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Rehabilitation of Precast Deck Panel Bridges

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by

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A thesis submitted in partial fulfillment
of the requirements for the degree of
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Permanent Repairs, Full Depth Precast Panels

DEDICATION

For my lovely daughter Lina.

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All praise is due to GOD, most gracious most merciful, for giving me the ability, determination and drive to complete my graduate studies at this stage of my career. I would like to first thank my parents and sister, Mustafa, Amina, and Dr. Hayat Alvi, for their encouragement; and my wife Misbah for her loving support and understanding.

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ABSTRACT

USF completed a research study in 2005, which prioritized the replacement of 85 deteriorating composite precast deck panel bridges. This thesis re-evaluates the original recommendations in the wake of failures of two of these bridges in 2007. Since funding will not allow all identified bridges to be replaced, it was necessary to determine the most effective repair methods. To assess USF's recommendations, a forensic study was undertaken in which the most current inspection and work program documents on the two failed bridges were reviewed and FDOT personnel interviewed. The best repair procedures were determined by reviewing repair plans, specifications, reports and site visits. The study found the two bridges that failed had been correctly prioritized by USF (No. 1 of 18 and No. 8 of 15). A new, accelerated repair method encompassing complete bay replacement was developed in a pilot project funded by the Florida Department of Transportation.

1. INTRODUCTION

1.1 Introduction

The use of composite precast deck panel slabs on bridges was initially implemented in the construction of highway bridges in Illinois in the early 1950's. This innovation was never part of the bridge design process but rather the result of value engineering during the construction phase. The precast deck panel was used as a stay-in-place form and a cast-in-place (CIP) component was placed on top and in between the panels as shown in Figure 1.1, which considerably reduced construction time. By implementing this method, field forming was only needed for the exterior girder overhangs.

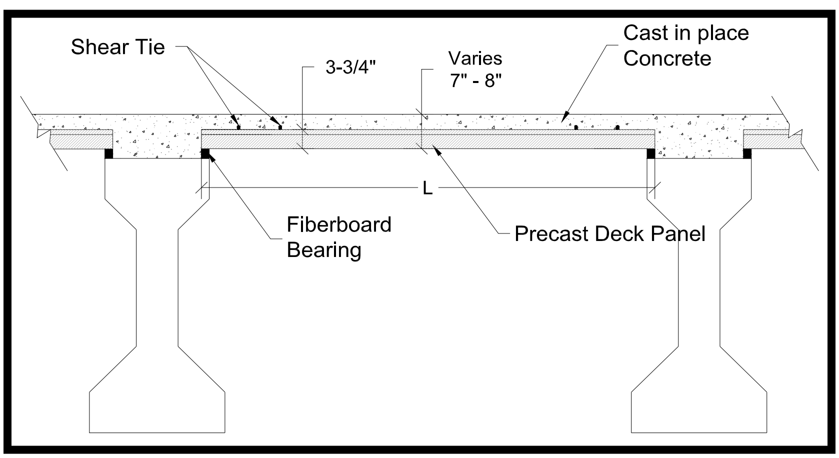


Figure 1.1 Composite Deck- Precast Deck Panel and CIP Concrete

In the early to mid 2000s, Florida had approximately 200 precast deck panel bridges, with the majority of them, 127 being located in Districts 1 and 7; this is including 18 on the Leroy Selmon Crosstown Expressway (Crosstown Expressway) of the Florida Department of Transportation (FDOT) (see Figure 1.2) [1]. Precast panel sizes vary with girder spacing but are typically 10 ft. x 10 ft. in plan and 3½ in. to 4 in. thick. In design, it is assumed that the panel acts compositely with the CIP reinforced concrete slab for resisting live loads as shown in Figures 1.3 and 1.4.

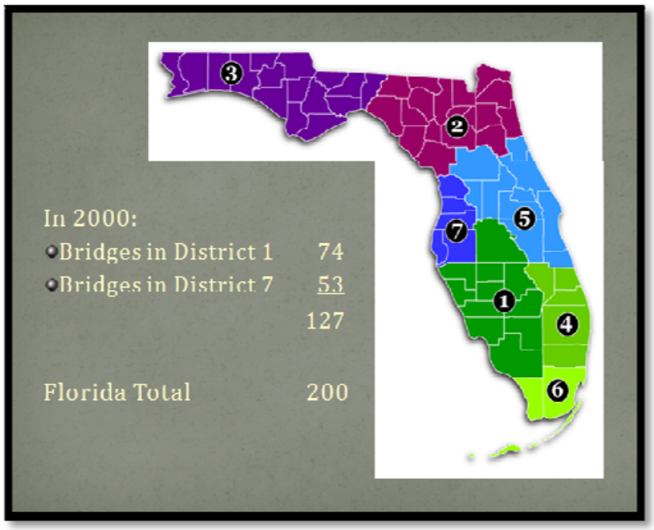


Figure 1.2 Precast Deck Panel Inventory in 2000

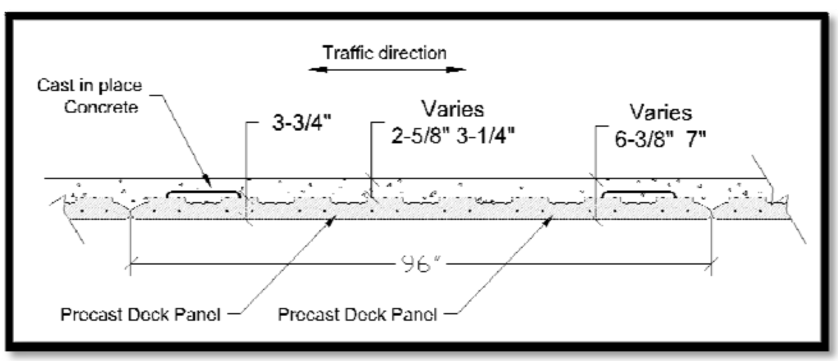


Figure 1.3 Precast Deck Panel with CIP Component

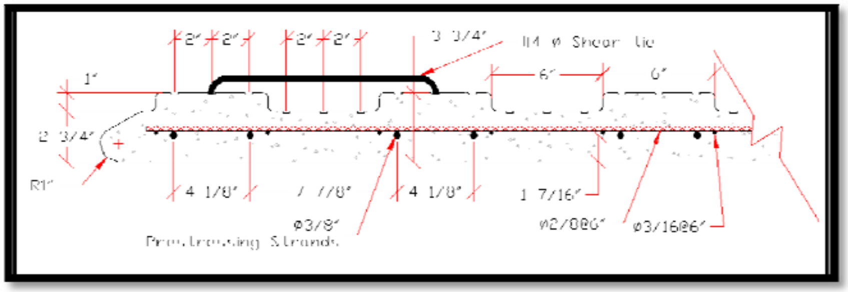


Figure 1.4 Precast Deck Panel Reinforcement Details

Despite successful performance in other states and satisfactory performance in other FDOT districts, precast deck panel bridges have a long history of premature deterioration in Districts 1 and 7 that has resulted in excessive maintenance for the FDOT and impacts to the traveling public. Previous research has attributed this to contractors using flexible fiberboard bearing material supports to simplify construction [2].

The FDOT Districts 1 and 7 Structures Maintenance Office (DSMO) has responded to numerous maintenance problems on precast deck panel bridges throughout the I-75 corridor in southwest Florida. Initially, the response was reactionary, geared towards emergency situations in which a localized failure of the bridge deck resulted in lane closures. Between 2000 and 2003, five failures occurred. Over time, the DSMO began a proactive approach to monitoring, early detection and repair to avoid disruptive emergency situations.

The DSMO had established a method to systematically replace selected precast deck panel bridges on I-75 in both Districts 1 and 7 with full depth, CIP concrete decks. The short-term goal was to replace the decks on bridges with high average daily traffic (ADT) and the long-term plans were to replace the decks on all precast deck panel bridges in both districts. The FDOT had allocated $78 million in 2001 for a period of 10 years to replace deck panel bridges on I-75 running through Districts 1 and 7, and $65 million in 2003 for the Crosstown Expressway Viaduct Bridges.

In 2005 The University of South Florida (USF) completed a comprehensive study for FDOT. The objective of the study was to prevent any further failures by identifying and prioritizing deck replacement of high risk bridges in Districts 1 and 7.

However, since finalization of the study, two subsequent sudden bridge deck failures have taken place. Both failures occurred in 2007 within District 7: the first one on the Crosstown Expressway and the other on I-75.

1.2 Objectives

The purpose of this research is to (1) reassess the prioritizations assigned to the two bridges that experienced deck failures subsequent to the finalization of the 2005 USF Study, (2) to provide an update of current status of composite precast deck panel bridges in FDOT's Districts 1 and 7, and (3) assess the effectiveness of repair methods used on this type of deck system.

This was accomplished by participating in the emergency response teams for both subsequent failures, gathering information, such as bridge inspection reports, monthly (deck panel) inspection reports, special engineering reports, plans, funding reports, 5-Year Work Program report and construction status reports from the FDOT as well as meeting with key FDOT, consultant and contractor personnel.

1.3 Thesis Organization

This report is organized into seven chapters and two appendices that describe various components of the research. Chapter 2 presents a literature review on other studies that have been published on composite precast deck panel bridges. Chapter 3 provides details of the deck failures that occurred in 2007. Chapter 4 assesses the recent failures in comparison to USF's rankings. Chapter 5 gives an update on the current status of composite precast deck panel bridges Districts 1 and 7. Chapter 6 assesses the effectiveness of the repair procedures used on composite deck panel bridges and the summary and conclusions are presented in Chapter 7.

2. BACKGROUND

2.1 Prior Research

Research related to the deficiencies of composite precast deck panel bridges has been undertaken since as early as 1982. A brief description of the research in chronological order is as follows:

2.2 University of Florida Study (1982)

In 1982, The University of Florida Study by Callis, et al. [2] was performed as a result of the excessive deck cracking on the Peace River Bridge. It concluded that:

- The decks in their present cracked condition are structurally adequate to carry normal traffic. In spite of the simple action of the decks, flexural stresses are not excessive.
- The shear stresses in the Peace River Bridge are substantially higher than that of conventional bridge decks or panel bridges with positive bearing at the ends of panels. The increase in shear stress is brought about by the combination of the lack of bearing of or the end of the panels and the loss of bond on the end of the panels which is primarily due to creep of the panels under the action of the prestress.
- The observed cracking on the top of the deck is probably primarily due to the volume changes brought about by the differential shrinkage between panels

and CIP component. However, temperature changes and live load stresses certainly increase the tensile stresses and the degree of cracking.

- Adding extra transverse or longitudinal steel is not felt to be sufficient to ensure adequate fatigue life of panel bridges.
- Removing the fiberboard and replacing it with a material providing positive bearing (mortar) would greatly increase the fatigue life expectancy of the Peace River Bridge. Whether this action is economically justifiable depends on further studies of the shear fatigue behavior of the bridge.
- Future panel construction projects should include a detail that provides positive bearing for panels. Strand extensions may also be useful.

2.3 University of Florida Study (1983)

In 1983, The University of Florida study by Fagundo, et al. [3] was a follow up to the Peace River Bridge Study, with the objectives to evaluate the potential for shear fatigue failure of existing panel bridges constructed using details that did not provide positive bearing under the ends of the panels and to compare the performance of composite precast deck panel bridges constructed using several support details against the performance of conventional reinforced concrete decks. The report concluded that:

- Composite decks without positive bearing act as simply supported beams with maximum positive moment in the center and negative moment at the ends.
- Replacing fiberboard with grout would reduce shear stresses at ends.
- Panel decks with positive bearing should have a service life comparable to conventional decks.

2.4 University of Florida Study (1984)

In 1984, the third report on this subject from The University of Florida was authored by Fagundo, et al. [4] with the objective of developing an immediate management plan that would allow FDOT to decide upon a reasonable, not necessarily optimum, program for grouting and repairing the bridge decks. The report concluded that:

- Bridges in which reasonable bond is maintained between the ends of the panels and the CIP component concrete should not exhibit any significant longitudinal spalling. Thus many of the panel bridges should not exhibit any significant longitudinal spalling for a very long service life.
- It is known that the major factors that cause the loss of bond on the end of panels are: poor end treatment of the panel such as sawing, lack of strand extensions, creep of the panels after the deck is poured, shrinkage of the deck, and live load stresses after the deck is placed. However, it is impossible to predict for any given bridge the probability of loss of bond and the associated spalling.
- Bridges that exhibit longitudinal spalling can be repaired by the M1 procedure which includes grouting under the panel. Once the repaired these bridges should have normal service lives.
- Panel bridges with reduced longitudinal steel, particularly those with longer panel pans (girder spacing) have the potential for transverse spalling.
- Transverse spalls that occur can be repaired by the M2 procedure as modified in Chapter 4, and should restore the deck to the extent that it would give a long service life.

Furthermore, the study recommended that:

- No large scale grouting or repair of panel bridges that have not exhibited any significant spalling is recommended. This is based on the expectation that the majority of the panel bridges will not exhibit significant spalling during their service life.
- If any M1 or M2 repairs are made in a bridge span, that span should be thoroughly surveyed for delamination by the chain procedure and all areas that are suspected of being damaged should be repaired. Also, bearing should be restored to all panels within that span.
- If a damaged bridge contains multiple spans, than all spans should be thoroughly investigated for delaminating. If significant delaminations appear then serious consideration should be given to repairing all damaged areas on the bridge. Further, any span that is repaired by either M1 or M2 procedures should be grouted under all panels. This recommendation is based on the fact that construction costs and inconvenience to the traveling public would be greater if a bridge is repaired one span at a time rather than all spans simultaneously.
- Data on bridges that are repaired should be carefully kept both with regards to physical variables and costs. Thus, after a few years an empirical prediction can be made of future costs.
- Field testing as part of research already planned should be directed towards verification of the repair techniques under field conditions. Attention should be given to testing spans with varying amount of longitudinal reinforcement.

- The management plan for the panel bridges outlined in steps 1 through 5 should be reasonably cost effective based on the present state of knowledge. To develop a truly optimum management plan a comprehensive research program extending over several years is required.
- It suggested that the M1 and M2 repair methods be incorporated with the modification described in section 4.2. These modifications should improve the ductility, strength, and durability of the repaired area.

2.5 University of South Florida Study (2005)

In 2005, The University of South Florida performed a study by Sen, et al. [5] with the objective of examining the deterioration process that leads to sudden failures in composite precast deck panel bridges and in turn to develop a strategy to assist the FDOT in the prioritization for replacement of these bridges in Districts 1 and 7 to avoid such failures. This study is reviewed more thoroughly because one objective of the author's research is to reassess the recommendations of the USF Study.

The USF study analyzed five localized failures that occurred in composite precast deck panel bridges in Districts 1 and 7 between 2000 and 2003. Table 2.1 summarizes relevant information relating to these failures. As indicated in the table, all failures had some type of repair while some had a combination of repairs. All the failures were narrowed down to only two cities within the two districts, Sarasota and Tampa. A survey was also conducted to determine the performance of deck panel bridges in other districts. No failures had occurred in Districts 2, 3, 5 and 6. District 4 reported failures in two bridges: Bridge No. 940126, carrying I-95 (Southbound) over the Florida Turnpike and

Bridge No. 940127, I-95 (Northbound) over the Florida Turnpike but no details were provided.

All failures occurred in bridges where the deck was nominally 7 inches thick. No failures occurred in deck panel bridges with thicker slabs. The percentage of trucks in the ADT varied between 8 and 30%.

Table 2.1 Localized Deck Failures 2000 to 2003

Bridge No.	District	Year Built	Age at Failure (Years)	NBI Rating Before Failure	Days Since Last Insp.	Rain 7 Days Prior to Fail (in.)	ADT	Failure Size (In.)	Loc. in Panel	Comment
	Bridge Location	Failure Date					%Truck			
170146	1	1981	19	6	90 days	0	34,000	18 x 24	Edge or Corner?	Failure at asphalt patch within full depth spall repair
	Sarasota, I-75 NB Over Bee Ridge Rd	2/12/2000		(Sat)			10%			
170086	1	1980	20	7	180 days	2	34,000	36 x 60	Corner Support	Localized full depth CIP repair
	Sarasota, I-75 NB Over Clark Rd	11/27/2000		(Good)		0.68	9%			
170085	1	1980	20	7	210 days	4	34,000	18 x 18	Corner	Asphalt patch adjacent to M1 repair
	Sarasota, I-75 SB Over Clark Rd	12/20/2000		(Good)		0.2	10%			
100332	7	1980	22	5	2 days	2	23,000	48 x 30	Near corner	Asphalt Patch
	Tampa, Cross-town Viaduct WB Span 38	10/2/2002		(Fair)		0.55	8%			
100332	7	1980	23	5	23 days	3	23,000	24 x 36	Edge	Failed M1 repair with flexible patch material
	Tampa, Cross-town Viaduct WB Span 70	9/5/2003		(Fair)		1.1	8%			

All failures occurred under the wheel loads applied close to the face of the girders where initial longitudinal cracks developed. Also in all five cases, the failure occurred in the right lane, (i.e. slow lane), where large and heavier loads (i.e. eighteen wheeler trucks and permit vehicles) generally travel. Failure was normally in the edge or corner panels whose boundaries developed reflective longitudinal and transverse cracking.

A deterioration model based on the field observations and analysis of localized failures was developed in the study (see Figure 2.1). However, as the structural behavior of composite precast deck panel bridges depends on several factors, not all of which can be quantified, it makes it almost impossible to accurately predict future service life using numerical analysis. On the other hand, inspection data that tracks progression of cracking can be more successful in predicting localized failure.

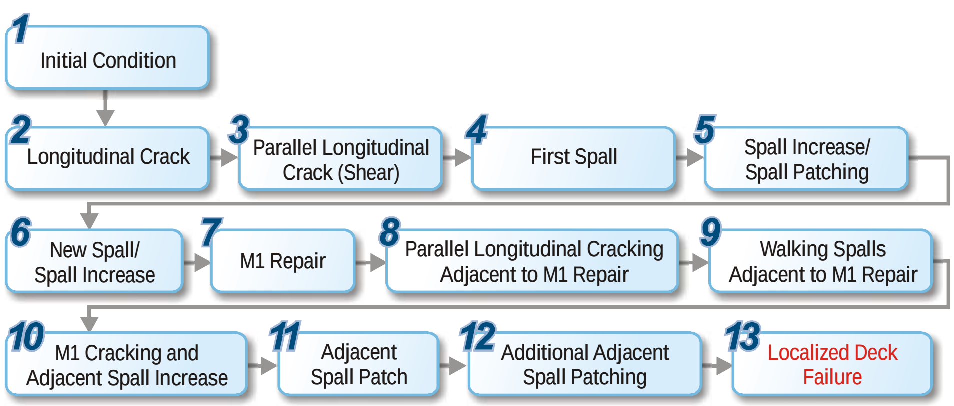


Figure 2.1 Deterioration Model

The simplified model indicates that longitudinal cracks first develop along the girder lines. This is followed by occasional reflective transverse cracking. Such defects appear within 5 years of construction. These cracks may not change for nearly 10 years after which there is more widespread transverse cracking. Longitudinal and transverse cracking result in spalling and delamination that require repair. This is an important stage in the deterioration process because the type and quality of repair will dictate if the long-term performance of the bridge is satisfactory or poor. In most cases, such damage occurs in regions where the panel is improperly supported on fiberboard bearing material. Where deck repairs are combined with proper panel bearing (e.g., by placing non-shrink grout or injecting epoxy), repairs are satisfactory. Where this is not carried out, and repairs are limited to surface repairs, there is progressive degradation as shown in Figure 2.1, which can lead to failure. In several instances, failures occurred at locations where temporary repairs had not been replaced.

Simplified calculations performed in the USF study proved that punching failures could result at loads below the design wheel load. This assumed the CIP deck to provide no resistance and the panel to be supported on fiberboard bearing material with well developed cracking along the transverse and longitudinal panel boundaries. The failure load was calculated to be around 15 kips. Otherwise, failure loads were nearly four times higher.

Equipped with this information, the USF team created replacement prioritization for bridges in District 1, District 7 and the Crosstown Expressway, respectively (See Tables A.1-A.5).

2.6 Summary and Conclusions

Various studies have been conducted on the problems with composite precast deck panel bridges. One such study was completed by USF in 2005. It investigated the five failures occurring in Districts 1 and 7 between 2000 and 2003, with the goal of prioritizing high risk bridges for replacement and consequently to eliminate sudden failures in the future.

3. DECK FAILURES IN 2007

3.1 Introduction

The main objective of the USF Study was to prioritize high risk bridges for replacement and in turn eliminate further sudden failures. However, since finalization of the study in 2005, two subsequent failures have take place. Both failures occurred in 2007 and within District 7. Details of the subsequent failures are provided in Table 3.1.

Table 3.1 Localized Deck Failures Following USF Study

Bridge No.	District	Year Built	Age at Failure (Years)	NBI Rating Before Failure	Days Since Last Insp.	Rain 7 Days Prior to Fail (in.)	ADT	Failure Size (Inches)	Loc. in Panel	Comment
	Bridge Location	Failure Date					%Trucks			
100332	7	1980	27	5	565	0.21	23,000	18 x 8	Edge	Failed localized patch repair
	Tampa, Cross-town Viaduct WB Span 39	3/5/2007		(Fair)			8%			
100436	7	1983	24	5	685	0.54	46,250	12 x 24	Edge	Failed localized patch repair
	I-75 over E. Broadway Ave., CR 574 and CSX Railroad	9/11/2007		(Fair)			8%			

3.2 Bridge No. 100332

The first failure occurred on March 5, 2007, on Bridge No. 100332, The Crosstown Expressway (Westbound) Viaduct, Span 39, Lane 2 (see Figures 3.1 and 3.2), The failure size was approximately 18 in. x 8 in. hole through the deck and it occurred within the outside edges of a nearly 5 ft. x 7 ft. existing repair patch. It was located on the edge of the panel and the edge of the beam.

Figure 3.1 Bridge No. 100332, Span 39 Failure (Deck Top)

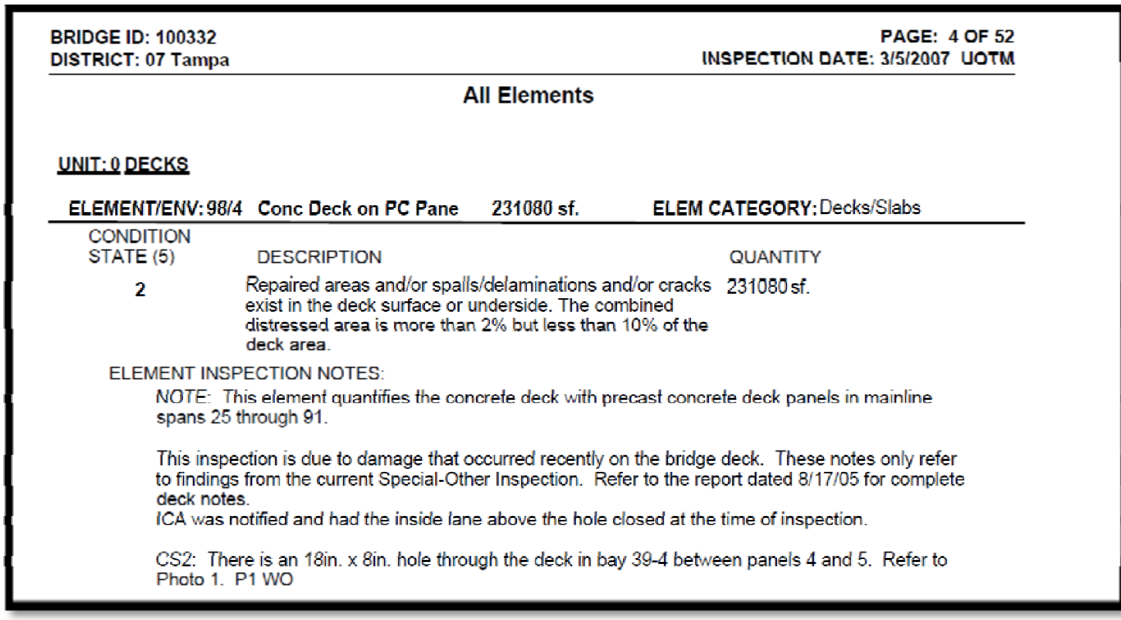
Figure 3.2 Bridge No. 100332, Span 39 Failure (Deck Underside)

Bridge No. 100332 is a 91 span (67 spans with precast deck panels) structure carrying the Crosstown Expressway westbound with two 12 ft. lanes and a 8 ft. wide right shoulder and 4 ft. wide left shoulder. The average daily traffic (ADT) in 2007 was 23,000, with 8% truck traffic. The superstructure consists of AASHTO Type III prestressed concrete beams. The failure happened on Span 39, which is approximately 55 ft. – 1½ in. long, and the beams are spaced at 8 ft. -1 ¼ in. supporting a typical 7 in. thick composite deck.

The last biennial inspection report prior to the failure was finalized August 17, 2005, 565 days before failure, and assigned the deck an NBI Rating of 5 (fair) [6]. The report makes general statements applying to all precast deck panel spans stating that the deck top has light to moderate wear and is typically populated with minor cracks. Also that the deck top and deck undersides have minor multi-directional cracking in isolated locations and some of the deck top cracks have minor associated spalls. It noted that on some of the deck underside many cracks have light efflorescence. The report also indicated that there are minor delaminations in isolated locations up to 1.5 ft. x 3 in. along the construction joints.

More specific details on deficiencies were acquired from the monthly deck panel inspection reports [7] indicating that longitudinal cracks were sealed in October 2006. Some new spalls developed at the edge of an existing repair in the south wheel path of Lane 2 in January 2007. The spalls increased to high priority in the February report indicating there is a 2 ft. x 2 ft. x 2 in. spall with delamination on the topside of the deck and another of the same size on the deck underside. These deficiencies progressed into a punch through failure occurring on March 5, 2007, consisting of an 18 in. x 8 in. hole, which had to be addressed with an emergency repair applying full depth bay replacement across two panels (see Figure 3.3) [8].

The failure occurred after a heavy rainfall event [9]. The precipitation according to the National Oceanic and Atmospheric Administration (NOAA) archives was 0.21 in. seven days prior to the failure.



BRIDGE ID: 100332
DISTRICT: 07 Tampa

PAGE: 4 OF 52
INSPECTION DATE: 3/5/2007 UOTM

All Elements

UNIT: 0 DECKS

ELEMENT/ENV: 98/4 Conc Deck on PC Pane 231080 sf. ELEM CATEGORY: Decks/Slabs

CONDITION STATE (5)	DESCRIPTION	QUANTITY
2	Repaired areas and/or spalls/delaminations and/or cracks exist in the deck surface or underside. The combined distressed area is more than 2% but less than 10% of the deck area.	231080 sf.

ELEMENT INSPECTION NOTES:

NOTE: This element quantifies the concrete deck with precast concrete deck panels in mainline spans 25 through 91.

This inspection is due to damage that occurred recently on the bridge deck. These notes only refer to findings from the current Special-Other Inspection. Refer to the report dated 8/17/05 for complete deck notes.
ICA was notified and had the inside lane above the hole closed at the time of inspection.

CS2: There is an 18in. x 8in. hole through the deck in bay 39-4 between panels 4 and 5. Refer to Photo 1. P1 WO

Figure 3.3 Excerpts from Emergency Inspection Report

3.3 Bridge No. 100436

The next failure took place on September 11, 2007, on Br. 100436, I-75 (Northbound) over E. Broadway Ave., CR 574 and CSX Railroad, in Span 4, Lane 3 (See Figures 3.4 and 3.5). The failure was approximately 12 in. x 24 in. and was located inside an existing repair patch, on the edge of the panel as indicated in the emergency inspection report in Figure 3.6 [10]. Like its predecessor, the failure was also located on the edge of the panel and the edge of the beam.

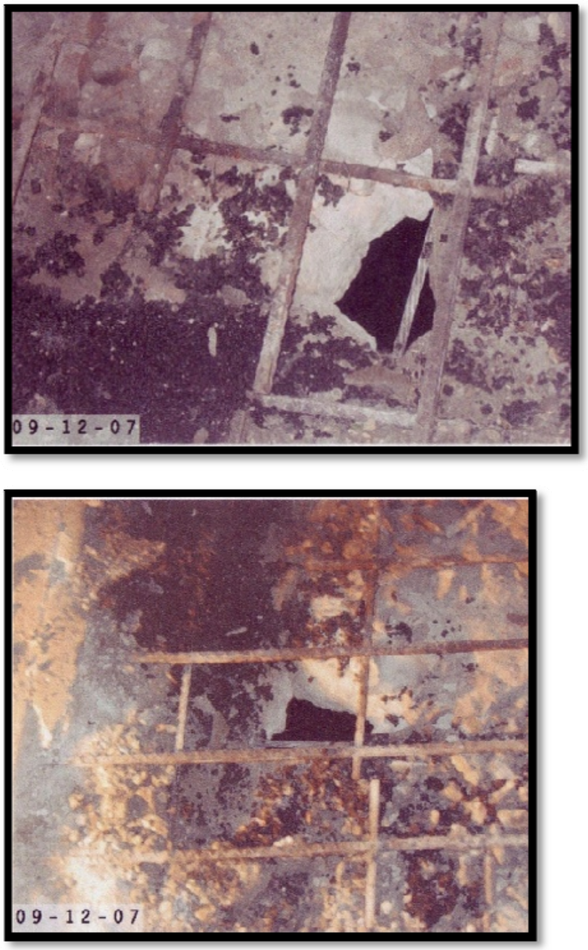


Figure 3.4 Bridge No. 100436, Span 4 Failure (Deck Top)

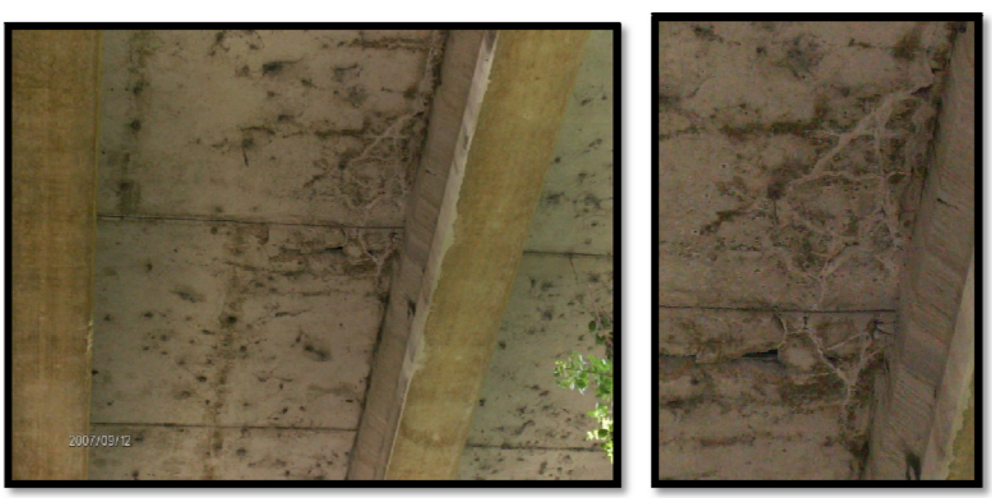


Figure 3.5 Bridge No. 100436, Span 4 Failure (Deck Underside)

BRIDGE ID: 100436
DISTRICT: 07 Tampa

PAGE: 3 OF 11
INSPECTION DATE: 9/11/2007 YGZJ

All Elements

UNIT: 0 DECKS

ELEMENT/ENV: 98/3 Conc Deck on PC Pane 21517 sf. ELEM CATEGORY: Decks/Slabs

CONDITION STATE (5)	DESCRIPTION	QUANTITY
2	Repaired areas and/or spalls/delaminations and/or cracks exist in the deck surface or underside. The combined distressed area is more than 2% but less than 10% of the deck area	21517 sf.

ELEMENT INSPECTION NOTES:

NOTE: This inspection is due to damage that occurred recently on the bridge deck. These notes only refer to findings from the current Special Inspection.

CS2: There is damage to the concrete deck from traffic impact, in the right side of the 3rd lane wheel path. The hole is in bay 4-5, about 26ft south of bent 5 at beam 6. Total area damaged is about 2sf.

The underside of the deck has two damaged panels. Refer to Photo 1. P1 WO The northern damaged panel has a visible hole and exposed strands at its south end. The other panel has visible cracking and staining along the east edge at the PS beam. The damage occurs at the eastern edge of the panels at beam 6. Refer to Photo 2. P1 WO

CORRECTIVE ACTION TAKEN:
Emergency repairs consisted of bolting a steel plate to the deck surface. Refer to Photo 3. Inspector did not arrive on-site until after the plate was installed

Figure 3.6 Excerpt from Emergency Inspection Report

Bridge No. 100436 is a five span structure carrying I-75 Northbound with three 12’ Lanes and 10’ shoulders on both, right and left sides, The average daily traffic (ADT) in 2007 was 46,250, with 8% truck traffic. The superstructure consists of AASHTO Type II, III and IV beams. The failure happened on Span 4, which is approximately 76 ft. in length and is comprised of Type III beams, spaced at 8 ft.-10 in., supporting a typical 7 in. thick composite deck.

The last biennial inspection report prior to the failure was finalized October 27, 2005, 685 days before failure, which assigned the deck an NBI Rating of 5 (fair) [11]. The report documented that there are numerous patches made with “epoxy type” material in the deck top, longitudinal cracking with maximum widths of 1/16 in. over the edge of beam lines and transverse cracks up to 12 ft. long x 1/8 in. wide over the precast deck panel joints in all spans. It stated that most transverse cracks are spaced 8 ft. apart, which is the approximate length of a deck panel on this bridge. Due to the problem with excessive cracking throughout the bridge, a deck cracking diagram was prepared for the inspection files as shown in Figure 3.7 [12].

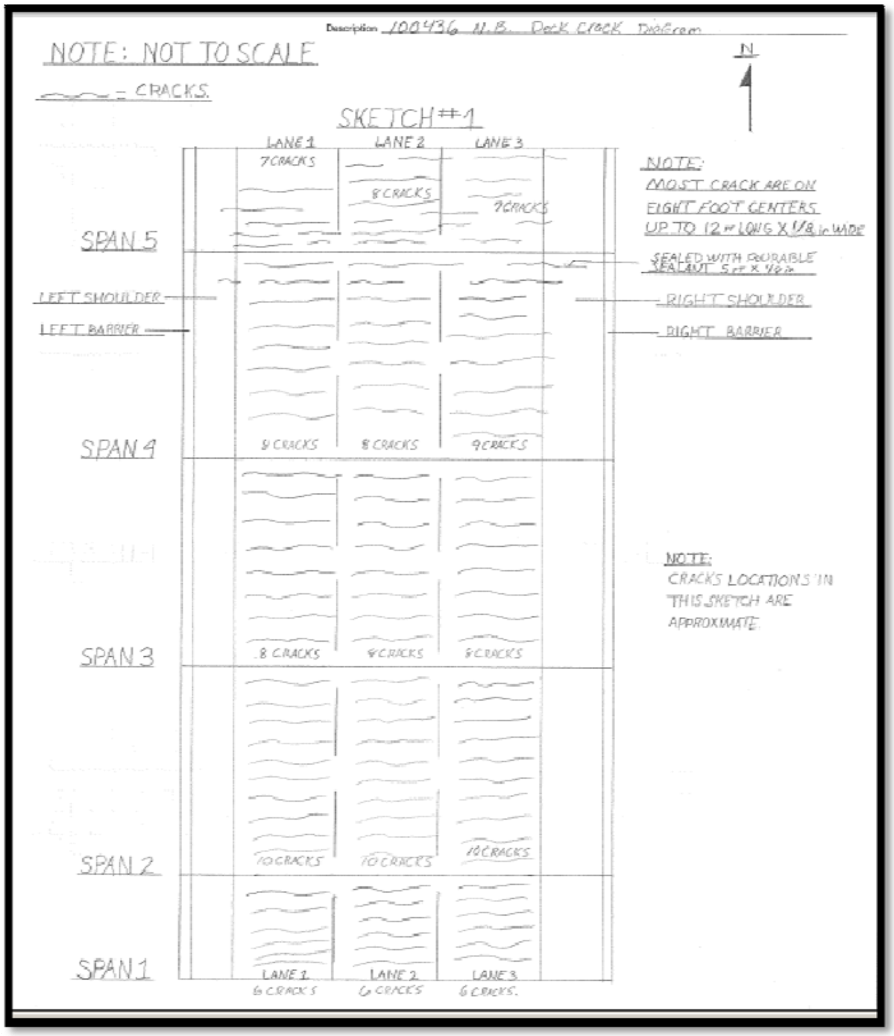


Figure 3.7 Bridge No. 100436, Transverse Cracking Pattern

The monthly inspection reports indicate that localized repairs were made in Span 4 in April, July and September of 2005 and in February of 2007. The reports also indicate that there is a problem with transverse cracking throughout the bridge. Most of the cracking is 1/8 in. wide but three cracks in particular were identified with an approximate width of ¼ in. [13]. The reports indicate that the three ¼ in. cracks have been present since August of 2006 as shown in Figure 3.8.

Figure 3.8 Bridge No. 100436, Span 4 Failure (3/4 in. Transverse Crack)

The precipitation was 0.54 in. seven days prior to the failure according to the NOAA archives.

3.4 Summary and Conclusions

The USF Study's goal was to prioritize high risk bridges for replacement and in order to eliminate any additional sudden failures in the future. However, since finalization of the study in 2005, two bridges experienced sudden failures. The failures took place on Bridge No. 100332, Crosstown Expressway Viaduct, Span 39 and Bridge No. 100436, I-75 over E. Broadway Ave., CR 574 and CSX Railroad. Both failures occurred in 2007 and were located in District 7. Both failures were inside the limits of existing repairs, were located on the edge of the precast panel, and edge of beam and occurred after rain events.

4. ASSESSMENT OF USF PRIORITIZATION RELATIVE TO NEW FAILURES

4.1 Bridge No. 100332

The USF study had ranked this bridge as No. 1 for replacement on the Crosstown prioritization listing as shown in Table A.5. First priority replacement ranking was given to this bridge because it ranked the highest in the categories of Failing Repair Count, Weighted Index, FDOT Ranking, ADT, Importance, Normalized, Risk and Safety as quantified below:

- Year Built: 1975
- Spall Count: 344.7
- Failing Repair Count: 44.9
- Weighted Index: 1018.7
- FDOT Rank: 1
- ADT: 23,000
- Importance Rank: 1
- Normalized Risk: 1.000
- Risk Rank: 1
- Safety Rank: 1

However, despite being ranked with highest replacement priority on the Crosstown Expressway, the FDOT was unable to acquire all the funding required to replace the bridge decks in time to avoid the failure. It is because this bridge was programmed for deck replacement along with its twin structure, Bridge No. 100333, Crosstown Expressway (Eastbound), which makes a total of 134 precast deck panel spans to be replaced. The large number of spans to be replaced required considerably more time to obtain funding than the typical three span bridges on I-75. Replacement funding in the amount of $65 million had been programmed in 2003 for this project [14]. This amount was approved for use in 2009 and the deck replacement project was advertised and four design build firms were shortlisted on March 12, 2010. However, it is currently on hold because The Tampa-Hillsborough Expressway Authority (THEA) requested that the bridges also be widened along with deck replacement [15]. Although widening was given as an option in FDOT's request for proposal, the additional $65 million required for this work was not initially programmed. Consequently the project is at this time on hold and THEA is trying to obtain "stimulus funding" from the Federal Government to include the additional work. The FDOT is expecting to select a firm from the four shortlisted parties in early 2011 and begin work by midsummer.

Taking all of this information into account, USF's replacement prioritization for Bridge No. 100332 was very accurate. It was justified in being ranked No. 1. Had the FDOT been able to acquire the replacement funding for this bridge earlier, the failure might have been avoided.

4.2 Bridge No. 100436

The USF Study had ranked this bridge as No. 8 for replacement. The following factors were taken into consideration for ranking:

- Year Built: 1983
- Spall Count: 5.5
- Failing Repair Count: 1
- Weighted Index: 17
- USF Inspection Condition: Acceptable
- FDOT Rank: 7
- ADT: 44,500
- Normalized Risk: 0.142
- Safety Rank: 9
- Importance Rank: 6
- Risk Rank: 8

The main reason why this bridge was not prioritized higher for replacement at the time of the study was because it only had one failing repair. The research team concluded that the very low count of failing repairs was a good indication that the bridge would not fail [16]. However, immediately after the study was finalized four repairs failed consecutively leading up to the date of deck failure as noted in the monthly inspection reports.

The FDOT was not able to perform emergency repairs using the DSMO's preferred method of full depth bay replacement with CIP concrete because, due to the high

ADT on I-75, the lanes could not be closed to traffic to allow the concrete to cure as required. Instead, the DSMO instructed their asset maintenance contractor to temporarily fasten a ¾ in. thick steel plate with anchor bolts on the deck top over the failure as shown in Figure 4.1. Then timber bracing was installed in the bays underneath to prevent beams from torsion (see Figure 4.2). A few days later, the steel plate was removed and replaced with high strength, fast setting concrete repair material. This repair was performed within a few hours during a night time lane closure [17].

Figure 4.1 Bridge No. 100436, Span 4 Failure (Temporary Steel Plate)

Figure 4.2 Bridge No. 100436, Span 4 Failure (Timber Bracing)

Approximately eleven months after the failure, on August 19, 2009, DSMO’s monthly inspection of deck panel bridges cited significant deflection and deterioration in the repair patch material (see Figure 4.3). Asset maintenance personnel were summoned onsite, and it was agreed by all parties that an immediate repair was required. The lane once again had to be temporarily closed to traffic for a few hours. The deteriorated patch was removed and replaced with sound material and additional timber bracing was installed in both north and south directions of the existing shoring between the beams [18].

Figure 4.3 Bridge No. 100436, Span 4 (Failure in Patch Repair Material)

USF had ranked Bridge No. 100436 No. 8 out of 15 in priority for replacement. The main reason for not ranking this bridge higher provided by the USF research team was because it had only one failing spall repair. However, it is evident that six of the seven bridges ranked for replacement ahead of this bridge had no failing repairs. The same six also had a lower Weighted Index. Six bridges ranked lesser in priority in FDOT Rank, Safety Rank, Normalized Risk, Safety Rank, Importance Rank and had and lower ADTs.

According to the Cracking Diagram in Figure 3.7, the transverse cracking pattern appears to be consistent with spacing of the precast panels. Apparently lack of longitudinal continuity between precast deck panels resulted in the transverse cracks propagating in the "component" or CIP concrete portion. These cracks prevent the deck system from behaving compositely resulting in reduced punching shear capacity as supported by the punching shear calculations performed in the USF Study. If the precast panel support is poor due to the fiberboard bearing material, then the punching shear is resisted by only two sides. In this case, depending on factors like overload and material properties, the punching shear failure of the panel becomes possible.

Considering that ranking categories of Bridge No. 100436 were higher in priority than the six other bridges ranked before it and discovering that this bridge had a prevalent problem with transverse cracking, it is determined that it would probably have been more accurate to rank this bridge at replacement priority No. 2, before the other six bridges. However, since Bridge No. 100436 was on the replacement prioritization list and ranked approximately midway between the 15 bridges on that list, it is the author's judgment that USF's ranking was on target.

4.3 Summary and Conclusions

The USF prioritization for replacement of Bridge No. 100332 was accurate and totally justified in being ranked No. 1.

The prioritization of Bridge No. 100436 was also on target. It was on the replacement prioritization list and ranked No. 8, approximately midway between the 15 bridges on that list.

The USF Study replacement rankings for both bridges that subsequently failed in 2007, Bridge Nos. 100332 and 100436 were justifiable.

5. PRESENT STATUS OF PRECAST DECK PANEL BRIDGE REPLACEMENT

5.1 Status

Utilizing the $78 million acquired in 2001along with implementing strategies such as including deck replacement within interstate widening projects, FDOT has been working vigorously on replacing the existing composite precast deck panel systems on bridges in Districts 1 and 7 with CIP concrete decks. At this point, the decks of 51 bridges in both districts combined have been replaced with CIP concrete decks. The majority of the funding by far was consumed in District 1 [19]. The breakdown is as follows:

- District 1- bridges carrying or over I-75: 36
- District 7- bridges carrying I-75: 9
- Crosstown Expressway Bridges: 6

The FDOT has shortlisted four design build firms for deck panel replacement on Crosstown viaduct bridges. The contract is pending final selection and execution, which is expected to happen sometime in early 2011 and work should begin before the end of the year. However, due to the limited availability of funding and the current condition of the state's economy, the remaining deck panel bridges in both, Districts 1 and 7, will have to be addressed with repairs until additional funding, if any, can be acquired for complete deck replacement. Currently three precast deck panel bridges that were on District 7's Work Program since 2000, Bridge Nos. 100468, 100469 and 100470, were moved from the district's planning category of "deck replacement" to "deck repairs." In addition, two more precast deck panel bridges, Bridge Nos.100358, 100359, have been added to the work program for deck repair as well [20]. Along with the lack of funding, deck replacement on high ADT highways such as I -75 is no longer feasible due to the volume of traffic backup that is created as a result of closing lanes. District 7's lane closure policy justifies closing lanes based on traffic counts. If the traffic count is too high as the case on I-75, the district will not allow any lane closures for planned projects. For this reason it is imperative to research the repair and rehabilitation methods to address precast deck panel bridge deficiencies and determine the effectiveness of each application.

The USF Study recommended replacement prioritization tables for District 1, District 7 and the Crosstown Expressway. Replacement rankings were provided for bridges that needed replacement as well as for bridges in good condition. As part of this research, the tables have been updated with current information regarding NBI condition rating and replacement status.

Table 5.1 Recommended District 1 Bridge Replacement Sequence

No.	Bridge ID #	Location	Current Condition NBI Rating	Replaced
1	130090	I-275 NB Over I-75	N.A.	Yes
2	130112	I-275 SB R to I-75 NB & I-75 And I-275 Ramps	5 (Fair)	No- removed from program
3	170081	I-75 Over Palmer Blvd	N.A.	Yes
4	170080	I-75 Over Main A Canal	N.A.	Yes
5	030188	I-75 over CR-846	Information unavailable	N.A.

Table 5.1 (Continued)

6	170094	I-75 NB Over Havana Road	N.A.	Yes
7	170099	SR-681 SB Over CSX RR	6 (Satisfactory)	No
8	170089	I-75 Over River Road/Cr 777	N.A.	Yes
9	170100	SR-681 NB Over CSX RR	7 (Good)	No
10	010064	Oil Well Road Over I-75	6 (Satisfactory)	No
11	030187	I-75 Over CR-846	Information unavailable	N.A.
12	170096	I-75 SB Over Jacaranda Blvd	N.A.	Yes
13	170079	I-75 Over Main A Canal	N.A.	Yes

Table 5.2 District 1 Bridges in Good Condition

No.	Bridge ID#	Location	Current Condition NBI Rating	Replaced
1	10059	I-75 Over CR-776	N.A.	Yes
2	10065	Airport Rd Over I-75	7 (Good)	No
3	10066	CR-768 Over I-75	7 (Good)	No
4	10067	US-17 Over Florida St.	6 (Satisfactory)	No
5	10068	US-17 Over Florida St.	6 (Satisfactory)	No
6	10075	Carmalite St. Over I-75	6 (Satisfactory)	No
7	10090	US-17 Over Lavilla St. & Rr	6 (Satisfactory)	No
8	10091	US-17 Over Lavilla St. & Rr	6 (Satisfactory)	No
9	120085	US-41 Over Imperial River	7 (Good)	No
10	120086	US-41 Over Imperial River	Information unavailable	N.A.
11	120088	SR-685 Over Matanzas Pass	6 (Satisfactory)	No
12	120114	Slater Rd. Over I-75	6 (Satisfactory)	No- moved out to 2020
13	120126	I-75 NB Over Alico Rd./Canal	Information unavailable	N.A.
14	120127	I-75 SB Over Alico Rd./Canal	Information unavailable	N.A.
15	130085	I-75 NB Over SR-64	N.A.	Yes
16	130089	Erie Rd Over I-75	6 (Satisfactory)	No
17	130107	Mendoza Rd Over I-75	7 (Good)	No
18	170082	I-75 Over Palmer Blvd.	N.A.	Yes
19	170083	I-75 SB Over SR-780	7 (Good)	No
20	170084	I-75 NB Over SR-780	7 (Good)	No
21	170090	I-75 Over River Rd.	N.A.	Yes
22	170091	I-75 SB Over Jackson Rd.	N.A.	Yes
23	170092	I-75 NB Over Jackson Rd.	N.A.	Yes
24	170093	I-75 Over SR-80	N.A.	Yes
25	170095	I-75 NB Over Jacaranda Blvd.	N.A.	Yes

Table 5.3 Recommended District 7 Bridge Replacement Sequence

No.	Bridge ID#	Location	Current NBI Rating	Replaced
1	100468	I-75 SB Over Woodberry Rd.	6 (Satisfactory)	No
2	100347	I-75 NB Over SR-674	N.A.	Yes
3	100470	I-75 SB Over CSX RR	6 (Satisfactory)	No
4	100358	I-75 SB Over Alafia River	5 (Fair)	No
5	100359	I-75 NB Over Alafia River	5 (Fair)	No
6	150122	I-275 NB Over 5th Ave. North	7 (Good)	No
7	100346	I-75 SB Over SR-674	N.A.	Yes
8	100436	I-75 NB Over Broadway/CR-574 / CSX RR	5 (Fair)	No
9	100338	US-41 Over Mackay Bay	5 (Fair)	No
10	100357	I-75 NB Over Riverview Drive	5 (Fair)	No
11	100356	I-75 SB Over Riverview Drive	5 (Fair)	No
12	100080	SR 60 WB Over Bypass Canal	5 (Fair)	No
13	100081	SR 60 EB Over Bypass Canal	5 (Fair)	No
14	100049	US-41Over Palm River	7 (Good)	No
15	100351	Valroy Road Over I-75	5 (Fair)	No

Table 5.4 District 7 Bridges in Good Condition

No.	Bridge ID#	Location	Current NBI Rating	Replaced
1	100398	I-75 NB Over Sligh Ave./Ramp D-1	7 (Good)	No
2	100339	US 301 Over Bypass Canal	6 (Satisfactory)	No
3	100377	Gibsonton Dr. Over I-75	5 (Fair)	No
4	100399	SR 582 WB Over Bypass Canal	6 (Satisfactory)	No
5	100424	Ramp B Over US 92	7 (Good)	No
6	100435	I-75 SB Over Broadway/CR574/CSX RR	6 (Satisfactory)	No
7	100469	I-75 NB Over Woodberry Rd.	6 (Satisfactory)	No
8	100471	I-75 Over CSX RR	6 (Satisfactory)	No
9	150121	I-275 SB Over 5th Ave	7 (Good)	No
10	150145	I-375 WB Over CR-689	7 (Good)	No
11	150146	I-375 EB Over CR-689	7 (Good)	No
12	150168	I-175 WB Over 6th St. S	7 (Good)	No
13	150169	I-175 EB Over 6th St. S	7 (Good)	No
14	150170	8th St. S. Over I-175	5 (Fair)	No

Table 5.5 Recommended Crosstown Expressway Replacement Sequence

No.	Bridge ID #	Location	Current Condition NBI Rating	Replaced
1	100332	SR 618 WB Over Hills River/ Downtown TPA	5 (Fair)	Scheduled in 2011
2	100333	SR 618 EB Over Hills River/ Downtown TPA	6 (Satisfactory)	Scheduled in 2011
3	100443	SR618 Over Ramp D & SR585/22nd Street & R/R	4 (Poor) Structurally Deficient	No
4	100453	SR 618 Over 50th Street (US 41)	N.A.	Yes
5	100448	SR 618 Over CSX RR	Removed to accommodate new elevated bridge.	N.A.
6	100451	SR 618 Over 39th Street	6 (Satisfactory)	No
7	100447	SR 618 Over RR	7 (Good)	No
8	100457	SR 618 Over Maydell Drive	N.A.	Yes
9	100449	SR 618 Over 34th Street & Creek	6 (Satisfactory)	No
10	100454	SR 618 Over 50th Street (US 41)	N.A.	Yes
11	100456	SR 618 Over CSX R/R	N.A.	Yes
12	100444	SR 618 Over SR 585 22nd St/CSX RR	6 (Satisfactory)	No
13	100455	SR 618 Over CSX RR	N.A.	Yes
14	100450	SR 618 Over 34th Street & Creek	Replaced to accommodate new elevated bridge.	N.A.
15	100452	SR 618 Over 39th Street	Removed to accommodate new elevated bridge.	N.A.
16	100446	SR 618 Over 26th Street	6 (Satisfactory)	No
17	100458	SR 618 Over Maydell Drive	N.A.	Yes
18	100445	SR 618 Over 26th Street	6 (Satisfactory)	No

5.2 Summary and Conclusions

At this point, the decks of 51 bridges in both districts combined have been replaced with CIP concrete decks. The majority of the funding was consumed in District 1, followed by District 7, then the Crosstown Expressway. However due to budget restrictions, aside from three bridges on the work program for replacement, the remaining composite precast deck panel bridges will have to be addressed with rehabilitation.

6. BRIDGE DECK REPAIR METHODS

6.1 Introduction

As stated in the previous chapter, because of the difficulty in acquiring funding due to the sluggish condition of the state and national economy at this time, the remaining deck panel bridges in both, Districts 1 and 7, will have to be addressed with rehabilitation rather than replacement as originally planned.

6.2 Repair Materials

Repair material and method of construction can make the difference between a good and poor repair. Therefore, prior to proceeding with the repair procedures it is crucial to discuss some relevant issues regarding repair materials. Repair material selection is not an easy task because there are too many material manufacturing companies and even more so of products to select from. The information provided by the manufacturers and distributors is incomplete or in worse cases misleading. Additionally, new materials as well as new repair methods are constantly introduced and changes are frequently being applied to tried and true products.

A good source for guidance on selecting a repair material is The American Concrete Institute's 546.3R-06, Guide for the Selection of Materials for the Repair of Concrete [21]. This publication was written with the goal to provide guidance on common repair material, emphasize relevant repair material properties, test procedures, minimum performance levels and applications for requirements and service environments.

The first step of the process is to perform an in-depth inspection of the problem in the field and to document the deficiencies, potential damage and damage cause. This is followed by an assessment of repair service conditions, repair objectives, desired service life and future maintenance.

The following are the most important repair material properties [22] along with the relationship of repair material (R) to concrete substrate (C) [23] listed in descending order:

1. Drying Shrinkage R<C
2. Tensile Strength R>C
3. Modulus of Elasticity R<C
4. Tensile Strain Capacity R>C
5. Thermal Expansion/Contraction R<C
6. Creep R>C
7. Compressive Strength R=C

Volume stability i.e., dry shrinkage, refers to the dimensional change of the repair material. The existing concrete, or substrate, is almost always stable and if the repair is not, high shear stresses occur at the interface that can lead to debonding, cracking and ultimately failure of the repair. Dry shrinkage is arguably the most important property for a durable repair.

Another important property is the tensile strength. This is the maximum unit stress a repair material is capable of resisting under axial tension.

The modulus of elasticity is the ratio of normal stress to corresponding strain for tensile or compressive stress below the proportional limit of the material. If the repair is not structural, then it is preferable that the repair material has a lower modulus of elasticity than the substrate. However, if the repair is structural, then the repair material should have a modulus of elasticity as close as possible to the substrate's property.

The tensile strain capacity is the concrete's resistance of cracking from slow rates of stress development under uniaxial tension. Investigation shows that the tensile strain capacity of concrete is a relatively independent parameter. [24].

The coefficient of thermal expansion is the change in linear dimension per unit length of a material per degree of temperature change. In situations where temperature is not controlled, such as in exterior and some interior applications, it is desirable for the repair material to have a coefficient of thermal expansion similar to that of the substrate concrete so that the two materials behave similarly under daily and seasonal temperature variations. If the coefficients vary significantly, the differential movements due to temperature fluctuations could affect the performance of the repair, and should be accounted for in the repair design.

Creep is time-dependent deformation due to sustained load. Because many repairs are not subjected to significant compressive forces, compressive creep may not be a significant property of repair materials. Creep can be important if stress is induced in the repair material due to restraint of shrinkage strains or due to factors such as thermal movement or the application of live loads.

Compressive strength is the measured maximum resistance of a material to axial compressive loading, expressed as force per unit cross-sectional area. This is the property that most material manufacturers and distributers like to tout with high numbers. However, a high compressive strength does not mean anything if the repair patch is separating from its substrate concrete due to shrinkage.

6.3 Repair Types

The DSMO uses seven fundamental repair methods used to address the deficiencies on precast deck panel bridges. Table 6.1 categorizes the repairs with their positive and negative aspects, indicating the stage of the deterioration model in which they are generally implemented along with an overall assessment on the effectiveness of the repairs.

Table 6.1 Repair Methods

Repair Types	Favorable Characteristics	Unfavorable Characteristics	Used at Stage of Deterioration model	Effectiveness of Repair
Crack Repair	Helps keep out debris and impurities that may accelerate deterioration.	Does not impede the deterioration process or help structurally.	1,3	Not effective
Maintenance Spall Patching (Asphalt)	Easy to place without much disruption to traffic. Very inexpensive repair.	Only for temporary use If left longer than a week, could be detriment rather than a benefit to the bridge.	4,6,9,11,12	Not effective
Localized Spall Repair	Provides a repair with compressive strength in comparison to the maintenance patching with asphalt.	Due to the nature of the deck panel system not acting compositely, the localized repairs start separating at the edges and new spalls described as “walking spalls”.	4,6,9,11, 13	Not effective

Table 6.1 (Continued)

Grout Packing	Good to slow down deterioration process by providing positive bearing and extending bridge life. No traffic impact.	Does not mitigate deficiencies that were present prior to grout packing.	2 thru 10	Good to slow down deterioration
M1 Repair	Repair replaces deteriorated CIP component by extending to the top of the precast panel.	Can separate from panel, start separating at the edges and new walking spalls start to appear. Process is moderately labor intensive and impacts traffic.	7	Better than spall repair but not very effective
Full Span M1 Repair with Grout Packing	Last longer than any other type aside from full depth bay replacement.	Process is labor intensive and causes impacts to traffic.	7	Effective
Full Depth Bay Replacement	Addresses the root cause of problem: elimination of vertical and longitudinal separation between the precast deck panel and CIP surfaces.	Costly, very labor intensive and causes significant impacts to the traveling public.	8 thru 13	Very effective

A more detailed write up on the repairs presented in Table 6.1 is provided in Appendix B.

Assessment of the prevailing repair procedures tabulated above indicates that crack repair, maintenance spall patching and localized spall repair are not effective because they do not mitigate the deterioration process caused by the panel and CIP sections not behaving compositely. Grout packing is a good method to slow down deterioration by providing positive bearing. However, if it is not applied at an early stage the deterioration that existed on the bridge will continue to intensify. Both M1 and M1 with grout packing are acceptable repair procedures, but as seen in the deterioration model, these repairs eventually start to weaken because there is still a separation between the precast panel and the cast in place section. Full depth bay replacement with CIP concrete is the most effective repair method because it addresses the root cause of the deterioration by eliminating the vertical and longitudinal interface between the precast deck panel and CIP concrete surfaces and provides positive bearing for the deck.

Even though full depth bay replacement is the most effective repair, it is difficult to apply this method to high ADT highways due to the extended lane closures required to accommodate concrete curing time, as was the case on the emergency repair for Bridge No. 100436, I-75 over E. Broadway Ave. and CSX Railroad. Therefore, the deficiency was temporarily repaired and shored in September 2007 and a repair project was programmed to start construction in late 2009.

The DSMO tasked Parsons Brinckerhoff (PB) to design a pilot project to replace the deficient bays on Bridge No. 100435 and 100436, twin bridges on I-75 that would minimize disruption to traffic. Traffic analysis and lane closure calculations indicated that this area of I-75 should only have nighttime lane closures due to high ADT conditions. Therefore the author, as PB’s design project manager, teamed with SDR Engineering, Inc. to design the partial deck replacement using full depth precast deck panels to achieve the DMSO’s goal [25]. The design detailed that the existing deteriorated deck be cut, demolished, removed and replaced with a full depth precast concrete deck only at night. This limitation required partial installation of sections of the full depth precast panel per night. It was specified that a minimum length of 30 ft. of full depth panel was to be installed per night. Near surface mounted (NSM) Carbon Polymer Reinforced Fiber (CFRP) bars were installed to transfer shear into the existing deck. The construction was executed during the second quarter of 2010 and although similar technology has been used in other states e.g., Issa [26], it was the first time that this method was applied in Florida. It is also innovative being that this is the only method of full depth precast panels that transfers forces longitudinally employing the use of NSM CFRP bars, rather than transversely. For this reason the design for this pilot project was closely monitored and scrutinized by the DSMO as well as FDOT's State Structures Design Office in Tallahassee.

Key repair illustrations from the design plans are shown in Figure 6.1 and a more detailed description of the project is as follows.

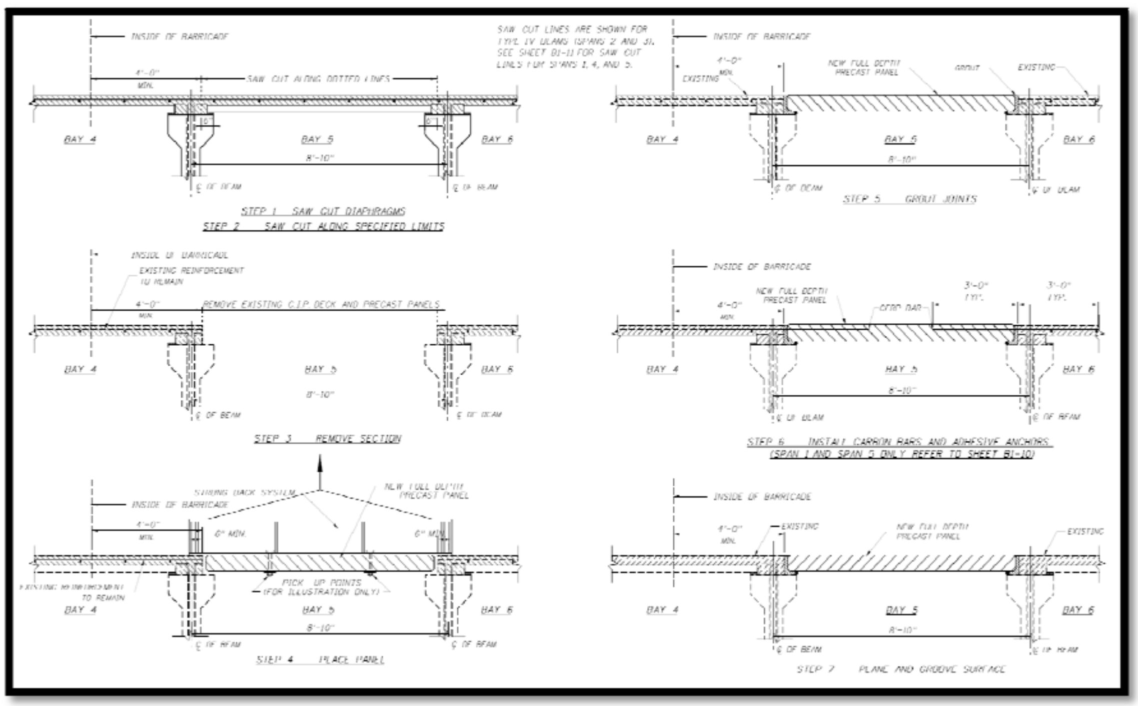


Figure 6.1 Full Depth Precast Panel Design Plan Details [25]

The full depth 6,000 psi Class IV precast concrete panels were cast in a prestressing yard and brought on to the site nightly as needed for construction.

The existing composite precast and CIP deck was sawcut, from the inside face of beam to the inside face of beam with an additional six inches on both sides. A platform was constructed under the bay for containment and subsequent disposal of debris. An overhead crane was used for the removal of the cut out existing section (see Figure 6.2) as well as for the placement of the new full depth precast concrete panel section as shown in Figure. A "strong back" system was used to suspend the precast panel from designed pick-up points the top as shown in Figure 6.3.

Figure 6.2 Existing Deck Cut and Removal Using Strong Back

Figure 6.3 New Deck Suspension Using Strong Back

The process started by cleaning the edges of the top flanges of the beams with light sandblasting. After sandblasting, potable water was applied to surfaces of top flanges and vertical faces of existing deck slab to obtain a "saturated surface-dry" condition prior to grouting of the longitudinal joints.

While the new panel is still suspended in place using the strong back, low pressure grout pumps were used to ensure full penetration of the epoxy grout below the edges of the panel to form proper seating as shown in Figure 6.4. The epoxy is kept in place between the top of beam and bottom of panel using backer rods, also shown in Figure 6.4. The epoxy was later placed on the panel and panel interface (i.e., transverse edges) of the previously installed full-depth panel section and the new section of panel to be installed. The new section of the full depth panel was pushed in place to form and seal the construction joint.

Figure 6.4   Installing Backer Rod for Pouring Epoxy and Finished Epoxy Joint for Bonding New Panel with Existing as well as Seating for New Panel

On the spans with AASHTO Type II Prestressed Concrete Beams, the shear connectors had to be cut and removed in order to fit the new precast panels into place and to ensure that the panels have proper bearing on the beams. Therefore, to transfer shear from the deck into the beams, adhesive bonded anchors were installed through preformed holes in the precast concrete panel slabs as shown in Figure 6.5. Holes with specific diameters were drilled using a rotary hammer drill and bit into the beams for placement of the adhesive anchors. It was specified to use a metal detector specifically designed for locating steel in concrete to avoid conflicts with the beams existing steel reinforcement.

Core drilling was performed to clear existing steel reinforcement. Next, the holes were cleaned using oil-free compressed air to remove loose particles accumulated from drilling.

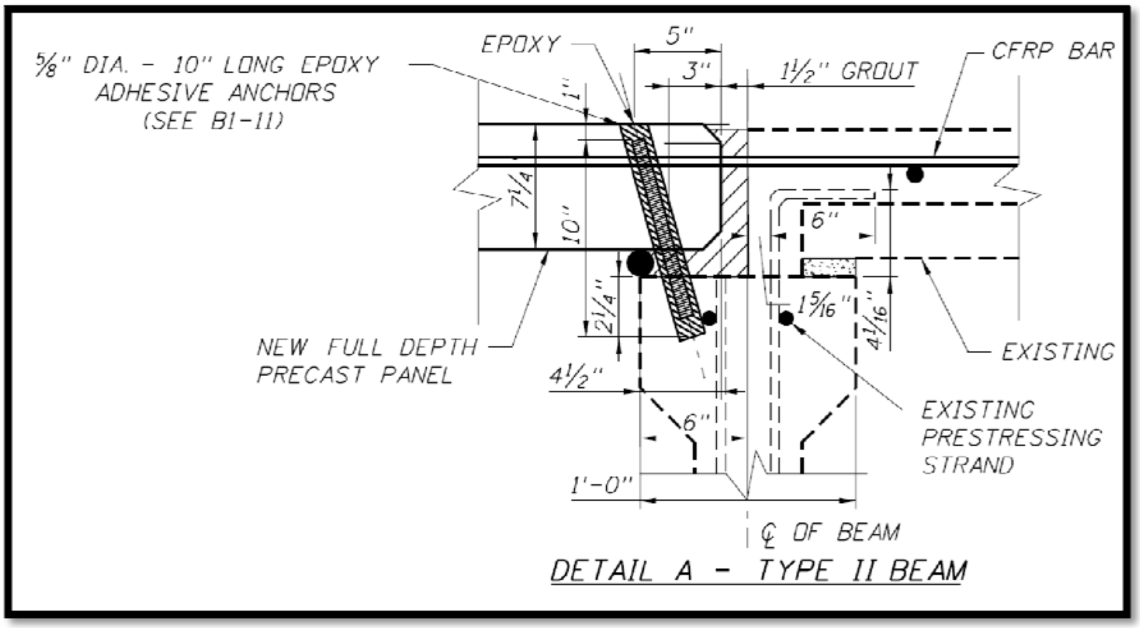


Figure 6.5 Adhesive Anchors

The anchors were installed and adequate quantities of the adhesive bonding material were used to fill the drilled hole to approximately 1/4 inch of the concrete surface measured after placement of the steel bar or anchor.

Grooves, approximately 3 ft. long were cut on both sides of the panel transversely, and extended 3 ft. into the existing deck. These were for the installation of the Near-Surface Mounted (NSM) CFRP Bars. The use of CFRP bars are required for flexural strengthening in the negative moment regions of the bridge deck.

All grooves, where the NSM CFRP bars were to be placed were half-filled with embedding paste. It was specified to avoid entrapped air voids between concrete substrate and the embedding paste.

The CFRP bars were cut to the specified length, cleaned and placed into the half filled groove and slightly pressed to force the paste to flow around the bar, completely filling the space between the bar and the sides of the groove. The groove was then filled with more paste until the surface leveled as shown in Figure 6.6.

Figure 6.6 Sawcutting Grooves into Deck and Placing CFRP Rods in Epoxy

The CFRP system was allowed to cure then a protective coating was applied on the surface of the CFRP system.

Limited planning and grooving of the new panel was required to match the finish and grade of the existing bridge deck.

The pilot construction project was successful by resulting in a sound repair with no disruption to daytime traffic. Lane closures were performed only during night and 30 lineal feet of the bay was replaced per night. A total of 8,831 ft. of bay replacement was performed using full depth precast panels, 2,944 ft. on Bridge No. 100435 and 5,887 ft. on Br. No. 100436. Deck following project completion is shown in Figure 6.7.

Figure 6.7 Completed Deck (Transverse and Longitudinal NSM CFRP Installation)

6.4 Summary and Conclusions

Being that the remaining composite precast deck panel bridges will have to be addressed with rehabilitation in lieu of replacement, this chapter reviewed the effectiveness of seven repair methods. Currently the most effective method is full depth bay replacement with CIP concrete. If it is not possible to implement this method due to budget constraints, then grout packing should be used to replace the fiber board bearing material with non-shrink grout or epoxy provide to positive bearing to slow down the deterioration process. However, if conventional full depth bay replacement is not feasible due to traffic restrictions, then the favorable method of rehabilitation is the use of full depth precast panels.

7. SUMMARY AND CONCLUSIONS

Precast deck panel bridges since the mid 1980s have been experiencing premature deterioration in Florida, which has been a great source of inconvenience to the FDOT in regards to time, money and impact to the traveling public. Five sudden deck failures occurred in Districts 1 and 7 between 2000 and 2003 as documented in Table 2.1.

In 2005 USF completed a comprehensive study for the FDOT. The main goal of the study was identifying and prioritizing deck replacement of high risk bridges in Districts 1 and 7, primarily to prevent the occurrence of similar failures. However, since finalization of the study in 2005, two subsequent failures have taken place.

The objectives of this research were to reassess the prioritizations of USF Study regarding the two subsequent failures, provide an update on the status of the composite precast deck panel bridges in Districts 1 and 7 and assess the effectiveness of repair methods used for this type of bridge deck system.

The sudden failures took place on Bridge No. 100332, Crosstown Expressway Viaduct, Span 39 and Bridge No. 100436, I-75 over E. Broadway Ave., CR 574 and CSX Railroad. Both failures occurred in 2007 and were located in District 7 as shown in Table 3.1.

Bridge No. 100332 was ranked as the No. 1 priority for replacement on the Crosstown prioritization list as indicated in Table A.5. First priority replacement ranking was given to this bridge because it Ranked No. 1 in the categories of Safety, Risk, Normalized Risk, Importance, ADT, FDOT Ranking, Weighted Index, Failing Repair Count, and Spall Count.

However, despite being assigned with highest priority for replacement on the Crosstown Expressway, the FDOT was unable to acquire the large amount of funding required for deck replacement in time to avoid the 2007 failure. Hence, the USF prioritization for replacement of Bridge No. 100332 was very accurate, and absolutely justified in being ranked No. 1. Had the FDOT been able to acquire the replacement funding for this bridge in time the failure might have been avoided.

Bridge No. 100436 was ranked as No. 8 for replacement. The USF Study Team's primary motive for not prioritizing this bridge higher for replacement at the time of the study was because it only had one failing repair. However, as exhibited by the monthly inspection reports, the repairs began deteriorating rapidly immediately after the study was finalized and continued until failure.

Six of the seven bridges ranked for replacement ahead of this bridge had no failing repairs. The same six also had a lower Weighted Index. Six bridges ranked lesser in priority in FDOT Rank, Safety Rank, Normalized Risk, Safety Rank, Importance Rank and had and lower ADTs. This bridge also began showing signs of widespread transverse cracking since the early 2000s. Most of the transverse cracking pattern appears to be consistent with spacing of the precast panels. They are 1/8 in. width but three had grown to a ¾ in. width over time.

Considering that ranking categories of Bridge No. 100436 were higher in priority than the six other bridges ranked ahead of it and discovering that this bridge had a prevalent problem with transverse cracking [20], it is determined that this bridge could probably have been more accurately ranked at replacement priority No. 2, prior to the six other bridges. However, since Bridge No. 100436 was on the replacement prioritization list and ranked No. 8, approximately midway between the 15 bridges on that list, it is the author's judgment that USF's ranking was justifiable.

In summary, the USF Study's replacement rankings for the bridges that subsequently failed in 2007, Bridge Nos. 100332 and 100436 were well-founded.

Since completion of the USF Study in 2005, a total of 51 composite precast deck panel bridges in Districts 1 and 7 have been replaced with full depth CIP decks. The majority of the funding went to District 1, followed by District 7 and the Crosstown Expressway as indicated in the following breakdown:

- District 1- bridges carrying or over I-75: 36
- District 7- bridges carrying I-75: 9
- Crosstown Expressway Bridges: 6

Three additional bridges are in District 1's work program to be replaced between now and 2020. However, due to the limited availability of funding and the current condition of the state's economy, the remaining interstate and high ADT highway deck panel bridges will have to be addressed with rehabilitation until additional funding, if any, can be acquired for complete deck replacement. The remaining high ADT bridges are as follows:

- District 1: 25
- District 7: 27
- Crosstown Expressway: 10

For this reason, research was performed on the repair methods used for precast deck panel bridge deficiencies to determine the effectiveness of each application.

The repair method and materials play a big part in the performance and service life of the deck. Deck repairs are suitable when good concrete repair material and construction methods are applied. Where this is not carried out there is progressive degradation as indicated in the deterioration model shown in Figure 2.1, which can lead to punch through failure of the deck. This was the case in all seven failures where asphalt patching was used for repair.

The effectiveness of seven repair methods was examined. Currently the most effective permanent repair for deck panel bridge deficiencies is full depth bay replacement. If full depth replacement is not possible due to traffic or budget constraints, then grout packing should be used to replace the fiber board bearing material with non-shrink grout or epoxy to provide positive bearing in order to slow down the deterioration process until the full depth bay replacement or entire span replacement can be accomplished. The conventional full depth bay replacement is not always feasible due to restrictions such as not being able to close down lanes on high ADT highways. In this case it was found that the favorable method of construction for bay replacement was the use of full depth precast panels. The DSMO performed a pilot construction project employing this method on Bridge No. 100435 and 100436 earlier in the year. Lane closures were performed only during night and 30 lineal feet of the bay was replaced per night. The project ended successfully by providing a sound repair, consisting of 8831 ft. of bay replacement using full depth precast panels. This was done without disrupting any daytime traffic.

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APPENDIX A: USF STUDY'S PRIORITIZATION TABLES

APPENDIX A (Continued)

Table A.1 Recommended District 1 Bridge Replacement Sequence

No.	Bridge No.	Location
1	130090	I-275 NB Over I-75
2	130112	I-275 SB R to I-75 NB & I-75 And I-275 Ramps
3	170081	I-75 Over Palmer Blvd
4	170080	I-75 Over Main A Canal
5	030188	I-75 over CR-846
6	170094	I-75 NB Over Havana Road
7	170099	SR-681 SB Over CSX RR
8	170089	I-75 Over River Road/Cr 777
9	170100	SR-681 NB Over CSX RR
10	010064	Oil Well Road Over I-75
11	030187	I-75 Over CR-846
12	170096	I-75 SB Over Jacaranda Blvd
13	170079	I-75 Over Main A Canal

APPENDIX A (Continued)

Table A.2 District 1 Bridges in Good Condition

No.	Bridge No.	Location
1	10059	I-75 Over CR-776
2	10065	Airport Rd Over I-75
3	10066	CR-768 Over I-75
4	10067	US-17 Over Florida St.
5	10068	US-17 Over Florida St.
6	10075	Carmalite St. Over I-75
7	10090	US-17 Over Lavilla St. & Rr
8	10091	US-17 Over Lavilla St. & Rr
9	120085	US-41 Over Imperial River
10	120086	US-41 Over Imperial River
11	120088	SR-685 Over Matanzas Pass
12	120114	Slater Rd. Over I-75
13	120126	I-75 NB Over Alico Rd./Canal
14	120127	I-75 SB Over Alico Rd./Canal
15	130085	I-75 NB Over SR-64
16	130089	Erie Rd Over I-75
17	130107	Mendoza Rd Over I-75
18	170082	I-75 Over Palmer Blvd.
19	170083	I-75 SB Over SR-780
20	170084	I-75 NB Over SR-780
21	170090	I-75 Over River Rd.
22	170091	I-75 SB Over Jackson Rd.
23	170092	I-75 NB Over Jackson Rd.
24	170093	I-75 Over SR-80
25	170095	I-75 NB Over Jacaranda Blvd.

APPENDIX A (Continued)

Table A.3 Recommended District 7 Bridge Replacement Sequence

No.	Bridge No.	Location
1	100468	I-75 SB Over Woodberry Rd.
2	100347	I-75 NB Over SR-674
3	100470	I-75 SB Over CSX RR
4	100358	I-75 SB Over Alafia River
5	100359	I-75 NB Over Alafia River
6	150122	I-275 NB Over 5th Ave. North
7	100346	I-75 SB Over SR-674
8	100436	I-75 NB Over Broadway/CR-574 / CSX RR
9	100338	US-41 Over Mackay Bay
10	100357	I-75 NB Over Riverview Drive
11	100356	I-75 SB Over Riverview Drive
12	100080	SR 60 WB Over Bypass Canal
13	100081	SR 60 EB Over Bypass Canal
14	100049	US-41Over Palm River
15	100351	Valroy Road Over I-75

Table A.4 District 7 Bridges in Good Condition

No.	Bridge No.	Location
1	100398	I-75 NB Over Sligh Ave./Ramp D-1
2	100339	US 301 Over Tampa Bypass Canal
3	100377	Gibsonton Dr. Over I-75
4	100399	SR 582 WB Over Bypass Canal
5	100424	Ramp B Over US 92
6	100435	I-75 SB Over Broadway/CR574/CSX
7	100469	I-75 NB Over Woodberry Rd.
8	100471	I-75 Over CSX RR
9	150121	I-275 SB Over 5th Ave
10	150145	I-375 WB Over CR-689
11	150146	I-375 EB Over CR-689
12	150168	I-175 WB Over 6th St. S
13	150169	I-175 EB Over 6th St. S
14	150170	8th St. S Over I-175

APPENDIX A (Continued)

Table A.5 Recommended Crosstown Expressway Replacement Sequence

No.	Bridge No.	Location
1	100332	SR 618 WB Over Hills River/ Downtown TPA
2	100333	SR 618 EB Over Hills River/ Downtown TPA
3	100443	SR618 Over Ramp D & SR585/22nd Street & R/R
4	100453	SR 618 Over 50th Street (US 41)
5	100448	SR 618 Over CSX RR
6	100451	SR 618 Over 39th Street
7	100447	SR 618 Over RR
8	100457	SR 618 Over Maydell Drive
9	100449	SR 618 Over 34th Street & Creek
10	100454	SR 618 Over 50th Street (US 41)
11	100456	SR 618 Over CSX R/R
12	100444	SR 618 Over SR 585 22nd St/CSX RR
13	100455	SR 618 Over CSX RR
14	100450	SR 618 Over 34th Street & Creek
15	100452	SR 618 Over 39th Street
16	100446	SR 618 Over 26th Street
17	100458	SR 618 Over Maydell Drive
18	100445	SR 618 Over 26th Street

APPENDIX B: REPAIR METHODS

B.1 Crack Repair

The USF study reported that at the second stage of the deterioration model is the occurrence of longitudinal cracks over the edges of the girders. This type of cracking starts early in precast deck panel bridges and is the most common type of cracking. This crack is mainly the result of creep induced by prestressing forces in the precast panel, and the differential shrinkage between the CIP concrete and the deck precast panel. Once the formation of longitudinal cracking has started, sporadic transverse cracks can also develop in the deck.

The cracking can be repaired with epoxy crack injection or crack sealant. Crack injection is a structural repair meaning that it ideally restores the structural strength of the deck to original. Crack sealing penetrates and covers the cracking in order to avoid water, chlorides and other impurities from entering inside the deck [B.1]. If it is determined that the crack is active, (i.e., opening and closing), then epoxy crack injection should not be used because it does not have the flexibility like crack sealant.

The transverse cracks on Bridge No. 100436 were sealed using a flexible sealant following the first failure on September 11, 2007.

B.2 Maintenance Spall Patching

After the occurrence of the second parallel crack, the concrete trapped between the two cracks is already internally cracked and starts to crumble. During the fourth stage of the deterioration model, a spall develops. At this stage, a new parameter is introduced, the effect of the rainwater forced inside the cracks by vehicles. Although this is difficult to quantify, bridge inspectors have observed this phenomenon over the years.

FDOT classifies deck patching in three different categories based upon depth [B.2]:

- Type A- Above the top layer of reinforcing steel
- Type B- At least one inch below the top layer of reinforcing
- Type C- Full depth replacement

The most common and simplest repair method is maintenance spall patching. It is used for spalls that are in the CIP portion of the deck. When a deficiency such as a spall would appear on the bridge deck (approximately ten years after construction as indicated in the simplified deck deterioration process depicted in Figure 2.1), it was common practice for the FDOT maintenance crews to patch it with flexible (i.e., "cold patch") asphaltic concrete as illustrated in Figure B.1.

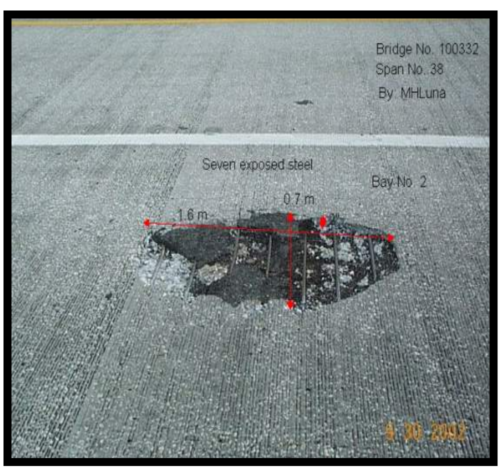

Figure B.1 Bridge No. 100332, Span 38- Asphalt Patch (2 Days Before Failure)

The asphaltic concrete patching is not labor intensive for the crews and can be performed in a matter of minutes with very minimal disruption to the traveling public. It is also a very inexpensive procedure. The maintenance crews would set up a temporary lane closure(s) as needed, clean debris out of the spall using hand tools and patch it using a ready mix bag. The purpose of asphalt patching was to alleviate immediate danger to the motoring public as well as to avoid the spall from getting worse. This method of repair was never meant as a permanent solution, and although it was always the DSMO's policy for the maintenance crew to return within a week and perform a permanent repair, sometimes due to other priorities of the crews, these temporary patches would remain for a longer periods of time [B.3]. This type of patch for extended periods of time has proved to be a detriment rather than a benefit to the bridge. This is especially the case when asphalt is used in steps 11 and 12 of the deterioration model, (i.e., when used to patch spalls inside or adjacent to a deficient M1 Repair). Instead of distributing the load evenly, when the flexible material, which has negligible compressive strength, was placed in the spall it would pound at the precast panel beneath it and the adjacent CIP section at its sides. In most cases this type of pounding action leads to an increase in the area and depth of the spalls and in some cases has led to cracking of the precast panel due to punching shear. In the absolute worst case scenarios, asphalt was used to repair existing repairs in the deck and the pounding resulted in punching a hole through the deck as shown in Figure B.2. In most of the punch through failures, rainy weather had been a catalyst. Water manages to find its way into the patched spall. Water is an incompressible fluid, even more so than the incompressible properties of asphalt. The wheel loading on the patch causes pumping action between the asphalt and the precast panel until failure. Although there is no solid proof, it is strongly believed that this is a major cause of punch through failure in the deck. Six of the seven failures in Tables 1 and 5 occurred after rainfall.

Figure B.2 Bridge No. 100332, Span 38- Asphalt Patch (Failure)

Additionally, as indicated in Tables 2.1 and 3.1, six out of the seven failures had asphalt repairs. Four were standalone asphalt patches and two were asphalt patches used to address deficiencies within existing repairs.

B.3 Localized Spall Repair

Unlike the maintenance spall patch, localized spall repairs are theoretically a permanent type of repair. It is classified by FDOT as Type B or C. This repair method is the immediate follow-up step to the maintenance spall patch for the maintenance crews. It is mainly used for deficiencies that lie within the depth of the CIP portion of the deck, but they have also been used for full depth repair. These repairs are performed using a concrete repair material. This repair method is not so labor intensive, can be done at a relatively low cost, and when using high strength fast setting material, it can be performed using nighttime lane closures, reducing major impacts on the traveling public.

Since bridge deck repair usually involves closing lane(s), the material most often used is some type of rapid-setting concrete repair material. Most brands of this material usually attain 4000 psi in 4 hours.

Although this is a permanent repair, the FDOT has not had much success with the longevity of these repairs. Due to the nature of the deck panel system not acting compositely, the localized repairs start separating at the edges and new spalls described as "walking spalls" start appearing in front of these repairs (see Figure B.3).

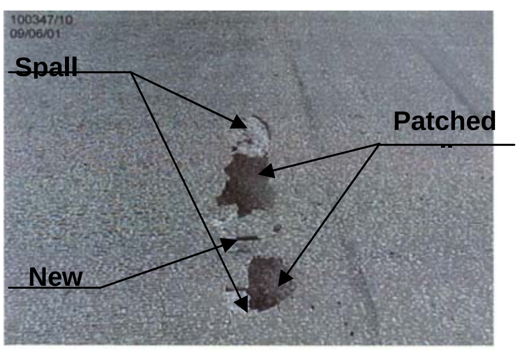

Figure B.3 Patched Spalls and Walking Spalls

Depending on all the associated factors, new spalls can appear in the areas adjacent to the repaired spall after some time. After the spall is created, the residual shear capacity of that region is almost zero, even after it has been patched. Therefore, the shear that was to be supported by that region now has to be redistributed to sections adjacent to the spall. This creates additional stresses in that region, and accelerates its deterioration generating new spalls, which are also generally treated with flexible repair material (see Figure B.4).

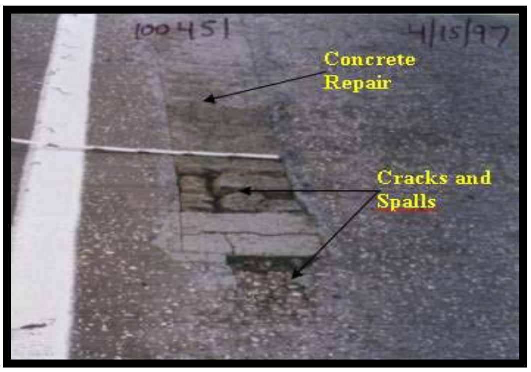

Figure B.4 Localized Spall Repair Starting to Spall at the Edge

One of the seven failures reported in Tables 2.1 and 3.1 occurred at an area which had been repaired by localized full depth spall patching.

B.4 Grout Packing

The majority of deck panel bridges in Florida have been built with fiberboard bearing material or what is commonly referred to as "roofing felt" to support the precast deck panels on the girders. By use of this Fiberboard bearing material, positive bearing is not provided at the ends of the precast panel. Due to the effects of creep and shrinkage, the initial separation and longitudinal crack indicated in Deterioration Stage No. 2 is inherent to precast deck panel construction. However, the few bridges in Florida that had used positive bearing have performed much better and in turn have had longer service lives. The most important conclusion drawn from the forensic study in the 2005 USF report is that the lack of positive panel bearing is clearly the main factor responsible for the occurrence of major deck deterioration such as cracking, delamination, spalling, failing repairs, and in the worst case localized punch-through deck failures. Hence, grout packing is a good method of repair. The fiber board bearing material is replaced with non-shrink Portland cement grout or epoxy grout to provide positive bearing, (see Figure B.5).

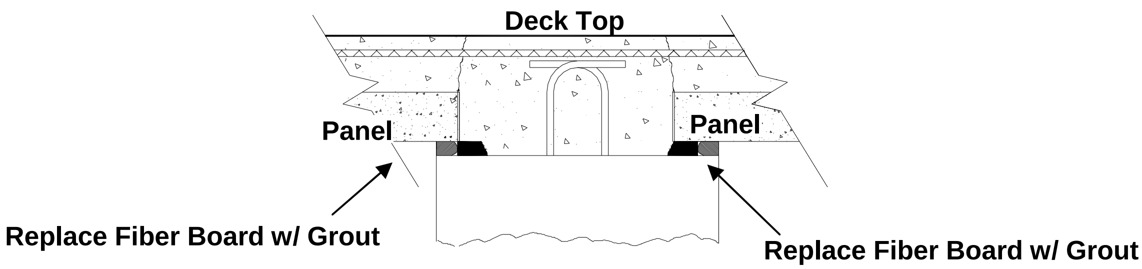

Figure B.5 Bearing Detail after Grout Packing Repair

Grout packing is one of the most effective repair methods used to extend the service lives of precast deck panel bridges. It is very cost effective in comparison to other effective repair methods and it does not cause any interruption in traffic to the facility carried by the structure because the work can be performed utilizing a bucket truck or scissor lift underneath the bridge.

It is important to note that none of the failures reported in Tables 2.1and 3.1 were retrofitted with grout packing.

B.5 M1 Repair

Generally, after several patch and re-patches, an M1 repair is done in the affected area. The M1 repair is used to repair longitudinal spalling along the edge of a beam as illustrated in Figure B.6. The M1 and M2 were FDOT's recommended methods of repair in the 1980s [B.4].

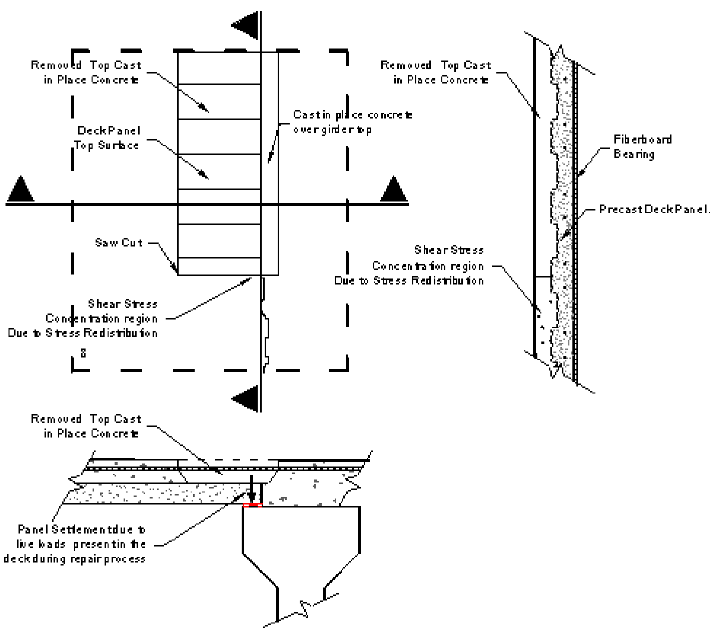

Figure B.6 M1 Repair Procedure (Stage #7)

Unlike localized repairs, the depth of the M1 goes to the top of the precast panel. Although the M1 repairs hold up better than localized repairs, again due to the bridge deck system not acting compositely, they start separating at the edges and walking spalls start occurring in front of these repairs.

Two of the seven failures in Tables 2.1 and 3.1 were associated with deteriorating M1 repairs. On Bridge No. 170085 there was a walking spall, patched with asphalt adjacent to an M1 repair and on Bridge 100332, Span 70, asphalt was used to patch a deficiency within an existing M1 repair.

B.6 Full Span M1 Repair with Grout Packing

This somewhat modified M1 repair is also used to repair longitudinal spalling along the edge of a beam. The difference with this repair is that the CIP concrete portion on top of the precast beams, as well as on top of the beams, is fully removed and additional steel is added to the area on top of the beams. The fiber board bearing material is replaced with non-shrink Portland cement grout or epoxy as discussed previously to provide positive bearing. This repair is extended longitudinally throughout the length of the span.

The durability of the modified M1 repair and the condition of the deck area around it depends on the following parameters:

1. Time period between spall, spall repair, and M1 repair,
2. Possible internal damage to the panel induced from previous stages,
3. Possible internal damage to the panel induced from removal of CIP concrete,
4. Bonding between the old concrete and the repair material,
5. Stress redistribution to adjacent areas (after removal of the damaged CIP concrete that deck region is no longer transferring shear to the supports, so that shear is redistributed to the transverse edges of the repair),
6. Repair material,
7. Presence of panel shear connectors embedded in the M1 repair,

8. Time interval between repair and passage of traffic.
9. And finally the most important parameter, removal of the fiberboard and its replacement by non shrink epoxy.

This procedure is labor intensive, costly and causes interruption to traffic. However, with the exception of full bridge bay replacement, it is the most effective repair method because it fills the spalled area under the wheel lines with sound incompressible material and provides positive bearing for the deck panels. Nevertheless, even these repairs can end up with deficiencies such as longitudinal cracks within them or adjacent to them.

None of the failures reported in Tables 2.1 and 3.1 had M1 and grout packing as the method of repair.

B.7 M2 Repair

Although the M2 repair method was not encountered in any of the authors inspections or failures listed in Tables 2.1 and 3.1, it deserves to be mentioned because it was prescribed as a good method of repair by the FDOT in the 1980s [B.3]. The M2 repair, shown in Figure B.7, is used to fix the problem of cracking and spalling along the transverse joints of the precast panel. The unsound material is removed approximately six inches on each side of the transverse joint and an inverted T-beam is formed with the bottom of the precast panel sitting on the flange of the inverted T-beam. The flange of the T-beam is required to be at least 24 inches wide. The inverted T-beam needs to be provided with positive bearing on the girders [B.3].

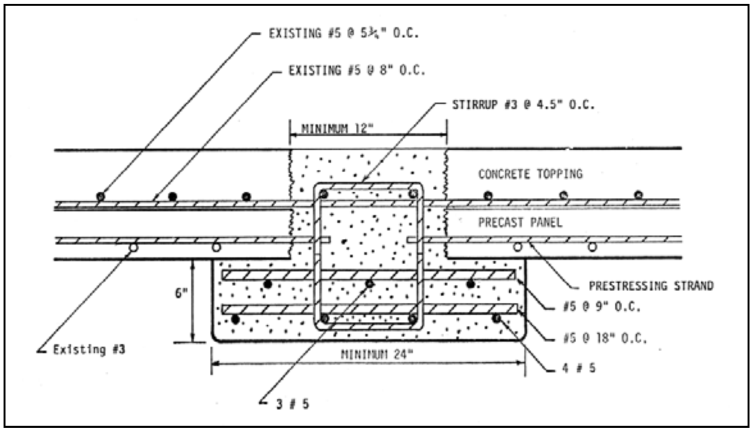

Figure B.7 M2 Repair Procedure [3]

The M2 repairs are costly relative to other repair methods and cause impact to the traveling public.

B.8 Full Depth Bay Replacement

Full depth bay replacement is the most effective repair method for deficient precast deck panel bridges. In fact it is the directive of the DSMO to use this method for all permanent repairs. At a minimum, it is done in a bay (the transverse distance between two beams) and throughout the length of the span. Sometimes the entire deck on the span or all bays is replaced with full depth CIP concrete.

When only a bay is replaced, the CIP concrete and precast panel is demolished, leaving only the reinforcing steel grid which was within the CIP section for continuity, then removed using jack hammers. A new bottom steel mat is designed as shown in Figure B.8 [B.5] and placed as an alternate to the precast panel.

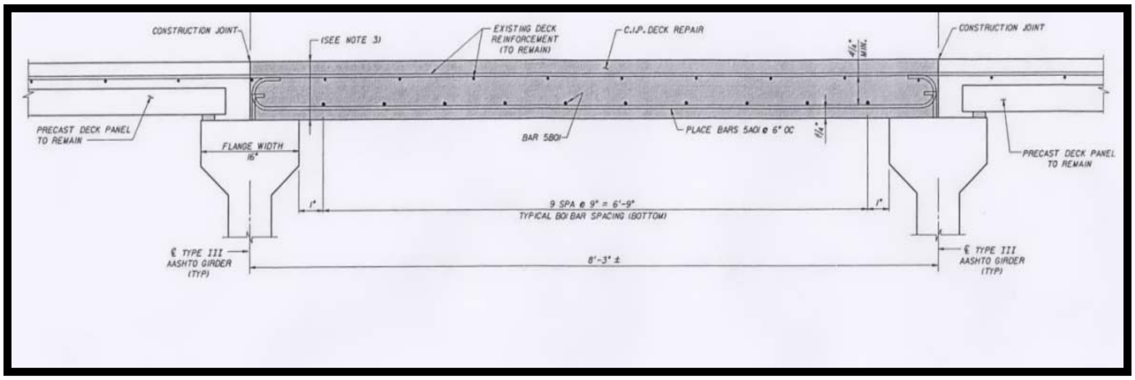

Figure B.8 Full Depth Bay Replacement Detail

A standard compression test is performed on 6 in. x 12 in. test cylinders at 24 hours, 48 hours, and 72 hours after the concrete is poured and finished. Although there is always pressure from the public and elected officials, the bridge is not opened to traffic until the minimum required compressive strength per design calculations is attained. After the concrete has gained the required strength, the bridge or repaired area is opened to traffic.

The conventional bay replacement is the most expensive repair method and causes significant interruption to the traveling public. However, it is the most effective repair method because it addresses the root cause of the problem which is the elimination of the vertical and longitudinal separation between the precast deck panel and CIP surfaces. None of the failures reported in Tables 2.1 and 3.1 occurred on decks which had been repaired by full depth bay replacement.

It is difficult to apply this method to high ADT highways due to the extended lane closures required to accommodate concrete curing time. This was the case on Bridge No.

100436, I-75 over E. Broadway Ave. and CSX Railroad. It is the last recorded failure as shown in Table 3.1. This was temporarily repaired and shored in September 2007. Traffic analysis and lane closure calculations indicate that the I-75 in this area can only have nighttime lane closures due to high ADT conditions.

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