

School of Civil and Environmental Engineering

Structural Engineering, Mechanics and Materials Research Report

Evaluation of Performance and Maximum Length of Continuous Decks in Bridges – Part 2

Final Report

Prepared for

Office of Materials and Research Georgia Department of Transportation

GDOT Research Project No. 10-29 Task Order No. 02-81

by

Eric Davidson, Donald White and Lawrence Kahn

Contract Research

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16. Abstract: Field experimental measurements and analytical studies showed that the link-slab reinforcement experiences almost no axial load due to thermal and gravity loading. One of the fundamental reasons for the low loads in the link-slab reinforcement is the fact that there is a small space (tolerance) between the dowel restraint bars located within the bearing at the bottom of each girder and the cast-in holes in the girder. Further, concrete shrinkage causes a crack to occur at the construction joint; the crack width is larger than the deformations caused by thermal and gravity loading. Thus, no link-slab moments can be generated in the deck. The authors concluded that the top layer of longitudinal deck reinforcement would be sufficient for link-slab reinforcement.					
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Executive Summary

The purpose of this research was to evaluate the performance history of continuous bridge decks in the State of Georgia, to determine why the current design detail between adjacent simple spans works, to recommend a new design detail, and to recommend the maximum and/or optimum lengths of continuous bridge decks. The continuous bridge decks have continuous reinforcement over the juncture between two edge beams and a construction joint for crack control. This zone is often termed a link-slab.

The current link-slab detail in Georgia uses two #6 bars spaced between each #4 bar in the top layer of deck reinforcement (typically spaced at 1-ft 6-in on center).

Generally, this detail performs well. Its design was based on historically good performance.

Field experimental measurements and analytical studies showed that the link-slab reinforcement experiences almost no axial load due to thermal and gravity loading. Its need for dowel action is limited because of the use of edge beams below the deck. Therefore, the only significant loads present in the link-slab reinforcement are due to lateral and longitudinal forces in the deck, typically due to vehicle braking, wind and earthquake effects. One of the fundamental reasons for the low loads in the link-slab reinforcement is the fact that there is a small space (tolerance) between the dowel restraint bars located within the bearing at the bottom of each girder and the cast-in round or slotted hole in the girder. This space exists whether the bearing is considered "fixed" or "expansion". This small space permits sufficient movement of the girder to relieve any

longitudinal force at the bearing due to girder rotation; yet, the dowel still restrains any excessive transverse and longitudinal motion. The rotations in the link slab are centered approximately about the link-slab reinforcement, not the centroid of the girder-slab composite structure. Further, concrete shrinkage occurring soon after the deck is placed causes a crack to occur at the construction joint. The estimated size of the shrinkage crack is larger than the deformations caused by thermal and gravity loading. Thus, no link-slab moments can be generated in the deck.

The authors concluded that the top layer of longitudinal reinforcement would be sufficient for link-slab reinforcement and that the top layer could be continued across the construction joint as a replacement for the current #6 bars for the link-slab reinforcement. If the skew is less than 15 degrees, the authors believe that the transverse reinforcement may be continued across the construction joint as is done in Texas. Otherwise the transverse reinforcement should be discontinued as is done currently. Further analytical studies are needed to confirm the effects of skew.

The maximum length of the bridge deck before expansion joints are required is dependent on the type of the expansion joints and their installation. If Georgia DOT continues to use the Evazote joint design for the expansion joint at each abutment, then the maximum length of the bridge should be 400 ft.

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Chapter 1

Introduction

1.1 Purpose and Objectives

The purpose of this research was to evaluate the performance history of continuous bridge decks in the State of Georgia, to determine why the current design detail between adjacent simple-spans works, to recommend a new design detail, and to recommend the maximum and/or optimum lengths of continuous bridge decks. The continuous bridge decks in Georgia have continuous reinforcement over the juncture between two edge beams along with a construction joint for crack control; this zone is often termed a "link-slab".

Expansion joints are a recognized problem within the bridge engineering community. Expansion joints are costly to install and maintain for several reasons. Spalling of the concrete deck in the vicinity of the joint and leakage from the joint onto the superstructure can reduce bridge life. Water leakage through the expansion joints causes deterioration of the surrounding structure and can also lead to corrosion of reinforcement (Caner and Zia, 1998). Debris accumulation around the expansion joints can restrain movement which may damage the bridge (Caner and Zia, 1998). The expansion joints reduce the ride quality of the bridges, and noise reduction measures must be frequently implemented in residential areas because of the noise from the cars driving over the expansion joints (Bridge, et al. 2005).

In Tennessee, the Kingsport bridge, which was approximately 2700 ft long, was opened in 1981 with the only expansion joints at the end abutments. The performance of this bridge suggests that the length of a continuous deck can be over 2700 ft (Burdette, et al., 2003). The main problem for the bridge in-service was leakage at the original finger joints installed at the abutments. The finger joints were replaced by modular expansion joints in 1997. Continuous bridge decks have been used in conjunction with continuous reinforced concrete pavement without any expansion joints in the construction of the Westlink M7, which is a 40 km privately financed toll road linking the M5 and M7 Motorways in Sydney, Australia (Bridge, et. al., 2005). The bridge movements are absorbed by the continuous system within a transition zone at each end of the bridge (Bridge, et. al., 2005).

The Georgia Department of Transportation (GDOT) currently builds bridge decks without expansion joints throughout the length of the bridge for lengths up to 400 ft. The GDOT Bridge and Structures Policy Manual states that expansion joints "are to be kept to a minimum because they always seem to leak or otherwise cause maintenance problems" (Liles, 2009).

1.2 Scope

In this research, five bridges were instrumented to determine the motion at their expansion joints and at their construction joint/link slab details. Finite element analyses were performed to analytically investigate the construction joint and link slab behavior. The instrumentation of the Georgia bridges was intended to provide greater insight into the actual behavior of the continuous deck detail. The data illustrate why the current

detail works. The primary focus of the instrumentation was to determine performance under thermal loads. The researchers instrumented the bridges with dial gauges to measure longitudinal movement and DEMEC points to measure expansion and contraction across the deck construction joints and expansion joints.

Finite element analyses were used to evaluate the potential magnitude and distribution of strains in the link-slab region of continuous deck bridges. Models were developed to represent one of the instrumented bridges. Field measurements were conducted to validate these models.

This report discusses field instrumentation of a pair of Bartow County bridges that were instrumented in March of 2011 and monitored through June 2012. Other instrumented bridges include structure 067-0213-0 in Cobb County and structures 077-5139-0 and 077-5141-0 in Coweta County. Results are presented that indicate the behavior of the monitored bridges under thermal loading conditions over a period of a little more than one year. In addition, the report discusses the finite element modeling that has been performed.

1.3 Link Slabs

The research focused on the specific type of bridge construction that utilizes link-slabs in the deck over intermediate piers. With this kind of bridge construction, the girders are designed as simply supported. The deck is cast continuously over the length between the adjacent girders. The length of the deck connecting the two adjacent simplespan girders is commonly referred to as the link-slab (Caner and Zia 1998). Figure 1.3.1 shows a typical link-slab detail as recommended by Caner and Zia (1998). For the

purposes of this report, the terms link-slab and continuous deck will be used interchangeably.

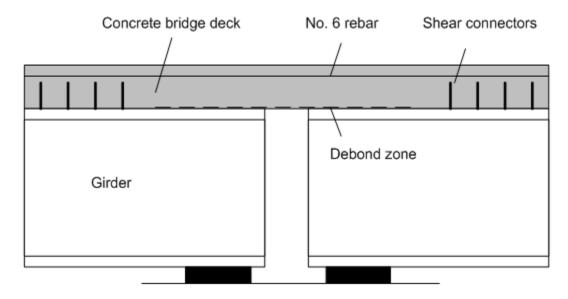


Figure 1.3.1: Typical link slab detail recommended by Caner and Zia (1998)

1.4 Construction Joints

Construction joints are used in the link-slab region over intermediate piers. The construction joints provide a location over the bridge pier at which a crack will tend to localize. A silicone sealant is used to seal the joint and to prevent water leakage that may cause corrosion of the deck reinforcement or damage to the structure underneath the joint. The silicone sealants are inexpensive and easy to maintain. The construction joint can be resealed if the original sealant is damaged as part of normal maintenance procedures. Figure 1.4.1 shows a detail of the standard construction joint used in continuous bridge decks by the GDOT. Figure 1.4.2 shows a photo of a construction joint in the field (Structure ID: 067-5186-0). Some damage to the concrete is evident in the vicinity of the joint. This aspect is discussed further in Section 3.2.

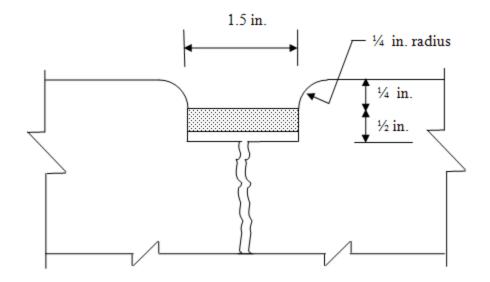


Figure 1.4.1: Standard construction joint for GDOT continuous deck detail



Figure 1.4.2: Construction joint in field with damage (Structure ID: 067-5186-0)

1.5 Expansion Joints

Expansion joints are full-depth joints that allow for expansion and contraction of the deck. GDOT currently uses expansion joints only at end abutments in bridges that are approximately 400 ft long or shorter (Liles, 2009). For bridges that are longer than 400 ft expansion joints at intermediate bents are "unavoidable" and "common" (Liles, 2009). Evazote expansion joints are preferred by the GDOT for continuous bridge decks (Capital Services, 2012). Figure 1.5.1 shows an expansion joint in the field (Structure ID: 015-5123-0). Figure 1.5.2 shows the installation of an expansion joint. Figure 1.5.1 shows some damage at the joint location. The extent of this damage observed in a number of bridges, and its significance, are discussed in Section 3.2.

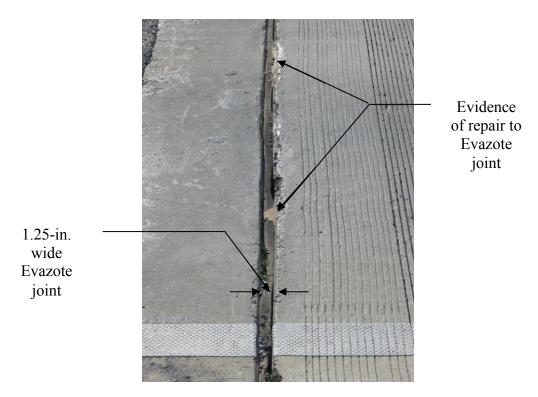


Figure 1.5.1: Evazote expansion joint in field (Structure ID: 015-5123-0)



Figure 1.5.2: Evazote expansion joint during installation

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Chapter 2

Link-Slab Details

In this chapter, link-slab details in Georgia, Texas, Florida, and North Carolina are discussed. The historical evolution of the Georgia detail is outlined. It is noted that the performance of each detail has been reported as satisfactory by the respective states.

2.1 History of Georgia Detail

GDOT began using a continuous deck detail in 1987. The original design for continuous decks was based on the design for continuous girder bridges. The design was guided by the AASHTO (1983) 3.24.10 provisions for longitudinal reinforcement in concrete decks. The AASHTO provisions stated that the total cross-sectional area of the longitudinal reinforcement shall not be less than 1% of the total cross-sectional area of the deck (AASHTO, 1983). The amount of reinforcement calculated for the longitudinal reinforcement across the construction joint was increased based on the design for heavily reinforced concrete beams and slabs to a reinforcement ratio of $\rho = 2\%$. When using 2% reinforcement and designing the slab for moment capacity, with width b = 12 inches and effective depth d = 6 inches, one would obtain a required area of steel of 1.44 in.²/ ft width, or #7 bars at 6 inches on-center spacing. One-half of this amount (selected arbitrarily) would yield #7 bars at 10 inches or #6 bars at 4 inches on-center. Two-thirds of the reinforcement was placed in the top layer, and one-third of the reinforcement would be placed in the bottom layer of reinforcement, as specified in AASHTO (1990) Section 3.24.10.

The resulting continuous deck design detail based on these decisions was two #7 bars at a length of 20 ft between each #4 bar in the top layer of reinforcement. The #4 bars in the top mat were spaced at 18-in. Under this design, the #7 bars and the top mat of #4 bars were continuous across the construction joint. An extra #4 bar was added in the bottom mat of longitudinal reinforcement. Based on AASHTO (1990) Specification Section 3.24.10, it was assumed that the #4 bar was placed in the middle half of the slab span. Figure 2.1.1 is a section view of an example bridge deck over a pier. It shows the layout of the #7 and #4 longitudinal reinforcing bars. The #7 bars are black circles, and the #4 bars are grey circles.

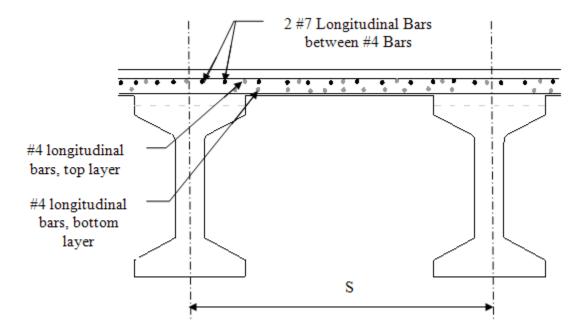


Figure 2.1.1: Section view through an intermediate bent at a construction joint with 1987 reinforcement layout (modified from GDOT SR 46 Over Oconee R. plan)

The amount of reinforcement going across the construction joint made it difficult for contractors to build and remove headers for containment of the concrete during the deck placement. In the late 1980s, the original design was modified to the one currently

used by the State of Georgia. The top mat of #4 bars was stopped 2 inches short of the construction joint on both of its sides. The #7 bars were replaced with #6 bars. The current design came about when the detail was modified for a second time in the early 1990s. The length of the additional #6 bars that went across the construction joint was reduced to 10 feet, 5 feet on each side of the joint. AASHTO (1990) Section 3.24.10 was used in determining the spacing of the reinforcement for the top and bottom layers in the deck. Figure 2.1.2 shows a plan view of the current Georgia continuous deck detail. An example cross section through the deck is shown in Figure 2.1.3.

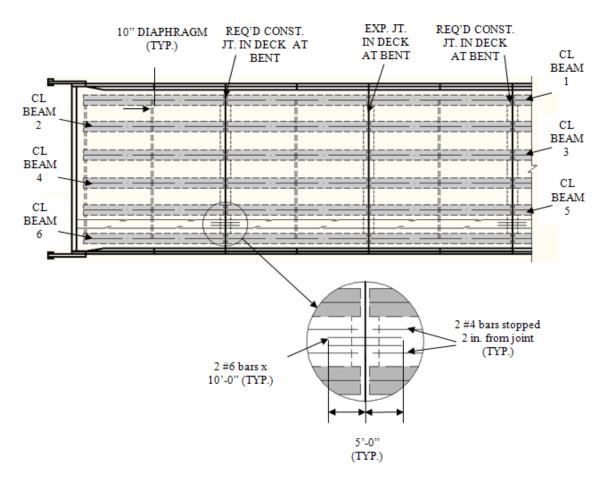


Figure 2.1.2: Plan view of current continuous bridge deck (modified from GDOT SR 46 Over Oconee R. plan)

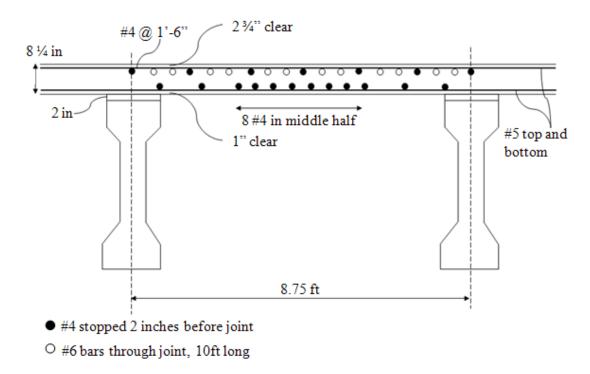


Figure 2.1.3: Example deck cross-section using current Georgia continuous-deck detail

The current design can be difficult to construct. Both the top and bottom mats of longitudinal and transverse deck reinforcement must be stopped 2 inches short of any construction joints in bridges that are skewed. This means that every transverse bar must be cut to a different length. Figures 2.1.4 and 2.1.5 show the configuration of the reinforcing steel around the construction joint in a representative skewed bridge.

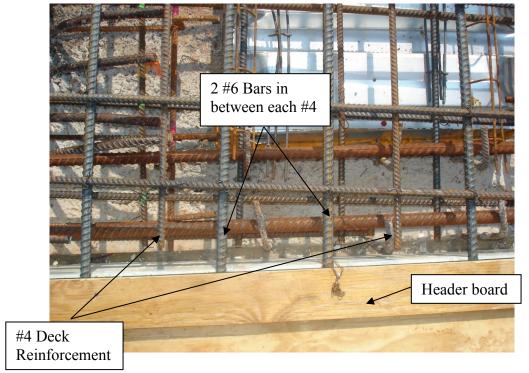


Figure 2.1.4: Georgia detail in the field

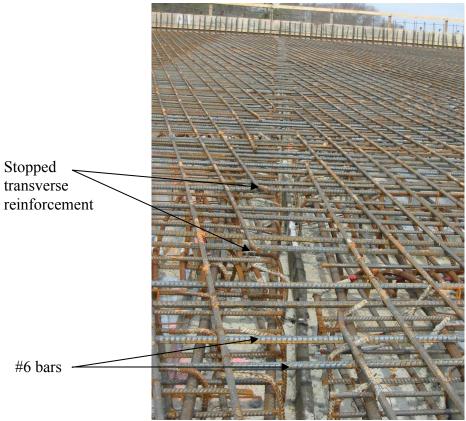


Figure 2.1.5: Skewed continuous deck bridge construction (bridge supporting US 27 at SR-1, Cedartown Bypass)

2.2 Texas Detail

The Texas detail does not use any additional bars across the link slab region. The design continues both the top and bottom mat of deck reinforcement across the construction joint (Simmons, 2001). For the longitudinal reinforcement, #4 bars are used in the top mat and #5 bars are used in the bottom mat. Transversely, #5 bars are used in both the top and bottom mats. Texas also addresses the constructability issues associated with skewed bridges by laying the transverse bars parallel to the skew in bridges with skews less than 15°. The transverse bars are perpendicular to the longitudinal direction in bridges with greater skews, but the bars are ended before reaching the construction joint. Plan views of the two variations of the Texas detail are shown in Figures 2.2.1 and 2.2.2. Bar sizes for the figures are given in Table 2.2.1. Figure 2.2.3 shows a typical cross section of the Texas continuous deck detail.

Table 2.2.1: TXDOT bar sizing for continuous deck detail

Bar	Size
A	#5
В	#5
D	#5
T	#4

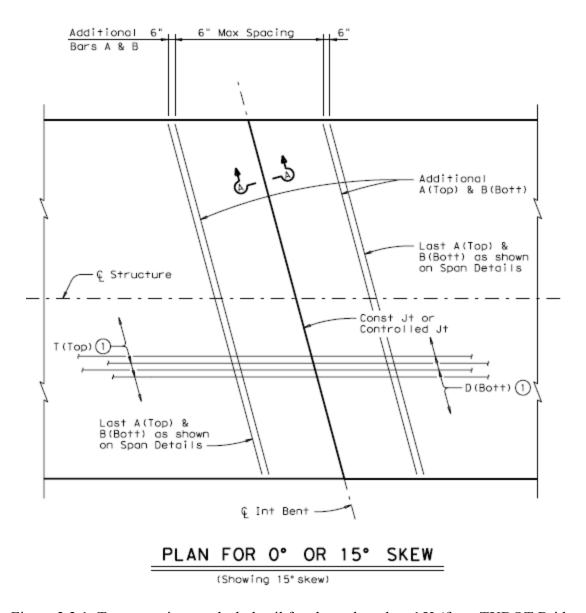


Figure 2.2.1: Texas continuous deck detail for skews less than 15° (from TXDOT Bridge Design Manual)

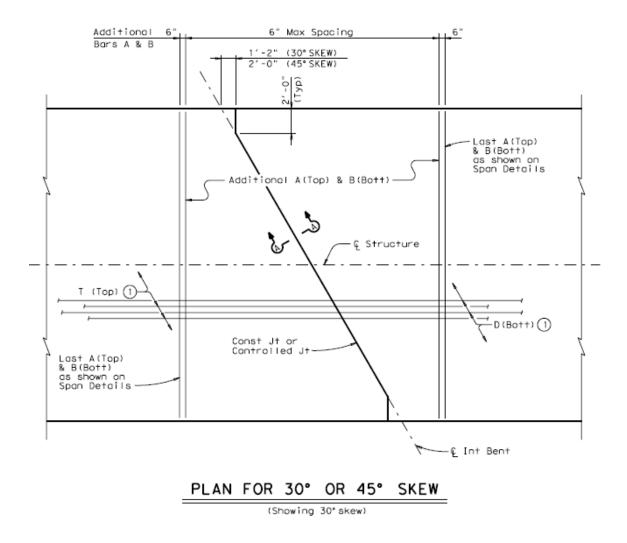


Figure 2.2.2: Texas continuous deck detail for skews of 30° or 45° (from TXDOT Bridge Design Manual)

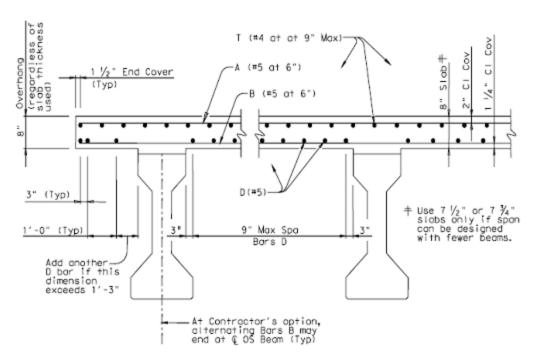


Figure 2.2.3: Texas DOT continuous bridge deck detail (Simmons, 2001)

2.3 Florida Detail

The continuous deck design detail used by Florida is found in section 4.2.6B of its bridge design manual (Robertson, 2009). Florida uses additional #5 bars in the top mat of longitudinal reinforcement in the link-slab region for its continuous deck design detail. All longitudinal reinforcement in the top and bottom layers is continuous over the construction joint. The Florida bridge design manual states that the additional longitudinal reinforcement over the joint shall meet the following criteria:

- "B. Size, space, and place reinforcing in accordance with the following criteria:
 - 1. No. 5 Bars placed between the continuous, longitudinal reinforcing bars.
 - 2. A minimum of 35 feet in length or 2/3 of the average span length whichever is less.
 - 3. Placed symmetrically about the centerline of the pier or bent, with alternating bars stagger 5 feet."

Florida also uses construction joints to control cracking in the bridge deck. The construction joints are placed at each end of the deck spans and at intermediate locations as required (Robertson, 2009). Figure 2.3.1 shows the continuous detail used by Florida.

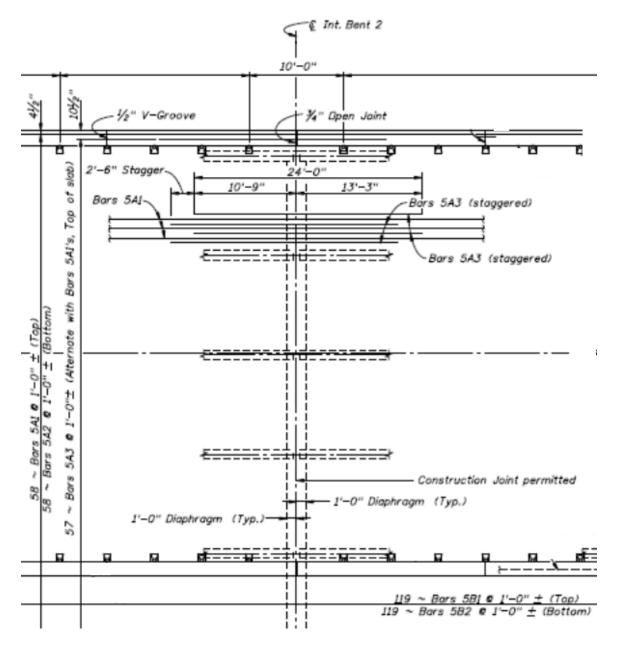
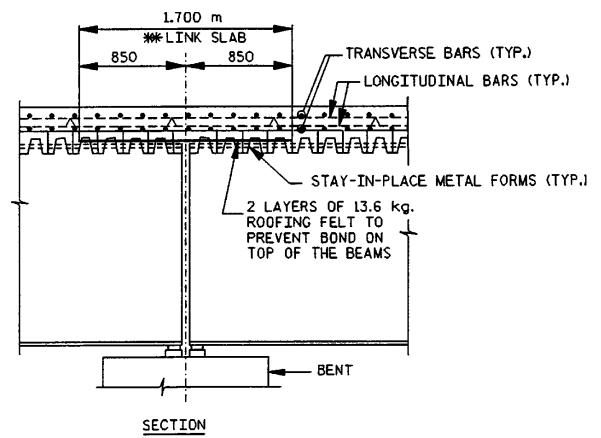


Figure 2.3.1: Florida continuous deck detail plan, 5A1 and 5A3 bars are #5 bars, and they run parallel to girders (from Robertson, 2009)

2.4 North Carolina Detail

North Carolina uses a design that has some different characteristics than the designs from the above states. The key difference for the North Carolina design is that the deck is debonded from the top of the girder in the link slab region (Wing and Kowalsky, 2005). Shear studs are removed and a felt material is inserted in between the deck and the girder for a length of 5% of the girder span length. The reinforcement that goes across the link slab is calculated based on the demand induced from the maximum end rotations of the simply supported girders when subjected to service loads. Figure 2.4.1 shows an example of the North Carolina design along with a steel girder. Florida and North Carolina do not provide information on transverse reinforcement or adjustments to the detail when skews are used.



*** NOTE: THE TOP OF THE BEAM IN THE REGION OF THE LINK SLAB SHALL BE SMOOTH AND FREE OF SHEAR CONNECTORS. NO WELDING OF FORMS OR FALSEWORK TO THE TOP FLANGE WILL BE PERMITTED IN THIS REGION.

Figure 2.4.1: North Carolina link slab design (from Wing and Kowalsky, 2005)

Chapter 3

Field Evaluation of Georgia Bridges

3.1 GDOT Maintenance Report Findings

GDOT maintenance reports were reviewed for bridges with prestressed, simple-span concrete girders to find any maintenance issues or consistent problems with the construction or durability of the continuous deck detail. The Maintenance Office does not specify which bridges have continuous decks in their maintenance reports. Therefore, bridges built with prestressed, simple-span concrete girders after the year 2000 were assumed to have continuous bridge decks, and the maintenance reports for those bridges were reviewed. The total number of bridges built after the year 2000 that meet these requirements is 244. During the review, the structure identification number, the GDOT district, the maximum span length, number of spans, structure length, leaks, leak location, deck condition and expansion joint types were recorded for each bridge if the information was available.

Using this information, the frequency of cracking, joint failure, or a combination of both was tabulated. The information distinguishes between abutment joints and joints within the span but does not indicate if the joints are expansion joints or construction joints. All abutment joints are expansion joints. Out of the 244 bridges, 93 of the bridges have cracking, joint leakage or joint failure reported; therefore approximately 38% of the bridges have a reported problem. Of the 93 bridges, 64 reported cracking. Forty-six of the 64 bridges have their cracking described as minor, very minor, or superficial which is

approximately 72% of the 64 bridges with cracking. Twenty-three of the 64 bridges attribute the cracks in part to shrinkage or settlement. Eighteen of the 64 bridges reported cracking at joints for a percentage of approximately 28%, or for an overall percentage of about 7%. Twelve of the 17 bridges had cracks occurring at abutments, and 1 of the 17 had cracking at only one joint. With 12 of the 17 joint cracks occurring at the abutments, only about 2% of the overall bridges reported cracking at construction joints in the bridge deck link slabs. The amount and types of cracking described are consistent with that described in the subsequent sections of this chapter.

Twenty-nine of the 244 bridges reported leaking at expansion joints or expansion joint failures, which is about 12% of the total reviewed. Six of the 29 reported joint failure, and 1 of the 6 was a joint failure at the abutment. Eleven of the 29 reported leaking joints occurred at abutments. Seventeen of the 29 bridges with joint failure or leaks occurred at joints in the bridge for approximately 7% of the total number of bridges. Three had leaking at only one joint, and one had leakage at all joints. Two of the 29 bridges with joint leaks were continuous deck bridges "built without construction joints", and leaking "occurred at the bottom of the deck at the [expansion] joints" (Mealer, 2007). No further information was given in the maintenance report regarding how the bridge was built.

The span lengths of the 93 of 244 bridges that reported cracking, joint leakage, or joint failure were also reviewed to see if span lengths affected the reported problems.

The span lengths for the 244 bridges ranged from 29 ft to 150 ft. The span lengths of the bridges reporting cracking were from 40 ft to 141 ft. The span lengths of the bridges reporting joint failure or leakage ranged from 39 ft to 140 ft. Neither of the groups

included the shortest or longest span reported. The total lengths of the bridges were also examined to identify any patterns. The total bridge lengths range from 68 ft to 3062 ft.

No evidence was found suggesting that the lengths of the spans or the overall bridge length affected the behavior of the bridges at the juncture of the edge beams in the linkslab detail. All reported issues were found in bridges with varying span and total lengths, and no consistent pattern was evident.

The problems with cracking, joint failure, and joint leakage usually appear to be unrelated to the continuous deck detail. Environmental conditions during construction, improper curing, and other issues can also lead to cracking in the bridge. Maintenance bridge inspectors attributed cracking in half of the bridges in part to shrinkage or settlement. Maintenance inspectors also recorded only five bridges with cracking specifically at joints other than at the abutments. The abutments of a bridge undergo a different set of forces than the joints in the bridge deck. The earth pressure, soil-structure interaction, and interaction with the approach slab should all be considered in addition to the shrinkage, creep, temperature, live, and dead loads on the bridge. These effects and the abutment design were outside the scope of this project. In addition, joint failure and leakage can be attributed to problems during construction or improper installation. Twelve of the bridges reported leakage or failure at abutments, and two of the bridges were "built without construction joints" which caused the deck to leak "at the bottom," on the underside of the deck (Mealer, 2007). No further information was given regarding the two bridges built without construction joints. Based on the GDOT maintenance report review, a small percentage of bridges have reported problems with cracking at the joints and joint leakage and failure. Only 2% of the total 244 bridges have cracking

occurring at construction joints. Seven percent of the total had joint leakage or failure. All problems were found in bridges of varying span lengths and total lengths with no evidence of length affecting any of the results. The problems could not be tied to the continuous deck detail. Construction conditions and installation of expansion joints should be considered. With such a small percentage of problems and no guarantee they were caused by the continuous deck detail, the current GDOT continuous deck detail appears to be working satisfactorily. The GDOT maintenance personnel did not report any consistent or noticeable problems with the continuous deck detail during interviews.

3.2 Selection of Bridge Sites

To evaluate the current design detail for continuous deck bridges used in Georgia, a number of existing bridges were selected and studied. Examining bridges built with the current GDOT detail for any type of distress provides insight into how well the design is working.

To begin the process of site inspection, accessible bridges were identified using the BIMS (Bridge Information Management System) software at the Georgia Department of Transportation. First, a list of all of the concrete bridges in Georgia was pulled from the system along with selected properties that were deemed helpful to narrow the list down to a reasonable number of bridges. These properties included the county where the bridge was located, the bridge's Structure ID number, the type of service under the bridge, year of construction, skew, deck condition, total length, and span length. For the bridges that passed over a waterway or relief for a waterway, the bridge height and water depth were also available.

The original list of 1470 bridges was narrowed by deleting bridges that did not meet the necessary requirements for the project. These requirements are given in Table 3.2.1 below. The total deck length was kept below 400 feet to facilitate access, as well as based on the recognition that bridges with a total length larger than 400 ft would be subdivided into multiple units by expansion joints within the bridge based on current GDOT practice. The number of spans needed to be at least two since the bridge must include at least one intermediate bent. The target span length was greater than 50 feet since bridges with spans of less than 50 feet would likely have little cracking. All bridge ages were considered, but bridges built after 1999/2000 were emphasized since they would use the current continuous-deck design detail. Specific selected types of "under bridge service" were chosen to avoid high-traffic roads under the bridge to allow research personnel to inspect safely and without disrupting traffic. Bridge height was kept to a maximum of approximately 20-25 feet, for ease of access during field inspection. Once the list was narrowed down to 206 bridges, additional properties were pulled from BIMS, including geographic coordinates and girder, for use in selecting bridges to survey and instrument.

Table 3.2.1: Required parameters for field study

Total Deck Length	Max 400 feet				
Number of Spans	At least 2 spans				
Span Length	Between 50 and 150 feet				
Bridge Age	1999/2000 and later for current design, but all ages were considered in query				
Under Bridge Service	Waterway, relief for waterway, pedestrian/bicycle path, miscellaneous				
Bridge Height	Ideally under 20 feet				
Girder Type/Shape	Precast, prestressed concrete (link slab)				

The first bridges selected for the project came from the "pedestrian/bicycle path" category of "under bridge service," because this was assumed to be the easiest type of bridge to access. These were bridges that passed over pedestrian/bicycle paths. There were eight bridges in this category, and the four that passed over the Silver Comet Trail (three in Cobb County and one in Paulding County) were selected for inspection due to their proximity to Atlanta and their ease of access. Since the bridge height was unavailable for any bridge that did not pass over water, an initial visit was made to these four Silver Comet Trail bridges to see which ones would be accessible for investigation. A photo of the bridge over Silver Comet Trail, Structure ID 067-5186-0, in Cobb County is shown in Figure 3.2.1. The three bridges in Cobb County were found to be appropriate for the study, and the parameters of these bridges are given in Table 3.2.2.



Figure 3.2.1: Girder access shown for Silver Comet Trail bridges, Structure ID 067-5186-0, in Cobb County

The next bridges chosen for inspection were two bridges under the same Structure ID number (015-5123-0) in Bartow County that passed over the waterway, Dry Creek. These bridges were chosen due to their proximity to Atlanta as well as the recorded water depth of zero feet, which was taken to indicate relatively easy access. The parameters of these bridges can also be found in Table 3.2.2. The height represents the maximum vertical clearance under the bridge.

The values for the deck condition shown in the table range from 0-to-9, with 0 meaning failure and 9 meaning excellent condition.

The bridge plans were obtained for all of the above bridges for use in mapping cracking in the decks, and so that additional details about the bridges could be

determined. This included the height for the Cobb county bridges since this information was unavailable in BIMS.

Table 3.2.2: Parameters of bridges selected for study

Structure ID	County	No. Spans	Max Span Length (ft)	Skew	Height (ft)	Year Built	Deck Cond	Girder Type
COB1 (067-0213-0)	Cobb	3	102	45°	24.25	1995	8	PSC "I" Beams
COB2 (067-5186-0)	Cobb	3	73	15°	23.0	1998	7	PSC "I" Beams
COB3 (067-5208-0)	Cobb	3	106	45°	23.58	1999	8	PSC "I" Beams
BAR1 (015-5123-0 North)	Bartow	3	70	15°	18.2	2006	7	PSC "I" Beams
BAR2 (015-5123-0 South)	Bartow	3	70	15°	18.2	2006	7	PSC "I" Beams
COW1 (077-5139-0)	Coweta	2	72	5°	16.0	2005	8	PSC "I" Beams
COW2 (077-5141-0)	Coweta	2	72	5°	16.0	2005	8	PSC "I" Beams

3.3 Inspection of Bridge Decks

The bridge decks of the five bridges introduced in Section 3.2 were inspected for cracks and other deterioration, and this information was then compared to the deck condition listed in the BIMS database. The two Bartow County bridges crossing Dry Creek are reviewed first. These two bridges are identical. The elevation and plan views are shown in Figures 3.3.1 and 3.3.2.

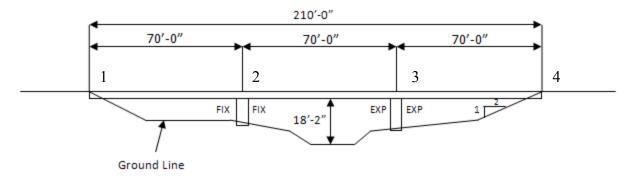


Figure 3.3.1: Elevation view of Bartow County bridge (BAR1 and BAR2)

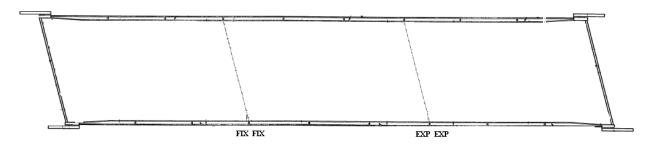


Figure 3.3.2: Plan View of Bartow County bridge

These bridges were found to be in very good condition, with only a few diagonal cracks at the corners of the deck at bents 1 and 4 in both bridges, likely from the skew, and a few other cracks perpendicular to the expansion joint at these bents. No cracks were found at either link-slab location (over bents 2 and 3). The lack of cracking was not consistent with the deck condition rating of 7, since this deck appeared to be in much better condition than one of the other bridges that received a rating of 8.

The elevation and plan views of the Cobb County Structure ID 067-0213-0 over the Silver Comet Trail and the abandoned CSX railroad are shown in Figures 3.3.3 and 3.3.4. This bridge was also in very good condition, with only a few small cracks at the north end of the bridge (bent 1), one crack and small spalling (missing pieces less than 1 in. in diameter) at bent 2, and some plant growth in the expansion joints at bents 1 and 4, which was consistent with its deck condition rating of 8.

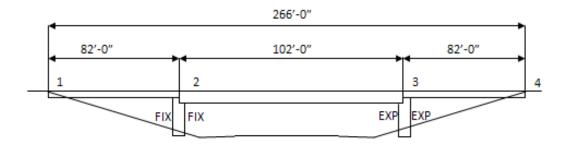


Figure 3.3.3: Elevation view of the Silver Comet Trail bridge (COB1)

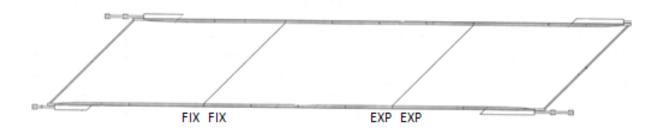


Figure 3.3.4: Plan View of Silver Comet Trail bridge, Cobb County

It can be observed that bent 2, where some cracking occurred, had fixed rather than expansion bearings on each side of the link slab. Figure 3.3.5 is a photo of the cracking adjacent to the construction joint at bent 2; Figure 3.3.6 is a photo of plant growth in the expansion joint at bent 1.



Figure 3.3.5: Cracking at bent 2 of Silver Comet Trail bridge (COB1)



Figure 3.3.6: Plant growth at bent 1 of Silver Comet Trail bridge (COB1)

The elevation and plan views of the Silver Comet Trail bridge, Cobb County, Structure ID #067-5186-0, are shown in Figures 3.3.7 and 3.3.8. This bridge had cracking in the corners of the deck at bents 1 and 4 due to the skew, but it also had a significant amount of spalling around the construction joints at bents 2 and 3, as well as in the middle of span 2. In addition, there were tiny meandering surface cracks along the entire deck over the entire length of the bridge, and an extremely wide opening in the expansion joint at bent 4, approximately 4.5 inches, to the extent that the joint filler had detached from the concrete at a number of points.

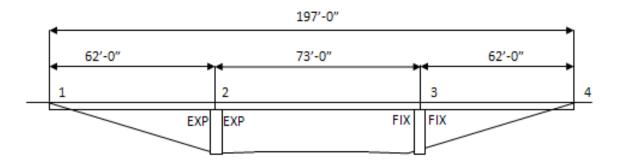


Figure 3.3.7: Elevation view of Cobb County bridge (COB2)

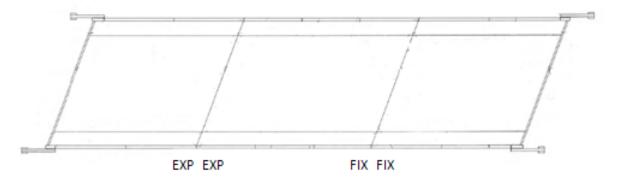


Figure 3.3.8: Plan View of Cobb County bridge (COB2)

This bridge had a deck condition rating of 7, which might be considered somewhat high due to the spalling at bents 2 and 3 and the widened expansion joint at bent 4. Photos of the damage at these locations are provided in Figures 3.3.9 and 3.3.10.

The elevation and plan views of the Silver Comet Trail bridge, Structure ID #067-5208-0, Cobb County, are shown in Figures 3.3.11 and 3.3.12.

The condition of this bridge deck was the worst of the decks examined during the study. There were 6.25" wide patches of new concrete in the deck adjacent to the construction joints over bents 2 and 3 with cracks perpendicular to the joints along their lengths. A 4'-11" piece of the patch in the deck over bent 3 was spalled off to the point where rebar was exposed. The orientation of the exposed reinforcement indicated that it was one of the bars in the edge beam that extends up into the deck. In addition, there were diagonal cracks at the corners of the deck at bents 1 and 4 due to the skew, and the expansion joint at bent 1 was 3 inches wide but in good condition and the joint filler was still attached to the concrete. Photos of the exposed reinforcement are shown in Figures 3.3.13 and 3.3.14.

The extensive damage at the above construction joint was most likely due to the following: the corners of the joint seemed to be squared off instead of rounded as the Georgia detail (Figure 1.3.4) specifies. This led to spalling of the concrete at the edge of the joint due to impact from vehicle tires. The patch repair then failed due to shrinkage of the repair material.



Figure 3.3.9: Spalling on bent 2 of Silver Comet Trail bridge (COB2)



Figure 3.3.10: Evazote joint (3.5"-4.5") at bent 4 of Silver Comet Trail bridge (COB2)

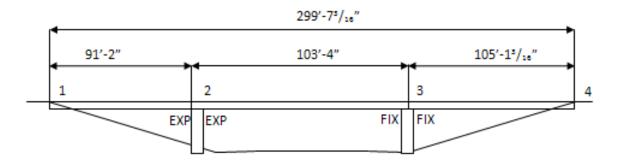


Figure 3.3.11: Elevation view of Cobb County bridge (COB3)

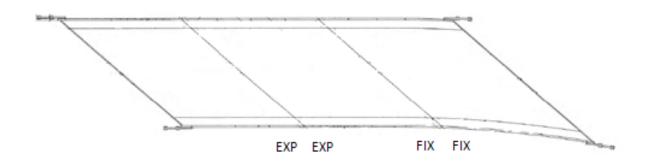


Figure 3.3.12: Plan View of Cobb County bridge (COB3)



Figure 3.3.13: Exposed rebar in patch over bent 3 on Silver Comet Trail bridge (COB3)



Figure 3.3.14: Exposed rebar in patch over bent 3 on Silver Comet Trail bridge (COB3)

Overall, cracking at the intermediate bents was observed in two of the five bridges, with the Silver Comet Trail bridge, Structure ID #067-5208-0, Cobb County, having the cracking at these bents concentrated within the concrete patches. Cracking that occurred at construction joints seemed to be primarily due to improper forming of the joints. The joints were squared off instead of being rounded as shown in Figure 3.3.15.

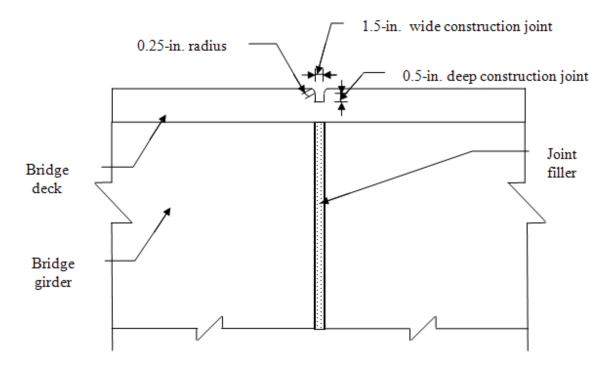


Figure 3.3.15: Detail of construction joint

Most of the cracking at all four bridges occurred at the abutments. It was hypothesized that problems with the expansion joints were largely due to improper installation. In bridges where the expansion joints were torn up or pulled away from the deck and abutment, it is possible that the evazote joints were not installed at the proper widths or specified temperatures.

3.4 Instrumentation of Decks and Girders

Five bridges were instrumented to evaluate the motion at the link slab construction joints and at the expansion joints at the ends of the bridges. These bridges are listed in Table 3.4.1. The instrumentation was intended to provide a better understanding of the behavior of the link-slabs. The primary focus of the instrumentation was to evaluate the movements under thermal loads. Wax plates were used to provide a coarse, inexpensive measurement of the girder movements. The five bridges were instrumented with dial gauges to measure longitudinal movement at the ends of bridge girders relative to the abutment or the bent. DEMEC points were mounted into the top of the deck adjacent to the construction and expansion joints to measure expansion and contraction across the joints. These field measurements, along with finite element modeling, were used to evaluate the bridge behavior.

Table 3.4.1: Instrumentation summary

ID	Structure ID	Wax Plates	Full Dial Gage Stands	Bottom Flange Dial Gages	DEMEC	Temperature
BAR1	015-5123-0 (South)	✓	✓	✓	✓	✓
BAR2	015-5123-0 (North)	✓		✓	✓	✓
COB1	067-0213-0			✓	✓	✓
COW1	077-5139-0			✓	✓	✓
COW2	077-5141-0			✓	✓	✓

3.4.1 Wax Plates

Wax plate displacement measurements were similar to those described by Chen, Helwig, Herman, & Writer (2007). Wax plates were placed at the bottoms of the ends of the girders in the center girder line of the Bartow bridge in August 2010 and were monitored through June 2012. This was the first type of instrumentation used due to its ease of fabrication and ease of placement.

The simple device shown in Figure 3.4.1 was comprised of two main components: a stylus, approximately six inches in length that was attached to the girder, and microcrystalline wax in a polystyrene Petri dish affixed to the pier cap with caulk. A two-inch square metal plate with a perpendicular arm and ¼-inch nut was attached to the concrete girder with a quick setting epoxy. The stylus, created from ¼-inch threaded rod and sharpened by a lathe on one end, could be adjusted vertically in the nut, as needed to reach the microcrystalline wax in the Petri dish. A second nut was used to prevent accidental movement of the stylus.

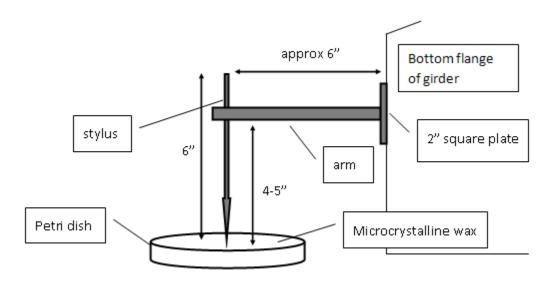


Figure 3.4.1: Detail of wax plate displacement device

Microcrystalline wax was used for the scratch-plate because it is soft enough to score, but melts at temperatures between 140-175° F, well above daily averages in hot climates such as in Georgia. The polystyrene Petri dish is less likely to break than traditional glass, which makes it a better option for use in the field. A ring of concrete-grade caulk affixed the Petri dishes, and served as a location guide for replacing old wax plates with new ones in the event that different days or seasons should be monitored separately. As shown in Figure 3.4.2 below, the wax had a blue alcohol-based stain that was sprayed over the wax so that a light scratch from the stylus would be easily identified through the colored coating on the white wax.



Figure 3.4.2: Photo of an in-place wax plate at the Bartow County bridges (StructBAR1, interior bent)

3.4.2 Dial Gage Stands

On the Bartow County bridge (BAR1), dial gauges (Figure 3.4.3) were installed at each end of the interior three lines of girders. In addition, DEMEC points were installed in the deck. These devices were used to measure the girder displacements at the bearing, the girder end rotations, and the deck movement at the construction joints associated with the link-slab details. These dial gage and DEMEC measurements were taken at different temperatures so that thermal movements could be determined.

The dial gauge system relied upon two assumptions. The first assumption was that the pier caps are very stiff and do not move. The second assumption was that the edge beams that are placed against the girders deflect the same as the girders. The top two dial gauges on each stand were placed against the edge beam. At the abutments, every measurement was taken against the end beams. Figure 3.4.3 shows a schematic of how the dial gauge systems worked along with a photograph of one placed in the field. The schematic shows how the systems were arranged at the intermediate bents. At these piers, the bottom dial gauge contacted a target on the girder's bottom flange. The photo shows one of the systems installed at an abutment at which the bottom gauge contacted the end beam.

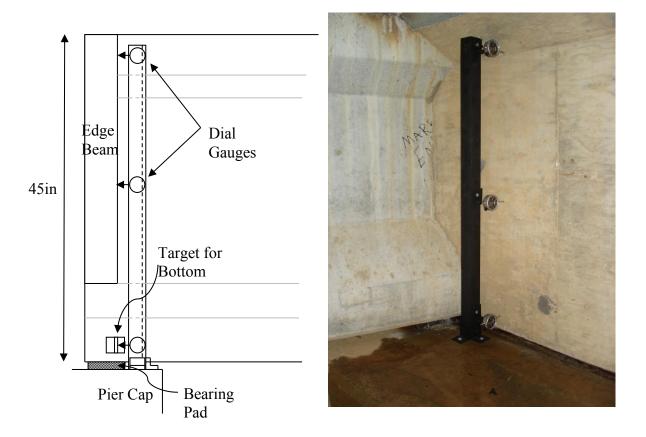


Figure 3.4.3: Dial gage system: Elevation diagram (left), Installed on Bartow bridge (right)

3.4.3 DEMEC Points

DEMEC points were installed in the top of the bridge deck at every joint (both expansion joints at end bents and construction joints at the interior bents). The DEMEC points were placed in the right shoulders of roadways for safety purposes. Figure 3.4.4 shows the locations of DEMEC points in the deck. Figure 3.4.5 shows a pair of DEMEC points installed at a construction joint.

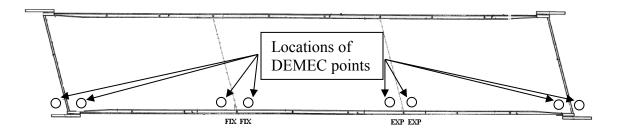


Figure 3.4.4: Locations of DEMEC points

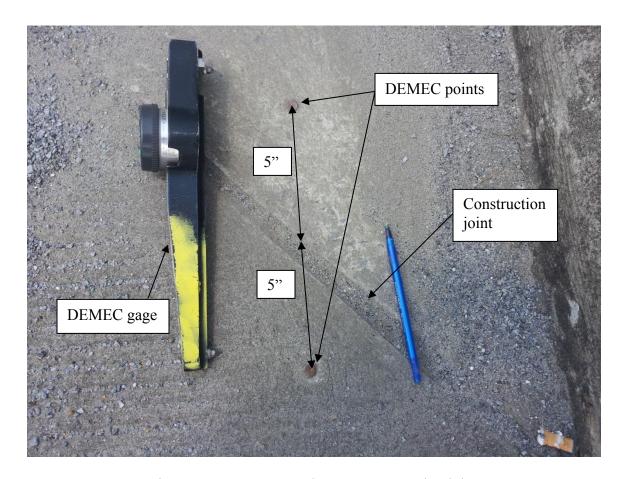


Figure 3.4.5: DEMEC points at a construction joint

A screw was inserted into the deck on each side of each joint. Each screw was located 5 inches from the joint. The tops of the screws contained a notch so that a DEMEC gauge could be inserted and give a reading of how far apart the two points were. These points were used to determine any contraction or expansion of the joints at the

deck surface due to temperature changes. Figure 3.4.6 shows a DEMEC reading being taken.



Figure 3.4.6: DEMEC reading

3.4.4 Bottom Flange Gages

The south Bartow bridge span (BAR1) was instrumented with dial gage stands that took measurements at three points through the depth of the girder. Four other bridges were instrumented with dial gages that took measurements only at the bottom flange of the girders. For these bridges, the dial gages were only installed on the center girder line. Table 3.4.1 shows a list of instrumented bridges and the type of devices installed on them.

3.4.5 Temperature Measurements

The dial gage and DEMEC instrumentation on the Bartow bridge BAR1 was in place from March 2011 through June 2012 (Note that the wax plates were placed seven months earlier because of recommendations by Chen, et al., 2007). Measurements primarily consisted of morning and afternoon readings of ambient air temperature, deck surface temperature, air temperature under the bridge, girder surface temperature, dial gauge readings, and DEMEC measurements.

Air temperatures were measured with a standard thermometer. All surface temperature readings were collected using a K-type surface thermocouple connected to an Omega HH11B digital thermometer. Figure 3.4.7 shows a temperature reading being taken at the deck surface. Morning and afternoon readings allowed for analysis of the bridge behavior corresponding to daily temperature variations as well as long-term seasonal variations.



Figure 3.4.7: Deck surface temperature reading being taken with thermocouple

Chapter 4

Results and Discussion of Field Evaluations

4.1 Wax Plate Results

The wax plates were located on the center line girders of both Bartow County bridges for more than a year. Measurements were taken at the bottom of the girders at their ends (essentially the girder longitudinal displacements at the bearings). Stations 1 and 4 corresponded to end abutments where expansion joints existed. Stations 2A and 2B are at bent 2. The station layout is shown in Figure 4.1.1. Both bearings at bent 2 are termed "fixed," having a 1.25-in. diameter pin inserted into a 2 in. diameter circular hole in the base of each girder. Stations 3A and 3B were at bent 3. Both bearings at bent 3 were termed "expansion," having a 1.25-in. diameter pin inserted into 1.75 in. wide by 5.625 in. long slotted hole in the base of the girders).

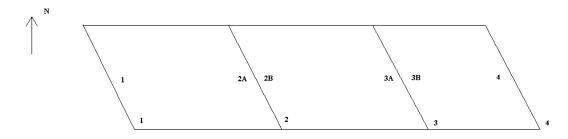


Figure 4.1.1: Station layout for Bartow County bridges

The results provided in Table 4.1.1 are values for both bridges. Both bridges exhibited identical behavior (i.e., the scratch lengths were the same). The plates were left in place for a full year so that the scratch lengths indicate full range of seasonal

expansion/contraction at the bearings. The plates were placed in August at a time of near maximum temperature. At intermediate temperatures, the stylus was observed to be located at a position somewhere in the middle of the scratch. Figure 4.1.2 shows the high and low recorded temperatures between March 2011 and June 2012. These maximum and minimum temperatures were taken as the ambient air temperatures that occurred over the period of time during which the wax plates were in the field. Figure 4.1.3 shows the plate located at station 4.

Table 4.1.1: Wax plate results

Lengths of Scratches in Wax Plates									
Bartow Bridge 1 (BAR1)									
Station	Station 1 2A 2B 3A 3B 4								
Length (in)	0.62	0	0	0.38	0.5	0.62			
Barto	Bartow Bridge 2 (BAR2)								
Station	1	2A	2B	3A	3B	4			
Length (in)	0.62	0	0	0.38	0.5	0.62			

Max Temp (°F)*	97
Min Temp (°F)*	26

^{*}Recorded Temperatures from National Weather Service

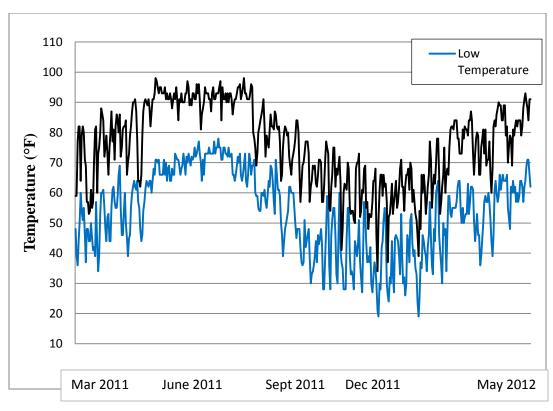


Figure 4.1.2: Daily high and low temperatures from March 2011 to June 2012 (from National Weather Service)



Figure 4.1.3: Wax plate after a year in the field

The seasonal temperature changes are primarily associated with an overall, uniform temperature change of the girders and deck. Therefore, the corresponding displacement at the end bearings should be similar to the amount of contraction or expansion that occurs at the expansion joint in the deck above the bearing. The wax plate marks indicated that the total movement at each expansion joint in this bridge with an overall span of 210 feet, was 0.62 inches. Figure 4.1.4 shows graphically the movements measured at the ends of the bridge.

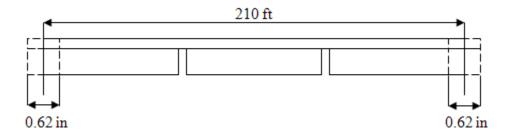


Figure 4.1.4 Movement at abutments due to seasonal temperature changes

The total length change values can be compared to the design thermal movements given in AASHTO (2007) 3.12.2.3. AASHTO equation 3.12.2.3-1 is:

$$\Delta_T = \alpha L \left(T_{MaxDesign} - T_{MinDesign} \right)$$

where

L = expansion length (in.)

 $\alpha = coefficient of thermal expansion (in./in./°F)$

The coefficient of thermal expansion for concrete can be taken as $6x10^{-6}$ in./in./°F (Neville, 1996). Using the above equation and an L value equal to each span length of 70 ft. = 840 in. with the temperature range of 70° F observed in the field, the total thermal

movement is 0.3528 in. per span. Summing the three spans gives a total expansion of 1.0584 in. The AASHTO value overestimates the total measured movement in this bridge due to the seasonal thermal effects by 100 (1.06 in - 0.62 in) / 0.62 in = 69%.

4.2 DEMEC Results

The five bridges listed in Table 3.4.1 had DEMEC points installed to determine the movements at the expansion joints and construction joints due to daily temperature variations. Readings were taken in the morning and in the afternoon. The surface temperature of the deck was taken at shoulder locations using the instrument described in Section 3.3.5. In addition, the length across the joints was measured in the morning and in the afternoon. The difference in the two values was taken as the movement across joint. Table 4.2.1 shows the values obtained for each bridge. Positive values indicate that the joint comes together and negative values indicate that the joint is spreading apart. This is illustrated in Figure 4.2.1.

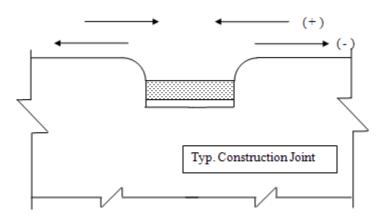


Figure 4.2.1: DEMEC sign convention

Table 4.2.1 DEMEC results, joint closing (+) and opening (-) (inches)

Bartow Bridge 1: 015-5123-0

	Bent							
Temp. Change								
(°F)	1	2	3	4				
32.4	0.076	0.0026	0.0016	0.0505				
32.5	0.119	-0.0106	-0.0087	0.0987				
29.4	0.1174	0.0043	-0.0025	0.0865				
35	0.048	0.0111	0.0017	0.0931				
25.2	0.098	0.004	0.0075	0.103				
20.4	0.079	0.001	-0.0021	0.0688				
21.3	0.081	0.0032	0.0011	0.079				

Bartow Bridge 2: 015-5123-0

	Bent								
Temp. Change									
(°F)	1	2	3	4					
31	0.105	0.0052	-0.001	0.0943					
32.5	0.121	0.0031	-0.0043	0.108					
29.4	0.0972	0.0037	-0.0009	0.095					
35	0.1223	0.0056	0.0004	0.0955					
25.2	0.096	0.0033	0.0016	0.0974					
20.4	0.084	-0.004	0.0013	0.077					
21.3	0.087	0.0027	-0.0017	0.069					

Cobb Bridge 1:067-0213-0

	Bent							
Temp. Change								
(°F)	1	2	3	4				
32.4	0.1119	0.001	-0.0026	0.0774				
32.5	0.124	0.0021	-0.0024	0.081				
29.4	0.103	0.0036	0.0027	0.083				
35	0.131	-0.004	0.0017	0.109				
27	0.097	-0.007	0.003	0.09				
22.6	0.098	0.0014	0.0011	0.084				
19.8	0.094	0.002	-0.0014	0.072				

Coweta Bridge 1:077-5139-0

	Bent						
Temp. Change							
(°F)	1	2	3				
28.1	0.071	0.002	0.068				
22.4	0.078	0.0013	0.074				
21.3	0.062	-0.0014	0.064				

Coweta Bridge 2:077-5141-0

	Bent						
Temp. Change							
(°F)	1	2	3				
28.1	0.09	-0.0011	0.077				
22.4	0.073	0.0019	0.075				
21.3	0.071	0.0017	0.061				

The DEMEC measurements were taken over a ten inch gauge length centered over the joint. At the time of their installation, the points were 10 inches apart.

Temperature increases generally resulted in the closing of the joint at the surface of the deck. Figure 4.2.2 is a plot of the above data and shows how each joint behaved for the Bartow Bridge 1. The figure shows consecutive readings that do not necessarily occur on consecutive days.

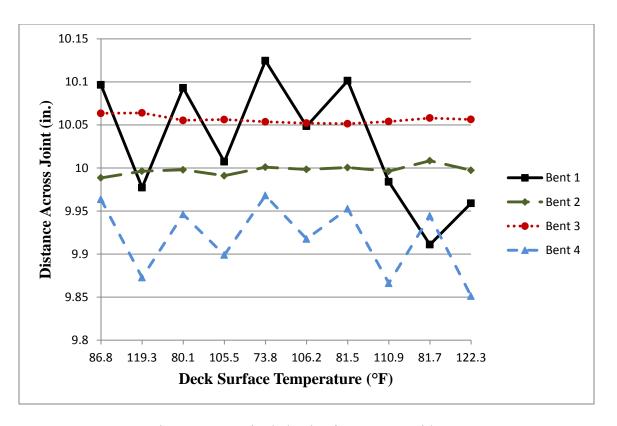


Figure 4.2.2: Joint behavior for Bartow Bridge 1

The data show that, as one would expect, in every case there is larger movement at the expansion joints (Bent 1 and 4) than at the link-slab construction joints. The movement at the expansion joints is also a closing of the joint in every case for temperature increase, which indicates deck expansion with temperature increase as expected. The expansion and contraction is smaller at the link-slab locations. The variations for the construction joints 2 and 3 are approximately the same, indicating that the behavior of the link slab is independent of the type of bearing that it is over (fixed-fixed or expansion-expansion).

4.3 Full-Height Dial Gage Stand Results

The dial gauge system shown in Figure 3.3.3 was placed on all three of the interior girder lines on the south Bartow bridge (BAR1) as indicated in Table 3.3.1. The station numbering is shown in Figure 4.1.1. The maximum daily temperature change in the deck was 35°F. On the same days that these temperature changes were measured, there was essentially no change in the temperature of the girders below the deck. Table 4.3.1 shows the average of the dial gauge results for each girder for each day that a maximum temperature change of 35 degrees was observed.

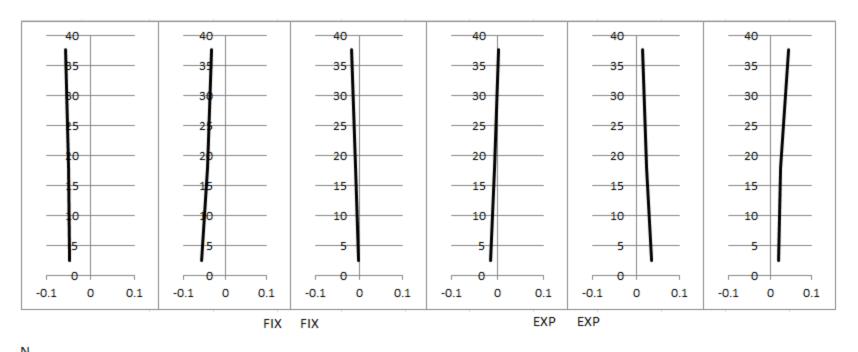
Table 4.3.1: Dial gage stand results (inch)

		1	2A	2B	3A	3B	4	Height above pier cap (in)
,	Top	-0.075	-0.042	-0.016	0.007	0.024	0.054	37.75
M	liddle	-0.057	-0.036	-0.002	-0.008	0.026	0.025	18
В	ottom	-0.042	-0.072	0.004	-0.019	0.054	0.028	2.5

Positive = movement toward the East

Negative = movement toward the West

The measurements of the longitudinal movement of the girders were taken at heights of 2.5 in., 18 in., and 37.75 in. above the top of the pier cap. Plotting these values gives an indication of the rotation at the end of the girder, and the measurements can be used to determine the location of the center of rotation of the system. Figure 4.3.1 shows the end displacement of each girder on the girder line highlighted in the plan view of the bridge. All the data for this bridge can be found in Appendix A.



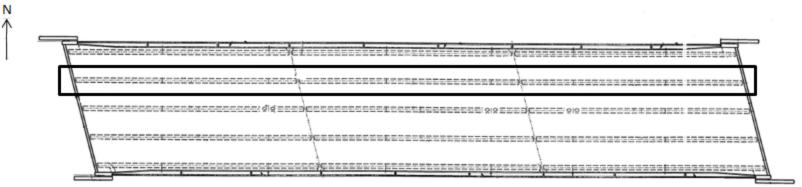


Figure 4.3.1: Typical girder end displacements

The middle four plots indicate that the center of rotation occurs in the deck at the intermediate fix-fix and exp-exp bearing, at an elevation near the center of the link-slab reinforcement and not at the centroid of the girder-deck composite system, which would be about 18 in. below the top of the deck. Figure 4.3.2 shows a typical measurement in relation to the girder and deck elevation.

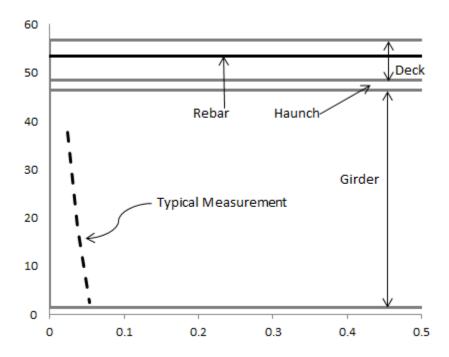


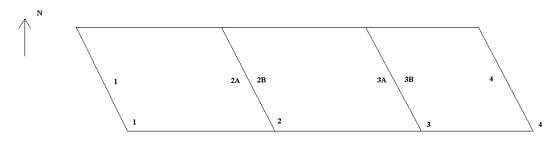
Figure 4.3.2: Typical measurement

4.4 Bottom Flange Dial Gage Results

The other four bridges had dial gauges measuring the displacements only at the bottom of the interior girders. Displacement values were similar for each day which had the same temperature change. Tables 4.4.1-4.4.3 give the displacement between the dawn and afternoon, which involved an increase in temperatures. Each table is paired with a diagram to clarify the sign convention.

Table 4.4.1: BAR2: 015-5123-0 Dial gauge results

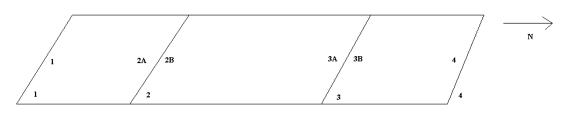
BAR2	Max. Temp. Change:		35°F			
Station	1	2A	2B	3A	3B	4
(in)	-0.062	-0.077	0.003	-0.049	0.072	-0.063



Positive = toward the East, Negative = toward the West

Table 4.4.2: COB1: 067-0213-0 Dial gauge results

COB1	Max. Temp. Change:		32.5°F			
Station	1	2A	2B	3A	3B	4
(in)	0.074	0.015	0.035	-0.015	0.081	-0.032

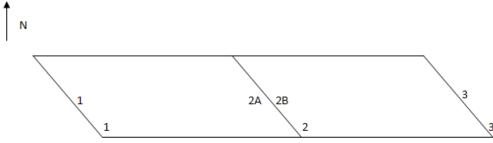


Positive = toward the North, Negative = toward the South

Table 4.4.3: COW1, COW2 Dial gauge results

COW1	Max. Temp. Change:		28.1°F	
Station	1	2A	2B	3
(in)	-0.051	-0.021	0.009	0.0312

COW2	Max. Temp. Change:		28.1°F	
Station	1	2A	2B	3
(in)	-0.047	-0.018	0.012	0.027



Positive = toward the East, Negative = toward the West

All of the dial gages show that there is no significant difference in the displacement at the bottom of the girder between expansion bearings and fixed bearings. For all the bridges, at least one of the fixed bearings at the fixed-fixed bent moved as much as the expansion bearings did. The center of rotation of BAR2 is located at about 2 inches below the top of the deck. The link-slab reinforcement is located about 3.25 inches below the top of the deck. This location is similar to the center of rotation for BAR1 that was described in section 4.3. Both bridges are composed of three equal length spans of AASHTO Type III girders.

4.5 Conclusions

There are two important considerations in the design of continuous deck bridges.

The first is how much overall thermal movement of the bridge occurs that needs to be

accommodated by the expansion joints at the ends of the bridges. The second is what forces are occurring in the link-slab regions that drive the demand for the reinforcement in that region. The wax plate measurements on the bridges BAR1 and BAR2 showed that the total amount of seasonal thermal movement in the bridges studied was less than that calculated using the AASHTO provisions.

Dial gage plus DEMEC measurements indicated that girder end rotations at the construction joints are centered at a point in the deck, potentially near the level of the link-slab reinforcement. This suggests that there are minimal axial forces in the link-slab bars that span this region. In addition, these measurements show that similar bearing movements occur at fixed bearings and at expansion bearings. The impact of these results is discussed further in the following chapters.

Chapter 5

Analytical Studies

5.1 Crack Opening in Link-Slab

The purpose of the construction joint placed at the center of the link-slab in the Georgia continuous deck detail is to force cracks to occur at that location. Field inspections have shown that significant transverse cracks are seldom occurring at any other locations in bridge decks. This is consistent with the cracking described in section 3.3. Due to joint sealants on the top of the deck and stay-in-place metal formwork underneath the deck, it was impossible to determine the existence of cracking at the construction joint locations in the bridges studied. The shrinkage of the deck concrete during the first few hours of curing is estimated in the following section and indicates strains that will induce cracking. It is expected that the cracking will be concentrated at the location of the construction joint in the link slab due to the presence of the edge beams below the deck on each side of the joint. The change in stiffness of the cross section at the link slab is hypothesized to cause a stress concentration which initiates restrained shrinkage cracking.

5.1.1 Early-Age Shrinkage Effects

ACI gives a typical value for the final unrestrained shrinkage strain in concrete structures to be 600×10^{-6} (ACI 224R-01). They give a concrete tensile-strain capacity of 150×10^{-6} , and if concrete is restrained, then cracking will occur. The bottom of the

deck is restrained from length changes by the tops of the girders and the stirrups or shear studs that project from the tops of the girders. If the above strain, which exceeds the tensile capacity of the concrete, is applied to an entire 70 ft. span such as in the Bartow Bridge 1, and if the corresponding movement were concentrated at the link-slab construction joint, the following crack width would occur at the link-slab.

 $crack = (600x10^{-6} - 150x10^{-6}) * 70ft * 12in = 0.378 inches$ Physically, the cracking would be distributed along the length of the bridge girders because of the deck reinforcement and the continuous restraint from the top of the bridge girders. The discrete crack at the link-slab construction joint, and any distributed cracks that occur within the link slab region, would have to be closed by compressive action in order to create compressive forces that would need to be reacted by the deck reinforcement in the link slab. That is, for an internal moment to develop within the deck within the link slab, the concrete on one side of a crack would have to make contact with the concrete on the other side; the compression force in the concrete would be equal and opposite to the tension in the link slab reinforcement. Rotations within the deck could be caused by bridge live loads and/or by thermal movements.

5.2 Effects of Bearing on Link-Slab Forces

Assuming a full depth crack exists through the deck at the construction joint, the concrete is unable to react any forces that could develop in the reinforcement that spans this region because one face of the crack is not in contact with the opposite face. The only way to develop any axial force in the reinforcement is to react that force by closure of the crack (or cracks). Ulku et al. (2009) and Okeil and ElSafty (2005) idealized the

bearing conditions of girders as being either rollers or hinges. The bottom of the girders can displace longitudinally at the bearings modeled as rollers, but they cannot at those modeled as "fixed", hinges in their models. Figure 5.2.1 shows the idealized support configurations used by Ulku et al. to model adjacent spans.

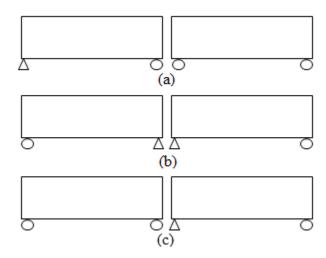


Figure 5.2.1: Support configurations (a) HRRR, (b) RHHR, (c) RRHR

Ulku et al. used finite element analysis and these support configurations to determine link-slab forces due to live load and the application of the AASHTO (2007) temperature gradient. The authors describe the results simply as the moment and force developed in the link-slab without describing how they are distributed in this region. Table 5.2.1 is taken from Ulku et al. (2009) and shows resulting axial forces from different support configurations.

Table 5.2.1: Forces in link slab for different support configurations (Ulku et al., 2009)

Case	Support	Moment ^a (ft-kips)			Axial Force ^a (kips)		
		Ideal Support	Bearing Stiffness			Bearing Stiffness	
			Lower Bound	Upper Bound	Ideal Support	Lower Bound	Upper Bound
Live	HRRR	-51	-50	-50	0	1	2
	RHHR	-19	-19	-19	159	160	159
	RRHR	-51	-50	-49	0	6	8
Positive temperature gradient	HRRR	61	61	61	0	-4	-5
	RHHR	44	44	44	-84	-84	-85
	RRHR	61	60	60	0	-5	-6
Negative temperature gradient	HRRR	-18	-18	-18	0	1	1
	RHHR	-13	-13	-13	25	25	26
	RRHR	-18	-18	-18	0	1	2

[&]quot;Sign convention: positive axial force-tension; negative moment-link-slab top fiber stress tension.

The above data show that significant axial force in the link-slab is only generated for a RHHR support configuration. Okeil and ElSafty (2005) stated that the tension force in the construction joint would be reduced "substantially" if any longitudinal movement was allowed by the bearings. Measurements of bearings that were described in Chapter 4 showed that even fixed bearings, which would be modeled as hinges, were able to exhibit longitudinal displacements up to about 0.08-in. (Table 4.4.1). These displacements are sufficiently large to alleviate the above forces.

5.2.1 Dowel Bar Lateral Restraint at the Bearings

The elastomeric bearings for bridges are designed as fixed or expansion. In both bearing setups in Georgia, a dowel bar is used to connect the girder to the pier cap through the bearing as shown in Figure 5.2.2. The dowel bar prevents the girder from excessive lateral and longitudinal movement with the expansion and contraction of the bridge. The standard diameter of the dowel bar is 1.25 inches, and its length is 1.5 ft.

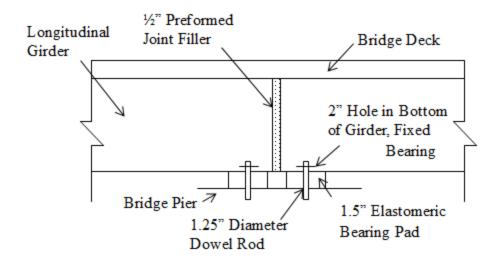


Figure 5.2.2: Dowel rod through pier cap, bearing pad, and girder (not to scale)

At fixed bearings, the dowel bar connecting the girder and pier cap is placed into a circular hole in the bottom of the girder. A photo of this hole is shown in Figure 5.2.3. The hole is 2 inches in diameter to allow for construction tolerances. The larger dimension of the hole compared to the dowel bar, as seen in Figure 5.2.4, allows for some lateral and longitudinal movement of the girder at the fixed bearing location.

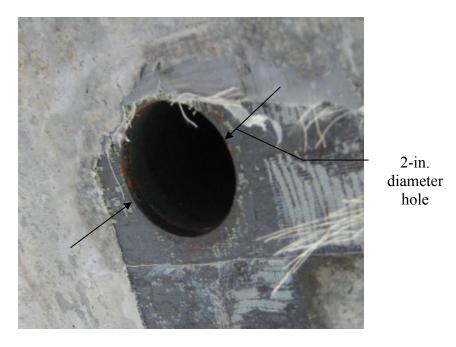


Figure 5.2.3: Hole in the underside of a girder at a fixed bearing



Figure 5.2.4: Dowel bar (1.25-in. diameter) inserted into the 2-in. diameter hole insert at a fixed bearing

The dowel bar in an expansion bearing is placed into a rectangular slot cast into the girder; this slot allows for more longitudinal movement than the fixed bearing. The amount of movement is limited by the length of the slot and the capacity of the

elastomeric bearing. Figure 5.2.5 shows the slot on the underside of a girder for the dowel rod at an expansion bearing. The bearing slot insert is approximately 5.625 inches long and 1.75 inches wide. Figure 5.2.6 is a picture of a dowel rod placed into the slot.

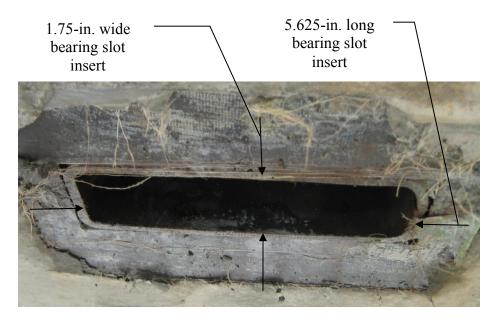


Figure 5.2.5: Bearing slot (1.75 in. x 6.75 in.) insert made of galvanized steel for expansion bearing set-up



Figure 5.2.6: Dowel rod inserted into expansion bearing slot insert in bottom of girder

At a fixed bearing, there is a 3/8-in. annulus between the dowel and hole if the dowel is in the center of the hole. This means that, on average, this amount of movement can occur without ever having to engage the dowel.

The total area of steel provided by the #6 bar reinforcement that extends across the construction joint at the level of the top mat of reinforcement is approximately equal to the total area of steel from the #4 longitudinal reinforcement in both the bottom and top mats. This amount of steel enables the link-slab to potentially carry a high amount of force. However, the amount of force that can be transmitted by the bearing pads and dowel bars is relatively small compared to the tension capacity of the #6 bars. Thus, the bearing pads and the annulus around the dowel bars limit the magnitude of the longitudinal forces transmitted in the system except as discussed below for longitudinal loads due to truck deceleration and seismic forces.

In addition, the amount of force that can be transmitted by a single dowel bar is relatively small compared to the tension capacity of the link-slab #6 bars even if the dowel were restraining the movement by coming into bearing against the side of the hole at a fixed bearing. As the girders move and if the dowel comes into contact with the side of the hole in the girder, the dowel bar is bent. The 1.23 in² of steel is capable of resisting 24.6 kips (at an allowable shear stress of 20 ksi) and can only withstand limited deformation. Therefore, the overall net force in the link slab due to any potential restraint from the fixed bearings is believed to be negligible.

5.2.2 Calculation of Elastomeric Bearing Pad Deformation

In order for a longitudinal displacement of a girder to occur, there must be some deformation of the bearing pad itself. The force required to deform the bearing would have to be reacted by the link-slab reinforcement. Figure 5.2.7 shows the dimensions of the bearing pads that are used in the BAR1 and BAR2 bridges.

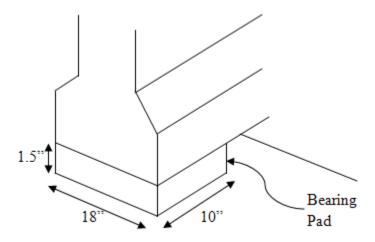


Figure 5.2.7: Bearing pad dimensions

These bearing pads are made of 60 durometer hardness neoprene. Ulku et al. give an upper bound shear modulus for this material as 0.230 ksi (2009). The maximum measured displacement of a bearing beneath a link-slab in the field was 0.077 in.

Assuming a conservative estimate of the shear deformation of 0.1 inch for a bearing, the force transmitted to the link-slab can be calculated. Figure 5.2.8 shows the assumed deformation of the bearing.

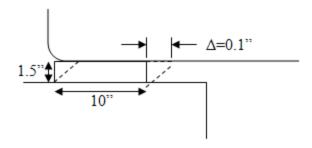


Figure 5.2.8: Deformation of bearing

Using the Bartow Bridge1 as an example, the force generated from the bearing associated with this amount of deformation was calculated. The dimensions of the bearing are those that are shown above in Figure 5.2.7. The calculation is given below.

Shear Strain,
$$\gamma = \frac{\Delta}{1.5"} = \frac{0.1"}{1.5"} = 0.0667$$

Shear Stress, $\tau = G * \gamma = 0.230 \ ksi * 0.06667 = 0.0153 \ ksi$
Force, $F = \tau * 18*10 = 2.76 \ kip$

The #6 reinforcing bars that span the link-slab region have an area of 0.44 in². In a given tributary width for a girder, there are 12 #6 bars in the deck. Assuming that all of the above force is transferred to the link slab, the stress in each bar of link-slab reinforcement is:

$$\sigma_{bar} = \frac{F}{12 * 0.44 in^2} = 0.523 \ ksi$$

This stress is well within the elastic range of the reinforcing steel. If the reinforcement were reduced to #4 bars, the stress in each bar would become 1.15 ksi. The calculation performed is also conservative. The above shear modulus of the pad is an upper bound value. The design shear deformation for the bearing pads is typically half of the pad

thickness which would be 0.75 inches in this case. If the above 24.6 kip estimate of the force from the dowel bar were used rather than the force that could be developed from the bearing pad, σ_{bar} would be 4.7 ksi, still a low stress compared to the yield strength of the reinforcing steel.

Furthermore, if the slab or the bars in the link-slab were reacting large longitudinal forces from the elastomeric bearing pads, maintenance reports and past experience would have revealed effects from this force (this force is a tension force due to simple-span girder rotations associated with downward vertical deflections of the girders, if one idealizes a fixed-fixed bearing as ideally fixed). Yet, no problems along these lines have been identified in the GDOT maintenance reports or with the bridge design office.

5.3 Finite Element Analysis

A finite element analysis was conducted for comparison with findings from the field and from simplified analysis. One-half of one span of a bridge system was modeled using GTStrudl 32. The model was based on the geometry of the BAR1 bridge. A Type III AASHTO girder was modeled with a two inch haunch, an 8.75 ft. wide, 8.25 in. deep deck, and 1 ft. wide edge beam. The girder, haunch, deck, and edge beam were modeled with GTStrudl's IPSL 3D solid elements. The link-slab reinforcement (#6 bars) was discretely modeled within the 3D solid mesh using space truss members. The primary difference between the model and BAR1 is that the model did not incorporate the skew

that exists in the BAR1 bridge, and only one girder and the tributary width of the slab are included in the model. The model is shown in Figure 5.3.1.

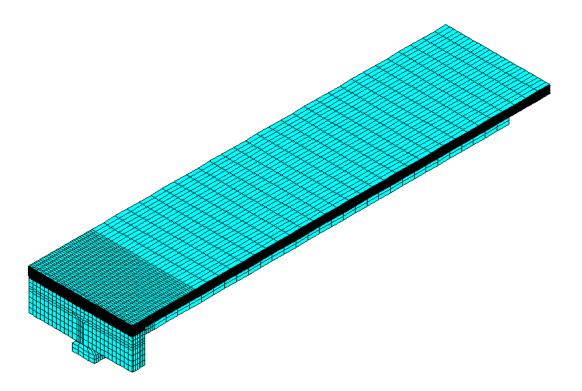


Figure 5.3.1: Isometric view of GTStrudl model

Symmetry boundary conditions were utilized at the midspan. Longitudinal displacements at the midspan were forced to be constant across the section so that no rotation would occur. At the end of the girder, free longitudinal translation was allowed at the bearing. At the bottom of the bearing, vertical displacement is restrained over the bearing area. At the link-slab, longitudinal motion was allowed, and the deck elements were modeled as if a full-depth crack was present. The only boundary condition was that the #6 link-slab rebar elements where constrained in the longitudinal direction at the

centerline of the construction joint. The AASHTO temperature gradient shown in Figure 5.3.2 was applied to the system.

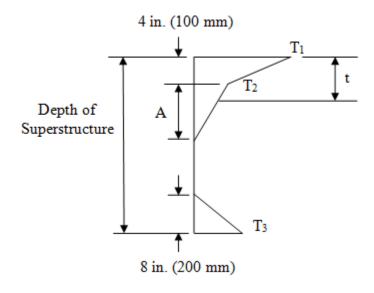


Figure 5.3.2: AASHTO (2007) temperature gradient

The temperature and dimensional values shown in Fig. 5.3.2 come from AASHTO (2007) Section 3.12.3 using Zone 3 temperature data. The temperature T₁ is 41°F, and T₂ is 11°F, but T₃ is assumed to be zero because no site-specific study has been performed to determine its value. A is taken equal to 12 inches because the bridge girders are greater than 16 inches in depth. The thickness of the deck, t, is 8.25 inches. The depth of the superstructure is 55.25 in. Figure 5.3.3 shows how the AASHTO gradient compares to a temperature profile obtained from the field by subtracting morning temperature readings from afternoon temperature readings.

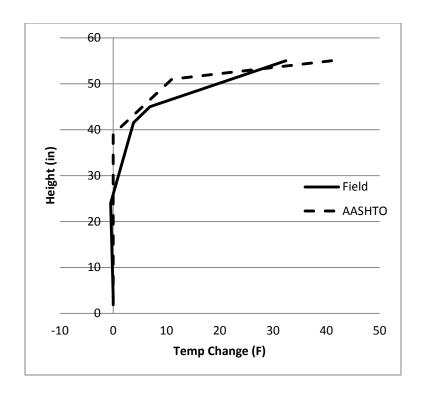


Figure 5.3.3: Comparison of Temperature Gradients

An elastic temperature analysis was conducted and a curvature similar to that observed in the field was observed. Figure 5.3.4 shows the deflected shape.

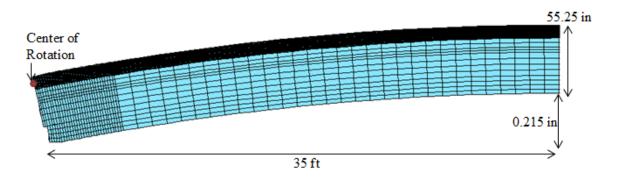


Figure 5.3.4: Deflection due to AASHTO (2007) temperature loading

The maximum longitudinal displacement at the top of the deck at the end of the link-slab was 0.05 inches. This is far less than the calculated crack width from shrinkage strain

discussed in Section 5.1.1. The top surface of the deck is unrestrained across the span due to the saw cut at the construction joints. The effects of the early-age shrinkage in the top surface would result in the calculated crack width at the joint. This means that it is unlikely that the concrete that has cracked from the early-age shrinkage will come back into contact due to the thermal movement. With no contact of the concrete, there will be essentially no moment created in the deck.

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Chapter 6

Conclusions and Design Recommendations

6.1 Conclusions

The current continuous deck design detail and its predecessors have served the State of Georgia well since the late 1980's. The current design detail has limited inservice problems as shown by the GDOT maintenance reports and field observations.

6.1.1 Performance History

The maintenance reports and interviews with several contractors and bridge maintenance office staff revealed little evidence of in-service problems with the current continuous deck detail. Of the 244 bridge maintenance reports reviewed, 2% of the bridges reported cracking at the link-slab construction joints in the bridge deck. Approximately 7% of the bridges from the bridge maintenance reports had experienced expansion joint failure or joint leakage. The bridge maintenance office and several contractors also did not report any in-service problems or concerns regarding the continuous deck detail during the site visits or interviews. From the maintenance reports and interviews, the continuous bridge deck design detail appears to be working well.

However, the contractors indicated difficulty in building the continuous deck detail because of the time and labor it consumes. The header required to form the construction joint is time consuming and labor intensive to build. Laying out the

reinforcement is also difficult, especially in skewed bridges where both the transverse and longitudinal deck reinforcement must be stopped 2 in. from the joint.

The field inspections conducted in this research indicated that most of the problems with continuous bridge decks were the result of poor practices in making the construction joints at the link slab details or improper installation of expansion joints at the abutments. Spalling at construction joints was due to improper forming of the joints. In a number of instances investigated in this research and where damage was observed at the construction joints, the joints were squared off instead of rounded with a ¼-in. radius as specified. The expansion joints with the greatest damage had pulled away from the bridge deck and/or the abutment. These joints created large gaps which allowed for the intrusion of water and other debris. The failures of these joints were likely due to installation at an inappropriate temperature, such that the initial width was not appropriate for the selected joint.

6.1.2 Analysis of the Current GDOT Continuous Deck Detail

Field results showed that girder ends that rest on so called "fixed" bearings, which have round holes for the dowel bars, have the same ability to displace longitudinally due to daily thermal loads as those that sit on so called "expansion" bearings, which have slotted holes for the dowel bars. Estimates of shrinkage strain showed that a crack of sufficient width could easily exist in the link-slab at the top of the deck so that the concrete on one side of the crack would not contact concrete on the other side of the crack. A finite element analysis showed that the deformations of a fully cracked deck due to thermal loadings are not sufficient to close such a shrinkage crack that may exist in

the deck. Therefore, no moment is created in the deck due to girder rotations.

Furthermore, because of the annulus around the dowel bars at the bearings, little to no force is generated in the bearings due to the girder rotations. Thus, little to no force occurs in the link slab reinforcement crossing the construction joint.

The link-slab reinforcing bars provide bridge deck continuity for longitudinal and transverse loads due to vehicle braking, wind and earthquake, and they act as dowel bars transferring shear between one side of the deck crack to the other. However, there is little shear transfer because of the presence of the edge beams supporting the deck.

6.2 Design Recommendations

6.2.1 Length Recommendations

The potential length of a bridge with a continuous bridge deck is limited by the type of expansion joint used to absorb the movement at the bridge abutments. In the State of Georgia, the Evazote expansion joint is preferred because of its low material and installation costs. However, the Evazote joint only has a maximum expansion and contraction range of 2.25 inches.

Field measurements showed that two bridges with total span lengths of 210 ft. each only required 0.625-in. of total movement to be accommodated at each abutment. If this is linearly extrapolated out to a 400 ft. span (the current GDOT limit), the total expansion/contraction required at an abutment would be 1.25 in. This is still well within the range of the Evazote joints mentioned above. Given the current limits adopted on the bridge length prior to installing expansion joints in Georgia, the problems that have been

encountered with Evazote expansion joints appear to be limited more by improper installation than by their capacity for exapansion/contraction. Further investigation is required before any adjustments to the determination of thermal movement should be considered.

6.2.2 Final Recommendations

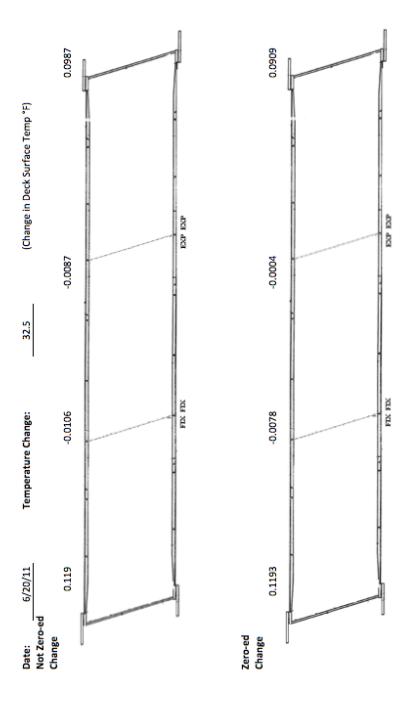
The final conclusions and design recommendations for continuous deck bridges are summarized below.

- 1. If Georgia DOT continues to use the Evazote joint design for the expansion joint at each abutment, then the maximum length of the bridge should be 400 ft. The maximum overall span length of a bridge that can be built with expansion joints only at the abutments is limited only by the deformation capacity of the type of expansion joint being used given proper installation.
- 2. Reduce link-slab reinforcement to #4 bars using the same requirements as those for determining the deck reinforcement as described in Chapter 2 of this document. It further is recommended that the top layer of deck reinforcement be continued across the joint as a replacement for the link-slab reinforcement in order to facility construction. The bottom layer of deck reinforcement should be discontinued as currently done, and the transverse reinforcement should be discontinued at the joint for bridges with a skew greater than 15 degrees.
- 3. Construction joints in the link-slab must be formed properly with the specified rounding of the joint edges. This will reduce the chances of spalling of the deck concrete around the joint and minimize the need for maintenance.

4. Expansion joints must be placed within the specified range of widths so that they can perform as expected. Better consideration of the temperature at the time of installation in setting the required ranges.

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Appendix A Bridge Measurement Data

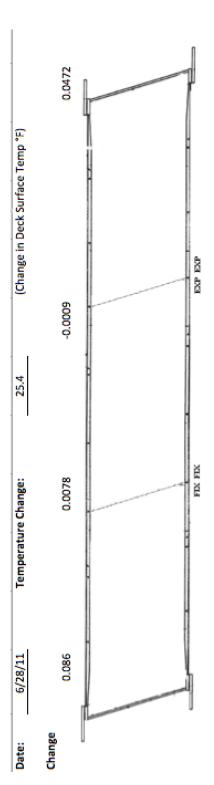


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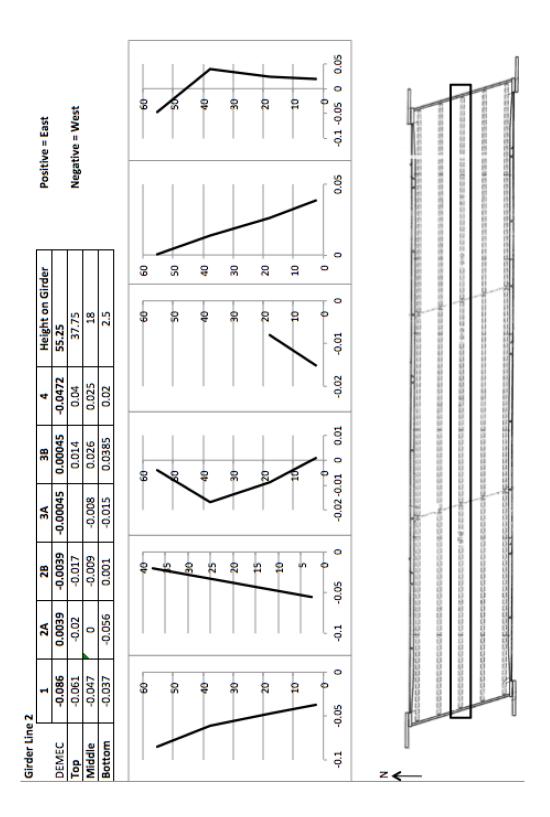
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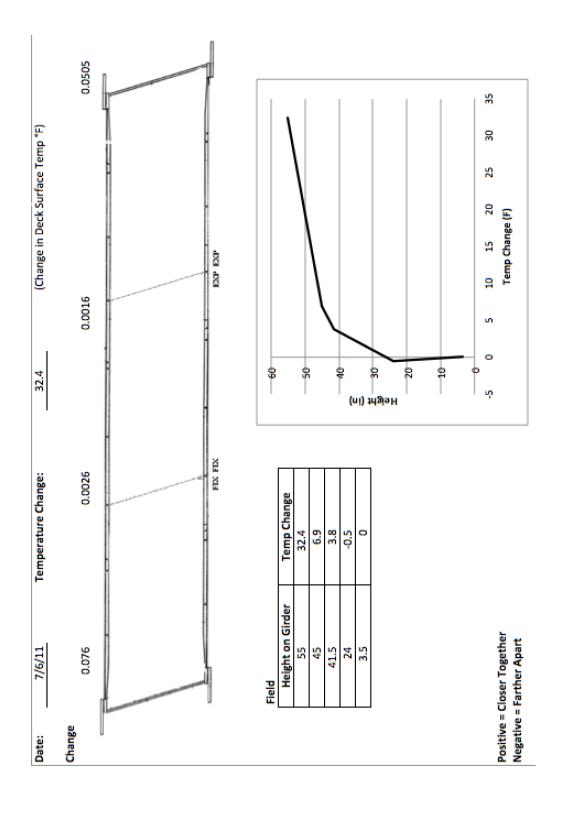


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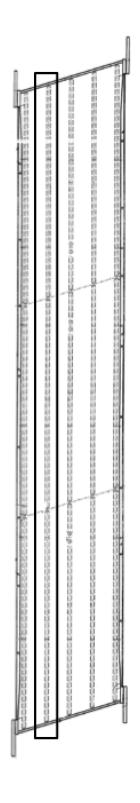
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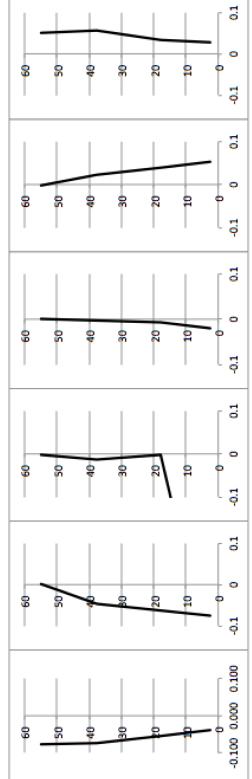
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