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Deterioration process and deck failure mechanism of Florida's precast deck panel bridges

Ivan A. Gualtero
University of South Florida

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Deterioration Process and Deck Failure Mechanism
of Florida’s Precast Deck Panel Bridges

by

Ivan A. Gualtero

A thesis submitted in partial fulfillment
of the requirements for the degree of
Master of Science in Civil Engineering
Department of Civil & Environmental Engineering
College of Engineering
University of South Florida

Major Professor: Rajan Sen, Ph.D.
Gray Mullins, Ph.D.
Ashraf Ayoub, Ph.D.

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DEDICATION

This work is dedicated to my parents and Tere. Thank you for all of the support that you have given me in my academic and professional pursuits.
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ABSTRACT

During the late 70’s and early 80’s, several precast deck panel bridges were constructed in Florida. These utilize prestressed precast panels as stay-in-place forms and are designed to act compositely with a cast-in-place deck which is poured subsequently. Such bridges offer advantages of quicker construction and lower costs. However, several such bridges built in Florida developed extensive cracking and spalling. Following localized failures, the Florida Department of Transportation have decided to replace all 127 precast panel deck bridges in Districts 1 and 7. Since deck replacement is contingent on funding, it is necessary to develop a rational procedure to decide the order in which they are replaced. This requires a better understanding of the deterioration process and failure mechanism in such bridge decks. The methodology used in this study was to first analyze in detail 5 cases of sudden localized deck failures to identify the causes of the failures and any common factors in the failed bridges. Also forensic studies were conducted on eight bridges scheduled for deck replacements during 2003 and 2004. In these studies it was possible to investigate in detail the condition of the deck at different stages of deterioration. Based on the information collected, a deck failure model was developed.
CHAPTER 1. INTRODUCTION

1.1 Precast Deck Panel System

Precast deck panel systems started as an option to reduce bridge construction cost and time by eliminating most of the field formwork needed, and reducing the amount of cast-in-place concrete to be placed in the deck. This system is basically a precast prestressed concrete panel that spans between bridge girders serving as a support for the cast-in-place topping. When the topping concrete sets, it acts compositely with the panel in resisting subsequent dead and live load.

This deck construction system was first introduced in the early fifties in the Illinois highway system. In the years following, its use was limited due to questions and uncertainties about its performance, typically generated because of the innovative nature of this construction system. In the seventies, Departments of Transportation in several states such as Florida, Texas and Pennsylvania conducted extensive research to find answers to these questions [14].

Following encouraging and positive results from different researches at that time, the panel deck construction system was finally accepted and incorporated into the American Association of State Highway and Transportation Officials (AASHTO) specification [1]. This, as well as its inherent economy led to wide spread use of this deck construction system in highways including some major interstate networks which were built at the time.
1.2 Deck Panel System Construction Details

As mentioned earlier, this deck construction method consists of a precast prestressed concrete panel that spans between the girders of the bridge (see Fig. 1.1).

![Figure 1.1. Typical Cross Section View](image)

For this construction system, alternate construction details are available; basically different panel details and different types of panel bearing. In the early stages of the introduction of this construction method, there were no standard construction details, so different states used different details.

1.2.1 Types of Precast Panels

There are two different types of panels: panel with ribs and panel without ribs (flat panel surface). The panels without ribs were used mainly in the initial years of introduction of this system. It was then found that a flat panel surface could lead to bond problems between the cast-in-place concrete and the panel. With the introduction of ribs in the panel, the bond between the panel and the topping concrete, was substantially improved.
Another construction detail that changed in precast panels was the length of the prestressing strand – whether it extended or stopped at the end of the panel. The extension is typically 3 inches. The idea of extending the strands came in an effort to obtain better control on shrinkage cracks and to improve the composite action between the vertical face of the panel and the topping concrete over the bridge girder. Different studies on the strand extensions have shown different results regarding the benefit of doing this [15]. This may be the reason why this construction detail was not used on Florida’s bridges.

1.2.2 Types of Panel Bearings

Based on the structural behavior panel bearings may be classified as (1) positive panel bearing and (2) negative panel bearing.
As shown in Fig. 1.3, in the positive bearing case, the panel overhangs a strip of soft bearing material (fiberboard). The overhang is at least 1½ in. from the interior vertical face of the strip and the vertical face of the panel. Then the topping concrete is poured with special care over the girders to assure that concrete is placed under the precast panel. After the topping concrete is set, the panel is not longer supported by the fiberboard strip, but by the concrete underneath the panel.

In the case of negative bearing (Fig. 1.3.), the fiberboard strip has the same width as the part of the panel that is supported by the flange of the girder. In other words, there is no concrete under the support of panel after the topping concrete is placed. This is called negative bearing because after the topping concrete is set the panel is no longer supported by the fiberboard but by the topping concrete on top of the panel. This type of panel bearing makes deck construction easier. Originally it was thought to have no negative effects on the structural deck behavior, but recent studies have shown that this is not the case.

### 1.3 Florida’s Deck Panel Bridges

Florida DOT has used different construction details for deck panel bridges throughout the years, including both positive and negative panel bearings.

In order to find the exact type of deck panel design used in each bridge, an extensive search was conducted at the FDOT District 1 and 7 maintenance office. In the search over a hundred bridge plans for deck panel systems were examined to obtain construction details. It was found that in almost all the bridges a full depth cast in place concrete deck instead of the deck panels was shown. Only in a few cases “as built” plans were found that showed deck panel construction details. This is shown in Figs. 1.4 - 1.5.

As shown in Fig. 1.4, the type of bearing used in these bridges is negative bearing with the soft bearing material covering the entire support surface of the panel. The recommended width of the bearing strip ranged from 1 in. to 1½ in. and the thickness
varied from ½” minimum to 1 ½” maximum. The material used as bearing material was fiberboard (*board composed of wood chips bonded together with resin*). Also it is noticeable that there no strand extensions in the panels.

![Diagram of shear connections](image)

**A) SHEAR CONNECTOR DETAILS**

![Diagram of deck cross-section](image)

**B) DECK CROSSSECTION VIEW**

![Diagram of panel bearing details](image)

**C) PANEL BEARING DETAILS**

*Figure 1.4* Construction Details of Florida’s Precast Panel Decks
Regarding the panels, they usually have ribs. These ribs were 6 in. wide and 1 in. high and were spaced at 6 in. intervals. The panel thickness was usually 3 ½ in. (including ribs), the panel length 8 ft and the panel length (span) varied from 5 ft – 9 ft depending on the girder configuration. The prestressing strands were typically located under the panel ribs leaving a minimum clearance of ¼ in from the bottom. The amount and the distribution of the prestressing strands depended on each deck design. (See Fig. 1.5)

![Figure 1.5 Precast Panel Prestressing Strands Configurations](image)

1.4 Use of Deck Panel Construction in Other States

In a detail research about the use of deck panel construction in other states [5] it was found that Texas is only one state where this construction system is widely used. Almost 85% of bridges in Texas use panel decks. Also these bridges have exhibited a performance comparable to full depth cast in place decks. Only longitudinal and transverse cracking have been observed on few occasions, but never sudden deck failures as in Florida.
Texas DOT Deck Panel Design Specifications [16].

(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 positive bearing under live loads.
(3) Polystyrene foam (Dow PL 300 Glue) used instead of fiberboard, available up to 4 in. thick.
(4) Panel overhangs bearing is 1 ½ in. minimum.

Texas does prohibit the use of panel decks 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*: Panel decks 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*: Panel decks 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 panel deck 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.
2.1 Introduction

In Florida many precast deck panel bridges were built during the period from 1980 to 1984, mostly in the Interstate Highway System, specifically in FDOT District 1 and 7. Of the 120 odd bridges, about 95% are located on the Interstate (I-75).

In most of the bridges constructed in Florida, after 2 or 3 years of its construction, the deck started to exhibit unusual longitudinal cracking on the deck surface. As a result FDOT funded research to determine how this early cracking would affect service life of the bridge and its maintenance, and to identify methods that could reduce the deterioration [11]. From these studies the FDOT came up with a repair method to improve the structural behavior of the bridges and stop the deterioration. This method consists of removal of the fiberboard bearing and its replacement by non shrink epoxy. In theory this method works, but in practice deterioration has continued in most of the bridges. This it thought to be due to poor workmanship on the repairs and the high degree of difficulty required to place epoxy in the narrow space between the edge of the panel and the top of the girder (see Fig. 1.3).

In this chapter typical deficiencies found in deck panel bridges will be described in detail. This information was collected from bridges in FDOT District 1 and 7. It was obtained from official FDOT bridge inspection reports.
Some of the typical deficiencies described in his chapter have resulted in sudden localized deck failures. Such failures are described in detail in Chapter 3.

The typical structural deficiencies found in the precast deck panel bridges may be divided in two groups:

1. Deck underside (Precast panel) deficiencies:
   - Transverse Cracks
   - Corner cracks
   - Delaminations
   - Spalls

2. Deck top deficiencies:
   - Failed repairs of spalls
   - Spalls
   - Delaminations
   - Transverse and longitudinal cracks.

2.2 Deck Underside Deficiencies

2.2.1 Bottom Transverse Cracks

This is a crack that appears in the bottom of the deck panels, transverse to the traffic direction. The average crack width is about 0.5 mm. It also has been found that this crack tends to run between two strands. This cracking can be found in the midspan, as well as close to the piers. Also it seems more likely to happen in steel girder bridges.
2.2.2 Panel Corner Crack

This crack is not as common as the transverse panel crack, the average crack width is about 1mm, this crack tends to be in a 45 degree angle and seldom is larger than 2 ft.
2.2.3 Panel Spalls and Delaminations

The occurrence of panel spall is not as common as other panel deficiencies. A panel spall can be found anywhere on the panel underside surface - there is no trend regarding its location. The regular spall size is 3 to 6 in. In case of delaminations, they are located almost always near the panel supports (see Fig. 2.3). It has been found that delamination seems to occur more frequently in steel girder bridges, than concrete girder bridges. The following picture was taken in I-75 over Alafia river (Bridge #100358 -59), one of the first bridges to exhibit problems in the deck in FDOT District 1.

![Panel Spall and Delamination](image)

**Figure 2.3.** Bottom Panel Spalls and Delaminations

2.3 Deck Top Deficiencies

2.3.1 Failed Repairs

Failed repairs are basically caused by the walking spall effect. This is when a deck spall is repaired (removing the adjacent concrete and placing epoxy or new concrete), and after a few days a new spall appears right next to the repair (see Fig 2.4). This new adjacent spalling also causes deterioration of the old repair. Depending on how
this deficiency is treated, it can lead to sudden deck failures. This is a very common deficiency in Florida’s precast deck panel bridges.

![Figure 2.4. Failed Repair (Walking Spall)](image)

2.3.2 Deck Top Spalls

There are 2 different types of spalls that can be found in the deck surface. The first type is the spall that is related to the deck concrete quality and bridge age, this can occur anywhere on the deck, and since is not related with the type of construction it can be found in any concrete bridge deck. The second type of spall is directly related with the deck panel construction. See Fig. 2.5. The typical spall occurs between 2 longitudinal cracks, and under the wheel path. The spall sizes vary depending the age of the spall, and the trend is to keep growing in the longitudinal direction if not special repair is done.
2.3.3 Deck Top Longitudinal and Transverse Cracking

Longitudinal cracks are very common in precast deck panels; this type of cracking is present in almost 90% of the deck panel bridges in FDOT Districts 1 and 7. It has been found that the longitudinal cracks are always located over the edges of the girders. Transverse cracks are not as common as longitudinal cracks. It has been observed that this type of crack is always located over the transverse panel joints.
CHAPTER 3. LOCALIZED FAILURES

3.1 Introduction

Between 2000 and 2003, localized failures occurred in five panel bridges in Districts 1 and 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 combination with information obtained in Chapter 5 to develop a rational deterioration and failure mechanism of these bridges.

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

In the following sections descriptions and analyses of each localized failure are presented in the same order as their listing in Table 3.1 in Sections 3.2-3.6. 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 is 5,000 psi. It is 3,000 psi for the cast in place concrete slab. Additional information regarding deck panel construction may be found in Chapter 1.

![Figure 3.1 Cross Section View of Bridge #170146 – Main Span](image)

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 \(^{[27]}\). Thirty percent of the ADT was truck traffic (ADTT). Details are summarized in Table 3.2.
### Table 3.2  Bridge #170146

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

* 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” [6]. A newspaper account made it 3 square ft - 18 in. x 24 in. [25]. However, no photographs of the damage are available.

Fig. 3.3 shows the location of the failed panel taken from reference [6]. It was redrawn to clarify the details. Failure occurred in “Span 2, Bay 10 at the edge of panel 13” [6]. 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. 6 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…”

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 [21, 25].

The first article [25] 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 [21] 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) [6]
3.2.2 Analysis

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 [2] 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 determine 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. Scanned excerpts from the relevant sections of the inspection report are included in Table 3.3.

The earliest report (May ’92) 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). In the report prepared in Nov’ 97 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”.

19
Table 3.3  Excerpts from Inspection Reports (Bridge #170146)

<table>
<thead>
<tr>
<th>Date</th>
<th>Description</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>11/24/99</td>
<td><strong>ELEMENT INSPECTION NOTES:</strong> 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 R/C panels. There is extensive random cracking with efflorescence at the north and north panel in Bay 4 of Span 3. There are several transverse cracks in the adjacent panel to the south. A portion of Bay 3 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 R/C panel.</td>
<td></td>
</tr>
<tr>
<td>11/05/97</td>
<td><strong>01.01 DECK (TOP)</strong> 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 occurred in spalled areas in span 1, right travel lane. A deck joint near the south end of a repair area, 12 ft from joint 1, has been repaired with an inappropriate material (asphalt), and is spalling around the edges.</td>
<td></td>
</tr>
<tr>
<td>12/13/95</td>
<td><strong>01.01 DECK (TOP) / SURFACING</strong> 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.</td>
<td></td>
</tr>
<tr>
<td>12/13/94</td>
<td><strong>01.01 DECK (TOP) / SURFACING</strong> 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 approximately 4.75 ft from abutment 4. There are also minor spalls that run along a longitudinal crack in the deck at span 2 in the right travel lane.</td>
<td></td>
</tr>
<tr>
<td>05/04/92</td>
<td><strong>Deck Component</strong> <strong>1.01 Deck (top) and 1.02 Deck (underside)</strong> 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 ft from abutment 4. There are also minor spalls that run along a longitudinal crack in the deck at span 2 in the right travel lane.</td>
<td></td>
</tr>
</tbody>
</table>

FDOT Bridge Inspection Report (Deck)
The final inspection report (Nov ’99) classifies the deck rating as ‘6’ (satisfactory) with a condition state of 3 since the combined area of distress 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 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 [17]. 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 [17].

3.2.4 Punching Shear

An estimate of the punching shear resistance can be obtained using code specified formula [2]. 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 [6].
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. [1]) 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 does not 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° failure surface. 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. 2. 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 an explanation as to why failure occurred.

**Table 3.4** Punching Shear Resistance Bridge # 170146

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Punching Shear Resistance*</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Full Composite Action</strong></td>
<td>CIP</td>
</tr>
<tr>
<td></td>
<td>$d_c = 4$ in.</td>
</tr>
<tr>
<td></td>
<td>$b_0 = 34$ in.</td>
</tr>
<tr>
<td></td>
<td>$V_{CIP} = 29.8$ kips</td>
</tr>
<tr>
<td></td>
<td>PANEL</td>
</tr>
<tr>
<td></td>
<td>$d_c = 2.56$ in. (ave)</td>
</tr>
<tr>
<td></td>
<td>$V_{PANEL} = 26.5$ kips</td>
</tr>
<tr>
<td></td>
<td>$V_{TOTAL} = 56.3$ kips</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>No Composite Action</th>
<th>CIP</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$V_{CIP} = 0$ kips</td>
</tr>
<tr>
<td>PANEL</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$d_c = 2.06$ in. (min)</td>
</tr>
<tr>
<td></td>
<td>$b_0 = 32.1$ in.</td>
</tr>
<tr>
<td></td>
<td>$V_{PANEL} = 15.3$ kips</td>
</tr>
<tr>
<td></td>
<td>$V_{TOTAL} = 15.3$ kips</td>
</tr>
</tbody>
</table>
### Table 3.4 (Continued)

<table>
<thead>
<tr>
<th>Effective Depth</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Traffic Direction</strong></td>
</tr>
<tr>
<td>2.06&quot;</td>
</tr>
<tr>
<td>2.06&quot;</td>
</tr>
<tr>
<td>Ø3/16&quot; @ 6&quot;</td>
</tr>
<tr>
<td>Ø2/8@6&quot;</td>
</tr>
<tr>
<td>Ø3/8&quot;</td>
</tr>
<tr>
<td>ø3/8&quot;</td>
</tr>
<tr>
<td>ø1/8 Min</td>
</tr>
<tr>
<td>6&quot;</td>
</tr>
<tr>
<td>6&quot;</td>
</tr>
<tr>
<td>6&quot;</td>
</tr>
<tr>
<td>1&quot;</td>
</tr>
</tbody>
</table>

**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.

* 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 in the slower right lane 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 indicates 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 is 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 is 5,000 psi and 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 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 [27]. 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>
<thead>
<tr>
<th>Table 3.5</th>
<th>Bridge #170086</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Bridge #170086 Characteristics</strong></td>
<td></td>
</tr>
<tr>
<td>Year Built</td>
<td>1980</td>
</tr>
<tr>
<td>Number of Spans</td>
<td>4</td>
</tr>
<tr>
<td>Lanes on Structure</td>
<td>3</td>
</tr>
<tr>
<td>ADT [3.1]</td>
<td>34,000</td>
</tr>
<tr>
<td>ADTT [3.1]</td>
<td>30%</td>
</tr>
<tr>
<td>Deck Condition Rating (2000)</td>
<td>7 (Good)</td>
</tr>
<tr>
<td>Composite Slab Thickness</td>
<td>7 in</td>
</tr>
<tr>
<td>Precast Panel Thickness</td>
<td>2-½ in (panel) 3-½ in (ribs)</td>
</tr>
<tr>
<td>Girder Type</td>
<td>AASHTO Type II and IV</td>
</tr>
</tbody>
</table>
3.3.1 Failure Details

Localized punching shear occurred late morning, Monday November 27, 2000. According to the consultant’s emergency response report [7], 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 [4] 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 [4, 22].

The first article [4] 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 [7]
The second article published the following day [22] 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.”

![Failed Panel Bridge #170086](Image)

**Figure 3.7** View of Failed Panel Bridge #170086 (*Courtesy Sarasota Herald*) [4]

### 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. 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 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)

<table>
<thead>
<tr>
<th>Unit: 0 Decks</th>
<th>FDOT Bridge Inspection Report (Deck)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ELEMENT/ENV: 98/4 Conc Deck on PC Pane 1309 sq.m.</td>
<td>ELEM CATEGORY: Decks/Slabs</td>
</tr>
<tr>
<td>CONDITION STATE (5)</td>
<td>DESCRIPTION</td>
</tr>
<tr>
<td>2</td>
<td>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.</td>
</tr>
</tbody>
</table>

**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.

<table>
<thead>
<tr>
<th>Date</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>05/08/00</td>
<td>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.</td>
</tr>
<tr>
<td>05/04/98</td>
<td>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.</td>
</tr>
<tr>
<td>06/19/96</td>
<td>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.</td>
</tr>
<tr>
<td>08/24/94</td>
<td></td>
</tr>
</tbody>
</table>

30
Table 3.6 (Continued)

<table>
<thead>
<tr>
<th>Deck Component</th>
<th>Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.01 Deck (top)</td>
<td>01/04/93</td>
</tr>
</tbody>
</table>

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. 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), 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 [17]. 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.
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” [7]. 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  Punching Shear Resistance Bridge # 170086

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Shear Resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder Top</td>
<td>$V_{\text{PANEL}} = 2 \sqrt{f'c} b_w d$</td>
</tr>
<tr>
<td>Girder Top</td>
<td>$V_{\text{PANEL}} = 13$ kips</td>
</tr>
<tr>
<td>Girder Top</td>
<td>$V_{\text{TOTAL}} = 13$ kips</td>
</tr>
</tbody>
</table>

- $b_w = 36$ in
- $d = 2.56$ in (ave)
- $f'c = 5000$ psi

3.3.5 Conclusions

A number of factors were responsible for this unusual failure. The most important of these was 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 [8] 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 [8] 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

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

* From National Bridge Inventory (2000)
3.4.1.1 Newspaper Account

Three articles regarding the failure were reported in the local newspaper, *Sarasota Herald Tribune* [23,24,26]. Of these only the first and last had relevant information.

The first article published on December 21, 2000 [26] 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 [23] 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).
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. 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.

Figure 3.10  Location of Failed Panel Bridge #170085 [8]
The first two reports 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 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 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), 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 “concrete 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)

<table>
<thead>
<tr>
<th>ELEMENT/ENV: 98/4</th>
<th>Conc Deck on PC Pane</th>
<th>1309 sq.m.</th>
<th>ELEM CATEGORY: Decks/Slabs</th>
</tr>
</thead>
<tbody>
<tr>
<td>CONDITION STATE (5)</td>
<td>DESCRIPTION</td>
<td>QUANTITY</td>
<td>RECOMMENDED FEASIBLE ACTION</td>
</tr>
<tr>
<td>2</td>
<td>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.</td>
<td>1309</td>
<td>0 Do Nothing</td>
</tr>
</tbody>
</table>

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.

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 2 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 .1m surrounding an asphalt patch at midspan adjacent to the west edge of pavement stripe.
Table 3.9  (Continued)

<table>
<thead>
<tr>
<th>Date</th>
<th>FDOT Bridge Inspection Report (Deck)</th>
</tr>
</thead>
<tbody>
<tr>
<td>06/19/96</td>
<td>G1.01 DECK (TOP)/SURFACING</td>
</tr>
<tr>
<td></td>
<td>All spans contain class 1 and 2 longitudinal cracks 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 x 15cm x 10cm). The center lane contains a spall (10cm x 15cm x 5cm). 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 chipping which will spill.</td>
</tr>
<tr>
<td>08/24/94</td>
<td>G1.01 DECK (TOP)/SURFACING</td>
</tr>
<tr>
<td></td>
<td>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.</td>
</tr>
<tr>
<td>04/93</td>
<td>Deck Component</td>
</tr>
<tr>
<td>04/03</td>
<td>1.01 Deck (top)</td>
</tr>
<tr>
<td></td>
<td>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.</td>
</tr>
</tbody>
</table>

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 [17]. 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.
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 region, faulty construction was undoubtedly a factor though precise faults cannot be pinpointed at this time.
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 was 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 [27]. 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

<table>
<thead>
<tr>
<th>Bridge #100332 Characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td>Year Built</td>
</tr>
<tr>
<td>Number of spans</td>
</tr>
<tr>
<td>Lanes on Structure</td>
</tr>
<tr>
<td>ADT (2002) [17]</td>
</tr>
<tr>
<td>Percent Truck ADTT [17]</td>
</tr>
<tr>
<td>Deck Condition Rating Span 38 (2001)</td>
</tr>
<tr>
<td>Deck Condition Rating Span 70 (2003)</td>
</tr>
<tr>
<td>Composite Slab Thickness</td>
</tr>
<tr>
<td>Precast Panel Thickness</td>
</tr>
<tr>
<td>Girder Type (Span 38)</td>
</tr>
</tbody>
</table>
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 38
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. 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 biennial inspection completed in Aug '01 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". 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)

<table>
<thead>
<tr>
<th>ELEMENT/ENV:99/4</th>
<th>Conc Deck on PC Pane</th>
<th>ELEM CATEGORY: Decks/Slabs</th>
</tr>
</thead>
<tbody>
<tr>
<td>CONDITION STATE (5)</td>
<td>DESCRIPTION</td>
<td>QUANTITY</td>
</tr>
<tr>
<td>2</td>
<td>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.</td>
<td>21588</td>
</tr>
</tbody>
</table>

**WORK ORDER RECOMMENDATION:**

REPR SPLs in Spans 26 27 29 39 41 44 47 53 57 58 70 75 80 & 81: 0.3MP are issued on a regular basis.

**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.
Table 3.11 (Continued)

FDOT Bridge Inspection Report (Deck)

<table>
<thead>
<tr>
<th>Condition State (5)</th>
<th>Description</th>
<th>Recommended Quantity</th>
<th>Feasible Action</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>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.</td>
<td>1 Spalls &amp; Delams NAR</td>
<td></td>
</tr>
</tbody>
</table>

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 appear 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 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 underside of the overhangs, predominately 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.

W/O - 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.

---

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.

---

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.

<table>
<thead>
<tr>
<th>SPAN</th>
<th>DEFICIENCY</th>
<th>LANE</th>
<th>LOCATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>38</td>
<td>Two class 1 spalls with cracking</td>
<td>North</td>
<td>Near pier 38 and 39</td>
</tr>
</tbody>
</table>
Table 3.11 (Continued)

FDOT Bridge Inspection Report (Deck)

<table>
<thead>
<tr>
<th>SPAN</th>
<th>DEFICIENCY</th>
<th>LANE</th>
<th>LOCATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>38</td>
<td>One Class 1 spall with cracking</td>
<td>North</td>
<td>Near Pier 38</td>
</tr>
</tbody>
</table>

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 be seen 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.
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 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.
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.

![Bridge # 100332 Span 70](image)

**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 subsequent 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.
3.6.1.1 Newspaper Account

Two articles regarding the failure were reported in the local newspaper, *Tampa Tribune* [19,20]. The first article [19] 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”.

![Diagram of bridge failure](image)

**Figure 3.20** Location of Failed Panel, Bridge #100332 Span 70
The second article [20] 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. 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 monthly 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 Span 70)

<table>
<thead>
<tr>
<th>ELEMENT/ENV: 98/4 Conc Deck on PC Pane</th>
<th>ELEM CATEGORY: Decks/Slabs</th>
</tr>
</thead>
<tbody>
<tr>
<td>CONDITION STATE (5)</td>
<td>QUANTITY</td>
</tr>
<tr>
<td>2</td>
<td>231080 sf.</td>
</tr>
<tr>
<td>REPAIRED AREAS AND/OR SPILLS/DELAMINATIONS AND/OR CRACKS EXIST IN THE DECK SURFACE OR UNDERSIDE. THE COMBINED DISTRESSED AREA IS 2% OR LESS OF THE DECK AREA.</td>
<td>231080 sf.</td>
</tr>
</tbody>
</table>

ELEM INSPECTION NOTES:

NOTE: This element quantifies the concrete deck with 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, spell, no steel, 2in x 4in x 1/2in.
- Span 26, panel 3-5, east edge adjacent beam 26-3, spell with exposed wire, 12in x 4in x 1/2in.
- Span 39, panel 4-5, SW corner, spell w/exposed wire, 12in x 12in x 1-1/2in.
- Span 41, panel 4-7, NW corner, spell/delamination, no steel, 10in x 10in x 3/4in.
- Span 47, panel 1-7, NW corner, spell w/exposed wire, 10in diameter x 3/4in.
- Span 47, panel 5-7, NE corner, spell w/exposed wire, 12in diameter x 1in.

Refer to photo 1. P3 WO

<table>
<thead>
<tr>
<th>ELEMENT/ENV: 98/4 Conc Deck on PC Pane</th>
<th>ELEM CATEGORY: Decks/Slabs</th>
</tr>
</thead>
<tbody>
<tr>
<td>CONDITION STATE (5)</td>
<td>QUANTITY</td>
</tr>
<tr>
<td>2</td>
<td>21588 sq.m.</td>
</tr>
<tr>
<td>REPAIRED AREAS AND/OR SPILLS/DELAMINATIONS AND/OR CRACKS EXIST IN THE DECK SURFACE OR UNDERSIDE. THE COMBINED DISTRESSED AREA IS 2% OR LESS OF THE DECK AREA.</td>
<td>21588</td>
</tr>
</tbody>
</table>

REPAIR SPLA in Spans 25 27 29 30 44 47 53 57 58 70 78 89 & 91: 0.3M3

WORK ORDER RECOMMENDATION:
- REPAIR SPLA in Spans 25 27 29 30 44 47 53 57 58 70 78 89 & 91: 0.3M3

ELEM 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 falling repairs and spills in the deck top and deck panel undersides. Refer to the Addendum for additional text.
Table 3.12 (Continued)

<table>
<thead>
<tr>
<th>ELEMENT/ENV: 98/4</th>
<th>Conc Deck on PC Pile</th>
<th>19328 sq.m.</th>
</tr>
</thead>
<tbody>
<tr>
<td>ELEM CATEGORY:</td>
<td>Decks/Giabs</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>CONDITION STATE(5)</th>
<th>DESCRIPTION</th>
<th>QUANTITY</th>
<th>RECOMMENDED ACTION</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Spalls, delaminations and/or cracks exist in the deck surface or underside.</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>The combined distressed area is 2% or less of the deck area.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

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 minor 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 turing 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 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.

WQ - Repair any spalls with exposed steel or any falling 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 0.05 m with exposed steel is present in this span. (See Photo D-27)

**G1.01 DECK (TOP)**

There are longitudinal, transverse and minor 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.

**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.

<table>
<thead>
<tr>
<th>SPAN</th>
<th>DEFICIENCY</th>
<th>LANE</th>
<th>LOCATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>70</td>
<td>Class 1 spall with exposed rebar and one incipient spall</td>
<td>South</td>
<td>Near midspan</td>
</tr>
<tr>
<td>70</td>
<td>Eight class 1 spalls with exposed rebar</td>
<td>South</td>
<td></td>
</tr>
<tr>
<td>70</td>
<td>One class 2 longitudinal crack entire length of span</td>
<td>North</td>
<td></td>
</tr>
</tbody>
</table>
Table 3.12  (Continued)

<table>
<thead>
<tr>
<th>G1.01</th>
<th>Deck (Top)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>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.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th></th>
<th>South</th>
<th>Near midspan</th>
</tr>
</thead>
<tbody>
<tr>
<td>70</td>
<td>Class 1 spall with exposed rebar and one incipient spall</td>
<td></td>
<td></td>
</tr>
<tr>
<td>70</td>
<td>Eight Class 1 spalls with exposed rebar</td>
<td></td>
<td></td>
</tr>
<tr>
<td>70</td>
<td>One Class 2 longitudinal crack entire length of span</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Bridge # 100332
Date: 7/31/02
Span 70, lane 1
Spall, 3 ft x 12 in 1.5 in, with exposed steel. INCREASE.

A

100332, span 70, lane 1
Photo 6
01/22/03

B
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

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

<table>
<thead>
<tr>
<th>Date</th>
<th>Deficiency Description</th>
<th>Feature Intersected</th>
<th>Photo</th>
</tr>
</thead>
<tbody>
<tr>
<td>8/12/03</td>
<td>Lane 1, span length full depth repair with perimeter RP - NC - STABLE</td>
<td>Hills, Riverview/Downtown</td>
<td>N</td>
</tr>
<tr>
<td></td>
<td>Lane 1, 2&quot; x 1&quot; x 1&quot; SPL/DEL with RX S. side of full depth repair &amp; E. of small RP - NEW</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7/11/03</td>
<td>Lane 2, two RP, trans. Crk. 1/16&quot; wide - RP MADE TO PERIMETER</td>
<td>Hills, River/Downtown</td>
<td>N</td>
</tr>
</tbody>
</table>

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.

![Tampa International Airport Precipitation (Aug 6 – Sep 5 2003)](image)

**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>
<thead>
<tr>
<th>Load Case (Panel Edge)</th>
<th>Punching Shear Resistance*</th>
</tr>
</thead>
</table>
| **Full Composite Action** | CIP  
  $d_e = 4 \text{ in.}$  
  $b_0 = 58 \text{ in.}$  
  $V_{CIP} = 50.8 \text{ kips}$  
  PANEL  
  $d_e = 2.56 \text{ in. (ave)}$  
  $V_{PANEL} = 41.4 \text{ kips}$  
  $V_{TOTAL} = 92.2 \text{ kips}$ |
| |  
  Tire Contact area: $b=20\text{ in}$  
  $l = 10 \text{ in}$  
  * Average values |
| **No Composite Action** | CIP  
  $V_{CIP} = 0 \text{ kips}$  
  PANEL  
  $d_e = 2.06 \text{ in. (min)}$  
  $b_0 = 54.12 \text{ in.}$  
  $V_{PANEL} = 21.7 \text{ kips}$  
  $V_{TOTAL} = 21.7 \text{ kips}$ |
| |  
  Tire Contact area: $b=20\text{ in}$  
  $l = 10 \text{ in}$  |

* 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). Again, failure was due to re-repair regions. 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 panel deck 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.

The primary goal of this chapter was to identify underlying trends that led to failure in order to develop a rational deterioration and failure mechanism of these bridges. 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 panel deck 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](image)

60
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 satisfactorily. 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

<table>
<thead>
<tr>
<th>Bridge #</th>
<th>Condition Rating</th>
<th>Last Inspection</th>
<th># of Rainfall events in past 7 days</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>170146</td>
<td>6 (Satisfactory)</td>
<td>3 months</td>
<td>0</td>
<td>Not identified</td>
</tr>
<tr>
<td>170086</td>
<td>7 (Good)</td>
<td>6 months</td>
<td>2 (0.68 in)</td>
<td>Not identified</td>
</tr>
<tr>
<td>170085</td>
<td>7 (Good)</td>
<td>7 months</td>
<td>4 (0.2 in.)</td>
<td>Identified</td>
</tr>
<tr>
<td>100332</td>
<td>5 (Fair)</td>
<td>2 days</td>
<td>2 (0.55 in.)</td>
<td>Identified</td>
</tr>
<tr>
<td>100332</td>
<td>5 (Fair)</td>
<td>23 days</td>
<td>3 (1.1 in.)</td>
<td>Identified</td>
</tr>
</tbody>
</table>
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>
<thead>
<tr>
<th>Bridge #</th>
<th>Year Built</th>
<th>Age at Failure (yrs)</th>
<th>ADT (ADTT)</th>
<th>Failure Size</th>
<th>Location in Panel</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>170146</td>
<td>1981</td>
<td>19</td>
<td>34,000 (30%)</td>
<td>18 in x 24 in</td>
<td>Edge or Corner?</td>
<td>Failure at M1 repair</td>
</tr>
<tr>
<td>170086</td>
<td>1980</td>
<td>20</td>
<td>34,000 (30%)</td>
<td>36 in x 60 in</td>
<td>Corner Support</td>
<td>Patch repair</td>
</tr>
<tr>
<td>170085</td>
<td>1980</td>
<td>20</td>
<td>34,000 (30%)</td>
<td>18 in x 18 in</td>
<td>Corner</td>
<td>Failure adjacent to M1 repair</td>
</tr>
<tr>
<td>100332</td>
<td>1980</td>
<td>22</td>
<td>23,000 (8%)</td>
<td>48 in x 30 in</td>
<td>Near corner</td>
<td>Asphalt Patch</td>
</tr>
<tr>
<td>100332</td>
<td>1980</td>
<td>23</td>
<td>23,000 (8%)</td>
<td>24 in x 36 in</td>
<td>Edge</td>
<td>Failed M1 repair with flexible patch material</td>
</tr>
</tbody>
</table>

* 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).
CHAPTER 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 panel deck 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>
<thead>
<tr>
<th>Bridge #</th>
<th>District</th>
<th>Built</th>
<th>Study Date</th>
<th>Bridge Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>130078</td>
<td>1</td>
<td>1981</td>
<td>6/03</td>
<td>I-75 SB over Moccasin Wallow Rd (Manatee County)</td>
</tr>
<tr>
<td>130079</td>
<td>1</td>
<td>1981</td>
<td>6/03</td>
<td>I-75 NB over Moccasin Wallow Rd (Manatee County)</td>
</tr>
<tr>
<td>170140</td>
<td>1</td>
<td>1981</td>
<td>1/04</td>
<td>I-75 NB over Toledo Blade Blvd (Sarasota County)</td>
</tr>
<tr>
<td>130075</td>
<td>1</td>
<td>1981</td>
<td>5/04</td>
<td>I-75 SB over CSR R/R (Manatee County)</td>
</tr>
<tr>
<td>100415</td>
<td>7</td>
<td>1983</td>
<td>6/04</td>
<td>I-75NB over US 92 (Hillsborough County)</td>
</tr>
<tr>
<td>100398</td>
<td>7</td>
<td>1984</td>
<td>6/04</td>
<td>I-75NB over Sligh &amp; Ramp D-1 (Hillsborough County)</td>
</tr>
<tr>
<td>100417</td>
<td>7</td>
<td>1983</td>
<td>7/04</td>
<td>I-75NB over Ramp B-1 (Hillsborough County)</td>
</tr>
<tr>
<td>130085</td>
<td>1</td>
<td>1981</td>
<td>8/04</td>
<td>I-75NB over SR-64 (Sarasota County)</td>
</tr>
</tbody>
</table>

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 is presented in Sections 4.3-4.10.

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 on 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:

1. No deck cracking.
2. Longitudinal deck surface cracks.
3. Transverse deck surface cracks.
4. Deck surface spalling including “walking” spalls.
5. Deficient M1 repairs.
6. Underside longitudinal and transverse panel cracking.
7. Condition of fiberboard bearing.
8. Effect of epoxy panel bearing.
9. 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 panel deck 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 is 5,000 psi. It is 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 panel deck 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 [27]

<table>
<thead>
<tr>
<th></th>
<th>Bridge #130078 (SB)</th>
<th>Bridge #130079 (NB)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Year Built</strong></td>
<td>1981</td>
<td>1981</td>
</tr>
<tr>
<td><strong>Number of Spans</strong></td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td><strong>Lanes on Structure</strong></td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td><strong>ADT (2003)</strong></td>
<td>26,500</td>
<td>27,000</td>
</tr>
<tr>
<td><strong>Percent Truck (ADTT)</strong></td>
<td>30%</td>
<td>30%</td>
</tr>
<tr>
<td><strong>Composite Slab Thickness</strong></td>
<td>7-½ in.</td>
<td>7-½ in.</td>
</tr>
<tr>
<td><strong>Precast Panel Thickness</strong></td>
<td>2-½ in (panel) 3-½ in (ribs)</td>
<td>2-½ in (panel) 3-½ in (ribs)</td>
</tr>
<tr>
<td><strong>Girder Type</strong></td>
<td>AASHTO Type II and IV</td>
<td>AASHTO Type II and IV</td>
</tr>
<tr>
<td><strong>Deck Condition Rating (2003)</strong></td>
<td>7 (Good Condition)</td>
<td>7 (Good Condition)</td>
</tr>
</tbody>
</table>
4.3.2 Inspection Method

As several panels (Fig. 4.3) had already been removed when I was able to access the site, 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.
A quick visual inspection was conducted to identify panels that exhibited typical deficiencies and a detailed 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**

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.
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. Please refer to Appendix B for detailed information on these deck cores.

To eliminate any doubt that the crack patterns were induced by the removal process, insitu inspection was instituted wherein 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.

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.
Figure 4.6  Overview of Findings from Bridge # 130079

- Diagonal Crack
- Vertical Crack
- Longitudinal crack at the surface
- Severe cracking and spalling

See Fig. 4.7
See Fig. 4.8
See Fig. 4.10
See Fig. 4.11

Separation at vertical panel face
Longitudinal surface crack
Diagonal Crack
Vertical Crack

No surface cracks
Small vertical crack
Separation at vertical panel face
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](image)

**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 of about 5 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 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 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 pane 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 that 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 can only be detected from sections that have been 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..3.3) 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](image)

This additional longitudinal cracking is caused by a bifurcation 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.

![Figure 4.11 Development of a Deck Surface Spall](image)
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.

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.

4.3.3.6 Findings on Panel Bearings

Regarding the precast panel’s bearing it was found that the bridge was built using only fiberboard to support the panels (negative bearing), but recently only in some areas of the bridge the fiberboard bearing has been removed and replaced by epoxy. The replacement of the fiberboard by epoxy was recommended in a previous research study [11], as a method to reduce future deterioration of the bridge deck, but exactly in the spots where the major deterioration was found, the fiberboard had not been replaced by epoxy.

4.3.3.7 Findings on Core Examination

Most of the cracks found on the cores show signs of water and dust infiltration (Cores 1-3, 1-4, 5-4, 5-6, 5-7) (Fig. 4.12). In the case of vertical cracks, some of them show 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 of 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 (core 1-1, 1-2, 5-4, 5-5). Along the longitudinal interface away from the panel edge there was no debonding (cores 5-3).

The depth of the deck, measured at each core location, varied from 7 ¾ in to 8 ½ in. See Appendix B for a detail 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 is 5,000 psi. It is 3,000 psi for the cast in place concrete slab.

**Table 4.3  Bridge #170140**

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

**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.

![View of Bridge #170140](image)

**Figure 4.14** View of Bridge #170140

### 4.4.3 Findings

An examination of the retrieved panels confirmed the findings from the previous bridge. 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.

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.
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 is 5,000 psi. It is 3,000 psi for the cast in place concrete slab.

**Table 4.4** Bridge #130075 [27]

<table>
<thead>
<tr>
<th>Bridge #130075 Characteristics</th>
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<tbody>
<tr>
<td>Year Built</td>
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</tr>
<tr>
<td>Number of Spans</td>
<td>3</td>
</tr>
<tr>
<td>Lanes on Structure</td>
<td>3</td>
</tr>
<tr>
<td>ADT (2003)</td>
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<tr>
<td>Percent Truck (ADTT)</td>
<td>30%</td>
</tr>
<tr>
<td>Composite Slab Thickness</td>
<td>7-½ in.</td>
</tr>
<tr>
<td>Precast Panel Thickness</td>
<td>2-½ in (panel) 3-½ in (ribs)</td>
</tr>
<tr>
<td>Girder Type</td>
<td>AASHTO Type II and III</td>
</tr>
<tr>
<td>Deck Condition Rating (2003)</td>
<td>5 (Fair Condition)</td>
</tr>
</tbody>
</table>
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.
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](image)

**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 panel deck 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 [14]. 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 causes separation of the interface between the M1 repair and the panel that cannot act compositely under flexural loading. The deterioration is 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 is 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 is 5,000 psi. It is 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) |
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.

![Cross Section View of Bridge #100415 span 2](image)

**Figure 4.22** Cross Section View of Bridge #100415 span 2

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.

![Bridge #100415 Span 2, Prior to Deck Removal](image)

**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 therefore 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 on the deck bearing in most of the deck. 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](image-url)
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 top 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

<table>
<thead>
<tr>
<th>Bridge #100398 Characteristics</th>
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<tbody>
<tr>
<td><strong>Year Built</strong></td>
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<tr>
<td><strong>Number of Spans</strong></td>
</tr>
<tr>
<td><strong>Lanes on Structure</strong></td>
</tr>
<tr>
<td><strong>ADT (2003)</strong></td>
</tr>
<tr>
<td><strong>Percent Truck (ADTT)</strong></td>
</tr>
<tr>
<td><strong>Composite Slab Thickness</strong></td>
</tr>
<tr>
<td><strong>Precast Panel Thickness</strong></td>
</tr>
<tr>
<td><strong>Girder Type</strong></td>
</tr>
<tr>
<td><strong>Deck Condition Rating (2003)</strong></td>
</tr>
</tbody>
</table>

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 is 5,000 psi. It is 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).

![Deck Overview of Bridge #100398](image)

**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.

![Inspection Methods Bridge #100398](image)

**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 [11].

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 panel deck 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](image)

98
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 on in almost 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 is 5,000 psi. It is 3,000 psi for the cast in place concrete slab.

<table>
<thead>
<tr>
<th>Table 4.7 Bridge #100417</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Bridge #100417 Characteristics</strong></td>
</tr>
<tr>
<td>Year Built</td>
</tr>
<tr>
<td>Number of Spans</td>
</tr>
<tr>
<td>Lanes on Structure</td>
</tr>
<tr>
<td>ADT (2001)</td>
</tr>
<tr>
<td>Percent Truck (ADTT)</td>
</tr>
<tr>
<td>Composite Slab Thickness</td>
</tr>
<tr>
<td>Precast Panel Thickness</td>
</tr>
<tr>
<td>Girder Type</td>
</tr>
<tr>
<td>Deck Condition Rating (2003)</td>
</tr>
</tbody>
</table>
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 part of District 7.

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

In this bridge was found 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 [11].

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](image)

It was also found the longitudinal cracks observed in the inspection conducted, were caused, as found in all the pervious studies, due to separation of the vertical face of the panel (Fig. 4.36). It is again proven 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 directly linked to the type of bearing used (negative bearing).
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 on this bridge.

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 panel deck 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 is 5,000 psi. It is 3,000 psi for the cast in place concrete slab.

<table>
<thead>
<tr>
<th>Table 4.8 Bridge #130085</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bridge #130085 Characteristics</td>
</tr>
<tr>
<td>Year Built</td>
</tr>
<tr>
<td>Number of Spans</td>
</tr>
<tr>
<td>Lanes on Structure</td>
</tr>
<tr>
<td>ADT (2001)</td>
</tr>
<tr>
<td>Percent Truck (ADTT)</td>
</tr>
<tr>
<td>Composite Slab Thickness</td>
</tr>
<tr>
<td>Precast Panel Thickness</td>
</tr>
<tr>
<td>Girder Type</td>
</tr>
<tr>
<td>Deck Condition Rating (2003)</td>
</tr>
</tbody>
</table>
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).
4.9.2 Findings

In this bridge was found that the panels were initially placed over a 1 ½ wide fiberboard strip, and a 2 in wide grout layer (Fig. 4.40), providing positive bearing support to the panels. But also was found that the fiberboard was later replaced by epoxy, as recommended in a previous research study [14] as a way to stop actual or to prevent future deterioration on deck panel bridges.

The panel bearings were inspected in a large area of the deck, and it was found that some spots where the precast panel wasn’t long enough to reach the grout, so those panels were only supported initially by the fiberboard and now by the epoxy repair (Fig. 4.40 4.41).

![Figure 4.40 Bridge # 130085 Original Panel Bearing Detail](image-url)
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, vertical cracking were detected (Fig. 4.43). The same cracking has been found in bridges with negative panel bearing. This proves that it is not related to the type of bearing used to support the panel.
As in bridges (100398 and 100417) the relative good condition of this bridge can be linked to the fact that the this bridge was built using positive (grout) bearing for the precast panels, plus the fact that the fiberboard was later replaced by epoxy, that could have helped to prevent deterioration in spots were the panel was supported only by the fiberboard (Fig. 4.42(b)).

4.10 Study Summary

The following table (Table 4.9) summarizes all the different types of panel bearings found in the bridges covered on the forensic study, and the link between the type of bearing and the condition of the deck.
Even though it was originally thought that all the deck panel bridges in FDOT Districts 1 and 7 were built supported only by fiberboard (negative bearing), it was found that 4 out of 7 bridges in the study had some kind of positive panel bearing -grout or concrete-. And all the major deterioration is linked to negative bearing created due to original design or construction inaccuracy.

Table 4.10 provides a summary of the most significant findings in each bridge included in the study. These findings cover all the typical deck top deficiencies in deck panel bridges.
### Table 4.10  Forensic Study Summary

<table>
<thead>
<tr>
<th>Bridge #</th>
<th>Study Date</th>
<th>New Finding</th>
<th>Verification</th>
</tr>
</thead>
<tbody>
<tr>
<td>130075</td>
<td>5/04</td>
<td>Epoxy Bearing Repair Cond. Cracking on M1 Repair</td>
<td>Longitudinal Deck Surface Cracking.</td>
</tr>
<tr>
<td>100415</td>
<td>6/04</td>
<td>Different types of bearing (Positive Negative)</td>
<td>Longitudinal Deck Surface Cracking. Deck Spalling and Delamination.</td>
</tr>
<tr>
<td>100398</td>
<td>6/04</td>
<td>Positive Panel Bearings</td>
<td>Longitudinal Deck Surface Cracking.</td>
</tr>
<tr>
<td>100417</td>
<td>7/04</td>
<td></td>
<td>Longitudinal Deck Surface Cracking. Positive Panel Bearings</td>
</tr>
<tr>
<td>130085</td>
<td>8/04</td>
<td>Grout + Epoxy Bearing Repair.</td>
<td>Longitudinal Deck Surface Cracking</td>
</tr>
</tbody>
</table>

From the previous studies it was possible to obtain valuable information regarding the deterioration of deck panel bridges. This information would have been very difficult to obtain only from lab tests.
CHAPTER 5. FAILURE MECHANISM

5.1 Introduction

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.

5.2 Deck Failure Mechanism Model

5.2.1 Stage #1 Initial Condition

The first stage is the initial condition of the bridge after being 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 – Relative 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 the 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.

5.2.2 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 (Fig. 5.1). Following the formation of longitudinal cracking sporadic transverse cracks can also develop due only to differential shrinkage.

![Diagram of Deterioration Stage #2](image)

**Figure 5.1** Deterioration Stage #2
5.2.3 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 bearing.

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

The 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 cause by the vertical cracking described in section 5.2.3. It was also found that this reduction is affected by the shape of the vertical crack. Fig. 5.2 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.

![Figure 5.2 Effect of Vertical Crack Shape in Shear Reduction](image_url)
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. 5.2). This shear failure is manifested by the appearance of a diagonal cracking emanating from the corner of the precast panel and 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. 5.3 (a)).

Figure 5.3 Shear Failures for Different Degrees of Shear Reduction
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 too high, the rebar acts like a “crowbar” in to concrete on top of the rebar, creating delamination in that area. (See Fig. 5.3(b)).

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.

![Figure 5.4 Deterioration Stage #3](image-url)
5.2.4 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 5.6  Deterioration Stage #4
5.2.5 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 5.7 Deterioration Stage #5](image-url)
5.2.6 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.

![Diagram of Deterioration Stage #6]

**Figure 5.8** Deterioration Stage #6
5.2.7 Stage #7 M1 Repair

Generally after several patch and re-patch processes 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 it is replaced by repair material. To do this, the edges of the section to be removed are cut and the concrete inside is removed using a jack hammer. Usually the intent is to remove 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) are the panel shear connectors embedded in the M1 Repair (8) How quickly was the repaired section opened to traffic. And finally the most important: (9) was the fiberboard removed and replaced with non shrink epoxy?

![Figure 5.9 M1 Repair Procedure (Stage #7)](image-url)
Figure 5.10  Deterioration Stage #7
5.2.8 Stage #8 Shear Failure Cracking Adjacent to an M1 Repair

Assuming that the panel bearing wasn’t replaced by epoxy, or that it was replaced but due to construction problems there is no full support of the panel by the epoxy, the deck area adjacent to an M1 repair, starts the deterioration process again with the appearance of the additional parallel cracking – shear failure cracking - described in (section 5.2.3). The parameters that affect the occurrence of this additional deterioration case are the same mentioned for stages #3 and #7.

![Diagram of Deterioration Stage #8](image_url)
5.2.9 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 5.12** Deterioration Stage #9
5.2.10 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 the wheels over it. Impact can also cause growth of adjacent spalls, and delamination between the panel surface and the M1 repair.

![Diagram showing the effects of cracking on M1 repair and adjacent spalling increase.](image)

**Figure 5.13** Deterioration Stage #10
5.2.11 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 generating redistribution of stresses to adjacent areas. Also in the case where no positive bearing has been provided to the panel, it will experience settlement every time the deck section is loaded due to the lack of stiff support.

![Figure 5.14 Deterioration Stage #11](image)
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, (4) degree of disrepair of the precast panel and the M1 repair.

5.2.12 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 5.15  Deterioration Stage #12
5.2.13 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 5.16 Deterioration Stage #13
5.3 Summary

The deck deterioration model described in this chapter can be summarized in the following failure tree.

**Figure 5.18** Deck Failure Tree
CHAPTER 6. SUMMARY AND CONCLUSIONS

6.1 Summary

The goal of this study was to identify the deterioration process and failure mechanism of deck panel bridges in Florida.

Typical deficiencies on deck panel bridges are described in Chapter 2. This information was collected from FDOT’s deck inspection reports available since the construction of the bridges to date.

Between 2000 and 2003, localized failures occurred in five panel bridges in Districts 1 and 7. Relevant information relating to these failures was collected and analyzed with the intent of identifying underlying trends.

Forensic investigations were carried out on seven deck panel bridges scheduled for replacement during 2003-2004 and located within easy driving of the USF campus. The objective was to obtain first hand evidence of deterioration and to gain understanding of the degradation process.

All the information collected was used to develop a deck failure model, and to identify the parameters that affect the structural behavior of the deck.
6.2 Localized Failures

6.2.1 Failure Trend

National Bridge Inventory deck condition rating was found to be a poor indicator for predicting panel deck 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.

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.

6.2.2 Environmental Factors

In four out of the five cases there was rainfall prior to failure. The most severe rainfall preceded the last failure (1.1 in.). The exact role of rainwater is not known. However, given that the concrete in the deck separates cleanly from the reinforcement, 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.
6.2.3 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 at the edge or corner panels whose boundaries developed reflective longitudinal and transverse cracking.

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

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

6.2.4 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 1.6).

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).
6.3 Forensic Investigation

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-thru deck failures. The lack of positive bearing can occur due to two main reasons:

1. When the initial deck design indicates the use of only fiberboard as bearing material for the panels (Fig.1.3).
2. Or in the case 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 FDOT Districts 1 and 7 were built using negative panel bearing (panel supported by fiberboard only) as originally thought. Four out of seven bridges covered in the study where built using positive panel bearings.

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

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.
6.4 Deterioration Model

Even though a deterioration model and a deck failure mechanism for deck panel bridges was successfully developed in this study, it is still difficult to accurately predict the future condition of different deck panel bridges using the model because most of the factors that influence how fast the deterioration can occur are very difficult to quantify.

1. Construction details: Type of panel bearing used (positive – negative), deck thickness (in most of the bridges real “as built” plans are not available).
2. Deck construction quality: Actual deck thickness, concrete quality (this can influence the formation of creep and shrinkage cracks), concrete cover for top rebars, panel length (does it have the right length to be supported by the grout) for positive bearing construction.
3. How is the shear capacity of the deck affected after the appearance of creep and shrinkage cracks? (see Fig. 5.2). This is very difficult to quantify.
4. Specifications and quality of previous deck repairs (epoxy panel bearing replacement, M1 repairs, spall patches). WHAT DOES THIS MEAN

6.5 Recommendations for Bridge Deck Replacement Prioritization

As mentioned in 6.4, to develop an efficient deck replacement prioritization plan for deck panel bridges, based on the prediction of the future structural behavior is not feasible.

The recommendation we can give after conducting this study is to conduct a more in-depth search in order to quantify the two most important factors that affects the condition of the bridge decks. These are the type of panel bearing detail used in each bridge (positive – negative bearing) and deck thickness. Even though searches were conducted at FDOT District 1 and 7 Bridge Maintenance office, these details were not found. These may be available in construction records archive at another FDOT office.
After identifying those parameters, this is the recommended approach for deck replacement prioritization:

**Table 6.2  Deck Replacement Prioritization Approach**

| Priority | Initial Bearing detail  
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(Fig. 1.3)</td>
</tr>
<tr>
<td>High</td>
<td>Negative</td>
</tr>
<tr>
<td></td>
<td>No</td>
</tr>
<tr>
<td></td>
<td>Heavy deterioration *</td>
</tr>
<tr>
<td></td>
<td>Replace the entire deck</td>
</tr>
<tr>
<td>Negative</td>
<td>Not in the entire deck, or too thin &lt; ¾ “(Fig. 4.20)</td>
</tr>
<tr>
<td></td>
<td>Heavy deterioration *</td>
</tr>
<tr>
<td></td>
<td>Replace the entire deck</td>
</tr>
<tr>
<td>Negative</td>
<td>Yes and more than ¾” thick (Fig. 4.16 d)</td>
</tr>
<tr>
<td></td>
<td>Deterioration in isolated locations only</td>
</tr>
<tr>
<td></td>
<td>Replace the entire deck</td>
</tr>
<tr>
<td>Positive</td>
<td>No</td>
</tr>
<tr>
<td></td>
<td>Deterioration in isolated locations only (Fig 4.40)</td>
</tr>
<tr>
<td></td>
<td>If feasible, replace only deteriorated deck sections covering entire bay</td>
</tr>
<tr>
<td>Negative</td>
<td>Yes and more than ¾” thick (Fig. 4.16 d)</td>
</tr>
<tr>
<td></td>
<td>Only typical cracking</td>
</tr>
<tr>
<td></td>
<td>Replacement may not be required, unless new deterioration appears</td>
</tr>
<tr>
<td>Low</td>
<td>Positive</td>
</tr>
<tr>
<td></td>
<td>Yes and more than ¾” thick (Fig. 4.41)</td>
</tr>
<tr>
<td></td>
<td>Only typical cracking</td>
</tr>
<tr>
<td></td>
<td>Replacement may not be required,</td>
</tr>
</tbody>
</table>

* Assume also typical longitudinal and transverse cracking.

**6.6 Future Work**

In order to fully validate the deterioration model developed in this study, each deterioration stage should be analyzed in detail using finite element analysis. The objective is to obtain additional information about the model that could not be obtained from forensic examination.
The numerical analysis of the model will be conducted by Dr. Niranjan Pai, Postdoctoral Fellow at the University of South Florida, as part of an FDOT research project. This will lead to the development of a deck replacement prioritization scheme for the panel deck bridges remaining in FDOT Districts 1 and 7.
REFERENCES


[2] ACI 318-2002, American Concrete Institute, Farmington Hills, MI.


APPENDICES
APPENDIX A: 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:

\[ b = 20 \text{ in} \]
\[ l = 10 \text{ in} \]

Determination of shear strength: As per ACI 11.12.2.1

Shear strength of concrete \( V_c \) is smallest of the following

\[ V_{c1} = \left[ 2 + \frac{4}{\beta_c} \right] \sqrt{f'_c b_o d} \quad \text{(Equation 11-33)} \]
\[ V_{c2} = \left[ \alpha_s \frac{d}{b_o} + 2 \right] \sqrt{f'_c b_o d} \quad \text{(Equation 11-34)} \]
\[ V_{c3} = 4 \sqrt{f'_c b_o d} \quad \text{(Equation 11-35)} \]

Where,

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

\( \beta_c \) = ratio of long side to short side of the concentrated area

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

\( \alpha_s \) = 20 (corner) \quad \alpha_s = 40 \text{ (center)}
Appendix A, (Continued)

Case 1 Full Composite Action (Corner)

![Figure A.1 Shear Failure Detail (Corner)](image)

-Cast in place slab

\[ f'_c_{\text{CIP}} = 3000 \text{ psi} \quad \quad \quad \quad \quad \quad d_{\text{CIP}} = 4 \text{ in} \]

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

\[ b_0 = \left[ 20 + \left( \frac{4}{2} \right) \right] + \left[ 10 + \left( \frac{4}{2} \right) \right] \quad b_0 = 34 \text{ in} \]
Appendix A, (Continued)

\[ V_{c1} = \left( 2 + \frac{4}{\beta_c} \right) \times \sqrt{\left( f'_{c_{-\text{CIP}}} \right) b_0 \times d_{\text{CIP}}} \]

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

\[ V_{c2} = \left( \alpha_s \cdot d_{\text{CIP}} + 2 \right) \times \sqrt{\left( f'_{c_{-\text{CIP}}} \right) b_0 \times d_{\text{CIP}}} \]

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

\[ V_{c3} = 4 \times \sqrt{\left( f'_{c_{-\text{CIP}}} \right) b_0 \times d_{\text{CIP}}} \]

\[ V_{c3} = 4 \times \sqrt{\left( 3000 \right) 34 \times 4} \quad V_{c3} = 29.8 \text{ kip} \quad \text{(Eq. 11-35)} \]

Shear strength of cast in place slab \( V_{c_{-\text{CIP}}} = 29.8 \text{ kip} \)

**Precast deck panel**

\( f'_{c_{-\text{pan}}} = 5000 \text{ psi} \)

\( d_{\text{pan}} = 2.56 \text{ in} \)

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

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

\[ V_{c1} = \left( 2 + \frac{4}{\beta_c} \right) \times \sqrt{\left( f'_{c_{-\text{pan}}} \right) b_0 \times d_{\text{pan}}} \]

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

\[ V_{c2} = \left( \alpha_s \cdot \frac{d_{\text{pan}}}{b_0} + 2 \right) \times \sqrt{\left( f'_{c_{-\text{pan}}} \right) b_0 \times d_{\text{pan}}} \]

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

\[ V_{c3} = 4 \times \sqrt{\left( f'_{c_{-\text{pan}}} \right) b_0 \times d_{\text{pan}}} \]

\[ V_{c3} = 4 \times \sqrt{\left( 5000 \right) 36.56 \times 2.56} \quad V_{c3} = 26.5 \text{ kip} \quad \text{(Eq. 11-35)} \]
Appendix A, (Continued)

Shear strength of pre-cast panel \( V_{c_{\text{panel}}} = 26.5 \text{ kip} \)

Total composite deck punching shear strength

\[
V_{\text{comp}} = V_{c_{\text{panel}}} + V_{c_{\text{CIP}}}
\]

\( V_{\text{comp}} = 56.3 \text{ kip} \)

Case 2. No Composite Action (Corner)

}\[
\beta_c = 2 \quad f'_{c_{\text{pan}}} = 5000 \text{ psi} \quad \text{Min } d_{\text{pan}} = 2.06 \text{ in} \quad \alpha_s = 20 \quad \text{(corner)}
\]

Figure A.2 Shear Failure Detail (No Composite - Corner)
Appendix A, (Continued)

\[
b_0 = \left( 1 + \frac{d_{pan}}{2} \right) + \left( b + \frac{d_{pan}}{2} \right)
\]

\[
b_0 = \left( 10 + \frac{3.06}{2} \right) + \left( 20 + \frac{2.06}{2} \right) \quad b_0 = 32.06 \text{ in}
\]

\[
V_{c1} = \left( 2 + \frac{4}{\beta_c} \right) \times \sqrt{\left( f'_{c_{, \text{pan}}} \right)} b_0 \times d_{pan}
\]

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

\[
V_{c2} = \left( \frac{2\times2.06}{32.06} + 2 \right) \times \sqrt{\left( f'_{c_{, \text{pan}}} \right)} b_0 \times d_{pan}
\]

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

\[
V_{c3} = 4 \times \sqrt{\left( f'_{c_{, \text{pan}}} \right)} b_0 \times d_{pan}
\]

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

Shear strength of pre-cast panel \[ V_{c_{, \text{rib}}} = 15.3 \text{ kip} \]
Appendix A, (Continued)

Case 3 Full Composite Action (Edge)

Figure A.3  Shear Failure Detail (Edge)

-Cast in place slab

\[ f'_{c_{CIP}} = 3000 \text{ psi} \quad \text{and} \quad d_{CIP} = 4 \text{ in} \]

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

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

\[ b_0 = 58 \text{ in} \]
Appendix A, (Continued)

\[ V_{c1} = \left( 2 + \frac{4}{\beta_c} \right) \times \sqrt{f'_{c_{-CIP}}} b_0 \times d_{CIP} \]

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

\[ V_{c2} = \left( \frac{\alpha_c \cdot d_{CIP}}{b_0} + 2 \right) \times \sqrt{f'_{c_{-CIP}}} b_0 \times d_{CIP} \]

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

\[ V_{c3} = 4 \times \sqrt{f'_{c_{-CIP}}} b_0 \times d_{CIP} \]

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

Shear strength of cast in place slab \( V_{c_{-CIP}} = 49.1 \text{ kip} \)

-Precast deck panel

\( f'_{c_{-pan}} = 5000 \text{ psi} \quad \text{d}_{pan} = 2.56 \text{ in} \)

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

\[ b_0 = 2 \left[ 20 + \left( \frac{4}{2} \right) + \left( \frac{2.56}{2} \right) \right] + \left[ 10 + 2 \left( \frac{4}{2} \right) + 2 \left( \frac{2.56}{2} \right) \right] \quad b_0 = 63.12 \text{ in} \]
Appendix A, (Continued)

\[ V_{c1} = \left( 2 + \frac{4}{\beta_c} \right) \times \sqrt{f_{c_{-pan}}'} b_0 \times d_{pan} \]

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

\[ V_{c2} = \left( \frac{\alpha_s \cdot d_{pan}}{b_0} + 2 \right) \times \sqrt{f_{c_{-pan}}'} b_0 \times d_{pan} \]

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

\[ V_{c3} = 4 \cdot \sqrt{f_{c_{-pan}}'} b_0 \times d_{pan} \]

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

Shear strength of pre-cast panel \( V_{c_{-panel}} = 41.4 \text{ kip} \)

**Total composite deck punching shear strength**

\[ V_{\text{comp}} = V_{c_{-panel}} + V_{c_{-CIP}} \]

\[ V_{\text{comp}} = 92.2 \text{ kip} \]
Appendix A, (Continued)

Case 4. No Composite Action (Edge)

![Diagram of Shear Failure Detail (No Composite - Edge)](image)

**Figure A.4** Shear Failure Detail (No Composite - Edge)

\[
\beta_c = 2 \quad f'_{\text{c, pan}} = 5000 \text{ psi} \quad \text{Min } d_{\text{pan}} = 2.06 \text{ in} \quad \alpha_s = 20 \text{ (corner)}
\]

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

\[
b_0 = \left( 10 + 2 \frac{3.06}{2} \right) + 2 \left( 20 + \frac{2.06}{2} \right) \quad b_0 = 54.12 \text{ in}
\]
Appendix A, (Continued)

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

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

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

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

\[ V_{c3} = 4 \times \sqrt{f'_{c,\text{pan}}} \times b_0 \times d_{\text{pan}} \]

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

Shear strength of pre-cast panel \[ V_{c,\text{rib}} = 21.7 \text{ kip} \]
Location of retrieved deck sections used for coring.

Deck sections #2 and #6 were rejected for the study due to heavy damage incurred during the removal process.

Figure B.1  I-75NB over Moccasin Wallow Bridge
### Table B.1  Core Details of Deck Section # 1

<table>
<thead>
<tr>
<th>CORE</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>1-1</strong></td>
<td>This section intercepts a crack 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><strong>1-2</strong></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>
</tbody>
</table>
Table B.1 (Continued)

<table>
<thead>
<tr>
<th>CORE</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-3</td>
<td>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 four small pieces and signs of water infiltrating the crumbled concrete were present. Total core thickness 7 5/8 in</td>
</tr>
<tr>
<td>1-4</td>
<td>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</td>
</tr>
</tbody>
</table>
Appendix B, (Continued)

**Table B.2** Core Details of Deck Section # 3

<table>
<thead>
<tr>
<th>CORE</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>3-1</td>
<td>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</td>
</tr>
</tbody>
</table>
# Appendix B, (Continued)

## Table B.3  Core Details of Deck Section # 4

<table>
<thead>
<tr>
<th>CORE</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>4-1</strong></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 is not in small pieces and there are no signs of spalling or delamination on the deck surface. Total core height is 8 in.</td>
</tr>
<tr>
<td><strong>4-2</strong></td>
<td>This core was taken over a transverse joint. A hairline crack extends 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>
</tbody>
</table>
Table B.3 (Continued)

<table>
<thead>
<tr>
<th>CORE</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>4-3</td>
<td><img src="image1.png" alt="Core Image" /></td>
</tr>
<tr>
<td>4-4</td>
<td><img src="image2.png" alt="Core Image" /></td>
</tr>
</tbody>
</table>
## Table B.4  Core Details of Deck Section # 5

<table>
<thead>
<tr>
<th>CORE</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>5-1</strong></td>
<td>This core was rejected due to heavy damage incurred during extraction.</td>
</tr>
<tr>
<td><strong>5-2</strong></td>
<td>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</td>
</tr>
</tbody>
</table>
### Table B.4 (Continued)

<table>
<thead>
<tr>
<th>CORE</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>5-3</td>
<td>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.</td>
</tr>
<tr>
<td>5-4</td>
<td>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.</td>
</tr>
<tr>
<td>5-5</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>
</tbody>
</table>
### Table B.4 (Continued)

<table>
<thead>
<tr>
<th>CORE</th>
<th>DESCRIPTION</th>
</tr>
</thead>
</table>
| 5-6   | 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. |
| 5-7   | 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. |