > Frame Loads > Point
- Input the values shown in Figure 3.7 and click "OK"

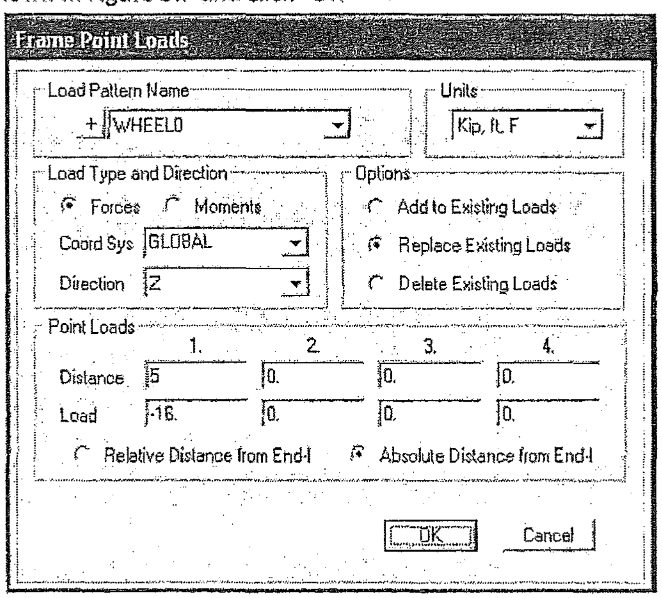


Figure 3.7. Frame Point Loads for WHEEL0, Frame 2

A similar procedure is done for the next live load location (WHEEL1). This is outlined as follows.

- Select the frame to be loaded (Frame 2).
- Assign > Frame Loads > Point
- Input the values shown in Figure 3.8 and click "OK"

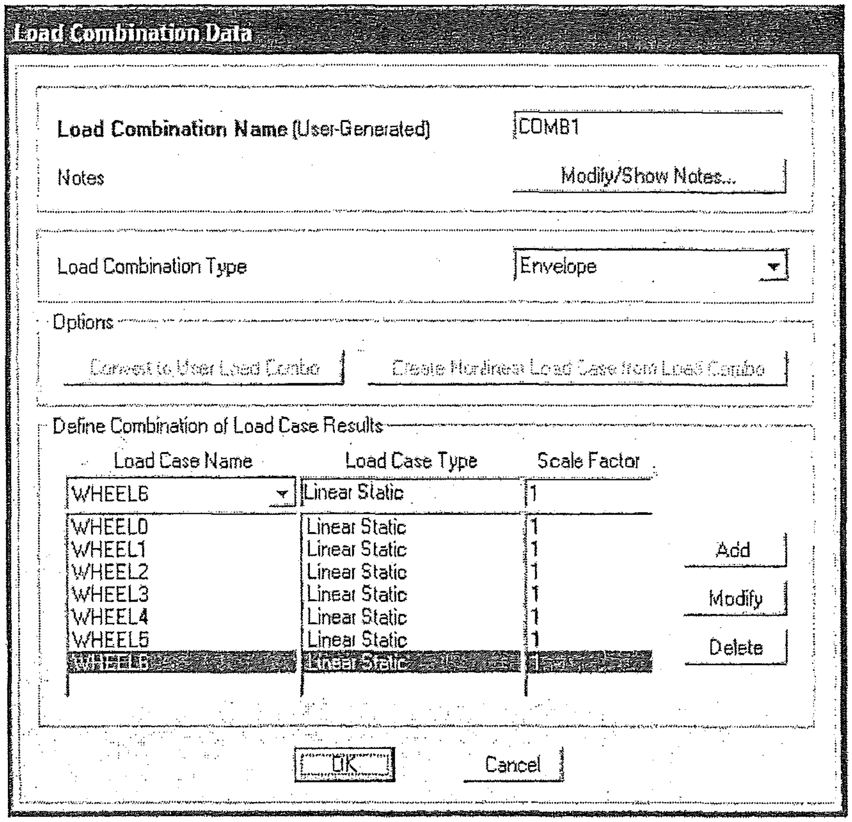


Figure 3.9. Load Combination Data

This concludes the live load procedure. From here all of the "WHEEL" load cases were analyzed. The following procedure is done to display the moment envelope for the live load. This moment envelope is captured by the "COMB1" load combination.

- Display > Show Forces/Stresses > Frames/Cables
- Input the values shown in Figure 3.10 and click "OK"

APPENDIX E

Deck Panel Connection Details

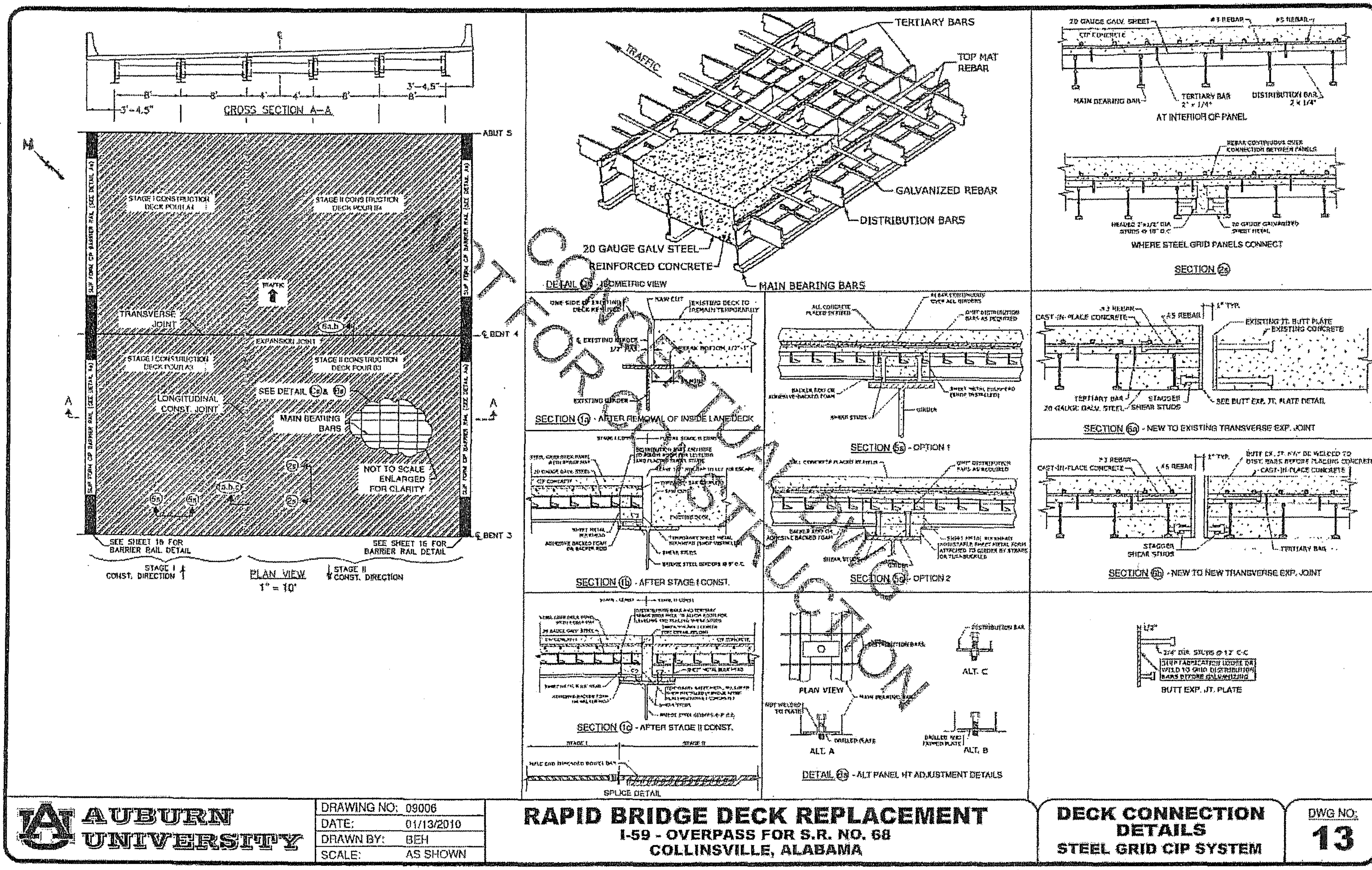
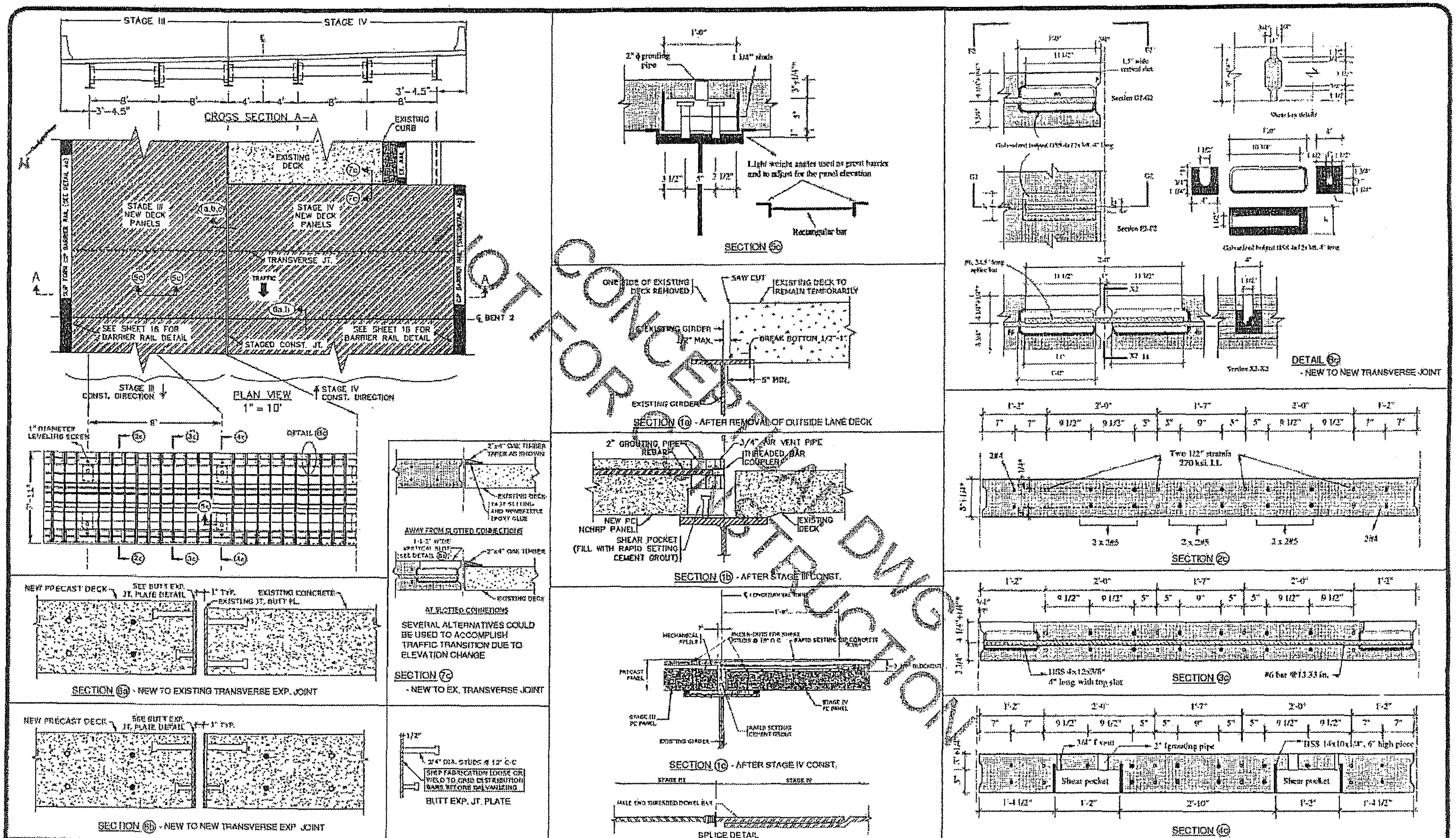


Figure E2. Deck Connection Details – Steel Grid CIP System



AUBURN UNIVERSITY	DRAWING NO: 09006 DATE: 01/13/2010 DRAWN BY: BEH SCALE: AS SHOWN	**RAPID BRIDGE DECK REPLACEMENT** I-59 - OVERPASS FOR S.R. NO. 68 COLLINSVILLE, ALABAMA	**DECK CONNECTION DETAILS** MODIFIED NCHRP PC SYSTEM	DWG NO: **15**

Figure E4. Deck Connection Details – Modified NCHRP PC System