UNIVERSITY OF SOUTH FLORIDA
Civil & Environmental Engineering

REPLACEMENT PRIORITIZATION OF PRECAST DECK PANEL BRIDGES

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16. Abstract

Precast deck panel bridges have a long history of poor performance in Florida. A spate of recent localized failures on major highways, led to a decision by the Department to replace selected bridges on I-75 in Districts 1 and 7 by full-depth, cast-in-place concrete slab over 10 years. The goal of this study was to develop a strategy that could assist in prioritizing this replacement.

A progressive degradation model was developed from a careful analysis of localized failures, on-site forensic investigation of deck panel bridges during their replacement, review of historical inspection data and finite element analysis. This model was subsequently integrated in PANEL - custom software written for this project. A special database containing inspection records for all precast deck panel bridges in Districts 1 and 7 extending over 20 years in electronic form was created for PANEL. This allowed PANEL to automate the prioritization process.

PANEL permits users to specify weighting factors for parameters such as safety, importance and cost. This information is then used to create lists that rank the order in which the replacement is to be carried out. Weighting factors used in this report for ranking were calibrated using the latest inspection data. As the database can be easily updated to include new inspection information and photographs, PANEL provides a dynamic resource that can be used by FDOT to review and revise its prioritization strategy in the future to take into consideration the latest available information.

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REPLACEMENT PRIORITIZATION OF PRECAST DECK PANEL BRIDGES

A Final Report on a Research Project

Prepared in Cooperation with the
State of Florida Department of Transportation and the
US Department of Transportation

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DISCLAIMER

CONVERSION FACTORS, US CUSTOMARY TO METRIC UNITS

Multiply	*by*	*to obtain*
inch	25.4	mm
foot	0.3048	meter
square inches	645	square mm
cubic yard	0.765	cubic meter
pound (lb)	4.448	newtons
kip (1000 lb)	4.448	kilo newton (kN)
newton	0.2248	pound
kip/ft	14.59	kN/meter
pound/in^2	0.0069	MPa
kip/in^2	6.895	MPa
MPa	0.145	ksi
ft-kip	1.356	kN-m
in-kip	0.113	kN-m
kN-m	0.7375	ft-kip

PREFACE

The investigation reported was funded by a contract awarded to the University of South Florida, Tampa by the Florida Department of Transportation (FDOT). The assistance and guidance of Mr. Jose Garcia and Mr. Steve Womble from the FDOT throughout the project is gratefully acknowledged.

The project benefited enormously from the input received during quarterly progress meetings held over the duration of the study. Special thanks are due to Dr. Jose Danon, Mr. Jose Rodriguez (District 7), Mr. Alan Hyman, Mr. Ron Meade, Mr. Todd Hammerle and Mr. Seta Koroitamudu (District 5). We are also grateful for the input provided by Mr. Keith Campbell (District 2) and Mr. Roberto M. Rubio (District 4).

This study could not have been completed without the assistance of the FDOT District 1 & 7 Structures and Facilities Office. We especially thank Mr. James Jacobsen, Mr. Manuel H. Luna, Mr. Alfonso Espitia and Mr. Richard Semple for helping in the compilation of over 1200 bridge inspection reports and plans for over 120 bridges.

The forensic investigations reported were greatly facilitated because of the cooperation and unconditional assistance of the deck replacement contractors. We thank Mr. Jovan A. Zepcevski (Zep Construction, Inc.), Ms. Arlene Barrios (FDOT) and Mr. Mark DeLorenzo (AIM Engineering & Surveying).

We thank Mr. Oscar Gomez-Fuentes, a graduate student in the Civil and Environmental Engineering, for his assistance. Mr. Nick Meloy contributed to the study as part of USF's College of Engineering's Research Experiences for Undergraduate (REU) program. We also thank Mr. Rusty Davison, Physics Teacher, King High School who assisted summers as part of the College's Research Experience for Teachers (RET) program.

EXECUTIVE SUMMARY

Precast deck panel bridges have an unfortunate history of poor performance in Florida. Recently, as a result of a spate of localized failures on major highways, a decision was taken to replace selected I-75 deck panel bridges in Districts 1 and 7 by full-depth, cast-in-place concrete slabs. The goal of this study was to develop a strategy that could be used to prioritize this replacement. In the study, however, all precast deck panel bridges in these Districts were considered.

Following compilation of inspection records spanning more than 20 years for over 120 bridges, a detailed analysis was carried out on the five precast deck panel bridges that experienced localized failures. The results of this investigation indicated that the common denominator in all cases was failure of re-repairs. Biennial inspection data was not found to be useful. However, monthly inspections that tracked the progression of deterioration were more effective in anticipating failures.

Three dimensional finite element analyses were conducted to determine if there were certain parameters such as type of girder, panel dimensions that made certain configurations particularly vulnerable to deterioration. However, in the absence of reliable information on the as-built structures, this analysis could not provide definitive results. The analysis was more successful in predicting that long term creep and shrinkage effects could lead to separation of the panel from the cast-in-place (CIP) slab at the vertical interface. This finding was subsequently corroborated in on-site forensic studies.

Eight precast deck panel bridges in various states of disrepair were inspected on-site before and during demolition of the deck. This study provided important insights into their performance. The cause of longitudinal reflective cracking was found to be due to separation of the prestressed panel from the CIP slab. The role of fiberboard bearings was clearly demonstrated. The fiberboard's inability to support panels resulted in non-composite action under shear that led to localized punching shear failures. Simplified code-based analysis indicated that such failure could occur at loads below design wheel loads. The study found that three of the eight bridges in good condition had panels that were partly supported by fiberboard and partly by grout – the prescription for successful performance in Texas.

Based on the information obtained from the above studies, a progressive degradation model was developed. This was integrated into computer software, PANEL that was specially developed for the study. PANEL accesses a database that was created to provide inspection records for all precast deck panel bridges in Districts 1 and 7 that extended over 20 years. This database can be updated by the user to include the latest inspection data. The program allows users to assign weighting factors for parameters such as safety, importance and cost. This information is then used to create lists that rank the order in which the replacement is to be carried out. The report provides rankings that were calibrated using the latest available inspection data obtained by the USF team in 2004. Based on this information, 46 of 85 remaining precast deck panel bridges are recommended for replacement. The remaining 39 are in good condition. By updating the inspection database, FDOT can use PANEL to re-prioritize rankings over the 10 years it will take to replace all the panel deck bridges in Districts 1 and 7.

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1. INTRODUCTION

1.1 Introduction

Precast deck panel bridges were first used in the construction of highway bridges in Illinois in the early 1950's. This type of construction offers significant economies; the stay-in-place (SIP) panel combined with a cast-in-place (CIP) topping can considerably reduce construction time as field forming is only needed for the exterior girder overhangs (Fig. 1.1).

Florida has approximately 200 precast deck panel bridges, 127 of which are located in Districts 1 and 7 (includes 18 on the Crosstown Expressway) of the Florida Department of Transportation. Precast panel sizes vary with girder spacing but are typically 10 ft x 10 ft in plan and 3½ - 4 in. thick. In design, it is assumed that the panel acts compositely with the CIP reinforced concrete slab for resisting live loads.

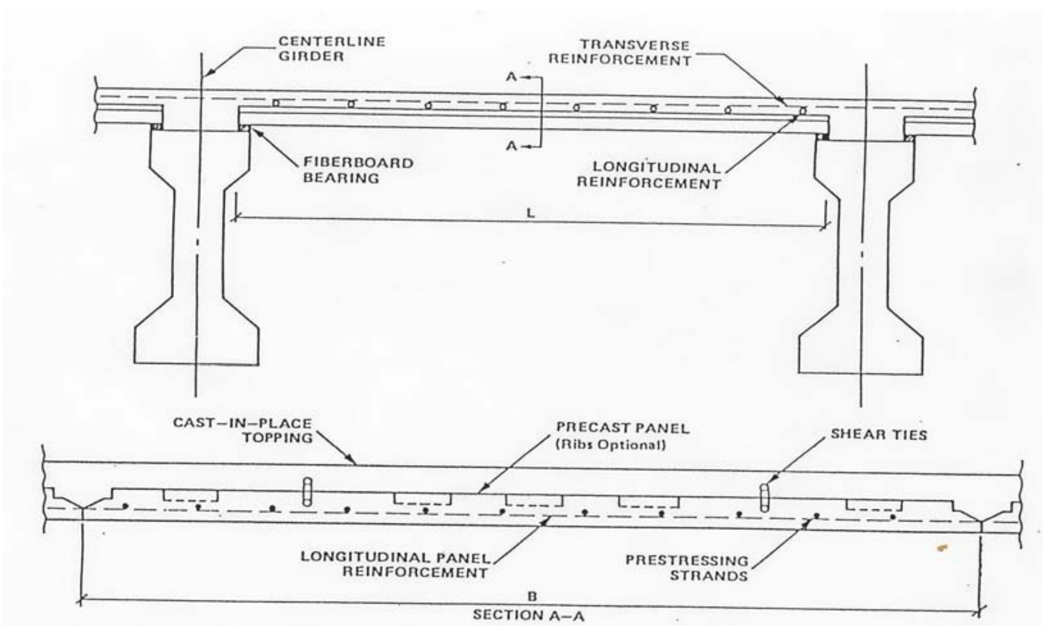


Figure 1.1 Fiberboard Supported Precast Panel Deck

Despite successful performance in other states, precast deck panel bridges have a long history of premature deterioration in Florida that has led to excessive maintenance and impacts to the traveling public. Previous research has attributed this to the use of flexible fiberboard supports that was used by contractors to simplify construction.

The District 1 & 7 Structures and Facilities Office have responded to numerous maintenance problems on deck panel bridge decks throughout the I-75 corridor in southwest Florida. Initially, the response was geared toward emergency situations in which a localized failure of the bridge deck resulted in lane closures. Over time, the District began a proactive approach to monitoring, early detection and repair to avoid disruptive emergency situations.

A program is underway to systematically replace selected deck panel bridges on I-75 in both Districts 1 and 7 by full-depth, CIP concrete decks. The short-term goal is to replace the decks of high ADT deck panel bridges with long-term plans to replace all precast deck panel bridges. To this end, the Department has allocated $78 million over 10 years for the replacement of selected deck panel bridges in Districts 1 and 7 focusing first on those that are severely deteriorated.

1.2.1 Objectives

The goal of this research project is to develop a simple, rational procedure that would assist in prioritizing the replacement of deck panel bridges in Districts 1 and 7.

In the original approach, it was envisaged that this goal would be met from three-dimensional finite element analysis with inspection records identifying damage progression. The actual model would be calibrated from laboratory testing and limited non-destructive field evaluation. However, a review of the literature in which DOTs across the country were contacted indicated that non-destructive evaluation using ground penetrating radar would not work for our situation. Moreover, in the absence of precise information on the as-built structures, finite element modeling could not provide results that could be used with any degree of confidence. In view of this, the method of approach was revised following discussions with FDOT engineers.

In the revised approach, instead of non-destructive testing more emphasis was placed on inspection data and on on-site forensic studies of deck panel bridges that were being replaced. The forensic studies provided data that allowed the development of a progressive failure model. This model was incorporated in customized software, PANEL that was developed for this study. PANEL provides a simple, speedy method for prioritizing deck panel bridges. More importantly, it is a dynamic resource since it has provisions that allow inspection records for all precast deck panel bridges in Districts 1 and 7 to be readily updated.

1.3 Organization of Report

This report is organized into nine chapters and five appendices that describe various aspects of the study. For convenience, all references cited are listed at the end of the respective chapters.

The collection of data on deck panel bridges and their performance was a crucial part of the investigation. **Chapter 2** provides an overview on the various sources that provided data for the study. **Chapter 3** presents an in-depth analysis of five localized deck panel failures that occurred from February 2000 to September 2002. **Chapter 4** provides results of forensic studies carried out on eight deck panel bridges that were replaced during June 2003 to August 2004. Based on the findings, a detailed degradation model was developed that was subsequently used by custom software PANEL in the prioritization. **Chapter 5** reports the findings of the finite element study. This presents results of a parametric study to investigate the relative importance of variables such as panel length, girder spacing, type of girder on likely performance. Additionally, it presents findings from an in-depth analysis to investigate the role of long term creep and shrinkage in the performance of panel deck bridges. **Chapter 6** provides results from the inspection of deck panel bridges carried out by the USF research team. Here a simple procedure was developed that allowed precise measurement of spall damage. This information was incorporated in plan drawings and used by the PANEL software for prioritizing replacement of deck panel bridges. **Chapter 7** provides a description of the PANEL software and the BRAILE input that was developed to convert information from inspection records on all deck panel bridges in Districts 1 and 7 into an electronic format. **Chapter 8** summarizes the results of the prioritization study. The principal findings and conclusions are presented in **Chapter 9**.

Five appendices supplement information presented in the above chapters. **Appendix A** provides code-based punching shear calculations. **Appendix B** contains results of a survey conducted to determine the experience of other state agencies with precast deck panel bridges. **Appendix C** provides detailed information on concrete cores taken in the forensic study. **Appendix D** provides information on an innovative pilot study conducted to automate bridge inspection using a camera mounted vehicle. **Appendix E** contains recent photographs of the precast deck panel bridges that are recommended for replacement in Chapter 9.

2. SOURCES OF INFORMATION

2.1 Introduction

The first step in the prioritization process was compilation of available data. This chapter provides an overview on the types of information collected and their sources. This is used in subsequent chapters to develop a degradation model that is integrated in PANEL software (Chapter 7). Information was collected for a total of 127 deck panel bridges - 74 bridges in District 1 and 53 in District 7 (including 18 on the Crosstown Expressway). The primary source of information was inspection reports spanning 20 years. Supplementary information was also obtained from the National Bridge Inventory and from reports prepared by consultants. Section 2.2 describes data collected from the National Bridge Inventory while Section 2.3 provides information on that obtained from FDOT's Inspection reports. Section 2.4 contains information on bridge plans obtained from FDOT while those from consultants are in Section 2.5. A summary is contained in Section 2.6.

2.2 National Bridge Inventory

The National Bridge Inventory (NBI) is a database developed by the US Federal Highway Administration that provides information on more than 600,000 bridges on public roads throughout the United States [2.1]. Fig. 2.1 shows a NBI query window.

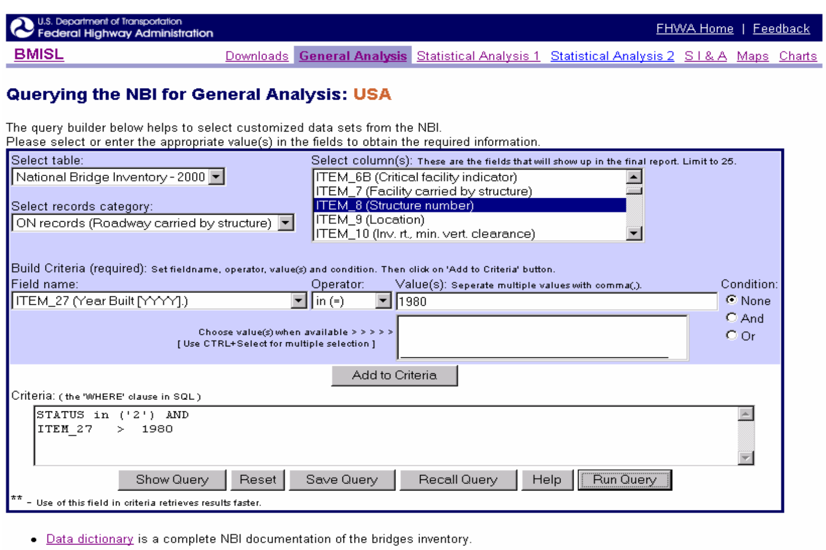


Figure 2.1 National Bridge Inventory Query Window

Data provided by NBI is in the form of a Microsoft Access file. This file contains 116 items of information on each bridge. A complete description of these items may be found in the FHWA's Recording and Coding Guide [2.2]. For the purposes of this study, only 20 items were required. The item numbers and their description are summarized in Table 2.1.

Table 2.1 NBI Data Items Used

Item No	Description
6	Features intersected
7	Facility carried by the structure
8	Structure number
16	Latitude
17	Longitude
27	Year built
28	Lanes on and under the structure
29	Average daily traffic
30	Year of average daily traffic
31	Design load
45	Number of spans in main unit
48	Length of maximum span
49	Structure length
58	Deck condition rating
90	Inspection date
91	Designated inspection frequency
107	Deck structure type
109	Average daily truck traffic
114	Future daily traffic
115	Year of future daily traffic

One of the most important items in the NBI database is the deck condition rating (item 58). This is the condition rating given by the bridge inspector based on the actual condition of the deck. Its definition is given in Table 2.2.

Table 2.2 Condition Rating Definition

Condition Rating	Description
9	**"EXCELLENT CONDITION"**
8	**"VERY GOOD CONDITION"** – No problems noted
7	**"GOOD CONDITION"** – Some minor problems
6	**"SATISFACTORY CONDITION"** – Structural elements show some minor deterioration.
5	**"FAIR CONDITION"** – All primary structural elements are sound but may have minor section loss, cracking, spalling or scour.
4	**"POOR CONDITION"** – Advanced section loss, deterioration, spalling or scour
3	**"SERIOUS CONDITION"** - Loss of section, deterioration of primary structural elements, fatigue or shear cracks in concrete may be present.
2	**"CRITICAL CONDITION"** – Advanced deterioration of primary structural elements
1	**"IMMINENT FAILURE CONDITION"** – major deterioration or section loss present in critical structural components
0	**"FAILED CONDITON"** – Out of service

For the data collected, the condition ratings varied from 8 “Very Good Condition” to 4 “Poor Condition”. Basically bridges with “Very Good Condition or Excellent Condition” are recently constructed bridges.

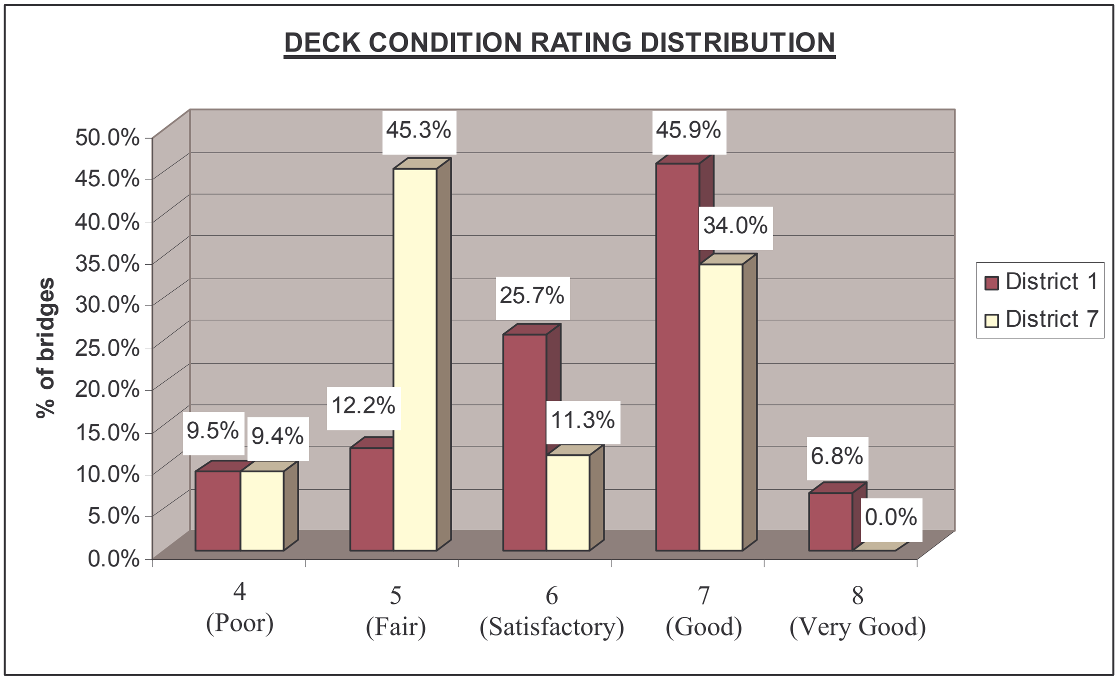


Figure 2.2 Deck Condition Rating Distribution (2002)

Fig. 2.2 shows a breakdown of the condition rating for precast deck panel bridges in Districts 1 and 7 (includes Crosstown Expressway) for 2002 based on information retrieved from NBI. Inspection of Fig. 2.2 shows that bridges in District 7 were generally in poorer condition compared to those in District 1 (about 55% rated fair or lower vs 22% similarly rated in District 1). Relevant information from the National Bridge Inventory database was stored in another database for use by the PANEL software.

2.3 Bridge Inspection Reports

2.3.1 Inspection Report Collection Procedure

Bridge inspection reports for Districts 1 and 7 were collected from the Bridge Maintenance office located in Tampa FL. This task was completed with the help of the Structures Maintenance Engineer at the Florida Department of Transportation at the time.

Due to the large volume of information that was required and the possible disruption that this could cause to the normal functioning of the archives division of the FDOT maintenance office where all the bridge documentation is kept, data collection was limited to Fridays, when many services are closed to the public.

Since historic inspection reports are not available in electronic form, it was necessary to physically review each inspection report, select the information to be used in the project, and make a photocopy of that information. Additionally, some photographs included in the reports were scanned.

In order to obtain information that could be used to analyze the progressive deterioration of the bridges over time, all inspection reports available for each bridge were collected. In most cases the initial inspection report was dated two years after bridge was first constructed. An average of ten bi-annual reports was collected for each bridge. With 127 bridges this meant a total of nearly 1200 reports. In addition, 290 photographs from the inspection reports were scanned and stored. This operation took nearly 6 months to complete.

After all reports had been collected, they were labeled, classified and archived at USF's project office in a way they could be easily consulted by the project's research staff. Subsequently, they were stored in a database that is used by the PANEL software for the automated prioritization scheme developed in this study.

2.3.2 Inspection Report Description

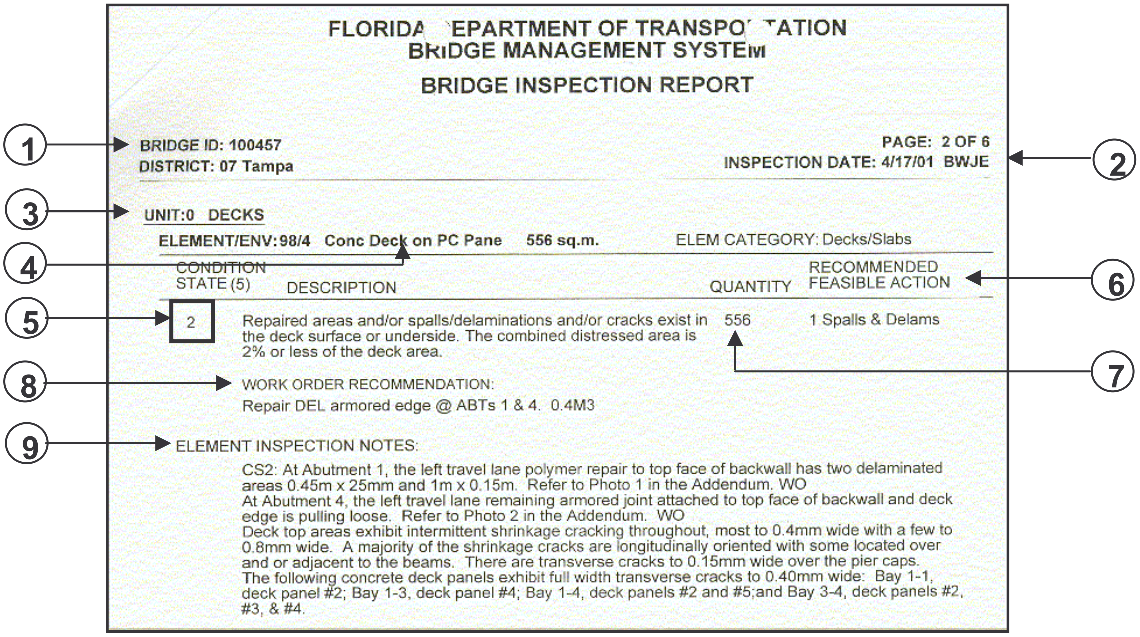
Bridge inspections are conducted every 2 years (unless more frequent inspections are required). In the inspection, the whole bridge is inspected, i.e. the superstructure, substructure and foundation (including scour analysis when required). For the purposes of this study, only relevant information relating to the deck was collected.

The information collected from the inspection reports include (1) actual and historical deck condition state, (2) historical deck condition ratings, (3) element inspection notes, (4) deficiency pictures, and (5) crack surveys (conducted only in 1983).

Over the years the format of the bridge inspection reports has changed several times. For the reports collected, three different formats could be identified. The information required for these different formats varied.

2.3.2.1 Latest Format (Used from 1999)

Fig. 2.3 illustrates the format used in the current inspection report. The information shown relates only to that describing inspection of the deck.


FLORIDA DEPARTMENT OF TRANSPORTATION
BRIDGE MANAGEMENT SYSTEM
BRIDGE INSPECTION REPORT

BRIDGE ID: 100457
DISTRICT: 07 Tampa

PAGE: 2 OF 6
INSPECTION DATE: 4/17/01 BWJE

UNIT:0 DECKS

ELEMENT/ENV: 98/4 Conc Deck on PC Pane 556 sq.m. ELEM CATEGORY: Decks/Slabs

CONDITION STATE (5)	DESCRIPTION	QUANTITY	RECOMMENDED FEASIBLE ACTION
2	Repaired areas and/or spalls/delaminations and/or cracks exist in the deck surface or underside. The combined distressed area is 2% or less of the deck area.	556	1 Spalls & Delams

WORK ORDER RECOMMENDATION:
Repair DEL armored edge @ ABTs 1 & 4. 0.4M3

ELEMENT INSPECTION NOTES:
CS2: At Abutment 1, the left travel lane polymer repair to top face of backwall has two delaminated areas 0.45m x 25mm and 1m x 0.15m. Refer to Photo 1 in the Addendum. WO
At Abutment 4, the left travel lane remaining armored joint attached to top face of backwall and deck edge is pulling loose. Refer to Photo 2 in the Addendum. WO
Deck top areas exhibit intermittent shrinkage cracking throughout, most to 0.4mm wide with a few to 0.8mm wide. A majority of the shrinkage cracks are longitudinally oriented with some located over and or adjacent to the beams. There are transverse cracks to 0.15mm wide over the pier caps.
The following concrete deck panels exhibit full width transverse cracks to 0.40mm wide: Bay 1-1, deck panel #2; Bay 1-3, deck panel #4; Bay 1-4, deck panels #2 and #5;and Bay 3-4, deck panels #2, #3, & #4.

Figure 2.3 Current Format in FDOT Bridge Inspection Report

As can be seen in Fig. 2.3, the information given in this type of inspection report consists of: (1) Bridge number, (2) Inspection date, (3) Unit (deck), (4) Element (Concrete deck in precast deck panel), (5) Condition state value (see Table 2.3), (6) Recommended feasible action, (7) Element quantity, (8) Work order recommendation, and, (9) Element inspection notes and photo addendum.

In this new inspection format the deck condition state is qualified using five standard condition state descriptions. The condition state definition is based on the approximate percentage of the distressed area relative to the total deck area. The different condition state descriptions are summarized in Table 2.3.

Table 2.3 Deck Condition State Definition [2.3]

Condition State	Definition
1	The surface and underside of the deck has no repaired areas, there are no spalls / delaminations in the deck surface or underside and the only cracking is superficial.
2	Repaired areas and/or spalls/delaminations and/or cracks exist in the deck surface or underside. The combined distress area is 2% or less of the deck area.
3	Repaired areas and/or spalls/delaminations and/or cracks exist in the deck surface or underside. The combined distress area is more than 2% but less than 10% of the total deck area.
4	Repaired areas and/or spalls/delaminations and/or cracks exist in the deck surface or underside. The combined distress area is more than 10% but less than 25% of the total deck area.
5	Repaired areas and/or spalls/delaminations and/or cracks exist in the deck surface or underside. The combined distress area is more than 25% of the total deck area.

A bridge inspector has to assign a condition state that best reflects the actual condition of the deck. Notice that these condition states do not provide much detail as to the real condition of the deck.

The element inspection notes allow the bridge inspector to provide more details on the condition of the deck. The inspection notes are basically provided when the inspector notices a significant deficiency and describes the type, characteristics and approximate location. Reference is also made to the photo addendum if pictures are available.

Notice that in this inspection format the description of the deck panel element includes both the top deck surface and the underside in the same inspection item (item 4 in Fig. 2.3). In many cases it is difficult to determine whether the deficiency is on the top deck surface or the underside, especially when the report does not include element inspection notes.

2.3.2.2 Report Format 1994-98

The inspection reports used during this period consisted of 5 sections. The first section is "*Report Identification*". Here all information used to identify the report is provided such as bridge number, bridge location, inspection date, type of inspection. The second part is the "*Condensed Inspection Report*". In this section the NCR (Numerical Condition Rating) for each bridge component is given. This has the same meaning as the ones in NBI summarized in Table 2.2.

Bridge and Inspection Info

Bridge No.: 130112 Location: 1.0 km East of US-41
County Section No.: 13175 Inspection Date: 10-06-97
State Road No.: 93 Inspector: D.R. Geiger
US Road No.: I-275 Mile Post No.: 0.910

B. COMPREHENSIVE REPORT OF DEFICIENCIES

Deck (Top & Underside)

G1.01 DECK(TOP)

All of the spans contain Class 1-2 longitudinal cracks primarily over the beams. Most of the spans also contain Class 1-2 transverse cracks and Class 1 map cracking. There are numerous areas in the deck which have been repaired and are cracking. There is a spall approximately 200mm x 150mm x 50mm (no steel) over bent 16. Also span 11 has several minor areas of exposed steel in the left shoulder. This exposure is due to lack of concrete cover and requires no repair.

G1.02 DECK(UNDERSIDE)

The majority of the precast deck panels on the steel beam spans 1-3 and 15-17 contain Class 1-2 transverse cracks. There are three panels in span 3 which were damaged (spalled at corners and along joint) while repairing deck spalls above. The expansion material placed on the beams for the deck panels to rest on is deteriorated and rotting where the deck surface has cracked allowing water to penetrate.

G1.03 EXPANSION JOINTS

The compression seal at bent 10 has a 50mm opening and is loose and allowing dirt and debris to enter the joint.

G2.02 BEAMS

The majority of the concrete beams contain Class 1 diagonal cracks at the interface of the top flange and web at each end of the beam.

Figure 2.4 FDOT Bridge Inspection Report Format (1994-98)

The third section of the report is the "*Comprehensive Report of Deficiencies*" or part B. As we can see in Fig. 2.4, this section provides a detailed report of the deficiencies found in each element of the bridge. Notice that in the case of the deck the information is sub-divided as 'deck top' or 'deck underside'.

The fourth section of the report provides an evaluation of previous corrective action. Here, a short description of the performance of previous repairs on any of the bridge elements is given.

The last part of the report is the "*Required Maintenance Repair and Rehabilitation*". The content of this section is based on the findings of the previous sections of the report. In most cases this section is written by a committee of structural engineers and revised by the Structures and Maintenance Engineer.

2.3.2.3 Report Format 1982-1992

This was the oldest type of inspection report found among the reports collected. Basically this report gives the same information provided as the one described previously, the only difference being the way the information is provided (see Fig. 2.5).

BRIDGE NUMBER 130112

INSPECTION DATE 01-14-85

B. COMPREHENSIVE REPORT

OF DEFICIENCIES

Element No. or Nos.

This structure has been found to be located in an class II corrosion area. Any deficiencies noted below will be made with this location taken in consideration. See sheet 4 for details.

NOTE: For crack class definitions see page 3.

Element Nos.

Element No. 3.1 - Deck (Top) - The deck top contains typical class I transverse cracks throughout all spans. Class I longitudinal cracks are located along the outside edge of the beams.

Element No. 3.3 - Joints (Expansion) - The expansion joints at Piers 10, 14, and 17 have loose elastometric compression seals. The worst joint, Pier 10 has an opening between the armor of 4½", the loose seal measures only 3 3/4". Other joints (Piers 14 & 17) have 1/8" gaps between armor and the sealing material.

Figure 2.5 FDOT Bridge Inspection Report Format (1982-1992)

2.3.3 Summary

In general, the information provided in the inspection reports includes an assessment of the deck and when any major sign of deterioration is found, a brief description of the damage along with its size and approximate location. Based on the structural condition, the bridge inspector assigns a condition rating number to the bridge deck which is also listed in the NBI database.

Table 2.4 compares information presented in the two formats. Often the format and the information presented in two consecutive inspection reports vary depending upon the inspector's understanding of the observed deficiency of the bridge deck.

Table 2.4 Comparison of Old and New Format

Characteristic	Old Format	New Format
Year	1981-1998	1999-till date
Definition of cracks/spalls	Crack class identifies width	Actual size mentioned
Condition rating (Deck)	No	Yes.
**Frequency of reports*	Two years	Two years
Units	US Customary	Metric
Deck deficiencies top and underside	Mentioned separately, under the heading "Deck Top" and "Deck Underside"	Mentioned together under the heading "Concrete Deck on PC Panel"
Joints nomenclature	Expansion joints	Pourable joint seal and Compression joint seal
Information	Detailed and good	Sometimes detailed, fairly good
Sketches of deficiency	Provided	Sometimes provided

**In special cases, inspections are carried out more frequently*

2.3.4 Monthly Inspection Reports

In addition to the bi-annual inspection reports, monthly inspection reports on selected deck panel bridges were also available. This was undertaken by FDOT to closely monitor the condition of certain bridges to reduce the risk of sudden localized deck failure. This was intended for internal use by FDOT.

Based on the information collected from these inspections, FDOT kept track of the progression of existing deficiencies so that appropriate repairs were conducted in a timely manner. Fig. 2.6 provides a sample monthly inspection report. Note that it is in a spreadsheet format; the same report covers several bridges. The most significant deficiencies are highlighted in yellow.

Dstrict 7 Deck Panel Bridges I-75/I-275/I-175/I-375						
Pr.	Bridge#	Span#	Deficiency	Feature Intersec.	Direction	Photo
			I-75			
	100351	1	Underside trans. Crks in panels Bay 1-1 Panel 2 trans. Crk 1/32" - NC - STABLE	I-75	overpass	N
		ALL	Several panels 1/64" wide trans. & diag. CKING - NC			
		3	Bay 3-3 Panel 2, 1/32" wide trans. Ck - NC			
*		4	Lane 2 , **8' X 3' REP area with (2) 6" x 3" SPL with RX - NC**		overpass	Jul-03
			Bay 4-4 Panel 3, trans. cracking. Efflo. 1/64" wide - NC			N
	100346	3	Two deck top repair Lane 3 - NC - STABLE	SR-674	SB	N
		1&4	Trans. Crks. Over deck panel joints in deck top - NC - STABLE.			N
*		4	**Lane 2 Long. Crks. 1/16" wide & 6" X 6" X 1/2" spl/del over BM 4-3 - NC**			Jul-03
*			**staining on web BM 4-3**			Jun-03
	100347	ALL	longitudinal cracks over and adjacent to the beams 1/32" wide - NC - STABLE	SR-674	NB	N
*		1	**Repair Lane 2, 10" x 10" x 1" SPL S. end of RP - NEW**		NB	Y
			RP Lane 2, 10' +/- from Pier 2 perimeter spalls 3" x 2" x 1" - **RP 8/03**			N
	100363	ALL	Trans. CRK over deck panel joints Long. Over beams up to 1/16" wide - NC - STABLE	CR-672	SB	N
		1	RP Lane 2, SPL/DEL repair 2' x 1' x 3" exp. rebar - RP 6/03 - STABLE			N
*			**Lane 2 14' from ABT 1 SPL @ RP 8" x 4" x 1" - NC**			Y
			Deck top repair Lane 3 - NC - STABLE			N
*		2	**Lane 2, 7' x 1' x 1/2" SPL/DEL RP 6' from P2 - NEW**			Y
			RP Lane 2, 3 epoxy patches 1' diameter - NC - STABLE			N
			RP Lane 3, 12.2' x 1.5' patch - N & S ends spl/del 3' x 1.5' area - RP 6/03 - STABLE			N
		3	RP Lane 2 two epoxy patches 1' diameter - NC - STABLE			N
			RP Lane 2 - NC - STABLE			N
			RP Lane 3 - NC - STABLE			N
			Lane 2 & 3 area of minor SPL/DEL 2' x 2', 12'+/- from Pier 4 - RP 6/03 - STABLE			N
		5	RP Lane 3, SPL @ N. end 18" x 8" x 1" no steel - RP 6/03 - STABLE			N
			Lane 4 @ ABT 6, 3.5' x 3" x 1" sp/del - RP 6/03 - STABLE			N
	100364	1	RP Lane 3 - NC - STABLE	CR-672	NB	N
			RP Lane 2, failed RP SPL 1' x1' x 2" - REP 6/03 - STABLE			N
		2	RP Lane 2 - NC - STABLE			N
		3	RP Lane 2, 10" X 10" X 1" sp/del N & S ends - RP 6/03 - STABLE			N
			Lane 2, 1' x 2" x 1" spl/del at Pier 4 - RP 6/03 - STABLE			N
		5	DK RP Lane 2, midspan 10" x 6" x 1/2" spl/del N. end of RP - **RP 8/03**			N
*			**RP Lane 3, 2' x 8" del. & patch material settled - NC**			Y
	100377	ALL	Shallow RPs lack of cover/trans. Cracks in deck panels 1/32" efflo. - NC -STABLE	I-75	overpass	N

Figure 2.6 Monthly Bridge Inspection Report

Inspections are performed by a team of two bridge inspectors. They assess the condition of the bridge deck by conducting a visual examination of the deck surface and its underside.

In order to identify progression of deficiencies, results of the current inspection are compared with those from previous inspection. Thus it was possible to determine if the deficiency was stable (and therefore not a threat to the integrity of the deck) or whether the deficiency increased between inspections that could have adverse consequences such as localized failure.

2.3.5 Limitations on Information Collected

Although inspection reports are the only means for obtaining information on the actual and historical condition of the bridge decks, there are important limitations that need to be recognized.

As may be noted, the actual deck condition is described in a qualitative manner that does not provide precise information on the deficiencies. The condition state only indicates an approximate percentage of the distressed deck area. More importantly, the assigned state, #2 (Table 2.3), is the same for almost all the bridges. It is only when major deficiencies occur that additional information is given in the Element Inspection Notes in one or two paragraphs. In some cases, a sketch giving the deficiency location is also included.

Another problem is the lack of consistency between the different biennial reports. As mentioned earlier, the format of the inspection report format has undergone changes since 1983 (the year when most of the deck panel bridges were built). The information reported is not the same. This means that the information available is incomplete for conducting a damage progression analysis.

More importantly, information on deck repairs is seldom available in the inspection reports. For example, if in one report a spall was identified and in the next report another spall was noted it is not possible to determine whether it was the same spall or whether the original spall had been repaired and this was a new spall.

The lack of precise information on the deck condition made it necessary for the USF research team to inspect all bridges (see Chapter 6).

2.4 Bridge Plans

Inspection reports are not intended to provide information on the deck geometry. This is needed to conduct numerical analyses and also to investigate the relationship between lane placement and deck deterioration.

Bridge plans were collected from FDOT District's 1 and 7 Bridge Maintenance Office. The plans were photocopied and stored at the USF project office for future use. Typical details are shown in Figs. 2.7 - 2.9.

The information obtained from the bridge plans includes (1) Bridge geometry (Plan & Elevation), (2) Construction data (Materials properties), (3) Deck structural details (Plan & Elevation), (4) Prestressed beam structural details and (5) Road geometry (Lane placement). Other information such as boring data, foundation layout, and pier structural details were available but were not needed in this project.

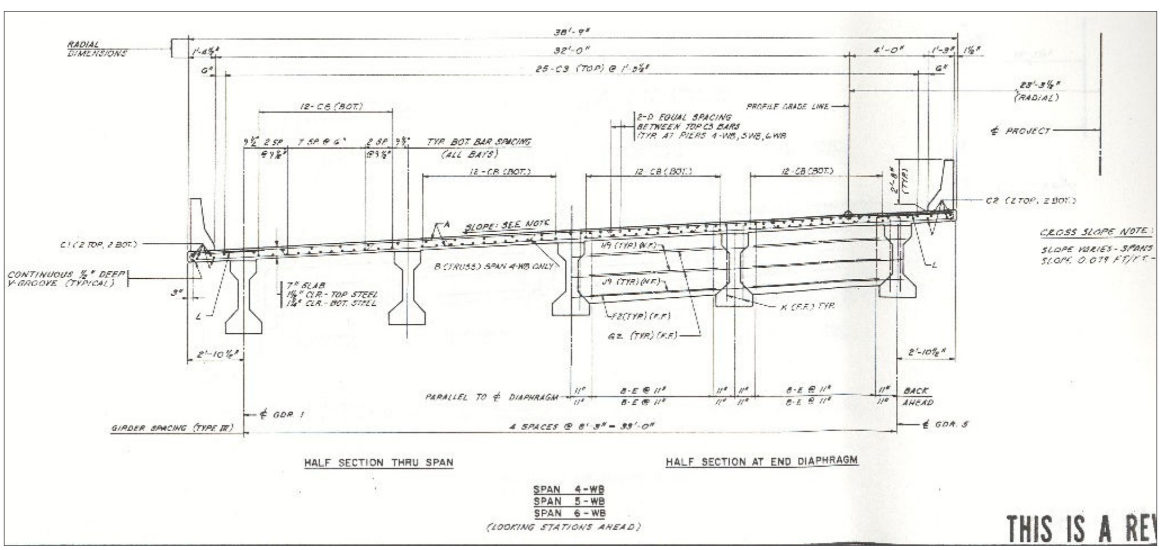


Figure 2.7 Typical Detail for Deck Panel Bridges

Unfortunately the deck structural information for almost all the bridges is based on the initial proposed design using a full-depth cast in place concrete deck. The as-built drawings were only available for a few cases.

For those few cases where the deck structural details reflect actual construction, information available consisted of (1) Precast panel layout, (2) Panel dimensions, (3) Panel and cast in place concrete materials properties, (4) Prestress data for the panels, (5) Construction notes and, (6) Fiberboard bearing details. Figs. 2.8-2.9 show typical information collected. In view of the lack of information, the same deck panel details are assumed for all deck panel bridges in Districts 1 and 7.

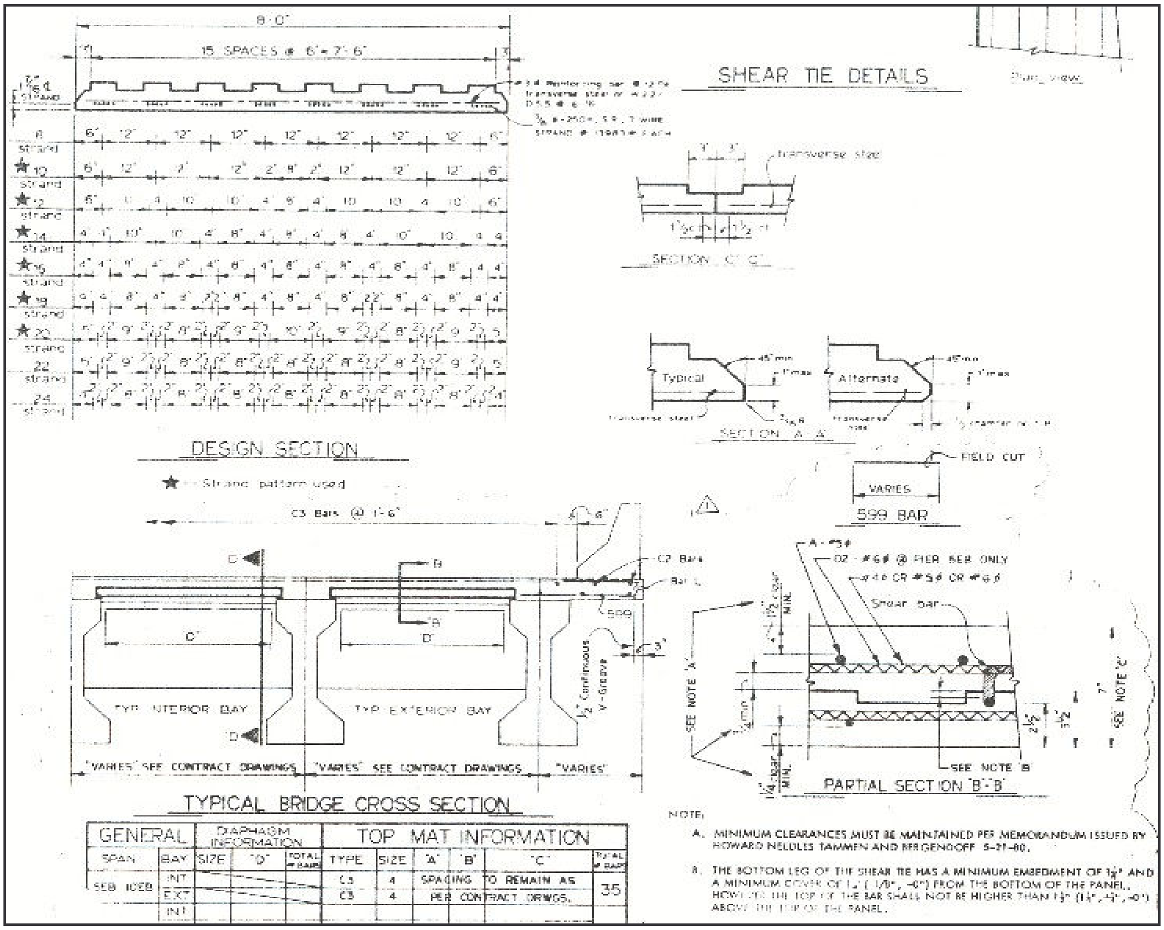


Figure 2.8 Precast Panel Construction Details

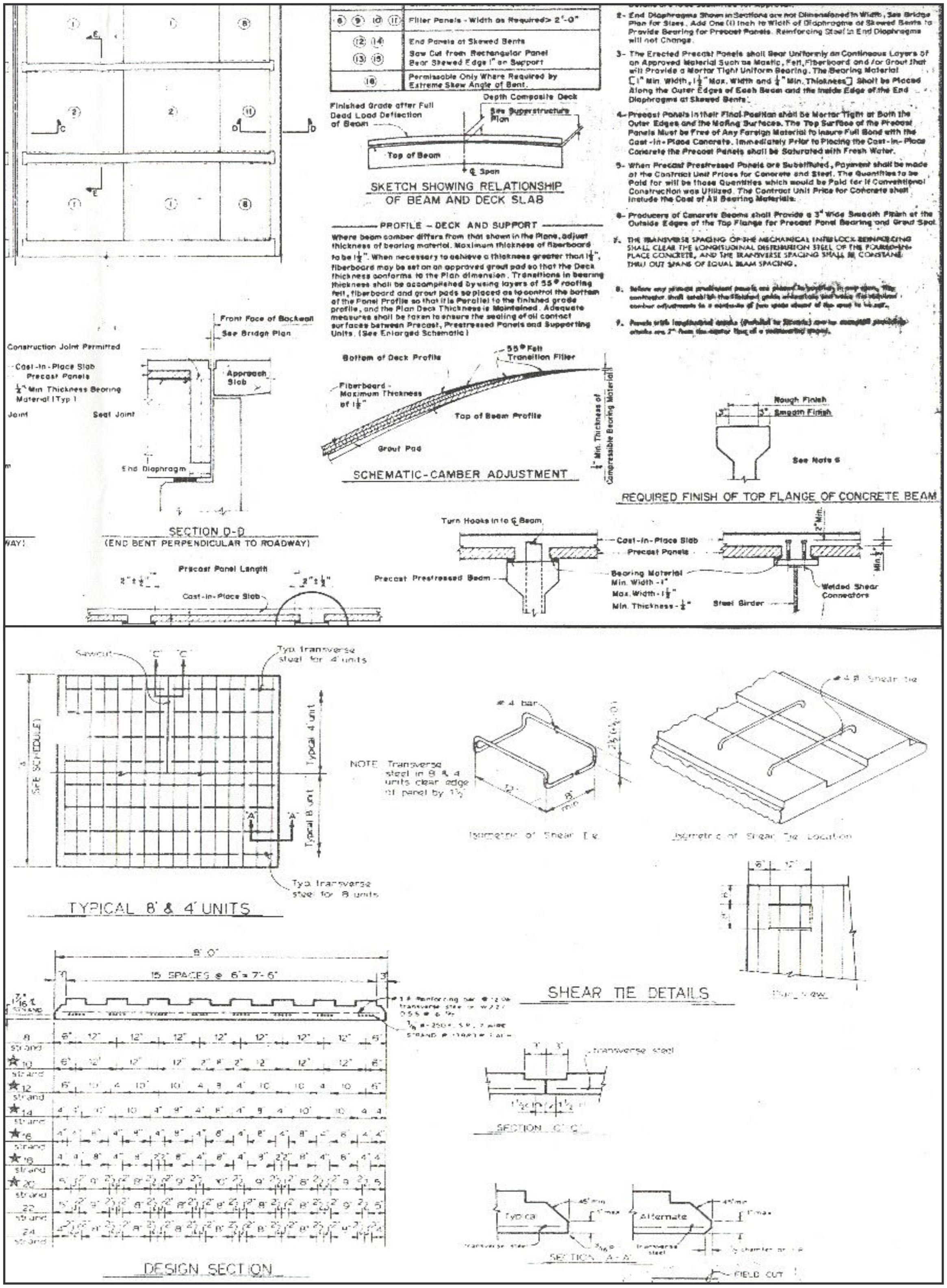


Figure 2.9 Precast Panel Construction Details (contd……)

2.5 Consultant Reports

Another important source of information for this research was reports on precast deck panel bridges, prepared by consulting firms working for FDOT. Among the reports obtained, the more relevant were: (1) *"Lee Roy Selmon CrossTown Expressway Deck Panel Bridge Assessment"* prepared by E.C. Driver & Associates [2.4], and (2)"*Bridge Emergency Response Reports*" prepared by E.C. Driver & Associates for the FDOT. [2.5 - 2.7].

A summary of the information obtained from the first of these reports is presented in Section 2.5.1. Reference to the remaining three reports is made in the next chapter that analyzes five localized panel deck failures.

2.5.1 Crosstown Expressway Assessment Report

This report [2.4] prepared by E.C Driver & Associates completed in January 2001, was required as part of the Crosstown Expressway reversible lanes project. This project required widening of six of eighteen existing deck panel bridges.

The deck surface of each bridge was visually inspected along with the underside. The location of cracks or spalls in the panels was noted. Surface distress including spalls, cracking and previous repairs were also noted. A description of the condition of each bridge along with a graphical representation of the findings was included in this report. A sample is shown in Fig. 2.10.

It may be seen from Fig. 2.10, that all the deficiencies in the bridge deck were identified and located over a deck layout, along with previous repairs.

In general the inspections showed that the bridges were in good condition. Most of the deck surfaces exhibited some degree of longitudinal cracking in conjunction of normal shrinkage cracking but the cracks were for the most part hairline cracks. Very few of the panels contained transverse cracks. These cracks were also for the most part hairline cracks.

The bridge deck assessment revealed that this type of construction has inherent design flaws that in some cases can lead to systematic deterioration of bridge decks ultimately resulting in localized deck failures.

Conclusions from this study indicated that there was no compelling reason why these bridges could not be widened. However, it was important to continue with frequent bridge inspection and deck repairs when needed, in order to reduce the probability of sudden localized deck failure.

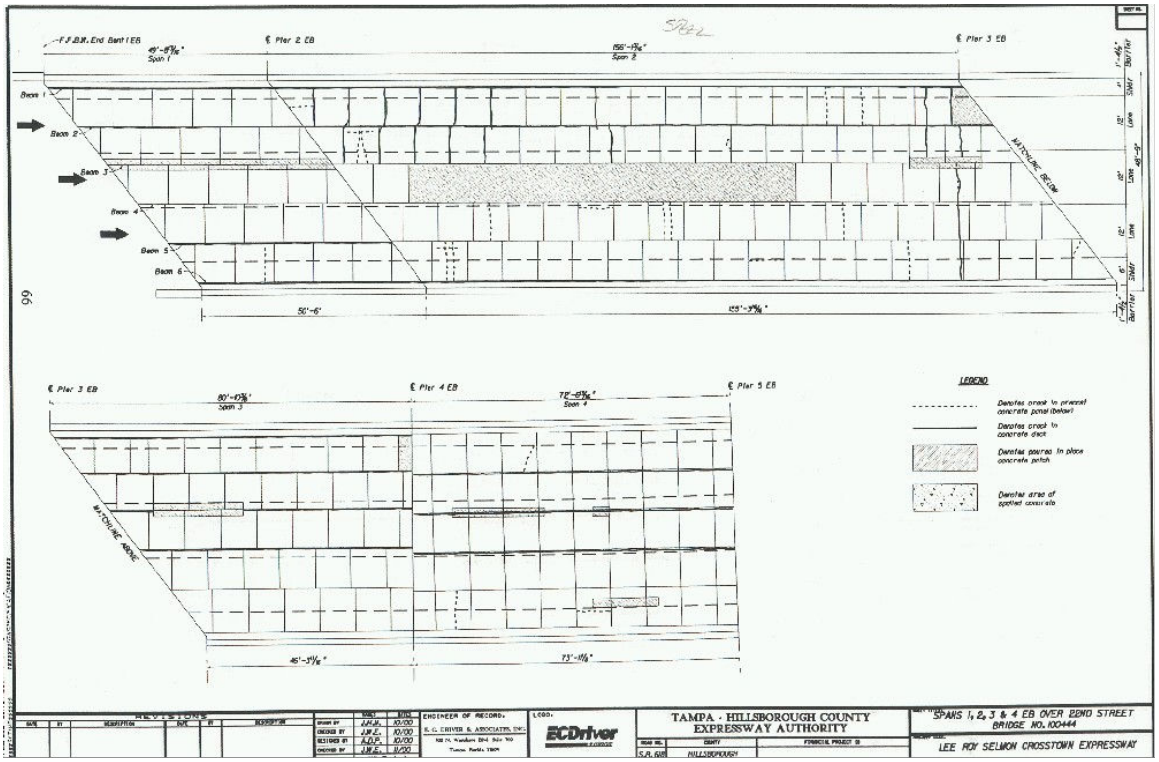


Figure 2.10 Deck Deficiency Survey in Crosstown Expressways [2.4].

2.6 Summary and Conclusions

- An average of ten biennial reports were collected for each bridge (construction date – to 2002), for an approximate total of 1200 reports. In addition, 290 inspection photographs were scanned and stored. Subsequently, they were stored in a database that is used by the PANEL software, and used to store historic information about the condition of these bridges since its construction date.

- Due to limitations in the FDOT biennial reports (see 2.3.5) it was decided to conduct a detailed deck inspections of all the deck panel bridges of Districts 1 and 7 (excluding Crosstown). This new inspection conducted by the USF research team provided detailed and accurate information used in the prioritization model.

- Inspection of bridge plans of all the deck panel bridges, indicated that almost all the bridges had construction details corresponding to a full depth cast in place concrete deck. Only a few bridges located on the Crosstown Expressway provided details of deck panel construction. (See 2.4).

References

2.1 U.S. Department of Transportation, Federal Highway Administration. (2003). National Bridge Inventory.

2.2 Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridge, Report No. FHWA PD-96-001, U.S. Department of Transportation, Federal Highway Administration.

2.3 Bridge Inspectors Field Guide, Structural Elements, Florida Department of Transportation, March 13, 2000.

2.4 "Lee Roy Selmon CrossTown Expressway Deck Panel Bridge Assessment" prepared by E.C. Driver & Associates, January 5, 2001.

2.5 Englert, J. (2000). Emergency Response Letter to Mr. Pepe Garcia dated Feb. 16, E. C. Driver and Associates, Tampa, FL.

2.6 Englert, J. (2000). Emergency Response Letter to Mr. Hamid Kashani on Bridge No 170086 dated Nov. 29, p.8. E. C. Driver and Associates, Tampa, FL.

2.7 Englert, J. (2000). Emergency Response Letter to Mr. Pepe Garcia dated Dec. 21, E. C. Driver and Associates, Tampa, FL.

3. LOCALIZED FAILURES

3.1 Introduction

Between 2000 and 2003, five localized failures occurred in precast deck panel bridges in District 1 & 7 (see Table 3.1. This chapter summarizes relevant information relating to these failures with the intent of identifying underlying trends, if any, for subsequent use in developing a rational prioritization scheme.

Table 3.1 Localized Deck Failures

Bridge #	District	Failure Date	Bridge Location
170146	1	2/12/2000	Sarasota, I-75 NB Over Bee Ridge Rd
170086	1	11/27/2000	Sarasota, I-75 NB Over Clark Rd
170085	1	12/20/2000	Sarasota, I-75 SB Over Clark Rd
100332	7	10/02/2002	Tampa, Crosstown Viaduct WB Span 38
100332	7	9/05/2002	Tampa, Crosstown Viaduct WB Span 70

In the following sections descriptions and analyses of each localized failure are presented in Sections 3.2-3.6 in the same order as their listing in Table 3.1. A summary of the principal findings is included in Section 3.7.

3.2 I-75 North Bound Over Bee Ridge Road, Bridge #170146

This 3-span bridge located in Sarasota, FL was built in 1981 and was 19 years old when it failed in February 2000. It has two 36 ft secondary spans (*span 1, span 3*) and a 118 ft 8 in. main span (*span 2*) to make the total bridge length 190 ft 8 in. The shorter spans were built using two AASHTO Type IV girders on the outside and five AASHTO Type II girders on the inside all spaced 8 ft 10 in. apart. In the main span, fifteen AASHTO Type IV girders are spaced at 4 ft 4 1/4 in. or 4 ft 4 5/16 in. on centers as shown in Fig. 3.1.

The deck has a 7 in. thick concrete slab with the precast panel component being either 2-½ in. or 3-½ in. (at the rib-section) thick as shown in Fig. 3.2. This panel thickness is typical for all the deck panel bridges in this area. The specified compressive strength of concrete for the precast panel was 5,000 psi. It was 3,000 psi for the cast in place concrete slab.

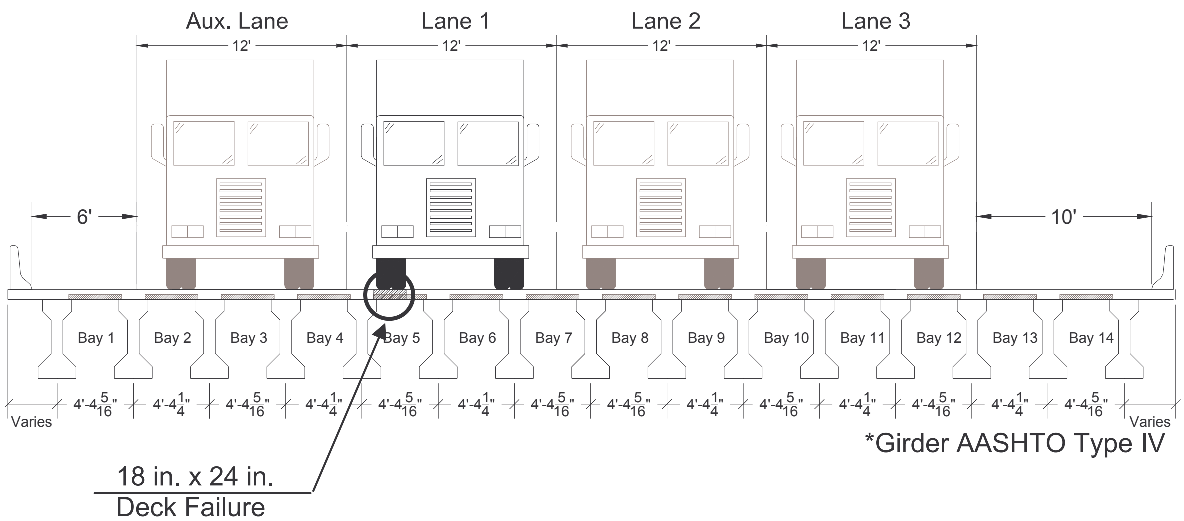


Figure 3.1 Cross Section View of Bridge #170146 – Main Span

The bridge has four 12 ft wide lanes, and 6 ft or 10 ft wide shoulders as shown in Fig. 3.1. There is an auxiliary lane that merges with traffic entering the interstate from Bee Ridge Road. The average daily traffic (ADT) in the bridge during the year 2000 was 34,000 [3.1]. Thirty percent of the ADT was truck traffic (ADTT). Details are summarized in Table 3.2.

Table 3.2 Bridge #170146

Bridge #170146 Characteristics	
Year Built	1981
Number of Spans	3
Lanes on Structure	4
ADT *	34,000
Percent Truck (ADTT)	30%
Deck Condition Rating (1999)	6 (Satisfactory)
Composite Slab Thickness	7 in.
Precast Panel Thickness	2-½ in (panel) 3-½ in (ribs)
Girder Type	AASHTO Type II and IV

* National Bridge Inventory (1999)

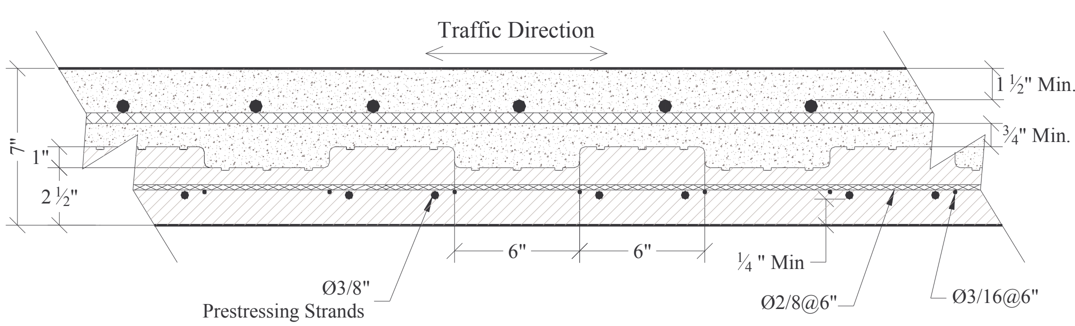


Figure 3.2 Composite Deck Section

3.2.1 Failure Details

Localized failure occurred suddenly in the main span on the morning of Saturday, February 12, 2000. A hole formed in a panel that was estimated to be about "*two feet square*" [3.2]. A newspaper account made it 3 square ft - 18 in. x 24 in. [3.3]. However, no photographs of the damage are available.

Fig. 3.3 shows the location of the failed panel taken from reference [3.4]. It was re-drawn to clarify details. Failure occurred in "*Span 2, Bay 10 at the edge of panel 13*" [3.2]. This location is also identified in Fig.3.1 as coinciding with the placement of the right truck wheel in the slow lane (lane 1) close to the face of a girder.

Ref. 3.2 also noted the following "*On the deck surface numerous asphalt and concrete type spall repairs had been performed over the years extending south from the hole about six more feet. From that point extending approximately fifteen additional feet, M-1 type repairs have been made. This consisted of asphaltic type material about 18 in. wide…"*

M1 repairs are the most common repair type in deck panel bridges in which the deteriorated section is removed by saw-cutting and replaced by high strength concrete or epoxy material.

3.2.1.1 Newspaper Account

In view of the limited information available, newspaper accounts of the failures were also reviewed. Two articles were printed in the local newspaper, *Sarasota Herald Tribune* [3.3, 3.5].

The first article [3.3] was published on February 13, 2000 with the headline "*Fallen asphalt closes lanes:* a *large pothole has developed again in the I-75 overpass at Bee Ridge*." The newspaper account stated "*No one was injured from the falling debris, but this is the second time in three months that a large pothole has developed in the overpass*"*…. FDOT crews last had problems with the overpass after a motorist saw a 18 in. hole in the south bound center lane in October*". No records of this 18 in. hole could be found.

A follow-up article [3.5] was published on February 15, 2000 with the headline "*FDOT will have I-75 hole fixed soon*". The article stated that "*Workers should be finished patching a hole in the northbound Interstate 75 overpass at Bee Ridge Road on Wednesday* [February 16]*, according to the Florida Department of Transportation"..* FDOT spokesman Marsha Burke stated "*It's old and is going to require maintenance. It's something that just happens with older bridges.*"

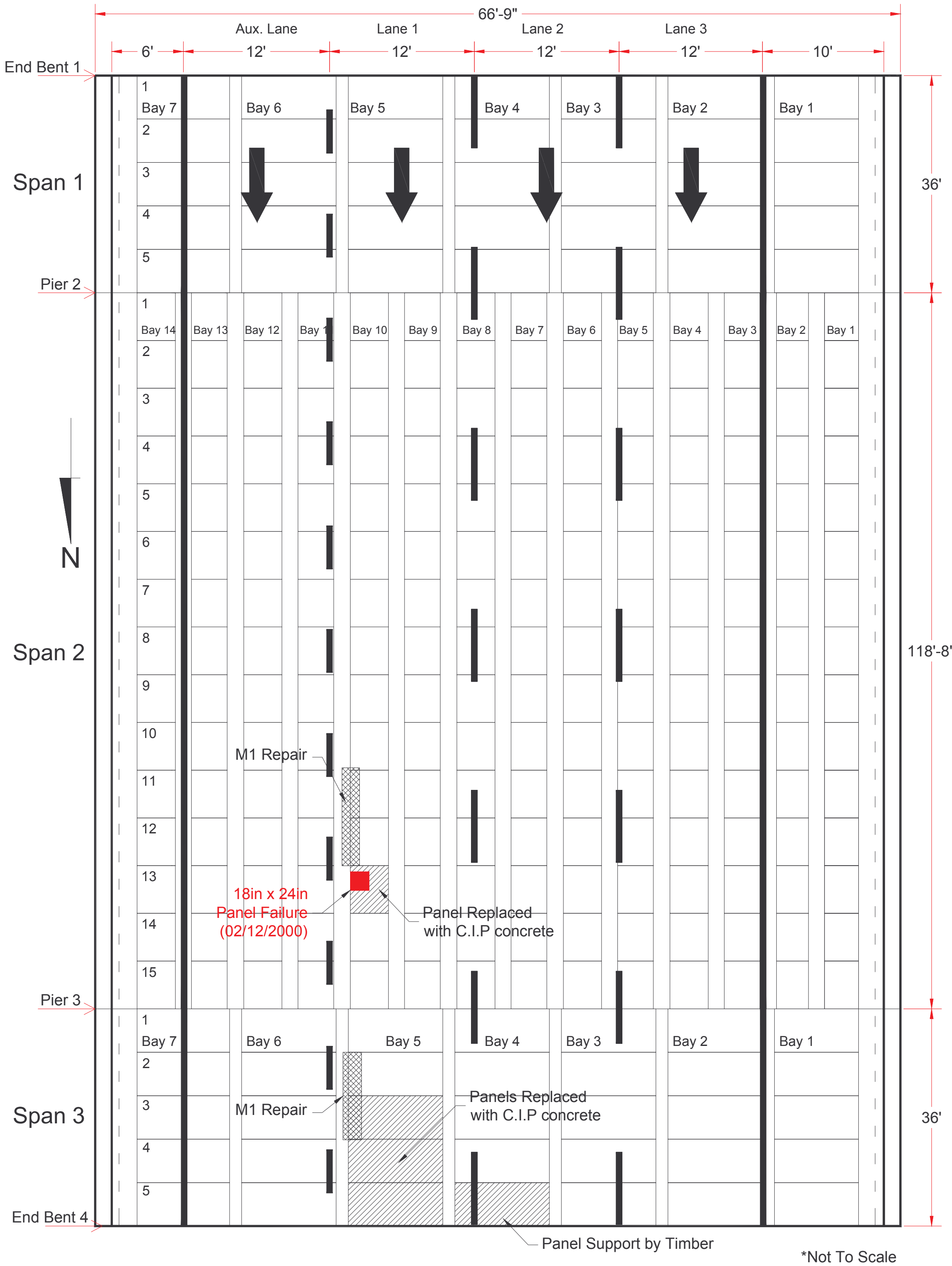


Figure 3.3 Location of Failed Panel, Bridge 170146 (I-75 NB) [3.4]

3.2.2 Analysis

As the goal of the project was to develop a rational replacement strategy, analyses were carried out to identify the likely cause of failure. The starting point of the investigations was a review of inspection reports and environmental factors. Additionally, a simplified code-based [3.6] punching shear analysis was carried out to provide a measure of the magnitude of the failure load. These are briefly described in Sections 3.2.2.1-3.2.2.3.

3.2.2.1 Inspection Reports

To help identify underlying trends, five consecutive inspection reports covering the period from 1992 to 1999 were reviewed. The final inspection in this sequence was carried out on November 24, 1999 less than 3 months before failure occurred on February 12, 2000. For authenticity, scanned excerpts from the relevant sections of the inspection report are included in Table 3.3 [3.7-3.11].

The earliest report (May '92) [3.7] notes the presence of Class 1 (0-1/64th in.) longitudinal cracks along inside girders. The bottom had "*occasional*" transverse cracks with efflorescence that had not changed since Jan 1984. Mention is made of spalls in span 3 adjacent to a previously patched area and span 2 (right travel lane where the failure occurred). This information is more or less repeated in the next two reports (Dec '94 and Dec '95) [3.8-3.9]. In the report prepared in Nov' 97 [3.10] dimensions of the spall in the right travel lane (6 ft 6 in. x 6 in.) are given. The inspector is also critical of the use of asphalt ("inappropriate material") for repair since it is "respalling around the edges".

Table 3.3 Excerpts from Inspection Reports (Bridge #170146)

	FDOT Bridge Inspection Report (Deck)
11/24/99	ELEMENT/ENV:98/3 Conc Deck on PC Pane 1187 sq.m. ELEM CATEGORY: Decks/Slabs CONDITION STATE (5) | DESCRIPTION | QUANTITY | RECOMMENDEI FEASIBLE ACTI(3 | Repaired areas and/or spalls/delaminations and/or cracks exist in the deck surface or underside. The combined area of distress is more than 2% but less than 10% of the total deck area | 1187 | 0 Do Nothing ELEMENT INSPECTION NOTES: Minor longitudinal cracks are present in the deck top generally above the edges of the beams. There are random minor transverse cracks, some with efflorescence on the P/C panels. There is extensive map cracking with efflorescence at the north end of the north panel in Bay 4 of Span 3. There are several transverse cracks in the adjacent panel to the south. A portion of Bay 5 of Span 3 has been replaced with a CIP section. There is a minor transverse crack with efflorescence at the south end of the repair area and a transverse crack in the adjacent P/C panel.

11/05/97	G1.01 DECK (TOP) There are Class 1 to Class 2 longitudinal cracks in all spans primarily over the beams. These cracks are typical in precast deck panel construction and have been previously noted as Class 1. Cracking has resulted in spalled areas in span 2, right travel lane. A 2m x 150mm area, 6m from joint 3 has been repaired with an inappropriate material (asphalt) and is respalling around the edges.
12/13/95	G1.01 DECK(TOP)/SURFACING The deck top contains class 1 longitudinal cracks which run along the edges of the interior beams. These cracks are typical in precast deck panel construction and appear unchanged. There are minor spalls that run along a longitudinal crack in the deck at span 2, in the right travel lane.
12/13/94	G1.01 DECK(TOP)/SURFACING The deck top contains class 1 longitudinal cracks which run along the edges of the interior beams. The bottom of the deck panels contain an occasional class 1 transverse cracks with efflorescence. These cracks were first noted in the Special Deck Inspection on January 25, 1984 and show little change since that date. There is a class 2 spall in the deck top of span 3 approximately 3'x6"x1" 1' from abutment4. There are also minor spalls that run along a longitudinal crack in the deck at span 2, (right travel lane).
05/04/92	Deck Component 1.01 Deck (top) and 1.02 Deck (underside) The deck top contains Class 1 longitudinal cracks which run along the edges of the interior beams. The bottom of the deck panels contain an occasional Class 1 transverse crack with efflorescence. These cracks were first noted in the Special Deck Inspection on 1/25/84 and show little change since that date. There is a Class 2 spall in the deck top of span 3 adjacent to a previously patched area, approximately 15' from abutment 4. There are also minor spalls that run along a longitudinal crack in the deck at span 2, (right travel lane).

The final inspection report (Nov '99) [3.11] classifies the deck rating as '6' (satisfactory) with a condition state of 3 since the combined area of distress was between 2% to 10% of total deck area. The longitudinal cracks described in all previous reports are mentioned though now there were "*random minor transverse cracks with efflorescence*". Details of cracking in Bay 4 of Span 3 and transverse cracking in Bay 5 of Span 3 are mentioned.

Significantly, no reference is made as to the condition of the deck in Span 2, right lane (where failure actually occurred). This had been identified in the four previous reports from 1992-1997 [3.7-10] shown underlined in Table 3.3.

3.2.3 Environmental Conditions

It had been speculated that rainfall can be a contributory factor towards failure. Fig. 3.4 shows the distribution of rainfall for Sarasota in the period from Jan 12-Feb 12 2000 [3.12]. In the week immediately preceding failure there was no rainfall. However, there was significant (over 1 in.) rainfall 2 weeks earlier on Jan 24.

For the record, on the day of the failure, the temperature varied from a minimum of 55°F to a maximum 80°F [3.12].

3.2.4 Punching Shear

An estimate of the punching shear resistance can be obtained using code specified formula [3.6]. The analysis is approximate since available information is limited, e.g. the exact location of the punching failure in the deck is unknown. Only the panel where failure occurred was shown in the sketch (Fig. 3.3) included in the consultant's emergency report [3.4].

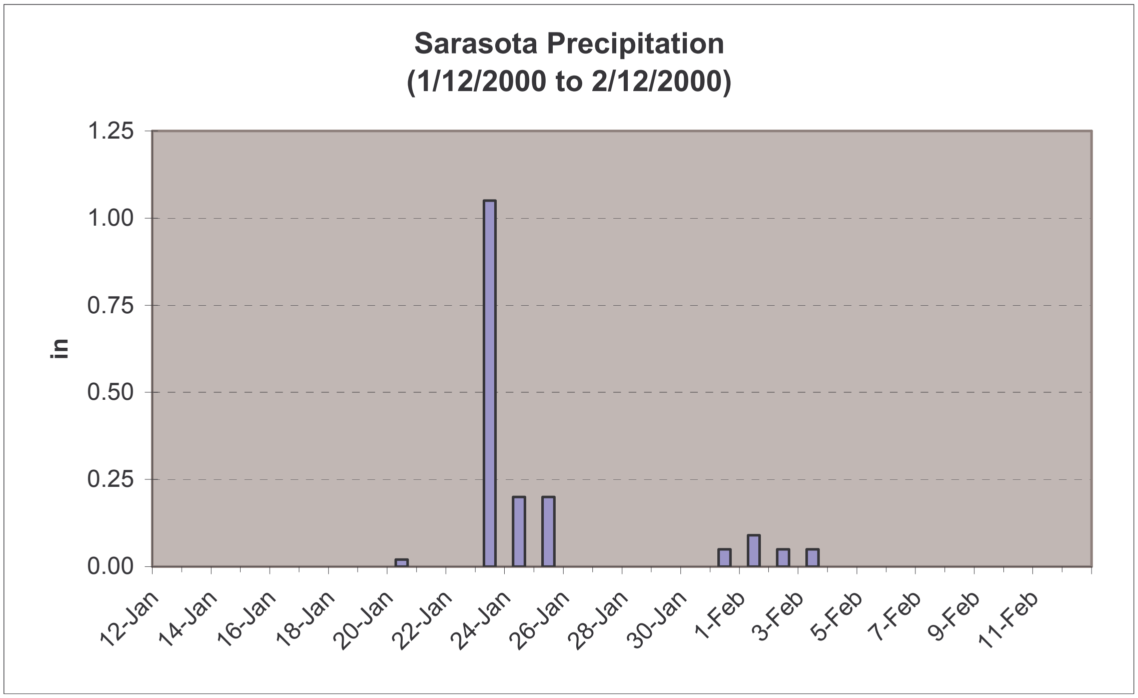


Figure 3.4 Sarasota Precipitation [3.12]

Two extreme cases are analyzed (1) full composite action and, (2) no composite action. For both cases, the wheel load (rectangular footprint, 10 in. x 20 in. [3.13]) is positioned at the critical section adjacent to the girder as shown in Fig. 3.1. Full composite action refers to the case where the wheel load is resisted by the entire 7 in. thick concrete slab (Fig. 3.2). This provides an upper bound on the maximum shear resistance. A lower bound on the shear resistance is provided when due to spalling and subsequent temporary repairs using flexible, asphalt-type material, the entire load is resisted by the precast, prestressed panel. In the analysis, the failure plane is assumed to be unaffected by the differing compressive strengths of the CIP (3000 psi) and precast prestressed panel (5000 psi).

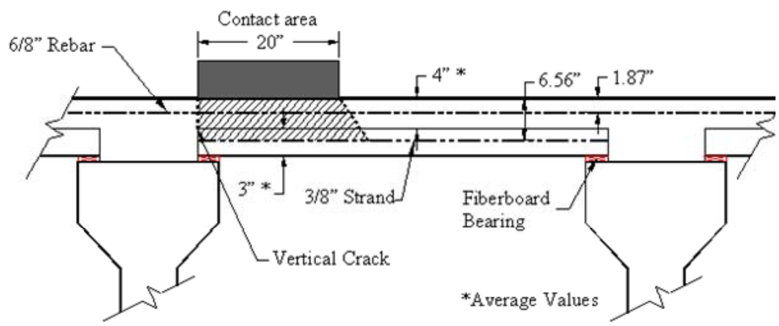
Inspection reports indicated that cracking developed along both the longitudinal and transverse edges of the panel. The fiberboard bearing cannot transfer loads to the girder and therefore, shear resistance was only provided by the two uncracked surfaces that extended half the effective depth away, $0.5d_e$, from the wheel for the assumed 45^0 failure surface [3.6]. Calculation of the punching shear load for both cases is summarized in Table 3.4. Complete calculations are shown in Appendix A.

In Table 3.4, b_0 signifies the failure perimeter as defined in Ref. 3.6. For the non-composite case, the minimum depth of the precast panel is used to calculate the effective depth. The calculation shows that the failure load varies from 15.3 kips to 56.3 kips. The former load is smaller than the AASHTO design wheel load *without the impact factor*. The dramatic reduction in punching shear resistance in the absence of contribution from the cast-in-place slab provides a possible explanation as to why failure occurred.

Table 3.4 Punching Shear Resistance Bridge # 170146

<table>
<tr><th>Load Case</th><th>Punching Shear Resistance*</th></tr>
<tr><td>

Full Composite Action



Tire Contact area: b=20in l = 10 in

</td><td>

CIP

d_e = 4 in.
b_0 = 34 in.
V_{CIP}= 29.8 kips

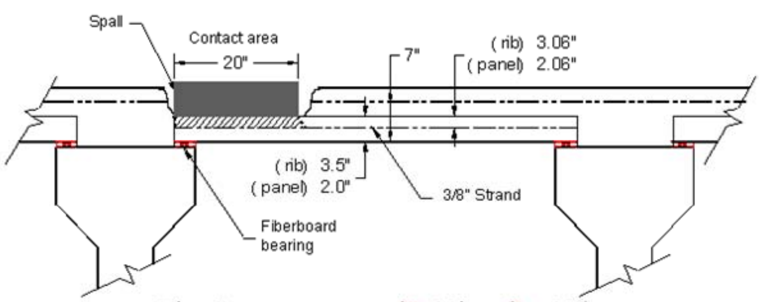
PANEL

d_e = 2.56 in. (ave)
V_{PANEL} = 26.5 kips

$\boxed{V_{TOTAL} = 56.3 \text{ kips}}$

</td></tr>
<tr><td>

No Composite Action



Tire Contact area: b_s=20in l_s = 10 in

</td><td>

CIP

V_{CIP}= 0 kips

PANEL

d_e = 2.06 in. (min)
b_0 = 32.1 in.

V_{PANEL} = 15.3 kips

$\boxed{V_{TOTAL} = 15.3 \text{ kips}}$

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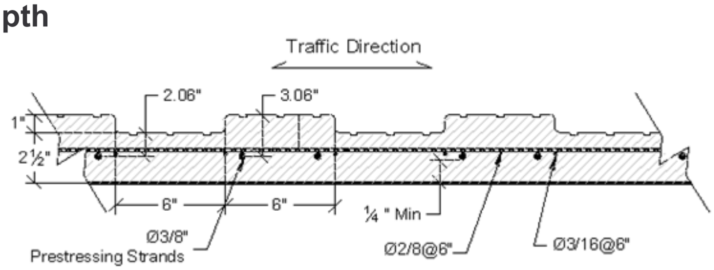
Effective Depth



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ASSUMPTIONS

1) Failure plane unaffected by the presence of higher compressive strength of the precast deck.
2) Fiberboard does not transfer loads. Shear resistance of cracked transverse and longitudinal panel boundaries are neglected

</td></tr>
</table>

* See Appendix A for detailed calculations.

3.2.5 Conclusions

Inspection reports indicate that longitudinal reflective cracks formed along the girder lines but remained dormant for over 10 years (1984-1994). Subsequently, there was more transverse cracking, spalling, repair and failure of re-repair culminating in localized failure. The dormant period suggests that failure may have been due to cumulative shear fatigue. Also, loads could also have been lower. However, no information on the distribution of truck traffic over lanes is available.

Simplified analysis indicated that regions of the deck where the cast-in-place slab did not resist any load could fail under design loads (Table 3.4). A review of the inspection records indicated that barring the final inspection, all four previous inspections had commented on the span where failure eventually occurred. Environmental factors may have played a role. Sustained rainfall could have led to bond degradation between concrete and reinforcement thereby lowering the shear capacity. Such effect would be limited to the cast-in-place slab.

3.3 I-75 NB Over Clark Rd, Bridge #170086

This four span bridge also located in Sarasota was built in 1980 and was 20 years old at the time of failure. It has two 88 ft 3 in. spans (*span 2, span 3*) and two 32 ft 6 in. secondary spans (*span 1, span 4*) for a total bridge length of 241 ft 6 in. The shorter spans use two AASHTO Type IV girders on the outside and five AASHTO Type II girders on the inside. These girders were all spaced 8 ft 10 in. apart as shown in Fig. 3.5. The two longer spans use seven AASHTO Type IV girders also spaced 8 ft 10 in. apart.

The composite slab was 7 in. thick. No specific details are available. However, they are likely to be similar to that shown in Fig. 3.2. The specified compressive strength of concrete for the precast panel was 5,000 psi and was 3,000 psi for the cast in place concrete slab. More details regarding deck panel construction may be found in Chapter 1.

The bridge has three 12 ft lanes, and two 10 ft wide shoulders as shown in Fig. 3.5. The average daily traffic (ADT) in the bridge during the year 2000 was 34,000 [3.1]. Thirty percent of the ADT was truck traffic. Details are summarized in Table 3.5. These are identical to that for the previous bridge.

Table 3.5 Bridge #170086

Bridge #170086 Characteristics	
Year Built	1980
Number of Spans	4
Lanes on Structure	3
ADT [3.1]	34,000
ADTT [3.1]	30%
Deck Condition Rating (2000)	7 (Good)
Composite Slab Thickness	7 in
Precast Panel Thickness	2-½ in (panel) 3-½ in (ribs)
Girder Type	AASHTO Type II and IV

3.3.1 Failure Details

Localized punching shear occurred late morning, Monday November 27, 2000. According to the consultant's emergency response report [3.4], failure occurred in span 4 (secondary span), bay 6, on the right lane where a 60 in. by 36 in. gaping hole developed near end bent 5 (Fig. 3.6). The report stated "*Half of the end panel adjacent to the expansion joint had been replaced at some previous time. This hole was the result of the failure of the remaining half of that panel*".

A photograph of the failed bridge panel obtained from the *Sarasota Herald* [3.14] is shown in Fig. 3.7. The entire concrete in the failed corner region was missing and debris can be seen lying on the road below. Some of the reinforcement had deformed plastically though none appear to be broken. However, the prestressing strands were ruptured. The location of the failed panel in span 4 is identified in the sketch provided in the consultant's report. As before, it has been re-drawn for clarity. This location is also identified in Fig. 3.5 as coinciding with the placement of the right truck wheel in the slow lane (Lane 1) close to the face of a girder.

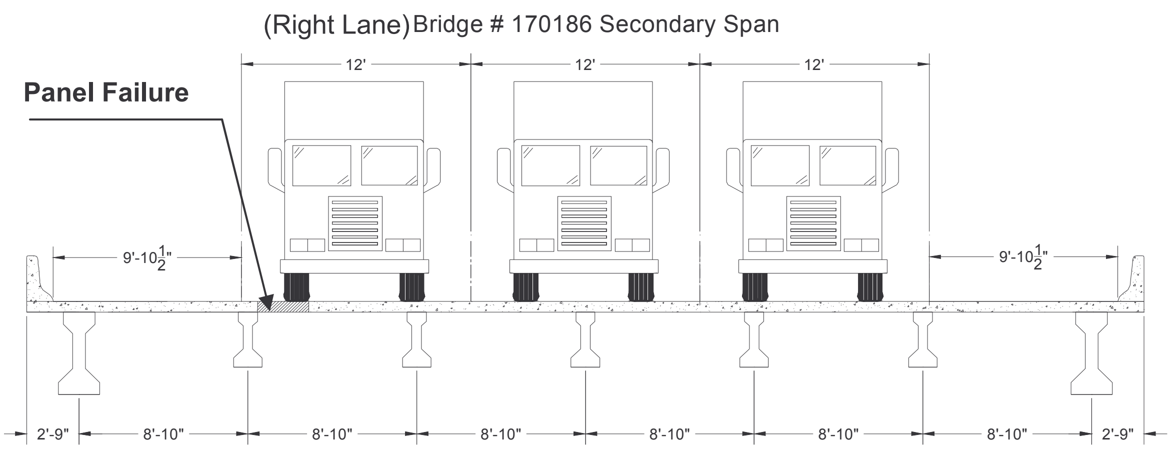


Figure 3.5 Cross Section View of Bridge #170086

3.3.1.1 Newspaper Account

Two articles related to the failure were reported in the local newspaper, *Sarasota Herald Tribune* [3.14-3.15].

The first article [3.14] published on November 28, 2000 with the headline "*Hole opens up in bridge on I-75 at State Road 72*". It noted that the hole that opened up was within "*a week after a state crew made repairs on the same spot*". The Florida Highway Patrol reported that there were "*no injuries or vehicle damage…*".

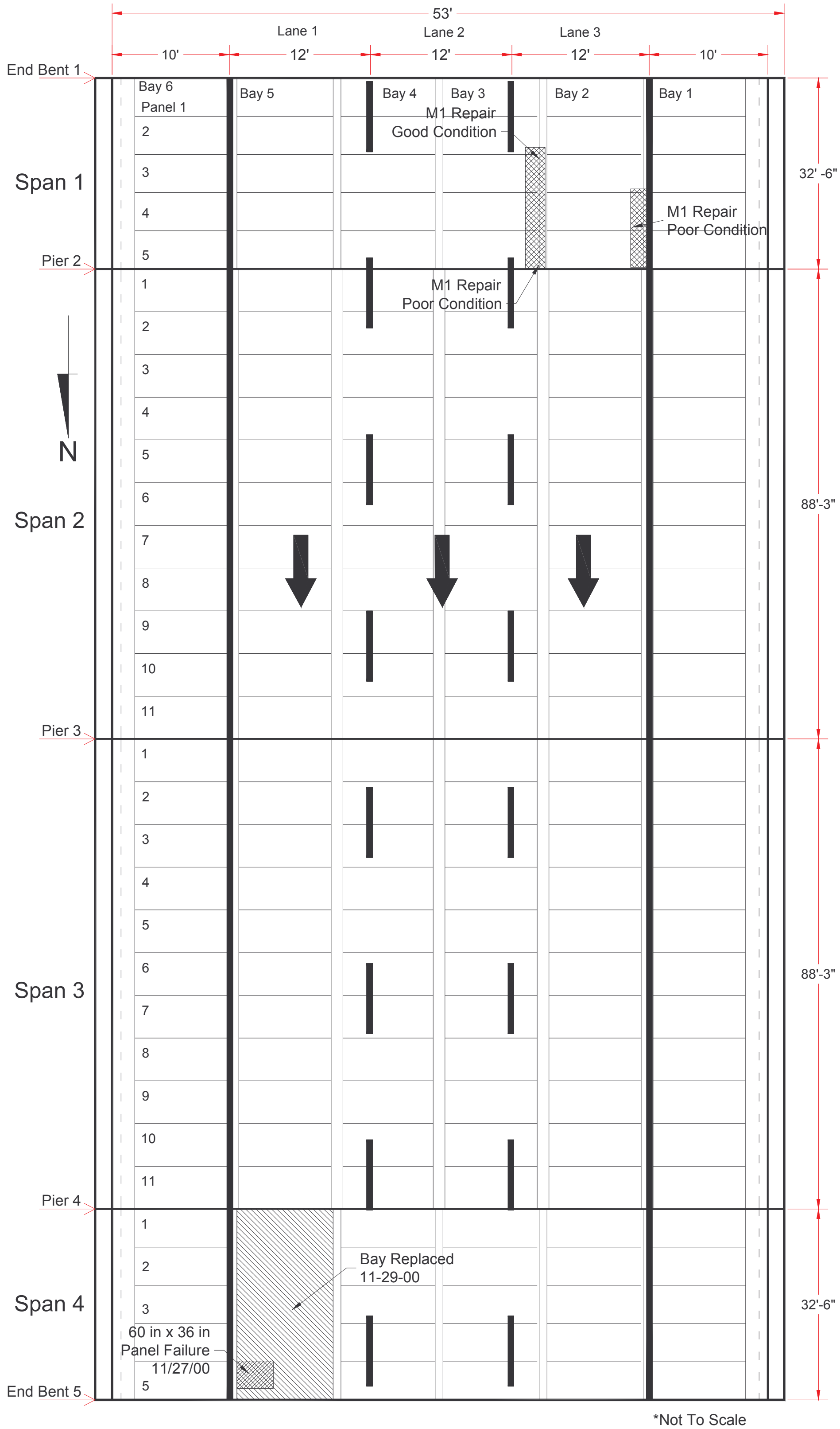


Figure 3.6 Location of Failed Panel, Bridge 170086 [3.4]

The second article published the following day [3.15] made the following observation "*The DOT offers assurances of daily checks and close inspections every 45 days, but those haven't predicted these failures. More effort and funds are needed immediately to make these bridges safe as soon as possible – before lives are lost. If protecting public safety requires shifting priorities or obtaining emergency funding, so be it.*"

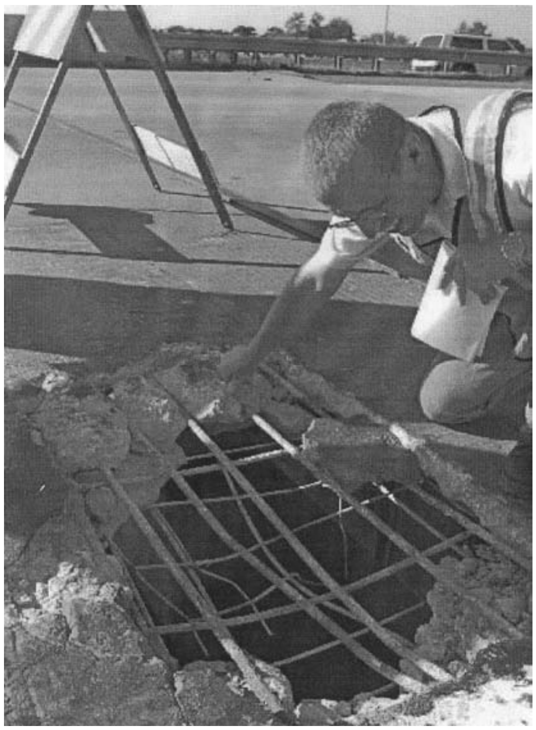


Figure 3.7 View of Failed Panel Bridge #170086
(Courtesy Sarasota Herald) [3.14]

3.3.2 Analysis

3.3.2.1 Inspection Reports

Table 3.6 contains relevant scanned excerpts from the last five inspection reports over the period Jan '93 to May '00 [3.16-3.20]. The last report (May '00) refers to the deck condition about six months prior to failure on Nov 27 '00.

The first three reports over the period Jan '93 to Jun '96 [3.16-3.18] are quite similar. Longitudinal cracks formed first along the girder lines followed by occasional transverse cracks at panel joints. As for the previous bridge (Table 3.3), the inspectors found "*no significant change*" over the 11 year period from May '85 to Jun '96.

Table 3.6 Excerpts from Inspection Reports (Bridge #170086)

	FDOT Bridge Inspection Report (Deck)
05/08/00	**UNIT:0 DECKS** **ELEMENT/ENV: 98/4 Conc Deck on PC Pane 1309 sq.m.** ELEM CATEGORY: Decks/Slabs CONDITION STATE (5) / DESCRIPTION / QUANTITY / RECOMMENDED FEASIBLE ACTION 2 — Repaired areas and/or spalls/delaminations and/or cracks exist in the deck surface or underside. The combined distressed area is 2% or less of the deck area. — 1309 — 0 Do Nothing ELEMENT INSPECTION NOTES: Minor longitudinal and transverse cracks are present on the deck top. Moderate abrasive wear is present throughout. There is a .1m x .1m x 10mm spall with no exposed reinforcing steel at the south end of an asphalt patch at the center of the west lane, 3m from the Abutment 5 joint. Minor cracks and spalls are present in and on the edges of random patch areas. Minor longitudinal and transverse cracks are present on random deck panels and in random repair areas.
05/04/98	G1.01 DECK (TOP) The deck top exhibits Class 1 to Class 2 longitudinal and transverse cracks throughout. The longitudinal cracks appear to run over or adjacent to the beams. Repairs made to the deck top in Span 1 exhibit Class 1 to Class 5 cracks and Class 1 spalls along the edges of the repairs. The deck exhibits moderate abrasive wear throughout. There is a deck repair 8m x 1.2m at Abutment 5.
06/19/96	G1.01 DECK(TOP)/SURFACING The deck top contains longitudinal class 1 cracks that run along the beams and occasional class 1 cracks at the panel joint. These cracks are due primarily to the deck panel type construction. These cracks have shown no significant change since May 1985.
08/24/94	G1.01 DECK(TOP)/SURFACING The deck top contains class 1 cracks that run longitudinal along the beams and occasional class 1 cracks at the panel joint. These cracks are due primarily to the deck panel type construction. These cracks were first noted in the May 1985 report and appear to show no change.
01/04/93	**Deck Component** **1.01 Deck (top)** **There are Class 1 and 2 cracks that run longitudinally along the beams, with an occasional Class 1 transverse crack at the panel joints. These cracks are due primarily to the deck panel type construction. These cracks were first noted in the report dated 5/85 and appear to show no change.**

However, significant deterioration was observed in the next inspection carried out in May ’98 [3.19]. Instead of “*occasional*” cracks reported earlier, longitudinal and transverse cracks had developed “*throughout*”. There was also severe cracking of repairs and spalls around the edge of the repair. The cracks were as wide as 1/8 in. (class 5). Mention is also made of deck repair over a large region about 26 ft x 4 ft at abutment 5. The description is not clear to tie it to eventual failure (see Fig. 3.7).

In the final report (May ’00) [3.20], top deck cracking is described as “minor”. This suggests that deficiencies identified earlier had been repaired. A small spall (4 in. x

4 in. x 0.2 in.) is mentioned as occurring at the "*center of the west lane, 3m (10 ft) from the abutment 5 joint*". The actual failure occurred at approximately the same location *but in the east lane*.

3.3.3 Environmental Conditions

Fig. 3.8 shows the distribution of rainfall for Sarasota in the period from Oct 27-Nov 27 2000 [3.12]. In the week immediately preceding failure there was about 0.68 in. of rain. It rained on 24th and 25th just 2 days before failure occurred. In this instance, rainfall may have been a factor. For the record, on the day of the failure, the temperature varied from a minimum of 53°F to a maximum 72°F.

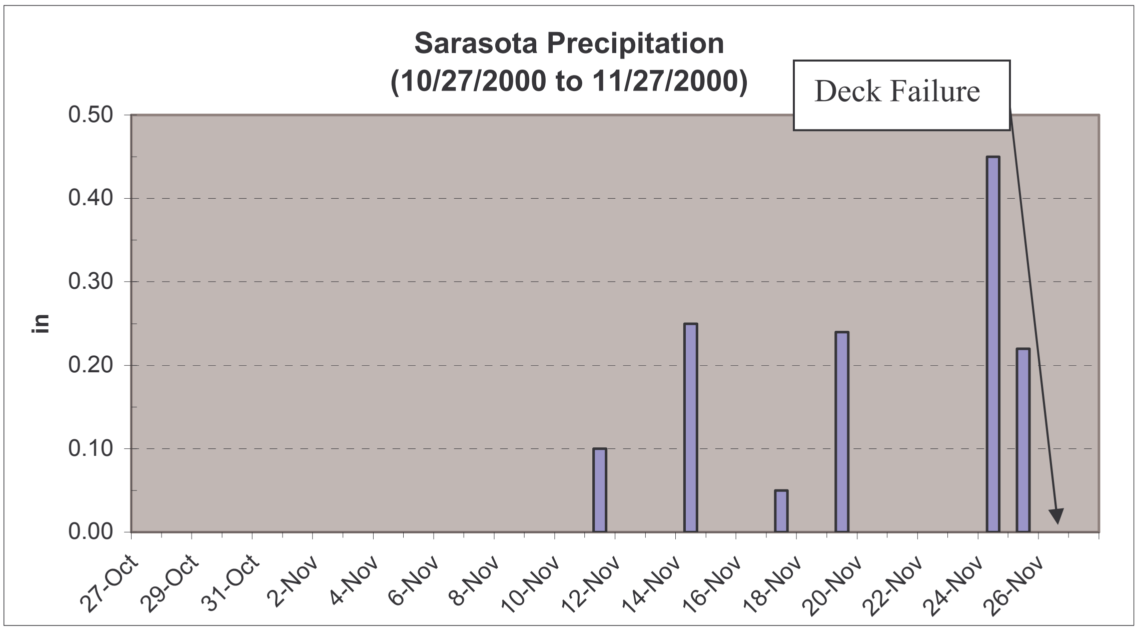


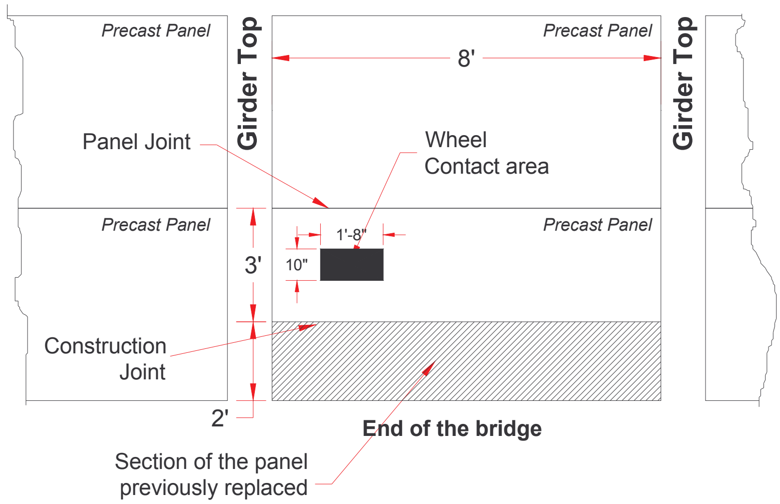
Figure 3.8 Sarasota Precipitation (Oct 27 – Nov 27 / 2000).

3.3.4 Punching Shear

According to the consultant's report cited earlier, "*half of the end panel adjacent to the expansion joint had been replaced at some previous time. This hole was the result of the failure of the remaining half of that panel*" [3.4]. The panel section that was replaced is marked in Fig. 3.7.

Assuming that no shear transfer was possible at the joint between the old and new panel, and reflective transverse cracking on the other side of the panel, the resistance of the slab is by one-way, not two-way shear. This "beam shear" type resistance is given by $2\sqrt{f'_c}\,b_w d$. Table 3.6 shows an estimate of the shear resistance taking b_w as 36 in. (the estimated unfailed length of a panel) with an average effective depth d of 2.56 in. Only the case where there is no composite action is considered since it gives lower loads. The calculated resistance is 13 kips smaller than the design load. The extent of the failed region is believed to be much greater in this failure because of the joint between the old and new panel (see Fig. 3.7, Table 3.7).

Table 3.7 Shear Resistance Bridge # 170086

<table>
<tr><th>Load Case</th><th>Shear Resistance</th></tr>
<tr><td>

Plan View</td><td>b_w= 36in
d = 2.56in (ave)
f'c = 5000 psi
$V_{PANEL} = 2\sqrt{f'c}b_w d$
V_{PANEL} =13 kips
V_{TOTAL} = 13 kips</td></tr>
</table>

3.3.5 Conclusions

A number of factors were responsible for this unusual failure. The most important of these may have been the joint between a panel segment – repaired and old - adjacent to an expansion joint (Fig. 3.7). In addition, there was heavy rainfall prior to failure that may have been a contributory factor by degrading the bond between concrete and steel. Unfortunately, there are too many unknowns to arrive at any definite conclusion.

The last inspection report six months prior to failure, mentions a spall close to the eventual failure location excepting that the west rather than the east lane was mentioned. It also noted damage to repaired areas in the form of cracking and spalling. The newspaper account stated that failure occurred at the same spot where temporary repairs had been carried out a week earlier. The shear failure load (Table 3.7) indicates that the deck could fail under design loads for this condition.

3.4 I-75 SB Over Clark Rd Bridge #170085

This 4-span bridge is identical to the one described on Section 3.3 and was also constructed the same year. A cross-section view is given in Fig. 3.9 while Table 3.8 provides a summary of relevant bridge details (this is identical to Table 3.5).

3.4.1 Failure Details

Localized failure occurred early morning on Wednesday December 20, 2000. According to the emergency response report [3.21] failure occurred in the first panel, bay 2 in span 3 adjacent to bent 3. The hole that punched right through the panel was estimated to be about 18 in. x 18 in. Fig. 3.10 shows the location of the failed panel. This is taken from reference [3.3] but was re-drawn for clarity. No photos of the localized failure are available.

From the cross section view (Fig. 3.9) it can be seen that failure again occurred in the right lane close to the panel support (girder face).

Table 3.8 Bridge #170085 Details

Bridge #170085 Characteristics	
Year Built	1980
Number of Spans	4
Lanes on Structure	3
ADT (2000)	34,000
Percent Truck ADTT	30%
Deck Condition Rating (2000)	7 (Good)
Composite Slab Thickness	7 in
Precast Panel Thickness	2-½ in (panel) 3-½ in (ribs)
Girder Type	AASHTO Type II and IV

* From National Bridge Inventory (2000)

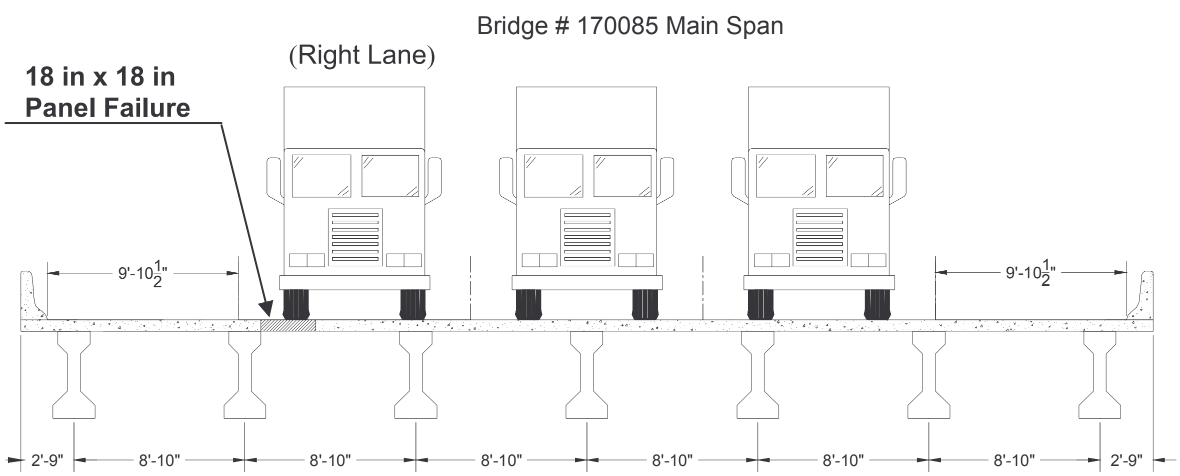


Figure 3.9 Cross Section View of Bridge #170085

3.4.1.1 Newspaper Account

Three articles regarding the failure were reported in the local newspaper, *Sarasota Herald Tribune* [3.22-3.24]. Of these only the first and last had relevant information.

The first article published on December 21, 2000 [3.22] stated that the hole was discovered at 7 am and that no one was injured. The reported size of the hole is 3 ft x 5 ft – same as in the previous bridge – possibly a mistake. The reporter quotes FDOT spokesman Gene O'Dell who said "*We just had our consultant inspect the Clark Road bridge two weeks ago and they said it was fine*". The last article published on December 23, 2000 [3.24] stated that the damage had been repaired and the bridge was opened to traffic. Mention was also made that a consultant was inspecting the bridge decks every 45 days and FDOT employees check them out once a month to "*see if there are any bad cracks, anything that will create a hole*" (O'Dell's quote).

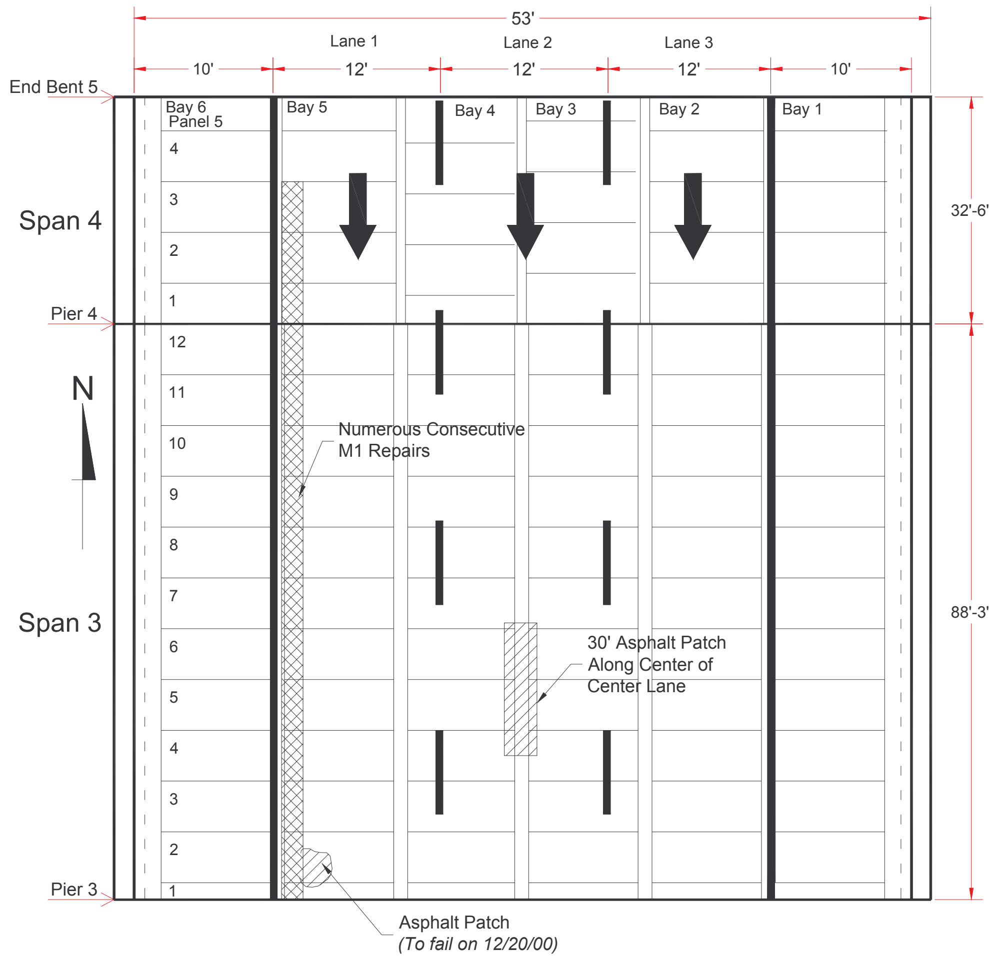


Figure 3.10 Location of Failed Panel Bridge [3.4]

3.4.2 Analysis

3.4.2.1 Inspection Reports

The five inspections preceding the localized failure were carried out on the same dates as the previous bridge (Table 3.6) in Jan '93, Aug '94, Jun '96, May '98 and May '00 [3.25-3.29]. The last inspection was completed about 7 months prior to the localized failure that occurred on Dec 21 ’00. Scanned excerpts from the complete reports are summarized in Table 3.9.

The first two reports [3.25-3.26] over the period Jan ’93 to Aug ’94 are quite similar to that for the previous bridge. Longitudinal cracks occurred first with occasional transverse cracks at panel joints. The inspectors state that the cracks first noted in the report dated May 1985 “*appear to* show *no change*”.

The next inspection carried out in Jun ’96 [3.27] reported more deterioration. All four spans contained longitudinal cracks along the beam lines with transverse cracking at the panel joint. Span 3 (where failure eventually occurred) had developed three spalls in the left lane ranging from 10 in. x 6 in. x 0.4 in. to 30 in. x 6 in. x 1 ¼ in. A fairly large 6 ft 6 in. x 6 in. x 1 ¼ in. spall had also developed in the center lane. In addition, patched areas in the left lane had cracked. This was expected to spall in the future.

Aside from longitudinal and transverse cracking in all spans, the inspection carried out in May ’98 [3.28] mentions that damage reported previously in span 3 had been repaired. However, cracks (up to 1/16 in.) and delamination had occurred in the repairs along the “*west edge pavement stripe*”. A delamination area 20 in. x 12 in. surrounding an asphalt patch at midspan in span 4 in the same region (west edge pavement stripe) had formed.

In the final report (May ’00) [3.29], top decking cracking is described as “minor”. The delamination in the middle of span 4 reported in the previous report had not grown in size. Fig. 3.11, scanned from the photo addendum of this inspection report, shows “c*oncrete and asphalt patches throughout spall span 3 1m x 50mm with exposed steel*”. The deficiency shown here happens to be at the exact location where failure occurred six months later. The deck condition rating of was given as 7, and the condition state of the bridge was reported as 2. None of the reports describe the underside of the deck. This suggests there was no cracking or efflorescence.

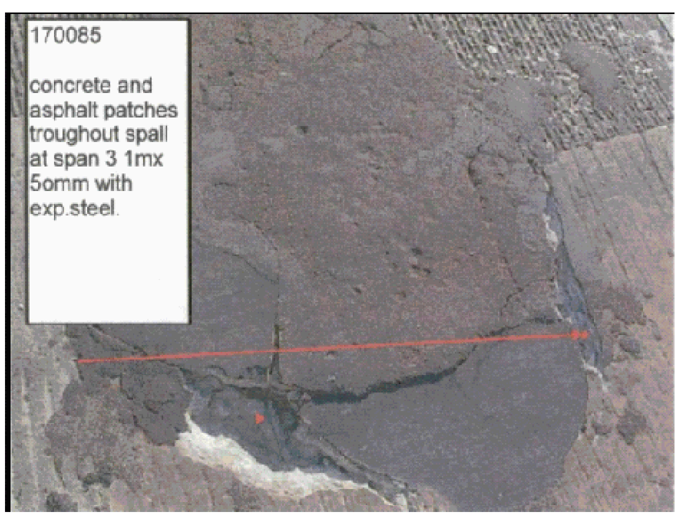


Figure 3.11 Deck Deficiency Six Months Before Failure, Bridge #170085

Table 3.9 Excerpts from Inspection Reports (Bridge #170085)

	FDOT Bridge Inspection Report (Deck)
05/08/99	ELEMENT/ENV: 98/4 Conc Deck on PC Pane 1309 sq.m. ELEM CATEGORY: Decks/Slabs CONDITION STATE (5) | DESCRIPTION | QUANTITY | RECOMMENDED FEASIBLE ACTION 2 | Repaired areas and/or spalls/delaminations and/or cracks exist in the deck surface or underside. The combined distressed area is 2% or less of the deck area. | 1309 | 0 Do Nothing ELEMENT INSPECTION NOTES: Minor longitudinal and transverse cracks are present on all spans. There is a .5m x .3m area of delamination surrounding an asphalt patch at midspan of Span 4 adjacent to the west edge of pavement stripe. There are random minor transverse cracks on the precast deck panels.
05/04/98	G1.01 DECK SURFACING There are Class 1 to 2 longitudinal and transverse cracks in all spans. The longitudinal cracks appear to be over the beams. The spalls previously noted in Span 3 have been repaired; however, these repairs exhibit Class 1 to 3 cracks and areas around the patches which are delaminated. This condition exists primarily along the west edge of pavement stripe. Span 4 has a delaminated area .5m x .3m surrounding an asphalt patch at midspan adjacent to the west edge of pavement stripe.
06/19/96	G1.01 DECK(TOP)/SURFACING All spans contain class 1 and 2 longitudinal crack which run along the beams. There are class 1 and 2 transverse cracks along the joints of the deck panels. Span 3 contains three spalled areas(25cm x15cm x1cm), (35cm x20cm x3cm)and(75cm x 15cm x 3cm)in the left lane. The center lane contains a spall (2m x 15cm x 3cm). These spalled areas are in line with the longitudinal cracking and will grow along the cracking. The patched areas in the left lane are cracked and crumbling which will spall.
08/24/94	G1.01 DECK(TOP)/SURFACING There are class 1 and 2 cracks that run longitudinally along the beams, with an occasional class 1 transverse crack at the panel joints. These cracks are due primarily to the deck panel type construction. These cracks were first noted in the report dated May 1985 and appear to show no change.
01/04/93	Deck Component 1.01 Deck (top) The deck top contains Class 1 cracks that run longitudinal along the beams and occasional Class 1 cracks at the panel joints. These cracks are due primarily to the deck panel type construction. These cracks were first noted in the 5/85 report and appear to show no change.

From the cross section view Fig. 3.9 we can see that the failure occurred again in the right lane and close to the panel support girder face.

3.4.3 Environmental Conditions

Fig. 3.12 shows the distribution of rainfall for Sarasota in the period from Nov 20-Dec 20 2000 [3.12]. In the ten days immediately preceding failure it rained on six occasions. It rained 0.05 in. the day before failure occurred. In this instance, rainfall may have been a factor. For the record, on the day of the failure, the temperature varied from a minimum of 38°F to a maximum 58°F.

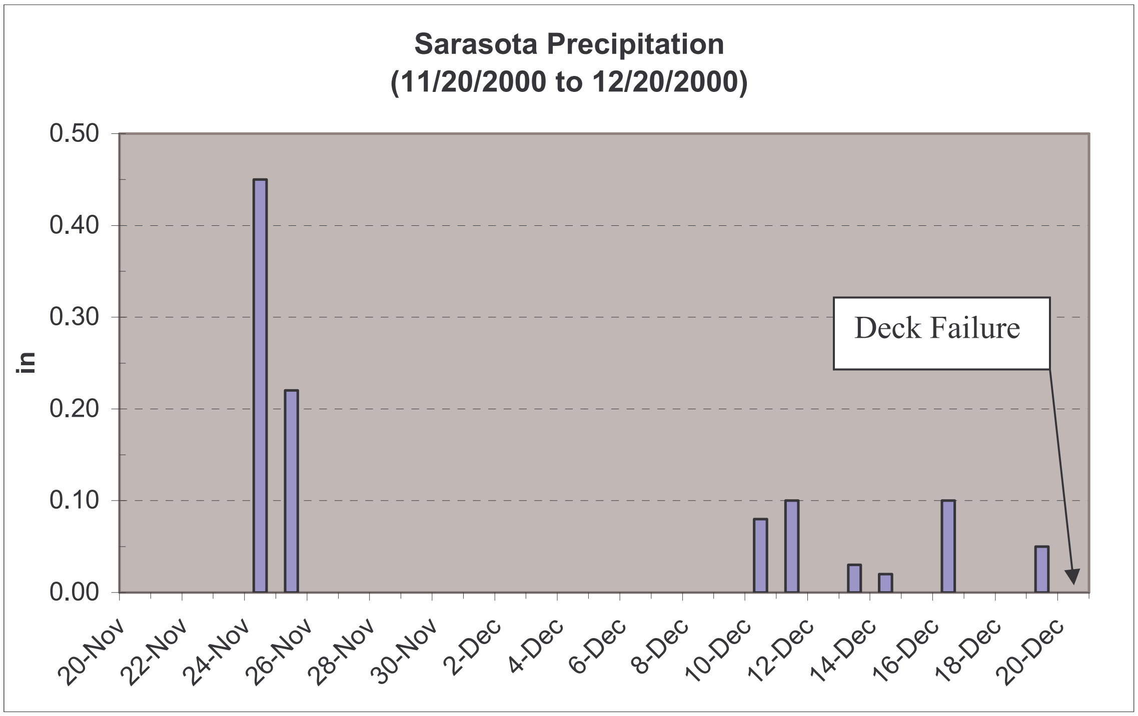


Figure. 3.12 Sarasota Precipitation (Nov 20 – Dec 20 / 2000).

3.4.4 Punching Shear

As the geometry and the material properties in the deck were identical to that in the previous bridges, the calculated punching shear failure load is also identical. The lower bound for the failure load is calculated to be 15.3 kips which is smaller than the design wheel load. See Table 3.4 for details.

3.4.5 Conclusions

The failure in this bridge was very similar to that in the first bridge (Section 3.2). Shear fatigue may have been responsible for failure. The failure load was estimated to be 15.3 kips (Table 3.4). The last inspection report stated that repairs had started to crack. As all three bridges failed in the same geographical location, faulty construction was undoubtedly a factor.

3.5 Crosstown Viaduct over Downtown Tampa, Bridge #100332 Span 38

This 9,600 ft bridge is the longest deck panel bridge in the area. It has a total of 91 spans, of which 24 were built in 1975 using a full-depth cast in place concrete slab. The remaining 67 spans were built in 1980 using precast deck panels. Two of 67 spans used steel girders (average span 170 ft) while the rest used prestress girders (average span 80 ft). This was one of the first deck panel bridges built on a main highway in District 7.

Span 38, where the failure occurred, was built using prestressed concrete girders. Its span length is 47 ft. The bridge section was 22 year old at the time of the failure.

The composite slab was 7 in. thick with the precast panel thickness varying between 2-½ in. or 3-½ in. (at the rib-section) as shown in Fig. 3.2. This panel thickness is typical for all deck panel bridges in this area. The specified compressive strength of concrete used for the precast panel is 5,000 psi. It is 3,000 psi for the cast in place concrete slab. More details regarding deck panel construction may be found in Chapter 1.

The bridge has two 12 ft lanes. The right shoulder is 8 ft wide and the left shoulder is only 4 ft wide, as shown in Fig. 3.13. The average daily traffic (ADT) during 2002 was 23,000 [3.1]. Eight percent of the ADT was truck traffic (Table 3.10).

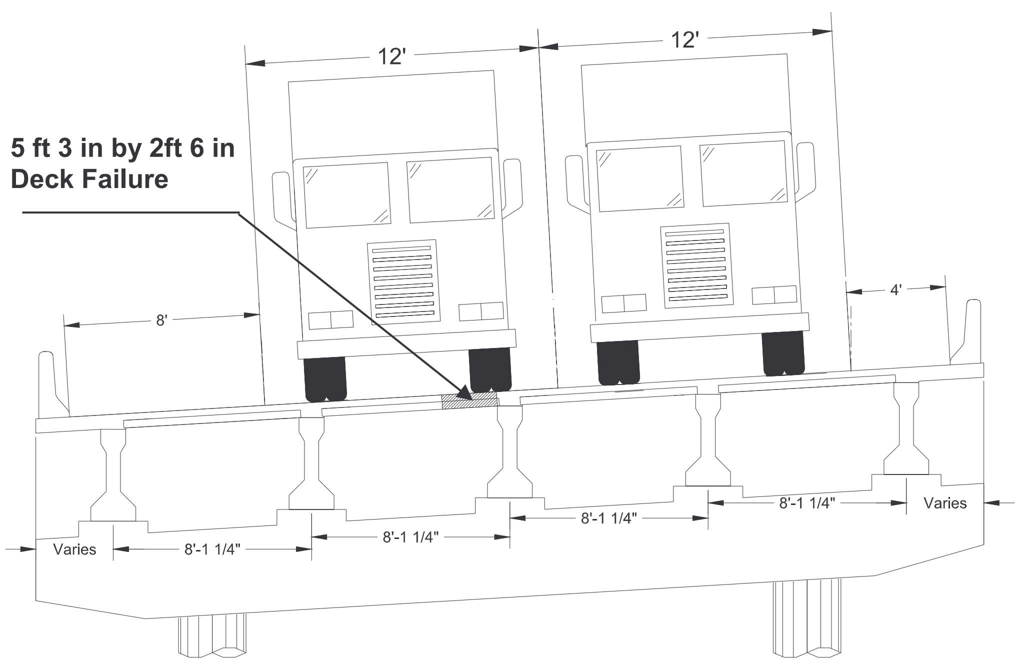


Figure 3.13 Cross Section View of Bridge # 100332, Span 38

Table 3.10 Bridge #100332 Details

Bridge #100332 Characteristics	
Year Built	1975 (spans 1 – 24) 1980 (spans 25 – 91)
Number of spans	91
Lanes on Structure	2
ADT (2002) [3.1]	23,000
Percent Truck ADTT [3.1]	8%
Deck Condition Rating Span 38 (2001)	5 (Fair)
Deck Condition Rating Span 70 (2003)	5 (Fair)
Composite Slab Thickness	7 in.
Precast Panel Thickness	2-½ in (panel) 3-½ in (ribs)
Girder Type (Span 38)	AASHTO Type IV

3.5.1 Failure Details

This failure was first noticed early morning on Wednesday, October 2 2002. It was located on the right lane close to mid-span. A gaping 5 ft 3 in. by 2 ft 6 in. hole formed (see Fig. 3.14). The same figure shows photos of the failed region and its underside two days prior to failure. Staining of the underside is visible. The concrete and repair material separated from the reinforcement which did not rupture. Fig. 3.15 provides a sketch showing the failure location on the deck.

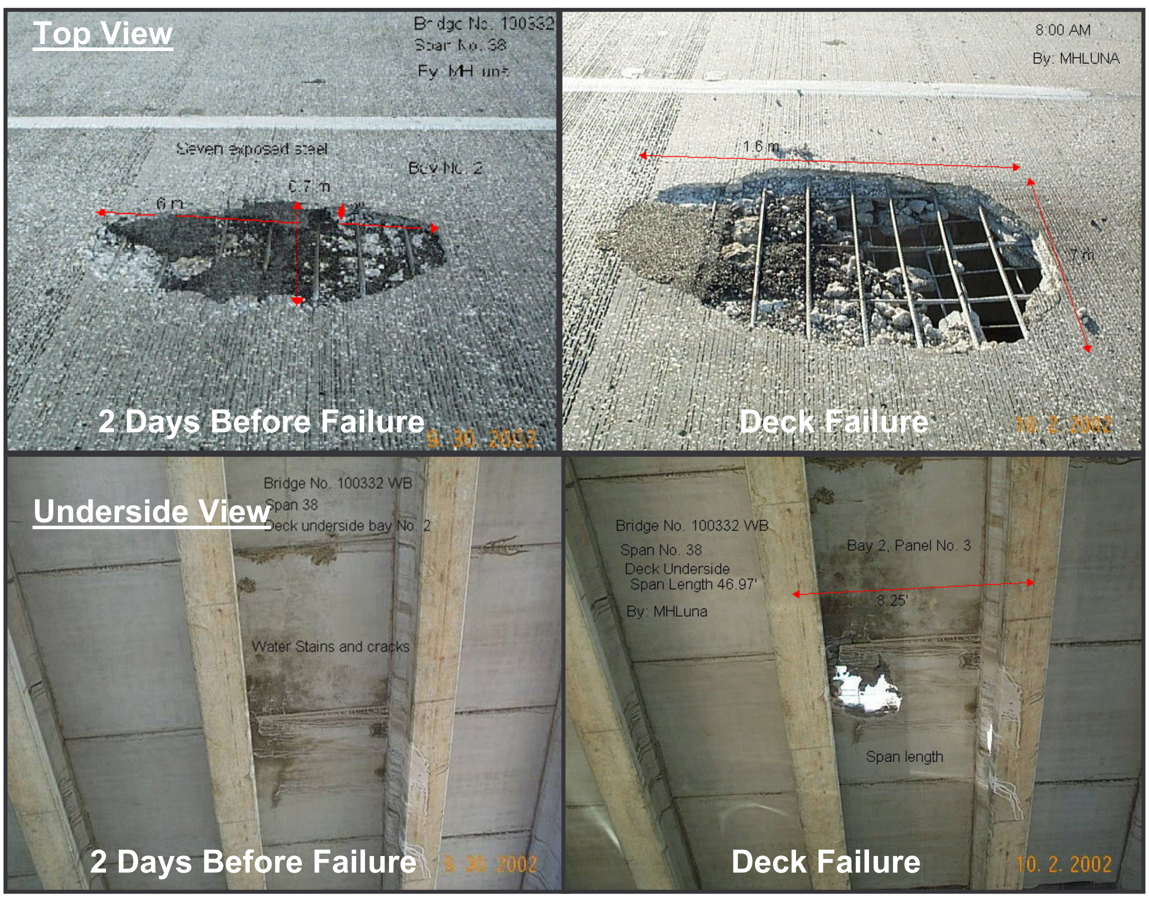


Figure 3.14 Localized Deck Failure. Bridge #100332, Span 38

This damage was repaired by demolishing the whole bay where the failure occurred and placing a new deck using full depth cast in place concrete.

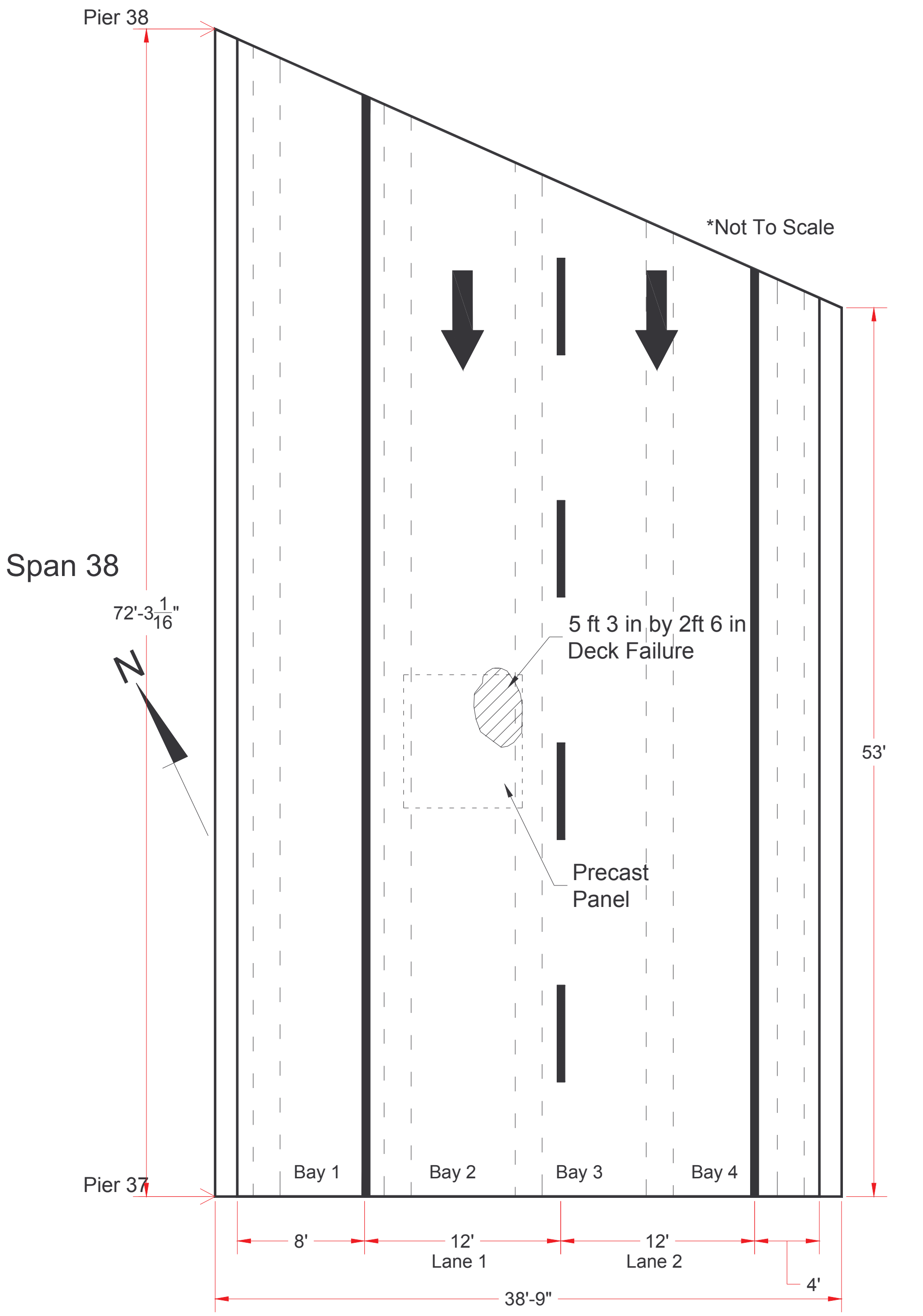


Figure 3.15 Location of Failed Panel, Bridge 100332 Span

3.5.1.1 Newspaper Account

No account of the failure was published in the local newspaper.

3.5.2 Analysis

3.5.2.1 Inspection Reports

The five inspections preceding the localized failure were carried out over eight years in May ’93, May ’95, Aug. ’97, Aug. ’99 and Aug. '01 [3.30-3.34]. The reports provide information on the entire bridge and for this reason there is minimal information relating to span 38 where the failure occurred. In the last biannual inspection completed in Aug '01 [3.34] approximately 14 months before the failure occurred on October 2 2002, the deck was given a condition rating of 5 (Fair) and a condition state of 2. No significant deficiencies relating to span 38 were documented. Scanned excerpts from these inspection reports are summarized in Table 3.11 for completeness.

Because of widespread deterioration of the bridge it was continuously monitoring by FDOT. Information from these monthly inspections provide invaluable information on the progression of degradation leading to failure.

Deterioration of the section that eventually failed was first reported on July 31 2002 as "*30 in. x 20 in. concrete delamination*" [3.35]. This was determined on the basis of a "hammer test" in which the suspected region is hit with a hammer and a hollow sound detected. By August 19 2002 the delamination had changed to a 48 in. by 10 ¾ in. spall. This spall was temporarily patched at that time. At the next inspection on September 30 2002, the patch was found to have failed. In addition, the extent of the spall had increased to 48 in. by 30 in. by 1.5 in deep (See Fig. 3.14). Temporary repairs were again carried out and the patch repaired. Two days later, this new patch failed and a 48 in. by 30 in. gaping hole developed at the site as shown in Fig. 3.14.

A USF research team visited the bridge one day after failure. Measurements taken at the site and from retrieved debris indicated that the deck was thinner than its nominal thickness. It was found to be 6-3/8 in. not 7 in. as specified in the plans stamped "as built" (see Fig. 3.16).

Table 3.11 Excerpts from Inspection Reports (Bridge #100332 Span 38)

FDOT Bridge Inspection Report (Deck)

08/29/01

ELEMENT/ENV: 98/4 Conc Deck on PC Pane 21588 sq.m. ELEM CATEGORY: Decks/Slabs

CONDITION STATE (5)	DESCRIPTION	QUANTITY	RECOMMENDED FEASIBLE ACTION
2	Repaired areas and/or spalls/delaminations and/or cracks exist in the deck surface or underside. The combined distressed area is 2% or less of the deck area.	21588	1 Spalls & Delams

WORK ORDER RECOMMENDATION:

REPR SPLs in Spans 26 27 29 39 41 44 47 53 57 58 70 76 88 & 91. 0.3M3

NOTE: DO NOT ISSUE WORK ORDERS. THI BRIDGE DECK PANEL DECK IS UN CONTINUOUS MONITORING AND ARE ISSUED ON A REGULAR BASIS. 11/2

ELEMENT INSPECTION NOTES:

NOTE: Previous quantity appears understated. Current quantity field verified.

This element quantifies the concrete deck with precast concrete deck panels in Spans 25 through 91 with the exception of Span 34.

Several deck panel undersides have transverse cracks up to 0.4mm wide in random locations. Refer to the Addendum for additional text.

There are several failing repairs and spalls in the deck top and deck panel undersides. Refer to the Addendum for additional text.

08/31/99

ELEMENT/ENV: 98/4 Conc Deck on PC Pane 19328 sq.m. ELEM CATEGORY: Decks/Slabs

CONDITION STATE (5)	DESCRIPTION	QUANTITY	RECOMMEND FEASIBLE AC
2	Repaired areas and/or spalls/delaminations and/or cracks exist in the deck surface or underside. The combined distressed area is 2% or less of the deck area.	19328	1 Spalls & Del

WORK ORDER RECOMMENDATION:

Repair the spalls in Spans 25,26,39,43,44,47,57,58,69,70,74,76,87,88 and 91.

ELEMENT INSPECTION NOTES:

Spans 25-91(Except 34) - The concrete deck has longitudinal, transverse and map cracking throughout. These cracks are up to 0.4mm wide. Some of these cracks have edge spalling. Surface abrasion is throughout the deck exposing the aggregate. Voids are in the concrete d where the aggregate is missing. Spalling is along the tyning grooves throughout the deck. Th has minor popoffs due to the removal of the roadway reflectors. Transverse cracks meander the full length of the construction joints. Edge spalling is at the expansion joints, some have repaired with nosing compound. Previous noted spalls, areas of reinforcing steel, and some of cracking have been patched. The patched areas seem solid and well bonded when sounde a hammer. Many of the patched areas have transverse and longitudinal cracks with corrosion stains, reflecting the underlying of reinforcing steel. Random cracks up to 0.6mm wide, and s some with short lengths of exposed reinforcing steel, are on the underside of the precast pane forms. The spalls are typically 150 mm to 500 mm in length or diameter and appear to be the of corrosion of the reinforcing strands or bars. Diagonal cracks up to 0.2mm wide, some with efflorescence are on the undersides of the overhangs, predominantly at the joints. Refer to th Addendum report for specific deficiencies listed by span number and photos of the deficiencie Corrective action was recommended and not completed on this element.
WO - Repair any spalls with exposed steel or any failing repairs in Spans 25,26,39,43,44,47,57,58,69,70,74,76,87,88 and 91.

Span-Unit

38-3 No significant deficiencies were observed during this inspection.

08/26/97	G1.01 DECK (TOP) There are longitudinal, transverse and map cracks throughout the deck top. At the construction joints, Class 1 to Class 2 transverse cracks typically meander along the full lengths of the joints. Edge spalling, some of which has been repaired with nosing compound is present at the expansion joint edges. Previously noted spalls, areas of exposed reinforcing steel, and some areas of cracking have been patched. The patched areas, when sounded with a hammer, seem solid and well bonded. For many of the patched areas, there are longitudinal and transverse cracks, some with corrosion stains reflecting the underlying reinforcing steel. Refer to photos 1 and 2 on pages 17 and 18 for a typical view.
05/15/95	G1.01 Deck (Top) NCR:6 All decks have distinct longitudinal and transverse class 1 cracks. The cracks vary in length from a few meters to the end tie width or length of the span. Class 2 and greater cracks with exposed rebar and other significant deficiencies are recorded in table 1 on pages 24 thru 27. SPAN: 38 — DEFICIENCY: Two class 1 spalls with cracking — LANE: North — LOCATION: Near pier 38 and 39
05/20/93	G1.01 Deck (Top) All decks have distinct longitudinal and transverse Class 1 cracks. The cracks vary in length from a few feet to the entire width or length of the span. Class 2 and greater cracks, as well as spalls that are exposing rebar and spalls that are of significant size, are recorded on Table 1 on Pages 19 and 20. Generally, the most serious deficiencies were spalls with exposed rebar and areas of honeycombed concrete. SPAN: 38 — DEFICIENCY: One Class 1 spall with cracking — LANE: North — LOCATION: Near Pier 38

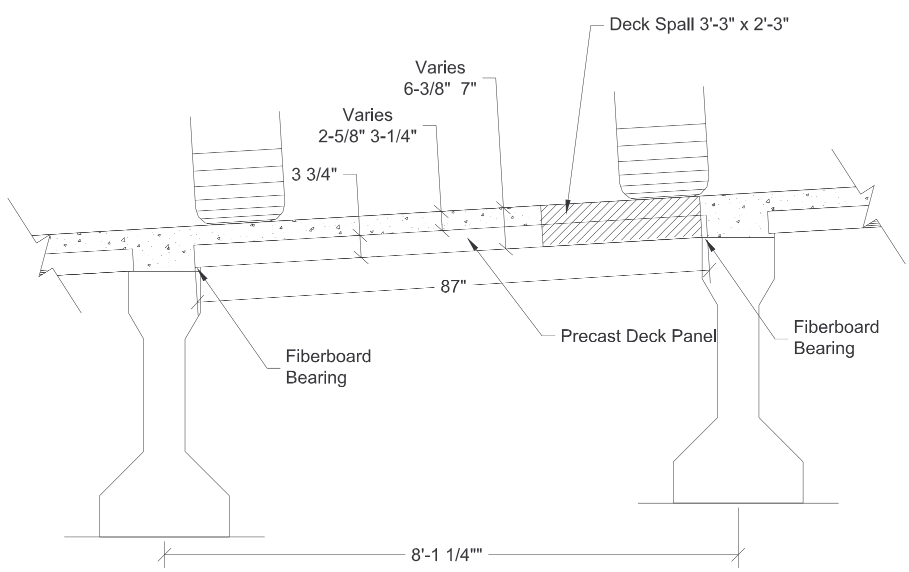


Figure 3.16 Deck Thickness Measurements and Details of Failed Section. Bridge # 100332, Span 38

3.5.3 Environmental Conditions

Precipitation readings at Tampa International Airport (6 miles from the bridge) for a period of one month before the localized deck failure are shown in Fig. 3.17. It may seen that that there was continuous rain over five days with a rainfall of 0.55 in. one week before the failure. However, no rain occurred 4 days before failure, For the record, on the day of the failure, the temperature varied from a minimum of 75°F to a maximum 88°F.

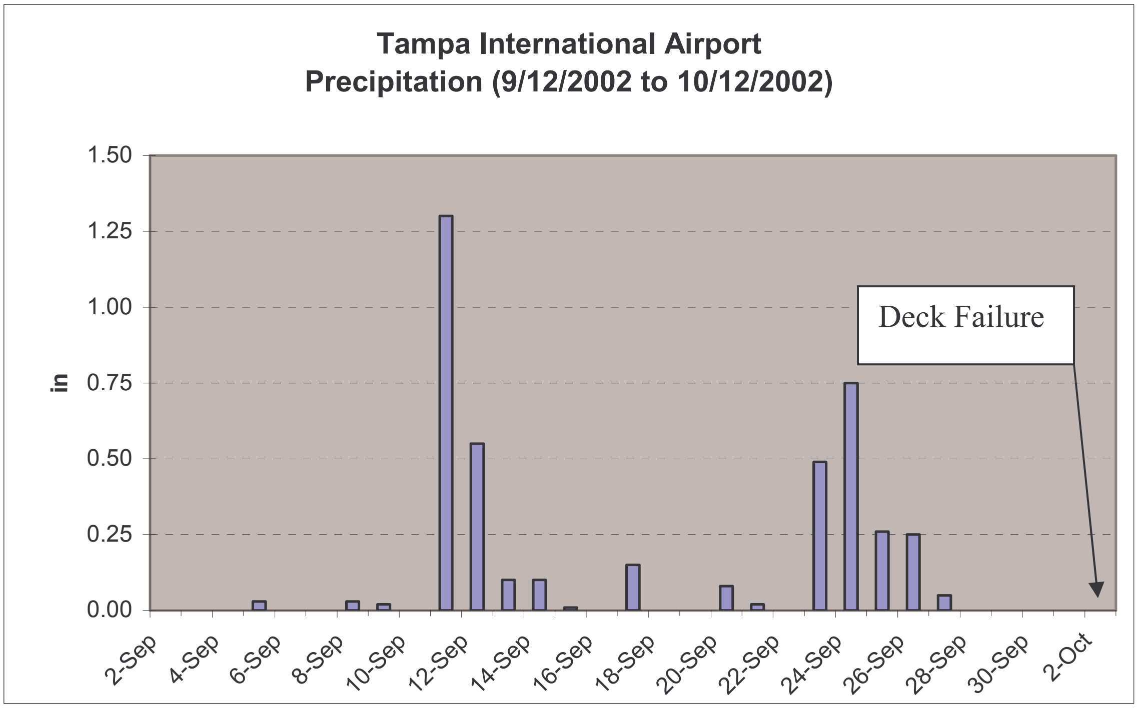


Figure. 3.17 Tampa International Airport (Sep. 02 – Oct. 02 2002)

3.5.4 Punching Shear

Although the deck was found to be thinner than its nominal value (see Fig. 3.15), the panel thickness was the same. In view of this, the lowerbound value of punching shear would still be the same - 15.3 kips. For details see Table 3.4.

3.5.5 Conclusions

The biennial inspection data just provides a snapshot on the condition of the bridge and is therefore not always very useful. Continuous monitoring data indicated that delaminations led to large spalls. If flexible materials are used for temporary repairs, they are unable to transfer wheel loads to the adjoining slab because of their low stiffness and localized failure can occur at loads below the design load (Table 3.4). Measurements indicated that the thickness of the deck could be smaller than nominal dimensions at specified locations. Rainfall could have been a contributory factor in this case.

3.6 Crosstown Viaduct over Downtown Tampa, Bridge #100332, Span 70.

This deck failure case occurred in the same bridge described in Section 3.5 but in span 70, (see Table 3.10 for general details). Span 70 is 65.5 ft long, and is built using type III AASHTO prestressed concrete girders. The girders are spaced center to center 6 ft 5 in.

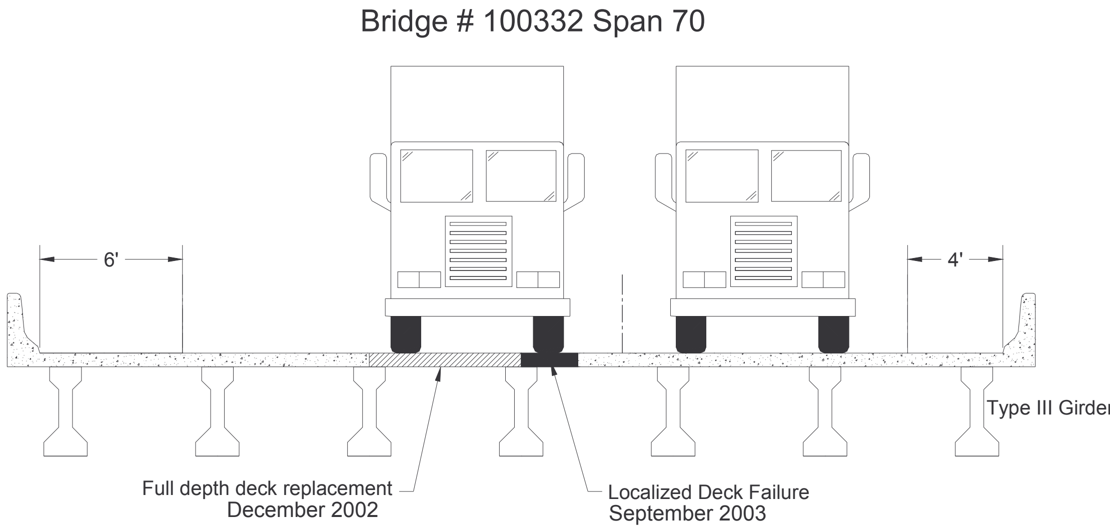


Figure 3.18 Cross Section View of Bridge# 100332, Span 70

3.6.1 Failure Details

Localized punching shear occurred in early morning, Friday September 5 2003. The failure was located close to the midspan and in the right lane. The failure region measured by the USF research team was estimated to be about 2 ft by 3 ft.

Photos of the failed section are shown in Fig. 3.19. A sketch showing the location of the failure in the deck is shownin Fig. 3.20. Initial spalling ahead of an M1 repair extended into the repair itself. Under subesequent loading, rebars were exposed in the spalled region. The concrete ultimately separated from the steel due to the impact of repeated wheel loads and a void formed.

Fig. 3.19 has three photos. The main photo is a close-up plan view of the damage from the top of the deck. Note that the rebars are not broken nor plastically deformed. Small sections of concrete just separated from the reinforcement. One of the prestressing strands can be seen to be intact. A second photo provides an overview of the deck. The third photo shows the extent of the opening in the deck from the underside. Water staining is clearly visible. This failure was repaired by demolishing the whole bay where the failure occurred, and the one adjacent in the left lane, and placing a new deck using full depth cast in place concrete.

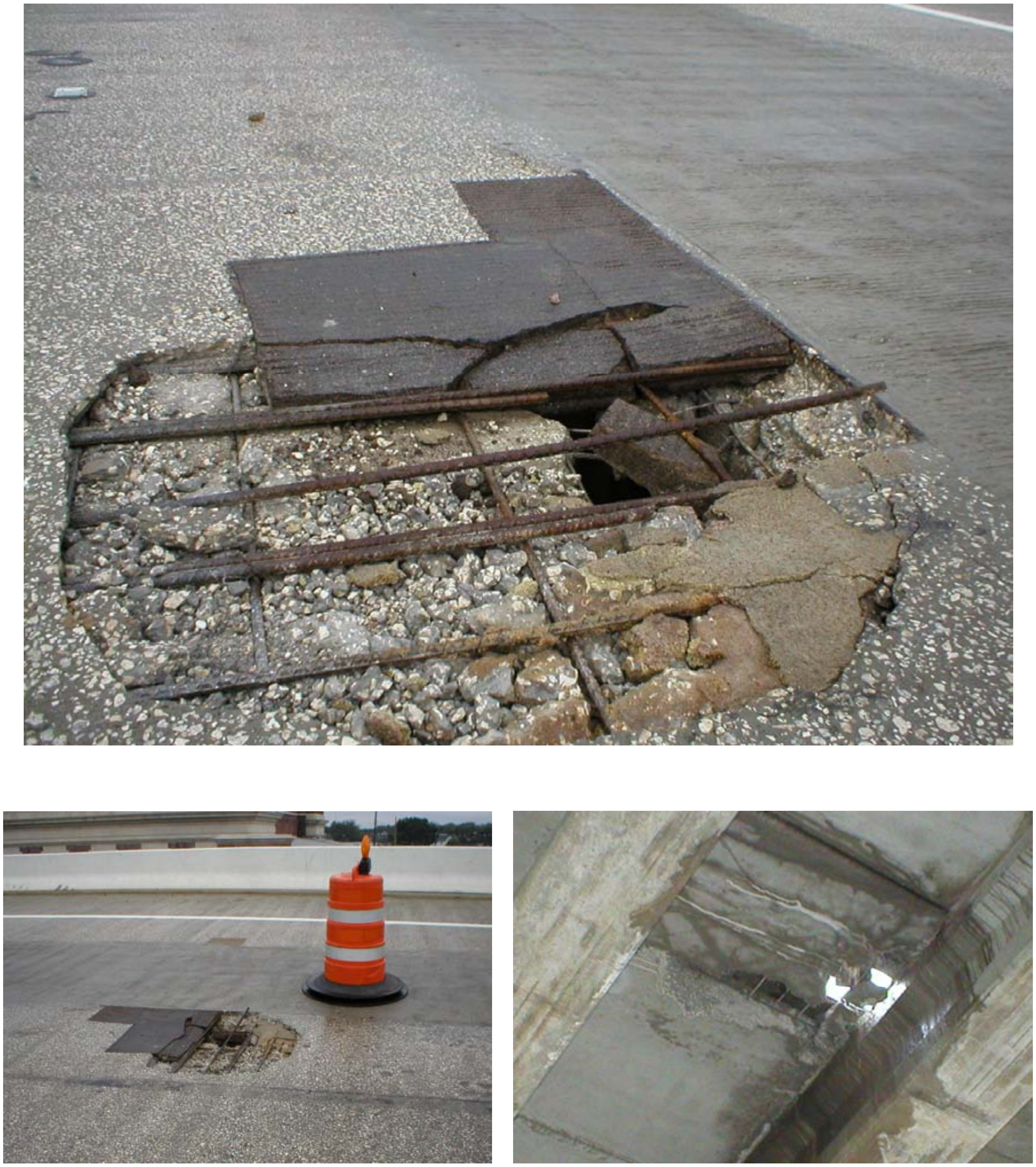


Figure 3.19 Localized Deck Failure, Bridge # 100332, Span 70

3.6.1.1 Newspaper Account

Two articles regarding the failure were reported in the local newspaper, *Tampa Tribune* [3.36, 3.37]

The first article [3.36] published on September 9 2003 with the headline "*Small Hole Paves Commuters' Way To A Traffic Jam*", makes reference to the large delays users are facing due to the deck failure. It also offered an explanation as to why the hole developed “*Florida’s endless down pours opened a small hole in a bridge on the Lee Roy Selmon Expressway on Friday, creating a huge mess for morning rush hour commuters that won’t improve until Sunday*”.

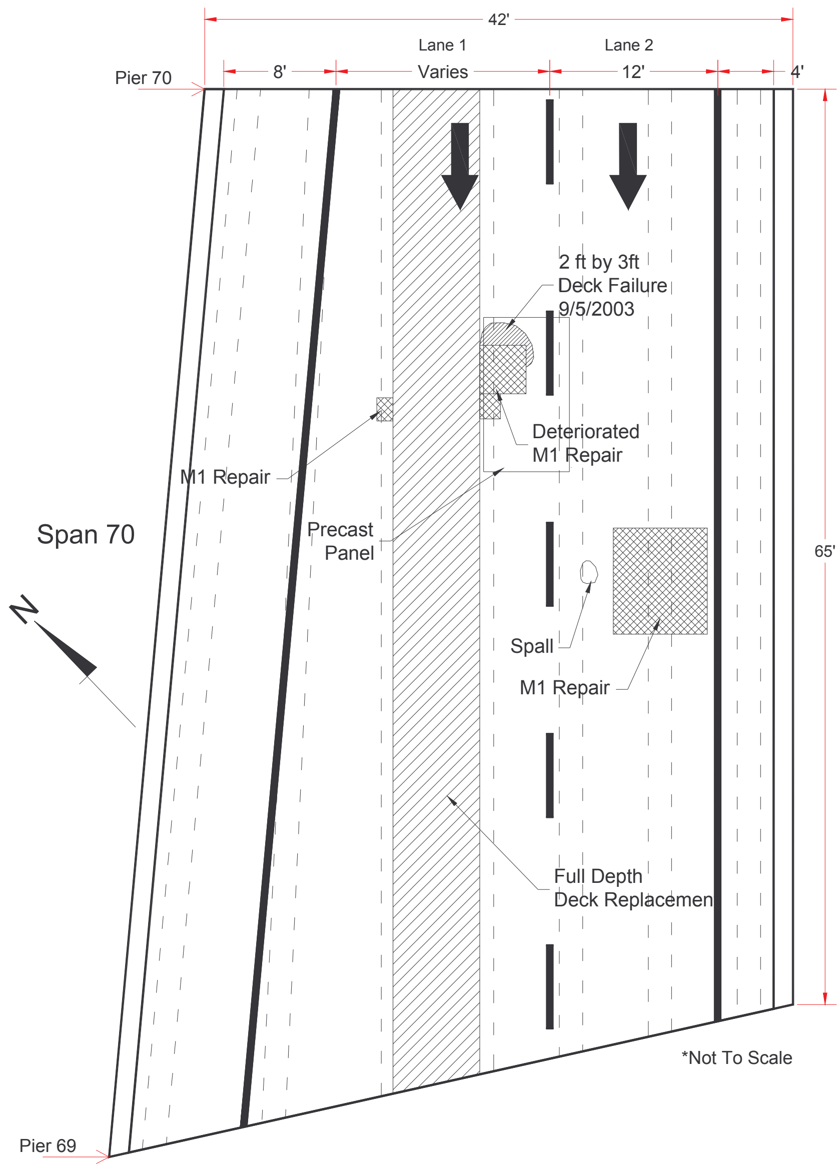


Figure 3.20 Location of Failed Panel, Bridge 100332 Span 70

The second article [3.37] published on September 10, 2003 with the headline “*Time Catches Up With Expressway*”. It stated that *“The 3-square-foot hole… was the site of an earlier temporary patch*”. Pat McCue, executive director of the local expressway authority was quoted as saying “*Truck traffic caused the layers to separate and crack in spots. Rainwater seeped into the cracks and, forced outward by the weight of traffic, crumbled the concrete, leaving a gaping hole*”. Ben Muns, the expressway authority's chief engineer was quoted as saying *"There's just no telling when the next one [hole] will be"*.

3.6.2 Analysis

3.6.2.1 Inspection Reports

The same five inspections reviewed for the previous failure in Span 38 describe the condition of the bridge over the eight year period from May ’93 to Aug. '01 [3.30-3.34]. As mentioned earlier, the reports provide information on the entire bridge and there is limited information relating to span 70 where failure occurred. Scanned excerpts from these inspection reports are summarized in Table 3.12 for completeness.

Additional monthy inspections (Table 3.13) noted that deterioration of the section that eventually failed was first observed on August 12 2003. It was described as a new 2' x 1' x 1" spall and delamination area with exposed steel. This had not been observed in the previous inspection carried out a month early on July 10.

Fig. 3.21 provides a photographic record of the events leading to failure. The first photo, A shows a 3 ft x 1 ft x 1.5 in spall that was observed 14 months prior to failure. The second photo, B, shows M1 repair carried out 8 months prior to failure. The last photo, C shows a spall developing ahead of the M1 repair taken 23 days before failure on August 12, 2001. The next picture in the sequence can be seen in Fig. 3.19 where the failed section can be seen.

Table 3.12 Excerpts from Inspection Reports (Bridge #100332)

	FDOT Bridge Inspection Report (Deck)
8/29/03	**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 2% or less of the deck area. — 231080 sf. ELEMENT INSPECTION NOTES: NOTE: This element quantifies the concrete deck w/precast concrete deck panels in spans 25 through 91 including span 34. The quantity has changed due to replacement of deck panels with CIP concrete and the addition of span 34. CS2: Deck panel deficiencies are listed below: Span 26, panel 2-4, east edge at beam 26-2, spall, no steel, 2in x 4in x 1/2in. Span 26, panel 3-5, east edge adjacent beam 26-3, spall with exposed wire, 12in x 4in x 1/2in. Span 39, panel 4-5, SW corner, spall w/exposed wire, 12in x 12in x 1-1/2in. Span 41, panel 4-7, NW corner, spall/delamination, no steel, 10in x 10in x 3/4in. Span 47, panel 1-7, NW corner, spall w/exposed wire, 10in diameter x 3/4in. Span 47, panel 5-7, NE corner, spall w/exposed wire, 12in diameter x 1in. Refer to photo 1. P3 WO

08/29/01

ELEMENT/ENV:98/4 **Conc Deck on PC Pane** **21588 sq.m.** ELEM CATEGORY: Decks/Slabs

CONDITION STATE (5)	DESCRIPTION	QUANTITY	RECOMMENDED FEASIBLE ACTION
2	Repaired areas and/or spalls/delaminations and/or cracks exist in the deck surface or underside. The combined distressed area is 2% or less of the deck area.	21588	1 Spalls & Delams

WORK ORDER RECOMMENDATION:

REPR SPLs in Spans 26 27 29 39 41 44 47 53 57 58 70 76 88 & 91. 0.3M3

NOTE: DO NOT ISSUE WORK ORDERS. THIS BRIDGE DECK PANEL DECK IS UNDER CONTINUOUS MONITORING AND W.O.'S ARE ISSUED ON A REGULAR BASIS. 11/20/01

ELEMENT INSPECTION NOTES:

NOTE: Previous quantity appears understated. Current quantity field verified.

This element quantifies the concrete deck with precast concrete deck panels in Spans 25 through 91 with the exception of Span 34.

Several deck panel undersides have transverse cracks up to 0.4mm wide in random locations. Refer to the Addendum for additional text.

There are several failing repairs and spalls in the deck top and deck panel undersides. Refer to the Addendum for additional text.

08/31/99

ELEMENT/ENV:98/4 **Conc Deck on PC Pane** **19328 sq.m.** ELEM CATEGORY:Decks/Slabs

CONDITION STATE (5)	DESCRIPTION	QUANTITY	RECOMMENDED FEASIBLE ACTION
2	Repaired areas and/or spalls/delaminations and/or cracks exist in the deck surface or underside. The combined distressed area is 2% or less of the deck area.	19328	1 Spalls & Delams

WORK ORDER RECOMMENDATION:

Repair the spalls in Spans 25,26,39,43,44,47,57,58,69,70,74,76,87,88 and 91.

ELEMENT INSPECTION NOTES:

Spans 25-91(Except 34) - The concrete deck has longitudinal, transverse and map cracking throughout. These cracks are up to 0.4mm wide. Some of these cracks have edge spalling. Surface abrasion is throughout the deck exposing the aggregate. Voids are in the concrete deck where the aggregate is missing. Spalling is along the tyning grooves throughout the deck. The deck has minor popoffs due to the removal of the roadway reflectors. Transverse cracks meander along the full length of the construction joints. Edge spalling is at the expansion joints, some have been repaired with nosing compound. Previous noted spalls, areas of reinforcing steel, and some areas of cracking have been patched. The patched areas seem solid and well bonded when sounded with a hammer. Many of the patched areas have transverse and longitudinal cracks with corrosion stains, reflecting the underlying of reinforcing steel. Random cracks up to 0.6mm wide, and spalls, some with short lengths of exposed reinforcing steel, are on the underside of the precast panel forms. The spalls are typically 150 mm to 500 mm in length or diameter and appear to be the result of corrosion of the reinforcing strands or bars. Diagonal cracks up to 0.2mm wide, some with efflorescence are on the undersides of the overhangs, predominantly at the joints. Refer to the Addendum report for specific deficiencies listed by span number and photos of the deficiencies. Corrective action was recommended and not completed on this element.
WO - Repair any spalls with exposed steel or any failing repairs in Spans 25,26,39,43,44,47,57,58,69,70,74,76,87,88 and 91.

Span-Unit

70-3 **The left anchor bolt for Beam 70-7 at Pier 71 is broken and the nut is missing. The right anchor bolt in the same location is also missing. Panel 2 in Bay 70-5 has a spall 0.08 m X 0.03 m X 0.01 m. A repaired area with transverse cracks up to 1 mm wide and a spall 0.16 m X 0.16 m X 0.03 m with exposed steel is present in this span. (See Photo D-27)**

08/26/97

G1.01 DECK (TOP)

There are longitudinal, transverse and map cracks throughout the deck top. At the construction joints, Class 1 to Class 2 transverse cracks typically meander along the full lengths of the joints. Edge spalling, some of which has been repaired with nosing compound is present at the expansion joint edges. Previously noted spalls, areas of exposed reinforcing steel, and some areas of cracking have been patched. The patched areas, when sounded with a hammer, seem solid and well bonded. For many of the patched areas, there are longitudinal and transverse cracks, some with corrosion stains reflecting the underlying reinforcing steel. Refer to photos 1 and 2 on pages 17 and 18 for a typical view.

05/15/9

G1.01 Deck (Top) NCR:6
All decks have distinct longitudinal and transverse class 1 cracks. The cracks vary in length from a few meters to the end tie width or length of the span. Class 2 and greater cracks with exposed rebar and other significant deficiencies are recorded in table 1 on pages 24 thru 27.

SPAN	DEFICIENCY	LANE	LOCATION
70	Class 1 spall with exposed rebar and one incipient spall	South	Near midspan
70	Eight class 1 spalls with exposed rebar	South	
70	One class 2 longitudinal crack entire length of span	North	

05/20/93

G1.01 Deck (Top)
All decks have distinct longitudinal and transverse Class 1 cracks. The cracks vary in length from a few feet to the entire width or length of the span. Class 2 and greater cracks, as well as spalls that are exposing rebar and spalls that are of significant size, are recorded on Table 1 on Pages 19 and 20. Generally, the most serious deficiencies were **spalls with exposed rebar and areas of honeycombed concrete.**

70	Class 1 spall with exposed rebar and one incipient spall	South	Near midspan
70	Eight Class 1 spalls with exposed rebar	South	
70	One Class 2 longitudinal crack entire length of span	North	

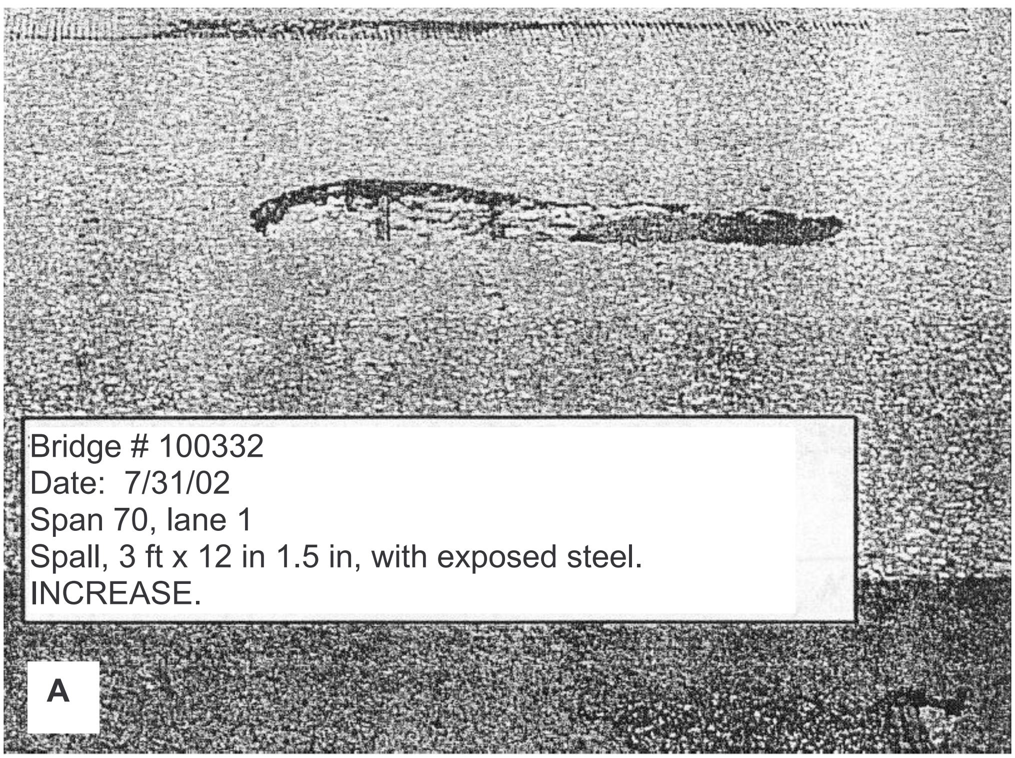


Figure 3.21 Deck Spall Bridge # 100332, Span 70. A) Initial spall 14 months before failure B) M1 repair over initial spall 8 months before failure, C) Spall next to the M1 repair, 23 days before failure.

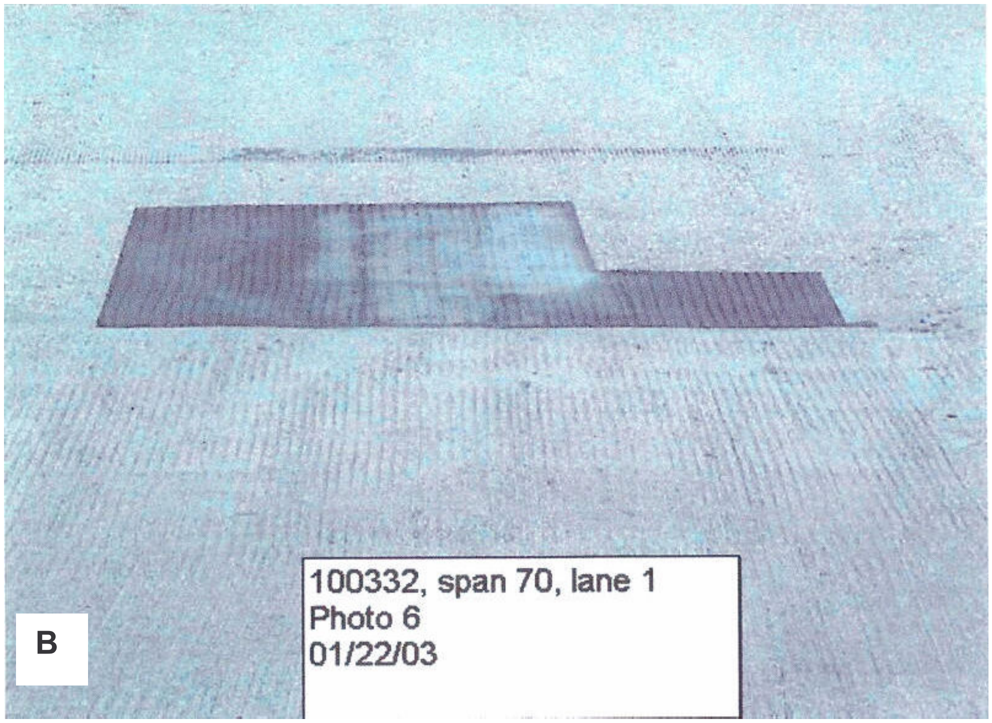


Figure 3.21 (continued) Deck Spall Bridge # 100332, Span 70. A) Initial spall 14 months before failure B) M1 repair over initial spall 8 months before failure, C) Spall next to the M1 repair, 23 days before failure.

Table 3.13 Excerpts from Monthly Inspection Reports (Bridge #100332)

Date	Bridge#	Span#	Deficiency	Feature Intersected	Photo
8/12/03	100332	70	Lane 1 span length full depth repair with perimeter RP - NC - STABLE	Hills. River/Downtown	N
			Lane 1, 2' x 1' x 1" SPL/DEL with RX S. side of full depth repair & E. of small RP - NEW		Y
7/10/03	100332	70	Lane 1 span length full depth repair with perimeter RP - NC - STABLE	Hills. River/Downtown	N
			Lane 2 two RP, trans. Crk. 1/16" wide - RP MADE TO PERIMETER 7/1/03		Y

No reference to the spot that failed

3.6.3 Environmental Conditions

The precipitation readings at Tampa International Airport (6 miles from the bridge) over a one month period prior to failure are shown in Fig. 3.22. Total rainfall one week before failure was about 1.1 inches. Two days before failure, rainfall of 0.8 in. was registered, 0.3 in. rain fell on the day of the failure. Thus, rain may have been a factor in degrading the concrete reinforcement bond that led to concrete pieces separating from the steel and creating a void in the deck. For the record, on the day of the failure, the temperature varied from a minimum of 74°F to a maximum 79°F.

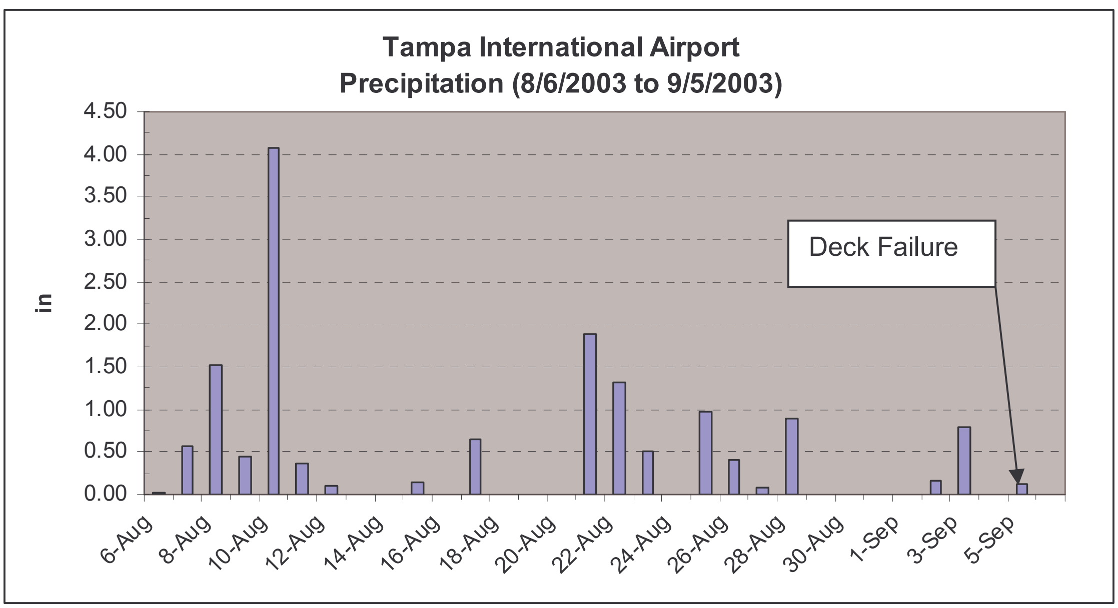


Figure 3.22 Tampa International Airport Precipitation (Aug 6 – Sep 5 2003).

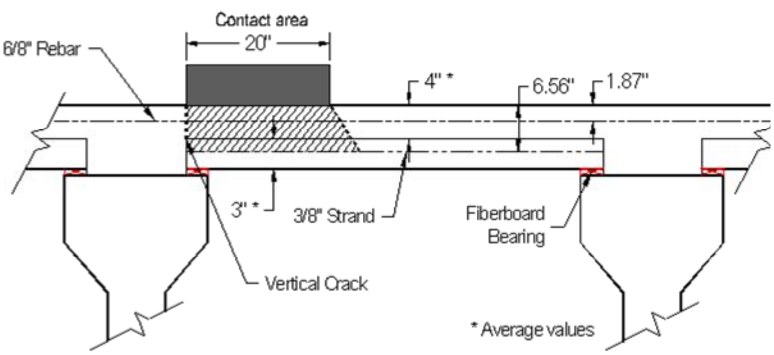
3.6.4 Punching Shear Analysis

The localized failure occurred within the panel. Consequently, two-way shear resistance was provide by three edges Table 3.14 summarizes the calculated punching shear values for the two extreme cases - full composite and panel slab only. It may be seen that the value of the failure load is higher in this case (21.7 kips vs 15.3 kips, Table 3.4). A photograph of the underside of the panel shows water damage and longitudinal cracking within the assumed region providing resistance. Thus, assumption of support from three surfaces is perhaps on the optimistic side in this situation.

Table 3.14 Punching Shear Resistance Bridge # 100332 Span 70

<table>
<tr><th>Load Case (Panel Edge)</th><th>Punching Shear Resistance*</th></tr>
<tr><td>

Full Composite Action



Tire Contact area: b=20in l = 10 in

</td><td>

CIP

d_e = 4 in.
b_0 = 58 in.
V_{CIP}= 50.8 kips

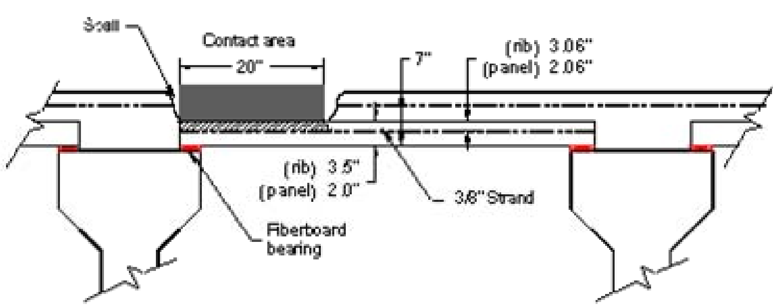
PANEL

d_e = 2.56 in. (ave)
V_{PANEL} = 41.4 kips

V_{TOTAL} = 92.2 kips

</td></tr>
<tr><td>

No Composite Action



Tire Contact area: b=20in l = 10 in

</td><td>

CIP

V_{CIP}= 0 kips

PANEL

d_e = 2.06 in. (min)
b_0 = 54.12 in.

V_{PANEL} = 21.7 kips

V_{TOTAL} = 21.7 kips

</td></tr>
</table>

* See Appendix A for detailed calculations.

3.6.5 Conclusions

Biennial inspection records were of limited value. However, monthly inspection records for this bridge provides a photographic record of the sequence in which failure occurs (see Fig. 3.21 and Fig. 3.19). Failure was in regions that had been re-repaired. Punching shear failure loads assuming resistance was provided from three surfaces overestimated the failure load. The condition of the underside of the bridge, especially if it shows signs of water stains may indicate impending localized failure. For this bridge, rainfall was a contributory factor as there was a fair amount of rain just prior to failure (see Fig. 3.22).

3.7 Summary and Conclusions

This chapter provided detailed information on five localized failures in deck panel bridges that occurred over the period between February 2000 and September 2003. These occurred at two locations - Sarasota and Tampa. One other failure was mentioned in the local newspaper (Section 3.2.1.1) in bridge #170146 but no records of this could be found. A survey was also conducted to find out about the performance of deck panel bridges in other districts. No failures had occurred in District 2. District 4 reported failures in two bridges - 940126 I-95 SB over Florida turnpike and 940127 I-95 NB over Florida turnpike but no details were provided (see Appendix B).

The primary goal of this chapter was to identify underlying trends that led to failure that could be incorporated in a rational prioritization scheme. To this end, attention was focussed on where failures occurred, inspection and environmental information. The principal conclusions are summarized below:

3.7.1 Failure Trend

National Bridge Inventory deck condition rating (Table 3.15) was found to be a poor indicator for predicting deck panel failures. All bridges that failed were rated between 5 (satisfactory) to 7 (good). Inspection records give a periodic snapshot on the condition of the bridge. Whereas biennial inspection data were generally unable to predict failure, monthly inspection records were far more successful in tracking problems that led to failure (see Table 3.15, Figs. 3.21/3.20). Based on the information provided in the inspection records for the five failures, the sequence leading to failure may be summarized as shown in Fig. 3.23.

Figure 3.23 Simplified Deck Deterioration Process.

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 (Tables 3.3, 3.6, 3.9) after which there is more widespread transverse cracking.

Longitudinal and transverse cracking result in spalling, delamination that require repair. In most cases, such damage occurs in regions where the panel is improperly supported on fiberboard. Depending on the materials and quality of the repair the deck can perform poorly or satifactorily. Where deck repairs are combined with proper panel bearing, e.g. by injecting epoxy, repairs are satisfactory. Where this is not carried out, and repairs are limited to surface repairs, there is progressive degradation (Fig. 3.21/3.20) which can lead to failure. In several instances, failures occurred at locations where temporary repairs had not been replaced.

Simplified calculations show that punching failures could result at loads below the design wheel load. This assumed the cast-in-place deck to provide no resistance and the panel to be supported on fiberboard with well developed cracking along the transverse and longitudinal panel boundaries. The failure load was calculated to be around 15 kips (Table 3.4). Otherwise, failure loads were nearly four times higher.

Table 3.15 Inspection Record

Bridge #	Conditon Rating	Last Inspection	# of Rainfall events in past 7 days	Comments
170146	6 (Satisfactory)	3 months	0	Not identified
170086	7 (Good)	6 months	2 (0.68 in)	Not identified
170085	7 (Good)	7 months	4 (0.2 in.)	Identified
100332	5 (Fair)	2 days	2 (0.55 in.)	Identified
100332	5 (Fair)	23 days	3 (1.1 in.)	Identified

3.7.2 Environmental Factors

In four out of the five cases there was rainfall prior to failure (Table 3.15). The most severe rainfall preceded the last failure (1.1 in.). Also, photos of the underside of the bridges that failed show water stains (see Figs. 3.7, 3.14, 3.19). The exact role of rainwater is not known. However, given that the concrete in the deck separates cleanly from the reinforcement (e.g. Fig. 3.19), it probably adversely affects bond and degrades the cohesiveness of the cement paste. Thus, it is reasonable to conclude that rainfall accelerates existing damage that can result in failure.

3.7.3 Failure Location

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 (Table 3.16). Failure was generally in the edge or corner panels whose boundaries developed reflective longitudinal and transverse cracking.

Table 3.16 Failure Comparison

Bridge #	Year Built	Age at Failure (yrs)	ADT (ADTT)	Failure Size	Location in Panel	Comment
170146	1981	19	34,000 (30%)	18 in x 24 in	Edge or Corner?	Failure at M1 repair
170086	1980	20	34,000 (30%)	36 in x 60 in	Corner Support	Patch repair
170085	1980	20	34,000 (30%)	18 in x 18 in	Corner	Failure adjacent to M1 repair
100332	1980	22	23,000 (8%)	48 in x 30 in	Near corner	Asphalt Patch
100332	1980	23	23,000 (8%)	24 in x 36 in	Edge	Failed M1 repair with flexible patch material

** National Bridge Inventory condition rating given in the bridge inspection prior to the deck failure*

3.7.4 Bridge Characteristics

All failures occurred in bridges where the deck was nominally 7 in. thick. No failures occurred in deck panel bridges with thicker slabs. The ADTT varied between 8-30% (Table 3.16).

Also it may be noted that the failures occurred in two twin bridges (NB and SB - 170086, 170085), and in a bridge adjacent to these two (170146). It is very likely that these three bridges were built with similar defects by the same contractor. The other two cases also occurred in the same bridge (100332 spans 38 and 70).

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3.32 Florida Department of Transportation. (1997)." *Bridge Inspection Report Bridge No 100332*", dated Aug 26.
3.33 Florida Department of Transportation. (1999)." *Bridge Inspection Report Bridge No 100332*", dated Aug 31.
3.34 Kisinger Campo & Associates. (2001)." *Bridge Inspection Report Bridge No 100332*" , dated Aug 29.
3.35 Luna M. *Personal Communications* (2003).
3.36 Sloan, J. (2003). *"Small Hole Paves Commuters' Way To A Traffic Jam"*. Article dated Sep. 9, Tampa Tribune.
3.37 Sloan, J. (2003). "*Time Catches Up With Expressway*". Article dated Sep. 10, Tampa Tribune.

4. FORENSIC INVESTIGATION

4.1 Introduction

In the previous chapter, five reported failures were investigated with a view towards identifying underlying trends that could be used to predict future failures. This chapter describes on-site investigations that were carried out to pursue the same objective: to gain enhanced understanding of the degradation process. In the study, several precast deck panel bridges scheduled for replacement during 2003-2004 and located within easy driving of the USF campus were investigated. A list of these bridges is given in Table 4.1.

Table 4.1 Forensic Studies

Bridge #	District	Built	Study Date	Bridge Location
130078	1	1981	6/03	I-75 SB over Moccasin Wallow Rd (Manatee County)
130079	1	1981	6/03	I-75 NB over Moccasin Wallow Rd (Manatee County)
170140	1	1981	1/04	I-75 NB over Toledo Blade Blvd (Sarasota County)
130075	1	1981	5/04	1-75 SB over CSR R/R (Manatee County)
100415	7	1983	6/04	I-75NB over US 92 (Hillsborough County)
100398	7	1984	6/04	I-75NB over Sligh & Ramp D-1 (Hillsborough County)
100417	7	1983	7/04	I-75NB over Ramp B-1 (Hillsborough County)
130085	1	1981	8/04	I-75NB over SR-64 (Sarasota County)

The bridges included in this forensic study were scheduled for a complete deck replacement for a variety of reasons not necessarily related to the state of disrepair. As a result, both badly deteriorated and those not so badly deteriorated decks were investigated. This made it possible to investigate the condition of the decks at different stages of deterioration.

The aim of the investigation was to compile a photographic record of the deterioration that could be used in developing a rational failure model. Forensic inspection methods were designed to obtain maximum information with minimal disruption to the contractor. The specific information of interest is summarized in Section

4.2. Self-standing sections relating to each bridge in Table 4.1 are presented in Sections 4.3-4.10. Section 4.11 describes a degradation model that was developed incorporating all the information compiled in this chapter. The main conclusions are summarized in Section 4.12.

The investigations reported could not have been carried out without the cooperation and unconditional assistance of the deck replacement contractors: Zep Constructions Inc. and AIM Engineering & Surveying.

4.2 Objectives

The main objective was to obtain first hand evidence of actual deck deterioration in order to get a better understanding of how deficiencies are initiated and how they propagate in typical deck panel bridges.

Specific information of interest was for identifying conditions that resulted in:

- Decks with no cracking.
- Longitudinal surface cracks.
- Transverse surface cracks.
- Deck surface spalling including “walking” spalls.
- Deficient M1 repairs.
- Underside longitudinal and transverse panel cracking.
- Condition of fiberboard bearing.
- Effect of epoxy panel bearing.
- Effect of different wheel locations.

Not all the information could be retrieved from a single bridge given that they were in different states of disrepair. In the sections that follow the same basic format will be followed: a description of the bridge that was replaced followed by the inspection method used and the principal findings.

4.3 I-75 NB and SB over Moccasin Wallow Rd. (Bridges #130079, #130078)

The replacement of the deck in these twin bridges was carried out in June 2003 by Zep Constructions. In three weeks, the existing deck panel was removed and replaced by a full-depth cast in place concrete slab. In all a deck area of 35,680 sq. ft was replaced.

4.3.1 Bridge Details

The I-75 NB and SB bridges over Moccasin Wallow in District 1 are located in Manatee County, a few miles north of the I-75 - I-275 intersection. These 3-span bridges were built in 1981 and were in service for nearly 23 years before replacement.

Each bridge has two approximately 100 ft. long main spans (*span 2, span 3*) and two 45 ft long secondary spans (span 1, span 4). The total length is about 290 ft.

In the north bound bridge, the shorter spans were built using two AASHTO Type IV girders on the outside and six AASHTO Type II girders on the inside all spaced 9 ft 3 1/2 in. apart. For the main span, nine AASHTO Type IV girders are spaced at 8 ft 1 1/2 in on centers as shown in Fig. 4.1. In the south bound bridge, the shorter spans use two AASHTO Type IV girders on the outside and five AASHTO Type II girders on the inside all spaced 8 ft 10 in. apart. In the main span, seven AASHTO Type IV girders are spaced at 8 ft 10 in on centers.

The deck had a 7.5 in. thick concrete slab with the precast panel component being either 2-½ in. or 3-½ in. (at the rib-section) thick as shown in Fig. 4.2. The specified compressive strength of concrete for the precast panel was 5,000 psi. It was 3,000 psi for the cast in place concrete slab.

The bridge has three 12 ft wide lanes, and 10 ft wide shoulders as shown in Fig. 4.1. There is an auxiliary lane that merges with traffic entering the interstate from I-275. These dimensions and the bridge cross-section are typical of all deck panel bridges in Districts 1 and 7 excepting that the deck thickness (7.5 in.) is slightly greater than the 7 in. norm.

In general the deck was in reasonable condition in both bridges with typical longitudinal and transverse cracking. Some regions had deteriorated and both M1 Repairs and spalling were present.

Table 4.2 Bridges #130078 and #130079 [4.1]

	Bridge #130078 (SB)	**Bridge #130079 (NB)**
Year Built	1981	1981
Number of Spans	4	4
Lanes on Structure	3	4
ADT (2003)	26,500	27,000
Percent Truck (ADTT)	30%	30%
Composite Slab Thickness	7-½ in.	7-½ in
Precast Panel Thickness	2-½ in (panel) 3-½ in (ribs)	2-½ in (panel) 3-½ in (ribs)
Girder Type	AASHTO Type II and IV	AASHTO Type II and IV
Deck Condition Rating (2003)	7 (Good Condition)	7 (Good Condition)

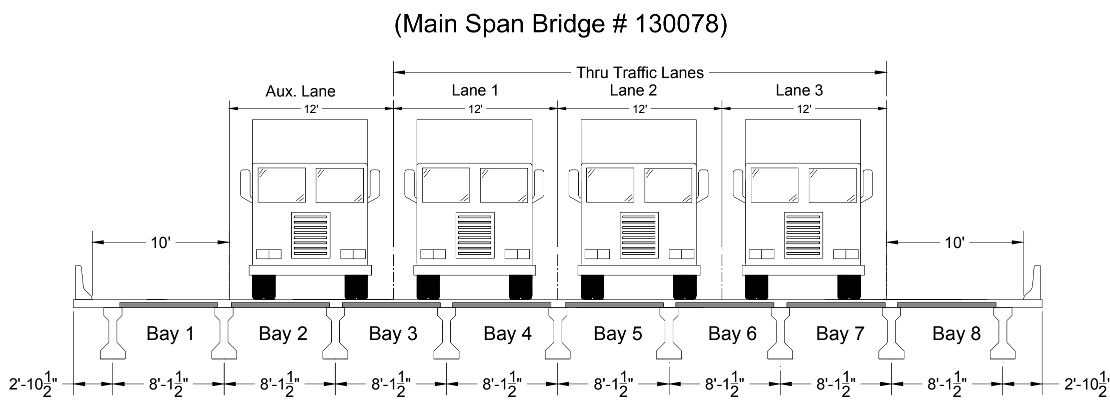


Figure 4.1 Cross Section View of Bridge #130078

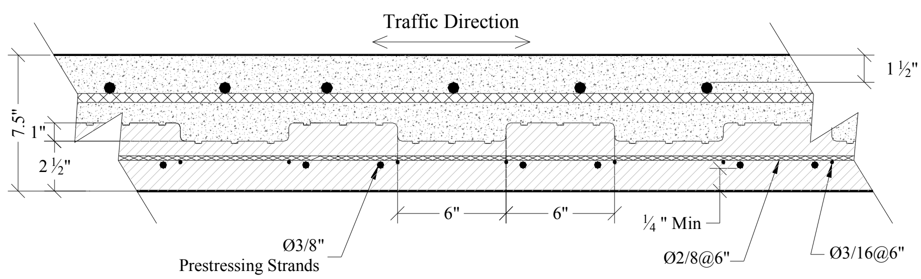


Figure 4.2 Composite Deck Section

4.3.2 Inspection Method

As several panels had already been removed when the USF research team was able to access the site (Fig. 4.3), three different procedures were to optimize the investigation. This involved (1) examination of already removed panels from the southbound bridge, (2) inspection of panels that had been identified prior to their removal and (3) inspection of panels in-place after adjoining panels had been removed. The last scenario provides the best information but is rarely possible since it can interfere with the contractor's work.

<u>Removed Panels</u>

A quick visual inspection was conducted to identify panels that exhibited typical deficiencies and a detailed report was done with all the information collected in the form of photographs, sketches and field notes. Care was taken to isolate existing deficiencies from those induced by the removal process.

Figure 4.3 Removed Deck Sections from SB Bridge #130079

<u>Marked and Removed</u>

This was undertaken for the east (right) half of the northbound bridge. Following a quick inspection of the deck, regions of special interest were identified (Fig. 4.4). These included sections with well defined typical deficiencies as well those with no apparent defects. A total of six panel sections were marked and removed. The average dimension of these sections was 8 ft by 10 ft.

The contractor removed the marked sections taking extra care to minimize additional damage and then stored them at an assigned place for subsequent detailed inspection.

Figure 4.4 Marked Deck Sections Removed From NB Bridge#130078

Inspection of these marked sections included detailed visual examination, crack survey of the deck surface and the cross section, and extraction of concrete cores (Fig. 4.5) from locations of special interest. A total of 15 cores were taken from the 6 deck sections. Appendix C has detailed information on these deck cores.

Figure 4.5 Coring of Marked Sections

<u>Insitu Examination</u>

To eliminate any doubt that the crack patterns were induced by the removal process, insitu inspection was instituted where deck sections were examined prior to their removal. This provided authentic information on crack propagation through the thickness of the deck. As mentioned earlier, this was possible when adjacent sections had already been removed to allow access to the vertical faces of the section. This inspection confirmed that the condition of the deck deficiencies was unaffected by the removal process.

4.3.3 Findings

Fig. 4.6 is a schematic drawing highlighting some of the findings. It provides details of their location in the deck cross section and also cross-refers to figure numbers where photographs of the particular deficiencies are provided.

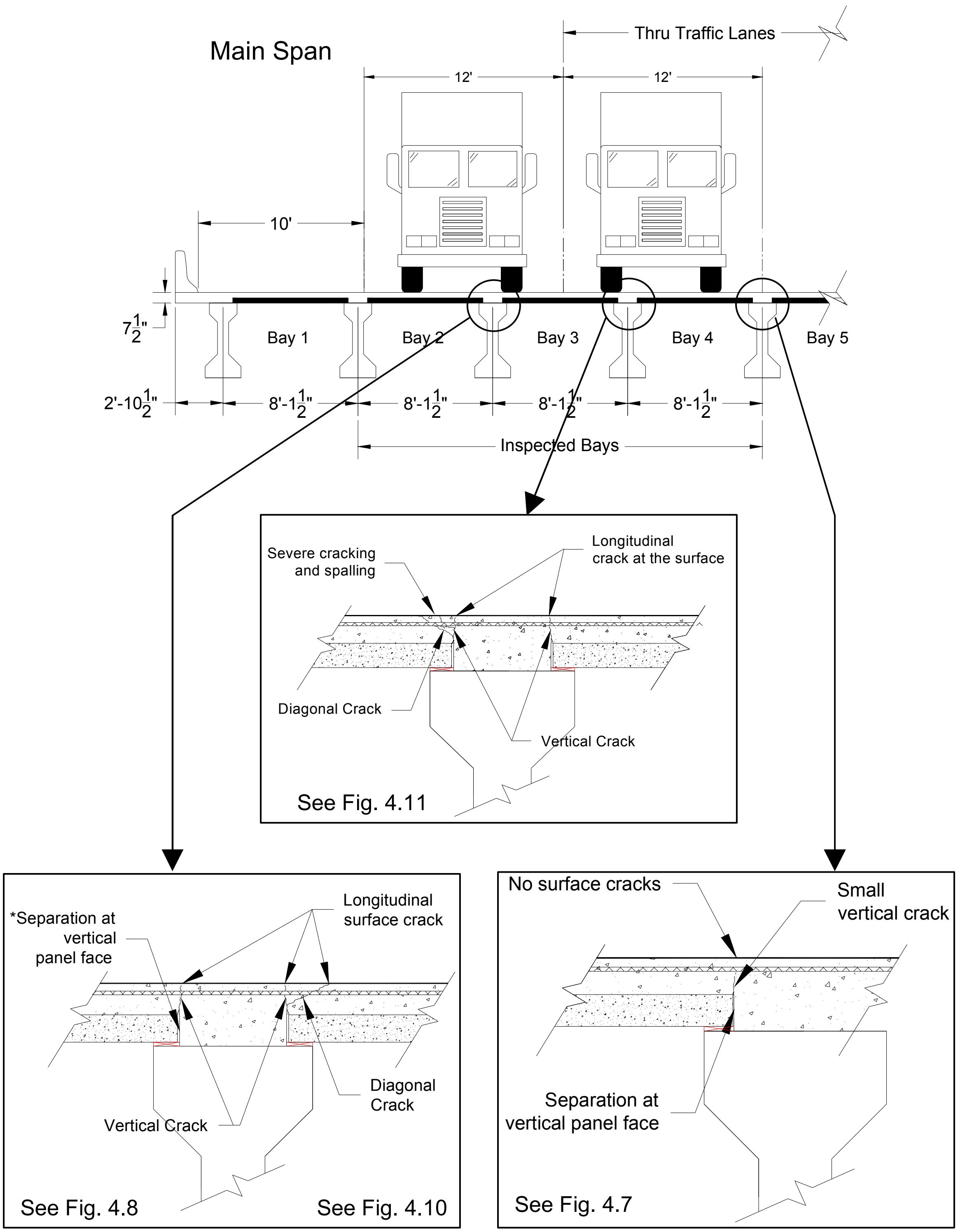


Figure 4.6 Overview of Findings from Bridge # 130079

In the following sections, detailed information is provided for each of the following findings some of which are shown in Fig. 4.6. These are:

1. No Deck Surface Cracking
2. Longitudinal Deck Surface Cracking
3. Transverse Deck Surface Cracking
4. Additional Longitudinal Cracking
5. Deck Spalling and Delamination.

4.3.3.1 No Deck Surface Cracking

Fig. 4.7 shows a retrieved panel with no surface cracking. The location of the prestressed girder support and the bearing pad has been drawn to provide better understanding.

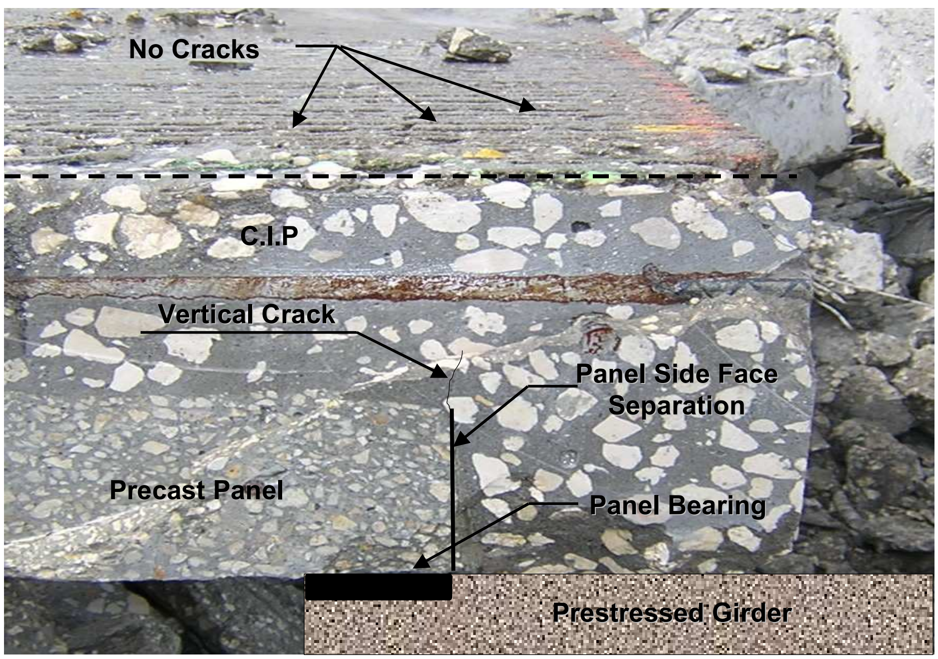


Figure 4.7 No Deck Surface Cracking

Inspection of Fig. 4.7 shows that there is separation of the precast panel from the cast-in-place concrete slab possibly due to long term differential creep and shrinkage movement. This separation is about 1 mm wide.

There is a vertical crack emanating from the corner of the panel that does not propagate all the way to the deck surface. This could be because the effect of creep and differential shrinkage was lower for this case, e.g. lower effective prestress, smaller age difference between casting of the panel and cast-in-place (CIP) deck.

4.3.3.2 Deck Surface Longitudinal Cracking

This is the most common deficiency observed in precast deck panel bridges found in almost all the deck panel bridges.

Fig. 4.8 shows how a typical longitudinal crack develops. This picture was taken with the panel in place in the bridge after the adjoining panel had been removed. The prestressed girder shown is the actual girder which supported the panel. The fiberboard bearing support is also visible.

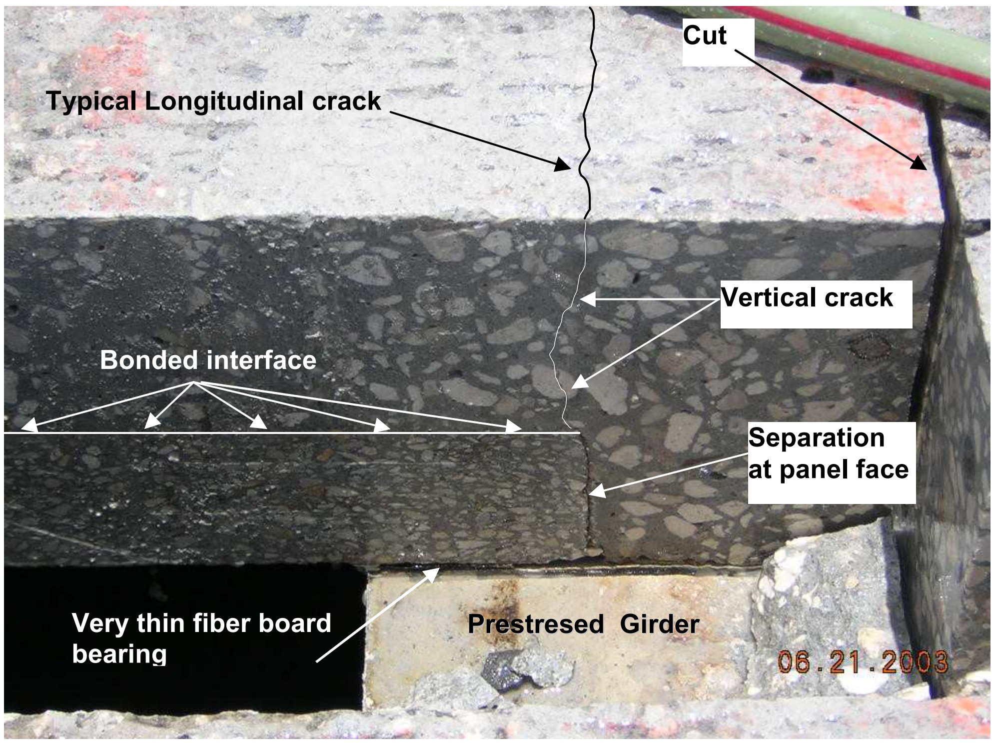


Figure 4.8 Development of Deck Surface Longitudinal Crack

Inspection of Fig. 4.8 shows clear separation of the vertical interface between the panel and the cast in place slab, i.e. the face of the panel completely debonded from the cast in place concrete. A vertical crack emanates from the top corner of the precast panel and propagates to the top of the deck. This pattern is replicated along the entire edge of the panel creating reflective longitudinal cracking on the deck surface.

It is important to recognize that this type of cracking can even be found on the shoulders of the bridge where live load is minimal. Thus, this type of cracking is not related to live load.

Also for this case and for all the sections inspected it was found that a very good bonded interface existed between the top face of the panel and the cast in place concrete. This indicates composite action under bending loads.

4.3.3.3 Deck Surface Transverse Cracking

Transverse deck surface cracking is not as common as longitudinal cracking. In most cases this is a hairline crack and it tends to remain stable without causing any further damage. In the forensic examination it was only detected in sections that were removed (Fig. 4.9).

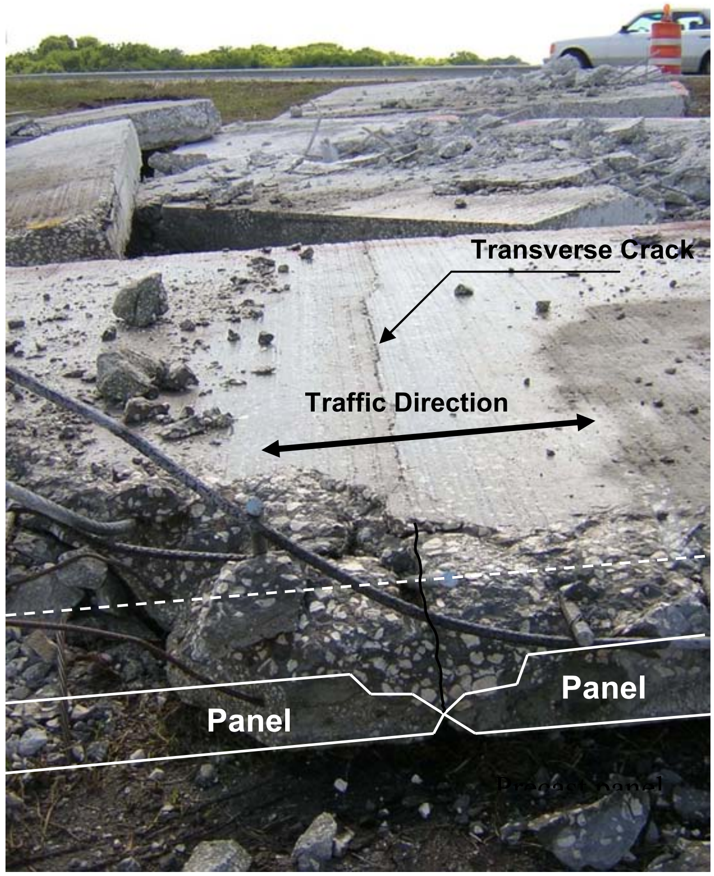


Figure 4.9 Deck Surface Transverse Crack

In Fig. 4.9 the location of the two adjoining panels has been drawn to provide better understanding. Cracking emanates at the joint and eventually propagates to the deck surface. Thus, it is a reflective crack that maps the location of the transverse panel joint on the deck surface. Where it does not reach the top surface, no cracking is visible.

4.3.3.4 Additional Longitudinal Cracking

In addition to the typical longitudinal crack running over the edge of the panels (See 4.3.3.2) another type of longitudinal cracking was found. This crack runs about 4 in. parallel to typical longitudinal cracks (Fig. 4.10).

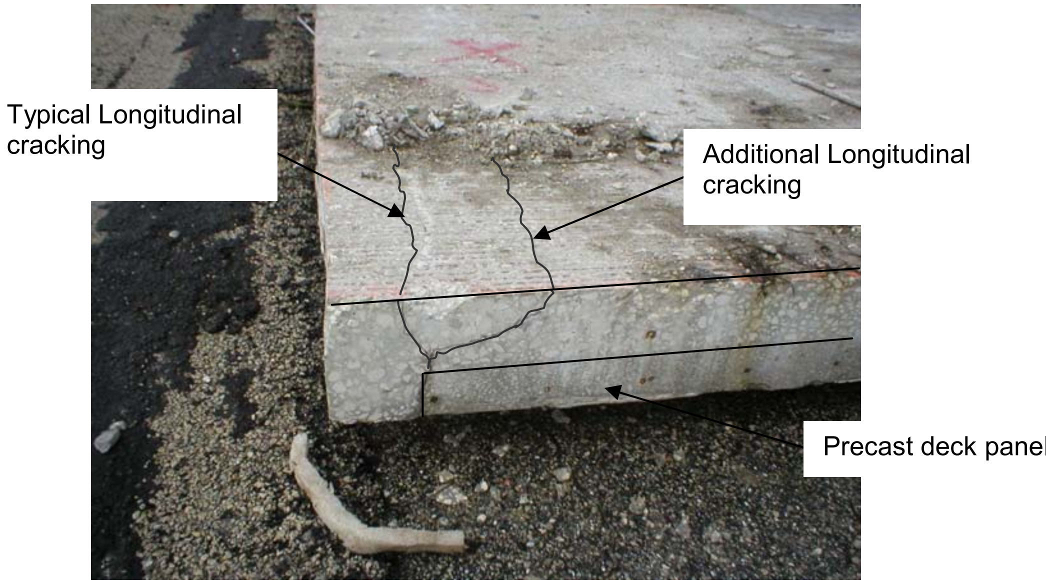


Figure 4.10 Additional Longitudinal Cracking

This additional longitudinal cracking is caused by a divergence of the vertical crack emanating from the corner of the precast panel. It propagates at an angle of less than 45 degrees to reach the deck surface, generating an additional longitudinal crack on the deck surface.

This type of crack is not as common as the typical longitudinal crack; it is only found in localized regions of the deck whereas other cracks tend to occur along the entire span. Also it was found that this additional cracking only occurs when a wheel load is located close to the panel support (see Fig. 4.6).

4.3.3.5 Deck Spalling and Delamination

This is one of the most important deficiencies in deck panel bridges. In the previous chapter examples are provided where sudden localized deck failures occurred at sites where temporary spalling repairs had been carried out.

The deck section analyzed was removed from span 3, bay 3 from the north bound bridge (see bridge cross section detail, Fig. 4.6). The spall was located right under the wheel load with the wheel load positioned at the edge of the girder.

In this specific case, the spalled area studied was located next to an existing M1 repair. This is a common deficiency in deck panel bridges; it is also known as a “walking spall” because it always occurs next to a spall patch or repair.

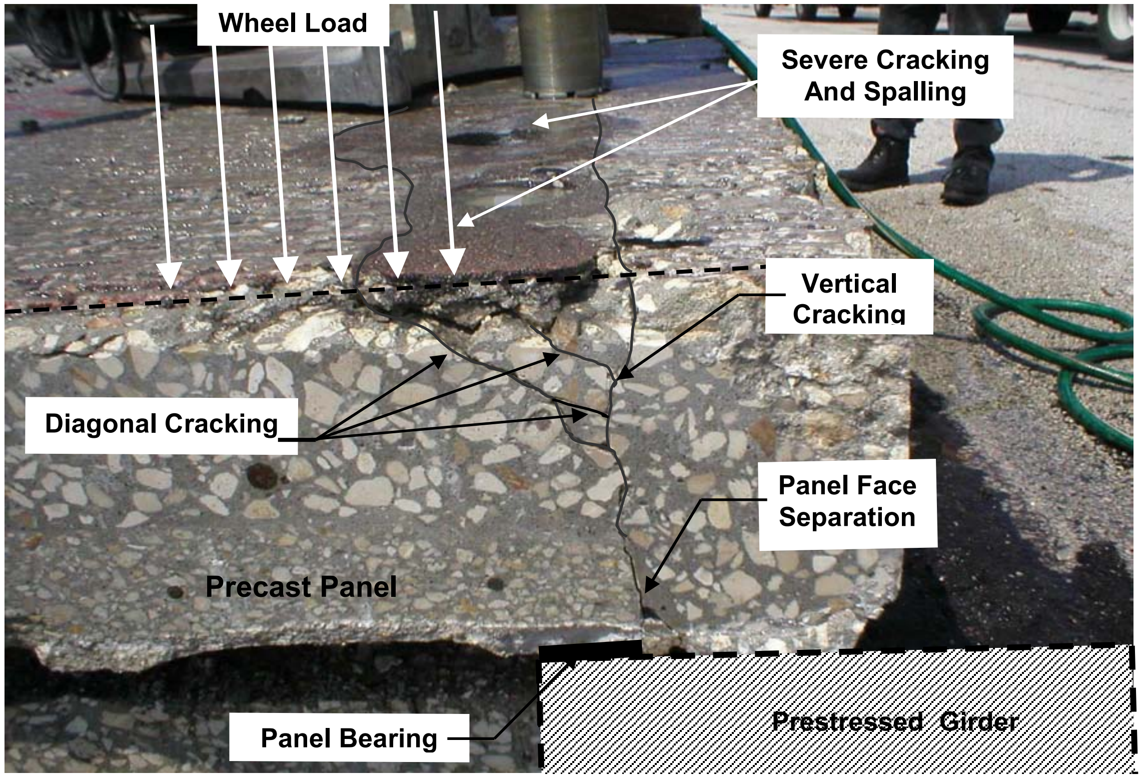


Figure 4.11 Development of a Deck Surface Spall

Fig. 4.11 is a photograph of the retrieved panel. A prestressed girder is drawn to provide contextual reference. Inspection of Fig 4.11 shows that it has all of the cracks described earlier, i.e. panel separation, vertical crack, diagonal crack, but with an increase in width of the cracks and additionally more diagonal cracks under the spalled area.

The longitudinal and diagonal cracking causes the concrete surface to break up into small pieces that can be easily detached from the deck by traffic creating the spall. Deck deterioration starts to accelerate due to the impact of the wheel loads on the spall.

4.3.4 Findings on Panel Bearings

Regarding the precast panel's bearing, it was found that the panels were supported on fiberboard (referred to as "negative bearing"). However, in some areas of the bridge, the fiberboard bearing had been removed and replaced by epoxy. This replacement had been recommended in a previous research study [4.3] as a method of reducing future deterioration of the bridge deck. It was found that major deterioration had occurred at exactly the same spots where the fiberboard had not been replaced by epoxy.

4.3.5 Findings on Core Examination

Most of the cracks found on the cores showed signs of water and dust infiltration (Cores 1-3 [Fig. 4.12], 1-4, 5-4, 5-6, 5-7 in Appendix C). In the case of vertical cracks, some of them showed these signs only over half the depth indicating that the prestressed slab was uncracked. But when the section deteriorated, infiltration occurred over the entire deck depth (Cores 5-6, 5-7).

In most cases (Cores 1-3, 1-4), concrete at the top of diagonal crack was crumbled and showed signs of water infiltration.

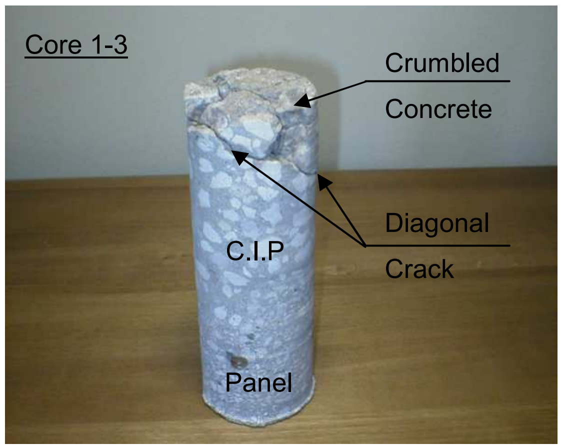


Figure 4.12 Crumbled concrete in top of a diagonal crack. (Core 1-3)

M1 repairs debonded only near the panel edges in the vertical direction as well as at its horizontal interface with the cast-in-place slab (Cores 1-1, 1-2, 5-4, 5-5). Along the longitudinal interface away from the panel edge there was no debonding (Core 5-3).

The depth of the deck, measured at each core location, varied from 7 ¾ in to 8 ½ in. See Appendix C for a detailed description of each core.

4.4 I-75 NB over N Toledo Blade Blvd. (Bridge #170140)

The deck replacement of this bridge was performed in January 2004 by Zep Constructions Inc. Fort Myers FL. At that time the bridge was widened and an additional 12 ft. lane added on the left side.

4.4.1 Bridge Details

This 3-span bridge located in Sarasota County, FL was built in 1981. Its deck was in service for 23 years before replacement. The bridge has a main span (*span 2*) of 107 ft 8 in. and two 41 ft secondary spans (*span 1, span 3*). Its overall length is 189 ft 8 in.

The shorter spans were built using two AASHTO Type IV girders on the outside and three AASHTO Type II girders on the inside all spaced 9 ft 3 in. apart. In the main span, seven AASHTO Type IV girders are spaced at 6 ft 2 in on centers as shown in Fig. 4.12.

The deck has a 7.5 in. thick concrete slab with the precast panel component being either 2-½ in. or 3-½ in. (at the rib-section) thick as shown in Fig. 4.2. The specified compressive strength of concrete for the precast panel was 5,000 psi. It was 3,000 psi for the cast in place concrete slab.

Table 4.3 Bridge #170140 [4.1]

Bridge #170140 Characteristics	
Year Built	1981
Number of Spans	3
Lanes on Structure	2
ADT (2003)	19,000
Percent Truck (ADTT)	30%
Composite Slab Thickness	7-½ in.
Precast Panel Thickness	2-½ in (panel) 3-½ in (ribs)
Girder Type	AASHTO Type II and IV
Deck Condition Rating (2003)	7 (Good Condition)

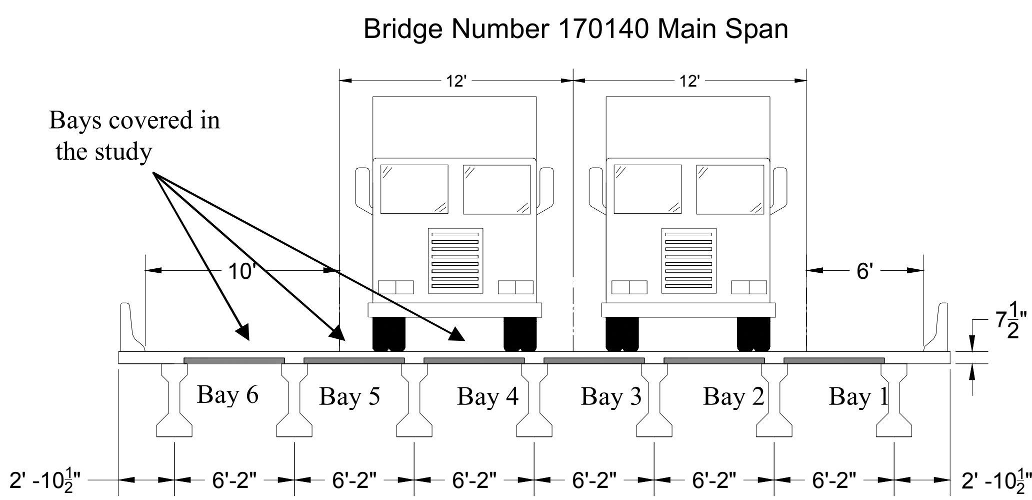


Figure 4.13 Cross Section View of Bridge #170140

The bridge has two 12 ft wide lanes, a 10 ft wide shoulder on the right of the traffic, and a 6 ft shoulder on the left, as shown in Fig. 4.13. It is in District 1 and is located 30 miles south of Sarasota.

The part of the bridge where the study was conducted was in apparent good condition. It only exhibited typical longitudinal and some transverse cracking. No previous repairs were found on the deck.

4.4.2 Inspection Method

The methodology used for this bridge was the same as the one used on the I-75 NB over Moccasin Wallow. Deck sections of special interest were marked for careful removal and subsequent detailed inspection (Fig. 4.14). However, no cores were extracted from the deck sections.

A) Marked section

B) Removed section

Figure 4.14 View of Bridge #170140

4.4.3 Findings

An examination of the retrieved panels confirmed the findings from the previous bridge (Fig. 4.15). Longitudinal cracks emanated from the corner of the prestressed panel and propagated through the slab thickness to emerge as visible cracks (Fig. 4.16a, 4.16b). Additional parallel cracking due to divergence of the crack emanating from the panel corner was also observed. However, the parallel cracking on the deck surface only appeared intermittently as shown in Fig. 4.16c.

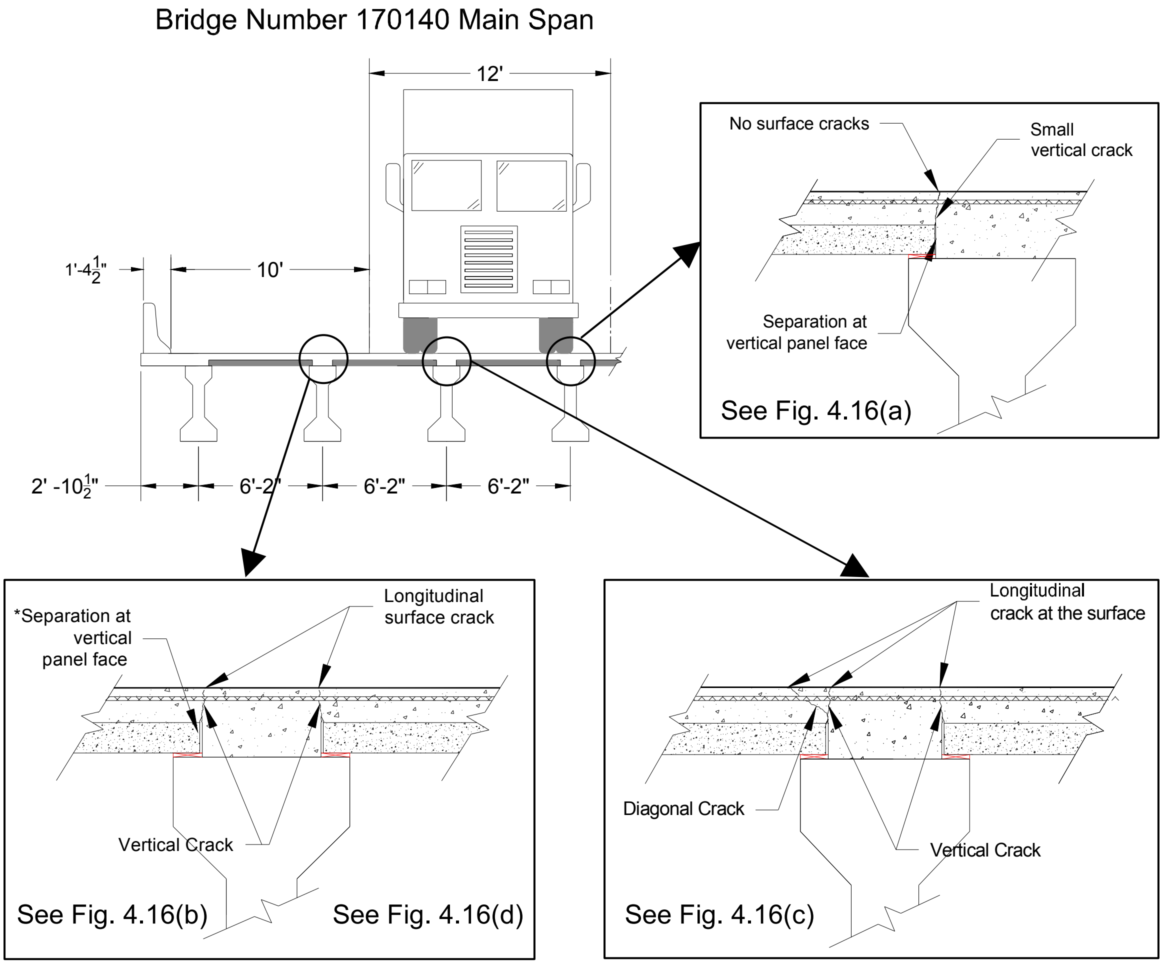
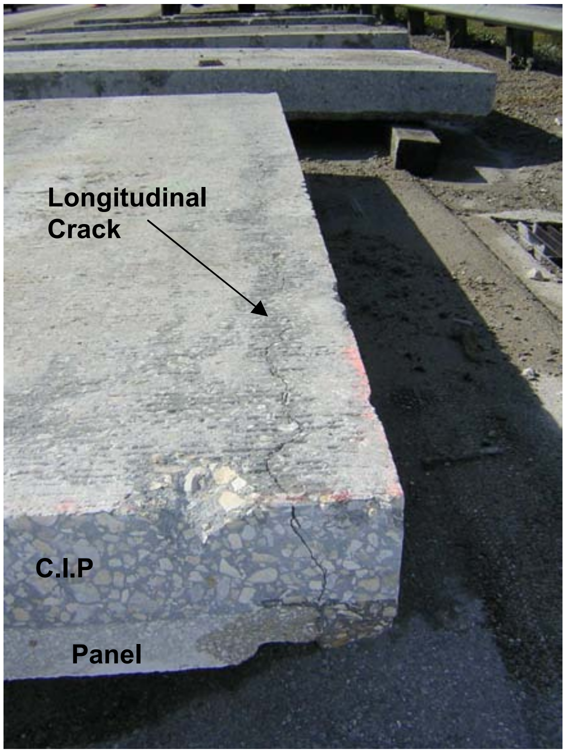
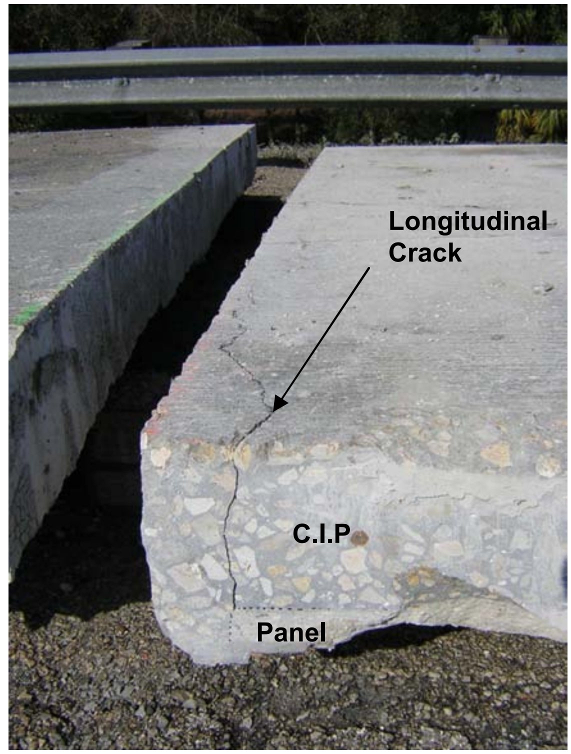


Figure 4.15 Findings Overview, Bridge # 170140

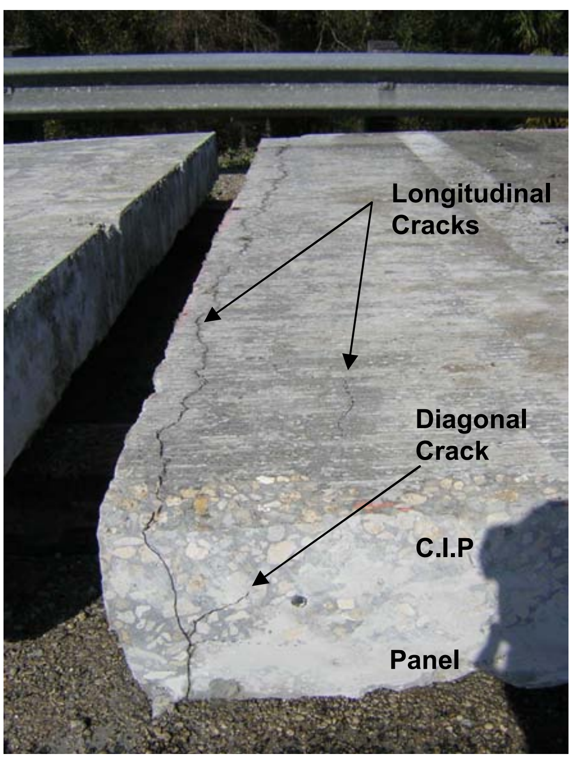
As before, there was separation between the precast panel and the cast-in-place slab at its vertical interface. This was suspected to be due to long term creep and shrinkage as stated earlier. An example was also found of the deck panel being supported by epoxy instead of fiberboard. In this instance, the extent of the longitudinal cracking was reduced (Fig. 4.16d). Overall, there were no dramatic new findings, simply confirmation of what was found earlier.



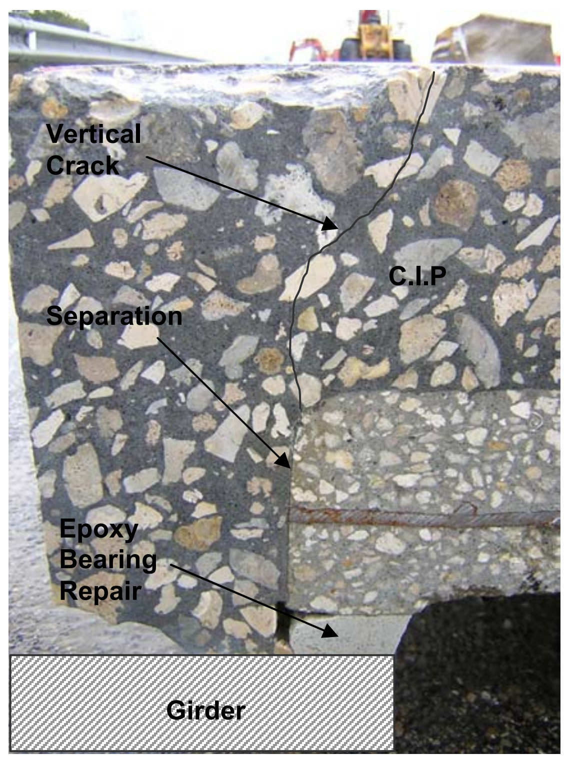
(a) Longitudinal Cracking



(b) Longitudinal Cracking



(c) Additional Longitudinal Cracking



(d) Epoxy Support

Figure 4.16 Retrieved Panels from Bridge #170140

4.5 I-75 SB over CSX R/R. (Bridge #130075)

The deck replacement of this bridge was performed in May 2004 by Zep Constructions Inc. Fort Myers FL.

4.5.1 Bridge Details

This 3-span bridge located in Sarasota County, FL was built in 1981. Its deck was in service for 23 years before replacement. The bridge has a main span (*span 2*) of 79 ft 2 in. and two 45 ft 5 in. secondary spans (*span 1, span 3*). Its overall length is 170 ft.

The shorter spans were built using two AASHTO Type III girders on the outside and five AASHTO Type II girders on the inside all spaced 8 ft 10 in. apart as shown in Fig. 4.16. For the main span, nine AASHTO Type III girders are spaced at 6 ft 7½ in on centers.

The deck has a 7.5 in. thick concrete slab with the precast panel component being either 2-½ in. or 3-½ in. (at the rib-section) thick as shown in Fig. 4.2. The specified compressive strength of concrete for the precast panel was 5,000 psi. It was 3,000 psi for the cast in place concrete slab.

Table 4.4 Bridge #130075 [4.1]

Bridge #130075 Characteristics	
Year Built	1981
Number of Spans	3
Lanes on Structure	3
ADT (2003)	36,500
Percent Truck (ADTT)	30%
Composite Slab Thickness	7-½ in.
Precast Panel Thickness	2-½ in (panel) 3-½ in (ribs)
Girder Type	AASHTO Type II and III
Deck Condition Rating (2003)	5 (Fair Condition)

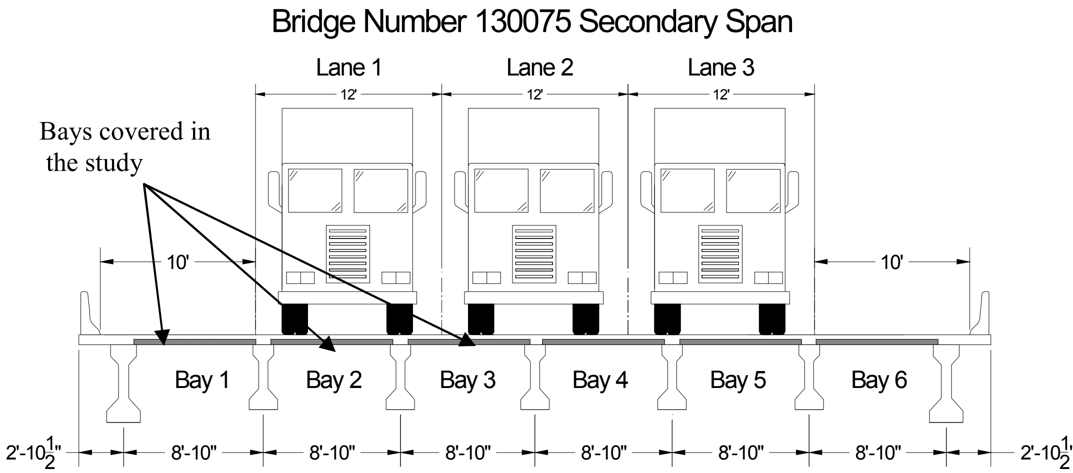


Figure 4.17 Cross Section View of Bridge #130075

The bridge has three 12 ft wide lanes, and 10 ft wide shoulders, as shown in Fig. 4.17. It is in District 1 and is located 2 miles north of Ellenton.

From the inspection performed before the deck removal, typical longitudinal and transverse cracking plus various M1 repairs, some of them stable and some unstable (Fig 4.18) were found.

Figure 4.18 Deck Overview of Bridge #130075

4.5.2 Inspection Method

This bridge was not part of the original investigation. Access was arranged at the last minute when much of the deck had already been removed. In view of this a different approach had to be employed.

In the modified approach there was no time for marking sections and then having them carefully removed by the contractor. Consequently, it was necessary to perform a quick inspection to locate and document major deck deficiencies. Following this inspection, each deck section was inspected in place after the adjoining section had been removed (Fig. 4.19a). Also deck sections that had been removed were also inspected, Fig. 4.19b.

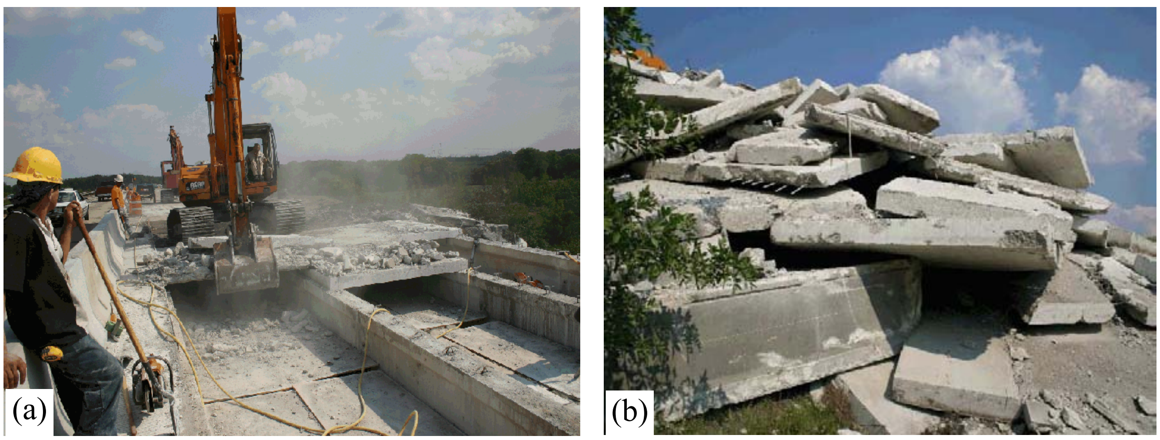


Figure 4.19 Inspection Methods Bridge #130075

4.5.3 Findings

As in previous examinations, panel face separation, and vertical cracking other typical cracking described in detail earlier were detected (see Fig. 4.20). New information relating to panel support was found.

Fig. 4.20 is a view of a section of the deck panel and the prestressed girder. The panel on the left is supported by epoxy while the section on the right is on fiberboard Thus, the replacement was partial and not over the entire deck as recommended in a previous research study [4.3]. Inspection of Fig. 4.20 shows that when epoxy was used to replace the fiberboard bearing it only penetrated over approximately one third the bearing width leaving a region that was unsupported (bay 3). Note the emergence of a vertical crack from this unsupported region. A similar crack appears from the edge of the fiberboard support on the right (bay 2). This was more heavily loaded and required an M1 repair. The divergence of the vertical crack caused separation of the interface between the M1 repair and the panel that cannot act compositely under flexural loading. The deterioration was more severe in bay 2 because of a combination of heavier loads and fiberboard supports.

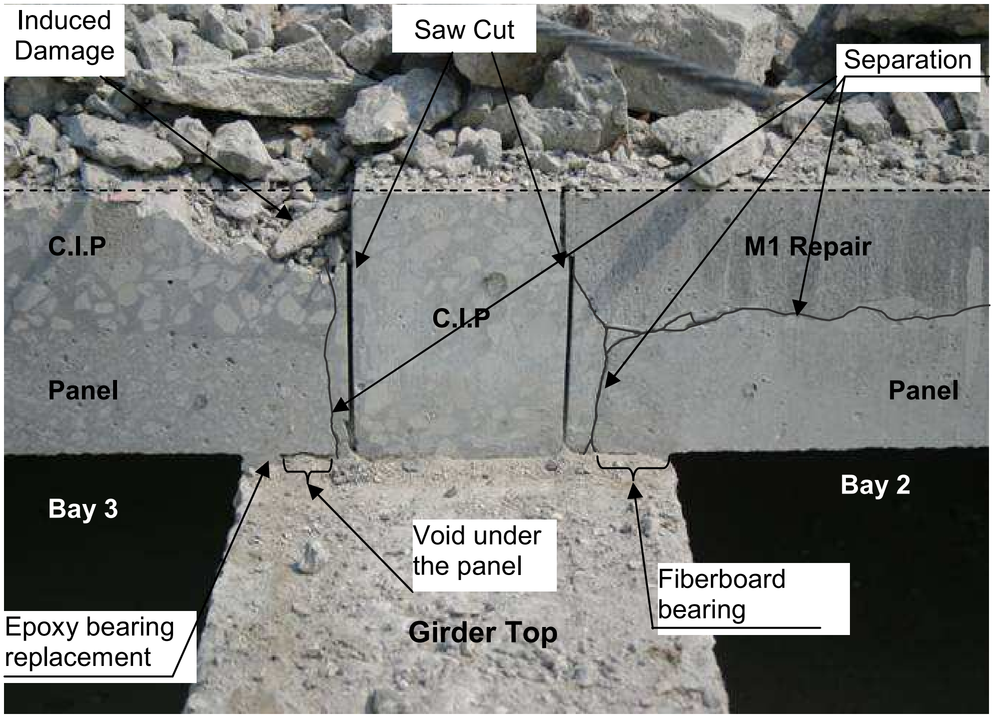


Figure 4.20 Deck Cross Section View over Girder # 3

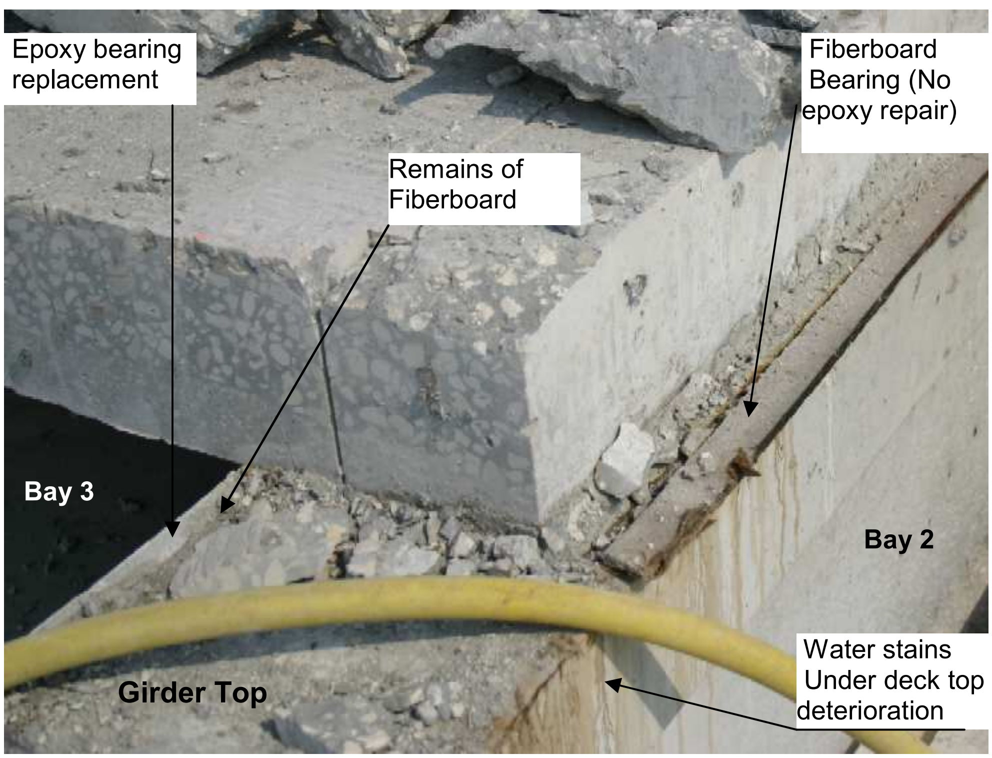


Figure 4.21 Panel Bearing Condition, over Girder # 3

Due to the examination of the deck sections before removal, it was possible to prove that the typical cracking found in this bridge and in previous cases was not caused by the deck removal process.

4.6 I-75 NB over US 92. (Bridge #100415)

The deck replacement of this bridge was performed in June 2004 by AIM Engineering & Surveying. This was conducted simultaneously with two other deck panel bridges (#100398 #100417) that are part of I-75- I-4 interchange.

4.6.1 Bridge Details

This 3-span bridge located in Hillsborough County, FL was built in 1983. Its deck was in service for nearly 21 years before replacement. The bridge has a main span (*span 2*) of 107 ft. and two 40 ft 7in secondary spans (*span 1, span 3*). Its overall length is 188 ft 2 in. Only the main span (*span 2*) was built using precast deck panels, the other spans being built using full depth cast in place concrete.

The shorter spans used two AASHTO Type IV girders on the outside and five AASHTO Type II girders on the inside all spaced 10 ft 1 in. apart. The main span has ten AASHTO Type IV girders spaced about 6 ft 9 in on centers as shown in Fig. 4.21.

The deck has a 7.5 in. thick concrete slab with the precast panel component being either 2-½ in. or 3-½ in. (at the rib-section) thick as shown in Fig. 4.2. The specified compressive strength of concrete for the precast panel was 5,000 psi. It was 3,000 psi for the cast in place concrete slab.

Table 4.5 Bridge #100415

Bridge #100415 Characteristics	
Year Built	1983
Number of Spans	3 (only span 2 –deck panel)
Lanes on Structure	4
ADT (2003)	43,000
Percent Truck (ADTT)	30%
Composite Slab Thickness	7-½ in.
Precast Panel Thickness	2-½ in (panel) 3-½ in (ribs)
Girder Type	AASHTO Type II and IV
Deck Condition Rating (2003)	5 (Fair Condition)

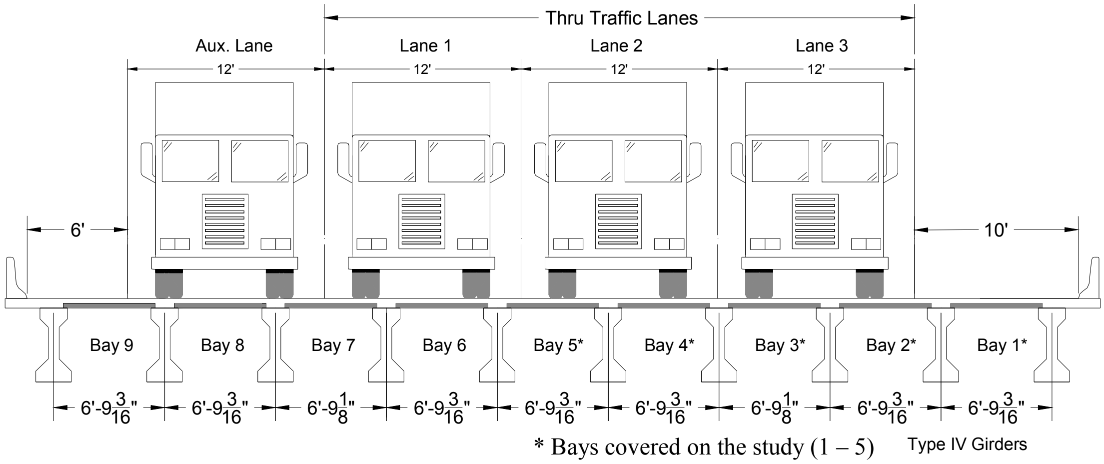


Figure 4.22 Cross Section View of Bridge #100415 span 2

This bridge has three main lanes, a 12 ft wide auxiliary lane, and two shoulders – one 6 ft wide and the other 10 ft as shown in Fig. 4.22. It is in District 7.

The forensic study was conducted on span 2, bays 1 to 5. This section of the bridge exhibited longitudinal and some transverse cracking typical of deck panel construction. Also along bay 5, there were two deteriorated M1 repairs and several walking spall patches as shown in Fig. 4.23. This figure also identifies the cut patterns used by the contractor.

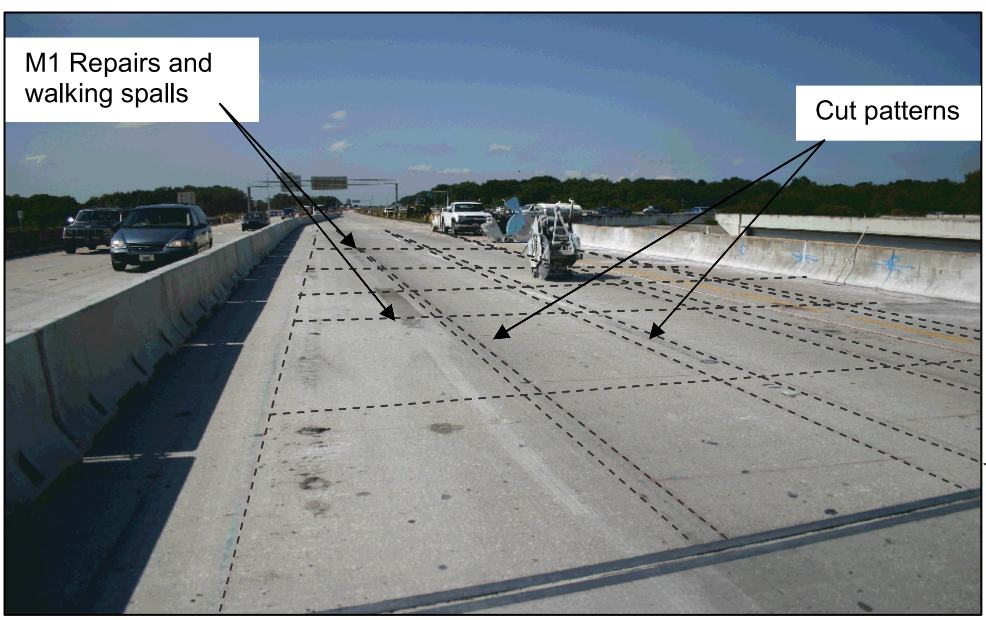


Figure 4.23 Bridge # 100415 Span 2, Prior to Deck Removal

4.6.2 Inspection Method

The intent was to follow the same procedure used in earlier forensic studies. However, the contractor used a different cut pattern (Fig. 4.24) and regions of greatest interest (the supported edges of the panel along the girder lines) were not included in the removed section. Therefore analysis focused mainly on the deck sections that were left on the top of the girders. These provided information on the bearing support provided to the panels.

Before the deck was removed, a detailed inspection was conducted to document the deficiencies and to determine their exact location so that their position could be identified in the remaining deck section on the top of the girders. Special interest was placed on assessing the condition of the panel bearings along the bridge deck. The cut pattern used in this case (Fig 4.24) helped to provide a detailed and unaltered view of the deck bearing. The panel sections removed were also inspected but not much information was obtained from them.

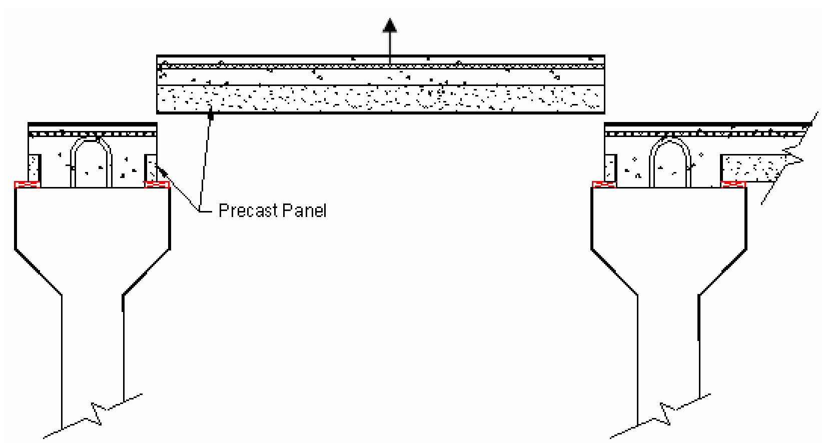


Figure 4.24 Cross Section View of Cut Pattern on Bridge #100415

4.6.3 Findings

4.6.3.1 Deteriorated M1 Repair and Walking Spalls

Fig. 4.25 shows the location of the M1 repair and the walking spalls in bays 4 and 5 in span 2. There are four numbered locations 1-4 in the plan view. These identify elevation views of the supporting girder and deck section after the panel had been cut out. The top left figure marked 1 shows the support for the panel at the M1 repair location. Note the longitudinal delamination in the cast-in-place (CIP) slab near the top. The figure marked 2 is a view of the panel after it was removed and placed on temporary barrier supports. The patch repairs and regions adjacent to it separated readily indicating loss of bond. The figure marked 3 is same as the one marked 1 except that it is located at a deteriorated region. A hammer head can be easily inserted indicating lack of bearing support and separation (also shown in the figure marked 4 where the concrete was removed. Separation of the vertical face (not visible) is also marked.

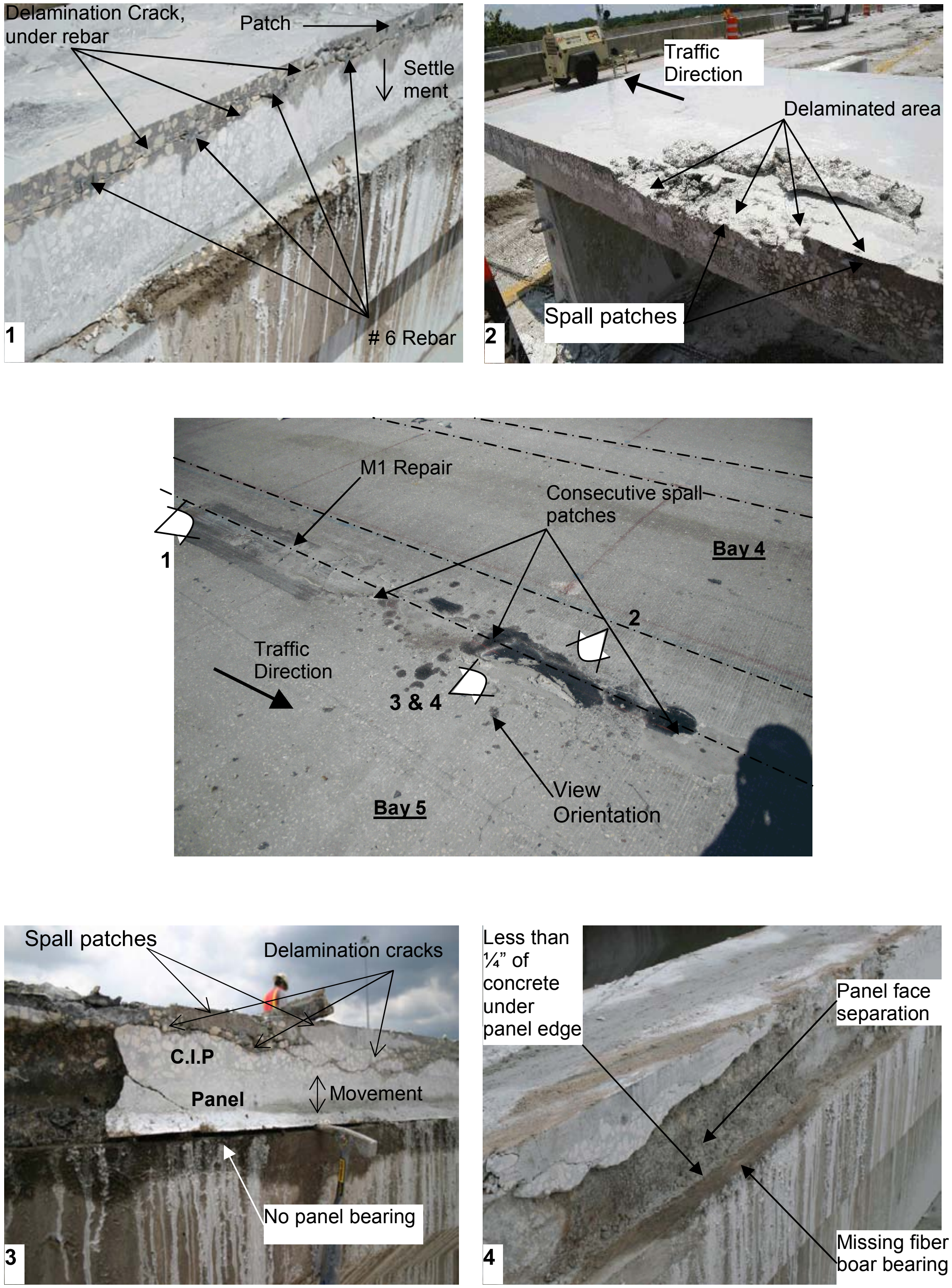


Figure 4.25 Examination of a Deteriorated Deck Section on Bridge #100415

4.6.3.2 Deck Panel Bearing

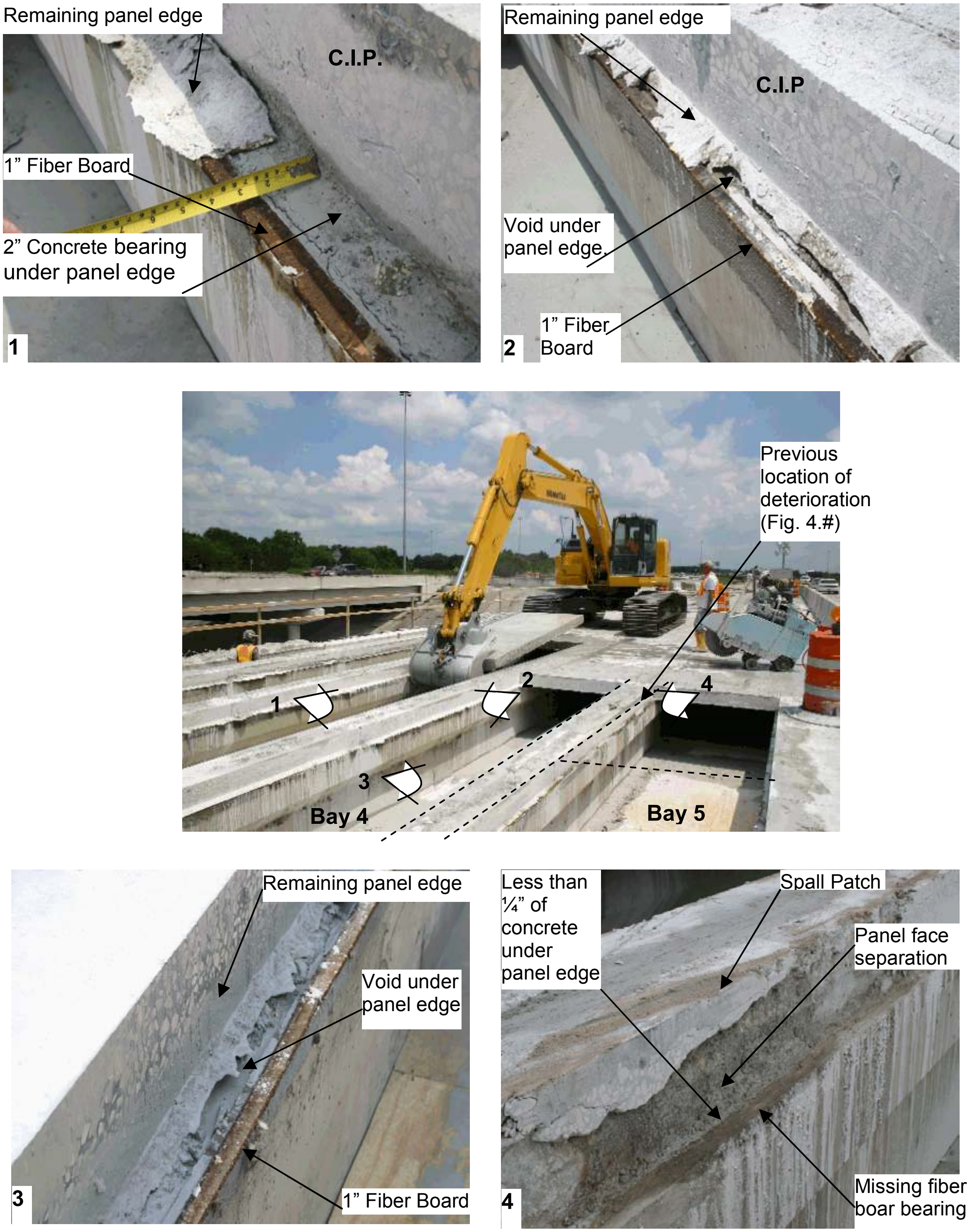


Figure 4.26 Panel Bearing Examination on Bridge #100415

Fig. 4.26 shows support for the panels at various locations along the bridge. Four pictures reflecting bearing locations marked 1-4 in the plan view are shown. The figure marked 1 shows a region where the slab was supported by 1 in. of fiberboard and 2 in. of concrete. This was unexpected from the Crosstown construction drawings and from previous research that indicated that the fiberboard was placed at the ends leaving no room for concrete to penetrate under the panel. Only vertical cracking was present in the panel with no delamination. Unfortunately, it cannot be seen because it was saw cut. The figures marked 2 and 3 show alternate locations where the concrete was unable to penetrate below the panel. The last figure, marked 4, also appearing in Fig. 4.25, shows lack of support that led to cracking and spalling of the deck. Thus, this figure provides evidence on the role of the bearing support on the performance of the deck.

4.7 I-75 NB over Sligh Ave & Ramp D-1 (Bridge #100398)

The deck replacement of this bridge was performed in June 2004 by AIM Engineering & Surveying. This deck replacement was conducted simultaneously with two other deck panel bridges (#100415 #100417) that are part of I-75- I-4 interchange.

4.7.1 Bridge Details

This 5-span bridge located in Hillsborough County, FL was built in 1984. Its deck was in service for nearly 20 years before replacement. The span lengths are as follows: Span 1 (south) 35 ft, Spans 2 and 3, 82 ft, Span 4, 54 ft 10 in, and Span 5 107 ft. Its overall length is 360 ft 10 in.

Span 1 has two AASHTO Type IV girders on the outside and six AASHTO Type II girders on the inside all spaced 10 ft 2 in. apart. Spans 2, 3 and 4, all have the same configuration as span 1 but used only girder type IV. The longest span (span 5) has eleven AASHTO Type IV girders spaced about 6 ft 7 ½ in on centers as shown in Fig. 4.27.

Table 4.6 Bridge #100398

Bridge #100398 Characteristics	
Year Built	1984
Number of Spans	5
Lanes on Structure	4
ADT (2003)	43,000
Percent Truck (ADTT)	30%
Composite Slab Thickness	7-½ in.
Precast Panel Thickness	2-½ in (panel) 3-½ in (ribs)
Girder Type	AASHTO Type II and IV
Deck Condition Rating (2003)	5 (Fair Condition)

The deck has a 7.5 in. thick concrete slab with the precast panel component being either 2-½ in. or 3-½ in. (at the rib-section) thick as shown in Fig. 4.2. The specified compressive strength of concrete for the precast panel was 5,000 psi. It was 3,000 psi for the cast in place concrete slab.

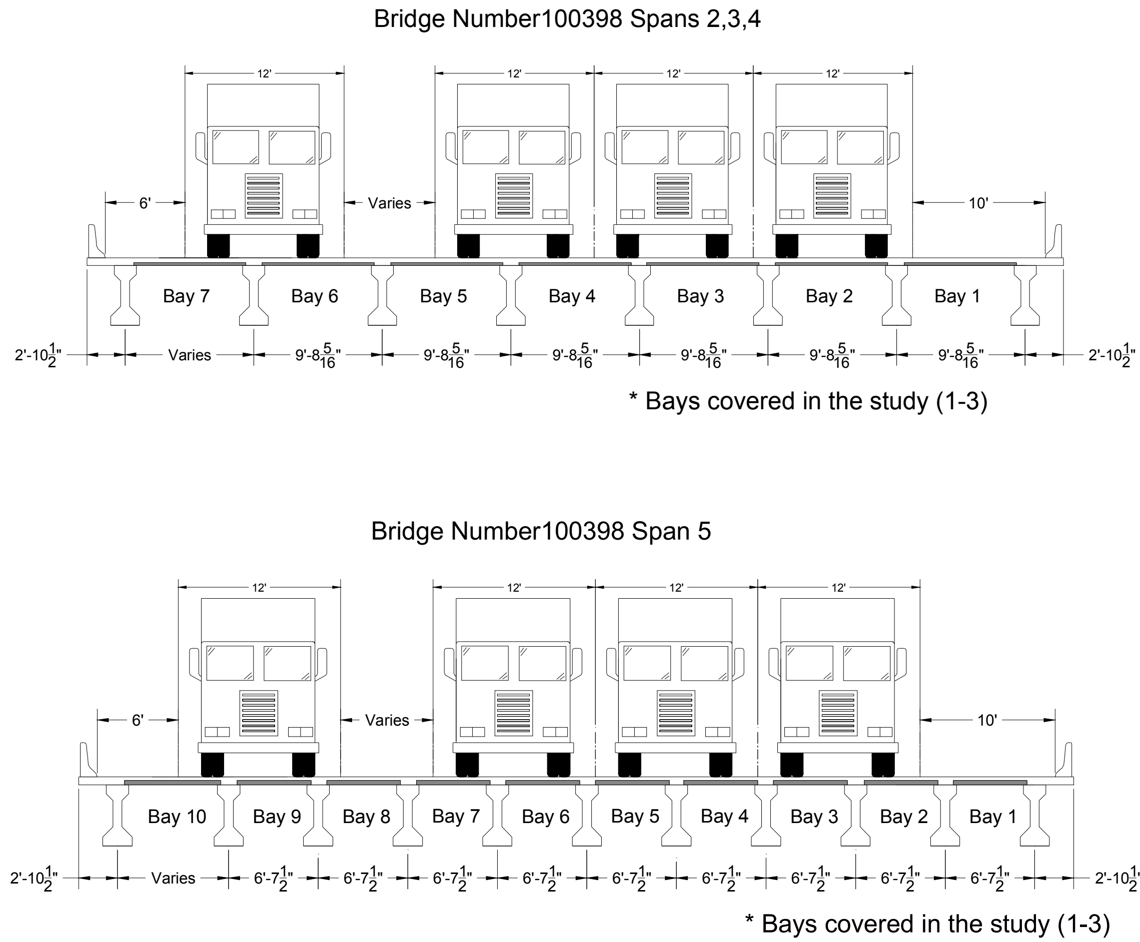


Figure 4.27 Cross Section View of Bridge #100398

After the half of the bridge to be replaced was closed, a detailed inspection of the bridge deck was conducted. During this inspection, no major deterioration was found, only typical longitudinal and some transverse cracks typical of deck panel bridges. Also no signs of previous repairs were found (Fig. 4.28).

Figure 4.28 Deck Overview of Bridge #100398

4.7.2 Inspection Method

Bearing in mind that in this bridge the contractor did not use the same cut pattern as the ones used in the previous bridge (Fig. 4.24), (they used a cut pattern similar to the one used on Moccasin Wallow Bridge, (Fig. 4.8), plus the fact that this bridge deck only exhibited random cracking but no major deficiencies, a different inspection method was used.

Figure 4.29 Inspection Methods Bridge #100398

First, random sections previously removed were inspected to identify panel face separation, vertical cracking or other type of typical internal deterioration (Fig. 4.29a). Then a detailed inspection of the panel bearing over the edges of the girders was conducted (Fig. 4.29b).

4.7.3 Findings

The most important finding was related to panel bearing support. It was found that the type of panel bearing used in this bridge was completely different, to the ones believed to be used in all deck panel bridges in Florida. Here a positive bearing was provided by a layer of grout placed next to a 1 in. fiberboard strip (Fig. 4.30). This system provided a stiff support for the panel. Soft fiberboard bearing is known to be responsible for premature deterioration of Florida's deck panel bridges [4.2].

Keeping in mind that this bridge was built on 1984, it is likely that in this bridge the panel bearing detail was changed to a positive bearing to prevent deterioration that had been observed in deck panel bridges built earlier in this area. This is the main reason why this bridge deck did not exhibit major deterioration after 20 years of service.

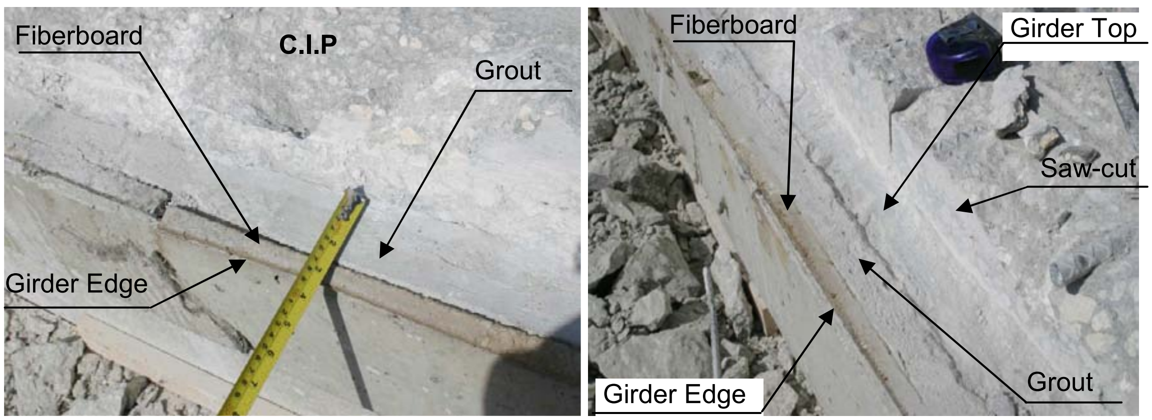


Figure 4.30 Panel Bearing Examination Bridge #100398

Despite the use of positive panel bearing in this bridge, typical deterioration such as panel face separation and vertical cracking was observed (Fig. 4.31). This proves that this kind of cracking is not related to the type of bearing used to support the panel. Positive bearing only prevents the occurrence of additional shear cracking that causes spalling in the deck surface, and may lead to sudden failures.

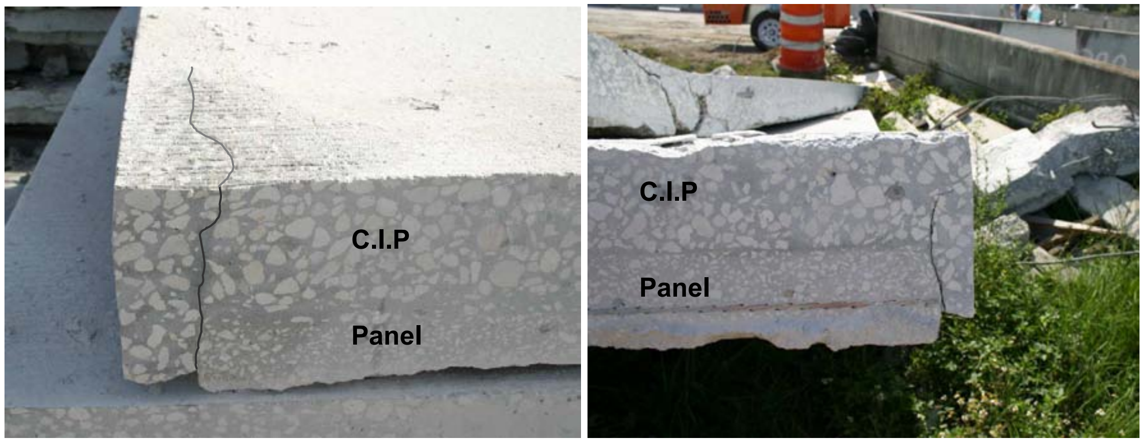


Figure 4.31 Vertical and Longitudinal Cracks Bridge #100398

Fig. 4.32 provides a summary of all the findings of the forensic investigation of bridge #100398. It shows the panel bearing detail used, and the typical panel face separation and vertical cracking found in all the deck sections inspected on this bridge.

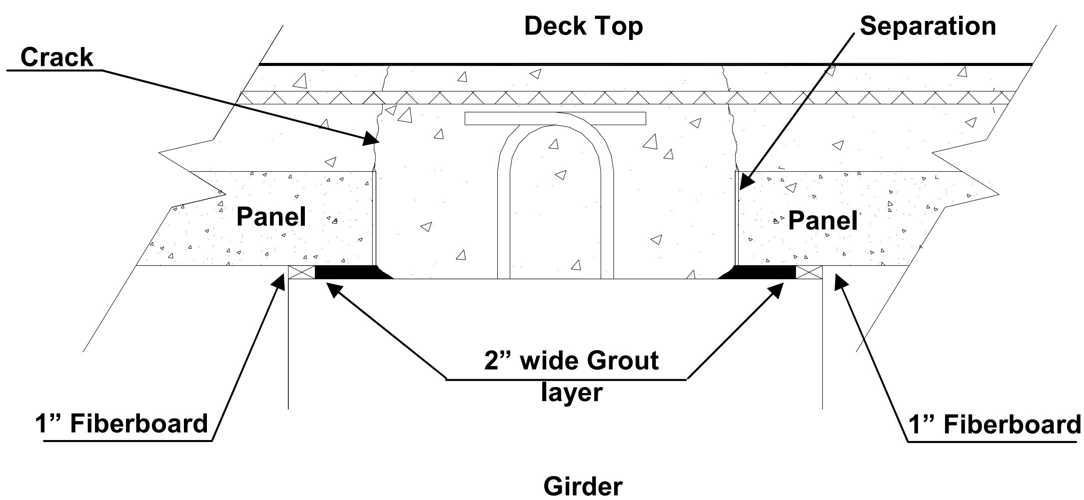


Figure 4.32 Findings Overview Bridge #100398

4.8 I-75 NB over Ramp B-1 (Bridge #100417)

The deck replacement of this bridge was performed in June 2004 by AIM Engineering & Surveying. This deck replacement was conducted simultaneously with two other deck panel bridges (#100398 and #100415) that are part of I-75- I-4 interchange.

4.8.1 Bridge Details

This 3-span bridge located in Hillsborough County, FL was built in 1983. Its deck was in service for nearly 21 years before replacement. The bridge has a main span (*span 2*) of 107 ft. and two 40 ft. 7 in. secondary spans (*span 1, span 3*). Its overall length is 160 ft 6 in.

The shorter spans used two AASHTO Type IV girders on the outside and four AASHTO Type II girders on the inside all spaced 10 ft 7-3/16 in. apart, except the center two spaced at 10 ft 7-1/4 in. The main span has seven AASHTO Type IV girders spaced about 8 ft. 10 in. on centers as shown in Fig. 4.33.

The deck has a 7.5 in. thick concrete slab with the precast panel component being either 2-½ in. or 3-½ in. (at the rib-section) thick as shown in Fig. 4.2. The specified compressive strength of concrete for the precast panel was 5,000 psi. It was 3,000 psi for the cast in place concrete slab.

Table 4.7 Bridge #100417

Bridge #100417 Characteristics	
Year Built	1983
Number of Spans	3
Lanes on Structure	3
ADT (2001)	48,290
Percent Truck (ADTT)	30%
Composite Slab Thickness	7-½ in.
Precast Panel Thickness	2-½ in (panel) 3-½ in (ribs)
Girder Type	AASHTO Type II and IV
Deck Condition Rating (2003)	5 (Fair Condition)

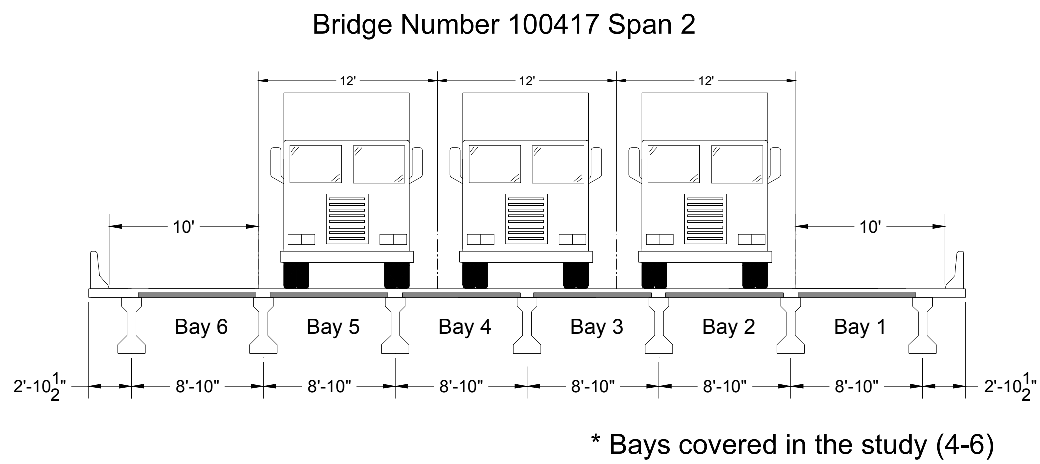


Figure 4.33 Cross Section View of Bridge #100417 span 2

This bridge has three main lanes, a 12 ft wide auxiliary lane, and two shoulders – both 10 ft wide as shown in Fig. 4.33. This bridge is located in District 7.

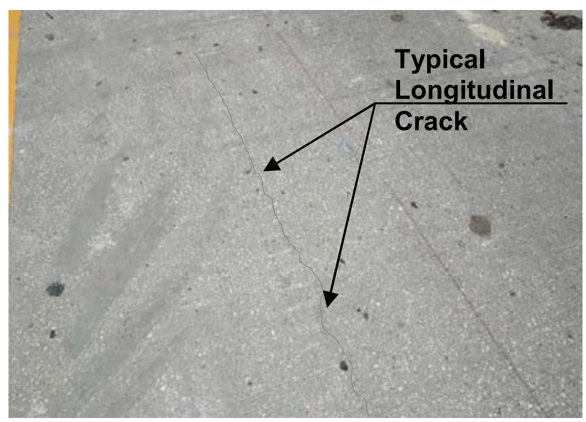


Figure 4.34 Deck Before Removal Bridge # 100417 (Bays 4-6)

After the half of the bridge to be replaced was closed, a detailed inspection of the bridge deck was conducted. During this inspection only typical longitudinal cracks along the entire deck and some transverse cracks were found. Both are typical of deck panel construction. Also no signs of previous repairs were found (Fig. 4.34).

4.8.2 Findings

This bridge had exactly the same type of panel bearing detail as in the previous bridge (Section 4.7). Here a positive bearing was provided by a layer of grout placed next to a 1 in. fiberboard strip (Fig. 4.35). This system provided a stiff support for the panel. Soft fiberboard bearing is known to be responsible for premature deterioration of Florida's deck panel bridges [4.2].

The use of positive panel bearing in this bridge is believed to be the reason why it did not exhibit major deterioration after 20 years of service.

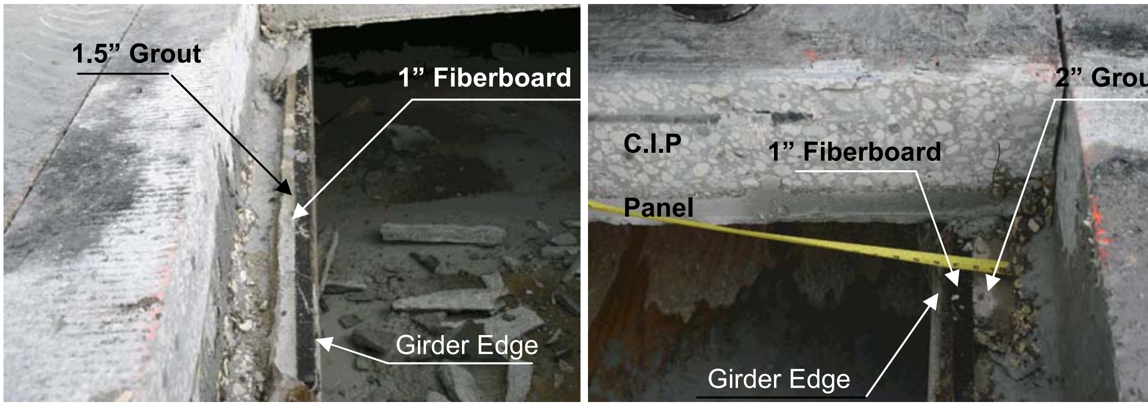


Figure 4.35 Panel Bearing Examination Bridge #100417

The inspection confirmed that longitudinal cracks were caused due to separation of the vertical face of the panel (Fig. 4.36). It also proved that the presence of deck surface longitudinal cracking is related only to the use of precast panels regardless the type of panel bearing used, whereas the occurrence of spalls or additional cracking was directly linked to the type of bearing used (“negative bearing”).

Figure 4.36 Panel Bearing Examination Bridge #100417

Fig. 4.37 provides a summary of all the findings of the forensic investigation of bridge #100417. It shows the panel bearing detail used, and the typical panel face separation and vertical cracking found on in almost all the deck sections inspected.

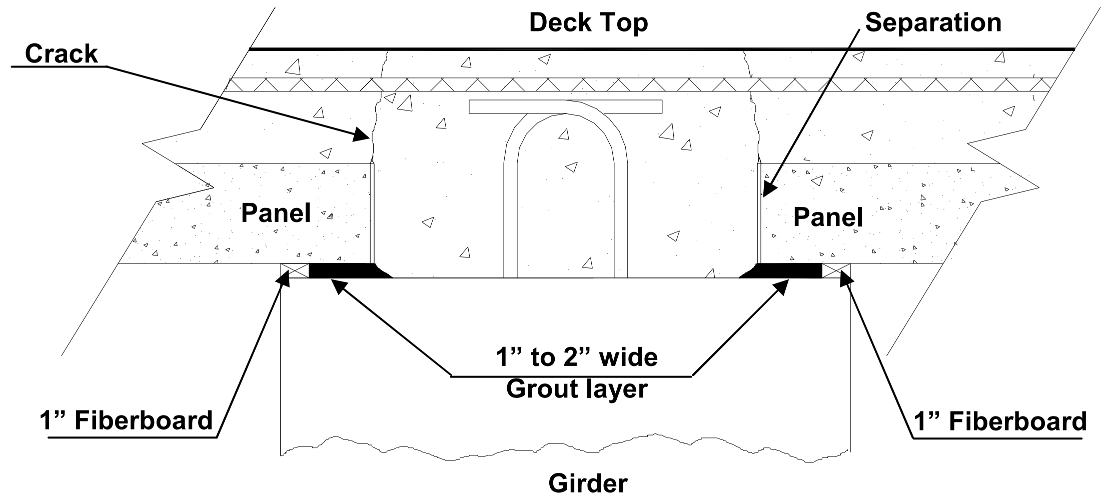


Figure 4.37 Findings Overview Bridge #100417

4.9 I-75 NB Over SR 64 (Bridge #130085)

The deck replacement of this bridge was performed in August 2004 by Zep Constructions Inc. Fort Myers FL. In about three weeks, the existing deck panel was removed and replaced by a full-depth cast in place concrete slab.

4.9.1 Bridge Details

This 4-span bridge located in Manatee County, FL District 1 was built in 1981. Its deck was in service for nearly 23 years before replacement. The bridge has two main spans of 108 ft 10 in. with two secondary spans of 43 ft. Its overall length is 303 ft 8 in.

The shorter spans used two AASHTO Type IV beams on the outside and six AASHTO Type II beams on the inside all spaced 8 ft 8 9/16 in. apart. The main span has eleven AASHTO Type IV beams spaced about 6 ft 9 in on centers as shown in Fig. 4.38

The deck has a 7.5 in. thick concrete slab with the precast panel component being either 2-½ in. or 3-½ in. (at the rib-section) thick as shown in Fig. 4.2. The specified compressive strength of concrete for the precast panel was 5,000 psi. It was 3,000 psi for the cast in place concrete slab.

Table 4.8 Bridge #130085

Bridge #130085 Characteristics	
Year Built	1981
Number of Spans	4
Lanes on Structure	4
ADT (2001)	25,000
Percent Truck (ADTT)	10%
Composite Slab Thickness	7-½ in.
Precast Panel Thickness	2-½ in (panel) 3-½ in (ribs)
Girder Type	AASHTO Type II and IV
Deck Condition Rating (2003)	7 (Good condition)

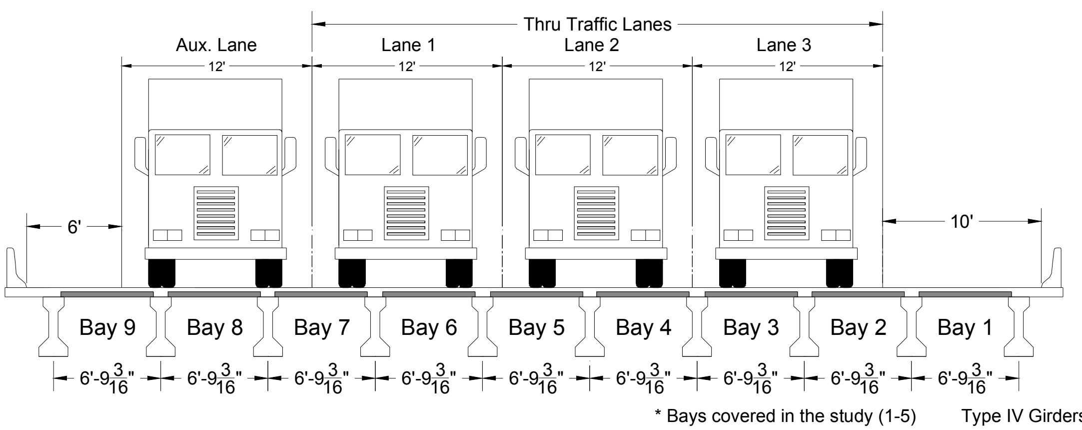


Figure 4.38 Cross Section View of Bridge #130085 span 2

This bridge has three main lanes, a 12 ft wide auxiliary lane, and two shoulders – one 6 ft wide and the other 10 ft, as shown in Fig. 4.38.

The forensic study was conducted on spans 2 to 4, bays 1 to 5. This section of the bridge and the rest of the bridge, exhibited only longitudinal and some transverse cracking typical of deck panel construction. No spalls or previous deck repairs were noticed on this bridge (Fig 4.39).

Figure 4.39 Bridge # 130085 Prior to Deck Removal

4.9.2 Findings

It was found that the panels in this bridge were initially placed over a 1 ½ wide fiberboard strip, with a 2 in. wide grout layer (Fig. 4.40), providing positive bearing support to the panels. However, the fiberboard had been replaced later by epoxy, as recommended in a previous research study [4.3], as a way to stop or to prevent future deterioration of the deck.

The panel bearings were inspected over a large area of the deck. It was found that in some regions the precast panel was not long enough, and therefore the grout could not flow below the panel and provide support. In such panels, support was only provided by fiberboard that had been subsequently replaced by epoxy (Fig. 4.40 4.41).

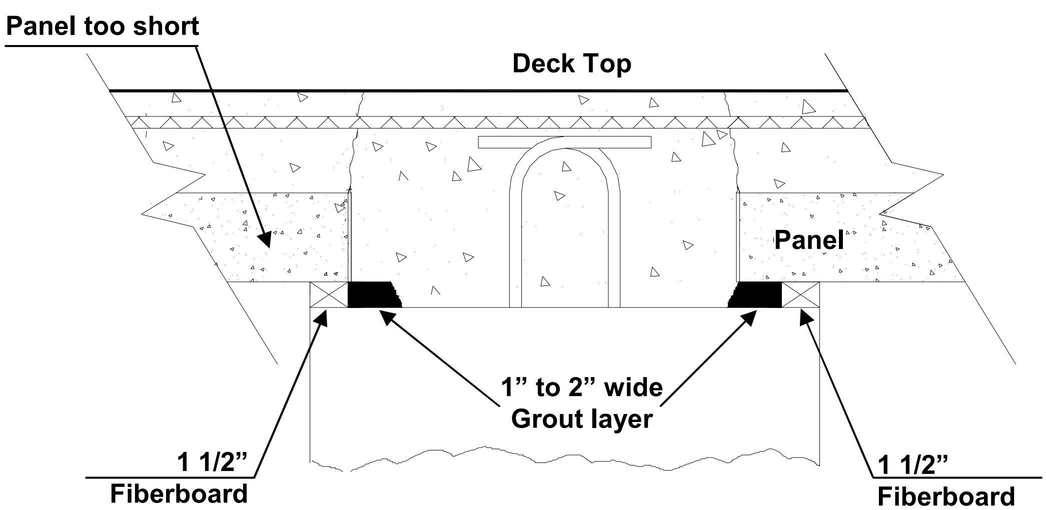


Figure 4.40 Bridge # 130085 Original Panel Bearing Detail

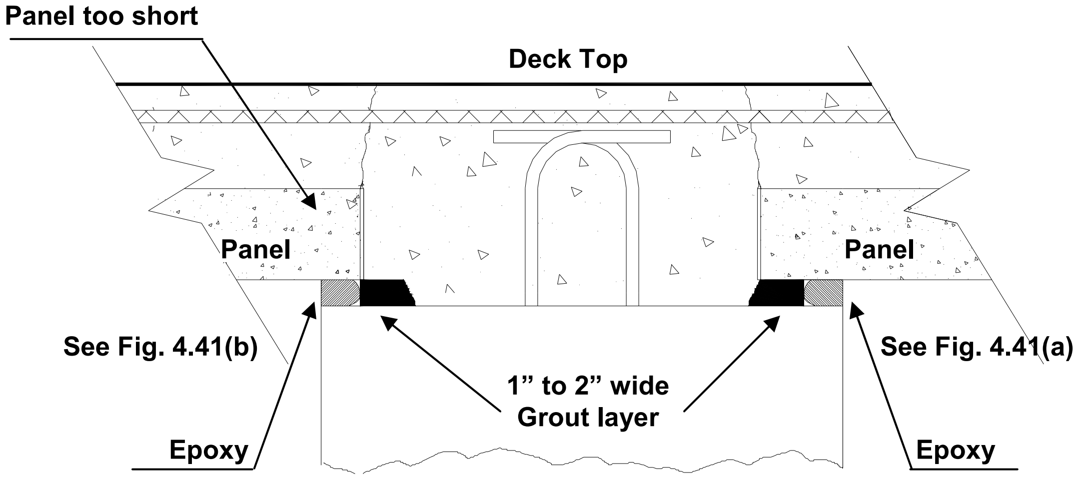


Figure 4.41 Bridge # 130085 Bearing Detail after Epoxy Repair

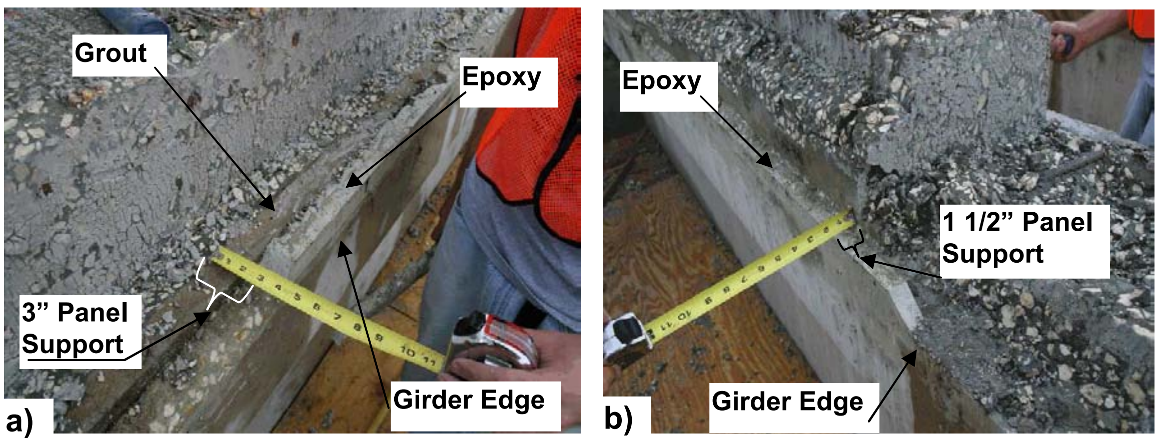


Figure 4.42 Bridge # 130085 Panel Bearing Details

Despite the use of positive panel bearing in this bridge, typical deterioration such as panel face separation and vertical cracking were detected (Fig. 4.43). Similar cracking was found in bridges on fiberboard bearings (“negative bearing”). This proves that such cracking is not related to the type of bearing used to support the panel.

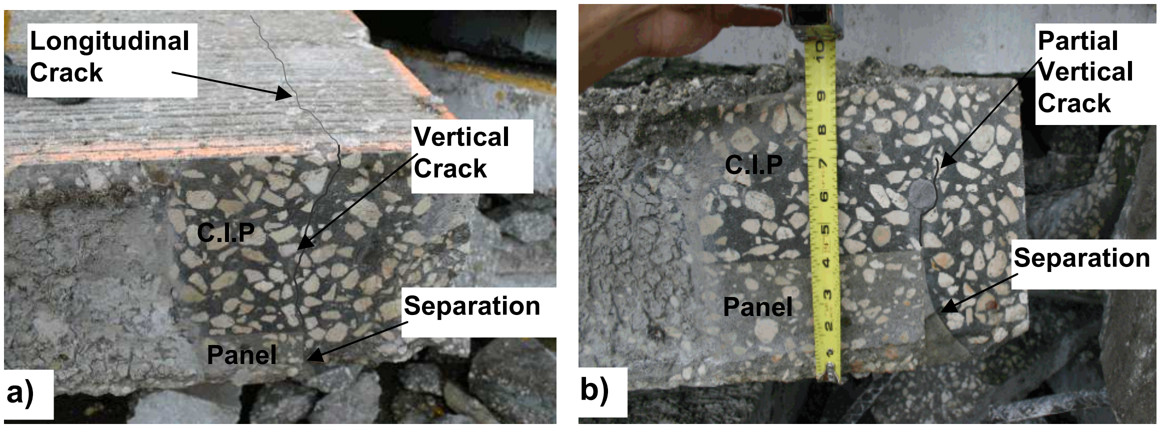


Figure 4.43 Surface Longitudinal Crack Bridge # 130085

As in bridges (100398 and 100417), the relatively good condition of this bridge can be attributed to the fact that this bridge was built using positive (grout) bearing for the precast panels. Also, replacement of the fiberboard by epoxy helped to prevent deterioration in regions where the panel was only supported by the fiberboard (Fig. 4.42b).

4.10 Study Summary

Table 4.9 summarizes all the different types of panel bearings used in the bridges covered on the forensic study and comments on the relationship between the type of bearing and the condition of the deck.

Table 4.9 Bearing and Deck Condition Summary

Bridge #	Study Date	Built	Bearing Type	Comments
130078 & 130079	6/03	1981	Originally fiberboard bearing. Partially replaced by epoxy bearing.	Major deficiencies found in regions where there was no epoxy replacement. This was probably because the fiberboard had become too thin to be removed and there was insufficient room for the epoxy to penetrate below the panel.
170140	1/04	1981	Originally fiberboard bearing. Full epoxy bearing replacement.	No major deficiencies were found. The fiberboard was thicker and full epoxy bearing replacement was possible.
130075	5/04	1981	Originally fiberboard bearing. Partially replaced by epoxy bearing.	Major deficiencies found in locations where there was partial replacement of the fiberboard by epoxy.
100415	6/04	1983	Fiberboard with panel overhang to allow concrete to flow under the panel and provide support.	Major deficiencies found in spots where no concrete went under the panel. Or the panel was too short to provide an overhang.
100398	6/04	1984	Fiberboard plus grout bearing.	Bridge deck in good condition, only longitudinal and transverse cracking.
100417	7/04	1983	Fiberboard plus grout bearing.	Bridge deck in good condition, only longitudinal and transverse cracking.
130085	8/04	1981	Originally fiberboard plus grout bearing. Now full epoxy bearing replacement plus grout.	Bridge deck in good condition because the panel is supported by epoxy and grout.

Even though it was originally thought that all the deck panel bridges in Districts 1 and 7 were supported only by fiberboard (negative bearing), it was found that in 4 out of 7 bridges in the study there was some type of positive panel bearing -grout or concrete-. And all the major deterioration was linked to negative fiberboard bearing or due to construction inaccuracy.

Table 4.10 provides a summary of the findings from each bridge included in the study. These findings cover all the typical deck top deficiencies in deck panel bridges.

Table 4.10 Study Summary

Bridge #	Study Date	Built	Finding	New
130078 & 130079	6/03	1981	No deck surface cracking. Longitudinal deck surface cracking. Transverse deck surface cracking. Additional longitudinal parallel cracking. Deck spalling and delamination.	All are new
170140	1/04	1981	Epoxy panel bearing repair Longitudinal deck surface cracking. Additional longitudinal parallel cracking.	Epoxy panel bearing repair
130075	5/04	1981	Epoxy bearing repair condition. Cracking of M1 Repair. Longitudinal deck surface crack.	Cracking of M1 Repair
100415	6/04	1983	Positive (grout) bearing. Negative (fiberboard) bearing. Longitudinal deck surface crack. Deck spalling and delamination.	Positive (grout) bearing. Negative (fiberboard) bearing
100398	6/04	1984	Positive panel bearings. Longitudinal deck surface crack.	
100417	7/04	1983	Positive panel bearings. Longitudinal deck surface crack.	
130085	8/04	1981	Grout + Epoxy bearing repair. Longitudinal deck surface crack.	

The forensic study made it possible to obtain invaluable information regarding the deterioration of deck panel bridges that could not otherwise have been obtained.

4.11 Deck Failure Mechanism Model

From the information obtained from the analysis of failed bridges (Ch. 3) and forensic investigations, it is possible to develop a model that identifies the progression in deterioration that can potentially lead to localized failure. This is described in the following sections.

4.11.1 Stage #1 Initial Condition

The first stage is the initial condition of the bridge after it was built. At this point we can identify two main groups of parameters that can affect long term performance of the bridge deck. These are:

1. Design Parameters – Relatively easy to quantify -
    a. Type of deck design. Deck panel, or full depth cast in place concrete deck
    b. Type of panel bearing. Positive panel bearing (grout, concrete), Negative bearing (fiber board)
    c. Deck geometry. Deck thickness, beam spacing, beam type, span length
    d. Deck material properties. Concrete f'c, water cement ratio
    e. Traffic volume. Average daily traffic (ADT), Average daily truck traffic (ADTT), Actual and estimated future values.
    f. Lane placement. Location of the wheel path relative to the edge of the girders.

2. Construction quality parameters - Very difficult to quantify –
    a. Deck thickness accuracy – it was found that in some cases the deck thickness was 10% smaller than the design value. And this type of deck is very sensitive to reductions in slab thickness.
    b. Top steel rebar cover. When the top steel rebar is very close to the surface of the deck chances of delamination and spalling are greater.
    c. Concrete properties. Actual water cement content ratio, concrete curing process, f'c value before the bridge was opened to traffic, capacity of the concrete to resist the environment, actual f'c values.
    d. Real panel bearing condition. It was found that poor workmanship can significantly affect the real condition of the panel bearing.

4.11.2 Stage #2 Longitudinal/ Transverse Cracking

The second stage is the occurrence of longitudinal cracks over the edges of the girders. This is the most common type of cracking in deck panel bridges and starts early. This crack is mainly the result of creep induced by prestressing forces in the panel, and the differential shrinkage between the cast in place concrete and the deck panel. Following the formation of longitudinal cracking, sporadic transverse cracks can also develop. The intersection of longitudinal and transverse cracking can lead to spalling in regions of the deck subjected to wheel loads.

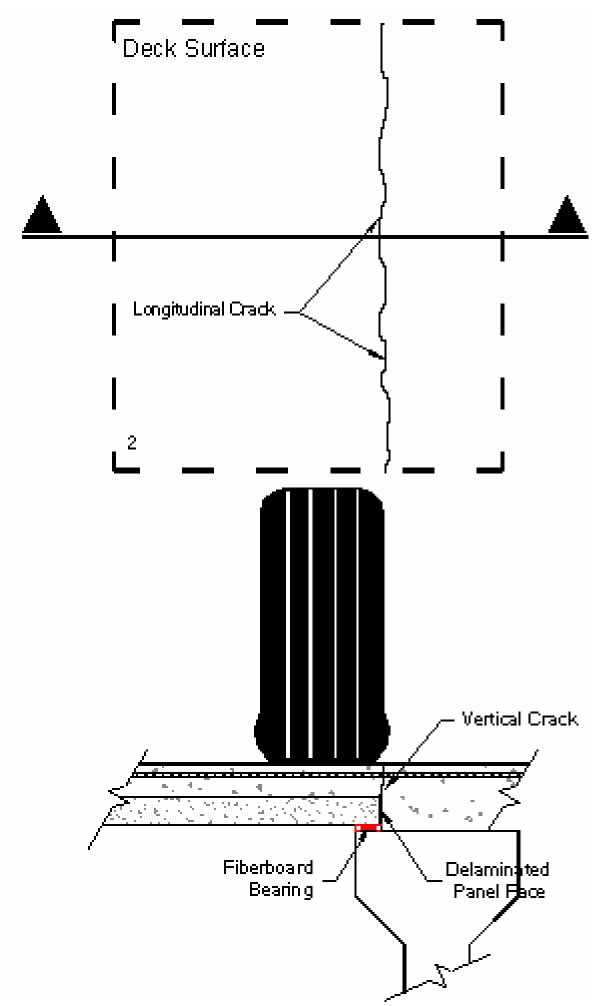


Figure 4.44 Deterioration Stage #2

4.11.3 Stage #3. Shear Failure Longitudinal Cracking

The third stage is the occurrence of additional longitudinal cracking on the deck surface, parallel to the cracking described previously. This additional cracking is the result of a shear failure of the cast in place concrete. This type of cracking is the first sign of future deck deterioration. This type of cracking is related only to panels supported on negative (fiberboard) bearing.

When the deck panels are supported only by fiberboard and no strand extension is provided in the panel face, all the shear load in this region is supported by the cast in place section on top of the panel edge, instead of being transferred to the entire composite section.

Shear failure that causes this crack occurs in part due to the reduction of the shear capacity in the already overstressed cast place concrete slab. This shear reduction is caused by the vertical cracking described in Section 4.11.2. It was also found that this reduction is affected by the shape of the vertical crack. Fig. 4.45 relates the shape of the crack to shear reduction. Reductions are higher when the crack extends towards the girder edge, and smaller when the crack extends towards the center of the girder since in the latter case, the load is directly supported by the girder.

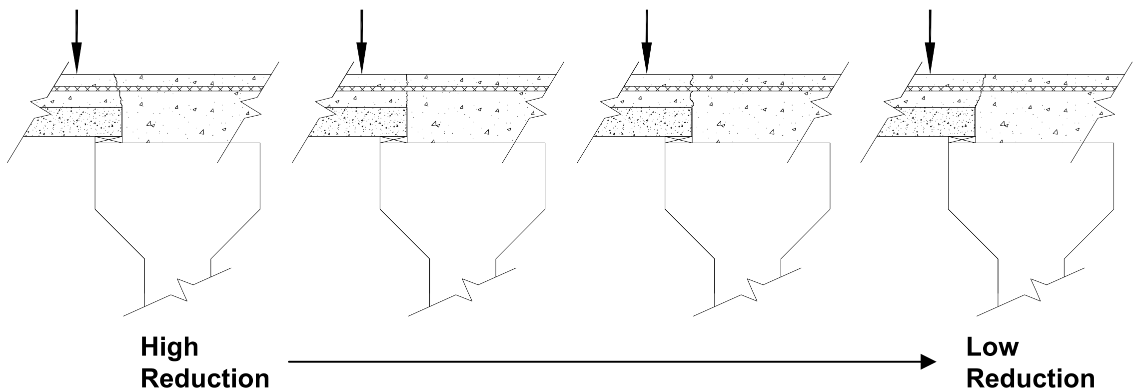


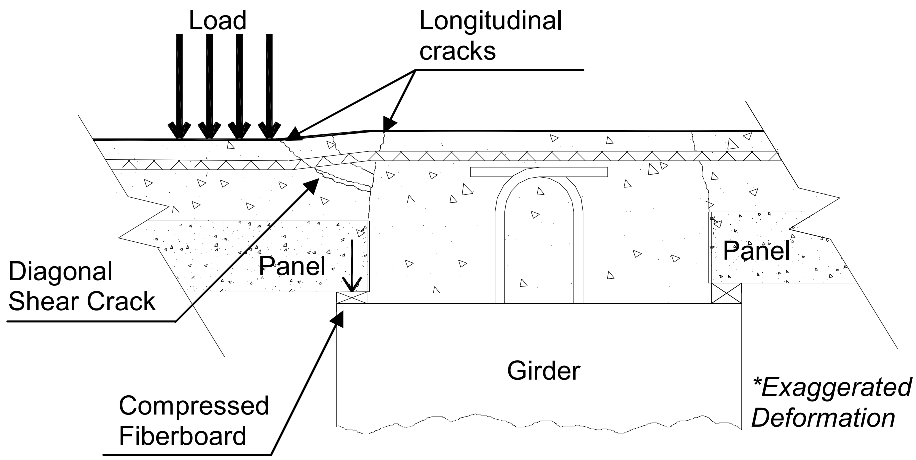
Figure 4.45 Effect of Vertical Crack Shape in Shear Reduction

From the forensic examinations, two different types of shear failures were identified.

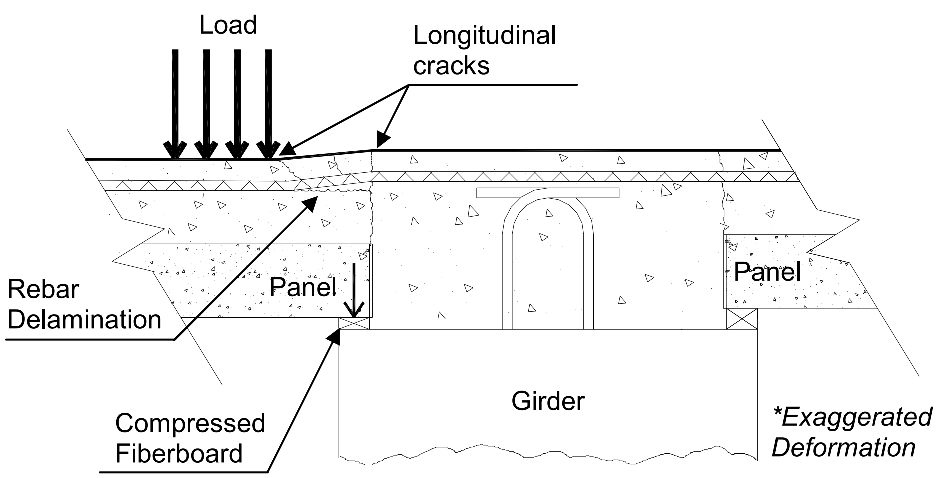
The first type of failure occurs when the cast in place concrete still has some capacity to transmit shear according to the shape of the vertical crack (Fig. 4.45). This shear failure is manifested by the appearance of a diagonal cracking emanating from the corner of the precast panel that propagates at an angle of less than 45 degrees. In most of the cases this crack reaches the surface generating additional longitudinal cracks (Fig. 4.46a.

The second type of shear failure occurs when due to the vertical crack the cast in place concrete has lost all its shear capacity. In this case, the shear is transmitted only by the steel rebar by dowel action. When the shear load is very high, the rebar acts like a "crowbar" digging into the concrete on top of the rebar, creating delamination in that area (see Fig. 4.46b).

Since this crack is related to shear, it is more likely to occur in the cases where the wheel loads are located close to the support of the panels. This is the load location that provides the highest shear value in the section of interest.



a) Low Reduction of C.I.P Shear Capacity.



b) High Reduction of C.I.P Shear Capacity.

Figure 4.46 Shear Failures for Different Degrees of Shear Reduction

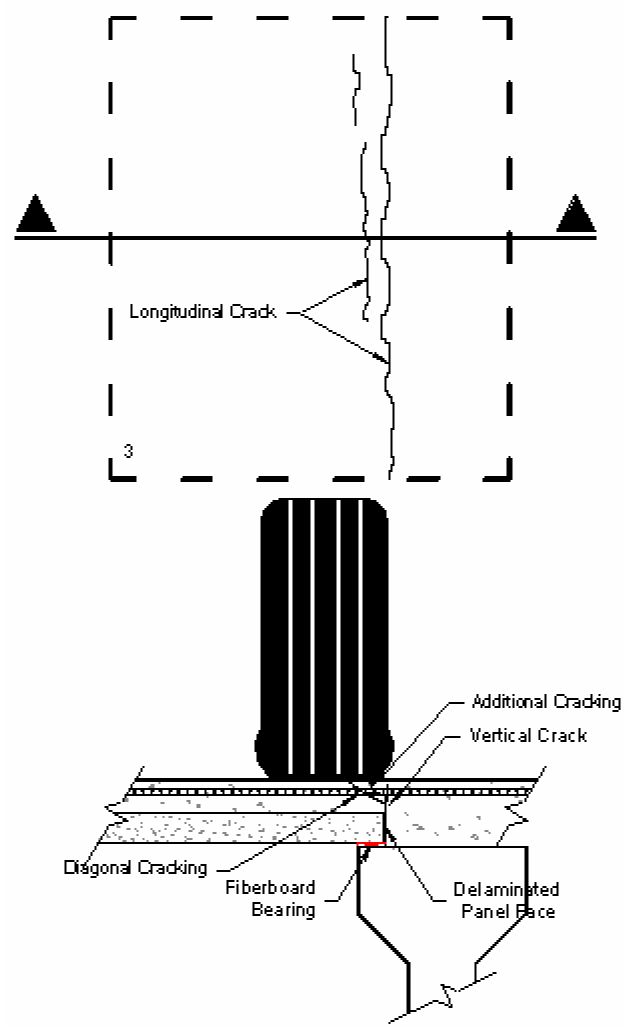


Figure 4.47 Deterioration Stage #3

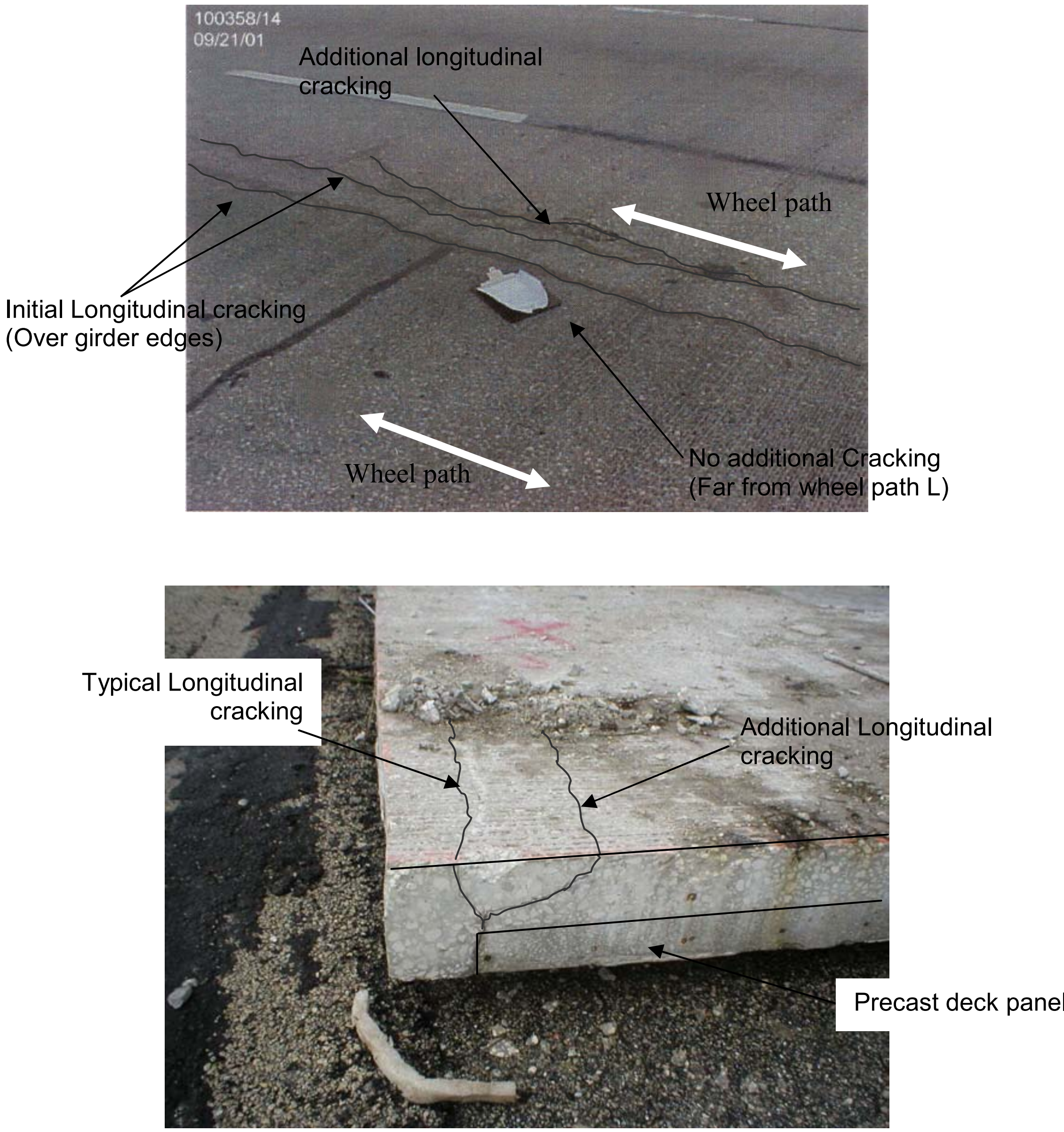
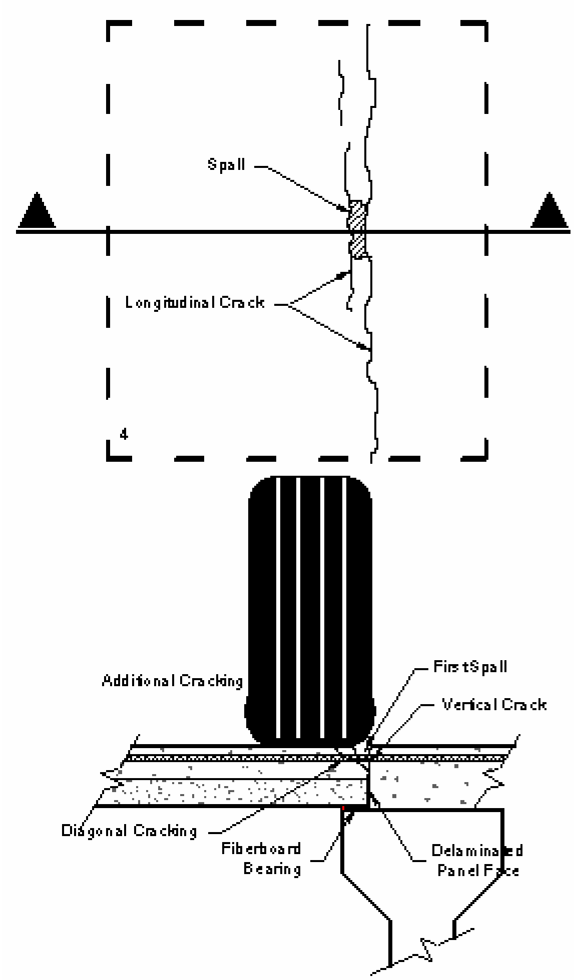


Figure 4.48 Examples of Deterioration Stage #3

4.11.4 Stage #4 First Spall

After the occurrence of the second parallel crack, the concrete trapped between the two cracks is already internally cracked and starts to crumble. As a result, 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.



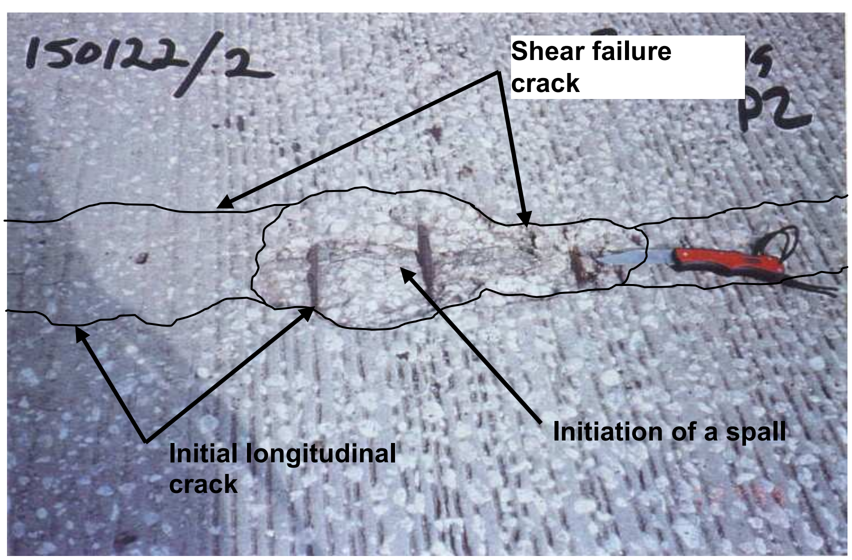


Figure 4.49 Deterioration Stage #4

4.11.5 Stage #5 Spall Increase, Then Spall Patch

After the occurrence of the first spall in stage#4 it will keep increasing in size basically due to the effect of the impact of the wheels at the edges of the existing spall. The maximum size of the spall and additional deterioration of the deck depends on how long it is left unrepaired. Usually the spalls are patched before they reach a relatively large size. For the majority of the cases, the repair consists of a temporary patch using a flexible material.

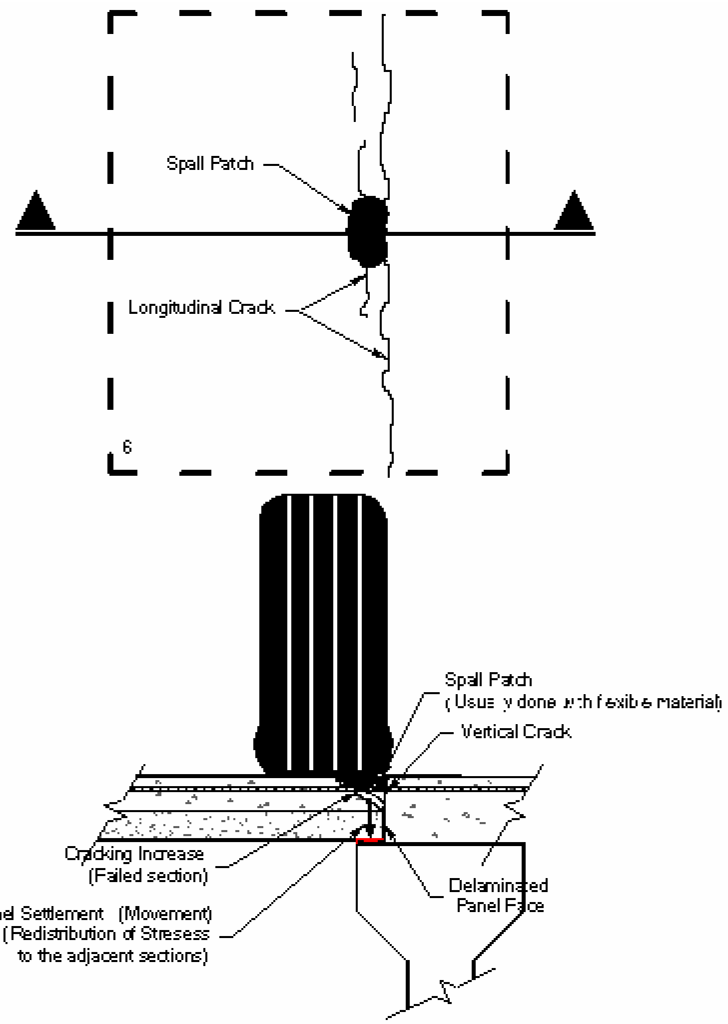


Figure 4.50 Deterioration Stage #5

4.11.6 Stage #6 New Spalling Plus Spall Increase

Depending on all the different factors mentioned earlier, new spalls can appear in the areas adjacent to the repaired spall after some time. Note that 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.

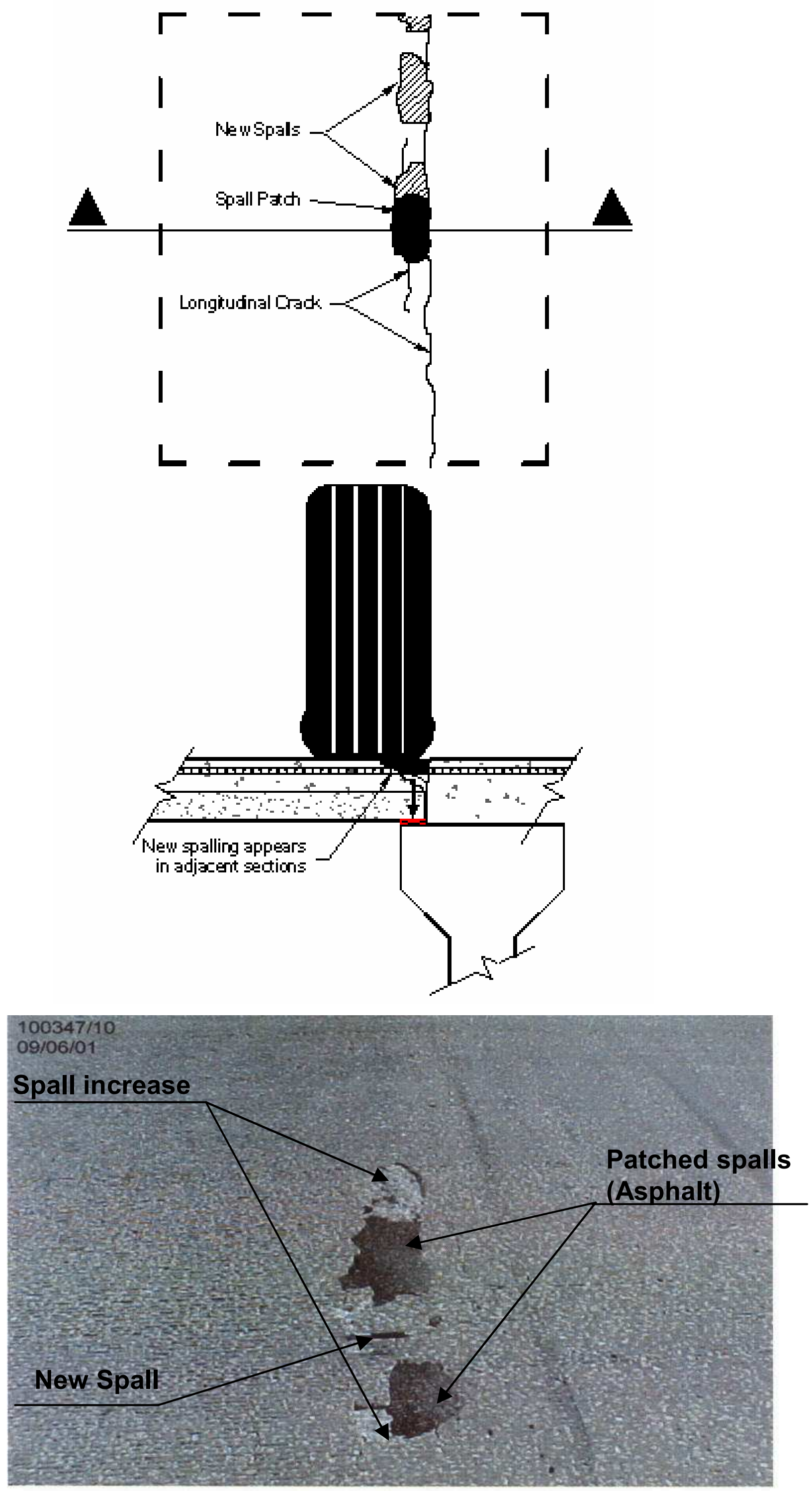


Figure 4.51 Deterioration Stage #6

4.11.7 Stage #7 M1 Repair

Generally after several patch and re-patches, an M1 repair is done in the affected area. An M1 repair consists of the removal of all the patched, spalled, and unsound concrete section, and its replacement by repair material. To do this, the edges of the section to be removed are cut and the concrete inside removed using a jack hammer. Usually the intent is to remove the cast in place concrete as close as possible to the deck panel surface. The opened surface is then cleaned and the removed concrete is replaced with different types of high strength epoxy materials. And in some cases the fiberboard bearing is replaced by epoxy.

The durability of the M1 repair and the condition of the deck area around it depends of 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 cast in place concrete, (4) Bonding between the old concrete and the repair material, (5) Stress redistribution to adjacent areas (after removal of the damaged cast in place 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. And finally the most important parameter,(9) removal of the fiberboard and its replacement by non shrink epoxy.

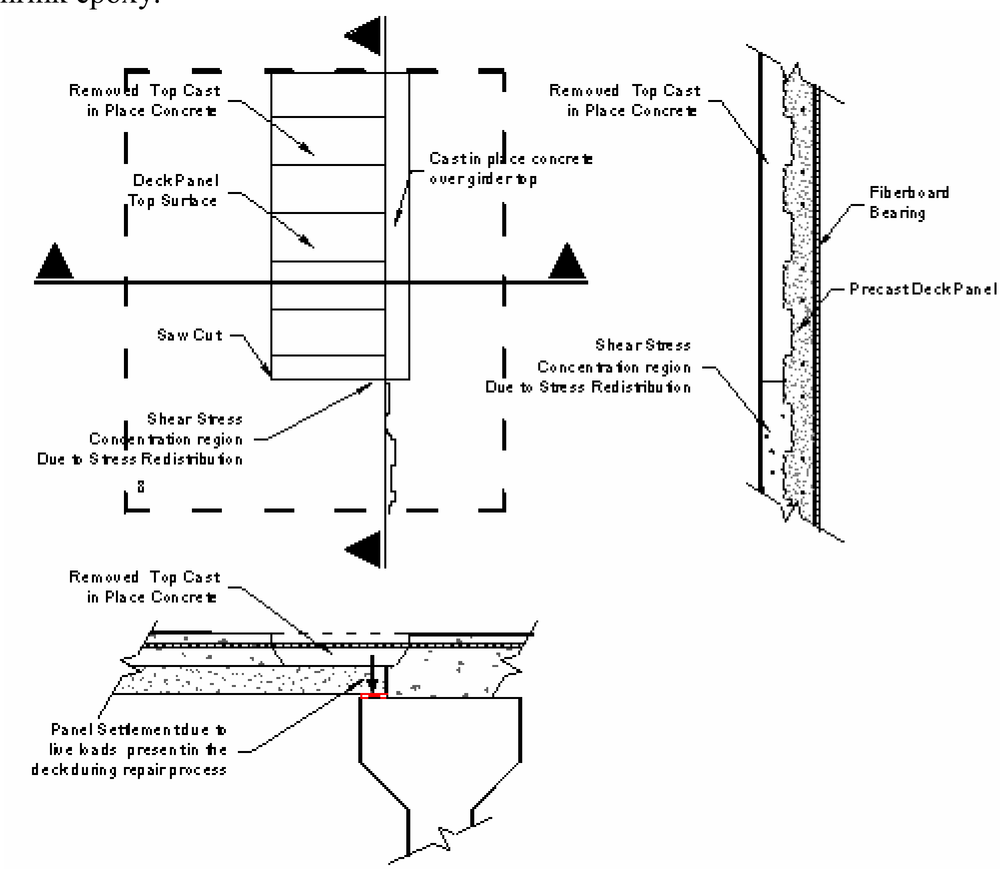
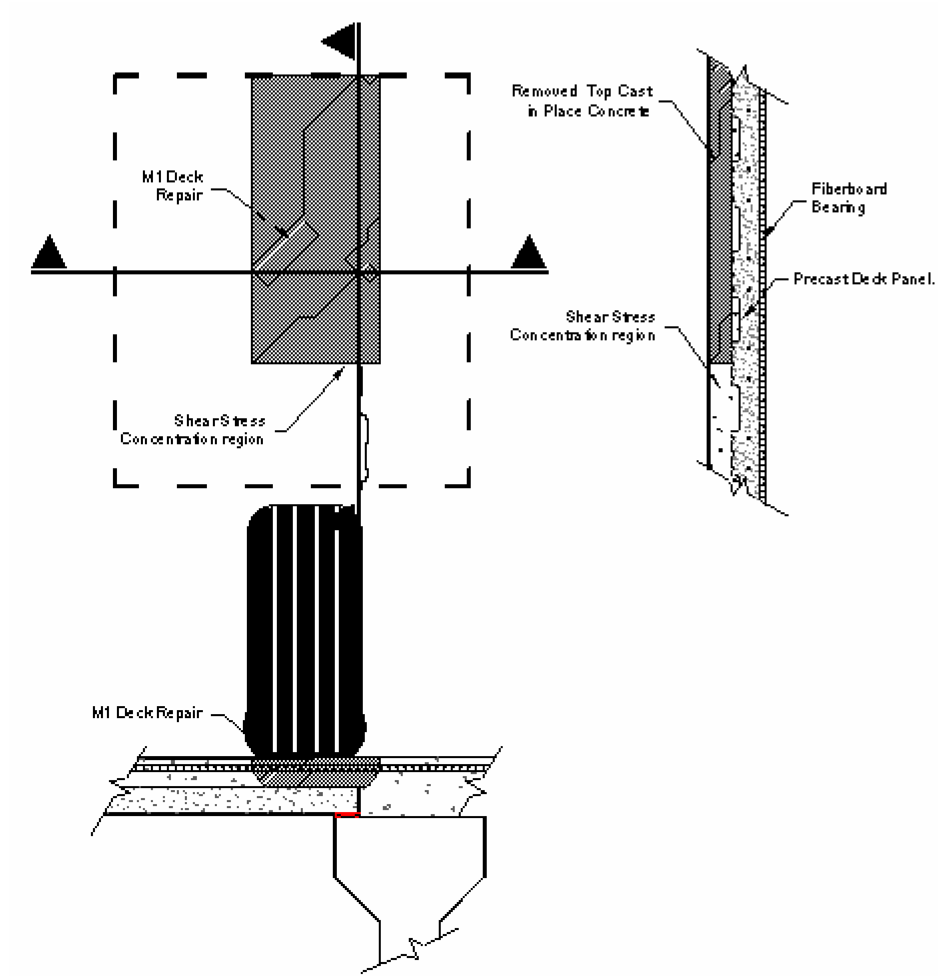


Figure 4.52 M1 Repair Procedure (Stage #7)



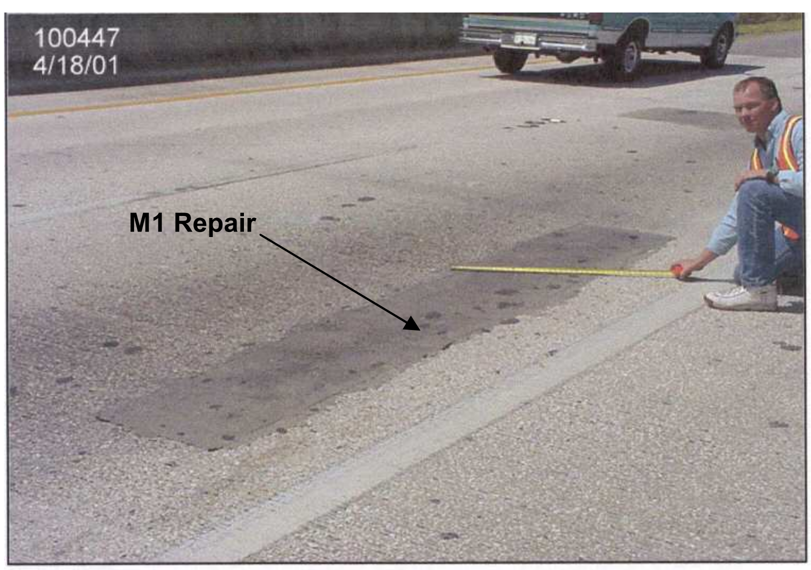


Figure 4.53 Deterioration Stage #7

4.11.8 Stage #8 Parallel Longitudinal Cracking Adjacent to an M1 Repair

Assuming that the panel bearing was not replaced by epoxy, or that it was replaced but due to construction problems there was no full support of the panel by the epoxy, the deck area adjacent to an M1 repair starts to deteriorate again with the appearance of the additional parallel cracking – shear failure cracking - described in (Section 4.11.3). The parameters that affect the occurrence of this additional deterioration case are the same mentioned for stages #3 and #7.

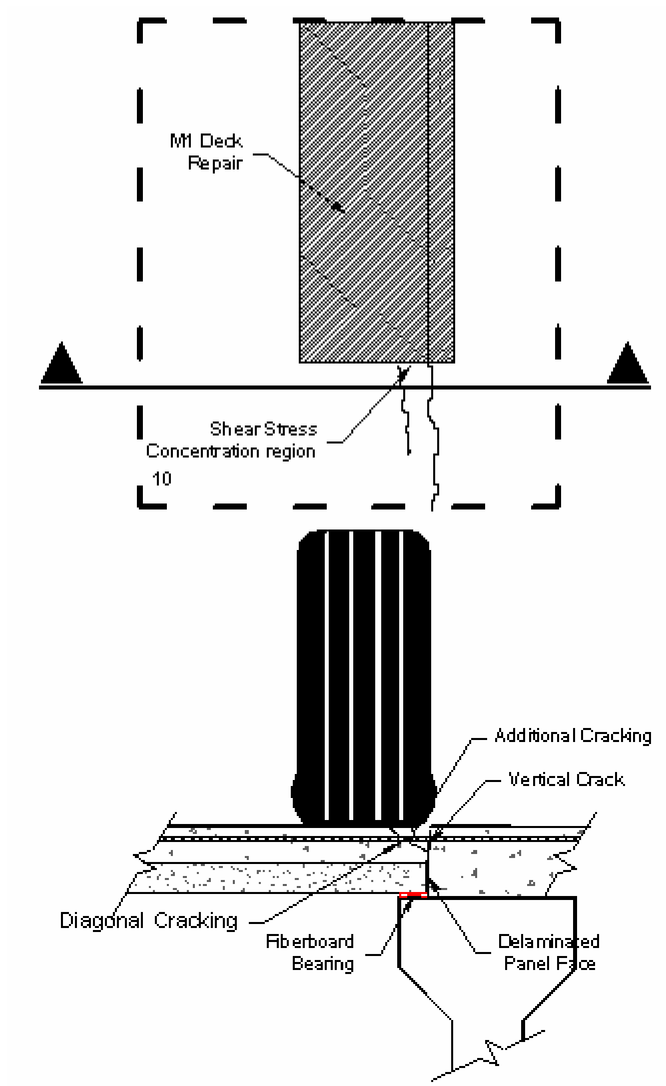
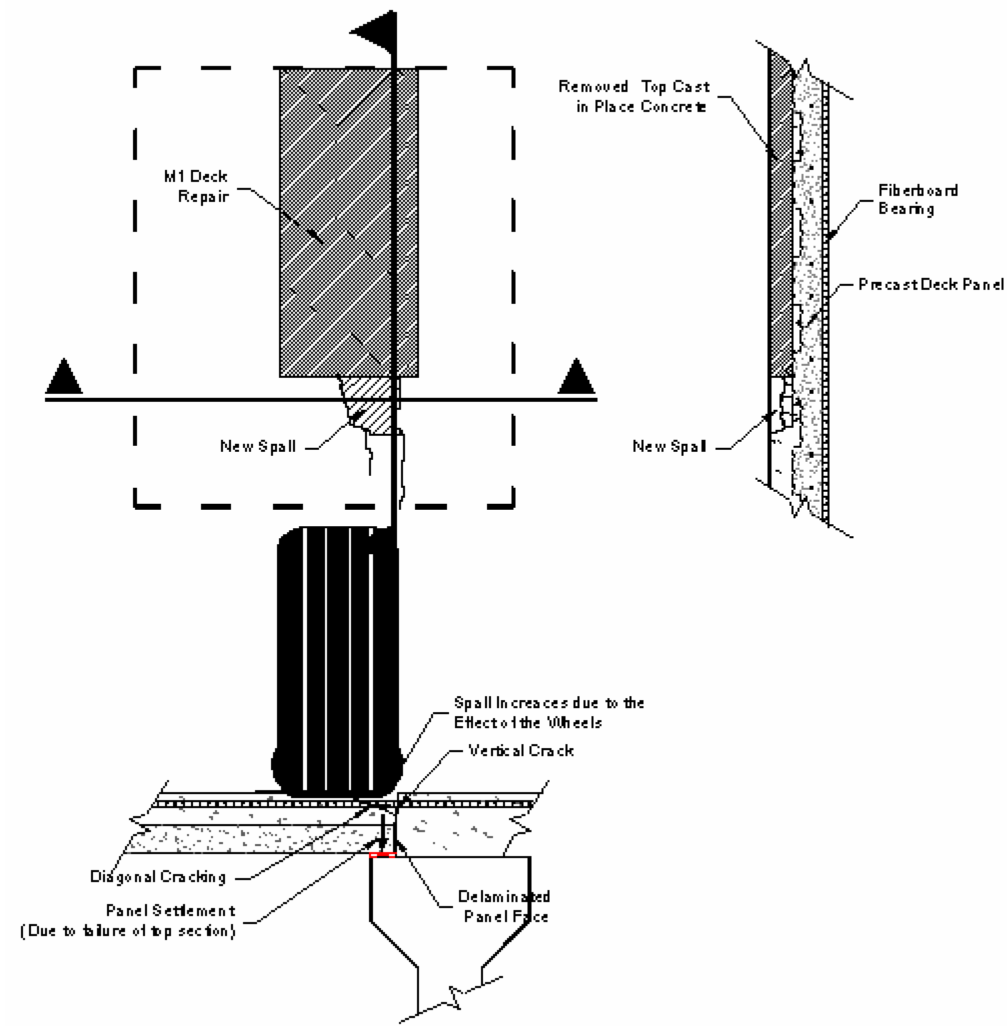


Figure 4.54 Deterioration Stage #8

4.11.9 Stage #9 Spalling Adjacent to an M1 Repair

After the occurrence of additional longitudinal cracking, a new spall develops and it follows the same mechanism mentioned in stage #4.



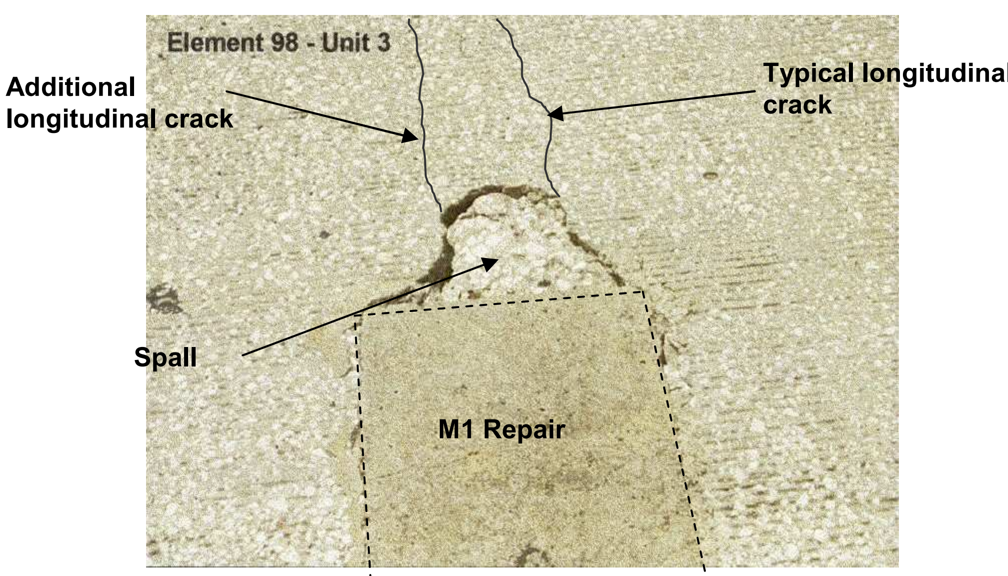


Figure 4.55 Deterioration Stage #9

4.11.10 Stage #10 Cracking on M1 Repair and Adjacent Spalling Increase

When the spall mentioned in stage #9 is not patched quickly, it is very likely that the M1 repair can be fractured due to the constant impact of wheels over it. Impact can also cause growth of adjacent spalls, and delamination between the panel surface and the M1 repair.

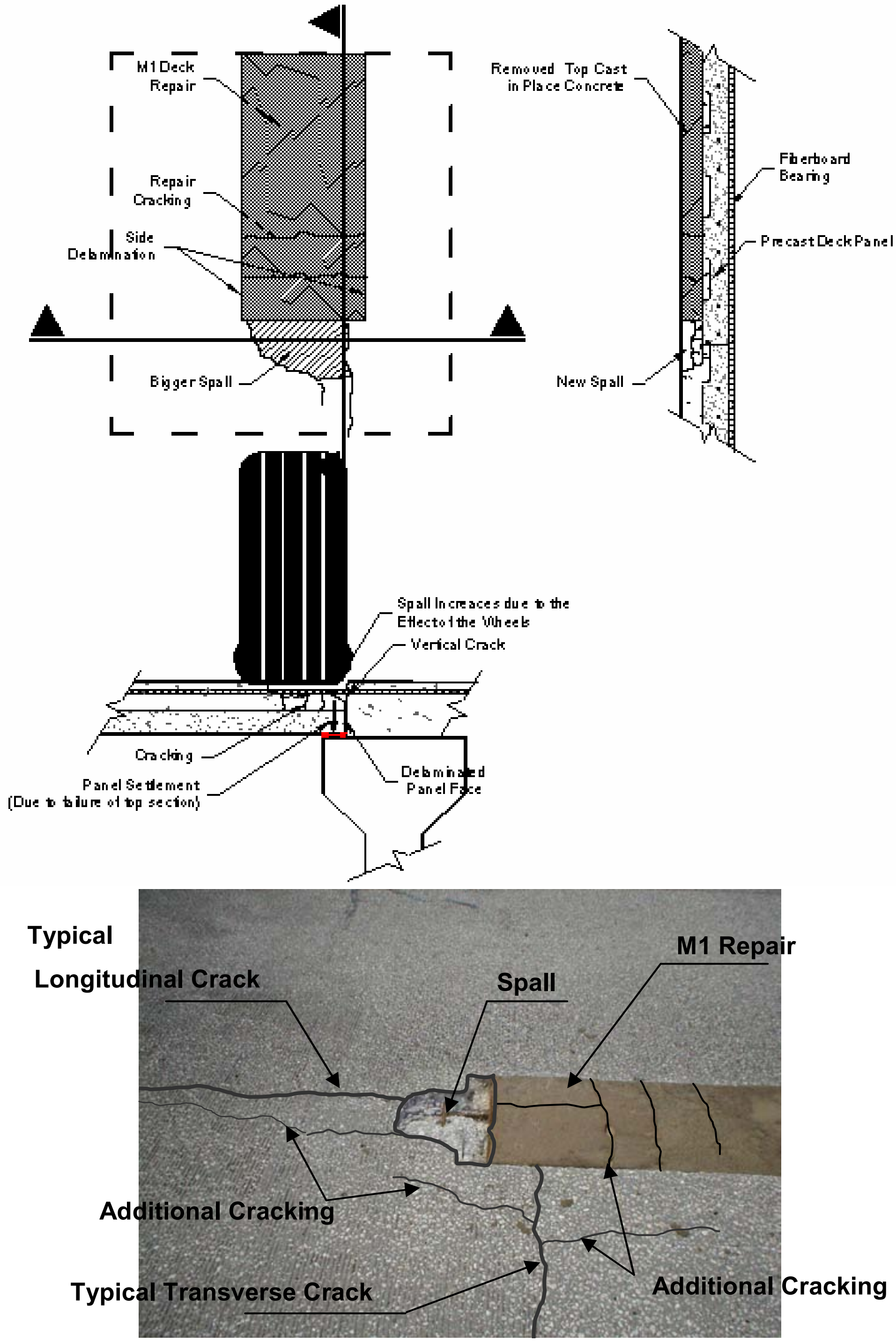


Figure 4.56 Deterioration Stage #10

4.11.11 Stage #11 Adjacent Spall patch

Right after the spall is noticed and depending on its size, it is patched. Usually a quick temporary repair is done, in most cases using flexible material. Then what we have here is a fractured and delaminated M1 repair, plus a flexible material patch. The structural capacity of this deck section is very limited leading to redistribution of stresses to adjacent areas. Also in the case where no positive bearing is provided, the panel will experience movement every time the deck section is loaded due to the lack of a stiff support. In some cases where the fiberboard is deteriorated or is missing, the panel can even touch the top of the girder every time it deflects. Due to the dynamic nature of the wheel loads, panel movement can generate a pulse between the panel and the top of the girder, creating a hammering action and introducing new stresses in the panel. The parameters that affect this stage are (1) Time period between spall beginning, and spall patch, (2) patch material, (3) lack of bond between repair and panel top and, (4) degree of disrepair of the precast panel and the M1 repair.

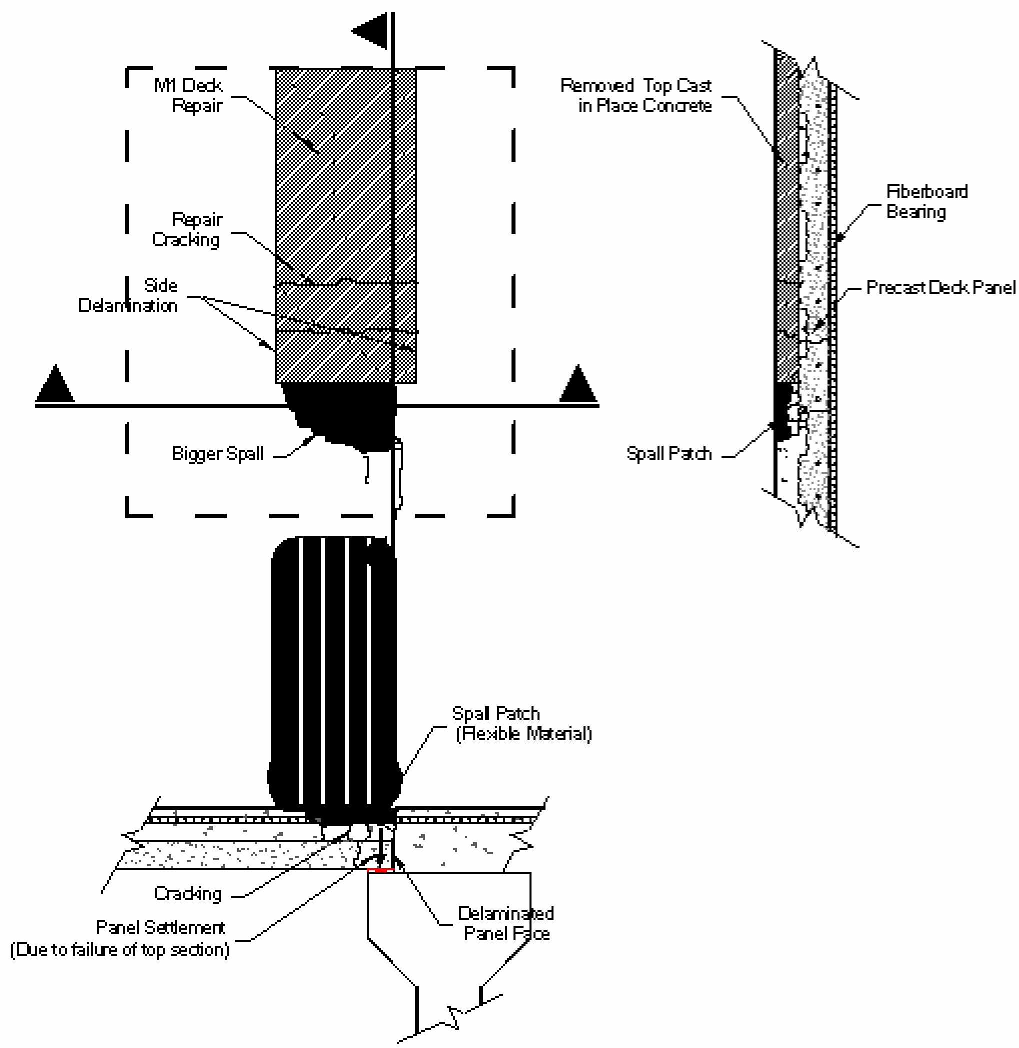


Figure 4.57 Deterioration Stage #11

4.11.12 Stage #12 Additional Adjacent Spalling

After stage #12, since the structural capacity of the section is not restored, deterioration of the deck surface will continue and can generate new spalls adjacent to the previous patch, and next to the edges of the M1 repair. At this point, the deterioration of the deck panel is accelerated by the effect of wheel loads applied over small chunks of concrete over the panel. This concentrates the wheel load over a very small region of the panel surface instead of distributing it over the entire deck section. As a result large stresses are generated in the panel which increases the probability of the occurrence of a punching shear failure of the panel.

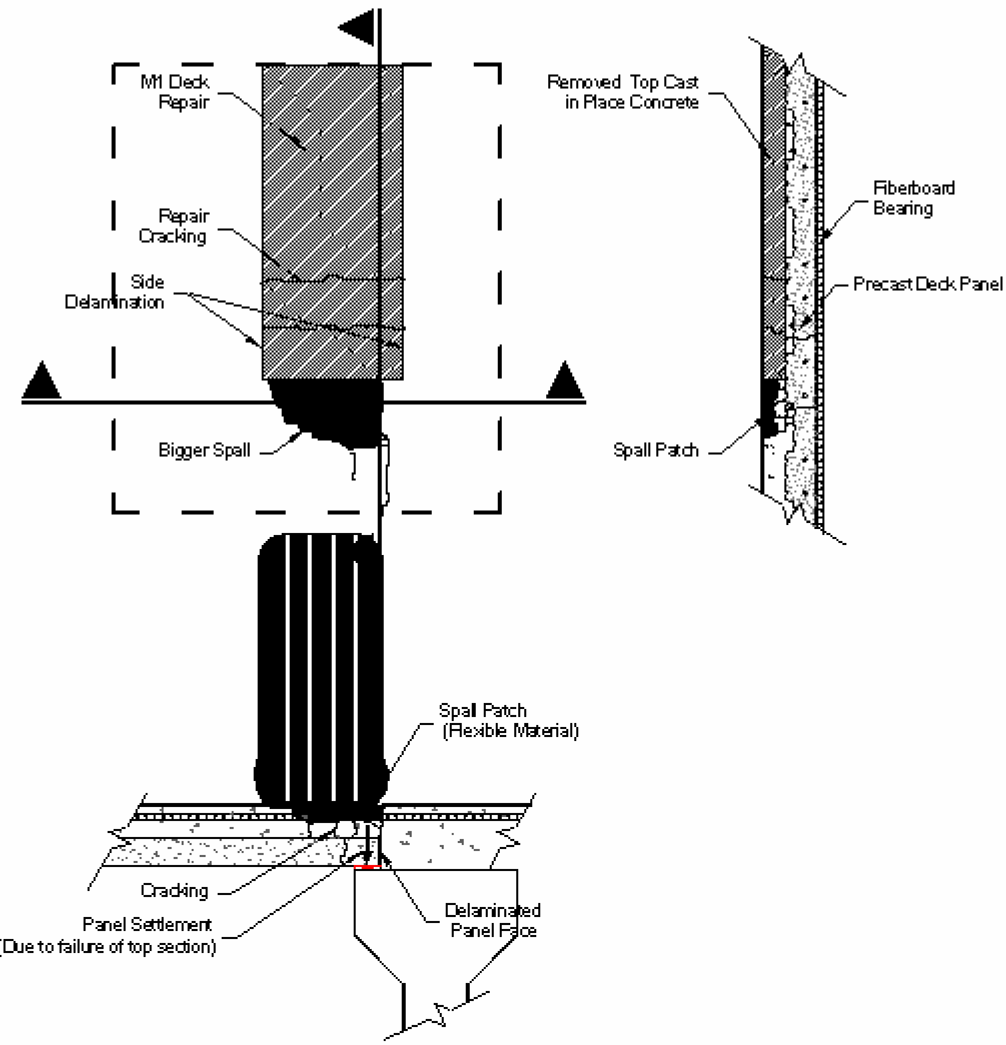


Figure 4.58 Deterioration Stage #12

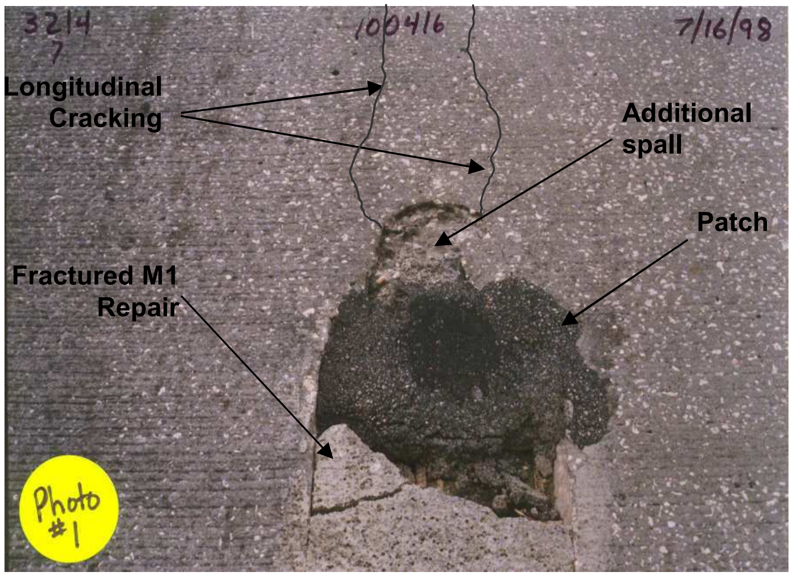
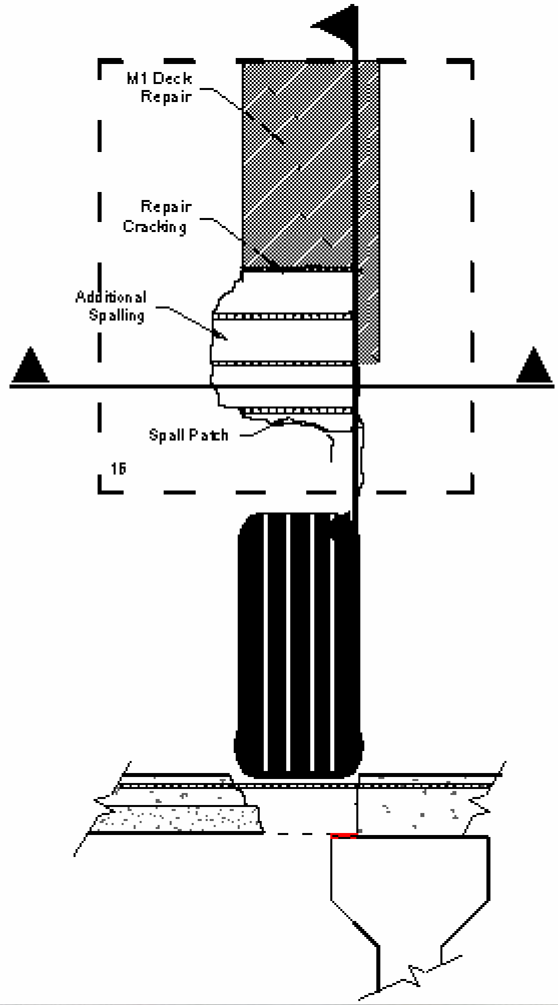


Figure 4.59 Example of Deterioration Stage #12

4.11.13 Stage #13 Deck Localized Failure

After experiencing all the previous deterioration stages, a localized failure is likely to occur. When this happens, the top steel bar is the only structural element that prevents the occurrence of the failure of the entire bay.



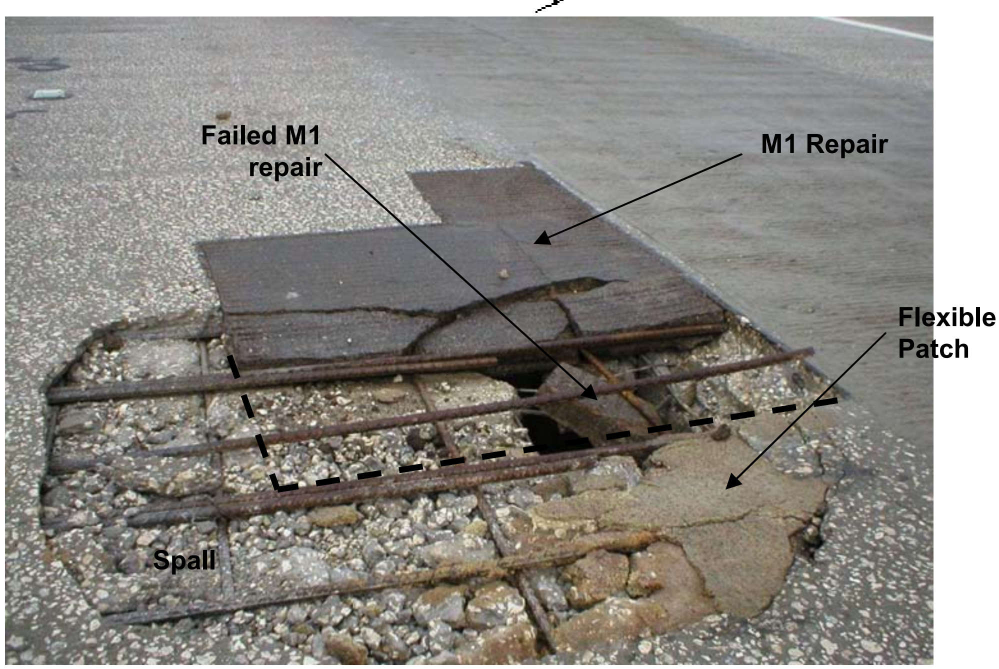


Figure 4.60 Deterioration Stage #13

4.12 Conclusions

- The most important conclusion is that the lack of positive panel bearing is clearly the main factor responsible for the occurrence of major deck deterioration such as delamination, spalling, failing repairs, and in the worst case localized punch-through deck failures. The lack of positive bearing can occur due to two main reasons:

    1. When the initial deck design indicates the exclusive use of fiberboard as a bearing material for the panels.
    2. Or in cases where positive bearing is specified in the design but due to construction deficiencies the panel may not be properly supported over stiff material. (Fig. 4.25 – 4.26).

- Not all the deck panel bridges in Districts 1 and 7 were built using negative panel bearing, i.e. panel supported by fiberboard only, as originally assumed. Four out of seven bridges investigated in the study provided positive support for the panels.

- The occurrence of deck surface longitudinal and transverse cracking is not related to the type of panel bearing whether positive or negative. It can be found for both types of bearings. This type of cracking has proven to remain stable through the years in bridges with positive panel bearing.

- The replacement of the fiberboard by epoxy to provide positive bearing in the panels is an effective way to prevent future deterioration. But it was found that replacement may be difficult to perform in certain cases, i.e. when the fiberboard is too thin and there is insufficient space for the epoxy to flow below the panel.

- Three common factors were found in all the deteriorated decks:

    1. Lack of stiff support for the deck panels (negative bearing)
    2. Wheel loads close to the supports (creating maximum shear stresses)
    3. Vertical crack (due to creep and shrinkage) that reduces the shear capacity of the cast in place concrete.

- The structural behavior of deck panel bridges depends on many factors as described in 4.11, and most of them are very difficult to establish, i.e. type of panel bearing used (no records were found regarding deck panel construction for most of the bridges). This makes it almost impossible to predict the deck condition in the future using numerical analysis.

References

4.1 U.S. Department of Transportation, Federal Highway Administration (2003). National Bridge Inventory.

4.2 Fagundo, F.E., Hays, C.O., Tabatabhai, H, H., and Soongswang, K., "The Effect of Crack Development and Propagation on the Maintenance Requirements of Precast Deck Bridges," Final Report No.U88, Department of Civil Engineering, Engineering and Industrial Experiment Station, University of Florida, Gainesville, July 1984, pp 152.

4.3 Hays, C.O., and Tabatabhai, H., "Summary of Research on Florida Precast Panel Bridges," Structures and Materials Research report No.85-1, Department of Civil Engineering, Engineering and Industrial Experiment Station, University of Florida, Gainesville, August 1985, pp.77.

4.4 Merill, B.D., "Texas Use of Precast Concrete Stay in Place Forms for Bridge Decks", Texas DOT, Bridge Conference, 2002, pp.1-19.

5. NUMERICAL MODELING

5.1 Introduction

A progressive deck failure mechanism based on forensic analysis and inspection data was presented in the previous chapter. This chapter presents numerical analysis of precast deck panel bridges with the aim of quantitatively verifying some of the important failure stages presented earlier. The main focus is on explaining the primary cause of longitudinal cracks observed in such bridges as being a result of creep and shrinkage effects. This is achieved through a non-linear finite element model of the deck, which accounts for the various stages encountered during the construction of such bridges. A brief description of the finite element model is presented in Section 5.2. Details of implementation of concrete Creep and Shrinkage in the analysis are discussed in Section 5.3. Results of numerical simulation of various construction stages are presented in Section 5.4. Section 5.5 presents results from a parameter study conducted with a more complex three-dimensional model to assess the service performance of such bridges. The chapter concludes with a summary presented in Section 5.6.

5.2 Two Dimensional Finite Element Model

In typical deck design, the deck is analyzed as a strip of unit width subjected to design moments specified in AASHTO. This is because the above simplification provides reasonable results since the deck resists loads mainly in one-way slab action. The finite element model used to study the behavior of deck panel also uses a simplified two dimensional representation of the deck. This greatly reduces the complexity and computational cost associated with modeling the non-linear behavior of concrete.

The two dimensional finite element model was developed using ANSYS 5.7, which is a general purpose finite element analysis software. The finite element mesh utilized for the study is shown in Fig. 5.1. It represents a cross-section of a bridge and models two precast panels and corresponding cast-in-place (CIP) concrete supported on three girders. Both the panel and CIP concrete are modeled using “PLANE183” element [5.1], which is a eight node, two dimensional element. It can be used for plain stress analysis and can model non-linear material behavior such as creep. The precast panel is supported on fiberboard, again modeled using PLANE183 element. The lower part of the CIP concrete and fiberboard are attached to spring elements “COMBIN14” to simulate girder support. COMBIN14 can be used as a two-dimensional spring element. The stiffness of these spring elements can be altered to model different girder stiffness. The lower nodes of the spring elements are all constrained along the vertical and horizontal directions.

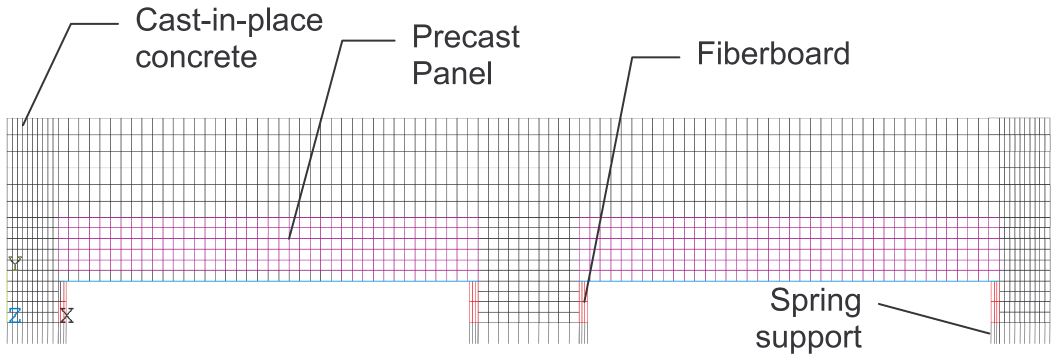


Figure 5.1 Finite element mesh.
(Note – Figure stretched vertically by a factor of 5 for clarity, Typical).

To assist in verification of results, the geometry of the panel used is based on the design example presented in Ref. [5.2]. The precast panel modeled is being 3 in. thick with 5" of CIP concrete to give a total deck thickness of 8 in. The panel used is 7 ft. 9 in. wide, while the width of the CIP portion and fiberboard supported over the girder is 22 inches. The precast panel concrete is modeled as 5000 psi strength concrete, while the CIP concrete uses a strength of 4500 psi. The bearing material is modeled with a modulus of elasticity of 350 ksi to model soft fiberboard [5.3] or 3000 ksi to model positive bearing from concrete or grout. Effect of prestressing (Grade 270, 0.1275 in^2/ft of panel) is modeled by applying equal and opposite forces at the ends of the panel.

The interface between the various components, i.e., between the panel and the CIP concrete, CIP concrete and the fiberboard are modeled using contact elements (CONTAC172). This allows the interface to be modeled as being perfectly bonded, and if needed, as being unbonded (i.e., there is no shear transfer at the interface). All girders in the model are constrained from horizontal translation. This models the condition that occurs near the diaphragms, where all girders are restrained from lateral motion by the diaphragm. The other case to consider is where the girders at not fixed, but provide resistance to lateral motion as result of the transverse stiffness (weak axis bending), which is representative of the condition near mid-span. Preliminary results indicate that stresses developed are qualitatively similar in both cases, however, they are more severe in the first case. Only results from the first model are presented here.

The mesh density was selected based on a convergence study with progressively dense mesh. The final model used for analysis consisted of about 6086 nodes and 2107 elements. As a result of the non-linear material properties and the use of contact elements, the solution utilized Newton-Raphson method [5.1]. The run time on a 3.0GHz Pentium 4 PC, with 2.5GB RAM was less than five minutes per analysis.

5.3 Implementation of Creep and Shrinkage in ANSYS

As stated earlier, the primary objective of the analysis was to quantify the amount of tensile stress developed due to differential creep and shrinkage. Creep and shrinkage behavior of concrete was modeled as using ACI 209 specifications. ANSYS does not include features to directly model concrete creep and shrinkage, however, it provides general creep models that can be adapted to obtain the desired creep behavior. The ACI 209 models creep as follows [5.4]

$$\varepsilon_c(t) = \frac{\sigma_c(t_0)}{E_c(t_0)}\left[1+\varphi(t,t_0)\right] \tag{5.1}$$

where

ε_c	=	concrete strain at time, t
$\sigma_c(t_0)$	=	instantaneous stress at time of loading, t_0
$E_c(t_0)$	=	modulus of elasticity at time t_0
$\varphi(t,t_0)$	=	creep coefficient at time t, given by

$$\varphi(t,t_0) = \frac{(t-t_0)^{0.6}}{10+(t-t_0)^{0.6}}\varphi_u \tag{5.2}$$

φ_u	=	ultimate creep, given by

$$\varphi_u = 2.35\gamma_c \tag{5.3}$$

Here γ_c is a factor that is a function of the relative thickness, ambient humidity and temperature. The multipliers were obtained assuming moist cured concrete at 70% relative humidity and the geometry of the deck given above. Essentially, the equation predicts the strain resulting due to creep at time, t.

The above creep model was implemented in ANSYS using the time-hardening model, which is described by the following equation

$$\Delta\varepsilon_c = C_1\sigma^{C_2}t^{C_3}e^{-\frac{C_4}{T}} \tag{5.4}$$

where

$\Delta\varepsilon_c$	=	incremental creep strain at time t
C_1, C_2, C_3	=	user defined constants
σ	=	element stress
ε	=	element strain

Using equation 5.1, incremental creep between any time, times t_1 and t_2 is given by

$$\Delta\varepsilon_c = \frac{\sigma_c(t_0)}{E_c(t_0)}\left[\varphi(t_2,t_0)-\varphi(t_1,t_0)\right] \tag{5.5}$$

By setting the following values for the coefficient and setting the time used by ANSYS (t_a) to compute the creep strain as shown below, the time hardening model can model the creep as described by ACI 209. Note that in the current implementation, t_a does not represent the actual age of the structure, but a value computed based on the given time using equation 5.10 to "trick" ANSYS into giving the current creep strain.

$$C_1 = \frac{1}{E_c(t_0)} \tag{5.6}$$

$$C_2 = 1 \tag{5.7}$$

$$C_3 = 1 \tag{5.8}$$

$$C_4 = 0 \tag{5.9}$$

$$t_a = \left[\varphi(t_2, t_0) - \varphi(t_1, t_0)\right] \tag{5.10}$$

Shrinkage strain at time t is obtained from the following equation specified by ACI 209

$$\varepsilon_{cs}(t, t_0) = \frac{t - t_0}{35 + (t - t_0)} (\varepsilon_{cs})_u \tag{5.11}$$

where

ε_{cs}	=	shrinkage strain at time t, since time t_0 = 7 days
$(\varepsilon_{cs})_u$	=	ultimate shrinkage strain, given by

$$(\varepsilon_{cs})_u = -780 \times 10^6 \gamma_{cs} \tag{5.12}$$

Here, γ_{cs}, is a factor that depends on relative humidity, temperature and volume-to-surface ratio, and computed using the parameter values used to compute γ_c for creep. Additional details on factors that influence creep and shrinkage can be found in reference 5.2.

The influence of the concrete shrinkage is modeled as by applying a temperature to the appropriate group of elements so as to produce the strain obtained by equation 5.11 at a specified time.

5.4 Modeling Construction Stages

As shown above, creep and shrinkage are a function of several factors. Of these the most important ones are the time of casting, time of load application and time since load application. The loads that influence the creep strains are the prestressing forces applied to the precast panel and the dead load. The stresses are also influenced by shrinkage, which is mainly a function of the number of days passed since casting.

The main objective of the current analysis is to quantitatively model the stresses developed due to creep and shrinkage, since these are thought to be the main cause of the widespread longitudinal cracking observed on precast deck panel bridges. Once the effect of shrinkage and creep are quantified, it may be possible to identify the causes for the observed variation in the extent of cracking on such bridges.

To accurately model creep and shrinkage, it is essential to model all the stages in construction of the bridge. The construction of the bridge can be divided into the following stages

a. The precast panel is cast and subjected to prestressing forces
b. The panel is stored for a certain period of time
c. The panel is then set on bridge girder and the CIP concrete is poured on the panel
d. The CIP concrete hardens and starts acting compositely with precast panel
e. The composite deck ages as time passes

The above five stages are modeled as follows in the finite element model

a. Prestressing load is applied to the panel as equal and opposite force on sides of the panel. The force is reduced by 15% to accounts for losses. Since strands are not modeled, losses due to elastic shortening, creep, shrinkage and relaxation are not computed by the model. However, the 15% loss is thought to be sufficient for the purpose of quantifying the stresses developed in CIP concrete.
b. Creep and shrinkage strains for the panel that result over the storage time are computed.
c. Dead load stresses at the time of pouring CIP concrete are computed.
d. The contact interface between the CIP concrete and the precast panel is activated so that the two start acting compositely.
e. Stresses developed due to differential shrinkage and creep are computed at different periods of time since construction.

5.4.1 Results

Figures 5.2 through 5.7 show results of the simulation of the above steps with a gap of 90 days between the casting of the panel and end of construction for the case with positive bearing in the form of color contour plots. Figure 5.2 shows results of step a listed above, i.e., prestressing the panel. Results are shown in the form of strains. Figure 5.3 shows the same panel strains but assuming 90 days of storage from the day of prestressing. Review of tabular result output (not shown) indicate that as a result of creep and shrinkage, the compressive strain increases to 741 microstrain from 151 microstrain developed initially due to 610 psi prestress.

Figure 5.4 shows the stress along the x axis, σ_x, developed due to dead load from the CIP concrete acting on the panel. The stress distribution shows classic bending behavior with maximum compression (about 1.04 ksi) at top of the panel and 175 psi at bottom. Note that the CIP concrete is still not capable of resisting loads at this stage.

Figures 5.5 through 5.7 show stresses resulting from differential creep and shrinkage at day 90 after the CIP concrete and panel start acting as a composite body. From Figure 5.5, it is seen that the primary response of the structure to shrinkage and creep is to develop tension in the CIP concrete. This is because the shrinkage of the CIP concrete is resisted by the panel, which, due to its age has undergone major share of its

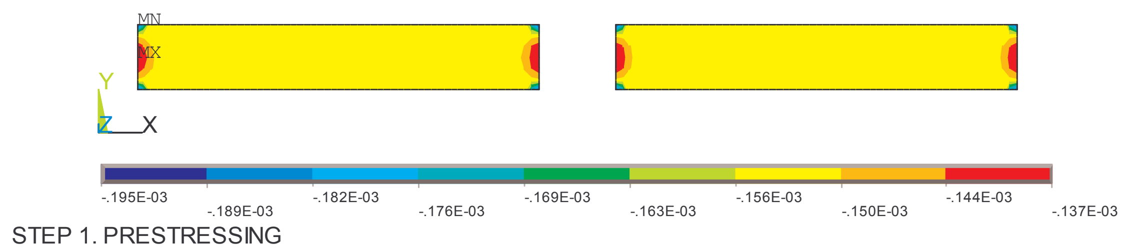


Figure 5.2 Strain ε_x on panels, due to prestressing.

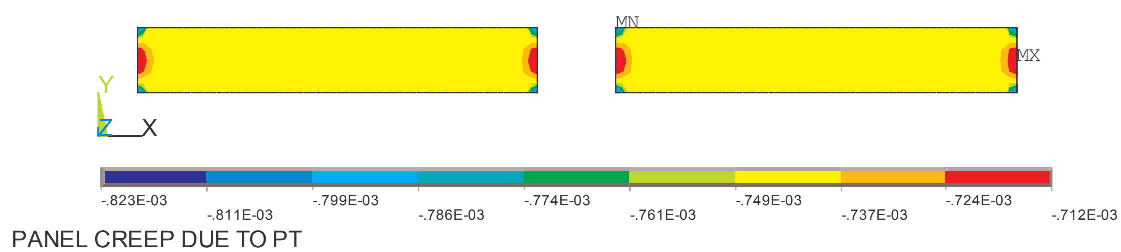


Figure 5.3 Strain ε_x on panels, due to prestressing, creep and Shrinkage 90 days after prestressing.

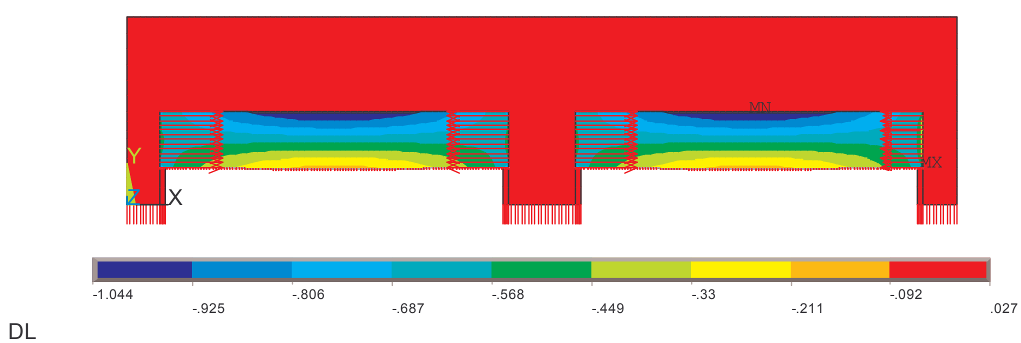


Figure 5.4 Stress σ_x (ksi) on panels, due to prestressing, creep, Shrinkage and CIP dead load 90 days after prestressing.

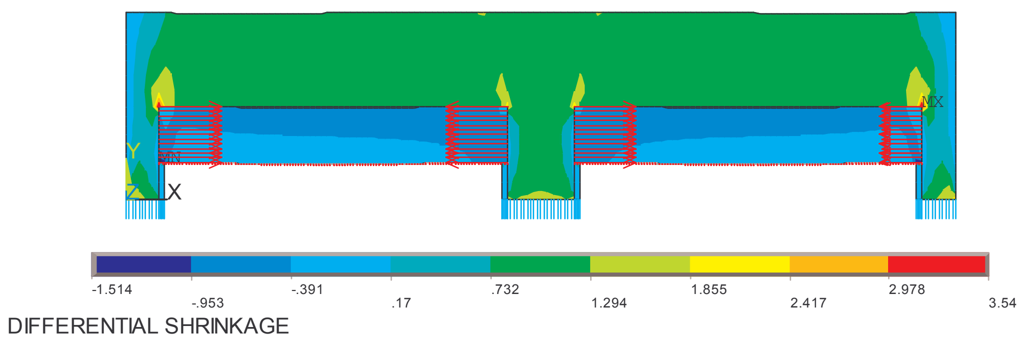


Figure 5.5 Stress σ_x(ksi) on composite system due to prestressing, creep, shrinkage and CIP dead load 90 days after construction.

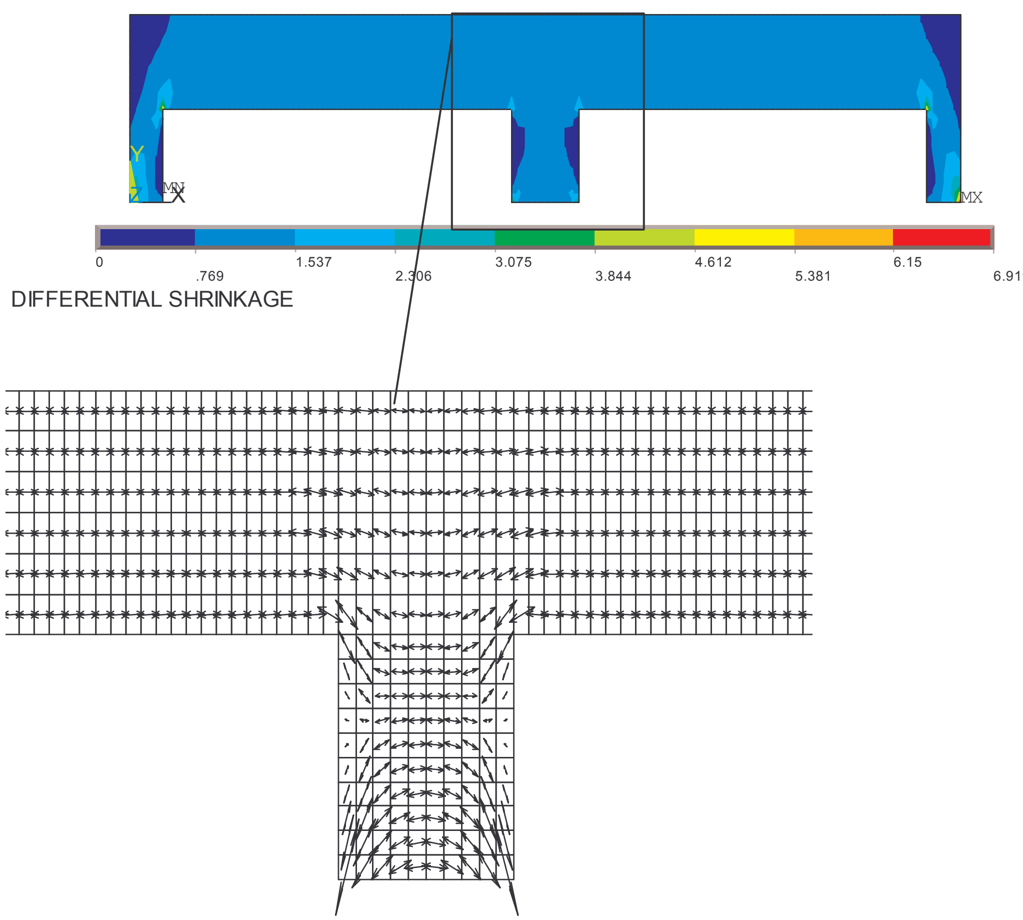


Figure 5.6 Principal Stress σ_1 on CIP concrete, due to differential Shrinkage and Creep load 90 days after construction. Blow up shows vector representation of the tensile stresses.

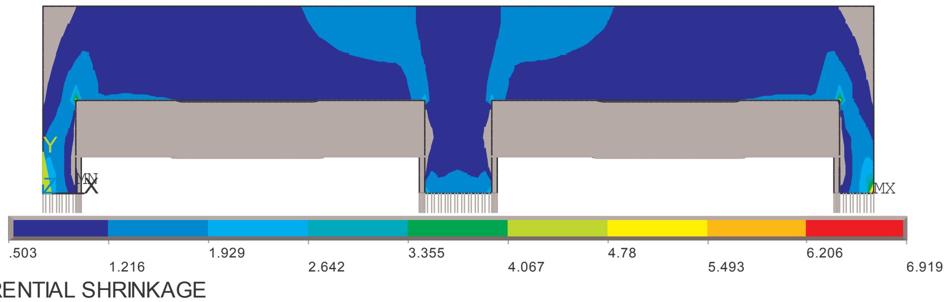


Figure 5.7 Regions with Principal Stress σ_1 > 0.503 90 days after construction.

shrinkage before the CIP concrete is poured. Figure 5.6 shows principal stresses distribution developed in the CIP concrete. The tensile nature of the stress is clearly seen in the vector representation of the principal stress shown in the close up of the central portion of the model. Figure 5.7 shows regions of the CIP concrete where the stress due to differential shrinkage and creep exceeds 0.503 ksi, which is the design tensile capacity of 4500 psi concrete. Given the large area under tensile stresses, one would expect more widespread cracking, however since the CIP portion of the deck is reinforced, and since shrinkage and creep loads are relieved on any cracking, the cracking is limited to areas near the panel to CIP concrete joint, where there is no reinforcement present.

It can be seen from the above example that differential shrinkage and to a lesser degree differential creep can easily lead to the cracking observed on deck panel bridges even with positive bearing. The results show that live-load is not required to initiate these cracks, although it would definitely be a factor in developing these cracks further and eventually causing spalls. This result is consistent with the observations of the forensic study presented in Chapter 4, where cracks were found even on shoulders where the live load cannot be a factor in the crack formation.

5.5 Preliminary Parameter Study

The initial project plan for prioritization involved detailed finite element analysis of individual bridges to estimate its residual capacity. As a preliminary step, a three dimensional model (Figure 5.8) was developed and used to run a parameter study to determine the parameters that lead to maximum live-load stresses from a HS-20 design load [5.3] placed near the girder line. Construction loading from creep and shrinkage were not considered in the analysis. The analysis conducted using a fractional factorial design [5.5] studied the effect of the ten parameters at levels listed in Table 5.1.

Fractional factorial design involves comparing results of models at high level of parameter (see Table 5.1, Column 3) with results obtained from low level of parameter (see Table 5.1, Column 2). For example, in the present case, the maximum deck service tensile stress obtained with Type II girder is compared with that obtained with Type IV girder to determine the significance of girder stiffness. The parameter is considered to be important if there is a significant difference in the results between the high and low value. In addition to the effect of the parameters listed in the table, fractional factorial design provides information on significance of the interaction between two variables. An example of interaction in the present case may be that bearing support condition (item 7 in Table 5.1) may not be significant as long as there is vertical composite action (item 5, Table 5.1). However, bearing support may become significant if there is no vertical composite action.

Table 5.1 Parameters values used for preliminary parameter study.

Parameter	Low	High
1. Girder Stiffness	Type II	Type IV
2. Span Length	10 X	20 X
3. Deck Concrete Quality	3400 psi	3750 psi
4. Composite Action (Horizontal)	Yes	No
5. Composite Action (Vertical)	Yes	No
6. Panel Thickness	3.5”	3.75”
7. Bearing Support Condition	No support	Positive support
8. Deck thickness	7”	7.5”
9. Panel Concrete	4000 psi	6000 psi
10. Panel Width (X)	5.25’	10.50’

Of these, composite action at horizontal interface and vertical interface was found to be the significant parameters. The utility of the above study was limited by the omission of creep and shrinkage, and as a result no useful conclusion could be arrived at. A sample result indicating the effect of horizontal interface composite action is shown in Figure 5.8, which represent the stresses developed at the horizontal interface of the CIP concrete. The Figure 5.8a shows the case where there is composite action at the horizontal interface, while 5.8b shows the case without composite action. The stresses in the section case are higher as indicated by the darker shading close to the region of load application.

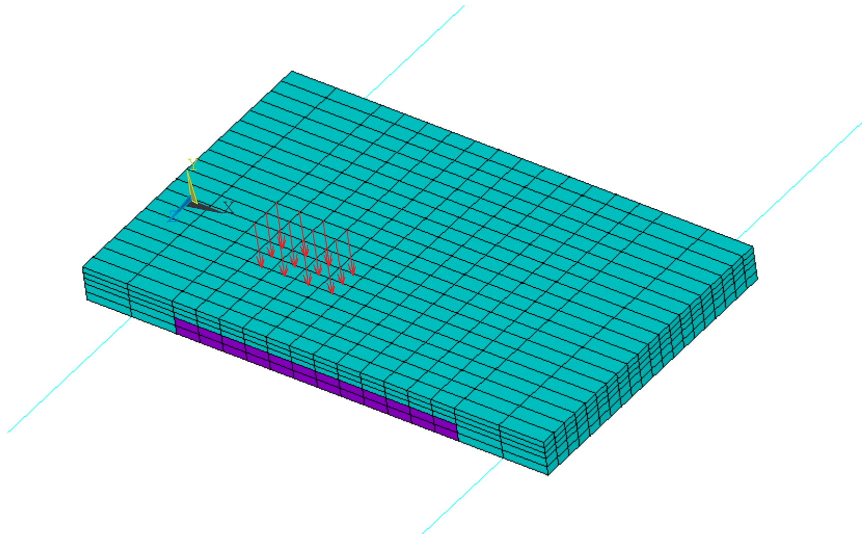
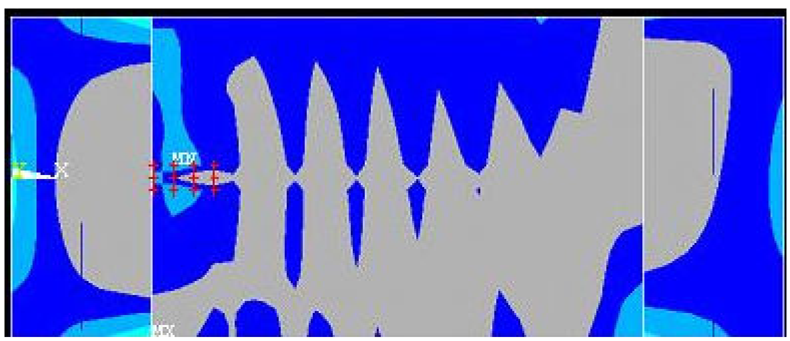
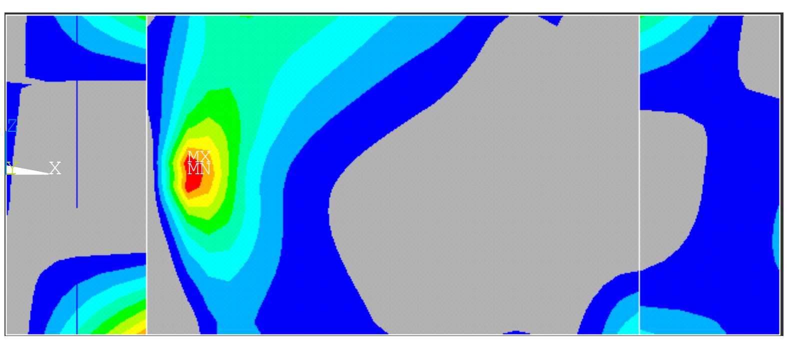


Figure 5.8 Three dimensional model used for preliminary parameter study.



(a)



(b)

Figure 5.9 Effect of horizontal composite action on stress in CIP concrete: (a) with composite action (b) without composite action.

5.6 Summary

The analysis presented in section 5.4 showed that differential shrinkage and creep can cause the cracking observed on bridges. Since shrinkage and creep are mainly a function of the time elapsed since casting, one can expect different results if the construction schedule is varied. For example, the tensile stresses will vary as the number of days the precast panel is stored is varied. Since the casting day and time elapsed between casting and installation of precast panels is likely to vary widely, so is the extent of cracking on bridges. As stated earlier, this has been found in the field, where identical bridges located on a viaduct and hence subjected to same to similar traffic display dramatically different extent of cracking. As a result other parameters, such as span length and panel dimensions play a secondary role.

References

5.1 ANSYS, Inc. (2000). ANSYS user manuals for Release 5.7. Cannonsburg, PA, ANSYS, Inc.

5.2 http://www.pjwhitehardwoods.com/products/fiberbrd_particlebrd.htm

5.3 Ross Bryan Associates Inc (1988), Recommended Practice for Precast Prestressed Concrete Composite Bridge Deck Panels, PCI Journal, March-April 1988.

5.4 ACI 209 Committee (1982), Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures.

5.5 D. Montgomery (2001), Design and Analysis of Experiments, 5 Edition, John Wiley and Sons, New York.

6. USF BRIDGE DECK INSPECTION

6.1 Introduction

In order to obtain up to date information on the condition of precast deck panel bridges, additional inspections were carried out by USF. Two series of inspections were carried out. During March 2004, a total of 101 precast deck panel bridges were inspected. A subsequent inspection was conducted at the end of June in which all 17 bridges identified as being in relatively poor condition in the first inspection were re-inspected.

Section 6.2 describes the inspection method developed by the research team. A summary of the findings from the first inspection is presented in Section 6.3 while Section 6.4 provides information on the re-inspection of selected bridges. The conclusions from this chapter are summarized in Section 6.5.

6.2. USF Inspection Method

Based on the evidence of deck panel deterioration reported in Chapters 3 and 4, the focus of the inspection was on the damage occurring on the top deck surface. Of particular interest were spalls, repairs and re-repairs. In the inspection, lasers were used to measure the size of the damage or repair to allow its progression to be monitored. Photographs of damage or repairs were taken and all information compiled was incorporated in AUTOCAD drawings of the plan view of the deck. These identified their approximate location in plan to provide an overview of the condition of the bridge. This information was stored electronically in a database and was used by the PANEL software for prioritizing deck replacement (see Chapter 8).

A laser distancemeter, DISTO™ classic[5] (manufactured by Leica Geosystems, Switzerland) was used (Fig. 6.1) was measuring the spall size. According to its manufacturer, the typical accuracy is ± 3 mm with a maximum of ± 5 mm [6.1]. This was found to be valid for direct measurements - not when triangulation was required as in our case. Here readings were found to be inaccurate.

Triangulation requires measurement of both distance and angles. Vertical and horizontal angles from a reference point were measured using a theodolite with a digital readout (Model: Sokkia DT6). The laser distancemeter was mounted on top of the theodolite so that it rotated in the same plane and about the same axes. A special fixture shown in Fig. 6.1 was fabricated for this purpose. Since the laser point could become invisible outdoors, a telescopic viewer BFT4 was also required. This was magnetically attached to the lasermeter. A complete photograph of the setup is shown in Fig. 6.1.

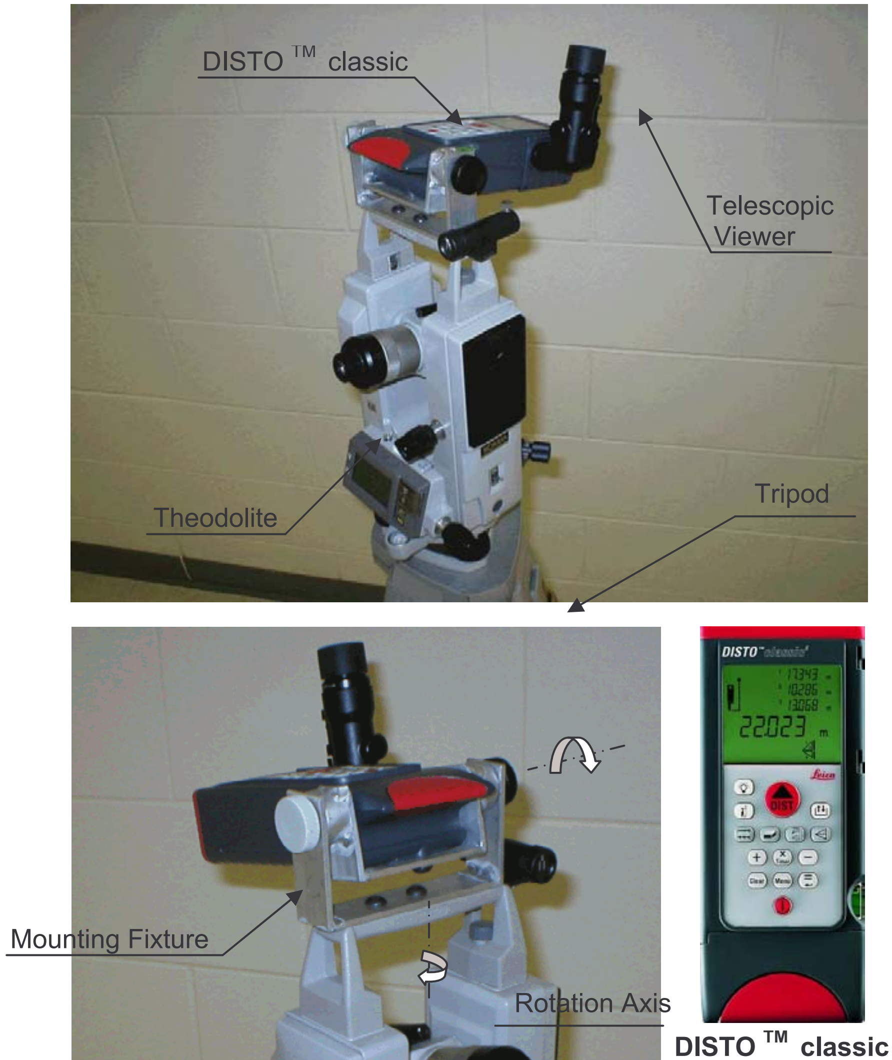


Figure 6.1 Laser Based Measuring Device (DISTO TM classic)

6.2.1 Procedure

Prior to the inspection, scaled drawings of the plans of each bridge were prepared. The plan drawings include dimensions and exact locations of the lanes and girders. Typically, two member crews were used for safety reasons. Safety precautions included (1) 'frequent stop' sign on the truck, (2) wearing of reflective vests, (3) hard hats and, (4) flashing lights in the vehicle. The following steps were employed:

1. A visual inspection of the entire deck was conducted from the shoulder and each deficiency photographed. The approximate location of the damage and its corresponding photograph number in the digital camera were recorded on the plan drawings.
2. The laser distancemeter was used to measure the length of the damage. Two measurements were made corresponding to the two extremities of the damage. At

the first point (P1), the distance to the extremity and the vertical angle were measured. The theodolite was then swiveled horizontally to the other extremity (P2) and both the horizontal and vertical angles to this point were measured along with the distance from the reference point. The readings were taken twice and recorded on the inspection data sheet for future processing (see Fig. 6.2).

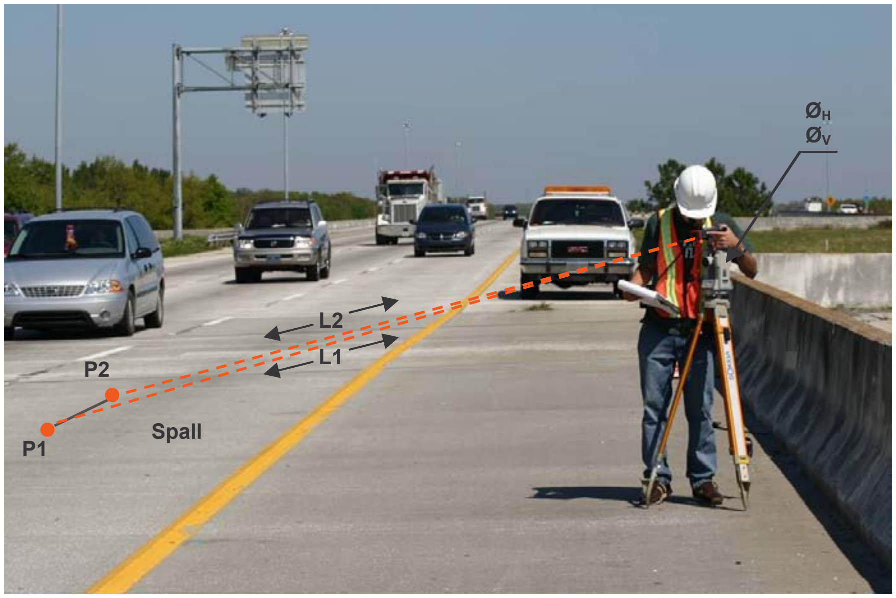


Figure 6.2 Measuring Spall Length

3. The dimensions of the damage were calculated from the measurements made in Step 2 using a spreadsheet (Fig 6.3).

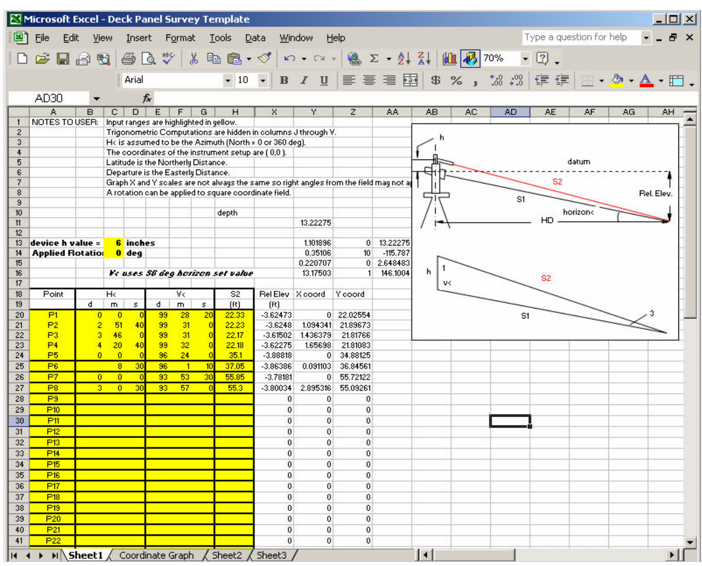


Figure 6.3 Spreadsheet Developed to Calculate Dimensions.

4. The location and extent of damage on the deck was plotted using AUTOCAD. A typical plot is shown in Fig. 6.4.

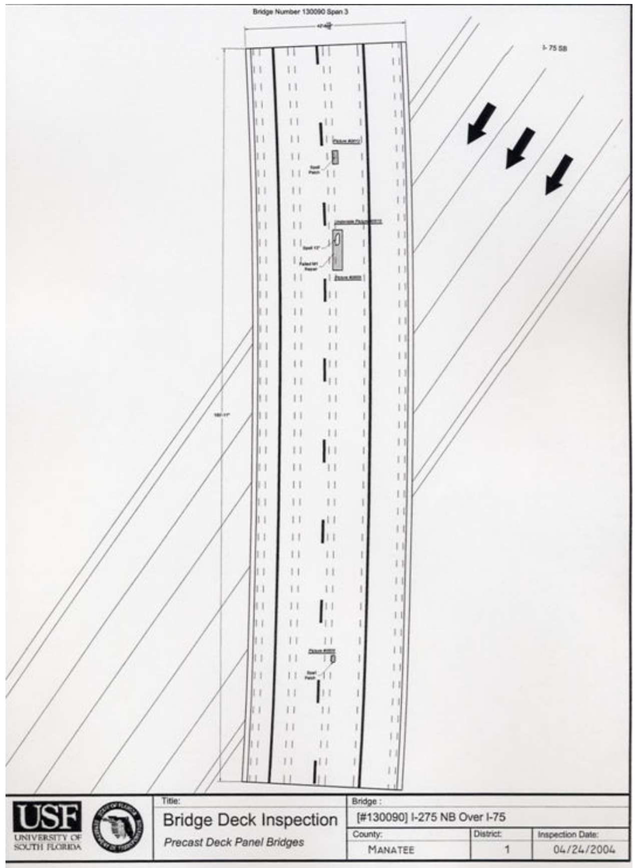


Figure 6.4 AUTOCAD Plot of Damage

6.3 March Inspection Results

As indicated in Section 6.2, all data compiled in the field were analyzed using an EXCEL spreadsheet to establish the length of the damage or the repaired region. Additionally, all photographs were reviewed.

Based on the number and condition of the deficiencies, decks were classified into five groups ranging from 'serious' to 'good'. 'Serious' (Group I) identified bridges that had many M1 repairs, failed repairs, walking spalls or isolated repairs. 'Poor' (Group II) identified bridges that had M1 repairs and walking spalls, 'Fair' (Group III) had few M1 repairs and spalls. Bridges classified as 'good' (Group V) or 'satisfactory' (Group IV) had only random cracking or stable repairs. These definitions are summarized in Table 6.1.

Table 6.1 Bridge Classification Groups

Group	Condition	Description
Group I	Serious	Many M1 repairs, failed repairs, walking spalls or isolated spalls.
Group II	Poor	M1 repairs, repaired walking spalls, spall patches.
Group III	Fair	Few M1 repairs, few spall patches.
Group IV	Satisfactory	Stable repairs, (Most of cases not related to deck panel), or full depth repairs
Group V	Good	Random cracking only.

District 1

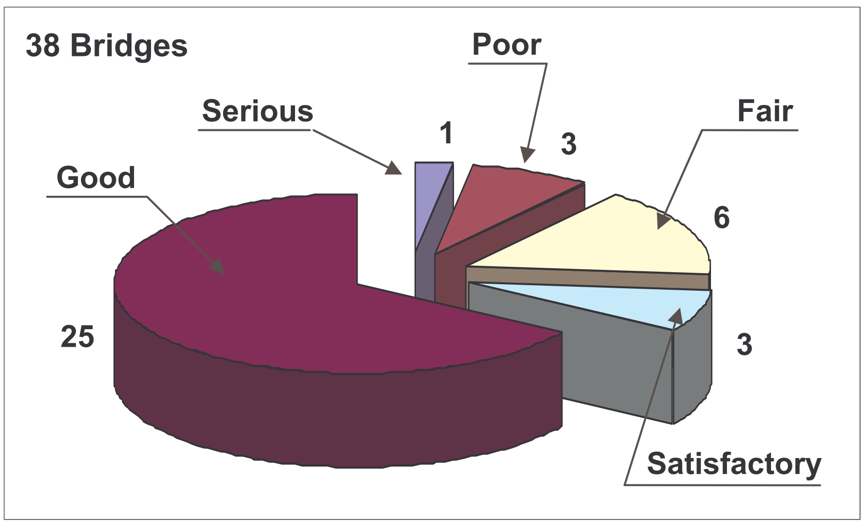


Figure 6.5 Inspection Summary - District 1

Of the 74 bridges in District 1, 38 that had not been replaced or were scheduled for replacement during 2004 were inspected.

A pie chart showing the results of the inspection of 38 bridges in District 1 is given in Fig. 6.5. Of the 38 bridges, only one was rated as '*serious*', three as '*poor*' and six as '*fair*'. These 10 bridges were candidates for the second inspection conducted three months later. The remaining 28 bridges were considered to be '*satisfactory'* (3) or '*good'* (25). Table 6.2 provides a summary of the findings for each bridge. Photographs showing typical deterioration in the worst four bridges are shown in Figs. 6.6-6.9. In each of these figures AUTOCAD drawings with the location of the damage in the plan view are also shown. Photographs and AUTOCAD drawings for these 38 bridges were stored in a database and can be accessed by the PANEL software (see Chapter 7).

Table 6.2 District 1 Bridge Classification

Condition	Bridge ID#	Location	Findings
Serious	130090	I-275 NB over I-75	1 failed M1 repair 11 M1 repairs 4 spalls 3 spall patches
Poor	170081	I-75 over Palmer Blvd.	5 M1 repairs 2 spalls 1 epoxy patch
	170080	I-75 over Main A Canal	3 M1 repairs 3 spalls
	130112	I-275 SB To I-75 NB Ramps	46 M1 repairs 5 full depth repairs 1 deck repair 3 patches
Fair	170100	SR-681 NB over CSX RR	1 M1 repair 1 spall patch
	170099	SR-681 SB over CSX RR	5 M1 repairs
	170094	I-75 NB over Havana Rd.	5 M1 repairs
	010064	Oil Well RD over I-75	1 spall 1 spall patch
	030188	I-75 over CR-846	5 M1 reapairs
	170089	I-75 over CSX RR	1 spall 2 spall patches
Satisfactory	030187	I-75 over CR-846	1 M1 repair
	170096	I-75 SB over Jacaranda Blvd.	2 full depth repairs
	170079	I-75 over Main A Canal	1 spall patch

Table 6.2 (continued) District 1 Bridge Classification

Condition	Bridge ID#	Location	Findings
Good	010059	I-75 over CR-776	No significant damage
	010065	Airport RD over I-75	No significant damage
	010066	CR-768 over I-75	No significant damage
	010067	US-17 over Florida St.	No significant damage
	010068	US-17 over Florida St.	No significant damage
	010075	Carmalite St. over I-75	No significant damage
	010090	US-17 over Lavilla St. & RR	No significant damage
	010091	US-17 over Lavilla St. & RR	No significant damage
	120085	US-41 over Imperial River	No significant damage
	120086	US-41 over Imperial River	No significant damage
	120088	SR-685 over Matanzas Pass	No significant damage
	120114	Slater Rd. over I-75	No significant damage
	120126	I-75 NB over Alico Rd./Canal	No significant damage
	120127	I-75 SB over Alico Rd./Canal	No significant damage
	130085	I-75 NB over SR-64	No significant damage
	130089	Erie RD over I-75	No significant damage
	130107	Mendoza RD over I-75	No significant damage
	170082	I-75 over Palmer Blvd.	No significant damage
	170083	I-75 SB over SR-780	No significant damage
	170084	I-75 NB over SR-780	No significant damage
	170090	I-75 over River Rd.	No significant damage
	170091	I-75 SB over Jackson Rd.	No significant damage
	170092	I-75 NB over Jackson Rd.	No significant damage
	170093	I-75 over SR-80	No significant damage
	170095	I-75 N2B over Jacaranda Blvd.	No significant damage

Serious (Group I)

- I-275 NB over I-75 (Bridge # 130090)

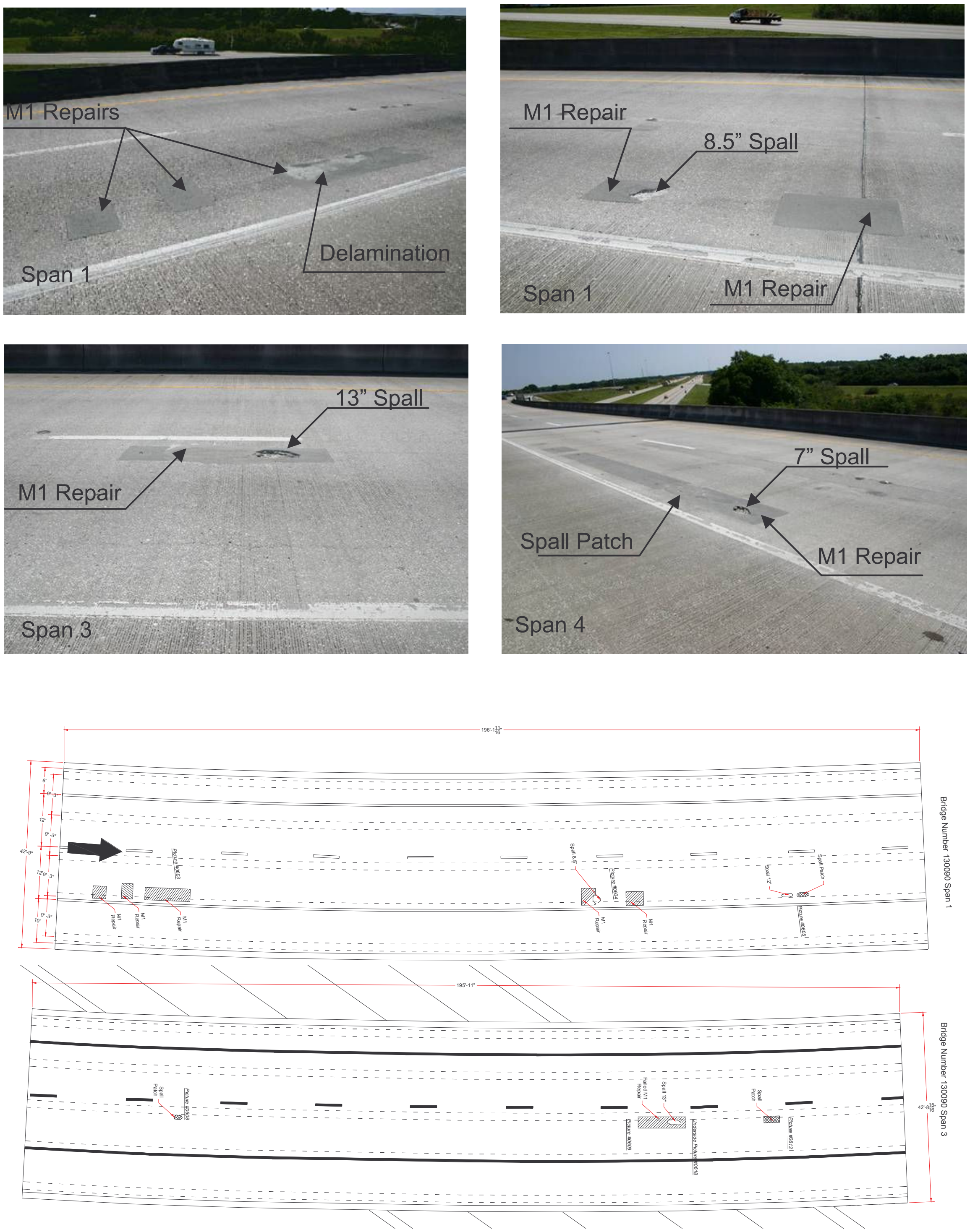


Figure 6.6 Deterioration in Bridge #130090 (March 2004)

Poor (Group II)

- I-275 SB to I-75 NB Ramp (Bridge #130112)

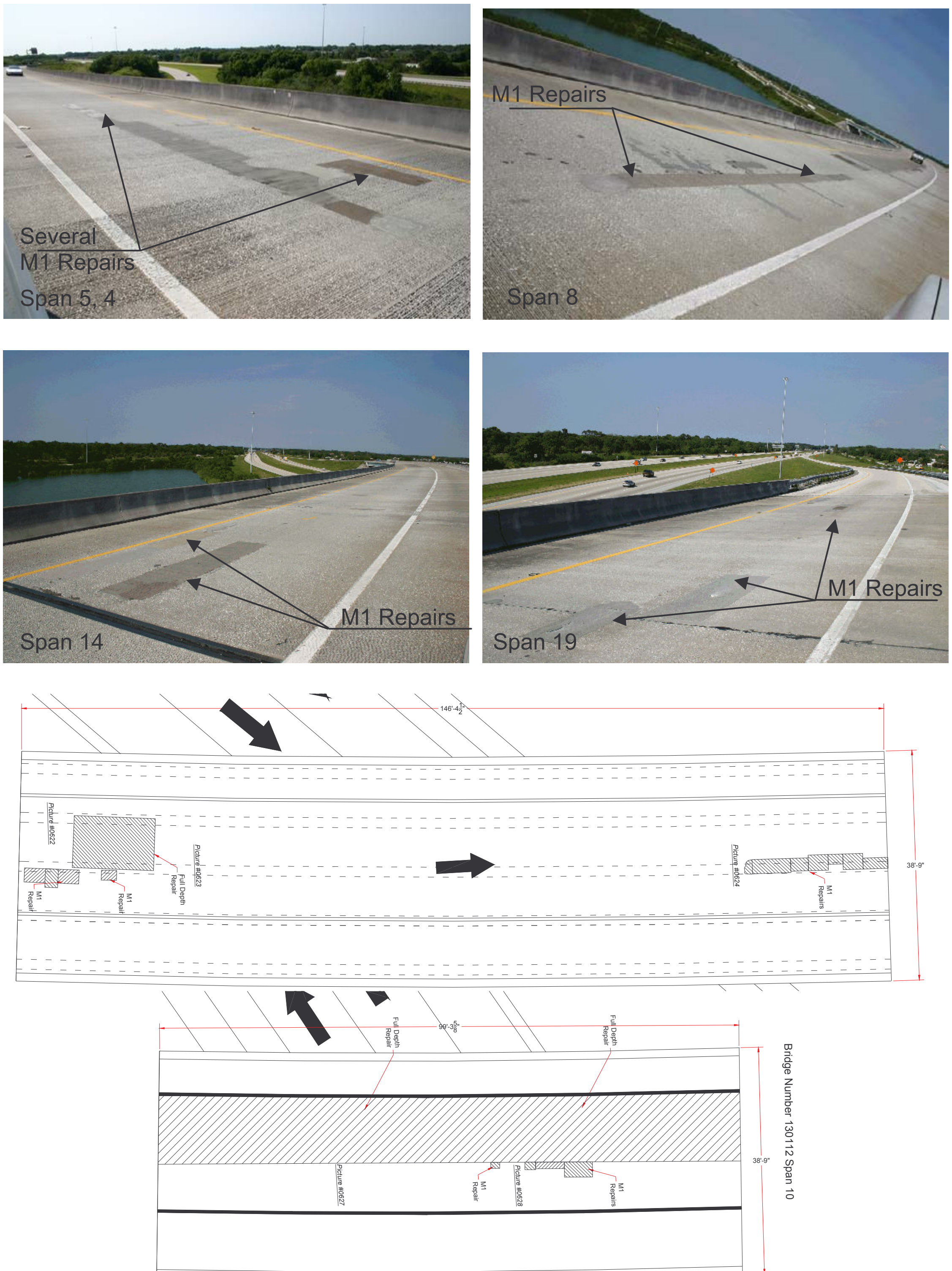


Figure 6.7 Deterioration in Bridge #130112 (March 2004)

- I-75 NB over Main Canal (Bridge #170080)

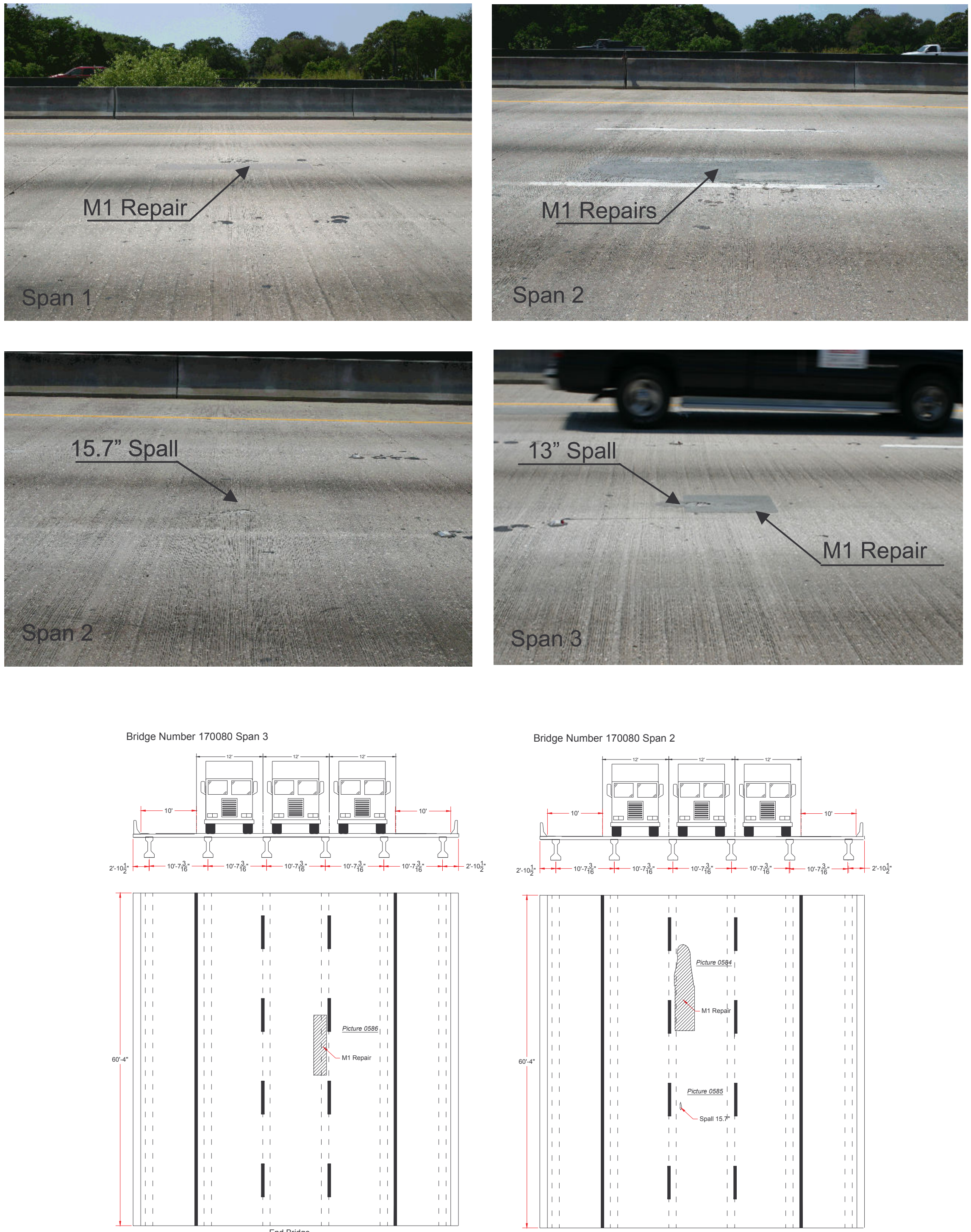


Figure 6.8 Deterioration in Bridge #170080 (March 2004)

- I-75 SB over Palmer Blvd (Bridge 170081)

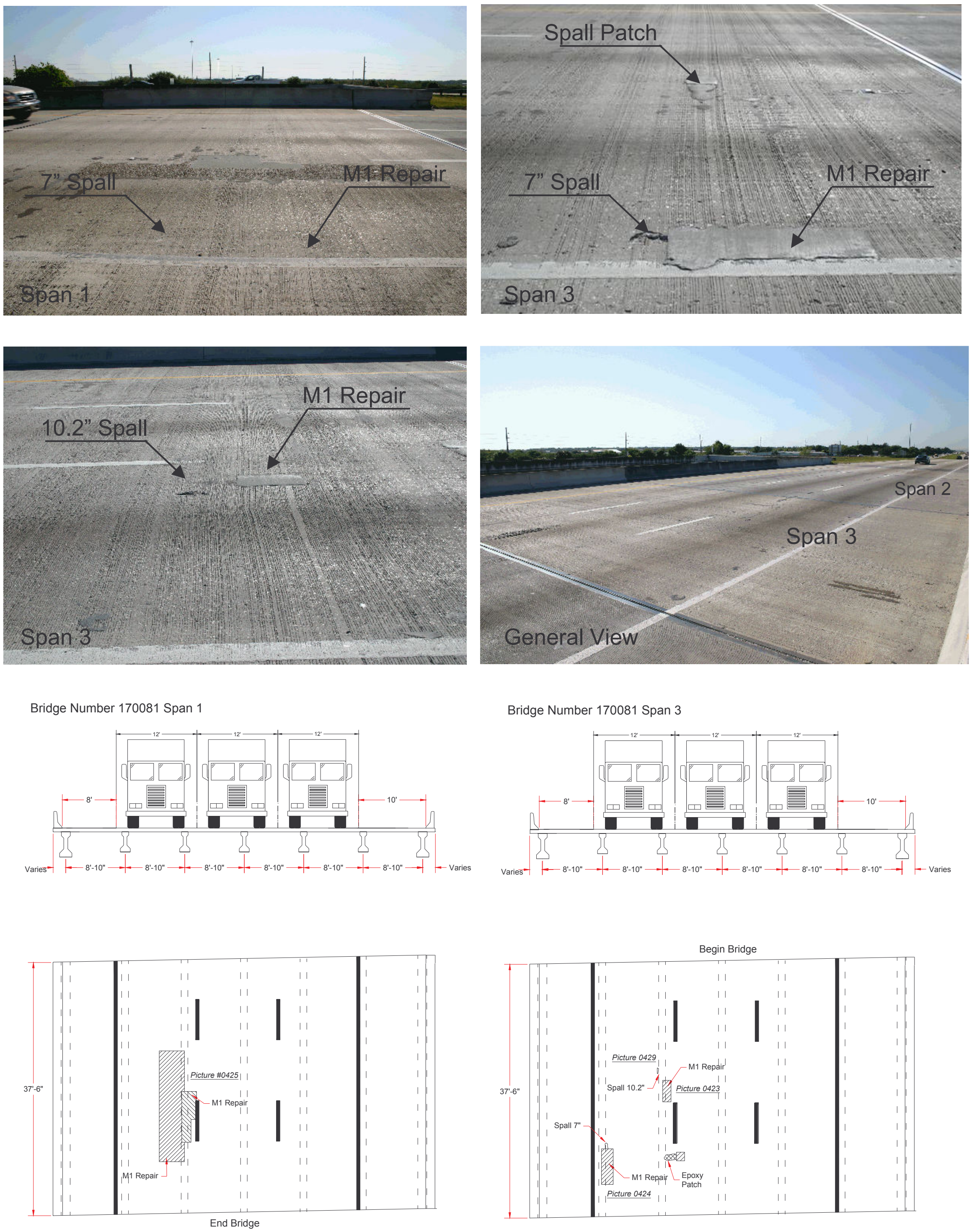


Figure 6.9 Deterioration in Bridge #170081 (March 2004)

District 7

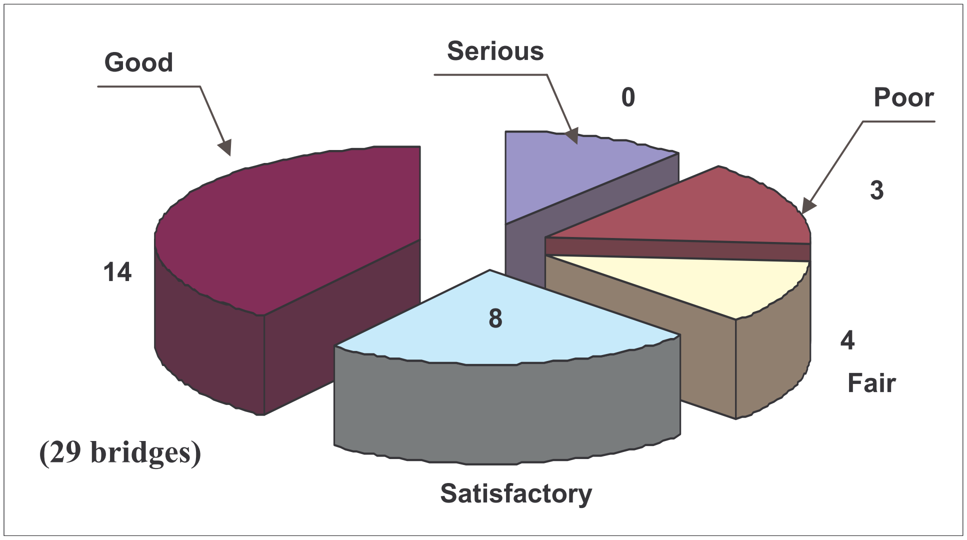


Figure 6.10 Inspection Summary - District 7

A total of 35 bridges (excluding Cross Town) were inspected of which 6 were scheduled for replacement during 2004. Only results for the remaining 29 bridges are presented here.

A pie chart showing the results of the inspection of 29 bridges is given in Fig. 6.10. Of the 29 bridges, none were rated as '*serious*', three were rated as '*poor*' and four others rated '*fair*'. These 7 bridges were candidates for the second inspection conducted three months later. The remaining 22 bridges were considered to be '*satisfactory'* (8) or '*good'* (14).

Table 6.3 provides a summary of the findings for each bridge. Photographs showing typical deterioration in the worst three bridges are shown in Figs. 6.11-6.13. As before, AUTOCAD drawings showing the location of the damage in the plan view are also shown in these figures. Photographs and AUTOCAD drawings for these 29 bridges were stored in a database and can be accessed by the PANEL software (see Chapter. 7).

Table 6.3 District 7 Bridge Classification

Group	Bridge ID#	Location	Findings
Serious**	100363*	I-75 SB over CR-672	18 M1 repairs 1 spall 16 epoxy patches
	100397*	I-75 SB over Sligh Ave. & Ramp D-1	-
	100415*	I-75 NB over US-92	-
	100416*	I-75 SB over Ramp B1	-
Poor	100347	I-75 NB over SR-674	1 M1 repair 1 major epoxy patch 2 epoxy patches
	100364*	I-75 NB over CR-672	7 M1 repairs 1 epoxy patch 3 joint repairs
	100468	I-75 SB over Woodberry Rd.	2 M1 repairs 1 epoxy patch
	100470	I-75 SB over CSX RR	1 M1 repair 1 spall
Fair	100346	I-75 SB over SR-674	2 M1 repairs
	100358	I-75 SB over Alafia River	3 M1 repairs 8 Full depth repairs 2 spalls 4 epoxy patches
	100359	I-75 NB over Alafia River	1 M1 repair 2 full depth repairs 2 epoxy patches
	150122	I-275 NB over 5^{th} Ave. N	1 M1 repair 3 epoxy patches
Satisfactory	100049	US-41 over Palm River	1 joint repair
	100080	SR-60 WB over Tampa Bypass Canal	1 repair 3 concrete patches
	100081	SR-60 EB over Tampa bypass Canal	2 M1 repairs 16 joint repairs
	100338	BUS-41 over MacKay Bay	4 M1 repair 2 spall patch 1 delamination 1 joint repair
	100351	Valroy Rd. over I-75	3 M1 repair 1 patch
	100356	I-75 SB over Riverview DR	3 full depth repairs
	100357	I-75 NB over Riverview DR	2 full depth repairs
	100436	I-75 NB over CR-574 & CSX RR	2 M1 repairs 1 joint repair

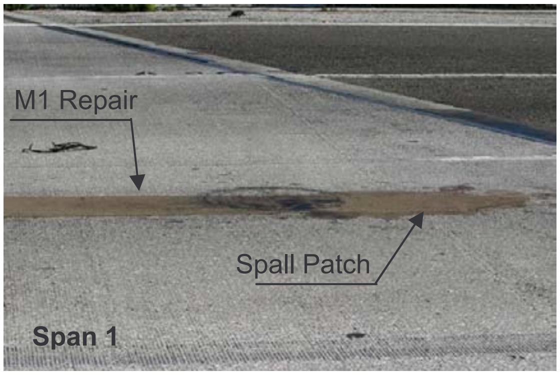
Table 6.3 (continued) District 7 Bridge Classification

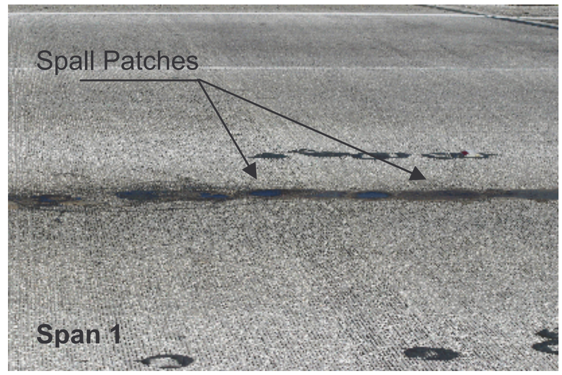
Group	Bridge ID#	Location	Findings
Good	100398	I-75 NB over Sligh Ave. & Ramp D-1	No significant damage
	100339	US 301 over Tampa Bypass Canal	No significant damage
	100377	Gibsonton Dr. over I-75	No significant damage
	100399	SR 582 WB over Tampa Bypass Canal	No significant damage
	100417*	I-75 NB over Ramp B-1	No significant damage
	100424	Ramp B over US 92	No significant damage
	100435	I-75 SB over CR-574 and CSX RR	No significant damage
	100469	I-75 NB over Woodberry Rd.	No significant damage
	100471	I-75 over CSX RR	No significant damage
	150121	I-275 SB over 5th Ave. S & 5th Ave. N	No significant damage
	150145	I-375 WB over CR-689	No significant damage
	150146	I-375 EB over CR-689	No significant damage
	150168	I-175 WB over 6th St. S	No significant damage
	150169	I-175 EB over 6th St. S	No significant damage
	150170	8th ST. S over I-175	No significant damage

* Deck to be replaced 2004 ** All bridges to be replaced in 2004

Poor (Group II)

- I-75 NB over SR-674 (Bridge #100347)





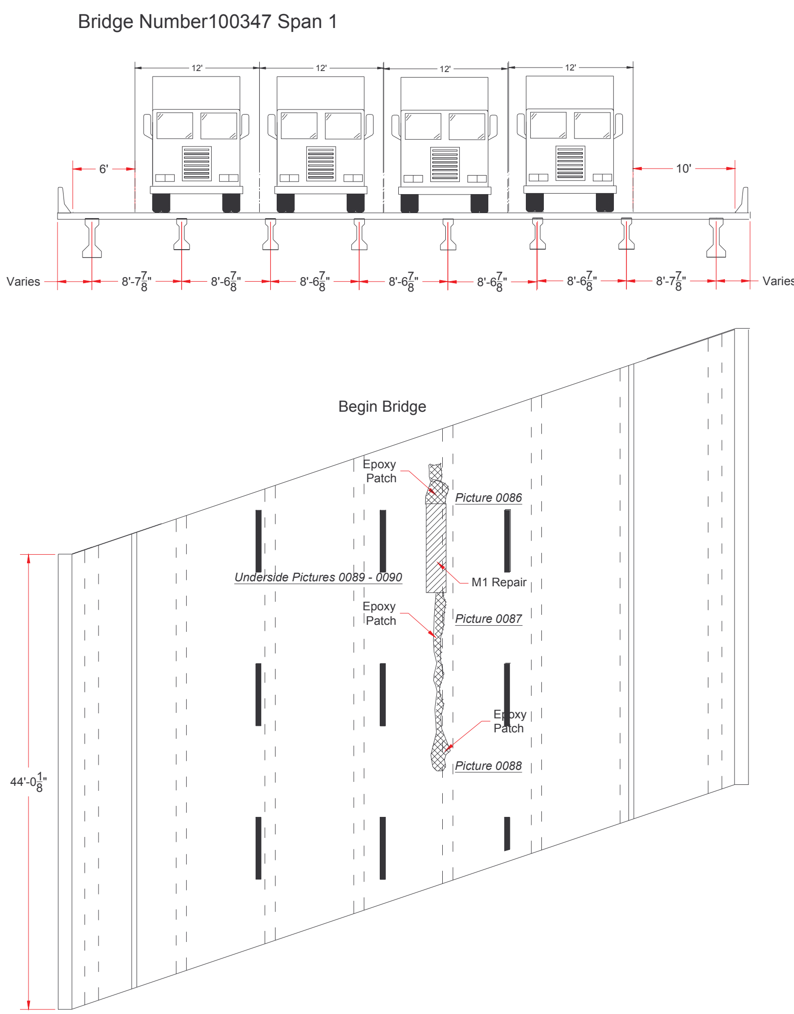


Figure 6.11 Deterioration in Bridge #100347 (March 2004)

- I-75 SB over Woodberry Road (Bridge #100468)

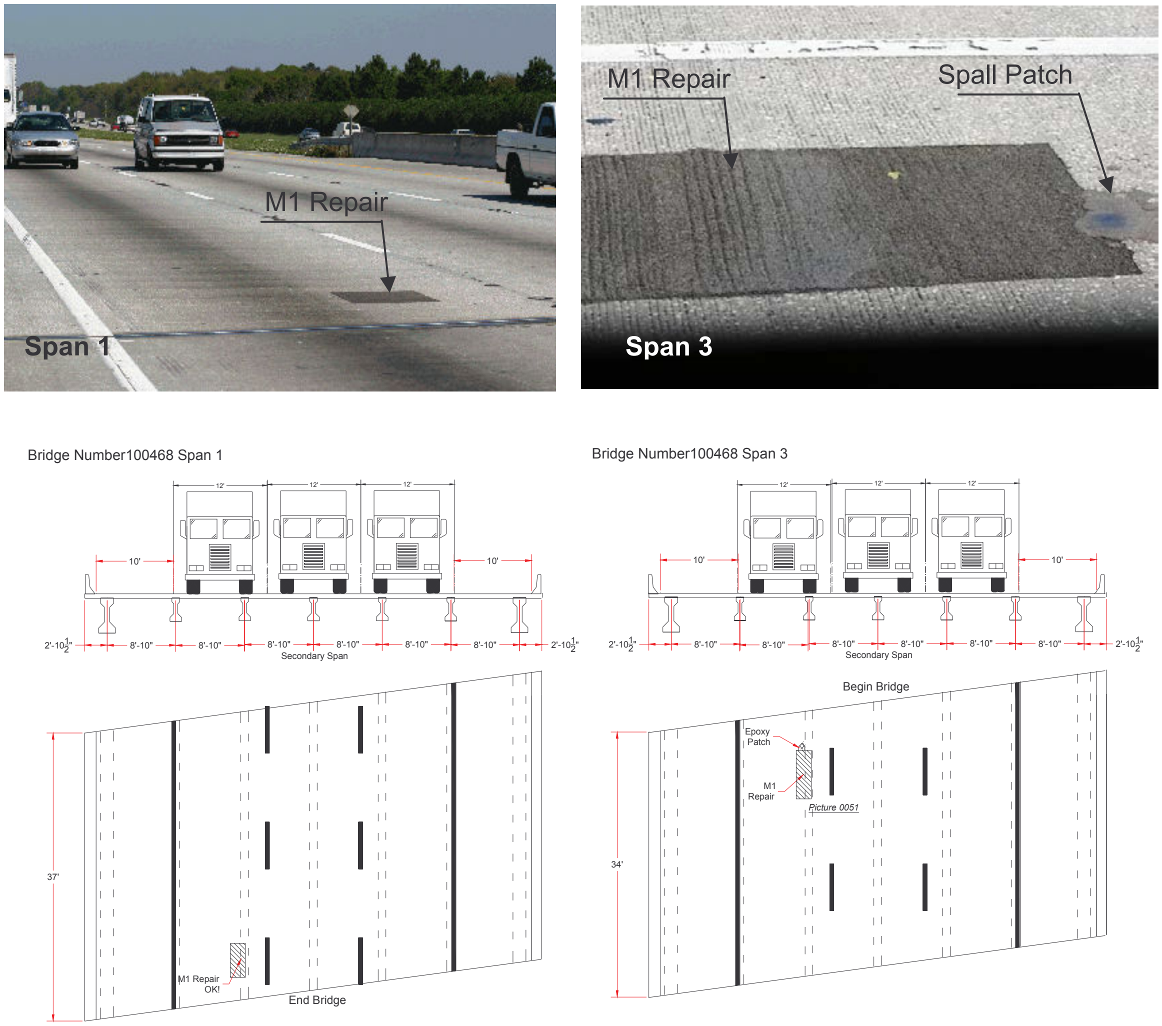


Figure 6.12 Deterioration in Bridge #100468 (March 2004)

- I-75 SB over Woodberry Road (Bridge #100470)

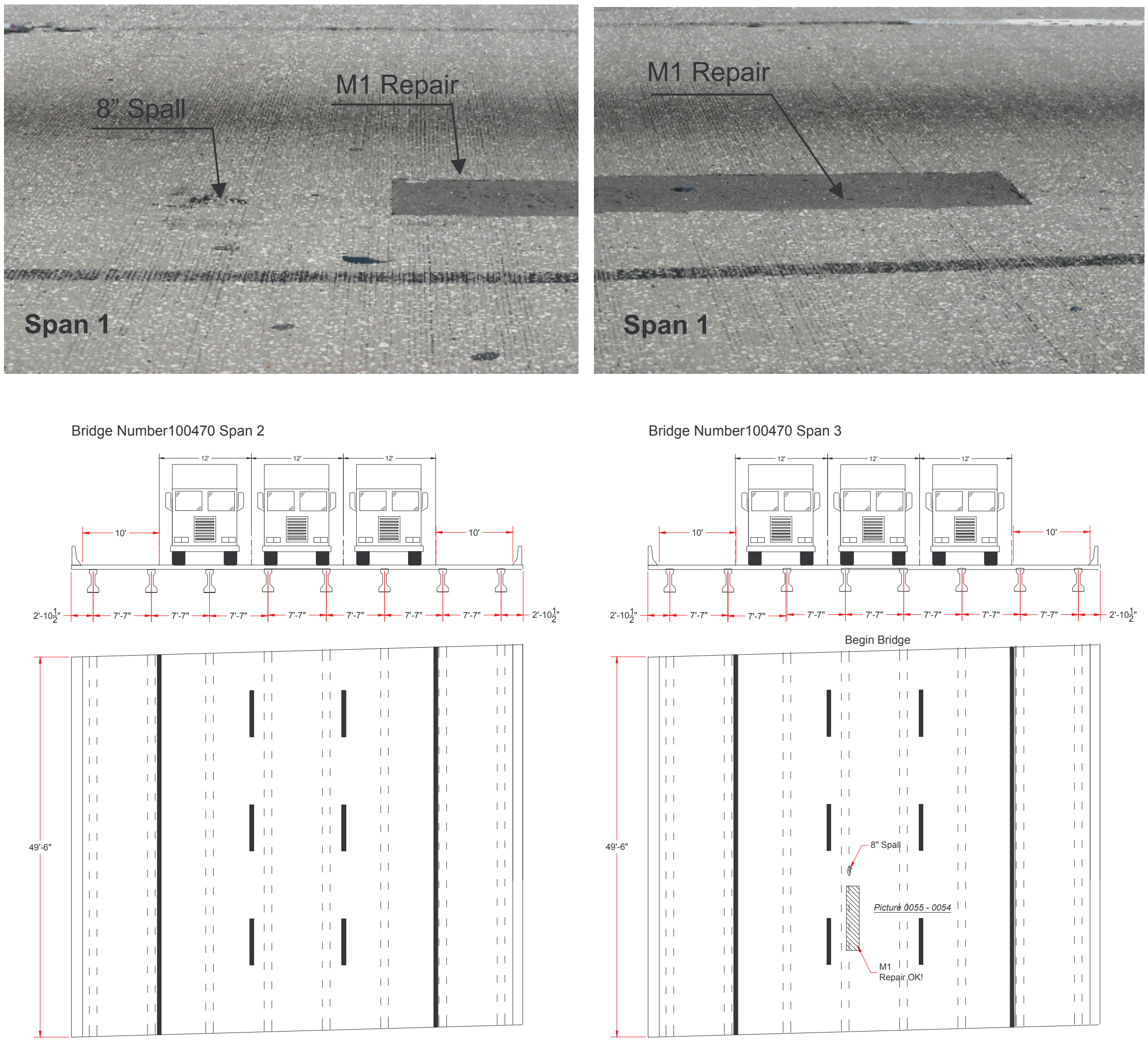


Figure 6.13 Deterioration in Bridge #100470 (March 2004)

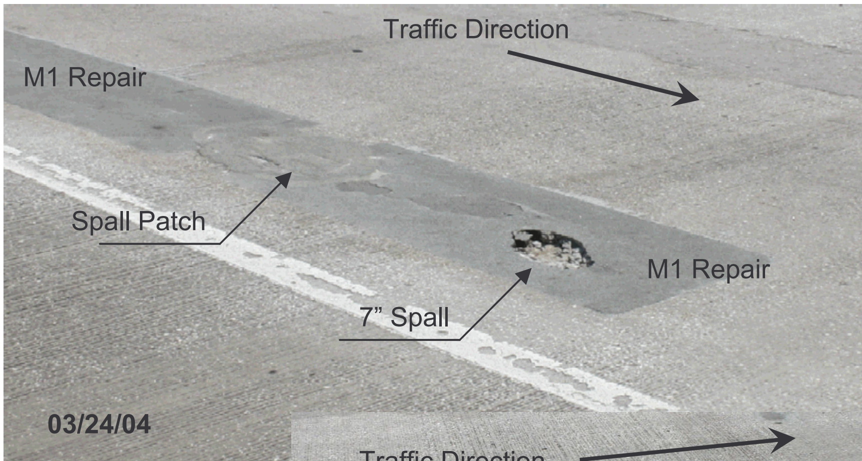
6.4 June Re-Inspection

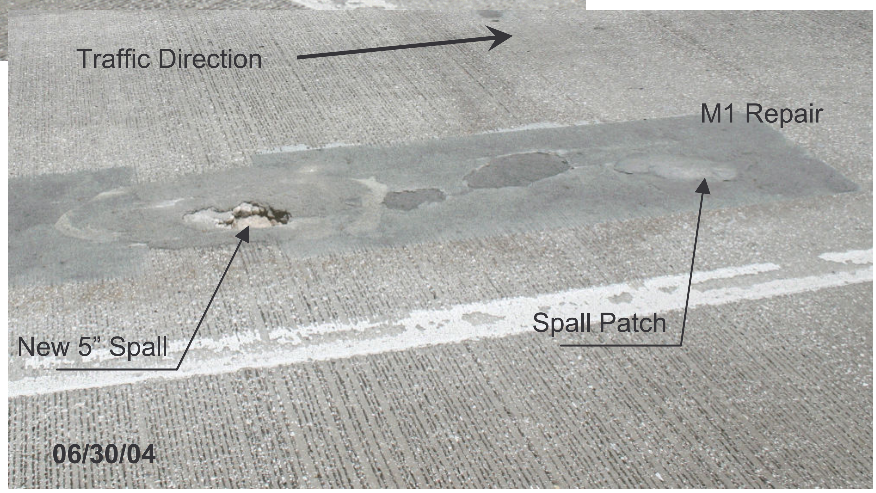
A total of 17 bridges – 10 in District 1 and 7 in District 7 – were classified in the first inspection as being in '*serious*' (1), '*poor*' (6) or '*fair*'(10) condition (see Tables 6.2-6.3). All these bridges were re-inspected at the end of June 2004. The aim was to identify progression of damage that could be used by PANEL to prioritize the replacement of these bridges.

Table 6.4 summarizes information on the change in the condition of these bridges over 3 months. A selection of representative photographs from 7 re-inspected bridges is shown in Figs. 6.14-6.20. These were taken from bridges classified originally as '*serious*' (1), '*poor*' (3) or '*fair*' (3). The selection was made to highlight examples of (1) new damage (2) increase in size of spalls (3) no deterioration (4) repair. Photographs and AUTOCAD drawings of all 15 re-inspected bridges are included in the PANEL database.

Table 6.4 Re-Inspection Summary

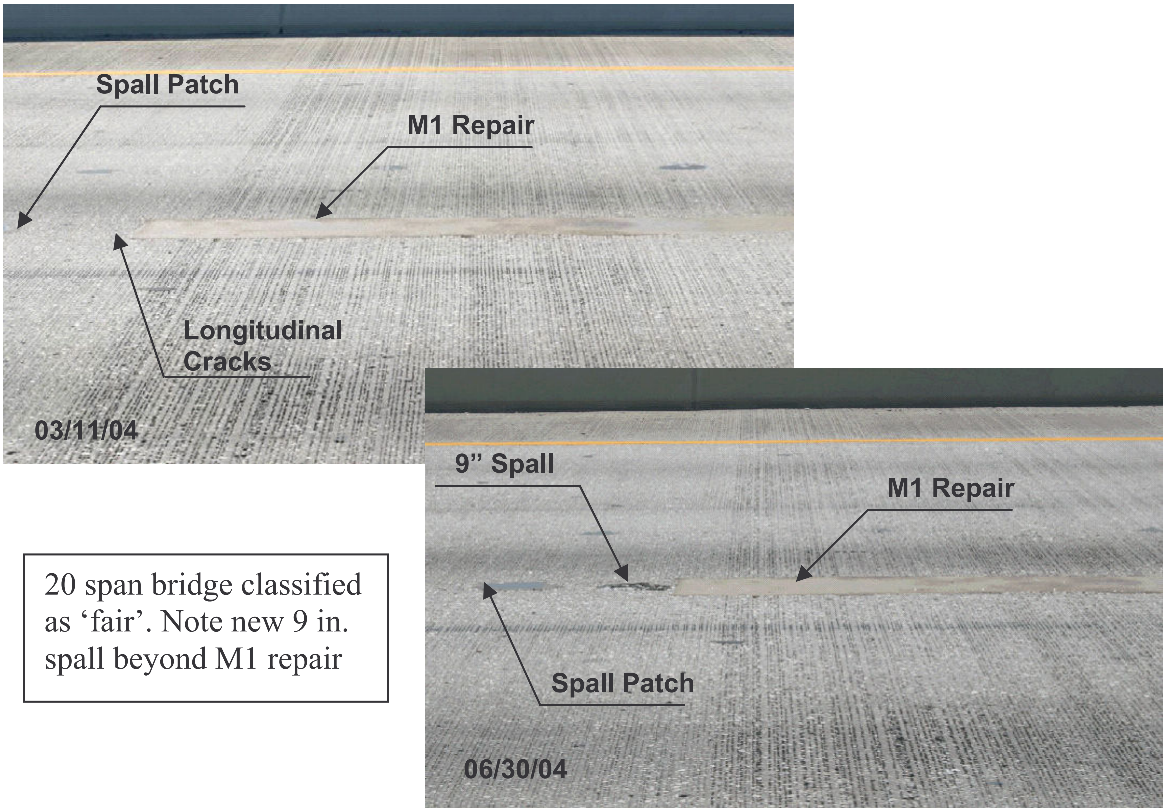
Group	Bridge ID#	Location	Change
Serious	130090	I-275 NB over I-75	4 New Spall Patches 1(5") New Spall No change in others.
Poor	170080	I-75 over Main A Canal	2 New Spall Patches 3 stable repairs
	170081	I-75 over Palmer Blvd.	(4") Spall increase (1.8") Spall increase 4 Stable repairs
	130112	I-275 SB TO I-75 NB Ramps	No change
	100347	I-75 NB over SR-674	No change
	100468	I-75 SB over Woodberry RD	2 New Spall patches in new spalls
	100470	I-75 SB over CSX RR	(2.5") Spall increase 1 Stable M1 repair
Fair	170089	I-75 over CSX RR	No Change
	170094	I-75 NB over Havana RD	No Change
	170099	SR-681 SB over CSX RR	No Change
	170100	SR-681 NB over CSX RR	No Change
	100346	I-75 SB over SR-674	No Change
	100358	I-75 SB over Alafia River	2 New Spall Patches
	100359	I-75 NB over Alafia River	New (9") Spall No Change in others
	150122	I-275 NB over 5^{th} Ave N	No change
	010064	Oil Well RD over I-75	No change
	030188	I-75 over CR-846	No change





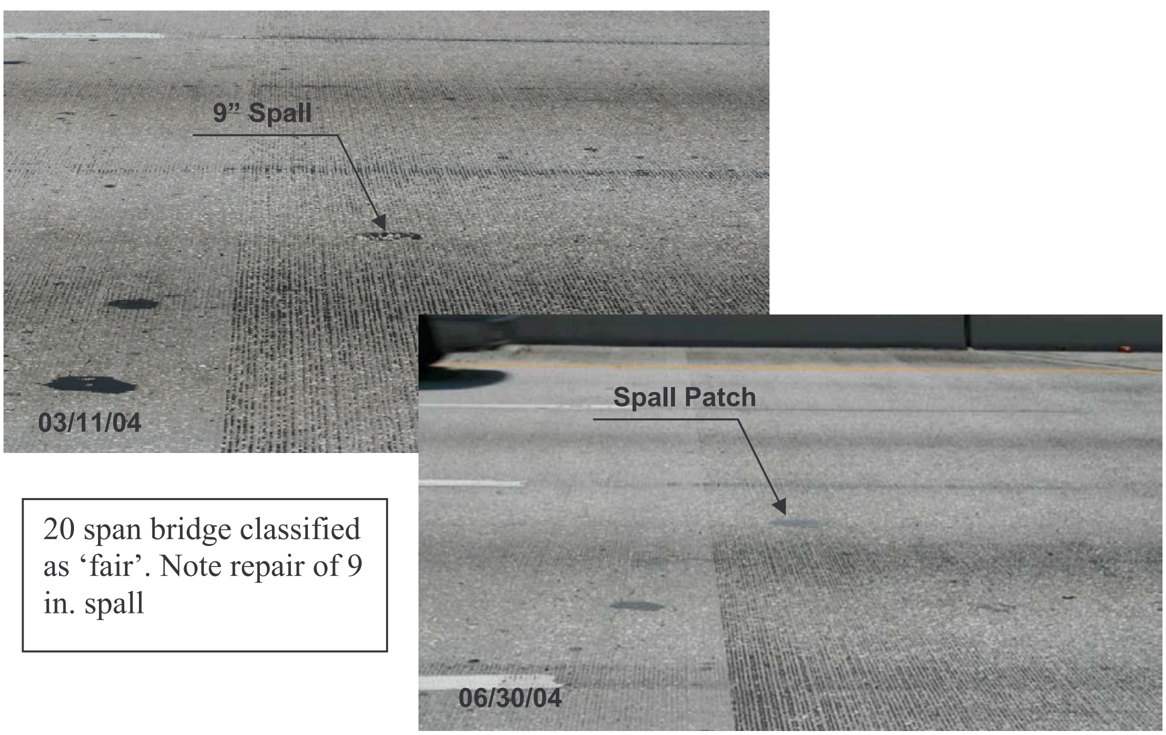
4-span bridge on steel beams classified as 'serious'. Note new spall and repaired patch

Figure 6.14 I-275 NB over I-75 Ramp River, Span 4 (Bridge #130090)



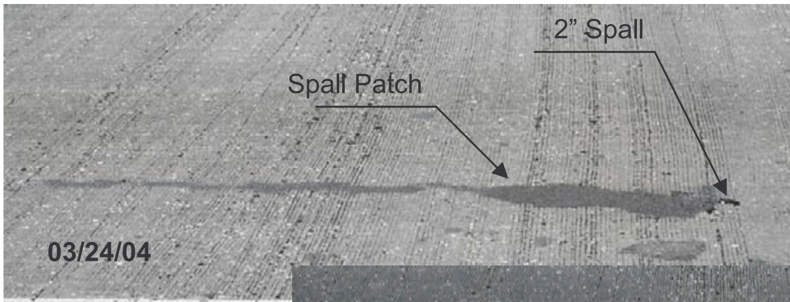
20 span bridge classified as ‘fair’. Note new 9 in. spall beyond M1 repair

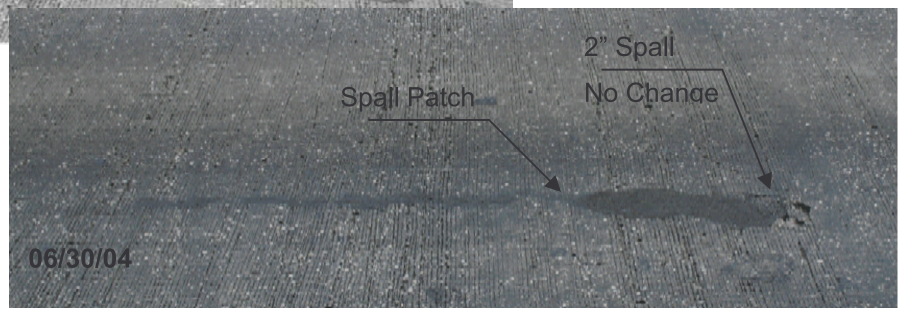
Figure 6.15 I-75 NB over Alafia River, Span # (Bridge 100359)



20 span bridge classified as ‘fair’. Note repair of 9 in. spall

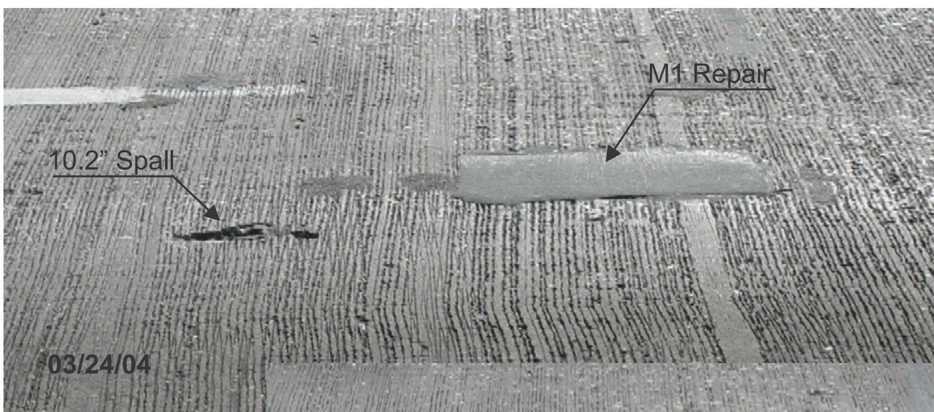
Figure 6.16 I-75 SB over Alafia River, Span # (Bridge 100358)

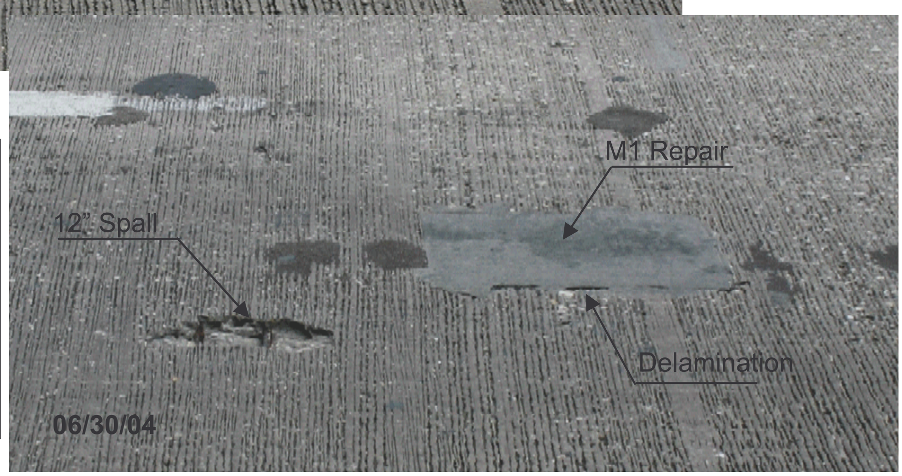




2-span bridge classified as ‘fair’. No change after 3 months

Figure 6.17 I-75 SB over River Road Span 2 (Bridge #170089)





3-span bridge classified as ‘poor’. Note increase in spall size and delamination of M1 repair

Figure 6.18 I-75 SB over Palmer Rd, Span 3 (Bridge #170081)

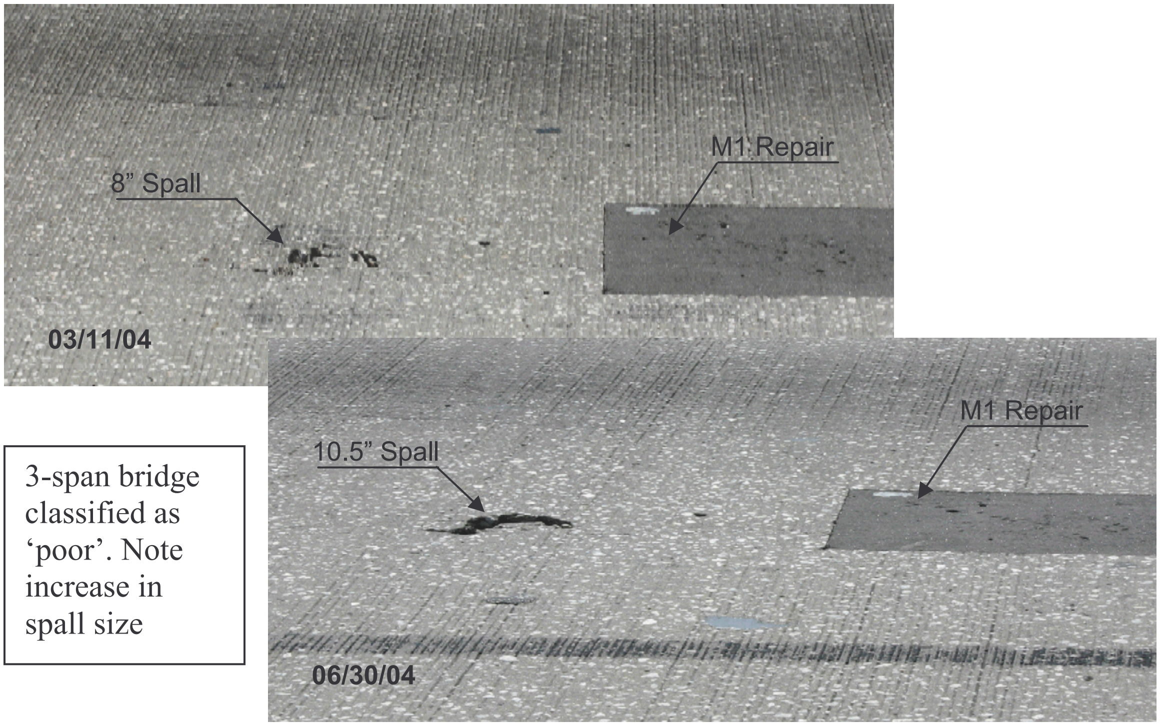


Figure 6.19 I-75 SB over Woodberry Road, Span 1 (Bridge #100470)

Figure 6.20 I-75 NB over Main Canal Span 2 (Bridge #170080)

6.5 Summary and Conclusions

A new inspection method was developed and used to inspect 101 deck panel bridges in Districts 1 and 7. In this method, dimensions of spalls are determined using a laser distancemeter mounted on a theodolite and the information mapped on plan view drawings of the bridge. A photographic record of the damage is noted. Based on the data compiled, bridges were classified ranging from '*serious*' to '*good*' (Table 6.1). Bridges identified as less than satisfactory were re-inspected after 3 months and their condition compared (Table 6.4). All data collected was stored for access by PANEL.

The new method is safe, efficient and reliable. The availability of PANEL makes it retrievable and makes it possible to take into account all available facts in deciding the prioritization of the deck panel bridges.

References

6.1 Leica DISTO™ classic[5] (2002). User Manual, version 1.2, Leica Geosystems, Heerbrugg, Switzerland.

7. PANEL SOFTWARE

7.1 Introduction

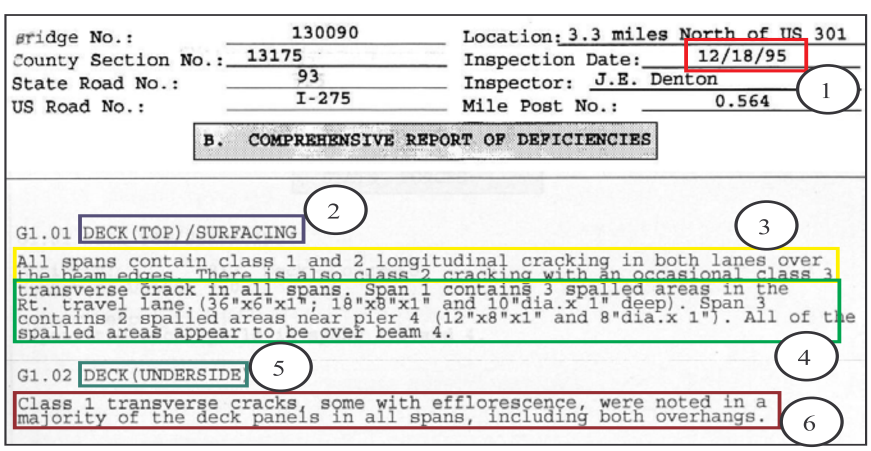
Previous chapter presents results of inspection conducted by the USF research team. The classification presented in the chapter is based on the data collected on the day of inspection. A more complete measure of bridge condition can be obtained through examination of data from historical inspection reports. This chapter presents a brief overview of the approach used to translate data from inspection reports into a format suitable for computer analysis in Section 7.2. This is followed by description of first version of the PANEL software in Section 7.3 followed by the latest version in Section 7.4. A summary of the chapter is presented in Section 7.5.

7.2 BRAILE

BRAILE is a computer data language developed to represent the data from the inspection report. It consists of commands that translate information from inspection reports to a computer language format. The data is entered into a text file, which is subsequently processed using PANEL. The language was developed by the research team at University of South Florida.

7.2.1 Translation of Information

Translation of information from the inspection report into a standard command format is presented here using an example. An old inspection report is shown in Figure


Bridge No.: 130090
County Section No.: 13175
State Road No.: 93
US Road No.: I-275
Location: 3.3 miles North of US 301
Inspection Date: 12/18/95
Inspector: J.E. Denton
Mile Post No.: 0.564

B. COMPREHENSIVE REPORT OF DEFICIENCIES

G1.01 DECK(TOP)/SURFACING

All spans contain class 1 and 2 longitudinal cracking in both lanes over the beam edges. There is also class 2 cracking with an occasional class 3 transverse crack in all spans. Span 1 contains 3 spalled areas in the Rt. travel lane.(36"x6"x1"; 18"x8"x1" and 10"dia.x 1" deep). Span 3 contains 2 spalled areas near pier 4 (12"x8"x1" and 8"dia.x 1"). All of the spalled areas appear to be over beam 4.

G1.02 DECK(UNDERSIDE)

Class 1 transverse cracks, some with efflorescence, were noted in a majority of the deck panels in all spans, including both overhangs.

Figure 7.1 Old Inspection Format

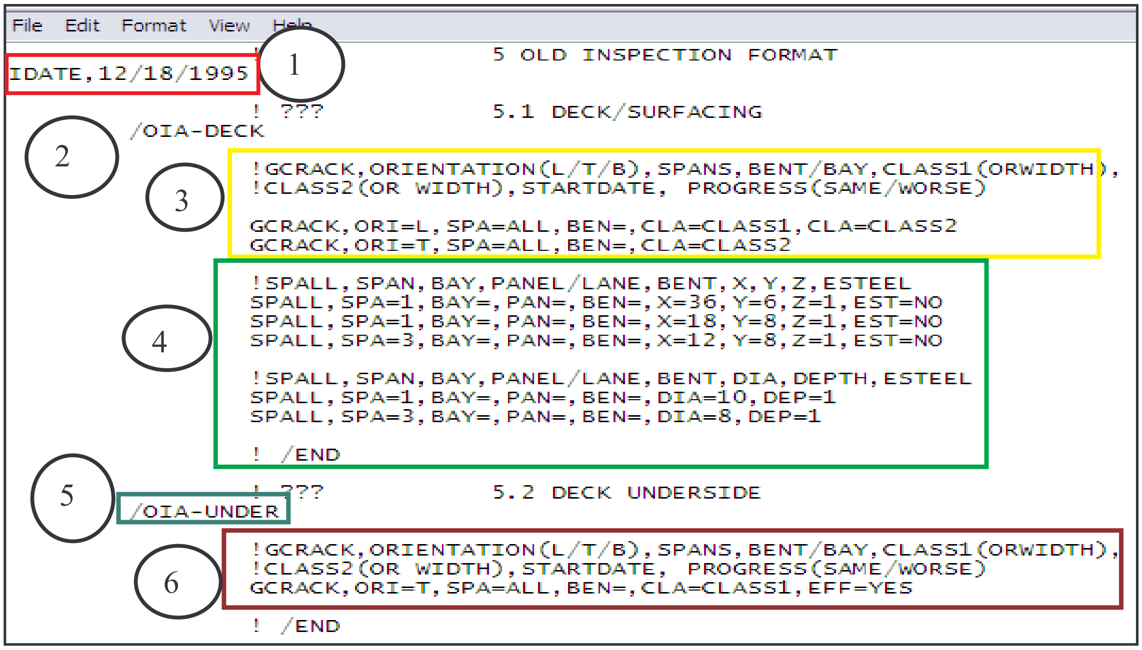


Figure 7.2 BRAILE File

7.1 for Bridge No. 130090. Only sections under the headings Deck (Top)/ Surfacing and Deck (Underside) are shown here. Information from regions 1 through 6 shown in the inspection file (see Figure 7.1) is translated to BRAILE commands in the corresponding marked regions (see Figure 7.2). Text following the exclamation point in the BRAILE file indicates comments, and is ignored during data processing. Figure 7.3 shows a new inspection report and the corresponding BRAILE file below it.

The BRAILE file for the new inspection format is shown below.

```
BRIDGE, 130090                                                          (Level 1)
        IDATE, 11/13/2001     !INSPECTION DATE                          (Level 2)

                /NI-DECKTOP  ! FOLLOWING COMMANDS REFER TO DECK TOP     (Level 3)
                        CONDITION, #=3                                  (Level 4)
                        !DELAM,SPAN,WIDTH,LENGTH,ESTEEL
                        DELAM,SPA=3,WID=67,LEN=19.6,EST=NO
                        !EREPAIR, TYPE, SPAN, LANE, X, Y, Z
                        EREPAIR,TYP=PATCH,SPA=1,LAN=2,X=11.8,Y=9.8
                        EREPAIR,TYP=PATCH,SPA=3,LAN=2,X=11.8,Y=7.8
                        EREPAIR,TYP=PATCH,SPA=4,LAN=2,X=7.8,Y=6
                        EREPAIR,TYP=PATCH,SPA=4,LAN=2,X=7.8,Y=6
                        EREPAIR,TYP=PATCH,SPA=4,LAN=2,X=7.8,Y=6
                        ! /END
                /NI-DECKUNDER  ! FOLLOWING COMMANDS REFER TO DECK BOTTOM
                        CONDITION,#=3
                        !GCRACK,ORIENTATION(L/T/B),SPANS,BENT/BAY,CLASS1(ORWID),
                        !CLASS2(OR WIDTH),STARTDATE, PROGRESS(SAME/WORSE)
                        GCRACK,ORI=T,SPA=ALL          ! MINOR CRACKS
```

BRIDGE ID: 130090 PAGE: 2 OF 8
DISTRICT: 01 Bartow INSPECTION DATE: 11/13/01 IFMK

UNIT:0 DECKS

ELEMENT/ENV:98/3 Conc Deck on PC Pane 2855 sq.m. ELEM CATEGORY: Decks/Slabs

CONDITION STATE (5)	DESCRIPTION	QUANTITY	RECOMMENDED FEASIBLE ACTION
3	Repaired areas and/or spalls/delaminations and/or cracks exist in the deck surface or underside. The combined area of distress is more than 2% but less than 10% of the total deck area	2855	1 Spalls & Delams

WORK ORDER RECOMMENDATION:

Rplc DEL patch spn 3 & asphalt patches spns 1 3 & 4. Grout panel BM joint under spalled areas. 10MH

ELEMENT INSPECTION NOTES:

NOTE: There is a sign mounted to the left face of the deck and beam 2-1.

CS3: The top middle support where the sign attaches to the deck is not mounted flush leaving a 10mm gap on the north end.
In the deck top, spans 1, 3, and 4 have several epoxy and grout patches in lane 2. In span 3, lane 2, there is a 1.7m x 500mm delaminated patch 14.5m from bent 3. This patch appears to be in good condition but when struck with a hammer, there is a distinct hollow sound. Refer to photo 1in addendum. P1 WO
The following areas have small asphalt patches:
Span 1, lane 2 at first construction joint from abut. 1, 300mm x 250mm;
Span 3, lane 2, 12.5m from bent 4, 300mm x 200mm;
Span 4, lane 2, midspan, 3 patches up to 200mm x 150mm. Refer to photo 2 in addendum.
In the deck underside, several of the deck panels have minor transverse cracks, some with efflorescence. In bay 4-2, the third panel from abutment 1 has a 200mm x 200mm x 10mm spall/delamination on the NW corner.
Bay 4-3, fourth panel from abutment 1, has a 400mm x 150mm x 20mm spall (no exposed steel) on the south end of the west edge.

ELEMENT/ENV:302/3 Compressn Joint Seal 26 m. ELEM CATEGORY: Joints

CONDITION STATE (3)	DESCRIPTION	QUANTITY	RECOMMENDED FEASIBLE ACTION
1	The element shows minimal deterioration. Adhesion is sound with no signs of leakage. There are no cohesion cracks. The adjacent deck and/or header is sound. If joint is armored, there are no signs of anchorage looseness.	13	0 Do Nothing

Figure 7.3 New Inspection Format

```
!SPALL,SPAN,BAY,PANEL/LANE,BENT,X,Y,Z,ESTEEL
SPALL,SPA=4,BAY=2,PAN=3,BEN=1,X=7.8,Y=7.8,Z=0.4,EST=NO
SPALL,SPA=4,BAY=3,PAN=4,BEN=1,X=15.7,Y=5.9,Z=0.8,EST=NO
!/END

/NI-CJS        ! FOLLOWING COMMANDS REFER TO POURABLE JOINT SEAL
        CONDITION,#=1
        !CJS,QTY,CONDITION (GOOD/POOR), RECOMENDATION(N=DO
        !NOTHING/CR=CLEAN J.& REPLACE)
        CJS,QTY=13,CON=GOOD,REC=N
        CONDITION,#=3
        CJS,QTY=13,CON=POOR,REC=CR
        ! /END
```

As seen from the above BRAILE file, inspection commands record the state of the bridge including the different deficiencies observed on the bridge deck. These commands indicate the type of deficiency and details such as their size, location and severity. Location is mostly reported in terms of span, bay and sometimes panel.

For example, the line
SPALL, SPA=4, BAY=2, PAN=3, BEN=1, X=7.8, Y=7.8, Z=0.4, EST=NO

indicates that there is spall in Span 4, Bay 2 and Panel 3 of size 7.8 in. x 7.8 in. x 0.4 in. and no exposed steel. Refer to online help on CD for the syntax and description of BRAILE commands.

It is worthy to note that most of the data entered have numerical values (such as the spall size) to assist in obtaining quantifiable results. Dimensions of all the parameters including spalls, delamination, patch etc. are recorded in inches.

Some important file conventions used for BRAILE are:

(1) Commands are case insensitive, i.e., they can be both upper and lower case.
(2) All command parameters are not mandatory. For example, its is not necessary to provide spall size if the information is not provided in the report.
(3) Command parameters do not have to be listed in any specific sequence. For example, the parameter SPA in the above example can be placed after EST=NO.
(4) As far as possible, the symbols used are intuitive. For example, SPA= Span, FREPAIR=failing repair etc.
(5) The value for each parameter is always preceded by the “=” sign.
(6) All the parameters are separated by commas.
(7) Comments are always preceded by exclamation mark “!”.

BRAILE data can be thought to be structured into five levels. These are described in Table 7.1. Levels indicate the association between the data. For example, the lowest level data (level 5) are command parameters (such as SPA), which are related to level 4 commands (such as SPALL). In the previous case, the level 5 command SPA indicates the span locating the deficiency noted in the level 4 command SPALL. The SPALL is in turn related to the level 3 command, such as /NI-DECKTOP, indicating that the spall occurs on the top of the deck. Level 3 commands are associated to level 2 command, which indicate the inspection date when the deficiency was recorded. Finally, level 2 commands are related to level 1 command, which indicates the bridge number that was inspected. The different levels represent progressive details of the inspection report, starting from the bridge number and inspection date and leading details of deficiencies on specific parts of the bridge.

7.3 PANEL VERSION 1.0

The information entered using BRAILE was processed using computer software called PANEL. It was developed using Visual Basic for Application feature of Microsoft

Excel. Broadly speaking, the software is similar to a database software since it basically retrieves desired information from data stored in BRAILE files.

Table 7.1 Comments on BRAILE Levels

Levels	Comments
1	Identifies the bridge number and remains same throughout the file. Written only once in a file
2	Identifies inspection date in the form mm/dd/yyyy. Several dates may be included in a single file.
3	Identifies the bridge component for which the deficiencies are recorded (e.g. deck top, deck bottom).
4	Identifies the deficiencies observed on the deck top or deck bottom (e.g. cracks, spalls, delaminations etc.).
5	Identifies details of the deficiencies, such as the span where it occurs or the size.

7.3.1 Outline of Data Processing Procedure

7.3.1.1 Selection of Data Files

Prior to retrieving any information from BRAILE files, the user must select the data files to search. In this study, the BRAILE files were divided into three groups, one each for District 1, District 7 and Crosstown bridges. Figure 7.4 shows the part of the software that enables the selection of the data set to process.

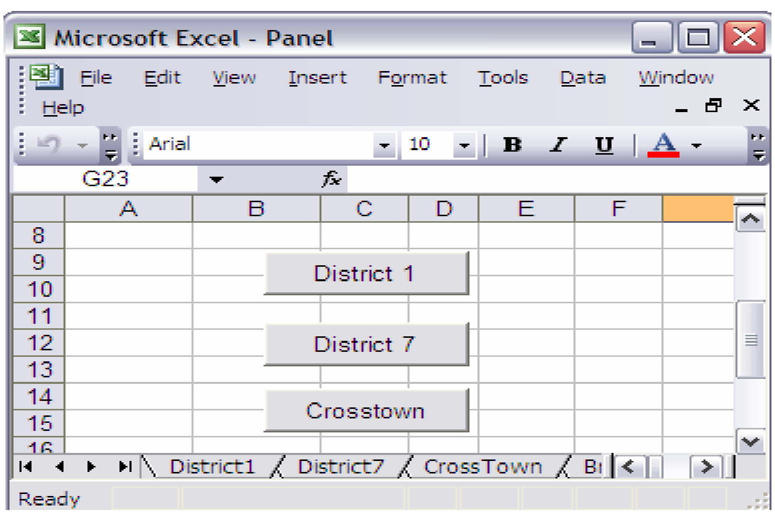


Figure 7.4 Selection of Data Set in PANEL

7.3.1.2 Updating Data

The spreadsheet has tabs with bridges from District 1, District 7 and Crosstown. The data from these can be updated by clicking the "Update Data" button (see Figure 7.5). This causes the program to display the "Search" tab (Figure 7.6) and conduct a series of searches to update the data in appropriate sheet as discussed below.

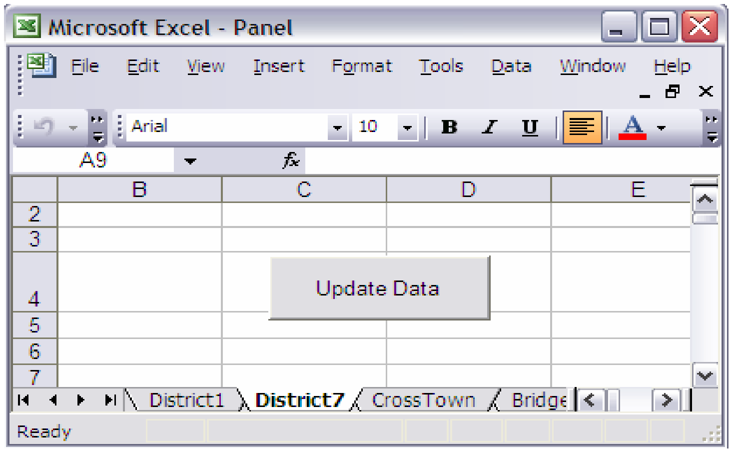


Figure 7.5 Spreadsheet for District 7

7.3.1.3 Data Processing

The "Search" tab enables the user to find desired information from stored BRAILE files. This is accomplished by specifying the different levels of commands to look for. For example, if one needs to determine the bridge numbers of from the files being processed, all that needs to be done is to search for data with level 1 command of "BRIDGE". This would cause PANEL to return the data lines containing the command BRIDGE along with the parameter specifying the bridge number. Other examples of search can be seen in Figure 7.6.

When instructed to "Update Data", the program performs a series of searches to return the number of deficiencies of different types, including failed repairs, existing repairs, spalls, bottom transverse cracks with efflorescence, bottom transverse cracks, bottom longitudinal cracks, top transverse cracks and top longitudinal cracks. The number of deficiencies obtained is for the entire bridge and not for each span. In addition, the number of inspections performed is also determined. This information is then recorded in the proper worksheet (District 1, District 7 or Crosstown) (see Figure 7.7).

The "Search" feature also has several other capabilities, such as for identifying all the bridges with a particular deficiency and obtaining deficiencies for a particular range of dates. Where applicable, such as in case of spall sizes, maximum, minimum and average values of deficiencies are also returned.

7.3.1.4 PANEL Output

As stated earlier, the data collected by the automated searches is stored in proper worksheet (see Figure 7.7). The data is stored in tabular format where rows correspond to the bridge number and the columns contain the number of specific deficiency.

7.4 PANEL 2.0

The original version of PANEL suffered from the drawback of being complicated to use since a very good knowledge of BRAILE is required to use advanced search features. Since it is essential that bridge information be constantly updated, a more user-friendly stand-alone version of the software was developed using Visual Basic. With the aim of improving the performance, data from BRAILE input files were added to a Microsoft Access database. This provided great improvement in speed and ease of use.

The basic features of PANEL 2.0 remain the same, i.e., the user can enter data about inspections, or process the data to obtain prioritization. The software implements the prioritization method described the next chapter. Figures 7.8 through 7.10 highlight input, prioritization and reporting features of the new version of the PANEL software. It also contains online help and features to customize the BRAILE language and the prioritization process. Complete details can be obtained from the online help included in the attached CD.

7.5 Summary

BRAILE command language along with PANEL provides an effective tool for assisting in replacement prioritization of panel bridges by providing quantitative data about these bridges from inspection reports. Although inspection reports data is not always current due to the two-year period between inspections, they still provide accurate and relevant information about these bridges that can be used to assess their true condition.

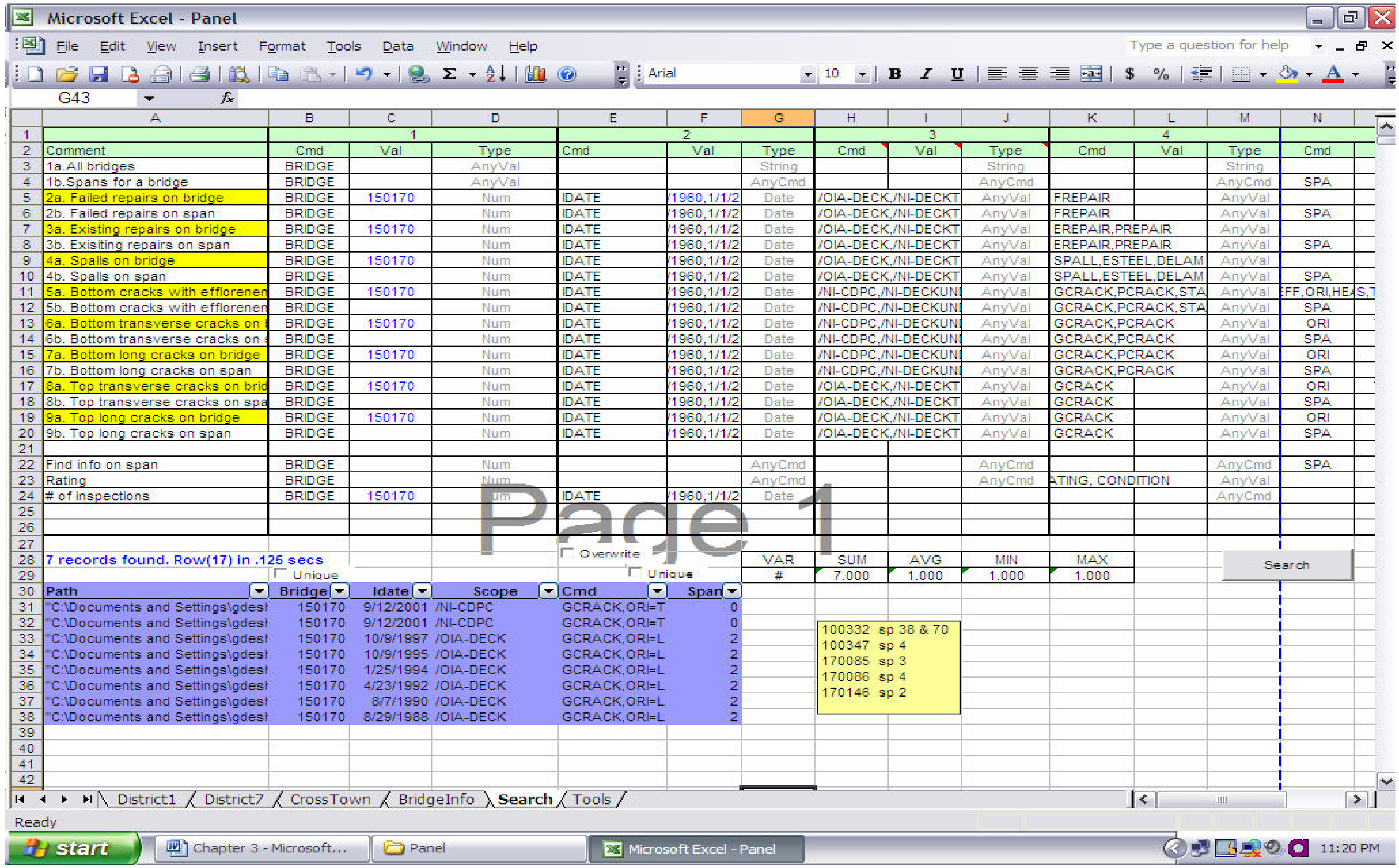


Figure 7.6 Data Processing Sheet

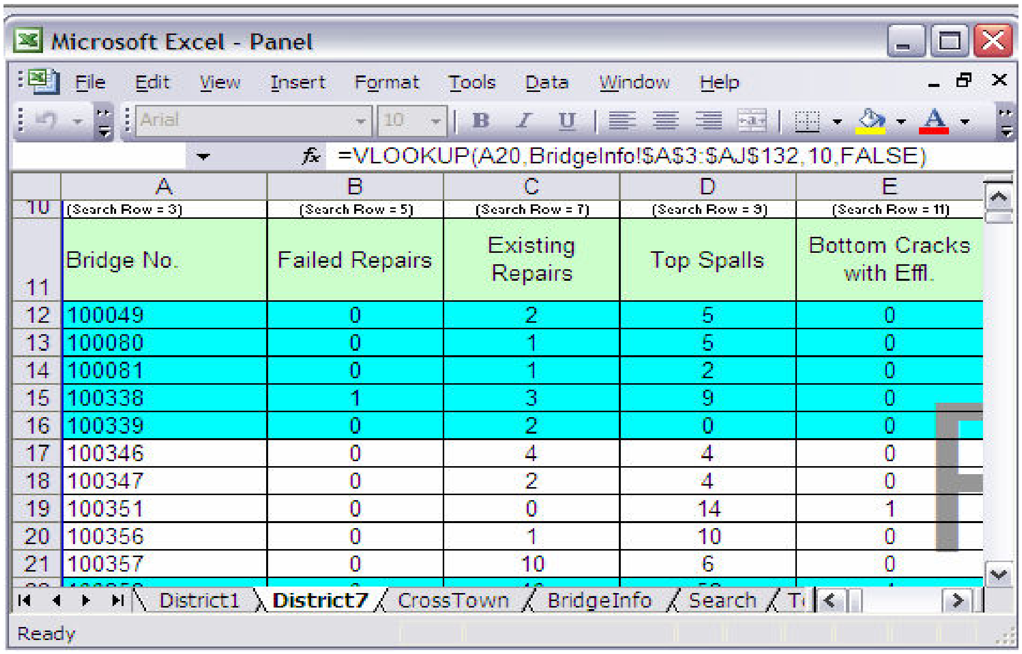


Figure 7.7 PANEL Output

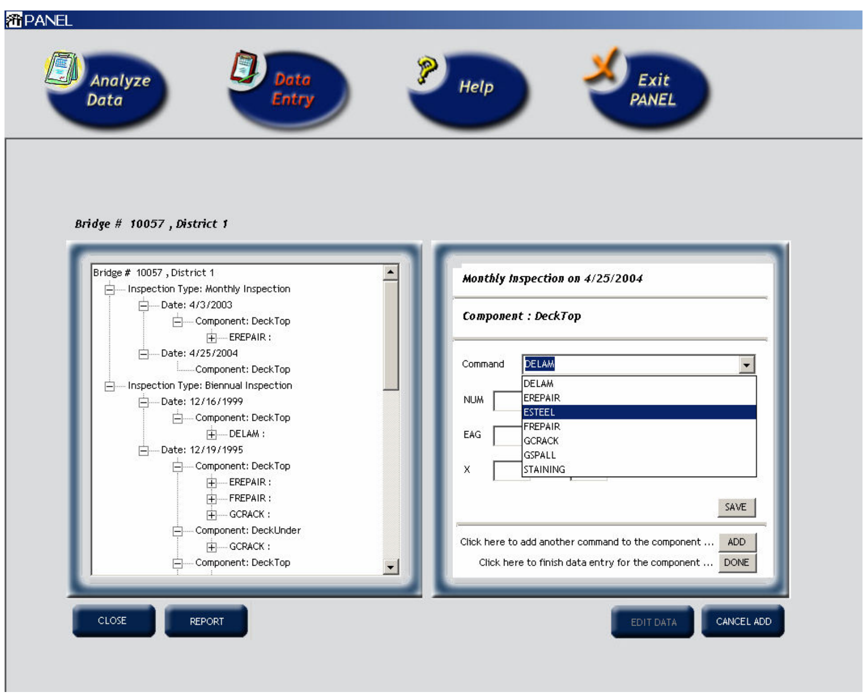


Figure 7.8 PANEL Data Input Screen

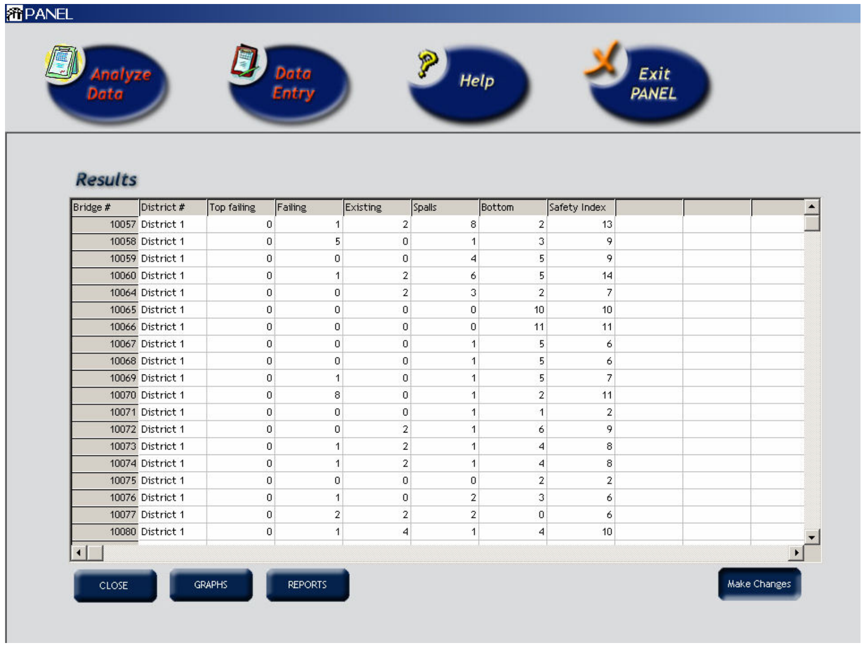


Figure 7.9 PANEL Output Screen

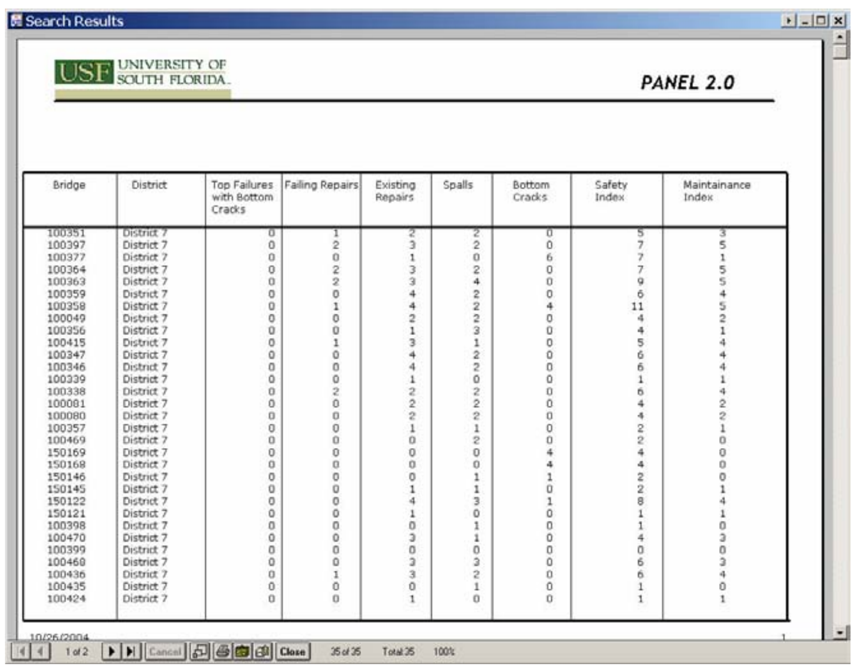


Figure 7.10 PANEL Sample Report

8. RESULTS

8.1 Introduction

This chapter contains results of prioritization performed using the PANEL software described in the previous chapter. The methodology used to assess the condition of deck using the inspection data is described in Section 8.2, followed by detailed results for District 1, District 7 and Crosstown in Sections 8.3 through 8.5 respectively. Section 8.6 presents a summary and conclusions.

8.2 Rationale

Many factors need to be considered when making engineering decisions. Of these the primary ones include safety, importance, and cost. These factors must be considered in the prioritization process used to identify bridges for replacement. Foremost is safety, i.e., bridges that are likely to fail must be replaced at the earliest. Second, it is also important to consider the importance of the bridge. Consequence of failure of bridges in evacuation routes can be serious, therefore they must be given high priority in replacement. Finally, it is important to consider economic issues, such as cost of frequent inspection and maintenance. Each of the three individual requirements of safety, importance and economics are discussed in more detail below.

8.2.1 Safety

As stated in Chapter 1, this project was undertaken as a response to several serious deck failures that resulted in deck panel bridges over the past 5 years. Fortunately, there were no reported injuries to the public, or property damage as a result of these failures. The primary objective of the project is to prevent similar failures by identifying and replacing the deck of high risk bridges.

Since there are a large number of bridges to be evaluated, a rational method must be used to identify high risk bridges based on the current understanding of the deterioration process. The detailed analysis of the five serious failures reported in Chapter 3 provides some clues on indicators of safety risks (see Section 3.7). The PANEL software was used to obtain data pertaining to these specific spans from all recorded inspection reports to identify indicators of poor bridge state. The data showed that all spans developed longitudinal cracks on the deck over girder lines within a few years of construction. A few transverse cracks were also reported. In one case (Bridge 100332, Span 70), there were reports of several spall on the deck underside (i.e., the panel) starting about 15 years after construction. There were also reports of a few cracks on the deck underside on span 38 of the same bridge around the same time. While, cracks are common to all spans that failed, their presence is not a very useful indicator of bridge condition, since it occurs in practically all deck panel bridges. The occurrence of underside deficiencies, such as cracks and spalls was observed in many of the bridges, and is an important indicator of the deck condition. However, it was not reported on all five cases. Of all the data available in inspection reports, the best indicator of poor deck condition was the record of spalling. All spans had a history of spalling with different degree of severity (see Table 8.1). Also, based on the model of progressive damage presented in Chapter 4, it is clear the spalls represent advanced stage of deterioration and thereby indicate serious problems.

Table 8.1 shows the historical spalling and repair data obtained using PANEL from biennial inspection for the specific span of bridges that had serious deck failures. While the number of spalls over time is by no means alarming except bridge 100332, span 70, it must be pointed that the more detailed analysis of these failure presented in Chapter 3 indicated many more repairs that are not recorded in the biennial inspection records. As a result, the data here is not useful to gage the absolute condition of the bridge, but it is useful as a comparative measure since most bridge inspections display the same lack of detail due to the two year interval between inspections.

Since spalling is the most consistent indicator of poor state of the deck, the number of spalls that occur in a bridge is used to measure the likelihood of failure of bridge deck. Since most inspections provide a snap-shot of the bridge at the time of inspection, they do not always contain records of previously recorded spalls that have been repaired. As a result, to get a complete idea of the historical performance of the bridge, a cumulative spall count is used. With the aim of giving more importance to recently recorded data, a time adjusted spall count, S_{ta}, is computed as follows:

$$S_{ta} = S_i \times \frac{T_i}{T_{last}} \tag{8.1}$$

where

S_{ta}	=	time adjusted spall count for a inspection date
S_i	=	number of spall reported in the inspection report
T_i	=	time (in days) between bridge opening date and inspection date
T_{last}	=	time (in days) between the bridge opening date and date of last inspection

In addition to the number of spalls recorded in an inspection report, the number of existing repairs is also an indicator of the spalling history of the bridge. In some cases, these may be repaired spalls that have been included as spalls in earlier reports. However, in many cases the repairs indicate repaired spalls that had developed between the previous inspection and the current inspection. In this case, the number of existing repairs must be included in the spall count to get a true estimate of the amount of

Table 8.1 Spalling and Repair History for Bridges with Localized Failures

Bridge	Span	Date	Spalls	Existing Repairs	Failing Repairs
170146	2	09/1984	-	-	-
		12/1985	-	-	-
		07/1986	-	-	-
		07/1988	-	-	-
		05/1990	-	-	-
		05/1992	1	-	-
		02/1994	1	-	-
		12/1995	1	-	-
		11/1997	2	-	1
		11/1999	-	-	-
170086	4	05/1985	-	-	-
		01/1987	-	-	-
		01/1989	-	-	-
		10/1990	-	-	-
		01/1993	-	-	-
		08/1994	-	-	-
		06/1996	-	-	-
		05/1998	-	1	-
		05/2000	1	-	-

Bridge	Span	Date	Spalls	Existing Repairs	Failing Repairs
170085	3	05/1985	-	-	-
		04/1987	-	-	-
		01/1989	-	-	-
		10/1990	-	-	-
		01/1993	-	-	-
		08/1994	-	-	-
		06/1996	3	-	-
		05/1998	-	-	-
		05/2000	-	-	-
100332	38	02/1985	1	-	-
		03/1987	-	-	-
		04/1989	-	-	-
		05/1991	1	-	-
		05/1993	1	-	-
		05/1995	2	-	-
		08/1997	-	-	-
		08/1999	-	-	-
		08/2001	-	-	-
100332	70	02/1985	-	-	-
		03/1987	1	-	-
		04/1989	-	-	-
		05/1991	1	-	-
		05/1993	10	-	-
		05/1995	9	-	-
		08/1997	-	-	-
		08/1999	-	-	-
		08/2001	-	-	-

spalling. Since it is not clear which of the two cases truly occurs, the cumulative spall count is computed as follows:

$$S_C = \sum S_{ta} + \max(E_{ta}) \tag{8.2}$$

$$E_{ta} = E_i \times \frac{T_i}{T_{last}} \tag{8.3}$$

where

S_C	=	cumulative time adjusted spall count
E_{ta}	=	time adjusted existing repair count for a inspection date, computed as follows
E_i	=	number of existing repairs reported in the inspection report

In addition to the spalls and number of repairs, another useful indicator of likelihood of failure is the number of failing repairs. Based on the progressive damage model presented in Chapter 5, it is known that failing repairs are indicators of a seriously weakened state of the concrete surrounding the repair due to poor bearing condition and damage from spalls and cracks. As a result failing repairs are given a higher weight when quantifying the safety risk associated with a bridge. Just as with the spall count, a time adjusted value is computed to give more importance to more recent data, and a cumulative measure is determined as follows.

$$F_{ta} = F_i \times \frac{T_i}{T_{last}} \tag{8.4}$$

$$F_C = \sum F_{ta} \tag{8.5}$$

where

F_{ta}	=	time adjusted failing repair count for a inspection date
F_i	=	number of failing repairs reported in the inspection report
F_C	=	cumulative time adjusted failing repair count

The final failure indicator of a bridge is obtained by combining the cumulative spall count and the cumulative failing repair count as follows.

$$C = S_C + W \times F_C \tag{8.6}$$

where

C	=	combined indicator of likelihood of failure
W	=	weight factor used to give more importance to failing repairs

The above equation gives a combined measure of the likelihood of failure from the cumulative spall count and cumulative failing repair count. Since it is known from the progressive damage model presented in Chapter 4 that failing repairs are indicators of serious deficiencies, they are given more importance than the number of spalls.

Various values of weight factor, W, were used and the ranking based on resulting combined indicator of failure, C, was compared with results of USF inspections reported in Chapter 6 for the top 25 bridges in District 1. The relative order of bridge classification based on C, and USF inspections is shown in Table 8.2. Clearly, a sign of good correlation between the two methods is that the bridges classified as "Serious", "Poor" or "Fair" appear higher in the list. This was found to occur for W = 15 or higher. As a result a value of W=15 was used to obtain the combined indicator of likelihood of failure.

Table 8.2 Effect of Weight Factor, W on Relative Ranking

W = 0	W = 1	W = 2	W = 3	W = 5	W =10	W =15	W =20
Poor	Poor	Poor	Poor	Poor	Serious	Serious	Serious
Serious	Serious	Serious	Serious	Serious	Poor	Poor	Poor
Fair	Fair	Fair	Fair	Acceptable	Acceptable	Acceptable	Acceptable
Fair	Poor	Poor	Poor	Fair	Poor	Poor	Poor
Poor	Fair	Fair	Acceptable	Poor	Fair	Poor	Poor
Fair	Fair	Acceptable	Fair	Poor	Poor	Fair	Fair
Acceptable	Poor	Fair	Poor	Fair	Fair	Fair	Fair
Poor	Acceptable	Poor	Fair	Fair	Fair	Fair	Fair
Fair	Acceptable	Acceptable	Acceptable	Acceptable	Acceptable	Acceptable	Acceptable
Good	Fair	Fair	Fair	Fair	Fair	Fair	Fair
Good	Good	Good	Good	Good	Good	Good	Good
Acceptable	Good	Good	Good	Good	Good	Good	Good
Good	Acceptable	Acceptable	Acceptable	Acceptable	Acceptable	Acceptable	Acceptable
Acceptable	Good	Good	Good	Good	Good	Good	Good
Fair	Fair	Fair	Fair	Fair	Fair	Fair	Fair
Good	Good	Good	Good	Good	Good	Good	Good
Good	Good	Good	Good	Good	Good	Good	Good
Fair	Fair	Fair	Fair	Fair	Fair	Fair	Fair
Good	Good	Good	Good	Good	Good	Good	Good
Good	Good	Good	Good	Good	Good	Good	Good
Good	Good	Good	Good	Good	Good	Good	Good
Good	Good	Good	Good	Good	Good	Good	Good
Good	Good	Good	Good	Good	Good	Good	Good
Good	Good	Good	Good	Good	Good	Good	Good
Good	Good	Good	Good	Good	Good	Good	Good

The above methodologies for determining the combined spall count and failing repair count, C, is a simple approach of quantifying the likelihood of failure. All the above quantities can be readily obtained using the PANEL software described in Chapter 7. One possible limitation in the approach is that it is influenced by the number of inspection reports used to obtain these values. A bridge with more inspection reports is likely to have a higher value of C. Since some of the older inspection reports were not available for a few bridges, this can potentially skew the results to make these bridges seem better than they may actually be. However, since the most importance is given to new inspection reports through the time adjusted spall and failing repair count, the above factor has limited impact on the results.

Another implication of this method is that bridges with larger number of spans are likely to have larger values of C, and thus higher likelihood of failure. That is, if two bridges have the same average value of C per span, the bridge with a larger number of spans represents a larger likelihood of failure. This is a reasonable conclusion when the objective is to replace entire bridges.

8.2.2 Importance

In addition to consideration for safety, it is essential to consider the importance of the bridge. A common measure of risk of a structure is obtained as a product of the likelihood of failure and the cost of consequences of failure [1]. As a result, although many bridges may have modest likelihood of failure, the consequences of failure may be more costly, therefore overall risk may be much higher. For example, there are deck panel bridges with low average daily traffic (ADT) that have significant cracking and spalling. Compared to such bridges, bridges with high ADT (such those on interstates), with lower amount of damage represent a higher overall risk due to the consequence of disrupting large number of vehicles on such bridges.

In the current study the importance of the bridge is assessed using the ADT reported in the National Bridge Inventory database (see Chapter 2). Bridges on important interstate systems, such as I-75 have high ADT's and therefore are given more importance.

8.2.3 Economic Issues

In addition to safety and importance, other issues, such as economics of replacement must be considered when obtaining the final prioritization list. Amongst other things, one of the items that contribute to the cost is the maintenance cost associated with frequent inspections. This however need not be considered separately, since the spall count used to measure the likelihood of failure gives a good measure of the required maintenance cost.

Some consideration for reducing the replacement cost are factors such as grouping the deck replacements with widening, and lowering the construction costs by replacing bridges that are located close to each other (such as twin bridges, northbound and southbound bridges over I-75 etc.). In the results presented, these factors are not explicitly considered, since information on widening plans etc. is not known at this time. However, the PANEL software includes features to account for such factors.

8.3 District 1 Results

Table 8.3 presents the cumulative spall count, S_C, and F_C along with other data for 78 deck panel bridges from District 1. The bridges are presented in the order of rank based on safety, i.e., bridges that are most likely to fail are listed first. The top four bridges in the list have either been replaced or scheduled for replacement by FDOT. This means that the currently utilized rationale used by FDOT for prioritizing replacement identified the worst bridges. The next two bridges in the list have been placed on low priority by FDOT. However, based on the methodology used here, these bridges are identified as having a higher likelihood failure. This is same conclusion arrive at in Chapter 6 through latest inspections performed by USF.

A look at the list reveals that 36 of the 74 bridges in the list have already be either replaced or have been scheduled for replacement by FDOT. For the most part, many of these bridges occur in the top of the list in Table 8.3, indicating these were bridges in poor condition. However, many of the bridges that have been identified as being in good condition by both the USF inspections and the methodology used here have been ranked as high priority for replacement by FDOT. These bridges have no history of significant spalling and therefore must not be made a high priority unless dictated by economic considerations, such as widening or grouping with other bridges in the proximity.

Replacement ranking based on importance is also presented in Table 8.3. As stated earlier, this is based on the ADT reported for the bridges in the National Bridge Inventory database. It can be seen that many of the bridges that are rated high based on importance are rated low on the safety rank. For example the top five important bridges 170083, 170084, 170081, 170080 and 170145 based have safety rankings of 57, 62, 23, 25 and 49. Of these bridges, 170145 has already been replaced. Most of the bridges that have been replaced or scheduled for replacement fall in the mid level of importance. Bridges with lowest ADT's in either good or fair condition have not been scheduled for replacement.

Ranking of the bridges based on risk, which is the product of the ADT and C normalized with respect to the maximum value in the table is also presented. This provides a means to identifying bridges that must be replaced to minimize the cost to the public associated with a failure. Of the top 35 bridges ranked based on risk, 30 have already been replaced or scheduled for replacement by FDOT. This indicates that the methodology used by FDOT is quite sound. The bridges that have not been replaced yet are 130012, 170081, 130090 and 030187. Of these the first four were classified as poor or serious based on the inspection results presented in Chapter 6.

Table 8.3 District 1 Prioritization

Bridge #	Year Built	T.A.C. Spalls [1]	T.A.C. Failing Repair [1]	Combined Failure Indicator [2]	USF Inspection Condition	FDOT Rank [3]	ADT	Normalized Risk [4]	Safety Rank	Importance Rank	Risk Rank
010058	1980	31.3	17.8	298.6	N/A	Replaced	22000	1.000	1	28	1
010057	1980	90.8	11.6	264.3	N/A	Replaced	22500	0.905	2	27	2
010081	1981	39.8	8.5	167.4	N/A	Replaced	22000	0.561	3	28	4
130075	1981	43.9	6.7	144.9	N/A	11 (2004)	36500	0.805	4	8	3
130090	1981	30.0	6.8	132.1	Serious	Low	8000	0.161	5	63	16
130112	1981	76.3	1.9	104.4	Poor	Low	26500	0.421	6	19	5
170130	1981	10.4	5.8	97.3	N/A	High (Replaced)	15500	0.230	7	59	8
170088	1979	17.7	4.0	77.6	N/A	1 (2004)	18500	0.218	8	45	9
130078	1981	3.3	4.7	74.2	N/A	10 (Replaced)	27000	0.305	9	17	6
120093	1978	15.3	3.8	72.2	N/A	High- (replaced)	24500	0.269	10	23	7
010076	1981	40.5	1.9	69.5	N/A	5 (2004)	18000	0.190	11	47	12
010073	1981	3.1	4.2	66.6	N/A	0	18000	0.182	12	47	13
010077	1981	9.5	3.7	64.8	N/A	5 (2004)	19500	0.192	13	38	11
010069	1981	2.7	4.0	62.2	N/A	0	17000	0.161	14	52	15
010080	1981	7.7	3.6	62.2	N/A	Replaced	22000	0.208	15	28	10
030187	1980	3.8	3.8	60.1	Acceptable	17	16500	0.151	16	53	17
170132	1981	29.4	1.0	44.0	N/A	4 (Replaced)	15500	0.104	17	59	23
010070	1981	1.1	2.6	39.5	N/A	0	18000	0.108	18	47	22
010082	1979	8.6	1.7	34.7	N/A	0	16500	0.087	19	53	24
120094	1978	0.9	1.9	29.6	N/A	High- (replaced)	27000	0.122	20	17	20
170129	1981	14.5	1.0	29.5	N/A	High (Replaced)	16000	0.072	21	55	29
170087	1979	10.0	1.2	28.2	N/A	1 (2004)	19500	0.084	22	38	26

[1] T.A.C. (Time Adjusted Cumulative values) are adjusted for the time of inspection to give more importance to new inspection data
[2] Failure Indicator Index = T.A.C Spalls + 15 × T.A.C. Failing Repairs
[3] ( ) represent status or year of scheduled replacement
[4] Normalized Risk =(ADT × Failure Indicator Index) / (Maximum Value from all bridges in the District)

Table 8.3 District 1 Prioritization (continued)

Bridge #	Year Built	T.A.C. Spalls	T.A.C. Failing Repair	Combined Failure Indicator	USF Inspection Condition	FDOT Rank	ADT	Normalized Risk	Safety Rank	Importance Rank	Risk Rank
170081	1979	12.6	1.0	27.5	Poor	7	40000	0.167	23	3	14
010060	1980	10.3	1.0	25.2	N/A	High (Replaced)	22000	0.084	24	28	25
170080	1979	8.4	1.0	23.3	Poor	3	40000	0.142	25	3	18
170146	1981	6.3	1.1	22.2	N/A	0	39000	0.132	26	6	19
130076	1981	6.8	1.0	21.4	N/A	11 (2004)	33500	0.109	27	12	21
170131	1981	6.3	1.0	20.8	N/A	4 (Replaced)	16000	0.051	28	55	31
170128	1979	5.0	1.0	19.5	N/A	6 (2004)	19500	0.058	29	38	30
030188	1980	18.3	0.0	18.3	Fair	17	29000	0.081	30	15	27
130079	1981	6.6	0.8	18.0	N/A	10 (Replaced)	26500	0.073	31	19	28
010074	1981	3.5	0.8	15.8	N/A	0	16000	0.038	32	55	33
170139	1981	6.0	0.6	15.1	N/A	0	18000	0.041	33	47	32
170099	1981	12.8	0.0	12.8	Fair	0	4950	0.010	34	66	43
170094	1979	10.4	0.0	10.4	Fair	9	23000	0.036	35	25	34
170096	1979	8.6	0.0	8.6	Acceptable	2	21000	0.028	36	33	35
010072	1980	6.0	0.0	6.0	N/A	High (Replaced)	21000	0.019	37	33	39
010064	1980	4.7	0.0	4.7	Fair	Low	720	0.001	38	73	59
010067	1981	4.4	0.0	4.4	Good	0	8400	0.006	39	61	50
010068	1981	4.4	0.0	4.4	Good	0	7700	0.005	40	65	52
170085	1980	4.2	0.0	4.2	N/A	0	36000	0.023	41	10	36
130084	1981	3.9	0.0	3.9	N/A	16 (2004)	26500	0.016	42	19	40
170079	1979	3.9	0.0	3.9	Acceptable	3	36500	0.022	43	8	37
010059	1980	3.9	0.0	3.9	Good	High (Replaced)	22000	0.013	44	28	42
170086	1980	3.7	0.0	3.7	N/A	0	36000	0.020	45	10	38
010083	1979	3.5	0.0	3.5	N/A	0	16000	0.009	46	55	44
170089	1979	2.9	0.0	2.9	Fair	13	18000	0.008	47	47	45
170127	1979	2.8	0.0	2.8	N/A	6 (2004)	18500	0.008	48	45	46

Table 8.3 District 1 Prioritization (continued)

Bridge #	Year Built	T.A.C. Spalls	T.A.C. Failing Repair	Combined Failure Indicator	USF Inspection Condition	FDOT Rank	ADT	Normalized Risk	Safety Rank	Importance Rank	Risk Rank
170145	1981	2.3	0.0	2.3	N/A	0	40000	0.014	49	3	41
120085	1976	2.0	0.0	2.0	Good	0	19500	0.006	50	38	49
130085	1981	2.0	0.0	2.0	Good	16 (2004)	25000	0.008	51	22	47
170100	1981	2.0	0.0	2.0	Fair	0	4950	0.002	52	66	58
130089	1981	1.8	0.0	1.8	Good	Low	500	0.000	53	74	62
010071	1980	1.6	0.0	1.6	N/A	High (Replaced)	21000	0.005	54	33	51
120088	1980	1.0	0.0	1.0	Good	0	24500	0.004	55	23	54
130107	1981	1.0	0.0	1.0	Good	Low	1500	0.000	56	68	60
170083	1979	0.9	0.0	0.9	Good	12	44750	0.006	57	1	48
120114	1979	0.8	0.0	0.8	Good	Low	1440	0.000	58	69	61
170090	1979	0.8	0.0	0.8	Good	13	19000	0.002	59	43	57
170095	1979	0.8	0.0	0.8	Good	2	28000	0.003	60	16	55
170092	1979	0.8	0.0	0.8	Good	14	23000	0.003	61	25	56
170084	1979	0.7	0.0	0.7	Good	12	44750	0.005	62	1	53
010065	1981	0.0	0.0	0.0	Good	Low	1200	0.000	63	71	63
010066	1981	0.0	0.0	0.0	Good	Low	1200	0.000	64	71	63
010075	1981	0.0	0.0	0.0	Good	Low	1250	0.000	65	70	63
010090	1981	0.0	0.0	0.0	Good	0	8400	0.000	66	61	63
010091	1981	0.0	0.0	0.0	Good	0	7900	0.000	67	64	63
120086	1976	0.0	0.0	0.0	Good	0	19500	0.000	68	38	63
120126	1979	0.0	0.0	0.0	Good	8	29500	0.000	69	13	63
120127	1979	0.0	0.0	0.0	Good	8	29500	0.000	70	13	63
170082	1979	0.0	0.0	0.0	Good	7	39000	0.000	71	6	63
170091	1979	0.0	0.0	0.0	Good	14	21000	0.000	72	33	63
170093	1979	0.0	0.0	0.0	Good	9	21000	0.000	73	33	63
170140	1981	0.0	0.0	0.0	N/A	15 (Replaced)	19000	0.000	74	43	63

8.4 District 7 Results

Table 8.4 presents the prioritization results for 35 bridges from District 7. As with District 1, the bridges are presented in the order of rank based on safety. The top four bridges in the list have been rated as either “Serious” or “Poor” based on inspections presented in Chapter 6. Of the four, Bridges 100416 and 100397 have also been ranked high by FDOT (rank 3 and 4). However, the top two bridges based on current methodology have been ranked 17 and 21 by FDOT.

Comparing the FDOT ranking presented in Table 8.4 with the safety based rank, it can be seen that only the two previously mentioned bridges fall in the group. Other bridges in the list, 100377, 100351 and 100398 have been ranked 31, 8 and 25 based on safety. These bridges have also been ranked as either “Good” or “Acceptable” in Chapter 6. It can be seen that unlike District 1, there is a significant difference between the ranking used by FDOT and one obtained from the current study.

Looking at the ranking based on importance, it is seen that the top 5 bridges based on importance are ranked 11, 5, 3,6 and 9 respectively by FDOT. These have a safety rank of 35, 25, 3, 13, and 30 respectively. With the exception of Bridge 100351, which has a safety rank of 8, the last 10 bridge rankings based on importance have rankings between 23 and 34, and have been classified as “Good” in Chapter 6. Of the bottom ten bridges based on importance, FDOT has ranked bridges 100377 and 100351 as 1 and 2 for replacement.

Ranking of the bridges based on risk, is somewhat similar to the safety based ranking. In fact the top five rankings based on risk also appear in the top five rankings based on safety, although in a different order. As a result comparison between FDOT ranking and safety based ranking presented earlier also applies to risk based ranking. The bottom ten in the list, have all been rated “Good” in Chapter 6. The top rated bridges based on FDOT ranking in the bottom ten list include Bridge 100377 (ranked 1 by FDOT) and 100435 (ranked 9 by FDOT).

Overall comparison of the different rankings suggests that many of the bridges currently ranked high by FDOT may be changed to lower priority.

Table 8.4 District 7 Prioritization

Bridge #	Year Built	Spall Count	Failing Repair Count	Weighted Index	USF Inspection Condition	FDOT Rank	ADT	Normalized Risk	Safety Rank	Importance Rank	Risk Rank
100363	1981	37.4	6	127	Serious	17	26000	0.639	1	18	3
100364	1981	12.9	7	118	Poor	21	31000	0.705	2	14	4
100416	1983	5.4	7	110	Serious	3	46824	1.000	3	3	1
100397	1984	10.9	5	84	Serious	4	43000	0.700	4	8	2
100358	1981	46.8	2	75	Fair	20	32500	0.474	5	12	5
100415	1983	7.0	4	67	Serious	8	43000	0.557	6	8	6
100338	1976	11.4	3	49	Acceptable	34	13000	0.123	7	25	9
100351	1981	8.5	2	37	Acceptable	2	2200	0.016	8	35	24
100436	1983	5.5	1	17	Acceptable	7	44500	0.142	9	6	8
100081	1970	12.7	0	13	Acceptable	32	17000	0.042	10	24	20
100356	1981	12.2	0	12	Acceptable	18	32500	0.077	11	12	14
100357	1981	10.6	0	11	Acceptable	19	31000	0.064	12	14	10
100468	1983	10.1	0	10	Poor	6	45500	0.089	13	4	7
100080	1970	10.1	0	10	Acceptable	31	26349	0.051	14	17	17
100346	1982	7.0	0	7	Fair	13	26000	0.035	15	18	16
150122	1980	6.8	0	7	Fair	23	33500	0.044	16	11	15
100359	1981	6.6	0	7	Fair	22	31000	0.040	17	14	13
100347	1982	6.5	0	7	Poor	14	24500	0.031	18	20	18
100469	1983	4.6	0	5	Good	10	44500	0.039	19	6	11

Table 8.4 District 7 Prioritization (continued)

Bridge #	Year Built	Spall Count	Failing Repair Count	Weighted Index	USF Inspection Condition	FDOT Rank	ADT	Normailized Risk	Safety Rank	Importance Rank	Risk Rank
100049	1968	4.4	0	4	Acceptable	30	21500	0.018	20	23	23
100471	1983	4.3	0	4	Good	15	24500	0.020	21	20	18
100470	1983	2.0	0	2	Poor	12	24500	0.009	22	20	21
150145	1979	1.9	0	2	Good	25	6100	0.002	23	32	28
150146	1979	1.7	0	2	Good	26	6600	0.002	24	29	32
100398	1984	1.3	0	1	Good	5	47500	0.012	25	2	12
150121	1980	1.0	0	1	Good	24	40500	0.007	26	10	22
100399	1979	0.9	0	1	Good	35	7300	0.001	27	28	30
100339	1978	0.9	0	1	Good	33	11900	0.002	28	26	29
150168	1979	0.9	0	1	Good	27	6450	0.001	29	30	25
100435	1983	0.9	0	1	Good	9	45500	0.008	30	4	27
100377	1983	0.9	0	1	Good	1	4500	0.001	31	34	31
100424	1983	0.8	0	1	Good	16	7500	0.001	32	27	33
150170	1979	0.8	0	1	Good	29	5010	0.001	33	33	34
150169	1979	0.8	0	1	Good	28	6450	0.001	34	30	26
100417	1983	0.0	0	0	Good	11	48290	0.000	35	1	35

8.5 Crosstown

Table 8.5 presents the prioritization results for 18 bridges from Crosstown. As done previously, the bridges are presented in the order of rank based on safety. The top five bridges in the list were ranked 1, 5, 2,17 and 7 by FDOT. This indicates reasonable agreement for bridges in poor condition.

Comparing the FDOT ranking presented in Table 8.5 with the safety based rank, it can be seen that for the most part they are ranked quite high too (1,3,13, 8 and 2). While top five items on the list show fairly good agreement, the bottom five contain two bridges (100451 and 100453) that are ranked 6 and 4 respectively, based on safety.

Looking at the ranking based on importance, it is seen that the top 5 bridge rankings based on importance are 1, 5, 4, 15 and 13 respectively by FDOT. These have a safety ranking of 1, 2, 8, 10, and 15 respectively. The bottom five bridges in the list contain bridges ranked 16, 3, 6, 8 and 17 by FDOT and 6, 13, 7, 9 and 4 based on safety. It is interesting to note that unlike bridges in District 1 and 7, here many bridges with low ADT are in a poor state. This suggests factors, such as poor construction may be responsible for the poor condition.

Ranking of the bridges based on risk is very similar to the safety based ranking in Crosstown. This is primarily because the risk is measured here as a product of the ADT and safety index, and since the ADT has a smaller variation, the risk ends up representing the effect of safety. As a result, the discussion presented above for comparison between FDOT ranking and safety based ranking also applies.

Table 8.5 Crosstown Prioritization

Bridge #	Year Built	Spall Count	Failing Repair Count	Weighted Index	FDOT Rank	ADT	Importance Rank	Normailized Risk	Risk Rank	Safety Rank
100332	1975	344.7	44.9	1018.7	1	23000	1	1.000	1	1
100333	1975	197.5	14.2	411.2	5	21000	2	0.369	2	2
100443	1981	53.8	6.7	153.9	2	16000	11	0.105	3	3
100453	1981	41.0	4.9	114.4	17	13530	18	0.066	4	4
100448	1981	2.8	3.5	54.9	7	16500	10	0.039	5	5
100451	1981	14.3	2.3	48.1	16	14330	14	0.029	6	6
100457	1981	9.7	2.0	39.6	6	14300	15	0.024	8	7
100447	1981	5.4	1.8	32.9	4	20800	3	0.029	7	8
100456	1981	17.7	0.0	17.7	8	14300	15	0.011	11	9
100449	1981	2.6	0.8	14.9	15	20760	4	0.013	9	10
100444	1981	11.8	0.0	11.8	12	16000	11	0.008	12	11
100454	1981	5.1	0.0	5.1	18	17290	8	0.004	10	12
100455	1981	5.0	0.0	5.0	3	14300	15	0.003	13	13
100446	1981	4.2	0.0	4.2	11	15800	13	0.003	16	14
100450	1981	3.2	0.0	3.2	13	18600	5	0.003	14	15
100458	1981	2.6	0.0	2.6	10	18300	7	0.002	17	16
100452	1981	2.0	0.0	2.0	7	18375	6	0.002	15	17
100445	1981	1.7	0.0	1.7	14	17000	9	0.001	18	18

8.6 Summary and Conclusions

The primary objective of the project was to develop a methodology for prioritizing replacement of deck panel bridges. A procedure for prioritization based on consideration of safety, importance, economic consideration and risk was presented in this chapter. The prioritized bridge lists for District 1, District 7 and Crosstown were presented with bridge ranking based on safety, importance and risk.

The three bridges from District 1 that had localized failure (170146, 170085, 17086) have been ranked 26, 41 and 45 based on safety out of the 74 bridges. The other bridge that had localized failures is ranked 1 based on safety amongst 18 bridges in Crosstown. While this might seem to indicate a lack of perfect correlation between the two, the main reason for the discrepancy is the lack of data as indicated by the analysis presented in Chapter 3. To rectify this problem, data for monthly inspection reports and USF inspections (Chapter 6) have also been used to arrive at the numbers. As a result, it is critical that the data used to obtain priorities in the future rely on frequent inspections.

Comparison of the proposed prioritization with current FDOT prioritization for District 1 revealed a generally good agreement between the two. Also, the agreement between the recommendations presented in Chapter 6 and those developed here were found to be quite good, although to some degree this was by design as discussed in Section 8.2.1. Prioritizations for District 7 and Crosstown vary considerably from the preliminary FDOT rankings.

9. SUMMARY AND CONCLUSIONS

9.1 Summary

Precast deck panel bridges have a long history of poor performance in Florida. A spate of recent localized failures on major highways, led to a decision by the Department to replace selected bridges on I-75 in Districts 1 and 7 by full-depth, cast-in-place concrete slab over 10 years. The goal of this study was to develop a strategy that could assist in prioritizing this replacement.

A progressive degradation model was developed from a careful review of localized failures, on-site forensic investigation of deck panel bridges being replaced, historical inspection data and finite element analysis. This model was subsequently integrated in PANEL - custom software written for this project. A special database containing inspection records for all precast deck panel bridges in Districts 1 and 7 extending over 20 years in electronic form was created for PANEL. This allowed PANEL to automate the prioritization process.

PANEL permits users to specify weighting factors for parameters such as safety, importance and cost. This information is then used to create lists that rank the order in which the replacement is to be carried out. Weighting factors used in this report for ranking were calibrated using the latest inspection data. As the database can be easily updated to include new inspection information and photographs, PANEL provides a dynamic resource that can be used by FDOT to review and revise its prioritization strategy in the future to take into consideration the latest available information.

9.2 Review of Findings

The following are some of the more important findings from the study. Additional conclusions are also provided at the end of each chapter.

9.3 Localized Failure

9.3.1 Inspection

Five localized failures occurring during 2000-2002 were carefully analyzed (Chapter 3). Biennial inspection data was generally unable to predict failure. All failed bridges were rated between 5 (satisfactory) and 7(good) according to the National Bridge Inventory Rating. Monthly inspection records that tracked progression of damage were more successful in anticipating failure.

9.3.2 Failure Location

All failures occurred under wheel loads applied close to the face of the girders where initial longitudinal cracks had developed. In all cases failure occurred in the right lane, i.e. slow lane. Failure was generally located at an edge or corner panel whose boundaries had developed reflective cracking (Table 9.1).

Table 9.1 Failure Summary

Bridge #	Year Built	Age at Failure (yrs)	Condition Rating *	ADT (%ADTT)	Failure Size	Location in Panel	Comment
170146	1981	19	6 (Satisfactory)	34,000 (30%)	18 in x 24 in	Edge or Corner?	Failure at M1 repair
170086	1980	20	7 (Good)	34,000 (30%)	36 in x 60 in	Corner Support	Patch repair
170085	1980	20	7 (Good)	34,000 (30%)	18 in x 18 in	Corner	Failure adjacent to M1 repair
100332	1980	22	5 (Fair)	23,000 (8%)	48 in x 30 in	Near corner	Asphalt Patch
100332	1980	23	5 (Fair)	23,000 (8%)	24 in x 36 in	Edge	Failed M1 repair with flexible patch material

** National Bridge Inventory condition rating given in the bridge inspection prior to the deck failure*

9.3.3 Cause of Failure

Simplified calculations showed that punching failures could occur at loads below the design wheel load (Appendix A) in situations where the cast-in-place concrete provided no resistance due to cracking and the panel was supported on fiberboard. The failure load for this case was calculated to be around 15 kips (Table 3.4). Otherwise, failure loads were nearly four times higher.

9.3.4 Environmental Factors

In four out of the five failures there was rainfall prior to failure (Table 3.15). The most severe rainfall preceded the last failure (1.1 in.). Rain may have contributed towards accelerating deck deterioration due to pumping action induced by the wheels that may have forced existing cracks to widen and reduce shear capacity.

9.3.5 *Bridge Characteristics*

All failures occurred in bridges where the deck was nominally 7 in. thick. No failures occurred in deck panel bridges with thicker slabs. The ADTT varied between 8-30% (Table 9.1).

Also all failures occurred in bridges in the same geographical region, (NB, SB - 170086, 170085 and in an adjacent bridge 170146 in Sarasota) or the same bridge (100332 spans 38 and 70 in Tampa). It appears likely that the failed bridges were built with similar defects by the same contractor.

9.4 Forensic Investigation

Eight on-site investigations were carried out on deck panel bridges before and during their demolition. The decks ranged from those that were in poor condition to those in relatively good condition with minimal visible cracking (Chapter 4).

9.4.1 *Panel Bearing Types*

Contrary to expectations, three out of the eight bridges investigated were found to be built with the panels supported by concrete or grout (Table 4.9). These bridges were in good condition except for one bridge (#100415) where spalling occurred. It was found that this was due to construction deficiency (the panel was not long enough to allow grout to flow underneath and was only supported by fiberboard as a result).

9.4.2 *Panel/CIP Separation*

In all the bridges inspected (see for example Figs. 4.7, 4.8, 4.10, 4.11), separation was observed at the vertical panel/CIP slab interface. This cannot be detected during inspection but it was predicted from finite element analysis (Chapter 5) as due to long term creep and shrinkage effects.

9.4.3 *Longitudinal Cracking*

This type of cracking is most common and occurs along the girder edges. It is initiated because of the vertical separation at the panel/CIP slab interface (Section 9.4.2). This cracking did not depend on live loads (it was found on hard shoulders) or the type of bearing (fiberboard or grout).

9.4.4 *Transverse Cracking*

This type of cracking was less common than the longitudinal cracking. This crack emanates at the transverse panel joint and progresses to the deck surface (Fig. 4.9). This is a reflective crack that does not affect shear capacity. It was not related to the type of bearing (fiberboard or grout) support.

9.4.5 Additional Longitudinal Cracking

The development of an *additional* longitudinal crack parallel to the initial crack described in Section 9.4.3 is the first sign of future deck deterioration, e.g. Figs. 4.10, 4.11, 4.16. This type of cracking *only* occurs when the panel is supported on (1) fiberboard bearing and (2) the panel boundary coincides with wheel lines.

9.4.6 Cause of Failure

The forensic study confirmed findings from previous studies in that lack of positive panel bearing support was responsible for poor performance. Lack of positive bearing may be due to:

1. Initial construction using fiberboard to support the panel.
2. Construction deficiencies, e.g. the precast panel had insufficient length (Fig. 4.25-4, 4.40) leaving no space for the grout to flow below the panel.

Not all the deck panel bridges in FDOT's Districts 1 and 7 were built using only fiberboard supports (Section 9.4.1). Three bridges examined constructed after 1983 had grout support for the panels similar to construction used in Texas (see Appendix B).

Three factors were common in all deteriorated decks:

1. Lack of stiff support for the deck panels (prevents composite action)
2. Wheel loads close to the supports (leads to highest shear stress)
3. Vertical crack at CIP- panel interface (due to creep and shrinkage). This reduces the shear capacity of the cast-in-place concrete, e.g. Fig. 4.8.

9.5 Deterioration Model

A deterioration model based on the field observations and analysis of localized failures was developed (Fig. 9.1). This provides information in the sequence in which deterioration can occur that may lead to localized failure. The formation of parallel longitudinal cracking is an important precursor to subsequent spalling at the critical shear location. Spalling starts a chain of events where the contribution of the cast-in-place slab in resisting loads reduces progressively. Where the shear capacity falls below the design load, localized failure can occur. More details are given in Section 4.11.

The structural behavior of precast deck panel bridges depends however on several factors not all of which can be quantified. This 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.

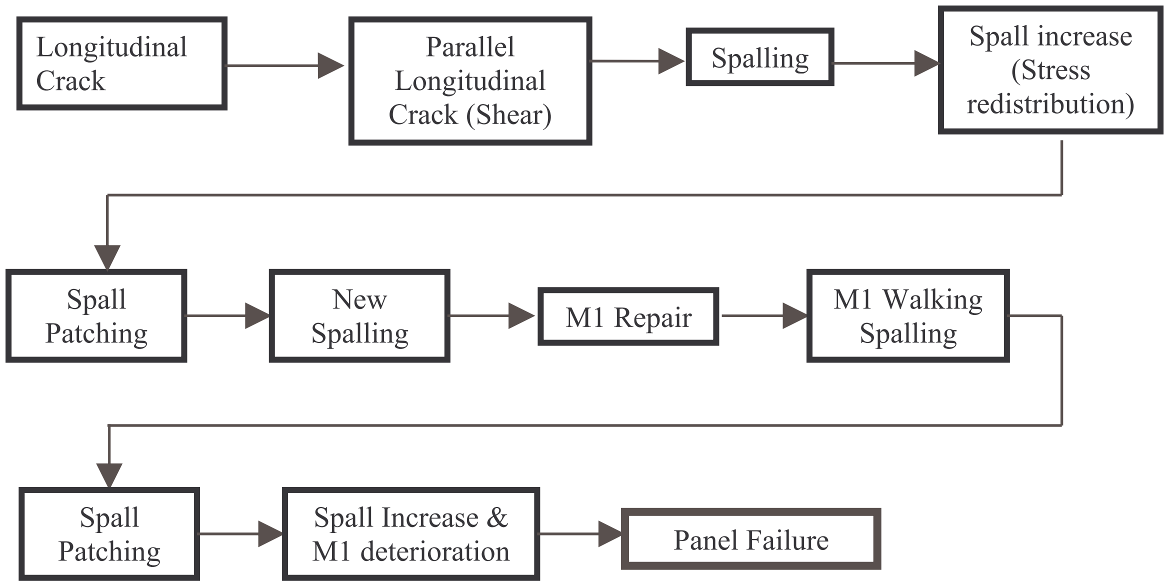


Figure 9.1 Deck Failure Tree

9.6 USF Bridge Deck Inspection Method

A new inspection method was developed that was intended to monitor crack progression in a safe and accurate manner by using a specially modified laser distancemeter. Data from this inspection is stored in electronic form for retrieval and processing by the PANEL software (see Tables 6.2, 6.3).

9.7 Finite Element Analysis

Finite element analysis of the staged construction of deck panel bridges revealed that one of the main causes of longitudinal cracking is differential shrinkage and differential creep between the panel and the deck. It was found that the amount of tensile stress developed and hence the amount of cracking was a function of the difference in the age of the precast panel and the CIP deck. Differential creep also contributed to the cracking but to a lesser degree.

9.8 PANEL Software

Custom software was developed to automate the prioritization process using data from biennial, monthly and USF inspections. A special computer data language called BRAILE (**BR**idge **A**bbreviated **I**nspection Report **L**anguag**E**) was developed to convert inspection reports into an electronic format. The software was used to prioritize replacement taking into consideration safety, importance and cost.

9.9 Recommendations

Of the 127 deck panel bridges in Districts 1 and 7, forty two have already been replaced or have been earmarked for replacement. Thus, prioritization proposed is for the remaining 85 bridges (38 in District 1, 29 in District 7 and 18 in Crosstown Expressway). All prioritization is based on a composite weighting factor that takes into consideration safety, importance and cost using the latest inspection records (see Chapter 6). Specific prioritization that considers each of these parameters separately may be found in Tables 8.3 and 8.4.

District 1

Of 38 deck panel bridges in District 1, thirteen need to be replaced. These are ranked in the recommended replacement sequence in Table 9.2. Eleven of these are on the interstate. The remaining two (#170099 and #170100) are on state roads. Photographs of the most recent condition are contained in Appendix E.

Table 9.2 Recommended District 1 Bridge Replacement Sequence

No.	Bridge ID #	Location
1	130090	I-275 NB & I-75
2	130112	I-275 SB R to I-75 NB & I-75 And I-275 Ramps
3	170081	I-75 & Palmer Blvd
4	170080	I-75 & Main A Canal
5	030188	I-75/SR-93 & CR-846
6	170094	I-75/SR-93 (NB) & Havana Road
7	170099	SR-681 SB & CSX RR
8	170089	I-75/SR-93 & River Road/Cr 777
9	170100	SR-681 NB & CSX RR
10	010064	Oil Well Road & I-75/SR93
11	030187	I-75/SR-93 & CR-846
12	170096	I-75/SR-93 SB & Jacaranda Blvd
13	170079	I-75 & Main A Canal

The remaining 25 bridges were found to be in relatively good condition. These are listed in Table 9.3. Based on the forensic studies conducted and reported in Chapter 5, this is likely due to (1) effective replacement of the fiberboard bearing or (2) use of grout bearing of the panels in the original construction. The replacement of these bridges could be deferred to a later date.

Table 9.3 District 1 Bridges in Good Condition

No.	Bridge ID#	Location
1	010059	I-75 Over CR-776
2	010065	Airport Rd Over I-75
3	010066	CR-768 Over I-75
4	010067	US-17 Over Florida St.
5	010068	US-17 Over Florida St.
6	010075	Carmalite St. Over I-75
7	010090	US-17 Over Lavilla St. & Rr
8	010091	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.

District 7

Of the 35 precast deck panel bridges in District 7, six will be replaced by the end of the year. Of the remaining 29, fifteen need to be replaced. These are ranked in the recommended replacement sequence in Table 9.4. Eleven of these are on the interstate. The remaining four are on state roads (#100338, #100080, #100081, #100351). Photographs of the most recent condition are listed in Appendix E.

Table 9.4 Recommended District 7 Bridge Replacement Sequence

No.	Bridge ID#	Location
1	100468	I-75 SB (SR-93a) & Woodberry Road
2	100347	I-75 NB & SR-674
3	100470	I-75 SB (SR 93A) & CSX RR
4	100358	I-75 SB & Alafia River
5	100359	I-75 NB & Alafia River
6	150122	I-275 NB & 5th Avenue North
7	100346	I-75 SB & SR-674
8	100436	I-75 NB & CR-574 & CSX RR
9	100338	US-41/22nd St & Mackay Bay
10	100357	I-75 NB & Riverview Drive
11	100356	I-75 SB & Riverview Drive
12	100080	SR 60 WB & Tampa Bypass Canal
13	100081	SR 60 EB & Tampa Bypass Canal
14	100049	US-41/SR-45 & Palm River
15	100351	Valroy Road & I-75/SR-93

The remaining 14 bridges (Table 9.5) were found to be in relatively good condition for the same reasons mentioned for District 1. The replacement of these bridges could be deferred to a later date.

Table 9.5 District 7 Bridges in Good Condition

No.	Bridge ID#	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 Tampa Bypass Canal
5	100424	Ramp B Over US 92
6	100435	I-75 SB Over CR-574 And CSX RR
7	100469	I-75 NB Over Woodberry Rd.
8	100471	I-75 Over CSX RR
9	150121	I-275 SB Over 5th Ave. S & 5th Ave. N
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

Crosstown Expressway

Although this was not part of the study, the performance of all 18 bridges was evaluated in the same manner. The replacement prioritization for these bridges is listed in Table 9.6.

Table 9.6 Recommended Crosstown Expressway Replacement Sequence

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

9.10 Comments on Recommendation

The study recommends that only 46 of the 85 precast deck panel bridges be considered for immediate replacement (Tables 9.2, 9.4, 9.6). The remaining 39 are in good condition (Tables 9.3, 9.5) and their replacement can be deferred. This is most probably because the panels are positively supported on epoxy or grout bearing as was discovered during the forensic examination (Chapter 4).

In this connection it is worth noting that deck panel bridges supported similarly have performed well elsewhere. Texas DOT reported (email from Mr. Brian Merrill, Manager, Construction and Maintenance Branch dated September 17, 2004) that of 1668 deck panel bridges in the state (includes 892 county bridges) only 1 was rated at condition five, 47 were rated at condition six and 1595 rated condition seven or higher (25 were not rated). They had very few cases of spalling. Thus, there are no inherent flaws in the system. Problems are due to poor implementation of the original concept in Florida.

9.11 Future Work

It is suggested that spot checks be made to verify whether the panels are positively supported or not in the bridges that are in good condition (Tables 9.3, 9.5). This can be done by field testing, e.g. monitoring deflections in selected bridges. Alternatively, consideration should be given to experimental NDT evaluations, e.g. Aerial Infrared Remote that may be able to identify anomalous areas accurately and expeditiously.

APPENDIX A

Punching Shear Calculations

Punching Shear Calculations

ASSUMPTIONS

1) Failure plane assumed to be linear.
2) Failure plane unaffected by the presence of higher compressive strength of the precast deck.
3) Prestressed panel assumed to be reinforced concrete for shear calculations.

NOTE:

Tire Contact area: $b = 20\ in$, $l = 10\ in$

Determination of shear strength: As per ACI 11.12.2.1

Shear strength of concrete V_c is smallest of the following

$$V_{c1} = \left[2 + \frac{4}{\beta_c}\right]\sqrt{f'_c}\,b_o d \qquad \text{(Equation 11-33)}$$

$$V_{c2} = \left[\alpha_s \frac{d}{b_o} + 2\right]\sqrt{f'_c}\,b_o d \qquad \text{(Equation 11-34)}$$

$$V_{c3} = 4\sqrt{f'_c}\,b_o d \qquad \text{(Equation 11-35)}$$

Where,

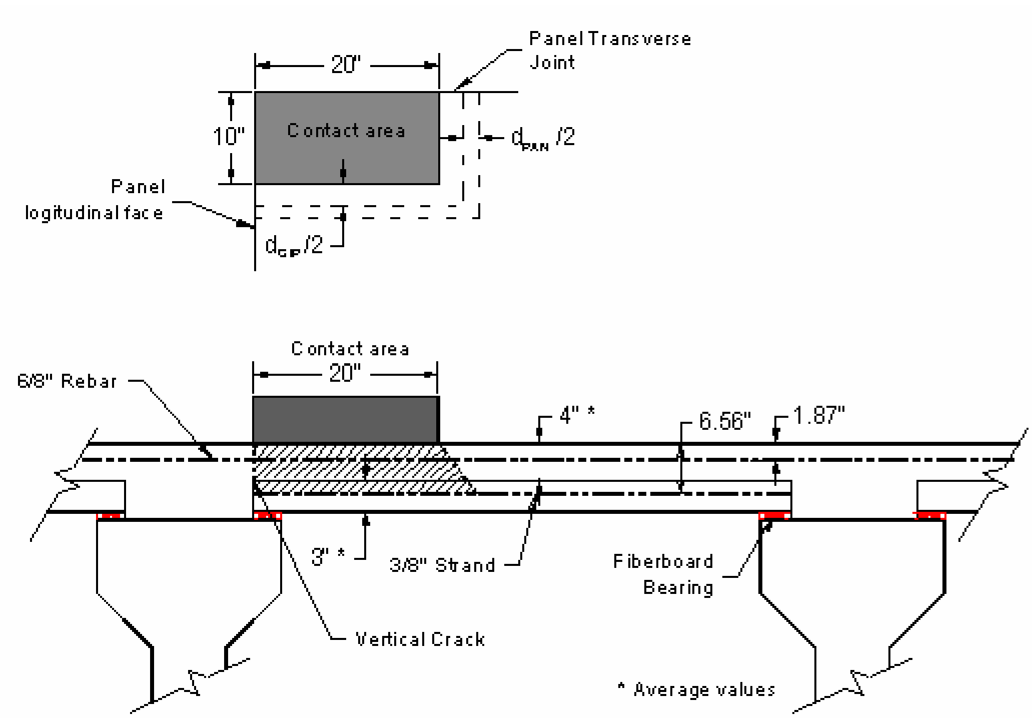
b_o = punching shear area at distance d/2 from the face of the loaded area

β_c = ratio of long side to short side of the concentrated area

$\beta_c = \left[\frac{20}{10}\right] = 2$

α_s = 20 (corner) α_s = 40 (center)

Case 1 Full Composite Action (Corner)



-Cast in place slab

$f'_{c\_CIP} = 3000$ psi $\qquad\qquad d_{CIP} = 4$ in

$$b_0 = \left[b + \left(\frac{d_{CIP}}{2}\right)\right] + \left[l + \left(\frac{d_{CIP}}{2}\right)\right]$$

$$b_0 = \left[20 + \left(\frac{4}{2}\right)\right] + \left[10 + \left(\frac{4}{2}\right)\right] \qquad b_0 = 34\ in$$

$$V_{c1} = \left(2 + \frac{4}{\beta_c}\right) \times \sqrt{\left(f'_{c\_CIP}\right)} b_0 \times d_{CIP}$$

$$V_{c1} = \left(2 + \frac{4}{2}\right) \times \sqrt{(3000)}\, 34 \times 4 \qquad V_{c1} = 29.8\ kip \qquad \text{(Eq. 11-33)}$$

$$V_{c2} = \left(\frac{\alpha_s \cdot d_{CIP}}{b_0} + 2\right) \times \sqrt{\left(f'_{c\_CIP}\right)} b_0 \times d_{CIP}$$

$$V_{c2} = \left(\frac{40 \cdot 4}{34} + 2\right) \times \sqrt{(3000)}\, 34 \times 4 \qquad V_{c2} = 49.9\ kip \qquad \text{(Eq. 11-34)}$$

$$V_{c3} = 4 \times \sqrt{\left(f'_{c\_CIP}\right)} b_0 \times d_{CIP}$$

$$V_{c3} = 4 \times \sqrt{(3000)}\, 34 \times 4 \qquad V_{c3} = 29.8\ kip \qquad \text{(Eq. 11-35)}$$

Shear strength of cast in place slab $V_{c\_CIP} = 29.8\ kip$

-Precast deck panel

$f'_{c\_pan} = 5000$ psi $\qquad d_{pan} = 2.56$ in

$$b_0 = \left[b + \left(\frac{d_{CIP}}{2}\right) + \left(\frac{d_{pan}}{2}\right)\right] + \left[l + \left(\frac{d_{CIP}}{2}\right) + \left(\frac{d_{pan}}{2}\right)\right]$$

$$b_0 = \left[20 + \left(\frac{4}{2}\right) + \left(\frac{2.56}{2}\right)\right] + \left[10 + \left(\frac{4}{2}\right) + \left(\frac{2.56}{2}\right)\right] \qquad b_0 = 36.56\ in$$

$$V_{c1} = \left(2 + \frac{4}{\beta_c}\right) \times \sqrt{\left(f'_{c\_pan}\right)}\, b_0 \times d_{pan}$$

$$V_{c1} = \left(2 + \frac{4}{2}\right) \times \sqrt{(5000)} \times 36.56 \times 2.56 \qquad V_{c1} = 26.5\ kip \qquad \text{(Eq. 11-33)}$$

$$V_{c2} = \left(\frac{\alpha_s \cdot d_{pan}}{b_0} + 2\right) \times \sqrt{\left(f'_{c\_pan}\right)} b_0 \times d_{pan}$$

$$V_{c2} = \left(\frac{40 \cdot 2.56}{36.56} + 2\right) \times \sqrt{(5000)} \times 36.56 \times 2.56 \qquad V_{c2} = 31.7\ kip \qquad \text{(Eq. 11-34)}$$

$$V_{c3} = 4 \cdot \sqrt{\left(f'_{c\_pan}\right)} b_0 \times d_{pan}$$

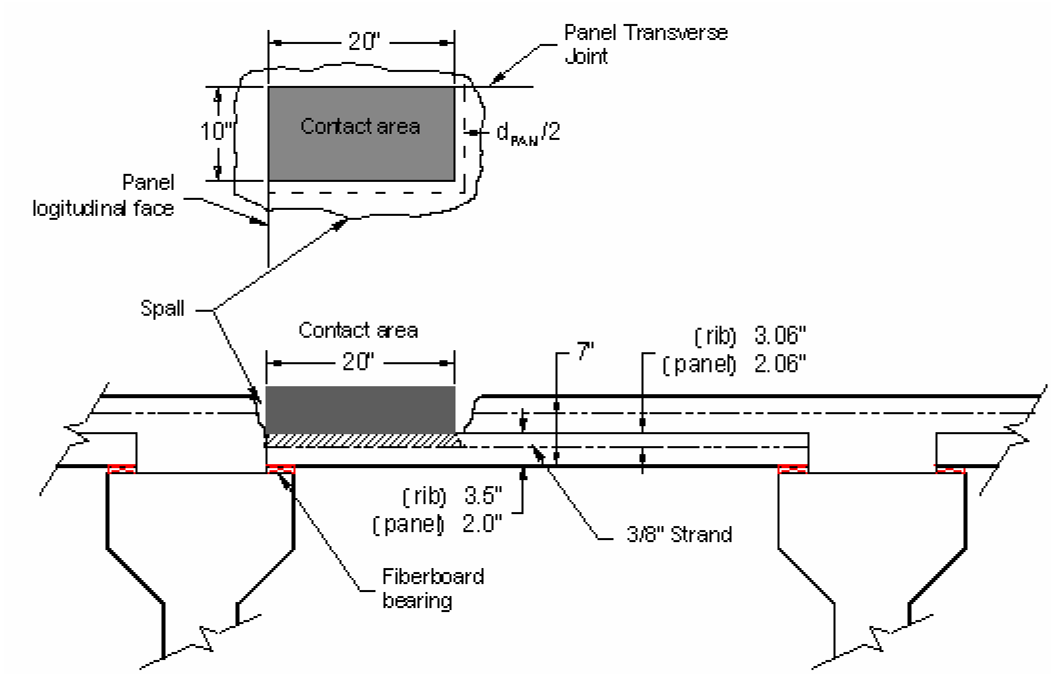
$$V_{c3} = 4 \cdot \sqrt{(5000)} \times 36.56 \times 2.56 \qquad V_{c3} = 26.5\ kip \qquad \text{(Eq. 11-35)}$$

Shear strength of pre-cast panel $V_{c\_panel} = 26.5\ kip$

Total composite deck punching shear strength $\mathrm{V_{comp}} = \mathrm{V_{c\_panel}} + \mathrm{V_{c\_CIP}}$

$$V_{comp} = 56.3\ kip$$

Case 2 No Composite Action (Corner)



$\beta_c = 2$ $\mathrm{f'_{c\_pan}} = 5000\ \mathrm{psi}$ Min $d_{pan} = 2.06\ in$ $\alpha_s = 20$ (corner)

$$b_0 = \left(l + \frac{d_{pan}}{2}\right) + \left(b + \frac{d_{pan}}{2}\right)$$

$$b_0 = \left(10 + \frac{3.06}{2}\right) + \left(20 + \frac{2.06}{2}\right) \qquad b_0 = 32.06\ in$$

$$V_{c1} = \left(2 + \frac{4}{\beta_c}\right) \times \sqrt{\left(f'_{c\_pan}\right)}\, b_0 \times d_{pan}$$

$$V_{c1} = \left(2 + \frac{4}{2}\right) \times \sqrt{(5000)} \times 32.06 \times 2.06 \qquad V_{c1} = 18.7\ kip \quad \text{(Eq. 11-33)}$$

$$V_{c2} = \left(\frac{\alpha_s \cdot d_{pan}}{b_0} + 2\right) \times \sqrt{\left(f'_{c\_pan}\right)} b_0 \times d_{pan}$$

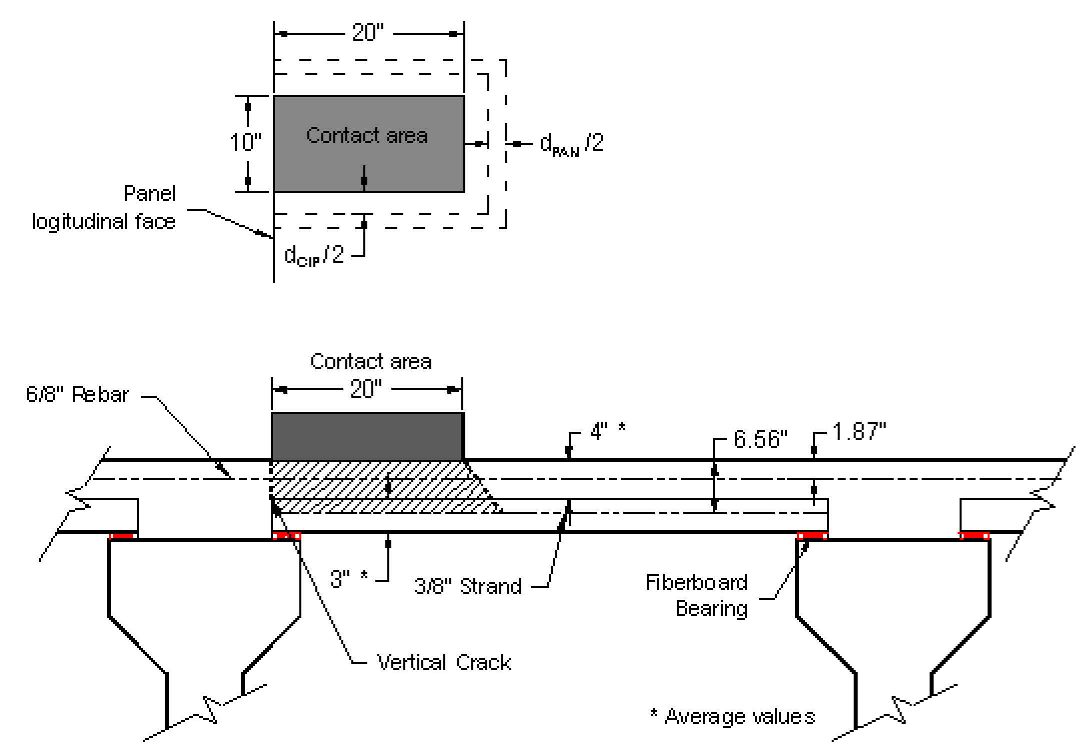
$$V_{c2} = \left(\frac{20 \times 2.06}{32.06} + 2\right) \times \sqrt{(5000 \cdot psi)} \times 32.06 \times 3.06 \qquad V_{c2} = 15.3\ kip \quad \text{(Eq. 11-34)}$$

$$V_{c3} = 4 \times \sqrt{\left(f'_{c\_pan}\right)} \times b_0 \times d_{pan}$$

$$V_{c3} = 4 \times \sqrt{(5000)} \times 32.06 \times 2.06 \qquad V_{c3} = 18.7\ kip \quad \text{(Eq. 11-35)}$$

Shear strength of pre-cast panel $V_{c\_rib} = 15.3\ kip$

Case 3 Full Composite Action (Edge)



-Cast in place slab

$f'_{c\_CIP} = 3000 \text{ psi}$ $\qquad d_{CIP} = 4 \text{ in}$

$$b_0 = 2\left[b + \left(\frac{d_{CIP}}{2}\right)\right] + \left[l + 2\left(\frac{d_{CIP}}{2}\right)\right]$$

$$b_0 = 2\left[20 + \left(\frac{4}{2}\right)\right] + \left[10 + 2\left(\frac{4}{2}\right)\right] \qquad b_0 = 58\ in$$

$$V_{c1} = \left(2+\frac{4}{\beta_c}\right)\times\sqrt{(f'_{c\_CIP})}b_0 \times d_{CIP}$$

$$V_{c1} = \left(2+\frac{4}{2}\right)\times\sqrt{(3000)}58\times 4 \qquad V_{c1} = 50.8kip \qquad \text{(Eq. 11-33)}$$

$$V_{c2} = \left(\frac{\alpha_s \cdot d_{CIP}}{b_0}+2\right)\times\sqrt{(f'_{c\_CIP})}b_0 \times d_{CIP}$$

$$V_{c2} = \left(\frac{40\cdot 4}{58}+2\right)\times\sqrt{(3000)}58\times 4 \qquad V_{c2} = 60.4\ kip \qquad \text{(Eq. 11-34)}$$

$$V_{c3} = 4\times\sqrt{(f'_{c\_CIP})}b_0 \times d_{CIP}$$

$$V_{c3} = 4\times\sqrt{(3000)}58\times 4 \qquad V_{c3} = 50.8kip \qquad \text{(Eq. 11-35)}$$

Shear strength of cast in place slab $V_{c\_CIP} = 49.1\ kip$

-Precast deck panel

$$\mathrm{f'_{c\_pan}} = 5000 \text{ psi} \qquad \mathrm{d_{pan}} = 2.56 \text{ in}$$

$$b_0 = 2\left[b+\left(\frac{d_{CIP}}{2}\right)+\left(\frac{d_{pan}}{2}\right)\right]+\left[l+2\left(\frac{d_{CIP}}{2}\right)+2\left(\frac{d_{pan}}{2}\right)\right]$$

$$b_0 = 2\left[20+\left(\frac{4}{2}\right)+\left(\frac{2.56}{2}\right)\right]+\left[10+2\left(\frac{4}{2}\right)+2\left(\frac{2.56}{2}\right)\right] \qquad b_0 = 63.12\ in$$

$$V_{c1} = \left(2+\frac{4}{\beta_c}\right)\times\sqrt{(f'_{c\_pan})}\,b_0 \times d_{pan}$$

$$V_{c1} = \left(2+\frac{4}{2}\right)\times\sqrt{(5000)}\times 63.12\times 2.56 \qquad V_{c1} = 45.7\ kip \qquad \text{(Eq. 11-33)}$$

$$V_{c2} = \left(\frac{\alpha_s \cdot d_{pan}}{b_0}+2\right)\times\sqrt{(f'_{c\_pan})}b_0 \times d_{pan}$$

$$V_{c2} = \left(\frac{40\cdot 2.56}{63.12}+2\right)\times\sqrt{(5000)}\times 63.12\times 2.56 \qquad V_{c2} = 41.4\ kip \qquad \text{(Eq. 11-34)}$$

$$V_{c3} = 4\cdot\sqrt{(f'_{c\_pan})}b_0 \times d_{pan}$$

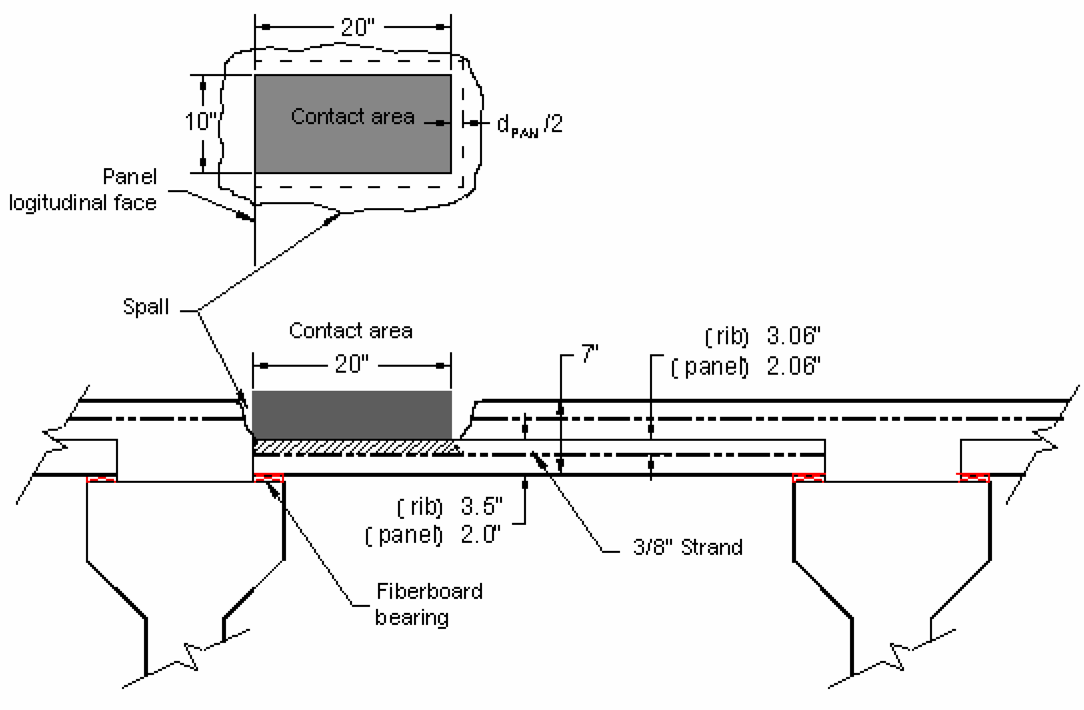
$$V_{c3} = 4\cdot\sqrt{(5000)}\times 63.12\times 2.56 \qquad V_{c3} = 45.7\ kip \qquad \text{(Eq. 11-35)}$$

Shear strength of pre-cast panel $V_{c\_panel} = 41.4\ kip$

Total composite deck punching shear strength $\mathrm{V_{comp} = V_{c\_panel} + V_{c\_CIP}}$

$$V_{comp} = 92.2\ kip$$

Case 4 No Composite Action (Edge)



$\beta_c = 2$ $f'_{c\_pan} = 5000$ psi Min $d_{pan} = 2.06\ in$ $\alpha_s = 20$ (corner)

$$b_0 = \left(l + 2\frac{d_{pan}}{2}\right) + 2\left(b + \frac{d_{pan}}{2}\right)$$

$$b_0 = \left(10 + 2\frac{3.06}{2}\right) + 2\left(20 + \frac{2.06}{2}\right) \qquad b_0 = 54.12\ in$$

$$V_{c1} = \left(2 + \frac{4}{\beta_c}\right) \times \sqrt{\left(f'_{c\_pan}\right)}\, b_0 \times d_{pan}$$

$$V_{c1} = \left(2 + \frac{4}{2}\right) \times \sqrt{(5000)} \times 32.06 \times 2.06 \qquad V_{c1} = 31.5\ kip \quad \text{(Eq. 11-33)}$$

$$V_{c2} = \left(\frac{\alpha_s \cdot d_{pan}}{b_0} + 2\right) \times \sqrt{\left(f'_{c\_pan}\right)} b_0 \times d_{pan}$$

$$V_{c2} = \left(\frac{20 \times 2.06}{32.06} + 2\right) \times \sqrt{(5000 \cdot psi)} \times 32.06 \times 3.06 \qquad V_{c2} = 21.7\ kip \quad \text{(Eq. 11-34)}$$

$$V_{c3} = 4 \times \sqrt{\left(f'_{c\_pan}\right)} \times b_0 \times d_{pan}$$

$$V_{c3} = 4 \times \sqrt{(5000)} \times 32.06 \times 2.06 \qquad V_{c3} = 31.5\ kip \quad \text{(Eq. 11-35)}$$

Shear strength of pre-cast panel $\qquad V_{c\_rib} = 21.7\ kip$

APPENDIX B

Deck Panel Bridge Survey

B.1 Deck Panel Survey

Information on the performance of precast deck panel systems available in the literature review is to a limited to a few states e.g. Florida, Texas, Pennsylvania, and Iowa. In view of this a nationwide survey was conducted to obtain additional information on the experience of other states with these bridges.

The National Bridge Inventory (NBI) database was used to identify states using composite bridge deck construction. Guidelines mentioned in the Recording and Coding Guide [1] were used to identify deck panel bridges (Table B.1). Unfortunately, the guide did not specifically mention any code for prestressed, precast deck panel systems. The probable code was determined by reviewing the NBI code for precast deck panel bridges in Districts 1 and 7.

Table B.1 Guidelines for Identifying Precast Deck Panel Bridges

Item	Description	Code	Description
43 A	Kind of material and/ or design	5	Prestressed concrete
43 B	Type of design and/or construction	02	Stringer/multibeam or girder
107	Deck structure type	2	Concrete precast panels

Thirty nine states were identified as *possible* owners of deck panel bridges. Information on deck panel bridges for a particular state was obtained from the NBI database and compiled in a tabular format as shown in Table B.2.

Table B.2 Information from NBI Database

State: Alaska
Code: 020

Source: National Bridge Inventory
Deck: Precast prestressed Panels
Total Number of Bridges:- 6

Bridge nos.	Year Built	Condition rating	Year of Condition Rating	Comments
9830005P0000000	1951	7	2001	Reconstructed in 1983
0983	1974	8	2000	First bridge built
0470	1974	8	2000	
0386	1974	7	2000	
0385	1974	7	2000	
0284	1974	7	2000	Last bridge built

B.2 Information Sent to DOT's

The following information was sent to selected DOT's:

(1) Letter from the Principal Investigator stating in brief the purpose of the survey and the feedback requested.
(2) Sketches showing typical deficiencies found in Florida bridges.
(3) NBI information summarized (Table B.2) in a tabular format.

Of the 39 states identified, 17 states were contacted since information on others was not readily available. The states contacted were Alabama, Alaska, Arkansas, Colorado, Georgia, Idaho, Indiana, Iowa, Kansas, Maryland, Michigan, Montana, New Jersey, New Mexico, Oklahoma, Louisiana and Oregon. Letters were also sent to Districts 2, 3, 4 and 5 under the jurisdiction of Florida Department of Transportation.

B.3 Response from DOT's

Fourteen of the 17 states contacted and FDOT's Districts 2, 4 and 5 responded to the survey. Some provided detailed information on composite deck design, specifications and performance. The responses are summarized below.

B.3.1 Alabama Department of Transportation

Out of the list of 100 bridges identified only 2 were composite deck bridges. They have been performing well without any deck maintenance problems for 25 years. The information was provided by Mr. Fred Conway from Alabama DOT.

B.3.2 Alaska Department of Transportation

Out of the list of 6 bridges sent, none were deck panels but were instead quad-stem precast prestressed concrete girders covered with a waterproof membrane and asphalt wearing surface. The following coding system is used by Alaska DOT to identify deck panel bridges:

Item 43 A = 5 – Prestressed Concrete
Item 43 B = 04 – Tee Beam
Item 107 = 9 – Other

Excepting 43 A, other coding is different from what was used to identify these bridges. When the NBI database was re-checked using the above coding system, it was found that Alaska DOT did not use deck panel bridges. The information was provided by Mr. Gary Scarborough and Mr. John Orbistondo.

B.3.3 Arkansas State Highway and Transportation Department

NBI database indicated that only two bridges were deck panel bridges. According to Mark Bradley, Staff Research Engineer Bridge, neither were deck panel bridges. Mark Bradley, Staff Research Engineer with DOT provided the information.

B.3.4 Florida Department of Transportation, District 2

Mr. Keith Campbell, District Structures and Facilities Engineer provided the following information on deck panel bridges in District 2.

(1) List of Deck Panel Bridges: There are 14 bridges under jurisdiction of District 2, FDOT (Table B.3)

Table B.3 List of Panel Bridges in District 2, FDOT

Bridge #	Facility	Feature Intersected	Year Built	Deck Condition Rating	Year of Rating
290083	US-42 (SR-25)	Suwannee River	1980	7	2003
720425	SR-134(TIMUQUANA)	Finishing Creek	1978	7	2002
720442	SR-202 WB(JTB)	Intercoastal Waterway	1979	7	2003
720443	SR-202 WB(JTB)	Cut Creek Branch	1978	7	2002
720444	SR-202 WB(JTB)	Cut Creek Tributary	1978	7	2002
720445	SR-202 EB(JTB)	Cut Creek Tributary	1978	7	2002
720451	SR-202 WB(JTB)	Cedar Swamp Creek	1979	7	2003
720452	SR-202 EB(JTB)	Cedar Swamp Creek	1979	7	2003
720460	SR-202 WB(JTB)	Hodges Blvd.	1979	7	2003
720461	SR-202 EB(JTB)	Hodges Blvd	1979	7	2003
740087	SR-200 WB & SR-A1A	Amelia River	1978	7	2002
740088	SR-200 EB & SR-A1A	Amelia River	1978	6	2002
760044	US-17 SB(SR-15)	Rice Creek	1981	7	2003
760045	US-17 NB(SR-15)	Rice Creek	1981	7	2003

(2) There have been no reported failures and none have been replaced.
(3) Many of the bridges replaced the fiberboard with epoxy. However, a detailed examination of two bridges (740087, 740088) indicated that 90% of the time the epoxy did not penetrate the full depth of the void.
(4) Latest Inspections reports were made available for all the bridges. The deficiencies that were noted are summarized in Table B.4.

Table B.4 Information on Deficiencies for District 2 Bridges

Bridge #	Deck Top	Deck Underside
290083	Insignificant longitudinal, transverse and random map cracks.	Moderate longitudinal and transverse cracks at random locations.
720425	Insignificant longitudinal, transverse, diagonal and map cracks in all spans	-
720442	Moderate longitudinal, transverse and diagonal cracks at random locations. 16" x 16" x3/4" spall in Span 16. Four pieces of averaging 2" each and one 24" piece of exposed reinforcing steel in Span 52.	Insignificant transverse cracks and minor spalls at random locations throughout the structure. 18"x12"x6" spall with exposed prestressing strands between beam 54-4 and 54-5.
720443	12"x4"x2" deep spall in the south deck overhang over pier 5.	-
720444	-	Insignificant longitudinal and transverse cracking at various locations throughout.
720445	Repaired area 15' x 25" wide in span 1, insignificant longitudinal and transverse cracking, down center of repair entire length. 24"x8"x2" deep spall at the end of repaired area. Spalled area has two reinforcing steel bars exposed 1" and 3".	Insignificant longitudinal and transverse cracking at various locations in several deck panels.
720451	Random insignificant transverse cracks and map cracking.	-
720452	Random insignificant transverse cracks, map cracking and moderate scaling in all spans. 6"x6"x1" spall in Span 1. 6 ½' x 3 ½' honeycombed area in Span 5.	-
720460	Insignificant longitudinal and transverse cracking at random locations throughout. 12"x8"x2" spall.	Several insignificant longitudinal cracks in Span 1 and Span 3. Minor spall in Span 2
720461	Insignificant transverse cracks	-
740087	Insignificant longitudinal cracks in most spans, random popout spalls with exposed steel, several up to moderate transverse full width cracks in Spans 24, Span 25 and Span 26.	Moderate transverse cracks throughout all the spans. Span 17 has 1.5 mm longitudinal crack. Deck Panel 6 between beams 25-3 and 25-4 has a 36 m x1.2 mm wide diagonal crack with a 0.1 m x 0.1 m x 0.05 m spalled area with an exposed prestressing cable. Deck Panel 26, between girders 25-1 and 25-2 is spalled in a 1.5 m x 0.46 m x 0.12 m area with exposed steel. 1.3 m x 1 mm wide longitudinal crack in Span 37.

Table B.4 (continued) Information on Deficiencies for District 2 Bridges

Bridge #	Deck Top	Deck Underside
740088	Insignificant longitudinal cracks in most spans. These cracks in Spans 10, 17, and 18 have areas along the cracks that are spalled. Random popout spalls with exposed reinforcing steel throughout all the spans. Several moderate transverse cracks which extend the full width of the deck.	Moderate transverse cracks throughout all the spans, several with efflorescence.
760044	Several spans have insignificant longitudinal, transverse and map cracks	Insignificant transverse cracks at random locations throughout the structure. 8”x5”x1 ¼” spall with 4” of exposed rebar in Span 4.
760045	Several spans have insignificant size of transverse and map cracks	Insignificant transverse cracks at random locations throughout the structure.

B.3.5 Florida Department of Transportation, District 4

Mr. Mathew Akers provided information on deck panel bridges in District 4. The district has 52 bridges as listed in Table B.5.

Table B.5 List of Panel Bridges in District 4, FDOT

Bridge #	Facility	Feature Intersected.
860177	EB Sunrise Blvd.	SR-91 Fla.Turnpike
860245	WB Oakland Pk Blvd	SR-91 Fla.Turnpike
860246	EB Oakland Pk Blvd	SR-91 Fla.Turnpike
860251	EB Sample Rd	SR-91 Fla.Turnpike
860304	I-75 (SR-93) NB	Arvida Pkwy (NW 196 AVE)
860305	I-75 (SR-93) SB	Arvida Pkwy (NW 196 AVE)
860316	Miramar Pkwy	I-75 (SR-93)
860320	Bass Creek Road	I-75 (SR-93)
860327	I-75 (SR-93) SB	Snake Creek
860328	I-75 NB (SR-93)	Snake Creek
860329	Sheridan St(72ND)	I-75 (SR-93)
860330	Stirling Road WB	I-75 (SR-93)
860331	Stirling Road EB	I-75 (SR-93)
860333	I-75 (SR-93) NB	US-27 (SR-25)
860335	Ramp G-H OVER US27	US-27 (SR-25)
860336	Ramp E-F OVER US27	US-27 (SR-25)
860350	SR-820	I-75 (SR-93)
860351	I-75 SB (SR-93)	C-4 Canal
860352	I-75 NB (SR-93)	C-4 Canal
860352	I-75 NB (SR-93)	C-4 Canal

Table B.5 (continued) List of Panel Bridges in District 4, FDOT

Bridge #	Facility	Feature Intersected.
860553	WB SW 10th ST	Service Road/SC Railroad
860557	EB SW 10th ST	Service Road/SC Railroad
860592	New Griffin Rd.	Dania Cut-Off Canal
864067	McNab Road	US 441 (SR-7)
880077	SR-656 (17th St.)	Intracoastal Waterway
880083	SR-60	Padgett Branch Marsh
890093	CR-76A	St. Lucie Canal
930265	PGA Blvd. (SR 786)	SR-91, Fla.Turnpike
930269	SR A1A	Intracoastal Waterway
930322	Linton Blvd	Intracoastal Waterway
930335	SB I-95 (SR-9)	PGA BLVD (SR 786)
930336	NB I-95 (SR-9)	PGA BLVD (SR 786)
930339	SR-811 (Alt A-1-A)	Loxahatchee River
930349	WB PGA Blvd.	Intracoastal Waterway
934275	Australian Ave SB	Okeechobee Blvd
934276	Australian Ave NB	Okeechobee Blvd
934940	Piper's Glen Road	LWDD E-3 Canal
935303	S. Lake Drive	Canal at Lake Ida N. End
935305	Mission Hill Rd	Canal E4 Lake Ida N. End
936551	Monet Road	Scotts Canal
940108	SB I-95 (SR-9)	Gatlin Blvd
940109	NB I-95 (SR-9)	Gatlin Blvd
940111	NB I-95 (S.R. 9)	CR 712 (Midway Road)
940112	SB I-95 (S.R. 9)	CR-712 Midway Road
940113	SB I-95 (SR-9)	Galiano Rd. & C-24 Canal
940114	NB I-95 (SR-9)	Galiano Rd & C-24 Canal
940115	SB I-95 (SR 9)	CR 709 & FECRR
940116	NB I-95 (SR 9)	CR 709 & FECRR
940122	SB I-95 (SR-9)	Ten Mile Creek
940123	NB I-95 (SR-9)	Ten Mile Creek
940126	SB I-95 (SR-9)	SR 91, Fla.Turnpike
940127	NB I-95 (SR-9)	SR 91, Fla.Turnpike

(1) Reported Failures: Out of the above list there were failures reported on two occasions in bridges 940126 and 940127.
(2) Many of the bridges replaced fiberboards with epoxy.
(3) Information on deck panel bridges replaced: Span 2 of bridges 940126 and 940127 were replaced. Some spans (numbers not provided) of bridges 940113, 940114 will be replaced.

Latest inspection reports and photos of deficiencies observed on the bridges that were and will be replaced were made available. For each bridge the information is summarized.

(1) *Bridge 940113*: Built in 1982 and has deck condition rating 7 (12/26/2002). Deck top has delamination; patch that sounds hollow and new spall is formed adjacent to north end of patch i.e. it is a failing repair. Also a spall with exposed rebar was present in Span 3. Span 4 has delamination on the underside of the deck. It seems likely that Span 3 will be replaced.

(2) *Bridge 940114*: Built in 1982 and has deck condition rating 7 (12/26/2002) Deck top has spalls and longitudinal cracks in Span 2 and the slab underside has converging cracks with light efflorescence in Span 6. It seems likely that Span 2 will be replaced.

(3) *Bridge 940126*: Built in 1982 and has deck condition rating 6 (7/9/2002). A portion of the deck on precast panels was replaced with a reinforced concrete deck in 1995. This replaced area of deck is located between BM2-3 and BM2-5 (full span length). There are two spalls with exposed steel in span 2. Patches are in good condition. There is loss of bearings under the concrete stay-in-place panel due to which there is heavy leakage of water.

(4) *Bridge 940127*: Built in 1982 and has deck condition rating 7 (12/26/2002). Span 2 is being replaced with reinforced concrete deck. There are numerous minor deck cracks throughout the structure. Sealant was applied to most visible of these cracks

B.3.6 Florida Department of Transportation, District 5

Mr. Ron Meade provided information on deck panel bridges in District 5. Details are listed in Table B.6. An explanation of the condition state is listed in Table B.7.

Table B.6 List of Panel Bridges in District 5, FDOT

Bridge #	Facility Carried	Feature Intersected	Number Of Spans	(98) Conc. Deck On PC Deck Panels Cond. State
750082	I-4	Par Avenue	3	2
750084	I-4	Formosa-Minnesota	8	2
750139	I-4	SR-423	4	3
750142	I-4	Central Florida Pkwy	1	3
750195	I-4	Par Avenue	3	3
750196	I-4	Formosa-Minnesota	6	2
750197	I-4	SR-423	4	3
750198	I-4	C 438 A	3	4
750200	I-4	Central Florida Pkwy	1	2
750256	I-4	SR-426	4	2
750261	I-4	SR-426	4	3
920100	I-4	Bonnet Creek	4	2
920101	I-4	Bonnet Creek	4	1
920034	US 192	Shingle Creek	5	AC Overlay

Table B.6 (continued) List of Panel Bridges in District 5, FDOT

Bridge #	Facility Carried	Feature Intersected	Number Of Spans	(98) Conc. Deck On PC Deck Panels Cond. State
790077	I-95	Tomoka River	12	2
790078	I-95	Tomoka River	12	2
790132	SR 40	Halifax River (Granada)	19	2
790124	SR 415	St. Johns River	37	2
770030	SR 434	Little Wekiva River	3	3
750316	SR 436	SR 528	5	3
750317	SR 436	SR 528	5	2
750315	SR 436 NB On Ramp	SR 528	3	2
790128	SR 44	Us 1	8	2
920146	SR 15/US 441	Blue Cypress Creek	4	AC Overlay
750085	SR 426	Lake Osceola Canal	1	AC Overlay
750013	SR 50	Econlockhatchee River	12	AC Overlay
750031	SR 50	Lake Sherwood	3	AC Overlay
700186	SR 507	SR 507 Over Crane Creek	3	AC Overlay
700185	SR 507 BABC OCK ST	Tillman Canal	3	AC Overlay
750319	SR 528	Daetwyler Road	3	3
750294	SR–482 (WB)	Floridas Turnpike	4	3
700140	SR-528 EB	SR-401	2	2
700074	SR-528 WB	SR-401	2	2
700174	US 192	Indian River	27	4
750004	US 17–92	Lake Estelle	3	AC Overlay
700173	US 192	Indian River Relief East	6	AC Overlay
700175	US 192	Indian River Relief East	6	AC Overlay
920032	US 192	Reedy Creek	6	AC Overlay
750003	US17–92	Lake Rowena	3	AC Overlay
920035	US–441	Bull Creek	6	AC Overlay
920940	US–441	Crabgrass Creek–West Br	3	AC Overlay

District 5 began replacing fiberboard under deck panels about 15 years ago when water damage (caused by longitudinal cracks) was noticed. Additionally, they made the Local Maintenance Units monitor the condition of all deck panel bridges with instructions to notify the district office of any problems. This was to ensure that they were properly repaired. They believe that these measures prevented major problems.

Table B.7 D5 PC Deck Panel Bridges Condition State Descriptions

98 - CONCRETE DECK ON PRECAST DECK PANELS (EA) This element defines those concrete bridge decks cast on precast deck panels. AC = Asphalt Concrete
CONDITION STATE DESCRIPTIONS
(1) The surface and underside of the deck has no repaired areas, there are no spalls/delaminations in the deck surface or underside and the only cracking is superficial.
(2) Repaired areas and/or spalls/delaminations and/or cracks exist in the deck surface or underside. The combined distressed area is 2% or less of the deck area.
(3) Repaired areas and/or spalls/delaminations and/or cracks exist in the deck surface or underside. The combined area of distress is more than 2% but less than 10% of the total deck area.
(4) Repaired areas and/or spalls/delaminations and/or cracks exist in the deck surface or underside. The combined area of distress is more than 10% but less than 25% of the total deck area.
(5) Repaired areas and/or spalls/delaminations and/or cracks exist in the deck surface or underside. The combined area of distress is more than 25% of the total deck area.

B.3.7 Georgia Department of Transportation

Following information was sent by Mr. Paul Liles, State Bridge Engineer regarding the survey:

(1) Out of the list of 45 bridges sent, 35 bridges were precast prestressed deck panel bridges. During 1982-1995 many more deck panel bridges (more than listed) were built in Georgia but the information is not in the bridge inventory. Decks are usually coded as full depth concrete decks and the use of deck panels does not show up in the NBI data.
(2) There is some cracking on the decks, but are held together by the top mat of bar reinforcement. No significant spalling is present. Bridges are performing satisfactorily and there is no need for deck replacement at this time.
(3) This method of construction was used as a contractor's proposed alternative to a full depth slab from 1982 to 1995. By 1995, the contractors stopped using deck panels completely and went back to using metal deck forms with full depth slabs.
(4) They stopped using this construction in 2000 and it was deleted from the 2001 specifications.
(5) Composite slab details used by Georgia DOT are as shown in Table B.8.

Table B.8 Composite Slab Details used by Georgia DOT

Slab	Description	Value
Precast panel	f'c	Min – 4000 psi at release Max – 5000 psi at 28 days
	Transverse reinforcement	0.11 sq. in. per foot of the panel
	Limiting compressive stress	100 psi
	Prestressing strands spacing	Max. – 1½ times composite slab thickness not more than 18 inches.
	Width	Min. – 4 ft. for interior panels 2 ft. for closure panels Max. – 8 ft. parallel to traffic
CIP	Longitudinal reinforcement	0.25 sq. in. per foot of slab width
	Cover to longitudinal reinf.	¾ in.
Bearing	Material	Mastic, polystyrene or fiberboard
	Thickness	Minimum – 1in. Maximum – 3 in.
	Bearing length	Minimum – 1 ½ in.

Georgia DOT Design Specifications for Panel Bridges

(1) Effective beam width considered for transfer of horizontal shear shall be the clear distance between edges of panels.
(2) Top surface of the panels shall receive a scored finish with a depth of scoring 1/8th inch in the panel.
(3) Panels shall overhang the bearing material by 1 ½ in. minimum.
(4) To ensure full bond between precast and CIP, interface should be free from any foreign matter. Immediately prior to placing the slab concrete, the panels shall be saturated with water.
(5) Panels used with AASHTO Type V beams min. overlap - 5 ½ in. with minimum bearing length - 2 in.

As seen from the specifications, panels overhang the bearing material by a minimum of 1 ½ in., allowing mortar to flow beneath the panels and act as a positive bearing. This is the likely reason for their satisfactory performance in Georgia.

B.3.8 Indiana Department of Transportation

Bill Dittrich, Bridge Inspection Engineer mentioned that the NBI item picked for identifying concrete deck panels was for precast channel shaped concrete beams used for the superstructure. There was a separate item in the INDOT database for INDOT bridges that have concrete deck panels under all or part of the bridge deck. Concrete deck panel was still an option for contractor's to use, but were not being used much any more.

The prestressed panels used were 2.5 to 3 in. thick. Much cracking (in the panels only) is found on the end panels on skewed bridges and the end panels have been replaced by cast-in-place deck. The bridges that have concrete deck panels have many transverse cracks in the panels.

A list of 98 bridges with the year built, year reconstructed along with the condition rating of the superstructure was provided. The first bridge was built in 1980 (145-13-06874) and the last one in 2002 (136-32-07782). All the bridges are in good condition and performing well as indicated by condition rating. The bridges mentioned in the NBI database were mostly county bridges.

B.3.9 Iowa Department of Transportation

Following information was sent by Bruce Brakke, Bridge Maintenance Engineer with the Department:

(1) Panels used are basically stay-in-place forms which become an integral part of the deck.
(2) The three bridges (16051, 238551, and 27191) out of the list of 30 were under the jurisdiction of Iowa DOT and were performing satisfactorily except one with few short hairline longitudinal cracks over the piers and few hairline transverse cracks at the bottom of the deck.
(3) They are permitted on bridges owned by Iowa DOT if all of the following condition are met
    (i) The bridge is constructed with pretensioned prestressed concrete beams.
    (ii) Intermediate diaphragms are steel.
    (iii) Skew is 45 degrees or less.
    (iv) The bridge is on a rural highway with a traffic volume ADT of less than 3000.
    (v) The bridge is not being built by staged construction.
(4) Panel Projection on to the beams is 4 in ± ½ in.
(5) Fiberboard bearing (3/4 in min to 3¼ in max. thickness and 1½ in wide) used, glued to the top edges of prestressed beams.
(6) Composite slab details used by Iowa DOT are shown in Table B.9.

Table B.9 Composite Slab Details used by Iowa DOT

Slab	Description	Value
Cast in Place	Thickness	3 in
	F'c	3500 psi
	Transverse reinforcement	#6@11 in. o.c.-Grade 60 2 ½ in. cover from top
	Longitudinal reinforcement	#5@9 in. o.c.-Grade 60
Precast Panel	Thickness	5 in.
	F'c	4500 psi at release 6000 psi(28 day)
	Prestressing strands	3/8 in. dia. 270 ksi stressed to 16.1 kips. Spacing 6 in., 1½ in. cover from interface
	Transverse reinforcement	6x6- W5.5x5.5 mesh or #3 spaced at 12 in o.c.

B.3.10 Kansas Department of Transportation

Kansas currently has 42 bridges with decks constructed using prestressed concrete panels. These panels are used only on prestressed beams. Most of these bridges are performing satisfactorily. Only deficiencies observed were longitudinal reflective cracks on some bridges. List of 42 bridges was provided with the year built and the NBI coding. The bearing used is extended or extruded polystyrene bedding material and is epoxied to the girders. Following coding system is used by the Kansas DOT to identify deck panel bridges:

Item 43 A = 5 – Prestressed Concrete
Item 43 B = 02 – Multi Stringer Beam or Girder
Item 107 = 02 – Concrete Precast Panels

Details of the composite bridge deck systems were also provided (Table B.10).

Table B.10 Composite Slab Details used by Kansas DOT

Slab	Description	Value
Cast in place	Thickness	145 mm
	f'c	35 MPa
	Transverse reinforcement	16S13 or 13S14
	Longitudinal reinforcement	19S3,19S6,or 19S8
Precast Panel	Thickness	80 mm
	f'c	35 MPa and 30 MPa at release
	Prestressing strands	10 mm ϕ –low lax
	Transverse reinforcement	10PA1
	Size	L = Max 2500 mm, Min 1800 mm W = 2090
	Initial Prestressing	76.7 kN
	Shear connectors	13PA2

The information presented here was provided by Kenneth F Hurst, P.E., Engineering Manager, State Bridge Office, Kansas DOT.

B.3.11 Louisiana Department of Transportation

Mr. Kevin John sent a list of bridges under jurisdiction of Louisiana DOT (Table B.11)

Table B.11 List of Panel Bridges in Louisiana DOT

Bridge #	Year Built	Condition Rating	Comments
085831132930761	1962	4	
085831090925341	1962	4	
085831078925301	1962	4	
085831123925711	1962	4	
085831146930231	1964	4	
85831143930321	1964	4	
085831093931491	1965	4	
084031167921981	1970	4	
084031208923431	1970	9	Replacement Structure
084030575924611	1978	4	
611708174001851	1994	7	
611708174001853	1994	7	

B.3.12 Michigan Department of Transportation

In 1980, Michigan DOT was directed not to use concrete deck panels. In 1994 and 1995 they experimented with concrete deck panels, but the panels were not prestressed. They are not used because of their performance. Instead metal deck forms are used. Mr. Roger Till of Michigan DOT provided a copy of report entitling "*Investigation of Precast Deck Panels Used in Spread Box Beam Bridges*", April 1997. The bridge (S01 of 25031) investigated was composite with stay in place precast panels overlaid by cast-in-place deck supported over prestressed concrete box beams. The precast panels were not prestressed, therefore do not represent Florida bridges.

From the investigation it was seen that numerous large cracks (longitudinal and transverse) were visible in the top deck surface. Following problems were observed in the precast panels:-

(1) Transverse hairline cracks were observed in the panels. Cracking generally occurred at the mid-panel and quarter points.
(2) There was a lack of concrete cover over the bottom mesh reinforcement. The mesh pattern reflected through the concrete exposing steel in random locations.
(3) In numerous locations, the panel seal mortar placed on the box beams was not continuously in contact with the panel, since it sloughed away from the precast panel.

The problems experienced by the bridge under investigation were similar to Florida bridges even though the precast panels were not prestressed. It may be concluded that the stay in place precast panels were not performing satisfactorily.

Recommendations given by the researchers were:-

(1) The minimum thickness of the panel must be 3 in. with 1 ½ in. cover from the top and the bottom to avoid reflective cracking in the panel due to limited concrete cover over the reinforcement mesh.
(2) Use epoxy coated mesh reinforcement to limit future bottom of deck spalls.

B.3.13 Montana Department of Transportation

According to Lee Walker, Program Specialist, Montana DOT did not use such (composite deck system) form of construction.

B.3.14 New Jersey Department of Transportation (NJDOT)

New Jersey DOT bridges use galvanized steel for stay-in-place forms. NJDOT has never used prestressed precast concrete deck panels as stay-in-place forms. The above information was provided by Al Virgilio, Supervising Engineer with NJDOT.

B.3.15 New Mexico Department of Transportation

Mr. Jimmy Camp, State Bridge Engineer with New Mexico DOT provided the following information:

(1) New Mexico mainly uses full depth concrete bridge decks.
(2) Until 1987 full depth decks was poured on false work. After 1987, full depth concrete decks on stay in place steel forms were used.
(3) Precast deck panels were not used until about 1999 except on twin bridges built in 1985 on NM 500 over the Rio Grande. One bridge has performed well, the other has not. Bridge No 8568 has a deck rating of 7. Bridge No 6224 has a deck rating of 4. The causes for the different behavior of the deck with same form of construction were not investigated.
(4) Since 2000 about 25 bridges with concrete deck panels were built. These bridges were built on DOT's Big I project and on US-70 project. Some premature concrete cracking has been observed but it is no worse than those in full depth concrete decks. The cracking has been filled with pourable epoxy crack fillers. All these decks are satisfactory for now.

The bridges with precast panels are in the early stage of use and are performing okay. It is therefore difficult to comment on the behavior of these bridges at this time.

B.3.16 Oklahoma Department of Transportation (OKDOT)

Mr. Eariquez sent a fax stating that the list of bridges (3 nos.) sent was accurate and the bridges are performing well, no cracking was observed. Thus the information sought using the NBI coding seemed to be accurate in this case.

B.3.17 Texas Department of Transportation [25]

Mr. Brian Merrill responded to the survey. Texas has 1668 such bridges on the state and county system. The majority of these bridges performed well (833 were rated as condition 8, 20 were rated as condition 9). Only 1 bridge was rated 5.

The bridges in Texas experienced cracking similar to those in Florida but had few cases of spalling. They believe that the key to their success was strict adherence to construction standards. Mr. Merrill noted "*It is essential that the concrete paste have sufficient room to flow under the panel edges to provide the final support for live load. Failure to do this will lead to excessive cracking and spalling*".

Table B.12 lists information on composite slab details used by Texas obtained from Mr. Merrill's publication [2] available at the Texas DOT web site. Composite slab details for a typical span of 8.7 ft used by Texas DOT are shown.

Table B.12 Composite Slab Details used by Texas DOT

Slab	Description	Value
Cast In Place	Thickness	4 in.
	f'c	4000 psi
	Transverse reinforcement (Grade 60)	Top layer - #5@6 in. O.C. Cover – 2 in. Bottom layer - #5@6 in. O.C. Cover – 1.25 in.
	Longitudinal reinforcement (Grade 60)	Top layer - #4@9 in. O.C. Bottom layer - #5@9 in. O.C.
Precast Panel	Thickness	4 in.
	f'c	5000 psi
	Prestressing strands	3/8 in. dia 270 ksi 6 in. O.C.
	Prestressing force	16.1 kips
	Tension limit	$6\sqrt{f'c}$
	Final stress	144 ksi

Texas DOT Design Specifications

(1) Panels at end of spans must have #3 bars extending into CIP portion
(2) Panels to be supported at least 1/4 in. above the girder so that mortar can flow under the panels to provide bearing to live loads.

(3) Polystyrene foam (Dow PL 300 Glue) used instead of fiberboard, available up to 4 in. thick.
(4) Panel overhangs bearing by 1 ½ in. minimum.

Texas prohibits the use of deck panel bridges for certain applications:

(1) *Curved steel girder bridges*: Texas DOT's Bridge Design Engineer prefers to have a monolithic deck on these units because of the complicated interaction between the deck, the curved girders, and the diaphragms.
(2) *Bridge widening*: Deck panels are not allowed in the bay adjacent to the existing structure because it is usually not possible to set the panels properly on the existing structure. It can be used on the other girders when the widening involves multiple girders.
(3) *Phased construction*: Deck panels are not often allowed in the bay adjacent to the previously placed deck because it is difficult to install a header form that leaves enough room for the panels to be set properly on the girders from the earlier stages.
(4) *Steel girders with narrow flanges:* Girders with flanges less than 12 inches wide make deck panel use difficult because the shear studs conflict with the panels. Standard details allow shear studs to be skewed across the flange width to facilitate the use of panels where sufficient flange width is available.

B.4 Summary

Fourteen DOT's and two FDOT districts, responded to the survey conducted. The information provided in the NBI database does not identify bridges with prestressed precast panels overlaid with cast in place concrete. Each state has its own way of coding such bridges that is not reflected in the NBI database. Often precast deck bridges are coded as stay in place precast panels overlaid by cast-in-place concrete. In some cases it seems to be correct as in case of Oklahoma DOT. In general, it is difficult to locate prestressed precast deck panels from the NBI database.

Stay in place steel forms are preferred instead of precast panels (Georgia DOT, Michigan DOT, and New Jersey DOT). Out of the DOT's who responded, none had investigated the cause of deficiencies (if present) excepting Michigan DOT. In case of Michigan, the precast panels used were not prestressed. The composite deck details used are different from Florida (Georgia, Iowa, Kansas, Texas). Panels overhang the bearing material to allow mortar to flow beneath the panel and act as a positive bearing to effectively transfer the shear to the girder below. In two cases (Kansas DOT and Iowa DOT) panels are used only when the supporting girders are prestressed. This ensures more rigidity to the deck in the transverse direction and as a result, less cracking is observed. Texas is the most prolific user of the deck panel system. Though longitudinal and transverse cracking were observed, the bridges have performed well with only 1 out 1668 bridges having a condition rating of 5. Almost 85 % of Texas bridges use panel deck construction.

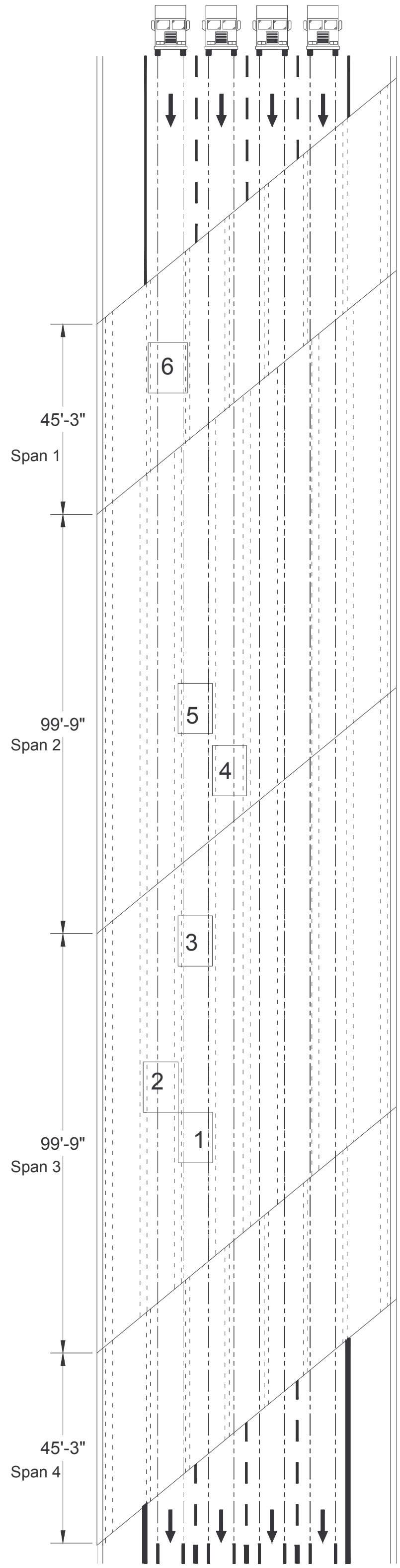
Thus, it can be concluded that the composite deck system when used is performing satisfactorily but nonetheless are avoided (except in Texas, where it's usage is encouraged). In some DOT's metal stay in place forms are preferred. Positive bearing and panel overhangs are found to lead to good performance of the bridge.

References

[1] Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridge, Report No. FHWA PD-96-001, U.S. Department of Transportation, Federal Highway Administration.

[2] Merill, B.D., "Texas Use of Precast Concrete Stay in Place Forms for Bridge Decks", Texas DOT, Bridge Conference, 2002, pp.1-19.

APPENDIX C
I-75 over Moccasin Wallow Rd
Core Evaluation



Deck sections # 2 and # 6 were rejected for the study due to heavy damage incurred during the removal process

Figure C1 I-75NB over Moccasin Wallow Bridge Core Locations

Table C.1 Core Details of Deck Section # 1

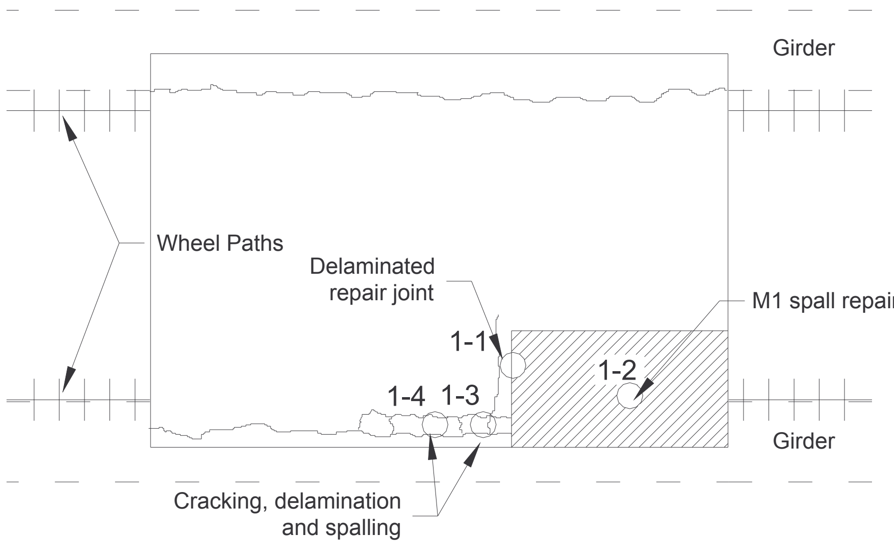
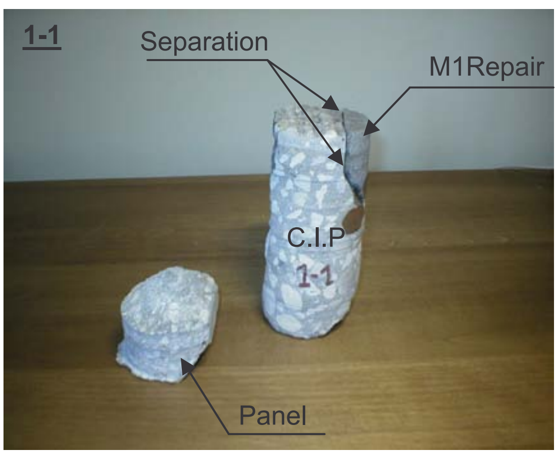
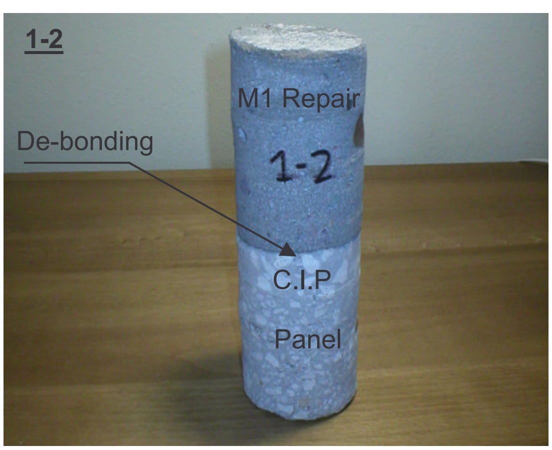
<table>
<tr><th colspan="2">Core Details of Deck Section # 1</th></tr>
<tr><td colspan="2"></td></tr>
<tr><th>CORE</th><th>DESCRIPTION</th></tr>
<tr><td></td><td>This section intercepts a crack and and an M1 repair. It is located about 1 ft from the supported edge as shown in the sketch. The core was extracted as two pieces with the panel completely separated from the cast in place slab. The M1 repair was completely debonded from the cast-in-place slab. Signs of water going thru the interface of the M1 repair and signs of rebar corrosion were also present.</td></tr>
<tr><td></td><td>This core was taken from the M1 repair section as indicated in the sketch above. The core was extracted in two pieces with the M1 repair completely debonded at its interface with the cast in place concrete. The total thickness was 7 5/8 in with the M1 repair being 3 5/8 in, the cast-in-place slab 1 in and the prestressed panel 3 in.</td></tr>
</table>

Table C.1 (continued) Core Details of Deck Section # 1

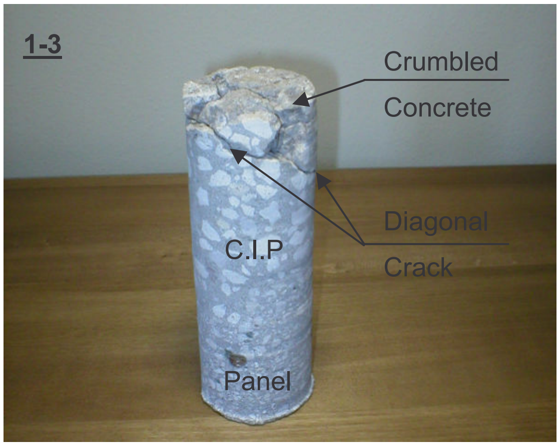
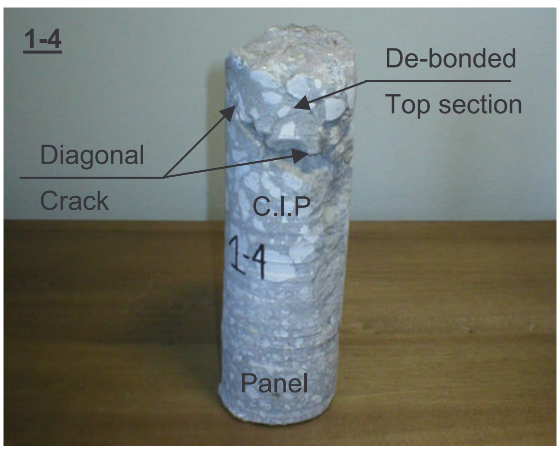
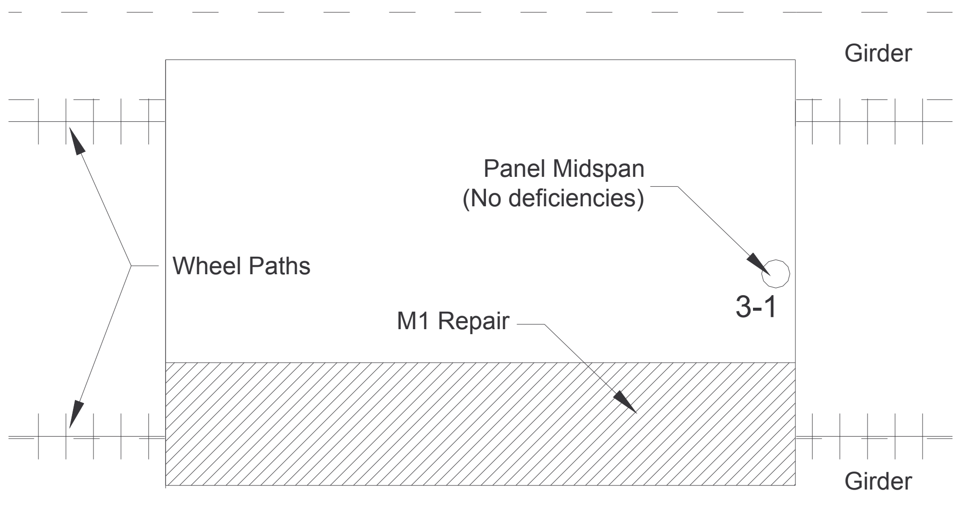
Core Details of Deck Section # 1	
	This core was taken between two parallel cracks close to the edge of the panel support. There was no debonding at the interface between the CIP slab and the panel. However, there was diagonal separation at the top (1/2 in at one end to 2 in at the other end). The concrete in this section was in four small pieces and signs of water infiltrating the crumbled concrete were present. Total core thickness 7 5/8 in
	The core was adjacent to 1-3 but was closer to the support. In this case, there was also no separation at the panel/CIP interface and a diagonal crack with the same slope (1/2 in at one end and 2 in. at the other end was present). However, the top segment was cracked but not in four pieces. Signs of water infiltrating the diagonal crack were found. Total core thickness 7 5/8 in

Table C.2 Core Details of Deck Section # 3

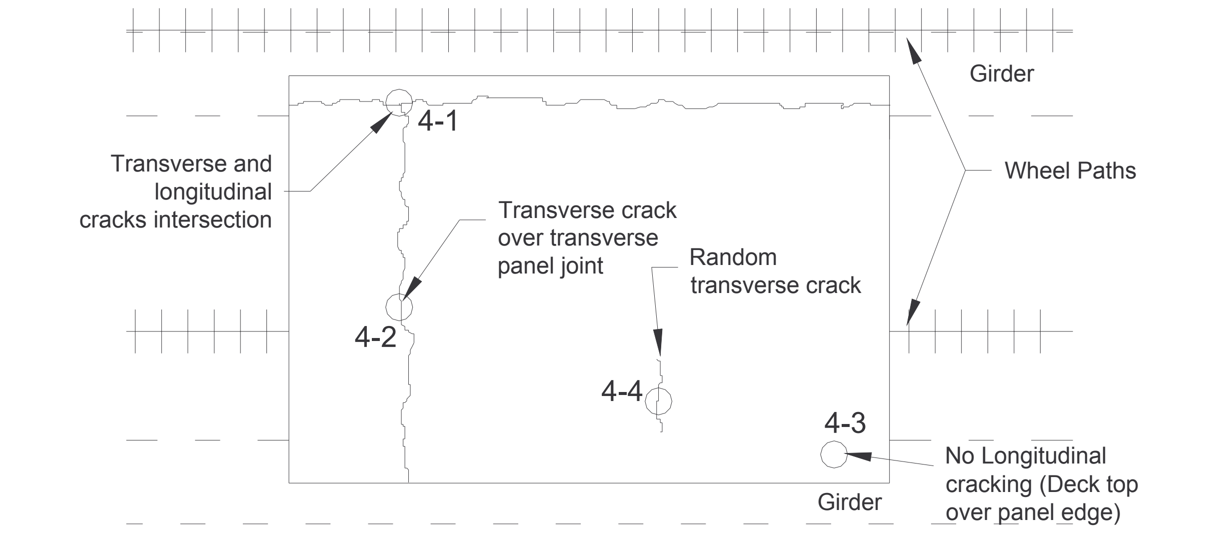
Core Details of Deck Section # 3



CORE	DESCRIPTION
	This core was taken at the middle of the panel where there was no deterioration. No deterioration was detected in this core. The bond between the CIP slab and the precast panel was excellent. Total core height 7 1/2 in

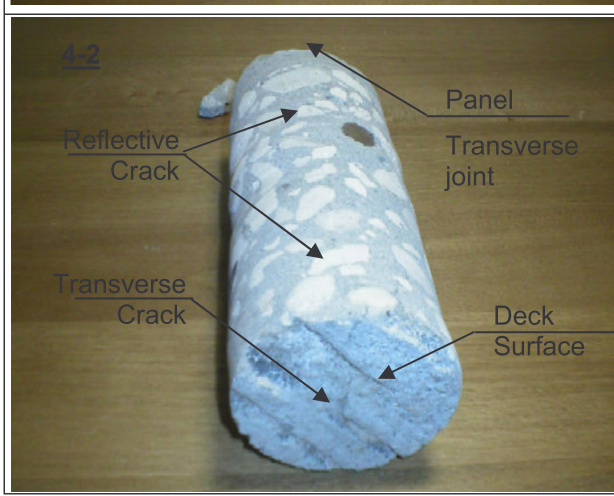
Table C.3 Core Details of Deck Section # 4

Core Details of Deck Section # 4



<table>
<tr><th>CORE</th><th>DESCRIPTION</th></tr>
<tr><td>


</td><td>
This core was taken at the intersection of a longitudinal and transverse crack as shown in the sketch above. The longitudinal crack extends all the way from the panel through the CIP slab. The transverse crack extends 2 in. below the top slab along the transverse panel joint. Despite the cracking, the concrete between the cracks was not in small pieces and there were no signs of spalling or delamination of the deck surface.

Total core height is 8 in
</td></tr>
<tr><td>


</td><td>
This core was taken over a transverse joint. A hairline crack extended all the way from the top surface to the transverse panel joint

The bottom part of the core (panel joint), was damaged during the extraction process.
</td></tr>
</table>

Table C.3 (continued) Core Details of Deck Section # 4

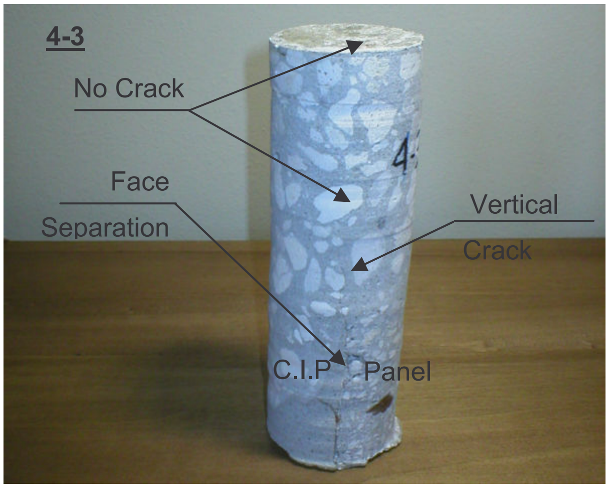
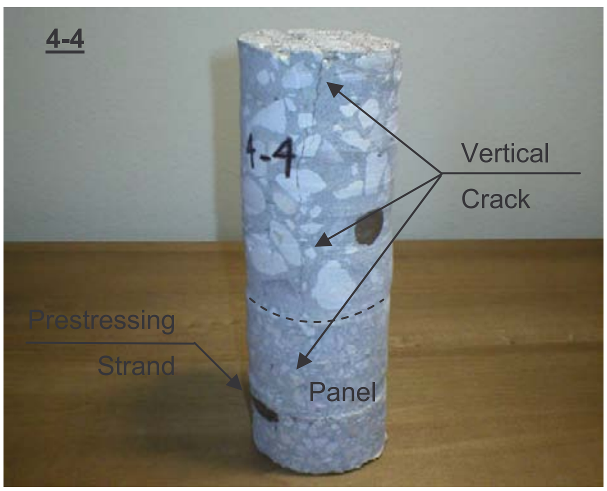
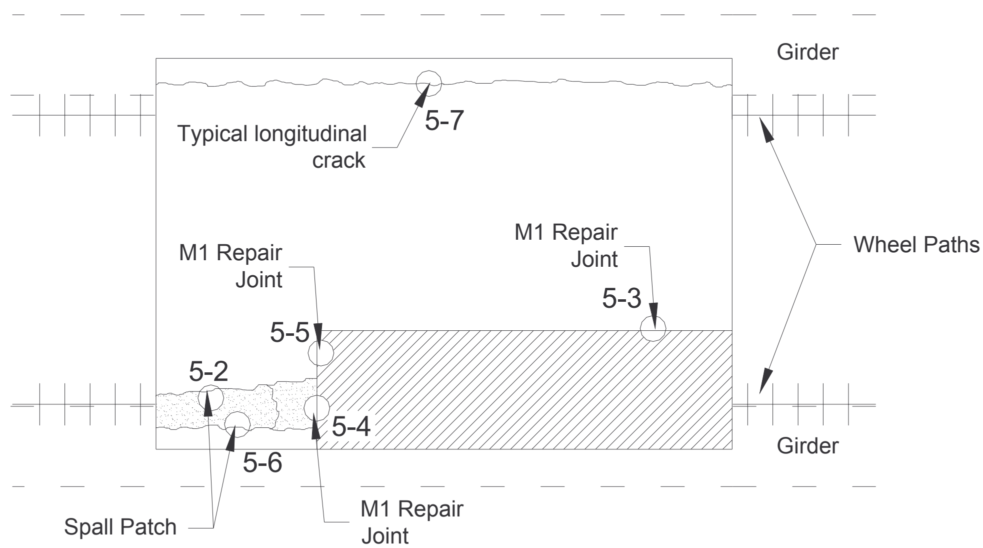
Core Details of Deck Section # 4	
	This core was taken over the edge of the panel where there was no surface cracking. There is separation between the vertical face of the panel and the CIP slab possibly due to creep and shrinkage. There is also a hairline vertical crack emanating from the corner that extends 1 ½ in upwards into the CIP slab. Total core height 8 1/2 in
	This core was taken at a section near the edge of the panel where there was a transverse crack. The crack extended all the way from the prestressing strand to the steel surface. The surface crack was an isolated crack with a length of less than 2 ft. Total core height 8 1/2 in

Table C.4 Core Details of Deck Section # 5

Core Details of Deck Section # 5



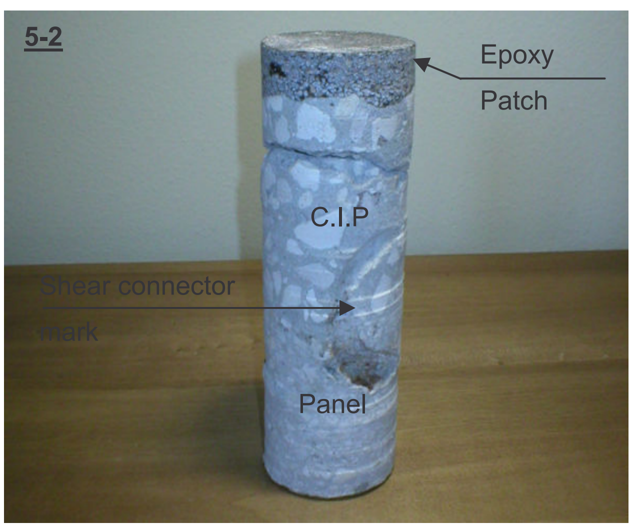
CORE	DESCRIPTION
<u>5-1</u>	This core was rejected due to heavy damage incurred during extraction.
	This core was taken from an epoxy repaired region between near two parallel cracks where it intercepted one of them. There was excellent bonding between the epoxy material and the CIP slab. The core was broken 2 in. from the top during the extraction process. This core has the mark of a shear connector embedded between the panel and the cast in place concrete. Total core height 8 in.

Table C.4 (continued) Core Details of Deck Section # 5

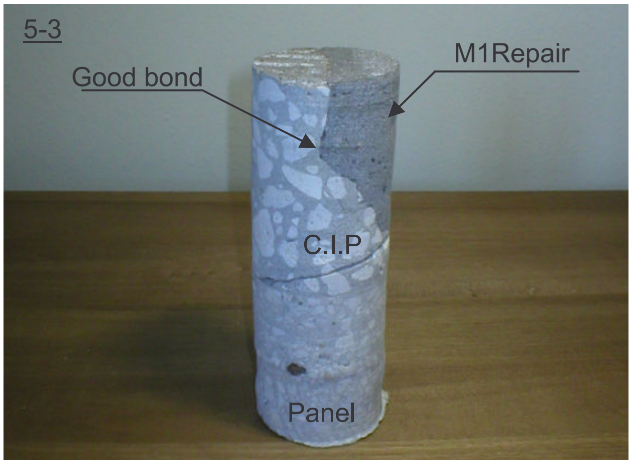
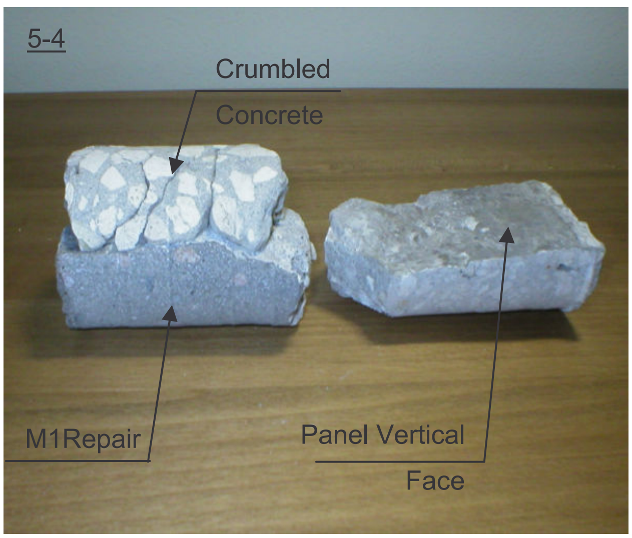
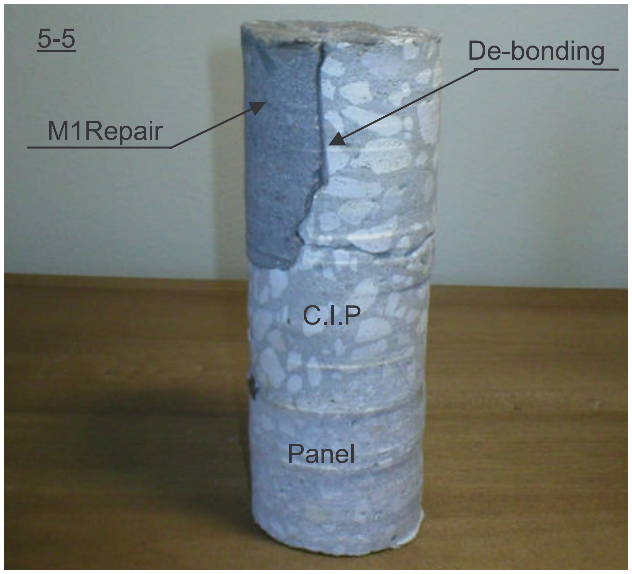
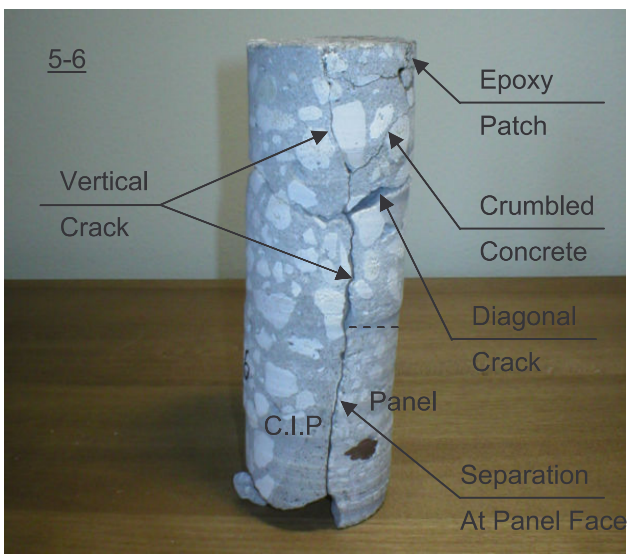
Core Details of Deck Section # 5	
	This core was taken from the edge of an M1 repair. There was good bond between the M1 repair and existing concrete. This core was also broken in half during the extraction process. Total core height 7 ¾ in.
	This core was taken at a transverse joint for an M1 repair. There was no bond between the M1 repair and the existing concrete. The concrete adjacent to the vertical repair joint was crumbled, and had signs of water infiltration. The panel vertical face easily separated from the adjacent cast in place concrete. Not all the pieces of the core could be retrieved.

Table C.4 (continued) Core Details of Deck Section # 5

<table>
<tr><th colspan="2">Core Details of Deck Section # 5</th></tr>
<tr><td>
</td><td>This core was taken adjacent to 5-4 but some distance away from the edge. It shows de-bonding between the M1 repair and the adjacent cast in place concrete.

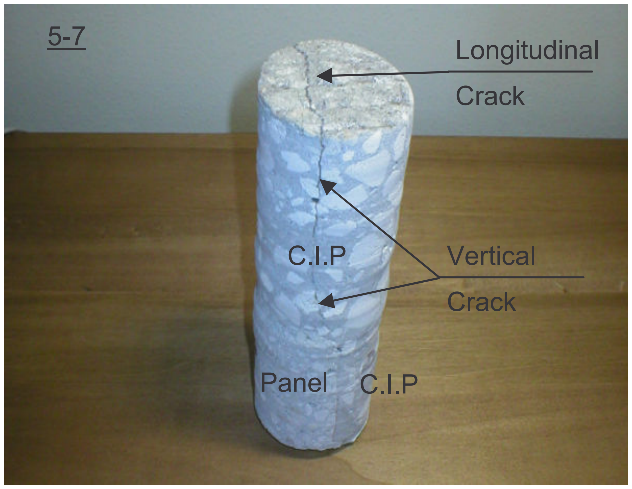
This core broke at the vertical edge of the M1 repair during the extraction process.

Total core height 8 in.</td></tr>
<tr><td>
</td><td>This core was taken adjacent to 5-2 near the edge of the panel where it crossed a longitudinal crack. It shows a vertical reflective crack extending over the entire depth of the core separating the precast panel from the CIP slab. There are signs of water and dust infiltration.

The top surface includes a partial epoxy patch which is bonded to the CIP slab. The penetration of the epoxy penetrating below has prevented the top surface from crumbling.

Total core height 8 in.</td></tr>
</table>

Table C.4 (continued) Core Details of Deck Section # 5

<table>
<tr><th colspan="2">Core Details of Deck Section # 5</th></tr>
<tr><td>
</td><td>This core was taken along a longitudinal crack located at the opposite supported edge of the panel from the previous cores. The same vertical crack detected in core 5-6 occurred but there was no additional damage.

There were signs of water penetration in the CIP slab.

Total core height 8 1/4 in.</td></tr>
</table>

APPENDIX D

Digital Image Bridge Deck Inspection

- Pilot Study -

D.1 Introduction

This appendix describes a pilot study conducted to determine the feasibility of carrying out bridge inspection using a camera mounted on a vehicle.

D.2 Current Deck Inspection Methods Used by FDOT

FDOT conducts bridge inspection by “Human Observation”. In this method, a bridge inspector walks around the deck, locating and classifying all the different deficiencies he can notice. This task is performed without any lane closures.

This method is labor intensive and the results in many cases may be inaccurate. Moreover, the information is inconsistent because different inspectors rate the severity of deficiencies differently since it is subjective. For example, deck cracking is loosely defined and the severity of a crack depends on lighting. In fact, under direct overhead sun light, some cracks are barely visible.

Figure D1 Current Bridge Inspection Method

Also, this method is very hazardous to the survey crew, since they have to walk alongside fast moving traffic (Fig. D1).

The findings from the inspection are included in the inspection report where the condition of the deck is summarized in a few lines. In case there are major deficiencies, photographs are taken along with sketches showing their location.

D.3 Digital Video as an Alternative Inspection Method

In this method data is digitally collected and subsequently processed. It consists of (1) data collection and (2) post-processing of data.

D.3.1 Data Collection

Data collection is performed using a high speed digital video camera mounted on the back of a truck (Fig. D2). The truck is equipped with a camera trigger mechanism, a roadway illumination system, a GPS and a laptop computer.

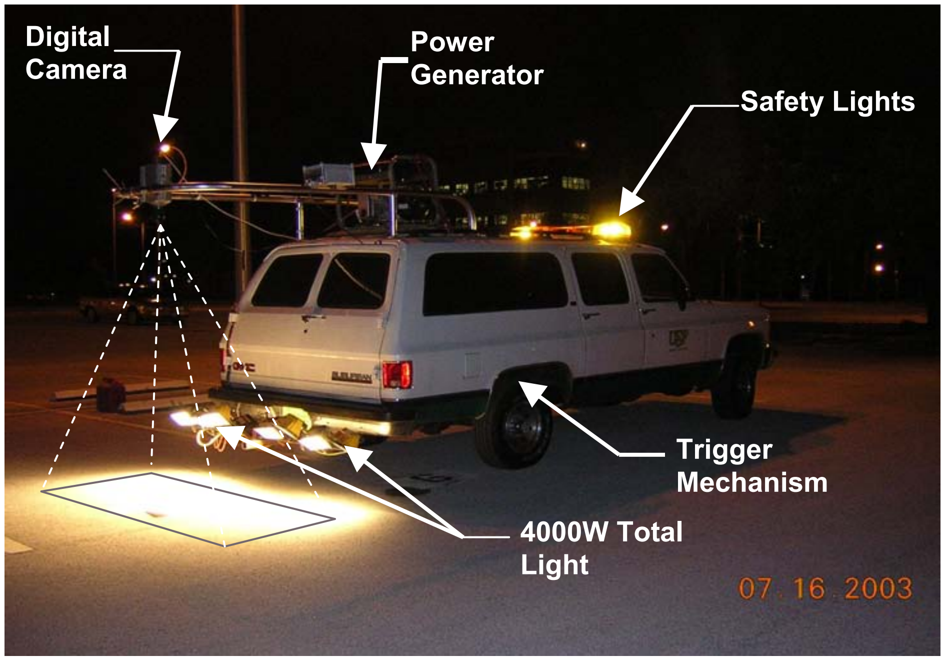


Figure D2 Digital Image Bridge Deck Inspection Vehicle

D.3.1.1 High Speed Camera

The camera selected for the pilot study was the "Phantom v5.1" model (Fig. D3) manufactured by Vision Research Inc. (www.visiblesolutions.com). The following are relevant specifications for this camera:

- 10 bit SR-CMOS 1024x1024 pixel sensor, color or monochrome.
- Trigger: Continuously, variable, pre/post.
- Sensitivity 400 ISO/ASA color.
- Up to 1000 pictures per second.
- Minimum exposure time 5 micro seconds.
- Internal Image memory.
  - Standard: 1024 Megabytes (Records 2000 images)
  - Optional: 2048 Megabytes (Records 4000 images)
  - Optional: Non volatile Flash Memory up to 4000 Megabytes.
- Power: 24 VDC and 1.5 Amp

Figure D3 Phantom v5.1 High Speed Digital Video Camera

This camera was selected because it had a flexible trigger mechanism (D.3.1.2), its relatively high resolution (1024 x 1024) (regular camcorder is 512 x 512), its digital video output for image post processing, and low exposure time that can prevent blurry images at high speed.

The output from the video camera can be a video or a series of still pictures or bitmaps. These individual pictures can be stamped with date and time, for later processing.

The camera is controlled using software that is installed in a laptop computer. This software allows control of the camera settings such as exposure time and options such as trigger setups and camera frame size. It also provides a live preview of the image captured by the camera (Fig. D4).

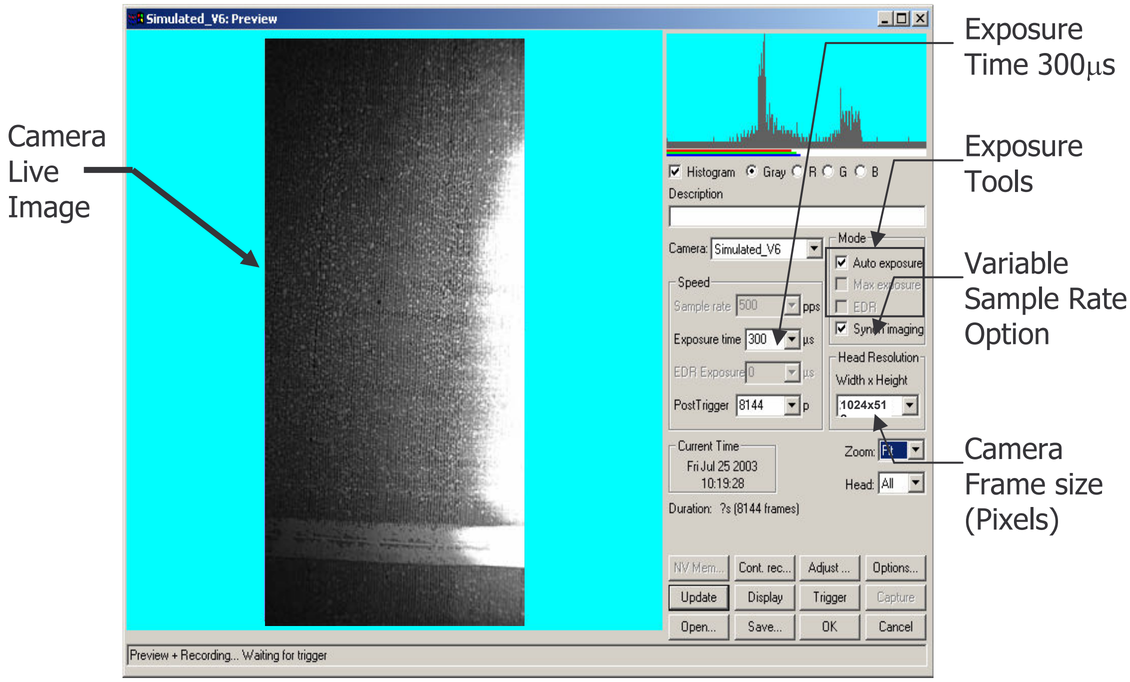


Figure D4 Phantom v5.1 High Speed Digital Video Camera

Based on the camera resolution and minimum crack size to be detected, it was decided to locate the camera 7 ft above the road surface (Fig. D4). With this camera configuration, the image covered a 7 ft 6 in. x 3 ft 8 ¾ in. surface area. Thus, to cover an entire traffic lane it required two passes as shown in Fig. D5.

The resolution of the camera in this configuration is 2mm by 2mm of road per pixel. This means that in theory the image will be clear enough to recognize cracks of at least 2 mm width (see D.3.1.6 for actual results).

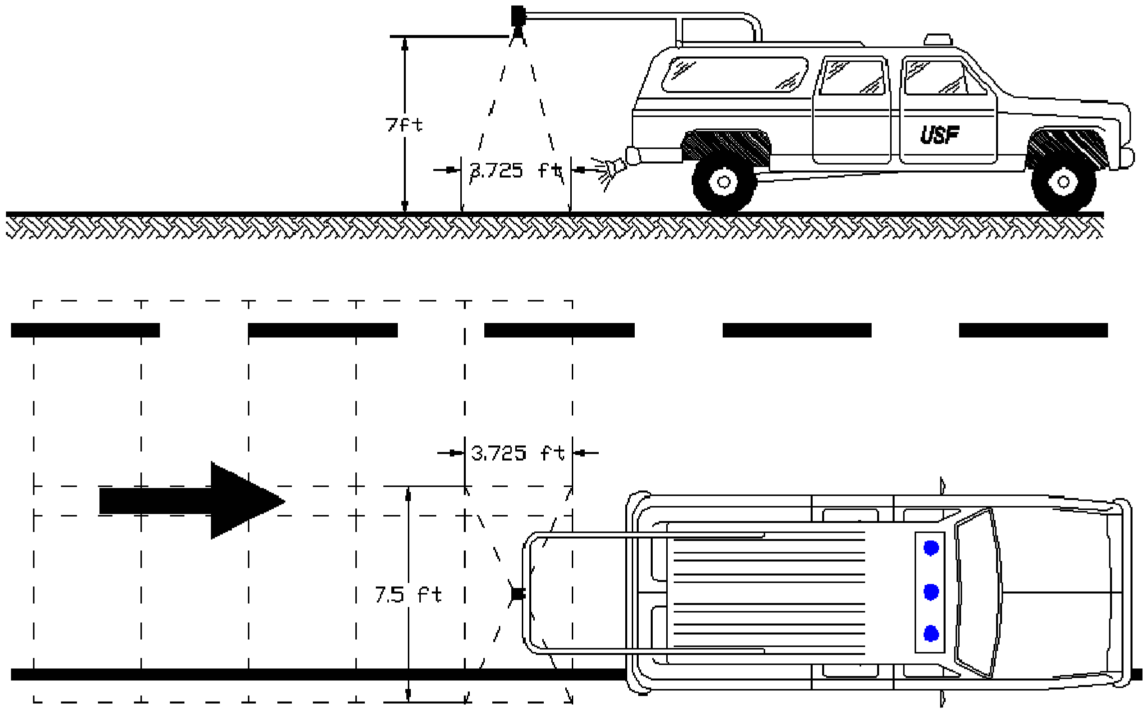


Figure D5 Camera Configuration

D.3.1.2 Camera Trigger Mechanism

The different trigger options for this specific camera are:

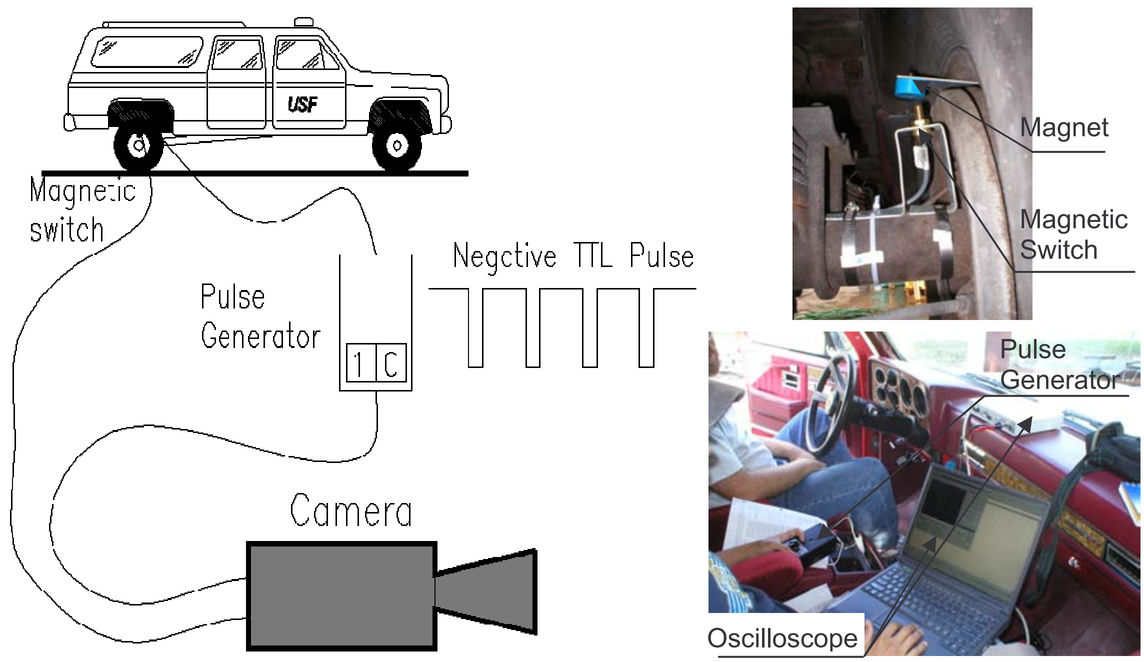
- **Continuous**: The camera can continuously record images till the internal memory is full.
- **Variable**: The camera collects just one image after the trigger event.
- **Pre-trigger**: The camera recalls images collected during a given period of time before the trigger event.
- **Post-trigger**: The camera starts collecting images during a given period of time after the trigger event.
- **Available trigger inputs**: TTL Pulse, or switch closure.

In this study, a variable trigger was used, activated by a TTL pulse (Square wave). The switch closure input was also tested, but better results were obtained with the TTL pulse.

The TTL pulse was generated using a pulse generator designed and constructed by the USF research team. This device generated a negative square wave every time the magnetic switch attached to the wheel closed the circuit (Fig. D.6)

The magnetic switch used was capable of operating in an aggressive environment (water, high temperatures, mud, and vibrations). It could operate at switching frequencies of up to 30 Hz, a switching voltage of 24 V or higher, and a switching current of 1.5A.

The trigger mechanism also included a manual switch in order to only activate the camera over the bridges to be inspected and remain shut otherwise.



System Diagram *Test*

Figure D6 Trigger Mechanism

For the vehicle selected, each wheel rotation was equivalent to 7 ft 4 in. of truck movement. Based on the camera resolution, each picture covered 3 ft 8 ¾ in. of road in the traveling direction. This meant that each picture had to be taken every half turn (180 deg rotation) of the wheel. To achieve this, two magnets were installed at two opposite quadrants of the wheel. In camera and trigger configuration, the final picture had a 3/8 in. overlap at each end which can be easily factored in during data processing.

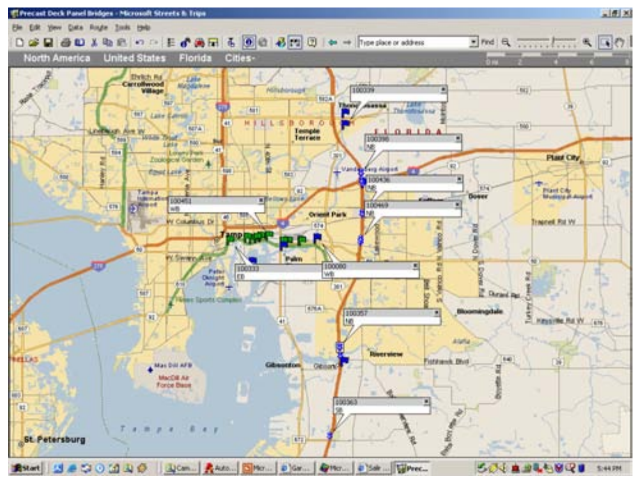
This trigger configuration assured that one picture could be obtained every 3 ft 8 in. of travel regardless of the speed of the truck. The maximum speed of the system was set as 70 mph to ensure no traffic disruptions on the highways.

D.3.1.3 GPS

A portable 12-channel Garmin GPS unit (Fig. D7) was incorporated in the system because of two reasons: (1) to know exactly when the inspection vehicle was approaching a bridge to be inspected, and (2) to know the exact location of each picture taken by the camera.

The location of each bridge to be inspected was entered in GIS software. For more complete information, every bridge location included a bridge number, traffic direction (NB/SB) and the District. The GPS unit was connected directly to the GIS software (Fig. D7) and it could show the exact location of the truck in the map where all the bridges were previously marked.

A) GPS Unit



B) GIS Software.

Figure D7 Bridge Location System.

To know the exact location of each picture taken, attempts were made to connect the GPS unit directly to the camera, so it could stamp the GPS coordinates on the picture. But this was not possible. So it was decided to store all the GPS readings in a database along with the respective time. Also each picture was stamped with the exact time when it was taken. The post-processing software could match the picture time with the coordinates stored in the database.

D.3.1.4 Data Storage

All the images captured by the camera were stored in the camera's internal memory that had a 1024 MB storage capacity. For the configuration used it was equivalent to approximately 2000 images. After the internal camera image memory was full, this information had to be downloaded to a computer. The camera would then be ready to start collecting pictures again. This meant that after 1.3 miles of deck inspection the data had to be downloaded from the camera memory.

The images were downloaded from the camera internal memory using a camera controlling software. In our case, a laptop computer with enough hard drive capacity to store all the data collected was used.

Figure D8 Setup inside Inspection Vehicle

Since two passes were needed to cover a full lane, the amount of data collected was about 1.4 Gigabytes per mile of lane surveyed. For this specific project we had an estimated 28 miles of bridge deck lanes to survey. This was equivalent to 39.2 Gigabytes of data in total.

D.3.1.5 Roadway Illumination System

After several daytime system trials, many problems were found due to variable sun light. Problems such as overexposure or underexposure after sudden sunlight intensity changes, or the effect of shadows (caused by other vehicles, trees or even the camera supporting frame) on the image, that in most cases led to underexposure in the shaded area, and overexposure in the rest of the image.

To avoid all these problems, it was decided to run the system at nighttime and implement a roadway illumination system.

The amount of light needed by the camera is dictated by the image exposure time. The lower the exposure time, the higher the amount of light needed, and vice versa. In our case a very low exposure time was needed in order to avoid blur caused by the high speed of the vehicle. It was found that a total of 4000 Watts of light was needed to cover the 7 ft 6 in. x 3 ft 8 ¾ in. picture area. Eight – 500W halogen lights were used.

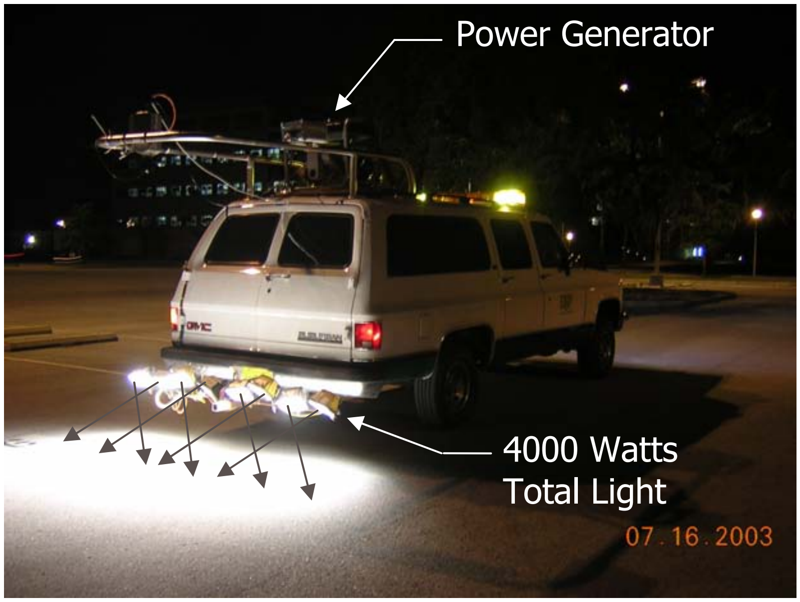


Figure D9 Roadway Illumination System.

D.3.1.6 Camera Illumination and Resolution Test.

Many tests were run on the entire video inspection system to find the best configuration (Fig. D8). The objective of this test was to find the best lighting arrangement in order to get better exposure of the cracks. It was found that it was better to locate the lights at a low angle and a cross pattern (Fig. D9) because in this case only the road surface is illuminated, not the inside of the crack, thereby creating the necessary contrast between the crack and the road surface.

In order to find the actual minimum crack size that could be detected with this camera, test images were taken over a cracked surface with known crack widths (0.5mm cracks), and also over crack calibration charts. These charts reproduced cracks from 0.1 mm up to 7 mm. Notice that in the final configuration picture (Fig. D10) it was possible to easily identify cracks as narrow as 0.5 mm.

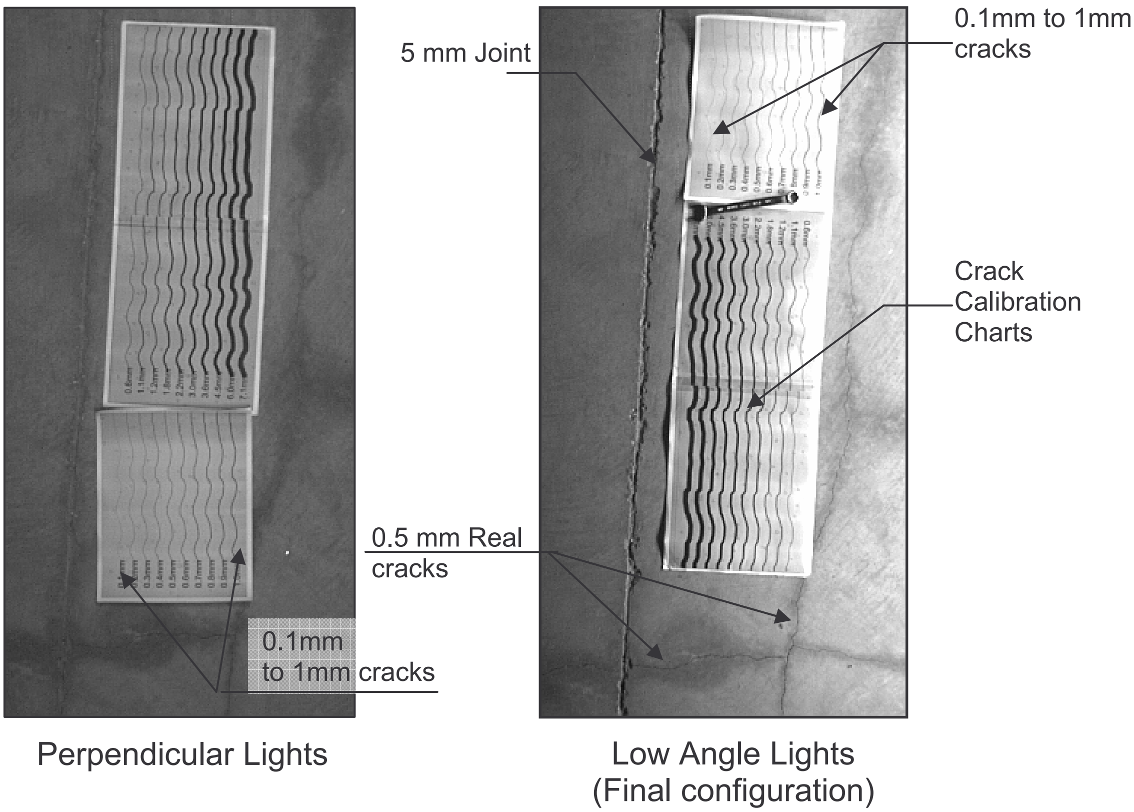


Figure D10 Camera Illumination and Resolution Test

D.3.1.7 Sample of System Results

Before starting data collection, a full inspection was carried out on one bridge to provide data that could be used for calibration. The bridge chosen for this purpose was the bridge on Gibsonton Road over I-75 bridge #100377, (See Table D.1). This bridge was selected because at that time it was considered by the FDOT as the number 1 priority for deck replacement due to multiple deck surface and underside cracking.

Table D.1 Bridge #100377

Bridge #100377 Characteristics	
FDOT District	7
Year Built	1980
Number of Spans	2
Span Lengths	168 ft
Deck Width	92 ft 9 in (Out to Out)
Lanes on Structure	4
ADT *	4,500
Percent Truck (ADTT)*	10%
Deck Condition Rating *	5 (Satisfactory)
Composite Slab Thickness	7 in.
Precast Panel Thickness	2-½ in (panel) 3-½ in (ribs)
Girder Type	Steel Girders.

* National Bridge Inventory (1999)

This inspection was conducted in July 2003 at night time. All the data collected was stored as series of ".bmp" files. A mosaic with all these individual pictures was assembled manually to create a big picture of the entire bridge deck. The assembling process was time consuming, but it was expected that this could be done automatically by the data processing software.

Fig. D11 shows the results of a section of the bridge at the end of the east bound lanes. Fig. D12 shows a close up view of Fig. D11 where a typical 0.5mm longitudinal crack was identified.

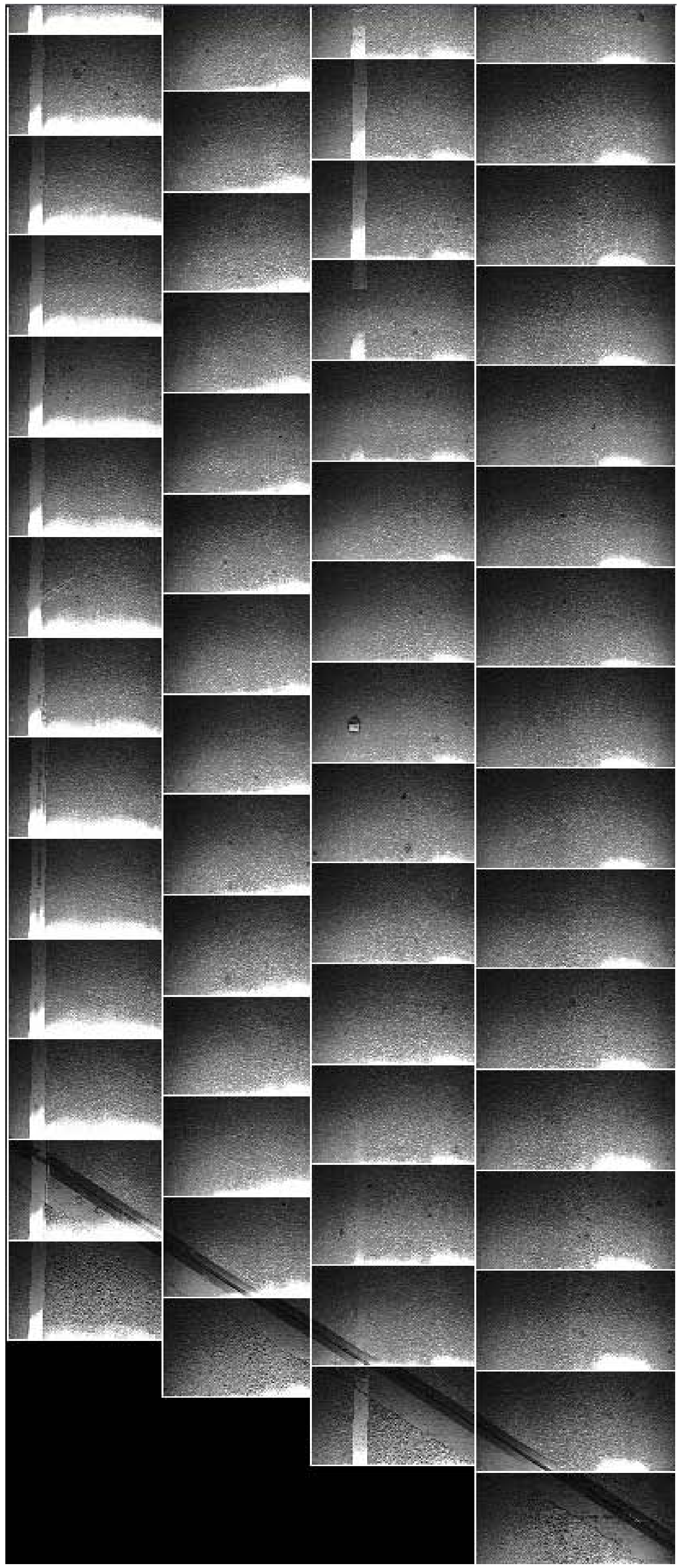


Figure D11 Results on Bridge # 100377 Gibsonton Rd EB over I-75

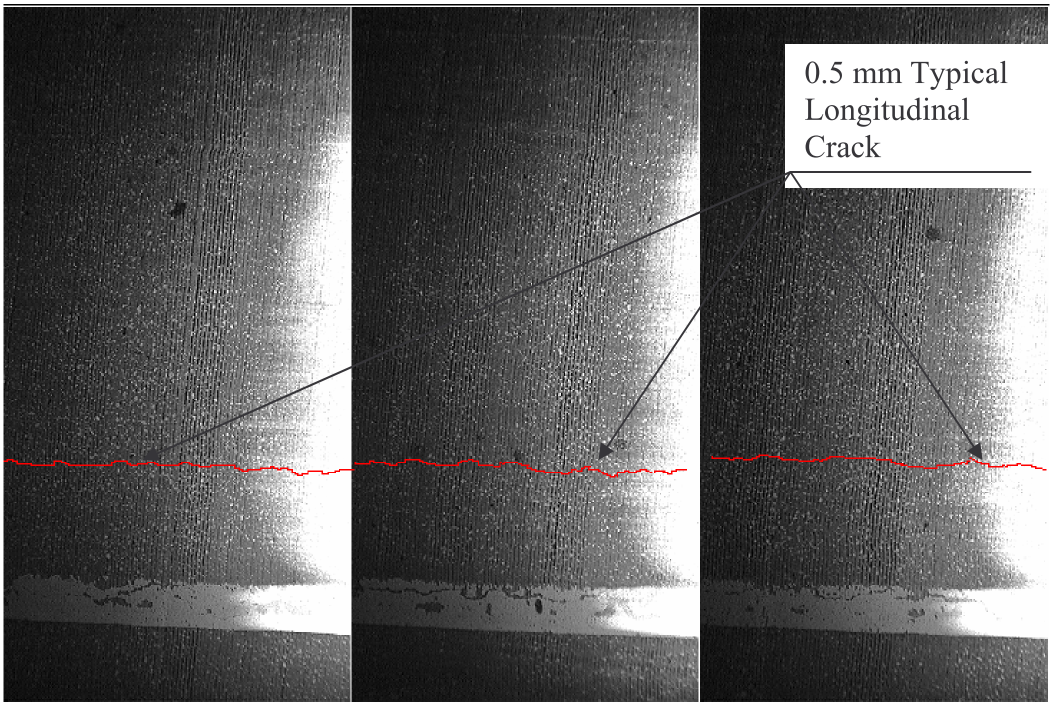


Figure D12 Longitudinal Cracking on Bridge # 100377

D.4 Data Processing

The data obtained from the data collection process was to be processed using powerful image processing software that was still in the first stages of development. The purpose of this software was to identify all possible deficiencies in the bridge decks using the images obtained with the high speed camera.

D.4.1 Software Input

The input for this software was to include the pictures from the camera in a digital format. Each picture would also include the respective date and time. In addition, GPS coordinates, bridge basic information, and bridge deck geometry in a CAD format would be provided. The bridge geometry was needed in order to locate the deficiencies identified by the software. (Fig. D13)

D.4.1.1 Software Output

The output would include: (1) deficiency details and classification, (2) amount and (3) exact location in the deck plan view, and (4) the possibility to recall the right picture for any specific location in the bridge deck - all in a digital format (Fig D13).

Figures D13 and D14 show screen shots of the tentative software layout to be used.

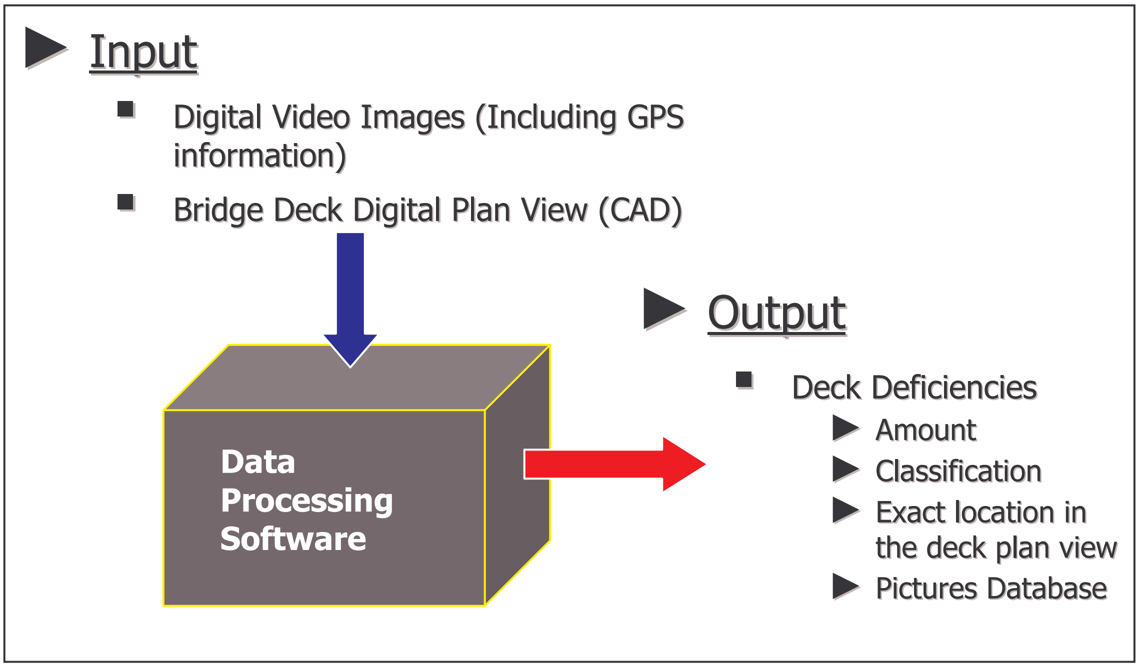


Figure D13 Data Processing Software Outline

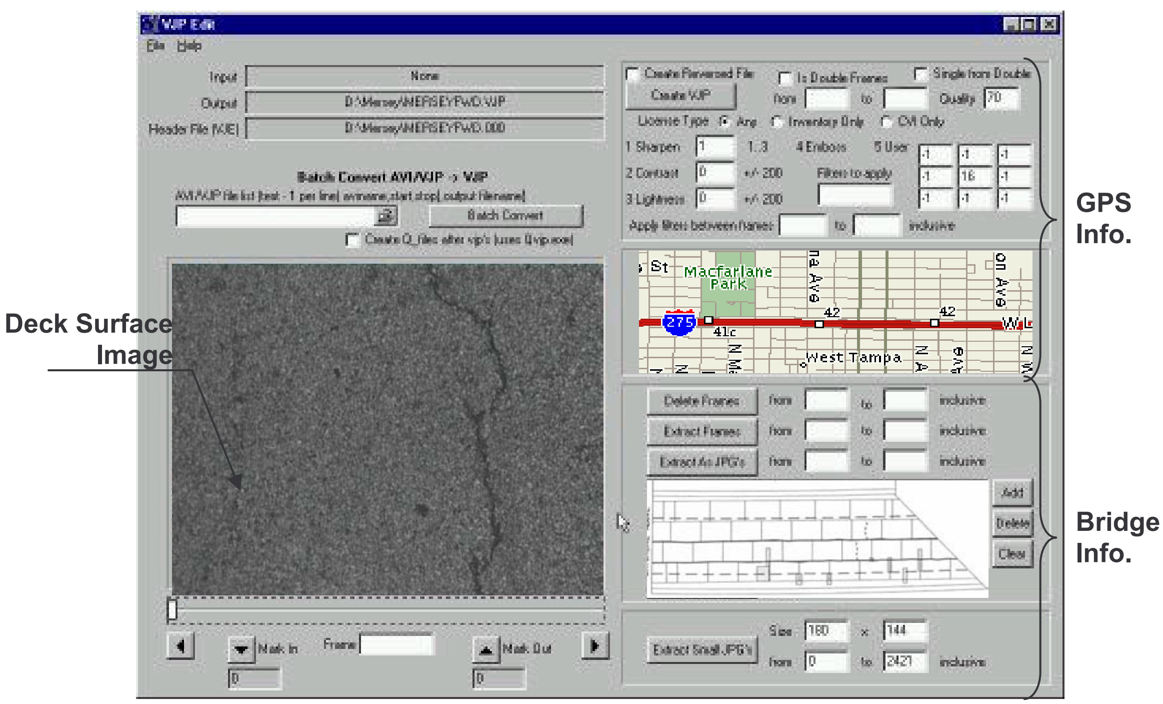


Figure D14 Tentative Software Layout

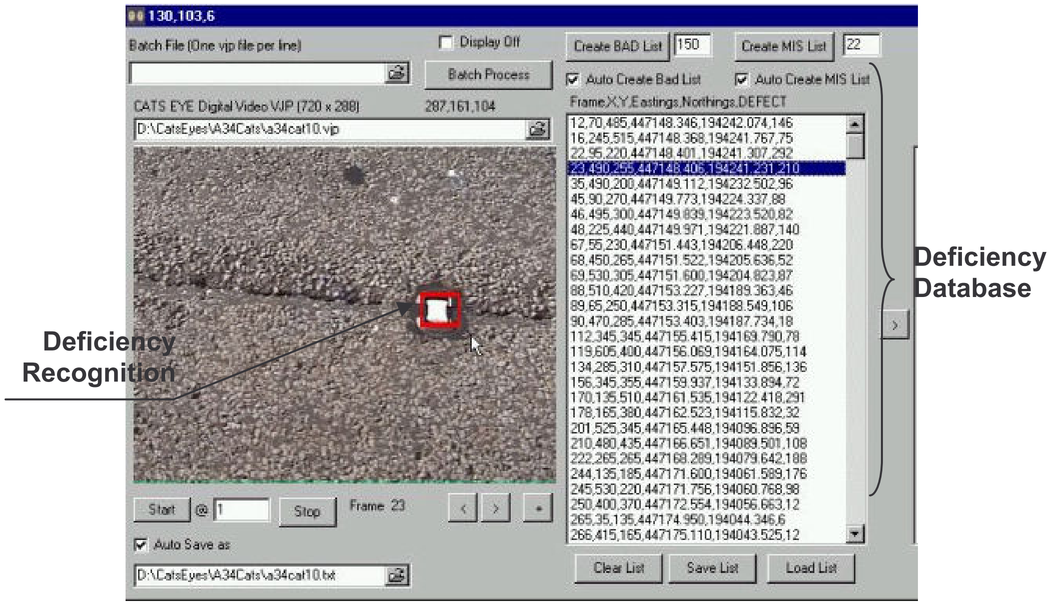


Figure D15 Tentative Layout of the Deficiency Recognition Window

D.5 System Weaknesses

Camera Performance

Even though the camera used in this pilot study had all the features needed for the study some performance problems were experienced. These were: (1) loss of data collected from the camera's internal memory, (2) Image exposure problems at different vehicle speeds and, (3) Electromagnetic noise interference of the image. When the study was conducted it was not possible to find a camera specifically designed for inspection. The camera used was designed to be used for high speed image analysis applied to ballistics, blasts, missile impacts, car safety (crash airbags), dynamics, elasticity, etc.

Image Resolution

Although the image resolution was acceptable in laboratory tests, the final resolution turned out to be much poorer in field applications due to the rough finish given to the deck surfaces. When the surface is smooth it is very easy to identify cracks, but when the surface is rough the cracks can be very hard to identify.

Data Collection

Two passes of the vehicle were required for inspecting each lane and in each pass the truck had to be driven steadily over the edge of the lane. This is not always easy on a busy interstate. Also inspecting exit or merge lanes required leaving and re-entering the interstate, i.e. additional driving. The final data collection could end up being inaccurate and time consuming.

Data Processing

The deficiency recognition system could easily confuse deficiencies with other objects on the deck surface such as oil stains, brake marks, dirt (tar, mud) and, lane marking.

D.6 Conclusions

- The idea of using a camera to record deficiencies is sound. However, there were several problems that still had to be solved in the data collection system. A lot of technical development is needed in the data processing software.

- The pilot study was stopped because of the high camera rental cost. It can be resumed after technical problems are solved and data processing software is developed.

APPENDIX E

Bridge Photographs

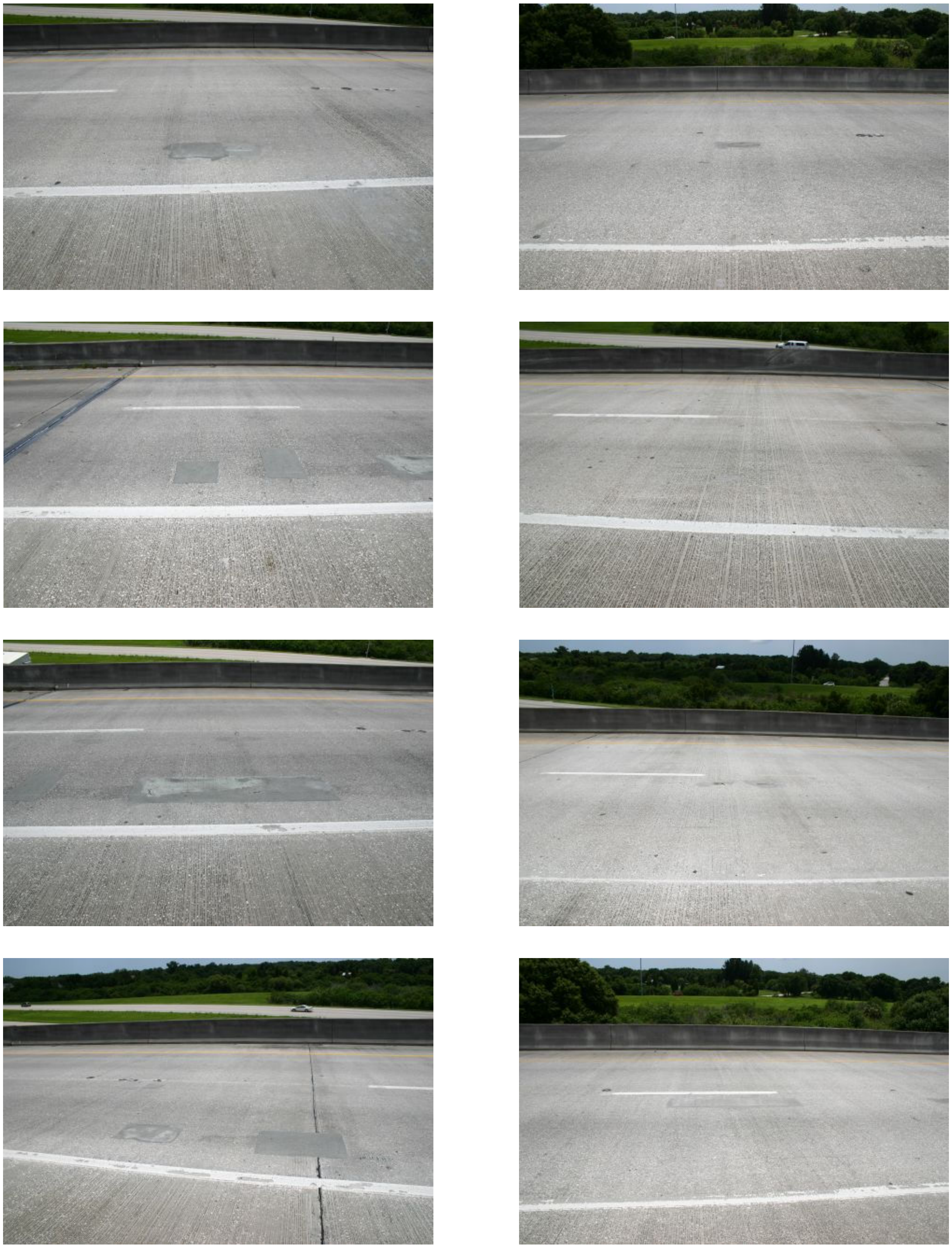


Figure E.1 District 1 Bridge # 130090, I-275 NB & I-75

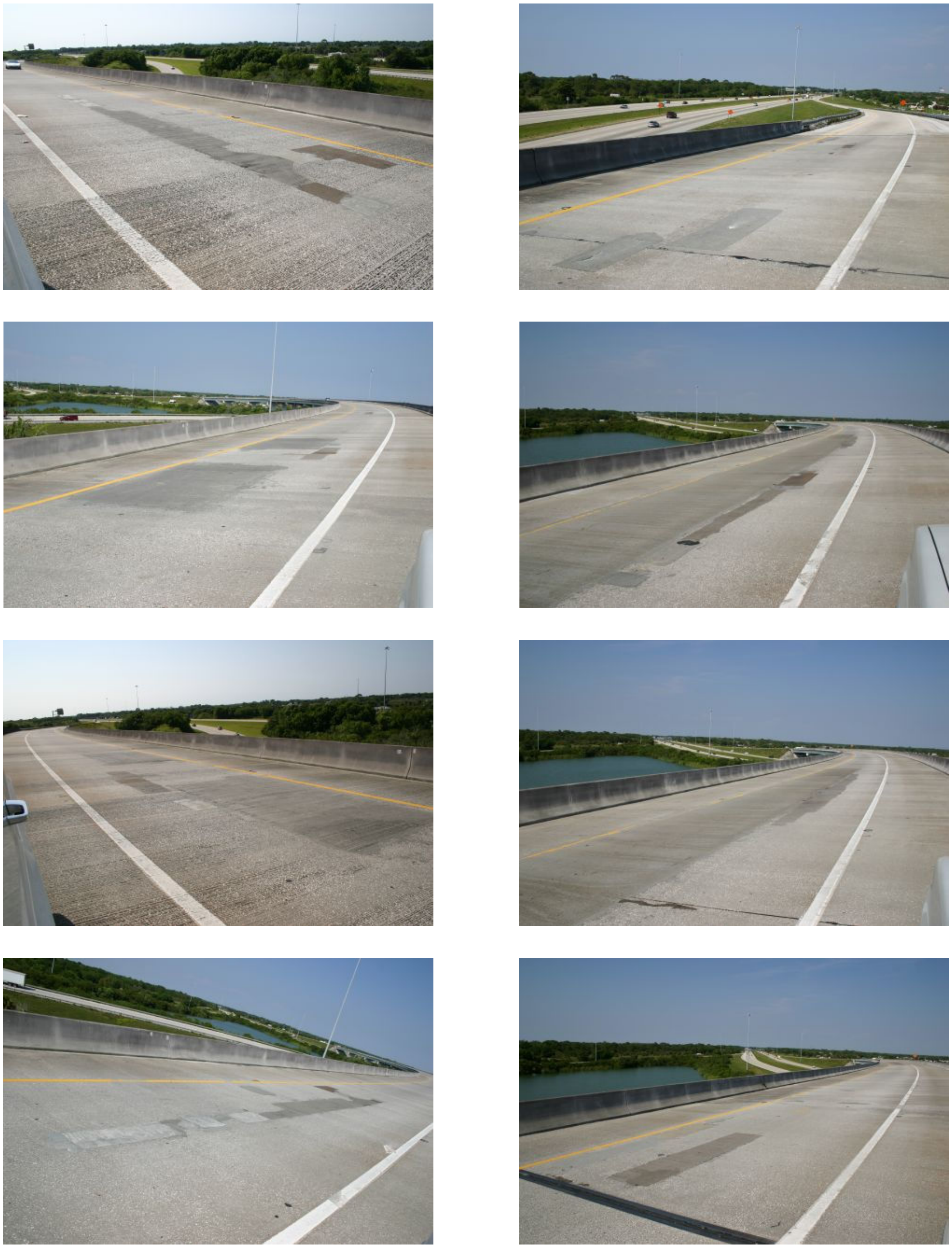


Figure E.2 District 1 Bridge # 130112, I-275 SB R to I-75 NB & I-75 and I-275 Ramps

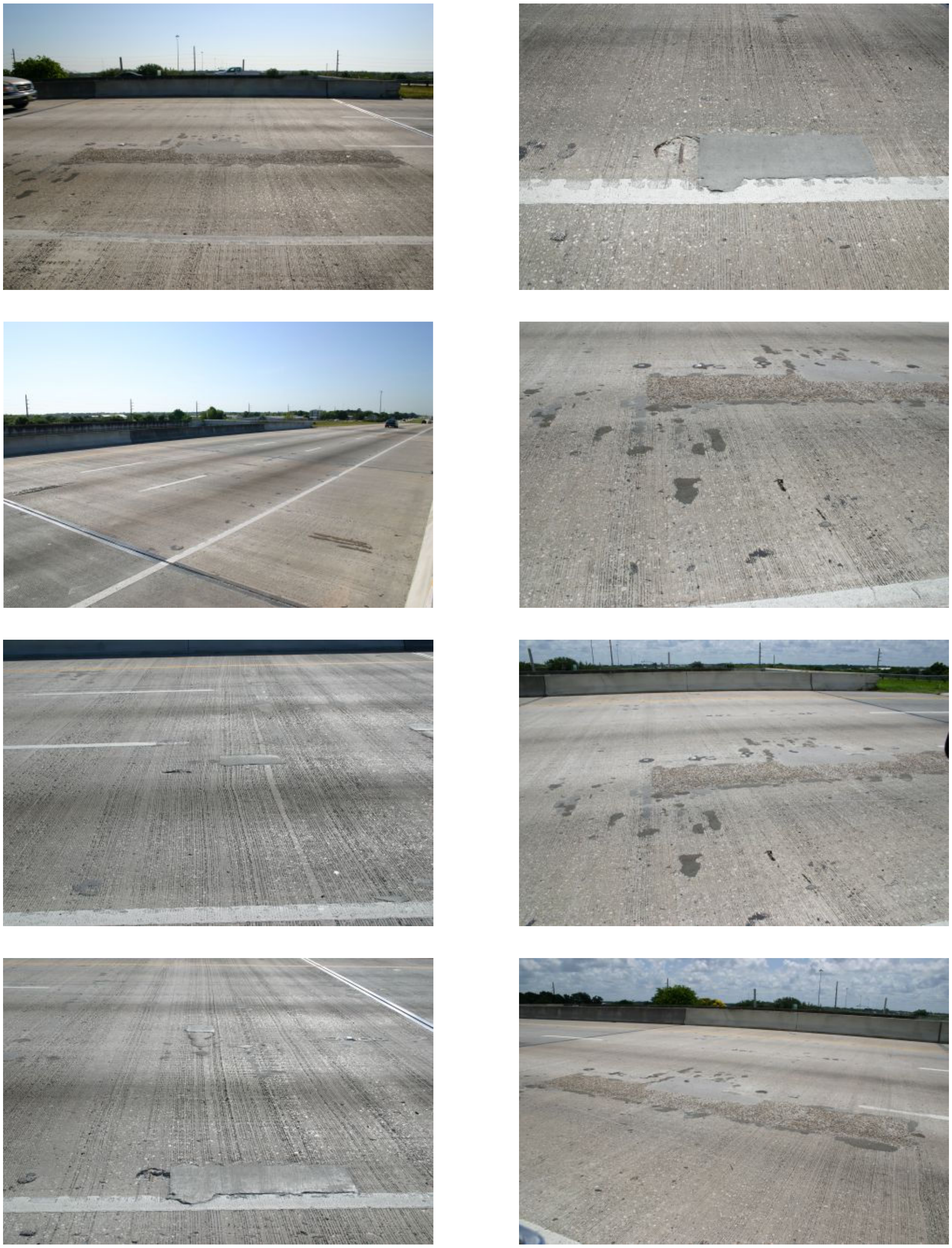


Figure E.3 District 1 Bridge # 170081, I-75 & Palmer Blvd

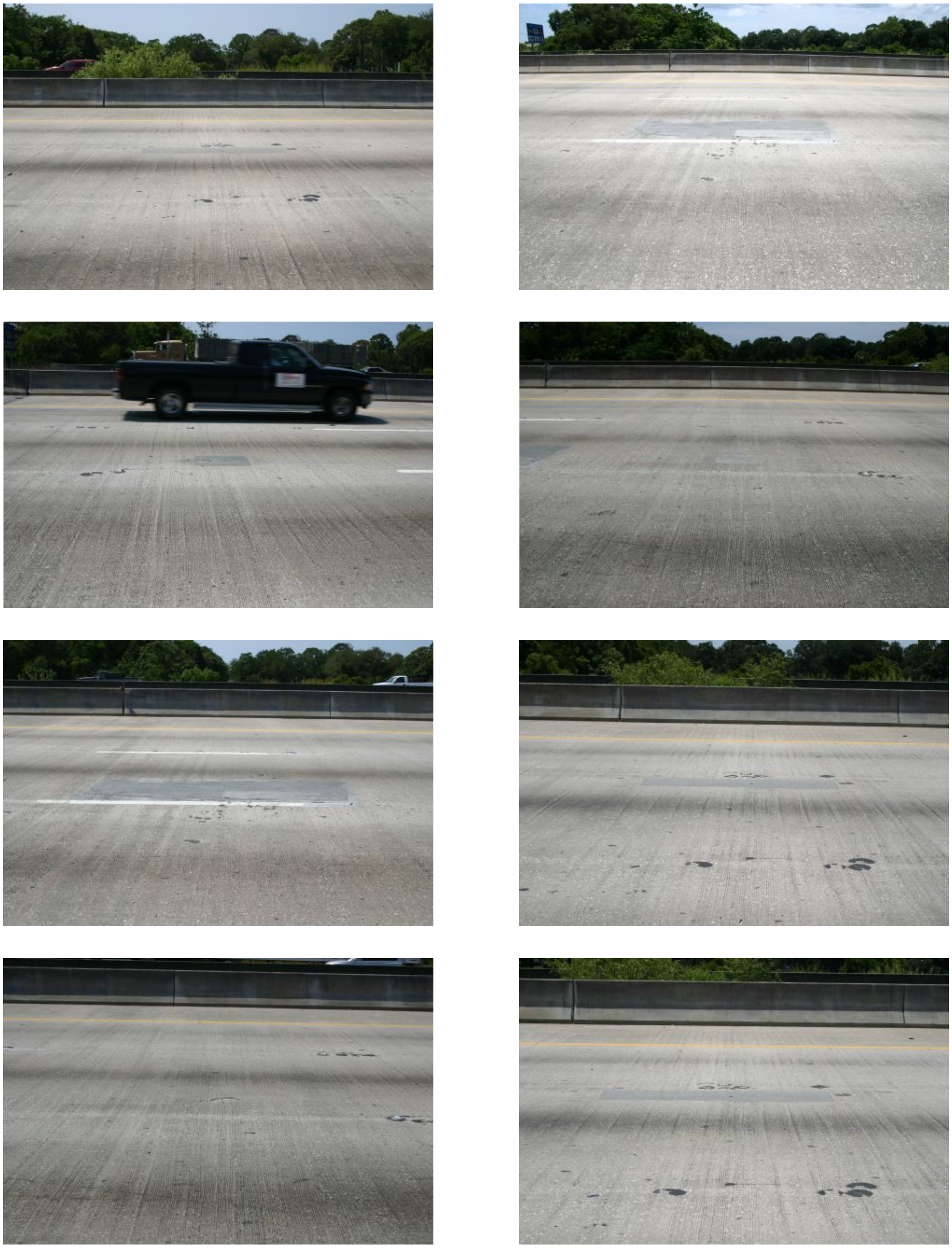


Figure E.4 District 1 Bridge # 170080, I-75 & Main A Canal

Figure E.5 District 1 Bridge # 030188, I-75/SR-93 & CR-846

Figure E.6 District 1 Bridge # 170094, I-75/SR-93 (NB) & Havana Road

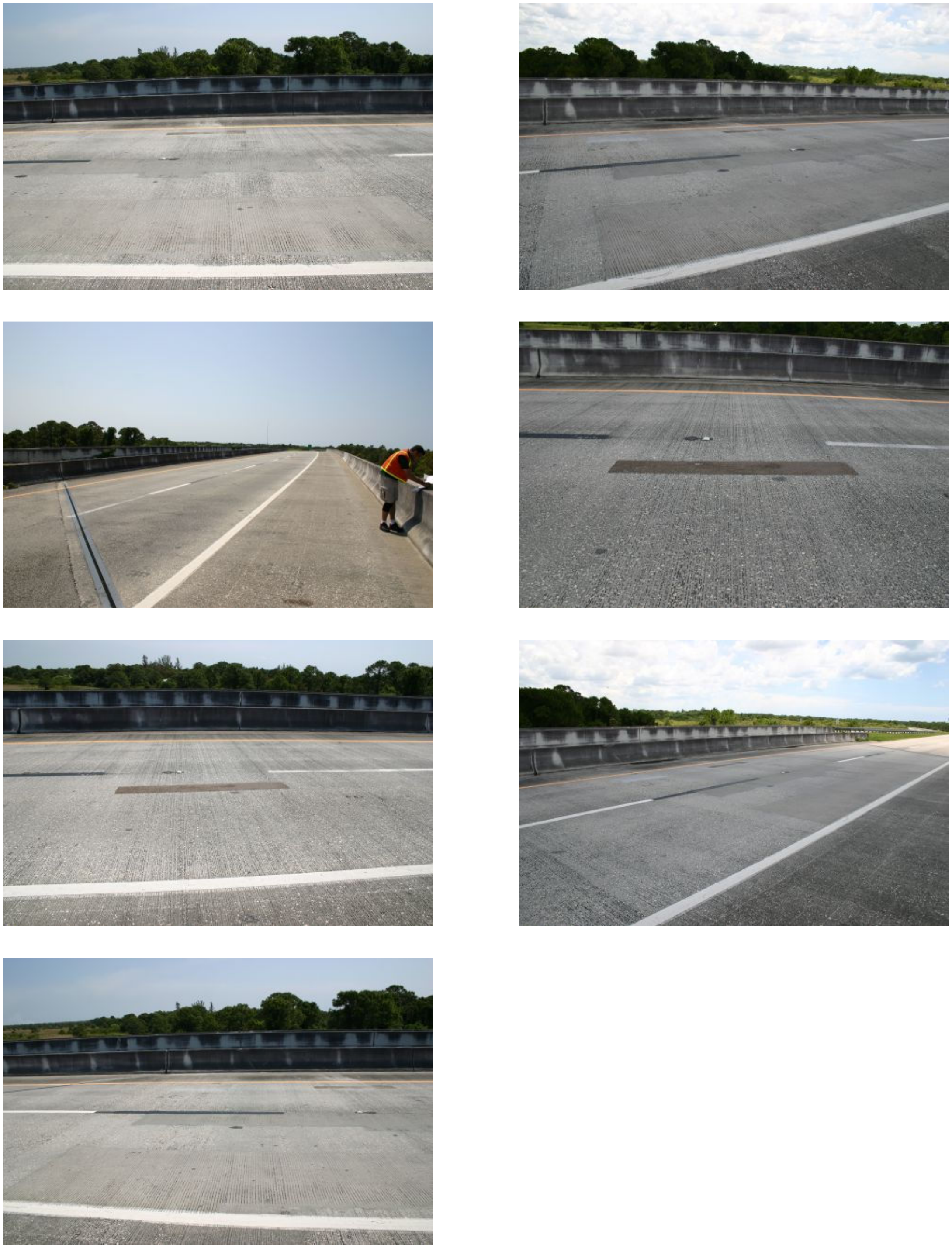


Figure E.7 District 1 Bridge # 170099, SR-681 SB & CSX RR

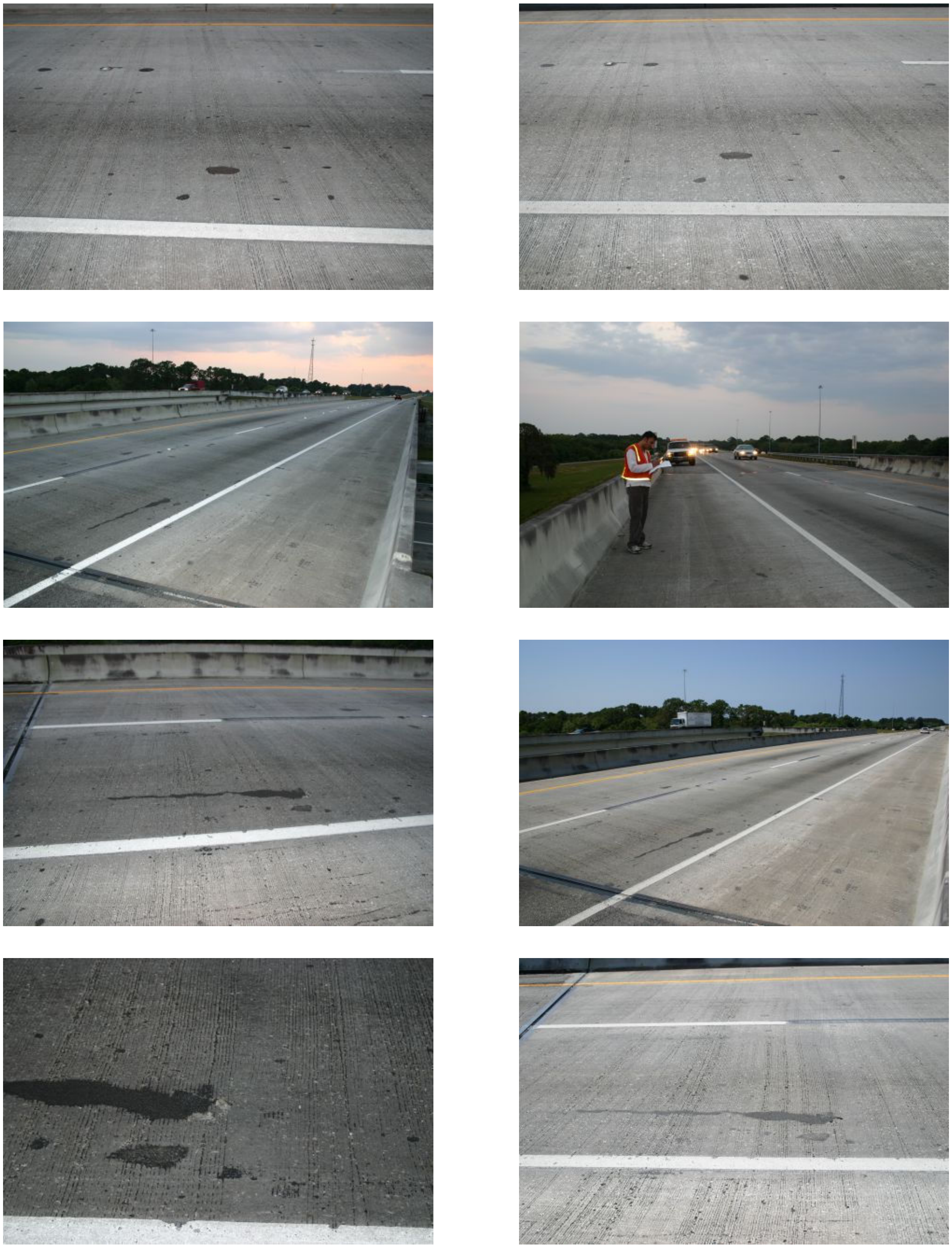


Figure E.8 District 1 Bridge # 170089, I-75/SR-93 & River Road/Cr 777

Figure E.9 District 1 Bridge # 170100, SR-681 NB & CSX RR

Figure E.10 District 1 Bridge # 010064, Oil Well Road & I-75/SR93

Figure E.11 District 1 Bridge # 030187, I-75/SR-93 & CR-846

Figure E.12 District 1 Bridge # 170096, I-75/SR-93 SB & Jacaranda Blvd

Figure E.13 District 1 Bridge # 170079, I-75 & Main A Canal

Figure E.14 District 7 Bridge # 100468, I-75 SB (SR-93a) & Woodberry Road

Figure E.15 District 7 Bridge # 100347, I-75 NB & SR-674

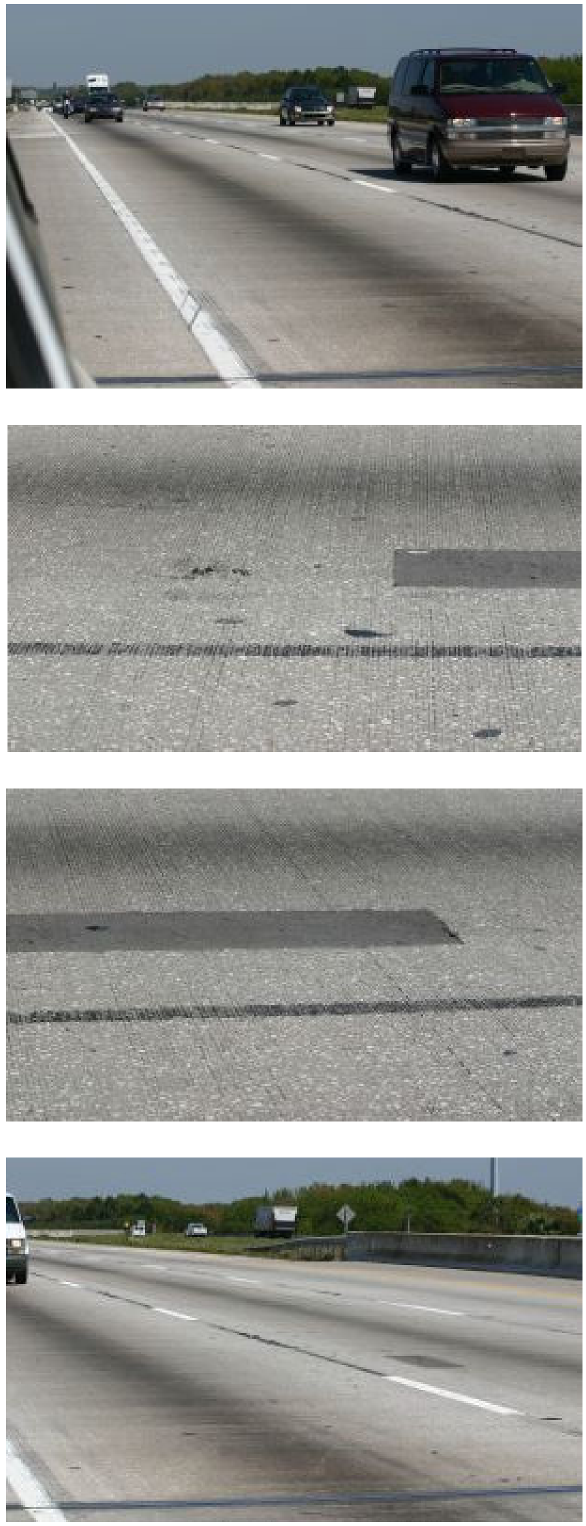


Figure E.16 District 7 Bridge # 100470, I-75 SB (SR 93a) & Csx Rr

Figure E.17 District 7 Bridge # 100358, I-75 SB & Alafia River

Figure E.18 District 7 Bridge # 100359, I-75 NB & Alafia River

Figure E.19 District 7 Bridge # 150122, I-275 NB & 5th Avenue North

Figure E.20 District 7 Bridge # 100346, I-75 SB & SR-674

Figure E.21 District 7 Bridge # 100436, I-75 NB & Cr-574 & Csx Rr

Figure E.22 District 7 Bridge # 100338, US-41/22nd St & Mackay Bay

Figure E.23 District 7 Bridge # 100357, I-75 NB & Riverview Drive

Figure E.24 District 7 Bridge # 100356, I-75 SB & Riverview Drive

Figure E.25 District 7 Bridge # 100080, SR 60 WB & Tampa Bypass Canal

Figure E.26 District 7 Bridge # 100081, SR 60 EB & Tampa Bypass Canal

Figure E.27 District 7 Bridge # 100049, US-41/SR-45 & Palm River

Figure E.28 District 7 Bridge # 100351, Valroy Road & I-75/SR-93