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MDOT

Causes & Cures for Prestressed Concrete I-Beam End Deterioration

August 2002

Wayne State University

Michigan Tech

Center for Structural Durability
A Michigan DOT Center of Excellence

Research

TESTING AND RESEARCH SECTION
CONSTRUCTION AND TECHNOLOGY DIVISION
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16. Abstract

Prestressed concrete (PC) for highway bridges was first introduced in Michigan in the 1950s. In 2002, Michigan had over 1900 prestressed box beam and 700 prestressed I-beam structures. A recent study focusing on the condition of Michigan's PC bridges revealed that while most were in fair or better than fair condition, older bridges are showing signs of deterioration, particularly in the ends of I-beam structures. End deterioration needs to be addressed through various inspection and repair techniques for these structures. The specific goals of this research were to a) *develop an inspection procedure* for prestressed concrete I-beam bridges that will clearly distinguish distress severity and disclose potential problems, b) *identify preventive maintenance* strategies to extend the service life of prestressed concrete I-beam ends, and c) *evaluate repair techniques* for I-beam ends to avoid performing complete beam replacement.

The information presented here begins with field survey inspection data and results of a multi-state survey to determine nationwide practices for inspection and repair of prestressed concrete I-beam ends. Analytical studies incorporated extensive field inspection data and showed beam-end deterioration to significantly influence the load path through the member to the bearing. An experimental investigation of shrinkage/cracking and adhesion of vertically patched shallow and deep repairs was conducted. Three repair materials were used in patches and specimens were thermally cycled. All repair materials showed cracking larger than 6 mils and no material met the minimum bond performance criteria of 400 psi. A master listing of suggested preventative maintenance and repair techniques is provided.

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CAUSES & CURES FOR PRESTRESSED CONCRETE I-BEAM END DETERIORATION

Submitted to the

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Executive Summary

The research study focus was the evaluation and abatement of girder end deterioration in prestressed concrete I-beam bridges. Details of the research conducted, including the quarterly report, notes of research meetings and detailed field inspection data can be found at http://webpages.eng.wayne.edu/durabilitycenter. The initial work task was the documentation of observed girder end condition by inspecting twenty prestressed concrete I-beam bridges constructed in Michigan between 1961 and 1998. The following table documents the list of inspected bridges and summary of girder end condition. Also, in the following table "corr" and "del" are abbreviations for corrosion and delamination, respectively.

Condition Summary

Bridge ID	County	Region	Year Built	No. of Spans	No. of Girders	Total No. of Beam-ends Inspected	% Beam-ends w/ No Cracks & No Corr	% Beam-ends w/ cracks	% Beam-ends w/ cracks & corr	% Beam-ends w/ del	% Beam-ends w/ spall	Sum (%)
29011 S03	Gratiot	Bay	1961	3	27	54	0	0	28	39	33	100
06111 S04	Arenac	Bay	1968	3	18	35	0	4	21	34	40	100
06111 S05	Arenac	Bay	1968	3	15	30	0	17	16	17	51	100
06111 S06	Arenac	Bay	1968	3	15	30	0	2	17	13	67	100
06111 S11	Arenac	Bay	1968	6	54	62	0	15	19	31	35	100
25042 S12-8	Genesee	Bay	1969	4	16	28	0	26	26	17	31	100
25042 S12-3	Genesee	Bay	1969	4	22	44	0	7	33	30	30	100
25042 S12-4	Genesee	Bay	1969	4	22	43	0	3	33	27	37	100
25042 S12-7	Genesee	Bay	1969	4	16	32	0	19	5	10	66	100
25132 S34	Genesee	Bay	1971	4	24	48	4	4	18	14	61	100
41025 S07	Kent	Grand	1961	4	24	47	0	0	9	11	80	100
41027 S06	Kent	Grand	1963	3	36	71	0	0	7	12	81	100
41029 S16-3	Kent	Grand	1964	3	24	46	0	0	13	16	72	100
41029 S16-4	Kent	Grand	1964	3	24	48	0	21	21	35	23	100
41029 S23	Kent	Grand	1972	3	24	48	0	58	6	10	25	100
67016 S09	Oceola	North	1984	1	6	11	0	18	18	55	9	100
67016 S10	Oceola	North	1984	1	7	14	0	64	14	7	14	100
53034 S05	Mason	North	1986	4	24	27	0	19	52	22	7	100
83033 S06	Wexford	North	1997	1	8	16	0	94	6	0	0	100
83033 S05	Wexford	North	1998	2	8	16	13	81	6	0	0	100
Total					414	750	0	17	18	20	44	100

Upon documenting the prestressed concrete I-beam end condition in Michigan a survey of State Departments of Transportation was conducted to document the observations in other states. A survey return-rate of 40 percent was achieved with 20 states responding. Responding states were located across the country. Two of Michigan's five neighboring states responded to the survey. One survey respondent indicated that the state did not have a prestressed concrete I-beam end deterioration problem and did not include a completed survey. Over 70 percent of the respondents indicated that they use some unique internal software for management of their state's bridge structural / safety data. All respondents indicated they do not gather specific inspection data on prestressed concrete beam end conditions. It was unclear from the survey responses if any states are using existing documentation (reports, etc.) to aid in the preventive maintenance of prestressed concrete I-beams.

While most states have not repaired prestressed I-beams for end deterioration, roughly 50 percent of the respondents indicated that their state DOT specifications would be used in the rehabilitation of prestressed concrete I-beam ends. The only responding state to indicate that I-beam end repair has been attempted was Michigan.

An inspection procedure is proposed specifically for early identification of prestressed concrete I-beam ends prone to deterioration. The inspection technique is based on data collected visually (at an arms length). In reviewing the observations and data obtained during the inspections, there are three inspection items of importance that are related to I-beam end deterioration. These items are presence of beam end cracking, bearing condition and beam end restraints, and drainage and expansion joint condition.

Categories of condition of deteriorated prestressed concrete beam-ends have been developed to define a distress level and an associated preventative maintenance or repair technique. Condition states for a prestressed concrete I-beam also describe the progression of distress at the beam-end with time. The information shown in the table below has been assembled to assist an inspection crew with accurately assessing the condition of a beam-end.

Condition States of Prestressed Concrete I-Beam Ends

Rating	Condition State
1	No cracks observed, no staining
2	Efflorescence, water-stains, and/or corrosion
3	Hairline Cracks. They can be horizontal, vertical, and/or diagonal
4	Map Cracks
5	Hairline Cracks with efflorescence, water-stains, and/or corrosion with a horizontal crack propagating from the sole plate
6	Cracked and Deformed Neoprene Pad, probably non-functional
7	Moderate Cracks
8	Moderate Cracks with efflorescence, water-stains, and/or corrosion
9	Major Cracks with efflorescence, water-stains, and/or corrosion
10	Delamination with Moderate and/or Major Cracks
11	Spall, Delamination, Corrosion, and Cracks
12	Spall, Exposed Reinforcement, and Corrosion

The study identified four major families of preventive maintenance approaches that can be applied to beam-ends. These techniques were: structure modification, surface insulating methods, electrical control methods, and environment modifying methods. The study developed three analysis tools for successfully executing a beam-end repair project. The tools are: testing procedures and distress severity criteria for PC I-beam end deterioration, cause-evidence relationships for beam end distress and an example performance matrix for preventive maintenance techniques.

The finite element modeling of a PC I-girder is performed to evaluate the causes of observed beam-end distress. The discrete beam analysis identified the effects of prestressing loads, and design changes with respect to tendon geometry and arrangement. Three types of prestressed concrete I-beams were modeled from existing bridges to determine the cause for initial bursting cracks at the end zones. The first model is a beam with straight tendons, the second beam is a Wisconsin type with and without bond breakers, and the third beam is with draped tendons. The stress formation at the end zones and cracking potential are studied. The table below describes the calculated shear and uniaxial stress in the girder ends. In all cases cracking is anticipated by high shear and tension stresses upon release of tendons.

Stresses in Prestressed I-Beam Ends Under Prestressing Loads

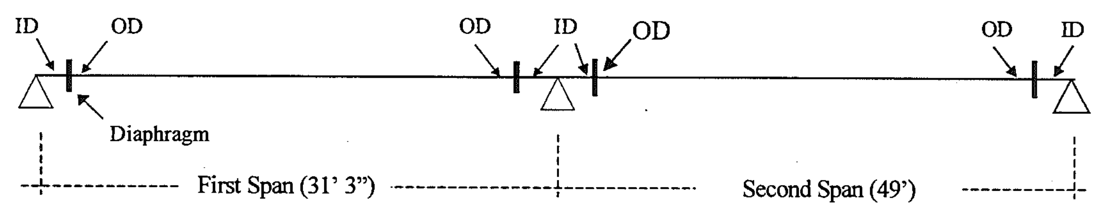
Girder Type Tendons		Maximum Shear Stress (ksi)	Axial Stress (ksi)	
			Compression	Tension
Straight (Uniform) Tendon		0.8	3.0	0.4
Draped Tendon		1.5	3.2	>f_{ct}
Bond-breakers	With Bond-breakers	3.0	5.4	>f_{ct} (424 psi)
	Without Bond-breakers	4.0	>>f_c' (5,000 psi)	>f_{ct} (424 psi)

The structural interactions between the bridge members and the load-transfer mechanism to the beam-ends is analyzed on a full bridge model. The analytical modeling is performed on the bridge with ID 06111-S04. The three-span bridge has exterior spans of 31 feet 3 inches in length and a mid-span of 49 feet in length. The bridge deck has a uniform width of 43 feet 2 inches with two lanes. The minimum deck thickness is 8 inches. Two types of AASHTO prestressed I-girder types, Type III and Type I, are used. Type III girders are located in the mid-span and as fascia girders in the exterior spans. Type I girders are located in the exterior spans as interior girders. The girders are designed as simply supported. The structural behavior of a bridge under several service-loading stages is analyzed. The impact of the diaphragms on beam-ends is investigated by describing diaphragms with different geometry and cross sections and having different material properties. The following table summarizes the shear and moments near beam-ends with various diaphragm types under various load conditions. It is seen that change in the stresses at the beam-ends are insignificant with the use of different diaphragms.

Full Bridge Analysis Results near the Beam-ends

Without Diaphragms			With Diaphragms						Steel Bracing						Loading			
Dead Load			Dead Load			Live Load			Dead Load			Live Load						
V (k)	M (in-k)	T (in-k)	V (k)	M (in-k)	T (in-k)	V (k)	M (in-k)	T (in-k)	V (k)	M (in-k)	T (in-k)	V (k)	M (in-k)	T (in-k)				
10	82	1	12	105	0	28	230	8	11	92	1	17	142	10	ID	A	Exterior	First Span Beams
8	193	3	8	242	7	8	518	18	8	206	3	9	282	26	OD			
8	193	3	8	211	11	9	301	27	8	206	3	9	275	23	OD	P		
10	82	1	11	91	1	15	126	6	11	92	1	16	136	9	ID			
12	106	0	11	95	0	21	181	9	12	102	0	30	253	10	ID	A	Interior	
10	243	1	11	221	0	31	440	24	10	238	1	30	625	26	OD			
10	243	1	10	237	0	19	397	5	10	238	1	20	413	5	OD	P		
12	106	0	12	103	1	22	182	0	12	102	0	21	172	1	ID			
16	137	2	18	150	1	NA	NA	NA	18	149	0	NA	NA	NA	ID	P	Exterior	Middle Span Beams
14	326	6	15	352	4	NA	NA	NA	15	347	1	NA	NA	NA	OD			
19	164	1	19	158	0	NA	NA	NA	19	158	0	NA	NA	NA	ID	P	Interior	
17	383	4	16	370	0	NA	NA	NA	16	372	0	NA	NA	NA	OD			

A: Abutment end
ID: Inside of Diaphragm (Span Side)
OD: Outside of Diaphragm (Beam-end Side)
M: Moment
NA: Not Available
P: Pier-cap end
T: Torsion
V: Shear Force



Forms of distress at the beam-ends include concrete spalling, delamination, cracking, and corrosion of reinforcement. The loss of concrete permits accelerated deterioration of reinforcing and prestressing steels, allows detensioning of prestressing steel, and increases the stress demand (bearing, shear, flexural) on the remaining section. Properly functioning repairs can restore cover to reinforcing and prestressing steels and re-establish the original intended cross section of the concrete. The two of the most important properties of concrete repair are crack resistance and substrate adhesion (bond). Crack resistance is needed to prohibit ingress of contaminants that can adversely affect the performance of the repair. Adhesion is required to assist the parent member in carrying loads as well as protecting the parent member (or repair) steel reinforcement from corrosion. A performance evaluation of vertical repair material was conducted and focused on evaluating crack development and repair bond tensile strength at the conclusion of a thermal cycling period. The performance measure for maximum repair cracking width for this study was 6-mils. Visual observations of the repair condition at the conclusion of the post-curing period revealed cracking within the repair material itself and at the repair-substrate joints (i.e. top and bottom repair joints). Not considering the cracking at these joints, some observations made were:

- All brands of repair material showed cracking greater than 6-mils in width.
- Those specimens with fine-width (2-mil) pattern cracking generally did not exhibit cracks within the repair greater than 6-mils in width.
- Those specimens repaired with materials produced with a liquid polymer exhibited more cracking than the material mixed with potable water.
- Repair depth (1-inch or 3-inch) did not have an impact on frequency of cracked specimens relative to the total number of specimens tested. About 1/3 of each repair depth group exhibited cracking greater than 6-mils.
- Post-curing environment showed 43% of specimens exceeded the 6-mil performance measure, whereas 28% of ambient cured specimens exceeded the measure.

For bond tensile strength, two sets of performance measures were observed. First, repairs cannot delaminate from the substrate and second, a bond tensile strength of 400-psi was required. Over 1/3 of the repair specimens did not meet the delamination performance criteria and none of the specimens were able to develop a bond tensile strength of greater than 400-psi.

The focus for the next phase of work should be related to safety assessment. In order to understand the relation between girder-end condition states and their load performance, the proposed approach is to development an analytical model for various bridge types calibrated by the full scale testing of decommissioned girders. The experimental component is further divided into laboratory experiments for condition characterization of in-service girders and field-testing of full-scale girders. Evaluation of actual beam structural capacity is needed to truly understand the damage due to deterioration. Full scale testing results of individual beams in flexure and shear can give insight into the actual strength reduction and can be compared with analytical models. Corresponding models can then be used for future bridge analysis of deteriorated structures to more accurately determine the capacity of the structure. In addition, repaired beams can also be tested for effectiveness in restoring strength.

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1.0 Introduction

1.1 Overview

Deterioration of in-service civil engineering infrastructure in the United States has become an issue in recent years (ASCE, 2002). Due in part to its versatility, availability, and economy, portland cement concrete has been selected as the building material of choice for roads, bridges, and dams, among other infrastructure elements. When properly designed and placed, portland cement concrete is also a very durable material (Mindess and Young, 1981). However, portland cement concrete can deteriorate during its service life if proper design and placement practices are not followed or if the service conditions are different from those anticipated. An example of insufficient durability is evidenced in the ends of some in-service prestressed concrete I-beams in Michigan bridges.

The basic concept of prestressed concrete dates back to the late eighteen hundreds, nearly as old as reinforced concrete. However, it was the material properties that prevented the application of prestressed concrete to structures like bridges. It was first Eugene Freyssinet of France who in 1928 first began the use of high strength steel wire for prestressed concrete, and he is known as the originator of practical prestressed concrete (Nawy, 2000). After Freyssinet's original work, prestressed concrete was increasingly utilized in both Europe and North America, and the first prestressed bridge was the Walnut Lane Bridge in Philadelphia, which was finished in 1951.

This bridge type quickly became popular because the beams could be built economically in plants, and their span lengths could compete with that of steel beams. Michigan's first prestressed concrete bridge was built in 1951 (Moore et al, 1970). In 1996, Michigan had 1,696 prestressed box beam and 696 prestressed I-beam structures that were owned and operated by the state (Needham and Juntunen, 1997).

Work performed by others has concluded that in midwestern states, such as Michigan, corrosion is more pronounced in older bridges near the beam-ends (Whiting et al, 1999). This is likely due to the beam-ends being located below poorly maintained deck joints and thus exposed to deicing salts draining from the deck surface. Photo 1-1 shows an example of the end deterioration problem that some of Michigan's prestressed concrete I-beam bridges face.

Corrosion induced deterioration of prestressed concrete bridges are more critical than similar deterioration in a conventionally reinforced portland cement concrete element for several reasons. One reason is that even minor corrosion of prestressing strands may affect the load carrying capacity of beams to a greater degree than in conventional reinforced concrete. Also, because of the relatively small diameter and large number of prestressing strands compared to the fewer number of larger bars that are used in conventionally reinforced concrete, a greater surface area is available for corrosion. Lastly, the increase in reinforcement density and surface area within a prestressed concrete section can lead to increased corrosion activity when compared to a conventionally reinforced member.

Photo 1-1. End deterioration of prestressed concrete I-beam ends

Recent practice in Michigan is to use a continuous-for-live-load (CLL) superstructure design where the diaphragms are cast integral with the deck over the piers. As shown in Figure 1-1, this design eliminates the joints over the piers and eliminates the direct exposure of the prestressing strands at the beam-ends to chlorides. While the current Michigan detail is similar, positive moment reinforcement is not used.

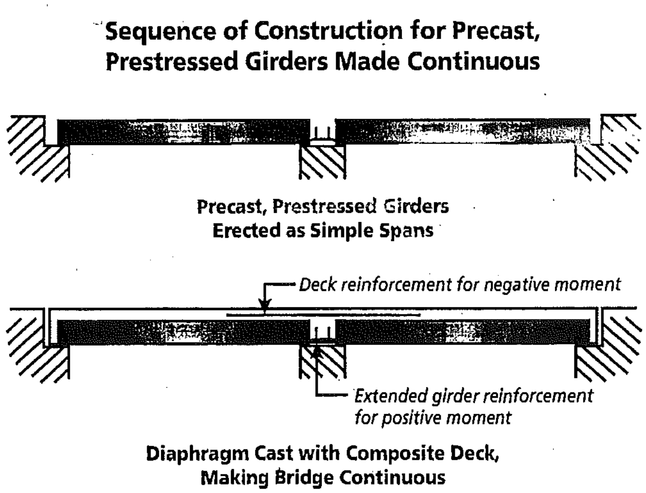

Figure 1-1. Example of Constructing Decks Continuous for Live Load (Rabbat and Aswad, 2002)

If this practice is effective, the corrosion problem will be constrained and corrective actions may be limited to prestressed concrete bridges constructed prior to CLL structures. For the remaining limited group of bridges with joints, there is a potential for corrosion of prestressed strands and mild steel reinforcement. There will be a need for inspection techniques that can identify problems in advance so that corrective action can be taken.

For existing deteriorated prestressed concrete bridges, however, there is a need for effective repair procedures. The Michigan Department of Transportation (MDOT) and the Federal Highway Administration (FHWA) developed one such repair procedure for prestressed concrete I-beams (Needham, 1999). The repair procedure was experimentally verified on one salvaged prestressed I-beam. The procedure was then applied to three in-service structures with some modifications, and it was economically beneficial compared to a total superstructure replacement (Needham, 2000). The effectiveness of this and other similar procedures, however, should be further verified using analytical tools and some experimental demonstrations.

Therefore, for bridges that may be at risk for end deterioration, there is a serious need for:

- **Evaluating** and **assessing** the deterioration level of these prestressed concrete bridges,
- **preventing** further deterioration from occurring, and
- **repairing** prestressed concrete bridges with corroding strands.

Research activity in the field of concrete repair has been on going for over 30 years, however at a relatively low volume, compared to other areas of concrete research. Through the completion of a state-of-the art literature review, multi-state survey, analytical modeling and experimental studies, this study examined the potential deterioration mechanisms that can affect a prestressed concrete beam-end, and what components of concrete durability are required to resist the deterioration. Although several deterioration mechanisms exist, one mechanism dominates all others in terms of frequency: corrosion-induced deterioration by deicer penetration.

1.2 Objectives

This research project, "Causes and Cures for Prestressed Concrete I-Beam End Deterioration," was sponsored by the Michigan Department of Transportation (MDOT) and performed by the Center for Structural Durability, a collaborative effort between Michigan Technological University and Wayne State University, the Research Team. Work has focused exclusively on prestressed concrete I-beams with end deterioration.

There were two main objectives for the research program:

- Development of an inspection procedure for existing prestressed concrete I-beams with potential end deterioration problems, and

- Evaluation of repair techniques for existing deteriorated prestressed concrete I-beam ends.

To satisfy these objectives, the project was organized in two phases with thirteen tasks. Sections 1.3 and 1.4 list the tasks with respect to their appropriate phase as they were presented in the original proposal. Also, any changes to the original tasks are discussed. Each task corresponds to one chapter of this report.

1.3 Phase I

Phase I contained Tasks 1-8 as listed below.

Task 1. State-of-the-Art Literature Review (Chapter 2)

The objective of the literature review was to identify, review, and synthesize information related to prestressed concrete I-beam end deterioration. It has been an ongoing study throughout the project, and the result is a comprehensive chapter describing many different aspects of prestressed concrete I-beams ranging from history of design to inspection, preventive maintenance and repair methods.

Task 2. Field Investigation (Chapter 3)

The objective of the field investigation was to conduct an extensive examination of 20 prestressed concrete I-beam bridges to observe the distress at the beam-ends. Bridges ranged from 2 to 40 years old and were previously FHWA condition rated between 4 (poor) and 8 (very good). The result is a complete documentation of the research team's process of selection of field specimens, field inspection and observations, and data organization.

Task 3. Identify Bridges Prone to Deterioration (Chapter 4)

The objective of this analysis was to identify bridges that are at risk of distress. Information gathered from the field investigation was used to describe why I-beams are in the current state of distress. This chapter thoroughly describes the methodology and results of the analysis.

Task 4. Conduct a Survey of State Highway Officials (Chapter 5)

The objectives of the survey include determining practices that are used for inspecting and repairing prestressed concrete I-beam ends, and identifying reports relating to the evaluation or repair of prestressed concrete I-beam ends. This task was completed in Phase I, and the result is a discussion of the returned surveys.

Task 5. Survey of PC I-beam End Inspection Techniques (Chapter 6)

A significant review of inspection techniques was provided in the literature review. This review, in conjunction with the field inspection of Chapter 3, has resulted in the proposed condition states and corresponding tabulated photos to aid inspectors in identifying the condition of a beam-end.

Task 6. Preventive Maintenance Techniques for Beam-End Deterioration (Chapter 7)

While families of preventative maintenance approaches were identified in the literature review, this chapter focuses on three analysis tools: tests applicable in classifying the severity of end distress, cause/evidence relationships for beam-end deterioration, and a performance matrix to aid in the selection of effective approaches.

Task 7. Project Website (Chapter 8)

The objective of the website was to create a centralized location of information pertaining to the projects that the Center for Structural Durability have completed or currently working on. This task was continually updated through both phases, and the result is a project homepage for the research project "Causes & Cures for Prestressed Concrete I-beam End Deterioration" that provides pertinent information.

Task 8. Interim Report

The Interim Report was submitted on October 25, 2001. It included all information that was completed within Tasks 1-7.

1.4 Phase II

Phase II contained Tasks 9-13 as listed below.

Task 9. Repair Techniques for Beam-End Deterioration (Chapter 9)

The initial task definition was to determine feasible repair/strengthening techniques including their benefit/cost analysis. Initial items within this task included identifying a performance matrix and identifying methods to assign repair techniques to different levels of distress based on literature review (Task 1), inspection and survey observations (Tasks 2-6), analytical study (Task 10) and laboratory testing results (Task 11). This matrix was discussed in Chapter 7, and benefit/cost analysis became beyond the scope/timeline of this project (see Chapter 14). Included in this chapter are basic suggestions for beam end repair, including comments/suggested changes for MDOT related documents.

Task 10. Analytical Modeling (Chapter 10)

The original purpose of this task was to develop an analytical model of a "prestressed concrete I-beam bridge" to be used for evaluating repair techniques. The task evolved to the evaluation of shear stresses during production, load paths near the girder-end under dead and live loads, and diaphragm effects in a bridge load response. Two models were developed (a discrete I-beam model and a full bridge model) using one of the inspected bridge as a reference for design and material characteristics. Models help to identify the cause of end cracking and can be used to potentially propose new reinforcement details or changes to precasting practices.

Task 11. Experimental Study of Repair Materials (Chapter 11)

The initial task definition was to conduct lab testing / implementation of the selected repair techniques based on Phase I outcome including 1) conduct specimen preparation 2) identify surface preparation technique and 3) state visible repair techniques for damage treatment. The outcome of this task was the experimental study of three repair materials used for partial patching of vertical and overhead repairs, and an evaluation of both shrinkage/cracking potential and bond tensile strength for all three materials.

Task 12. Data Analysis

This task has been incorporated into each respective chapter. For example, the analysis of field inspection data was included in Tasks 2 and 3, or Chapters 3 and 4, respectively. Results of the analytical study are included in Task 10 (Chapter 10), and laboratory data analysis is included in Task 11 (Chapter 11).

Task 13. Final Report

This document constitutes the final report. In addition to the above chapters, the following three chapters are included:

Chapter 12 – includes a discussion on a bridge management approach to managing the preventative maintenance and repair of prestressed concrete beam end distress.

Chapter 13 – provides a summary of the significant conclusions of this work.

Chapter 14 – as with any quality research project, many questions can be answered but some remain unanswered. This chapter includes a list of future studies for consideration.

2.0 State-of-the-Art Literature Review (Task 1)

2.1 Objectives and Approach

A state-of-the-art literature review was conducted for this project. The objective of the literature review was to identify, review, and synthesize information related to prestressed concrete I-beam end deterioration. Concentration areas for the review were established for the project, and included:

- Concrete durability and deterioration mechanisms
- Inspection, preventive maintenance, and repair tools for prestressed concrete bridges
- Past and current MDOT practices with prestressed concrete design and repairs

To effectively convey the results of the literature review, other topics related to prestressed concrete were reviewed and have been included in this report.

The resources of the Michigan Tech and Wayne State University libraries were used for this project. In addition to the collections of reference material housed at these facilities, a significant number of references were identified through electronic index searches including National Technical Information Service, Transportation Research Information Service, Engineering Village, and World Cat. An extensive review of information available on the Internet was also conducted. The resources of professional and government organizations were explored through web searches and telephone interviews. These organizations included:

- American Concrete Institute (ACI),
- ASTM International (ASTM),
- Federal Highway Administration (FHWA),
- American Association of State Highway and Transportation Officials (AASHTO),
- National Cooperative Highway Research Program (NCHRP),
- Transportation Research Board (TRB), and
- Michigan Department of Transportation (MDOT).

2.2 Overview of Prestressed Concrete

Prestressed concrete, a product of the twentieth century, announced a significant new direction in structural engineering. The idea of prestressing opened up new possibilities for form. Freyssinet outlined the "conditions for the practical use of prestressing" and had established the need for high strength steel, tensioning it to a high initial stress, and high strength concrete to reduce the loss of initial prestress to a minimum (Billington, 1976).

The first design guide for prestressed concrete was in the form of a recommended practice, rather than a building code. In the United States, the "ACI-ASCE Joint Committee Recommendations for the Design of Prestressed Members" published in 1958, included the state of art knowledge, which had developed with the limited use of prestressed concrete by the mid-1950's.

The Prestressed Concrete Institute, founded in 1954, published the first U.S. Building Code for prestressed concrete in 1961. At that time, the American Concrete Institute (ACI) Building Code Requirements for Reinforced Concrete (ACI, 1956) contained no reference to prestressed concrete, but the inclusion of new material on this subject was being considered for the next revision.

In 1963, the ACI Building Code Requirements for Reinforced Concrete (ACI, 1963) included a chapter covering prestressed concrete, much of which was carried forward into the 1971 revision of the ACI Building Code Requirements for Reinforced Concrete (ACI, 1971). Since 1971, annual revisions have been made. A similar evolution occurred with the AASHTO "Standard Specification for Highway Bridges" (AASHTO, 1996). The provisions for prestressed concrete in the current AASHTO Standard Specification for Highway Bridges are very similar to those of the ACI Code. Some differences are the allowable stress values and load factors that have been traditionally more conservative for bridges than buildings (Lin and Burns, 1981).

Representatives of the Federal Highway Administration (FHWA, formerly the Bureau of Public Roads), AASHTO, and the Prestressed Concrete Institute (PCI) developed a series of standard AASHTO-PCI sections for bridge beams to reduce the cost of design. Standard beam Types I through IV were developed in the late 1950's, and Types V and VI in the early 1960's. Additional modifications were made in the 1980's to further optimize sections (Rabbat and Russell, 1982). The current AASHTO shapes include these modifications (Rabbat and Russell, 1982).

2.3 Durability and Deterioration

There has been great concern about durability, especially in materials for structures that are subjected to harsh environments. Durability is the ability of concrete to resist weathering action, chemical attack, abrasion and other service conditions (ACI, 1990). Considerable emphasis is still given to high quality concrete as the first line of defense against corrosion. The heightened awareness of the importance of durability is evidenced by the ACI Building Code Requirements for Reinforced Concrete (ACI, 1989) that featured a new chapter devoted to the issue of durability.

The ACI 318-89 Code mandated the latest Post Tensioning Institute (PTI) standards for improved corrosion protection of unbonded tendons. Unbonded tendons used in corrosive environments must be completely encapsulated and be further protected with high quality greases. With an extra layer of protection provided by sheathing, post-tensioned structures are more corrosion resistant than pre-tensioned construction (Whiting et al, 1998). Because beam end deterioration has been observed, it suggests that prestressed concrete I-beams lack durability. This statement is only partially true, as prestressed concrete I-beams are subjected to diverse set of environmental and loading conditions.

Environmental conditions are not the only constituent behind the potential deterioration of a member. Concrete, prestressing steel type, and the environment affect whether or not a reinforcing steel will be susceptible to corrosion (Whiting et al, 1993). Concrete needs to have a water/cement ratio less than 0.45 and be free of chloride admixtures. Segregation of the concrete may occur at the bottom of cast members and the resulting concrete may lack acceptable durability (Emmons, 1994). In another example, concrete cover over top of slab reinforcement needs to be greater than 2-inches, per AASHTO, when exposed to deicers (AASHTO, 1996). AASHTO specifies that additional cover beyond the minimum 1.5-inches should be provided for prestressed beams when the contact of deicing agents is unavoidable.

Prestressing steel needs to be from cold-drawn wires; quenched and tempered wires are hard and brittle (Whiting et al, 1993). Whiting et al note that quenched and tempered wires are not approved for use in the United States.

What types of environmental conditions must prestressed concrete I-beams resist? In Michigan, the most common conditions that require concrete durability are thermal cycles, freeze-thaw cycles, exposure to acidic gasses (carbon dioxide), and exposure to deicing solutions. Other states may experience similar conditions (Enright and Frangopol, 2000). However, different aggressive conditions will not be discussed (alkali-silica reaction, delayed ettringite formation, etc.) in detail for this report. It is suggested that concrete deterioration can be categorized as dismemberment, dissolution, or erosion (Emmons, 1994). Forms of concrete distress can vary widely but are mainly dismemberment related for prestressed concrete I-beam end deterioration. This can include cracking, spalling, delaminations, and minor surface damage (Xanthakos, 1996).

2.3.1 Thermal Distortion

Nearly all Michigan bridges are situated in a location that allows fascia beams to be exposed to un-even (diurnal) solar heating. Like all materials, concrete expands when heated and shrinks when cooled. Having a temperature differential on an element such as a prestressed concrete I-beam would cause expansion on the outward-facing side and induce weak-axis bending stresses. Fixity of the top flange may cause an out-of-plumb condition for the beam web and induce additional stress into the member. A partially fixed beam end, such as one created by a frozen bearing may impose additional stress at the beam end. When stress build-up is relieved, tension cracks, shear cracks, or buckling may result (Emmons, 1994).

The coefficient of thermal expansion of the concrete and freedom from restraint affect the amount of thermal distortion related distress. To date, no information is available that suggests thermal distortion causes prestressed concrete I-beam end distress.

2.3.2 Freeze-thaw Deterioration

Without entrained air, the cement matrix surrounding the aggregate particles may fail when it becomes critically saturated and frozen (ACI, 1992). Freeze-thaw deterioration is a function of porosity, moisture saturation, number of freeze-thaw cycles, air entrainment, member surface, and aggregate quality (Emmons, 1994). An important fact to consider regarding freeze-thaw damage is that concrete that is dry or contains only a small amount of moisture is essentially not affected by even a large number of cycles of freezing and thawing (Kosmatka and Panarese, 1988). Evidence of freeze-thaw deterioration is usually in the form of small surface disintegration (Emmons, 1994). Freeze thaw damage is resisted by proper structure design to minimize exposure, low water-cement ratio (*w/c*), appropriate air entrainment, quality materials, adequate curing, and special attention to construction practices (ACI, 1992).

Observations by other researchers indicate that freeze-thaw damage is not presently occurring in Michigan's prestressed concrete I-beam ends (Ahlborn et al, 2001). Past practices have apparently adequately resisted this form of deterioration.

2.3.3 Corrosion-induced Deterioration

Corrosion of reinforcing steel is the single most destructive deterioration mechanism for reinforced concrete bridges in the United States (Weyers et al, 1993). An improved understanding of the influence of corrosion damage upon structural performance would assist owners and operators of structures to plan strategic, cost-effective remedial treatment (Cairns and Millard, 1999). Based on the observations of on-going research, damage to prestressed concrete I-beam ends is largely attributed to corrosion-related damage (Ahlborn et al, 2001). Corrosion can be generally defined as the deterioration of a substance or its properties because of a reaction with its environment (NACE, 1970). Numerous references were reviewed to gain an understanding of how steel reinforcement corrodes in concrete.

2.3.3.1 *Background*

In 1960, NCHRP Report No. 90 stated that corrosion of prestressing in concrete was not and would not be a problem if certain precautions were considered (Moore et al, 1970). These precautions include cover thickness in conformance with AASHTO specifications and protection of prestressing steel during shipment and storage. Issues being addressed by this project clearly indicate that some past practices (e.g. design, construction, and maintenance) have been insufficient and ineffective. Corrosion of metals in concrete has been studied for many years, both in the field and in the laboratory. Whiting et al collaborated and discussed several programs that intended to simulate corrosion of stressed reinforcement in the laboratory by immersion in a solution (Whiting et al, 1993). Researchers have also performed laboratory testing of prestressing steel in concrete to more closely simulate in-situ conditions (Moore et al, 1970). The Moore study also looked at two bridges, 2 and 11 years old. For these relatively young bridges, it was shown that the cement paste around the wires in the prestressing strands was providing corrosion protection.

Corrosion of steel reinforcement in concrete is commonly known to be an electrochemical process, rather than chemical or physical (NACE, 1970). One of two environments must exist for corrosion to occur, either aqueous or in air (Heldt, 2001). For electrochemical corrosion to occur, a cathode and an anode are required along with an electrical conductor and an aqueous medium (ACI, 1996). Any metal surface on which corrosion is taking place is a composite of anodes and cathodes electrically connected through the body of the material itself. Concrete provides the aqueous environment, containing water with dissolved oxygen (ACI, 1996). Figure 2-1 shows the basic corrosion cell than can occur in a section of prestressed concrete (Emmons, 1994).

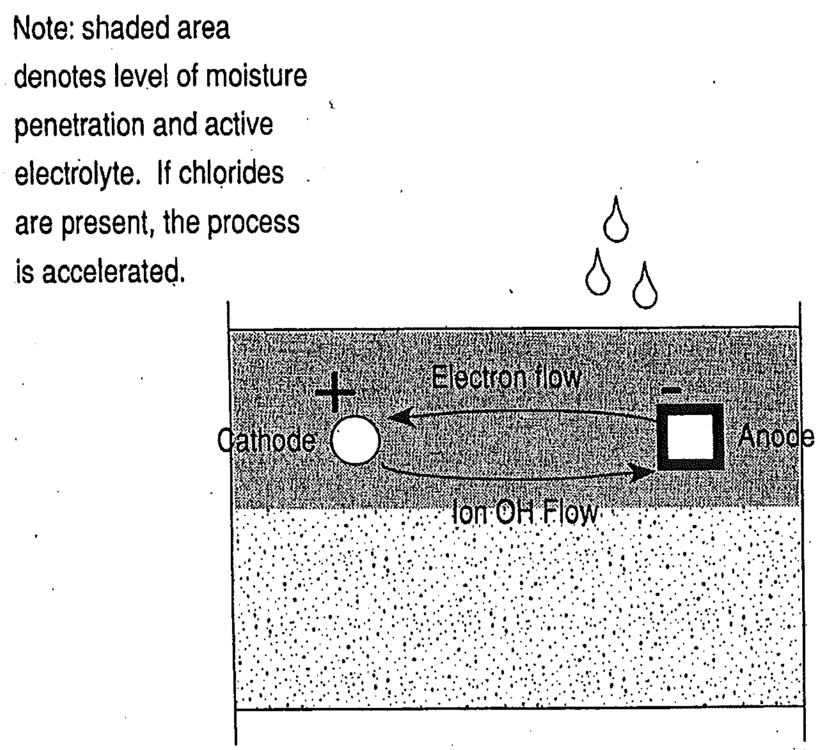

Figure 2-1. Basic Corrosion Cell Between Two Metals in Concrete (Emmons, 1994)

The presence of oxygen is essential for corrosion to occur. Oxygen is introduced into concrete most quickly through several wetting and drying processes. It has been shown that steel within concrete in a continuously wet environment has a slow rate of corrosion, even if the environment is seawater. For oxygen to be consumed in the cathodic half-cell reaction, it must be in an aqueous state. ACI 222R-96 shows that corrosion rate, discussed in later sections of this report, is paralleled with the oxygen concentration (ACI, 1996).

The inherently high pH of concrete is able to sustain a passive hydrated oxide film on the steel reinforcement. The film is actually a corrosion product that acts as a barrier to the anodic dissolution reaction (Jones, 1992). The composition of this thin and invisible film has been difficult to determine, however it is believed to be composed of chemical compounds of oxygen. The presence of the film is able to resist current flow at low voltages (driving forces). Current will flow at higher voltages on steel that has an adhered passive film due to the electrolysis of water (ACI, 1996).

Two types of corrosion are generally recognized in prestressed concrete: stress corrosion and pitting corrosion (Whiting et al, 1993; Leonhardt, 1964). Pitting corrosion is a localized form of galvanic corrosion (Novokshchenov, 1989). Pitting corrosion is most prevalent in reinforced concrete and prestressed concrete structures. Concrete carbonation and chloride ion penetration are the two primary causes of pitting corrosion. The presence of sufficient chloride ion with water, oxygen, and a corrodible metal leads to pitting corrosion. Carbonation reduces the alkalinity in the concrete and may lead to hydrogen embrittlement type of stress corrosion (Whiting et al, 1993). Stress corrosion cracking of chloride-laden concrete is unlikely to occur (Moore et al, 1970). Research by Legat et al showed that corrosion is mainly due to high concentration of chlorides and in a few cases the carbonization of concrete (Legat et al, 1996).

Different types of cements and pozzolans affect the pH of concrete. For example, blended cements may reduce alkalinity but increase electrical resistivity and decrease permeability. Aggregates have been shown to have little effect on corrosion of steel in concrete. However, exceptions are noted for porous aggregates or those aggregates previously exposed to seawater (ACI, 1996).

Dry, unsaturated concrete is more electrically resistive than wet concrete (ACI, 1996). Even in the presence of moisture, oxygen, and chloride ions (discussed in later sections of this report), corrosion damage can be prevented with highly resistive concrete (ACI, 1996). High quality concrete (low *w/c*, low permeability, certain admixtures) will also limit the ingress of moisture, oxygen, and chlorides. Concrete cover and degree of carbonation also affect the rate of corrosion (Tilly, 1987; ACI, 1996).

Epoxy coated strands showed excellent corrosion resistance performance in the laboratory in a study conducted by Perenchio et al (Perenchio et al, 1988; Whiting et al, 1993).

An electrochemical corrosion reaction can be separated into two component reactions called half-cell reactions. In the corrosion of steel reinforcement, iron is oxidized into ferrous ions at the anode, which in turn forms into ferrous oxide through several reactions (ACI, 1996). A second half-cell forms at the cathode, involving oxygen, water, and a free electron from the anodic reaction, generating a hydroxyl ion (ACI, 1996). Ferrous oxide precipitates on the surface of the reinforcing steel at the anode and occupies a volume several times greater than of the atomic / ionic iron consumed in the reaction (Kosmatka et al, 2001).

The anodic half-cell reaction has a standard redox potential (SRP, also known as standard electrode potential) of E=-0.440 mV. The SRP of the cathodic reaction is E=+0.401. SRP's are considered to be reversible potentials, in reference to a hydrogen half-cell having zero potential (ACI, 1996).

The SRP differs from corrosion potential (also known as rest potential or open circuit potential). The corrosion potential is related to the temperature, composition of the aqueous medium, and polarizations of the half-cells (ACI, 1996). It is the difference in magnitude of the two half-cell reactions (or SRPs) that determines the driving force of an electrochemical corrosion reaction.

Corrosion rate in concrete is typically influenced by environmental and other factors and controlled by one of two mechanisms. These mechanisms are the rate of oxygen diffusion through concrete and electrical resistance within the concrete (ACI, 1996). Both factors will affect the electrochemical corrosion reactions that produce or consume electrons (Jones, 1992). The rate of electron flow (current) is therefore a measure of the reaction rate. From manipulation of Faraday's Law, it can be shown that corrosion current density is proportional to the mass loss per unit time (Jones, 1992).

As previously mentioned, every half-cell reaction is reversible. The current density and potential at which a half cell reaction occurs in known as the exchange current density and half cell electrode potential, respectively (Jones, 1992). For an anodic (oxidation) half cell electrochemical reaction to occur, there must be a positive shift in the current density and potential from the equilibrium state. A cathodic (reduction) reaction will occur when a negative potential and positive current density is applied to the equilibrium state.

Deviation of half-cell electrode potential from the equilibrium state is known as polarization. Two kinds of polarization can occur simultaneously and are known as activation and concentration polarization (Jones, 1992). Concentration polarization occurs when the level of ions from the cathodic reaction are depleted more aggressively at the surface of the corroding metal, thus governing the rate of the reaction. Activation polarization occurs when one step in the electrochemical corrosion reaction controls the rate or charge flow.

2.3.3.2 *Carbonation-induced corrosion deterioration*

Carbonation of concrete is a reaction between acidic gases in the atmosphere or dissolved in water and the products of cement hydration (Emmons, 1994; ACI, 1992). This reaction produces carbonates ($CaCO_3$) and is accompanied by shrinkage. Concrete carbonation is a function of humidity, concrete permeability, and the concentration of carbon dioxide.

Carbonated concrete has properties that can be considered both beneficial and detrimental to concrete performance. Favorable effects of carbonation can be found in increased strength, hardness, and dimensional stability. Adverse effects of carbonation can be a porous and less wear resistant surface. Probably the most detrimental effect is a reduction in the concrete alkalinity, from a pH of around 13 to a pH of around 10 (ACI, 1992). When the pH of the concrete approaches 10, the passivity of steel is destroyed and more rapid corrosion may occur (Emmons, 1994).

Whiting et al, used petrography to determine the depth of carbonation in several prestressed concrete bridges in the Midwest (Whiting et al, 1993). At the time of the study, the Midwest bridges ranged in age from 31 to 36 years old. The depth of carbonation was generally on the order of 0.1 to 0.4 inches, with the exception of one bridge that had 0.6 to 1.3-inches of carbonated concrete. Cover to prestressing steel was greater than 1.5-inches for the same bridges. It is estimated that an upper limit for carbonation in concrete is 0.04-inches per year (Emmons, 1994).

It is uncertain whether or not Michigan's prestressed concrete I-beam ends are affected by carbonation-induced deterioration. Refer to the cited references for additional information on the mechanisms by which carbonation-induced corrosion damage occurs and is resisted.

2.3.3.3 *Deicer-induced corrosion deterioration*

Chloride ions have a well-documented detrimental role in reinforced concrete. Chloride ions are considered to be the major cause of premature corrosion of steel reinforcement (ACI, 1996). Even sound concrete is not immune from chloride ion penetration (ACI, 1996). Several theories exist as to the exact mechanism by which chloride ions affect corrosion. Two theories consider the chloride ion to be essentially a catalyst in the electrochemical corrosion reaction. Another theory states that chloride ions disrupt the performance of the passive oxide film on the reinforcement, in turn promoting corrosion (ACI, 1996). Other researchers (Whiting et al, 1993; Needham and Juntunen, 1997) have indicated that concrete alkalinity may be lowered when chloride concentration is increased to a certain level. However, steel corrosion may occur even in highly alkaline concrete if enough chlorides are present with water and oxygen (Emmons, 1994).

The amount of chloride ions in the concrete is typically expressed as a percentage of the weight of cement (ACI, 1992). The concentration of chlorides necessary to initiate corrosion varies greatly depending upon the pH of the concrete (Emmons, 1994). The amount of chloride ion required to accelerate corrosion (threshold) is 0.2 percent weight of cement, as determined by water-soluble testing techniques. Chlorides may be acid-soluble (naturally occurring aggregates) or water-soluble (admixtures) in form (Emmons, 1994). Water-soluble chlorides are of greatest concern to concrete durability since they "readily become free to attack surrounding reinforcing steel" (Emmons, 1994). As such, design Codes and many references suggest limiting the amount of water-soluble chloride ion to 0.06 percent weight of cement (ACI, 1999; ACI, 1996; Emmons, 1994).

A potential advantage of prestressing steel over reinforcing steel may be a higher resistance to chloride ion attack. A study (Rengaswamy and Rajagopalan, 1977) cited by Whiting et al (1993) indicated that 2 to 3 times the amount of chloride ion was required to initiate corrosion in prestressing steel, compared to mild reinforcement. Interestingly, design codes such as ACI 318-99 limit the maximum chloride ion content of concrete mixes more severely in prestressed concrete than in conventionally reinforced concrete (Whiting et al, 1993).

From a study conducted by Needham and Juntunen (1997), it was found that the bridge inspection superstructure rating and chloride ion content did not relate to beam end conditions, such as deterioration and water staining, for the prestressed concrete I-beam bridges studied. Other researchers have drawn opposite conclusions regarding the level of chloride ions and deterioration. Novokshchenov found elevated levels of chloride ion in prestressed concrete I-beams at deck joints where the greatest corrosion-related deterioration took place (Novokshchenov, 1989). In Michigan, Needham and Juntunen found that the chloride ion content was related to average daily traffic (Needham and Juntunen, 1997).

The origin of deicer agents onto prestressed concrete I-beam ends is fairly well understood. Numerous authors have recognized that the majority of prestressed concrete I-beam end distress occurs at transverse bridge deck joints (Tilly, 1987; Novokshchenov, 1989; Whiting et al, 1998; Jadun, 1990; Needham, 1999; Enright and Frangopol, 2000). The actions of traffic and environment degrade joints and lead to beam-end deterioration (Whiting et al, 1998). When the deck joint is not watertight, leakage of deicing solutions onto the beam end is permitted.

Tilly cites few overall problems with prestressed concrete bridges, but notes that the damage caused by leaking transverse deck joints was, even in 1987, a significant problem in the United States (Tilly, 1987).

Work by Novokshchenov in FHWA-RD-88-269 "Salt Penetration and Corrosion in Prestressed Concrete Members", cited by Whiting et al (1993), also showed that deck joints were the cause of corrosion-related deterioration in prestressed concrete I-beam ends (Novokshchenov, 1989). Novokshchenov cited several attributes of a failed joint, including deteriorated sealer, lack of sealer, and cracking of concrete at the joint. Novokshchenov also cited insufficient concrete cover as causing pre-mature corrosion failure.

Whiting et al evaluated 12 prestressed concrete bridges in various environments to document causes of corrosion and deterioration (Whiting et al, 1993). Whiting et al had similar conclusions to Novokshchenov in that the deicer solution passing through deck joints was the cause of the deterioration. However, another pathway for deicers was from deteriorated drains or gaps in the bridge railings (Whiting et al, 1993).

MDOT sought to determine chloride ion content in a select group of prestressed concrete bridges and perform an inspection of a portion of the prestressed concrete bridges around the state of Michigan (Jadun, 1990; Whtiting et al, 1993). Of the 12 bridges selected, it was reported that any deterioration on the beams was minor. Beam end deterioration was noted in I-beams and box beams in the study, and the distress was attributed to leaking deck joints. The study also showed that vehicular traffic underneath a prestressed concrete bridge increased the chloride ion contents of the beams, I-beams had higher chloride ion contents than box beams, and that box beams were in worse condition (Jadun, 1990; Whiting et al, 1993).

One research program was identified that sought to replicate leaking deck joints in a laboratory setting by periodically wetting I-beam ends with a 15% NaCl solution (Whiting et al, 1998). I-beams tested for the project were AASHTO Type II, cast specifically for the research project. Mix proportions for the laboratory beams included a *w/c* of 0.39 and an air content of 5.0% to 7.5% (Whiting et al, 1998). The beam end wetting approach did not produce distress for the eight months the equipment was operational. Further testing of the beam end specimens was consequently abandoned by Whiting (Whiting et al, 1998).

Enright and Frangopol studied prestressed concrete bridges in the Midwest and Colorado (Enright and Frangopol, 2000). They also concluded that the most common location of prestressed concrete I-beam distress was at the transverse deck joints. Further, this damage was corrosion related and due to chloride ion ingress (Enright and Frangopol, 2000).

2.3.4 Other Forms of Deterioration

Review of literature and conversations with state engineers have identified impact damage, alkali-silica reaction (ASR), and delayed ettringite formation (DEF) as some other forms of distress that may or have been documented for prestressed concrete I-beams. These distresses either do not appear to be common in Michigan (ASR and DEF), or routinely occur at locations beyond the beam end (impact damage).

2.4 Tools for Identifying Prestressed Concrete I-Beam End Deterioration

In the following subsections of this report, distinction is made between "current" and "new" approaches. The rationale for assigning a category to each technique is whether or not the approach is being used by Michigan or other states, as indicated from a multi-state survey (see Chapter 5).

2.4.1 Inspection / Assessment Techniques

From the suggestion of other researchers (Shanafelt and Horn, 1980) and for the purposes of this report, the term *inspection* is used to refer to the physical act of obtaining data on the condition of a structural element. *Assessment* is defined as the process of reviewing or making an interpretation of (inspection) data/conditions, structural analysis, and other decision-making processes. Although Shanafelt and Horn's work dealt with practices related to accidental (vehicular) damage of prestressed concrete bridge beams, this approach could be applied to assessment of beam end deterioration.

Both assessment and inspection practices are discussed in this section. As an example, the Michigan Pontis Bridge Inspection Manual has defined pre-determined assessment criteria for an inspector to follow for assigning a condition state and potential feasible actions (MDOT, 1999). In contrast, the Michigan Structure Inventory and Appraisal Coding Guide, used for National Bridge Inspection Standards (NBIS) inspections, allows for greater latitude in assigning condition ratings to a structural element (MDOT, 1997b). In this case, assessment is left to the judgment of the bridge inspector.

2.4.1.1 *Current Approaches*

Responses from a survey of state bridge engineers, discussed in Chapter 5, have indicated that all responding states use the *FHWA Bridge Inspectors Training Manual 90* (Manual 90) as a reference guide for the inspection and assessment of prestressed concrete I-beam bridges (Hartle et al, 1995). This document is the current standard for inspecting bridges and generally covers bridge mechanics, materials, and inspection practices. Guidelines are established to aid the bridge inspector in defined tasks. Of particular interest to this project are the suggested means and methods to inspect the ends of prestressed concrete I-beams.

The tools for cleaning, inspection, visual aid, measuring, and documentation that many states use for inspecting prestressed concrete I-beams are covered in Manual 90. Included in this list are chipping hammers, mirrors, and optical crack gauges. Conventional non-destructive evaluation equipment (pachometers, ultrasonic thickness gauge, etc.) is not part of the bridge inspector's equipment (Hartle et al, 1995).

Shanafelt and Horn identified similar tools that aid in the inspection of damaged prestressed concrete beams (Shanafelt and Horn, 1980) including a magnifying glass, flashlight, camera, and mirror. In addition to physical tools used by an inspector, good eyesight and a critical mind are essential personal qualities.

Photography, one of the technologies included in Manual 90, has benefited from advances in recent technology. The Tennessee Department of Transportation (TDOT) Bridge Inspection and Repair Office conducted an evaluation of the feasibility of using digital cameras to replace conventional cameras (FHWA, 1991). Digital cameras are greatly enhancing the usefulness of bridge inspections, since state-of-the-art computer programs allow engineers in the office quickly view the conditions of a bridge at the time the last inspection was performed.

Manual 90 provides inspection procedures for prestressed concrete I-beams (Hartle et al, 1995). Included in these procedures are specific instructions to "Examine the areas near the bearings and the cast-in-place end diaphragms for spalling concrete." among other detailed documentation requirements (Hartle et al, 1995).

The following is an except from the Manual's basic concrete inspection section:

When inspecting concrete structures, note all visible cracks, recording their type, width, length, and location. Any rust or efflorescence stains should also be recorded. Concrete scaling can occur on any exposed face of the concrete surface, and its area, location, depth, and general characteristics should be recorded. Inspect concrete surfaces for delamination or hollow zones, which are areas of incipient spalling, using a hammer or a chain drag. Delamination should be carefully documented using sketches showing the location and pertinent dimensions.

Unlike delamination, spalling is readily visible. Spalling should also be documented using sketches, noting the depth of the spalling, the presence of exposed reinforcing steel, and any deterioration or section loss that may be present on the exposed bars.

There are many common defects that occur on concrete bridges: cracking, scaling, delamination, spalling chloride contamination, honeycombs, pop-outs, wear, collision damage, abrasion, overload damage, reinforcing steel corrosion, prestressed concrete deterioration.

Each of these distresses is defined in the Manual. Cracks can be classified as hairline, medium, or wide cracks. On prestressed structures, all cracks are significant. When reporting cracks, the length, width, location, and orientation (horizontal, vertical, or diagonal) should be noted. The presence of rust stains or efflorescence or evidence of differential movement on either side of the crack should be indicated.

The manual has an eight-step inspection procedure for prestressed concrete I-beam bridge structures:

- *Examine the areas near the bearings and the cast-in-place end diaphragms for spalling concrete.*
- *Inspect the fixed diaphragms for diagonal cracking. This is a possible sign of shear failure caused by structure movement.*
- *Investigate the intermediate diaphragms for cracking and spalling concrete.*
- *Check beam flange surfaces for longitudinal cracks. This may indicate a deficiency of prestressing steel.*
- *Inspect the tension and shear zones of the beams for structural cracks. Any crack should be carefully measured with an optical crack gauge and documented.*
- *Examine underneath the beams for alignment and camber of the prestressed beams. Signs of deflection usually indicate loss of prestress.*
- *Investigate the beams for any collision damage. This is a major cause of damage to prestressed I-beams.*
- *Examine thoroughly any repairs that have been made previously. Determine if the repaired areas are functioning properly. Effective repairs and patching are usually limited to protection of exposed tendons and reinforcement.*

In terms of assessment, Manual 90 requires that inspectors rate bridge elements, including prestressed concrete I-beams, as a whole, rather than allowing individual locations of distress to lower an element's rating. However, inspectors are expected to modify the condition rating accordingly if an isolated distress (possibly beam end deterioration) influences the load carrying capacity or serviceability of the element (Hartle et al, 1995). The condition rating for a superstructure element, such as a prestressed concrete I-beam, is based on a scale of 0 to 9, in Manual 90. Manual 90 does not have a uniform damage severity classification for various concrete distresses. Shanafelt and Horn suggest ranking damage assessment into four categories: minor, moderate, severe, and critical (Shanafelt and Horn, 1980).

One recent study revealed significant variability from state to state on assigned condition ratings and field notes. In 1998 the FHWA's Nondestructive Evaluation Validation Center undertook a study to evaluate visual inspection of bridges (Graybeal et al, 2001). The Center performed a series of inspection trials among 49 state departments of transportation bridge inspectors. The bridges used in this study were located in northern Virginia and central Pennsylvania. "The primary data used to evaluate the routine inspections were the National Bridge Inspection Standards Condition Ratings assigned by the inspectors to the primary bridge components (deck, superstructure, and substructure)." In depth inspections were rated on the inspector's field notes. The study showed a significant variability from state to state on assigned condition ratings and field notes. The Center's recommendations were to revise the condition rating system to increase accuracy and reliability, increase the training of inspectors with respect to methods that identify reoccurring defects, and study the types and sizes of specific defects that will be found in an in-depth inspection.

Other studies have been performed to determine current policies and practices that may affect the accuracy and reliability of visual inspection (Rolander et al, 2001). A survey of state departments of transportation, a local-level department of transportation, and selected bridge inspection contractors showed how inspection management might influence the reliability of inspections (Rolander et al, 2001).

Effects of the deterioration should be understood through engineering assessments (Shanafelt and Horn, 1980). For accidental impact damage of prestressed concrete bridge beams, Shanafelt and Horn do not recommend field assessment of damage, due to the potential for making premature assessments. More innovative repair techniques appear to be generated by having repair assessments be office-based, away from the field process. However, ideas and alternatives for repair are suggested from the field inspector. Shanafelt and Horn concluded that complex engineering calculations were not required to develop in-place repairs, but that a basic level of calculations should be performed to verify restoration of strength and durability. Procedures for performing a capacity analysis of a prestressed concrete beam, considering compromised prestressing strands, have been developed by Xanthakos (1996).

Shanafelt and Horn provide recommendations for assessment of exposed, damaged, and severed strands. Nearly all states allow beams with one to three severed strands to remain in service (Shanafelt and Horn, 1980). Location of a damaged beam within a structure should be given consideration in the assessment stage. For example, fascia beams may not be loaded to the same levels as internal beams and therefore a greater amount of deterioration may be permitted (Shanafelt and Horn, 1985).

2.4.1.2 *New Approaches*

A manual of recommended practice for the inspection, assessment, and repair of deteriorated prestressed concrete bridge beams has been developed within NCRHP Report 280 (Shanafelt and Horn, 1985). These practices can be applied to deteriorated ends of prestressed concrete beams with engineering judgment (Ahlborn et al, 2001).

A rivet gun chipper is a tool that may be used by inspectors to evaluate the quality of concrete around a prestressing strand (Shanafelt and Horn, 1985). Deflection and elastic shortening techniques have been used in the past to estimate tension in exposed prestressing strands.

To evaluate box beams for a New York State DOT project, engineers used techniques consisting of visual examination, counting broken / deteriorated / corroded strands, and measuring remaining concrete section (Hag-Elsafi and Alampalli, 2000).

In addition to a visual inspection, Whiting et al, (1993) used a variety of survey techniques for a research project involving the inspection of prestressed concrete bridges. These techniques included half-cell potential measurements, delamination survey, cover measurements, corrosion rate measurements, petrographic analysis, chloride sampling and analysis, and penetrating sealer effectiveness. Techniques showing results fairly consistent with prestressed concrete I-beam end deterioration included half-cell potential measurements, corrosion rate measurements, and chloride sampling. The deicer-environment bridges reviewed by Whiting et al generally had deterioration consisting of cracked and spalled concrete with exposed corroding strand and stirrups.

Similar techniques were chosen by Arner and Panganiban (1986) and Kennedy (1991) for the investigation of deteriorating bridge decks. These techniques included half-cell potential testing, cover measurements, and chloride ion content testing. Arner and Panganiban suggested that delaminations could affect half-cell potential testing by creating an insulating plane in the concrete.

The result of a study conducted on the condition of prestressed concrete bridge elements located in corrosive environments is presented by Novokshchenov (Novokshchenov, 1989). Five bridges were subjected to a detailed inspection and other bridges in the study received a visual inspection. The bridges selected were in northern areas (subjected to deicing salts) and in southern areas (subjected to marine spray). Descriptions, visual examination results, corrosion amounts, concentration of chlorides, metallurgical analyses, and metallographic examination results are given with pictures and tables for each bridge surveyed. Recommendations about procedures, parameters and threshold values for detection of corrosive environments and assessment of the condition of prestressing steel in bridge components are also included in the Novokshchenov study.

Monterio et al (Monterio et al, 1998) studied a nondestructive method using a multi-electrode electrical resistivity array to determine the position of the reinforcing bars and their corrosion state. This work was performed by measuring the frequency dependence of the complex impedance of the bars along the surface of the concrete structure. By using this method, the background resistivity of the concrete can also be obtained. The method uses the direct relation between the complex impedance and the corrosion rate of the reinforcing bars to provide a rapid evaluation of the corrosion rate. Two advantages of the technique are that the measurements are taken on the surface of the concrete and that the method does not require removal of the concrete to connect the device to the bars. An experiment was conducted to test and determine the efficiency of the method. Four types of bars were used in the research which included a clean bar, a bar covered with gold, a painted bar, and a corroded bar. Apparatus, measurement technique, instrumentation, and the results of the experiment are presented in the paper. Similar research on the same content is conducted by Zhang et al. (Zhang et. al., 2001). As a conclusion in this study, the Surface Measurement Method is agreed to be an accurate method to investigate different corrosion states of the reinforcement.

New tools are available not only for the inspection of original bridge elements, but also for monitoring the performance of these repairs. Halstead et al (2000) used strain gauges to monitor potential cross sectional growth of strengthened columns due to the corrosion of reinforcing steel (expansive process). In the study, linear polarization probes were also included in the instrumentation package to measure corrosion rate of internal reinforcement.

Broomfield, et al (1999) studied corrosion monitoring using half-cell potential measurement, linear polarization and macro-cell current measurement methods. Linear polarization, concrete resistivity and other probes have been installed in new structures to monitor durability as well as in existing structures to evaluate rehabilitation strategies such as corrosion inhibitor applications and patch repairs. The types of sensors used, data collection techniques, results and interpretation are discussed.

Data obtained from nondestructive evaluation (NDE) of structures can greatly enhance the maintenance and management of infrastructure systems (Nogueira, 1999). Nogueira has proposed a methodology for a systematic application of NDE methods in periodical bridge inspection.

Pascale, et al (1999) described an experimental program aimed to assess the performance of fiber optic sensors (FOS) in civil engineering applications. This technique has been applied to a reinforced concrete bridge beam externally reinforced by fiber reinforced polymer (FRP) sheets. The paper includes the information about FOS, the experimental program, and a discussion for the results.

NDE for corrosion detection of embedded or encased steel reinforcement or bridge cables using time domain reflectometry is developed and demonstrated by Liu, et al (2001). Modeling, procedures, and a case study are presented.

In the paper presented by Titman (1999), wide ranges of applications of infrared thermography are explored, particularly relating to structural investigation situations. Some guidance is given on optimum timing, conditions, and viewing locations for the various situations are also described.

Settipani (1987) has shown that gammagraphy can be used to determine strand location, size, corrosion, and concrete defects. This technique has shown to be effective on elements 27-inches and less in thickness. According to Settipani, gammagraphy has a high initial cost compared to other non-destructive evaluation methods, but it provides more comprehensive data and a permanent inspection record.

Innovative ways to detect concrete delaminations have emerged in recent years. Henderson et al (2000) reported that an instrument named HollowDeck is an alternate approach to acoustic impact surveys. The technology used involves frequency analysis of sound waves to identify areas of concrete delamination.

Ganji et al (2000) and Gucunski et al (2000) have performed similar testing using a portable seismic pavement analyzer (PSPA). With the PSPA, Ganji et al were able to detect delaminations not found by chain dragging on bridge decks. Gucunski et al were able to place delaminations into four severity levels by interpretation of surface waves.

Due to the size of the PSPA and the time it takes to complete a single test (Gucunski et al, 2000), the PSPA appears to show little promise for use in prestressed concrete I-beam end inspection. HollowDeck technology appears to be similarly inappropriate for beam end inspection. However, non-destructive testing equipment like the HollowDeck and PSPA could be used as calibration and training tools for inspectors performing acoustic impact (delamination) surveys.

A useful tool in performing assessments of beam-end deterioration is the understanding of how distresses may have formed. Juntunen prepared an interim report containing a collection of photographs related to cracking of prestressed concrete I-beams (Juntunen, 2000). From review of this report, a summary of crack location, type, and potential cause has been prepared. This information is presented in Table 2-1.

Table 2-1. Prestressed Concrete Crack Type and Probable Cause

Location	Crack Type	Potential Cause (per Juntunen, 2000)							
		Loss of Bond	Pre-stressing Cut	Load Hit	Diaph. Bond	Strand Slip	Over-load	Insuff. Reinf.	Reinf. Corr'n
End of Member	Horizontal		✓		✓				
	Diagonal		✓	✓			✓	✓	
	Frown		✓						
	Vertical								✓
	Map							✓	
Remainder of Span	Longitudinal Flange	✓							
	Diagonal			✓		✓			
	Map							✓	

2.4.2 Preventive Maintenance Techniques

The efforts described in the literature review and current infrastructure management practices recognize that preventive maintenance is essential in extending the service life of many structures. Prestressed concrete I-beam bridges are no exception as evidenced by end deterioration problems. For the purpose of this report, preventive maintenance of prestressed concrete I-beams is distinguished from repair techniques by the fact that preventive maintenance can be applied to bridges where there is an existing sound substrate. If reconstruction of the substrate is or is part of the requirements for a given technique, then the technique shall be considered a repair technique. Repair techniques are discussed in later sections of this chapter.

2.4.2.1 *Current Approaches*

Preventive maintenance suggestions for existing concrete structures can be separated into four categories according to ACI (ACI, 1996). These categories are insulating methods, environment modifying methods, electron control methods, and a combination of two or more methods. Research performed for this project has suggested that perhaps another category exists: structure modification methods. A summary of all of the preventive maintenance or repair approaches discussed in this chapter, separated per category, is included in Appendix J of this report. All are discussed further in this section.

2.4.2.1.1 Surface Insulating Methods

Insulating methods should be considered as a preventive maintenance option when the parent concrete has been properly prepared (Weyers et al, 1993; ACI, 1996). The objectives of the covering should be to prevent contaminant, moisture, and oxygen intrusion into the concrete. This may include removal of chloride-contaminated concrete and arrest of the corrosion process prior to surface application. To not remove chloride-contaminated concrete prior to the application of an insulating method may accelerate corrosion (ACI, 1996).

PCA publication IS001 identifies 17 protective treatments (coatings and sealers) for use on concrete exposed to chlorides (PCA, 2001). Penetrating sealers in this list included certain formulations of:

- Drying Oils,
- Silanes, and
- Siloxanes

Linseed-oil surface sealers may be safely used on a prestressed concrete member (Weyers et al, 1993). Silanes and siloxanes are inexpensive, easy to apply, and allow the passage of water vapor while resisting a head of water. Silanes have shown better penetration into dried concrete than siloxanes and surfaces can easily be retreated for improved penetration (PCA, 2001). These sealers should not, however, be applied to concrete elements with active corrosion or excessive chloride ion contamination (Whiting, 1999), and silanes may not be an optimum solution when prolonged exposure to freezing and deicing chemicals is expected (PCA, 2001).

Researchers and consultants have also suggested the use of silanes and siloxanes in preventive maintenance (Tilly, 1987; Weyers et al, 1993; Emmons, 1994; Whiting et al, 1998). At least 17 different nationwide manufacturers of drying oils, silanes, and siloxanes exist (PCA, 2001). In addition, drying oils, such as boiled linseed oil, are commercially available.

Sealers are judged by many performance characteristics including water repellency, chloride screening, penetration depth, vapor permeability, alkaline resistance, and UV resistance (Weyers et, al, 1993). Coatings are compared with respect to their chloride screening, vapor permeability, and shrinkage properties. The surface of the substrate must be free of contaminants through surface preparation and cleaning methods to successfully apply coatings and sealers (Weyers et al, 1993). Weyers et al suggest patching surface defects with materials that can accommodate the "high internal stresses" generated by surface coatings.

In 1993, Whiting et al examined the performance of penetrating sealers in accordance with Strategic Highway Research Program (SHRP) guidelines (Whiting et al, 1992; Whiting et al, 1993). Two locations on one bridge were tested with some difficulty and variable results (Whiting et al, 1993).

More recent work in preventive maintenance was also identified in FHWA-RD-98-189. The use of 2-part epoxy, siloxane, silane, and multi-component (silane, siloxane, methyl methacrylate) materials for use as an insulator on various bridge elements was investigated (Whiting et al, 1998). These elements included piers, slabs, and I-beams. For the beam specimens, the silane sealer and the multi-component material were applied. Material performance was measured using half-cell potential testing, macrocell current measurement, and visual observation. Results of the testing indicated that after 50 ponding cycles, corrosion activity increased in the silane treated specimen, as indicated by half-cell potential, and macrocell current measurements. The multi-component material showed better performance, with seasonal fluctuations in current and potential. Whiting et al concluded that neither material showed promise for a long-term solution to corrosion resistance without consistent reapplication. Better new designs may be more effective in preventing beam end damage than performing periodic repairs or replacing members.

The sealing of cracks is recognized as a means to extend the life of prestressed concrete I-beam ends. Since 1984, MDOT has specified that longitudinal cracks greater than 10-mils in width in the beam end be sealed (MDOT, 1984b). Given the availability of several coatings to bridge cracks in concrete, coatings may have preference over crack treatment techniques.

Other countries are also using coatings as a preventive maintenance measure. The Victoria Department of Transportation in Australia conducted a project aimed at extending the durability of coastal bridge structures by application of protective coating systems (Andrews-Phaedonos et. al., 1997). The first stage of the study focused on a literature review of current world standards and codes related to laboratory based coating testing, assessment of coating performance requirements, evaluation of available products for testing, and a review of the most appropriate laboratory based assessment methods. The second stage included development of a reproducible concrete-type substrate, a chloride ingress-screening test, and their application to examples of four generic types of coatings.

A final means of insulating concrete from deterioration is the treatment of cracks. Durable repairs may be achieved by select epoxy injection of cracks (Shanafelt and Horn, 1980). Michigan has adopted the practice of sealing longitudinal cracks in prestressed concrete beams since 1984 (MDOT, 1984b). However, cracks in prestressed concrete formed by normal loading cannot be effectively repaired by epoxy injection (Shanafelt and Horn, 1980).

2.4.2.1.2 Electrical Control Methods

Only cathodic protection (CP) has been proven to stop corrosion of an existing structure (ACI, 1996). Cathodic protection is a method of controlling electron flow (current) in a structure (ACI 222R-96, 1996). In general, CP can be accomplished by either galvanic / sacrificial anode cathodic protection or impressed current cathodic protection systems (ACI, 1996; Xanthakos, 1996). The advantages and disadvantages of each system are briefly summarized in Table 2-2 (ACI, 1996):

Table 2-2. Comparison of Advantages and Disadvantages of Cathodic Protection Systems

Galvanic or Sacrificial Anode	Impressed Current
No external power required	External power required
Fixed, small driving voltage	Voltage variable over a wide range
Limited, small current output	Current output variable over a wide range
Interference of adjacent structure not likely	Interference can result
Overprotection not likely	Overprotection can result
Anodes rapidly consumed	Anodes slowly consumed
Sensitive to moisture and temperature conditions	Not sensitive to moisture and temperature

Possibly the more desirable cathodic protection system is a sacrificial anode system because it does not require external electricity to operate (ACI, 1996; CIAS, 2001). Sacrificial anodes are composed of a consumable metal such as zinc or aluminum and can be installed on the concrete surface or in an internal application (ACI, 1996) without external electrical requirements. Other consumable anodes include magnesium (ACI, 1996) and aluminum-zinc alloys (Funahashi and Daily, 1996). These systems can be applied to most structures and easily provide localized protection (CIAS, 2001). An impressed current system is not appropriate however where global reinforcement protection is desired, or in especially electrically resistant concrete where the driving voltages may be too low to provide sufficient protection (CIAS, 2001).

A unique type of sacrificial anode is a surface applied anode system. The 3M company has developed the 3M-4727 anode, a surface-mounted self-adhering sacrificial anode for cathodic protection of concrete structures (3M, 1998; 3M, 2001). 3M's literature indicates the anodes may be applied in a wide range of service environments in the vertical, horizontal, and overhead positions (3M, 1998). Like all sacrificial anodes, an electrical connection to the reinforcing steel (cathode) is required. Performance of the 3M-4727 anode has been correlated to NACE Standard RP290-00 (3M, 1998). Others have indicated that the criteria for protection of steel embedded in concrete are not clearly defined (ACI, 1996).

Sacrificial anode cathodic protection using the 3M-4727 anode did not show long-term success in recent Illinois trials on prestressed concrete I-beam ends (IDOT, 2001). Poor long-term performance of the Illinois installations was due to the breakdown of the hydro-gel material (IDOT 2001); a layer that aids in keeping the concrete moist for effective cathodic projection. According to the IDOT report, the hydrogel had changed over a "short period of time" from an ion conductor to an insulator (IDOT, 2001). A project is underway in Michigan that is also using this anode on I-beam ends (Needham, 2001); results of the anode performance have not yet been obtained. Other surface applied sacrificial anode systems include a technique involving thermal-sprayed zinc anodes (Bullard et al, 1996). The applicability of thermal-sprayed zinc anodes to Michigan bridges is being investigated in an internal MDOT project.

Surface-applied anodes appear to have a disadvantage over internal anodes in that they prevent future direct observation of the beam-end concrete. Norcure manufactures two types of internal anodes known as the Galvashield XP and Galvashield CC (Beaudette, 2001a). Both anodes are constructed of a zinc core surrounded by a cement matrix (Beaudette, 2001b; Beaudette, 2001c). In general, XP anodes are more appropriate for repair type projects while the CC anode could be used in a preventive maintenance application. Vector Corrosion Technologies has cited one project in Winnipeg, Manitoba, Canada, where the Galvashield CC anode was used to protect service life and prevent the onset of future corrosion problems (Vector, 2001). The Galvashield CC anode is cylindrical, 2-1/2-inches long and 1-5/8-inches in diameter. The anode is to be installed in predrilled holes in the concrete. A wire lead emitting from the anode is provided to permit an electrical connection to the reinforcing steel. Norcure specifies a maximum anode spacing of 28-inches for the CC anode. The Galvashield CC anodes are reported to have a service life of 10 to 20 years under normal conditions (Beaudette, 2001c).

In a study performed by the Concrete Innovations Appraisal Service, Galvashield anodes were found to be effective in conferring protection to reinforcing steel when used as intended (CAIS, 2001). However, the quantity of steel protected and service life of individual Galvashield anodes in a structure is still being studied. According to Daniel Burns of Vector Corrosion Technologies, the performance of the Norcure anodes is reportedly in accordance with non-published Vector recommendations (Burns, 2001).

Impressed current cathodic protection has been used since 1977 and is an option for preventive maintenance of prestressed concrete I-beams (CIAS, 2001). Impressed current systems force reinforcing steel into a passive cathodic state by applying a direct current from an anode to the reinforcing steel. Sufficient current is used to stop the anodic reaction at the steel and render the steel cathodic (CIAS, 2001). These systems are suitable for global protection, and can be applied via a variety of systems (CIAS, 2001). Impressed current systems do require constant adjustment and monitoring, as well as a permanent power supply (CIAS, 2001). However, the need for on-site electrical power and continual monitoring makes impressed current CP systems very unattractive for use in beam-end applications. In addition, overprotection (oxygen reduction) in impressed current systems may cause debonding at high current densities or hydrogen embrittlement for high yield strength steels (ACI, 1996).

2.4.2.2 *New Approaches*

2.4.2.2.1 Structure Modification Methods

There are several structure modifications that may help to reduce the beam end deterioration. These modifications include modifications to the deck, support member modifications, and primary framing modifications.

Possible deck modifications include repair or replacement of transverse deck joints, installation of a positive drainage waterproof overlay, or installation of a continuous for live load (CLL) deck. No evidence in the literature review was identified that indicated states are using a joint maintenance approach to prolonging prestressed concrete I-beam life. This is not to say that joints are not receiving attention from state transportation departments, but rather suggests that consideration is not being given to addressing the cause of the distress. Effective joint maintenance appears to be a logical approach to ensuring successful performance of prestressed concrete I-beam ends and other researchers have suggested joint maintenance as the first step in combating beam-end distress (Whiting et al, 1998). More effective joint maintenance, in combination with other approaches discussed in this chapter, can be extremely effective in preventing end deterioration. Design of joints should receive careful consideration, as not all are designed to be watertight. Installing joints and other bridge components in a manner that provides for positive drainage is an additional means of redirecting moisture and deicing solutions away from beam-ends (Whiting et al, 1998).

A recent trend in bridge deck construction has been to construct the decks with out joints, or CLL. In other words, the primary support members are designed to be simply supported members, carrying the dead and live loads. Bridge decks also carry superimposed dead and live loads, however the decks are made continuous over the supports (beam-ends). This design concept permits the elimination of transverse deck joints at the beam-ends and therefore reduces the risk of corrosion-induced concrete deterioration. To this end, reconstructing decks CLL is a preventive maintenance method.

Another option that may achieve the same results, as a continuous for live load deck is a waterproof deck overlay (Tortorete, 2001).

A field investigation performed by this and other researchers indicated frequent distress at the bottom beam flange near the sole plate (Ahlborn et al, 2001). This deterioration was commonly associated with corroding sole plates and masonry plates. Replacement of deteriorating bearings prior to concrete spalling is a means of preventing concrete distress.

Partial depth beam repair is a structure modifying approach that can be used as a preventive maintenance measure for beam-ends with potential deterioration. However, this technique is most applicable to repair projects and is therefore discussed in greater detail later in this chapter.

2.4.2.2.2 Surface Insulating Methods

Preventive maintenance solutions for prestressed concrete I-beams not currently being widely used have also been identified. Sealers and coatings are both considered surface treatments for concrete (PCA, 2001). However, penetrating sealers and surface sealers can be differentiated from coatings by their performance objectives. Sealers aim to prevent or decrease the penetration of liquid or gaseous media (e.g. chlorides, moisture, and sulfates) that can enter the pores of the concrete (PCA, 2001; ACI, 1990). Coatings are designed to act as a barrier to provide complete isolation between the concrete and the contaminating substance (PCA, 2001).

Numerous surface sealers are available for the protective treatment of prestressed concrete I-beam ends. Like penetrating sealers, surface sealers intend to limit the amount of moisture, chlorides, or other materials that can enter the pores of the concrete (PCA, 2001). Responses from the multi-state survey for this project (Chapter 5) did not indicate that any state transportation department is using surface sealers. A recent PCA publication identified 17 families of protective treatments (coatings and sealers) for use on concrete exposed to chlorides (PCA, 2001). Surface sealers in this list included certain formulations of:

- Acrylics,
- Methyl Methacrylate (MMA), and
- High Molecular Weight Methacrylate (HMWM)

At least 22 different manufacturers of acrylics and methyl methacrylates are available (PCA, 2001). A complete listing of these manufacturers is included in PCA publication IS001 (PCA, 2001).

Michigan is moving toward using coatings to protect prestressed concrete I-beam ends (Till, 2001b). In the MDOT Bridge Committee Meeting Minutes of June 14, 2001, MDOT indicated the intent to use an elastomeric sealer to protect beam-ends from chloride-induced corrosion deterioration. Based on the results of the multi-state survey (see Chapter 5), no other state departments of transportation are using coatings to protect beam-ends.

Several coatings were also identified in PCA as being suitable for resistance to chlorides (PCA, 2001). Coatings showing the most promise included:

- Bituminous Paints, Mastics, and Enamels,
- Polyesters,
- Urethanes,
- Epoxies,
- Neoprene,
- Coal Tar-Epoxy,
- Chemical Resistant Mortars and Grouts,
- Sheet Rubber Goods,

- Acrylics,
- Methyl Methacrylate (MMA), and
- High Molecular Weight Methacrylate (HMWM)

Several manufacturers produce products in multiple families of coatings. A recent effort by the Florida Wire and Cable Company has been to develop and bring to commercial production an epoxy coated prestressing strand (Breen, 1990). NCHRP Report 313 indicated excellent creep characteristics and found "epoxy coated prestressing strand to be remarkably tough and effective against corrosion in both pretension and post-tensioned applications" (Perenchio et al, 1989). Fiberglass tendons for anti-corrosion have also been introduced (Breen, 1990).

Partial depth beam repair is a surface insulating approach that can be used as a preventive maintenance measure for beam ends with potential deterioration. However, this technique is most applicable to repair projects and is therefore discussed later in this chapter.

2.4.2.2.3 Environment Modifying Methods

Elimination of water, removal of chlorides, removal of oxygen, impregnation of polymers, and elimination of stray electrical currents are means to create a less corrosive concrete environment (ACI, 1996).

Chloride ion extraction (CIE) and re-alkalization of concrete are two methods that are available to change the properties of concrete. This change generally modifies the environment from one conducive to corrosion-induced deterioration (e.g., high levels of chloride ion, low-alkalinity) to a less corrosive environment. CIE is accomplished electrochemically (ACI, 1996) and involves the use of an electrolyte, ion exchange resin, noble anode, and the reinforcing steel. In CIE, a low voltage direct current is impressed upon the reinforcing steel and chloride ions are driven toward the noble anode (Beaudette, 2002a; ACI, 1996). In the process, the chloride ions become trapped in the ion exchange resin (ACI, 1996). ACI notes that CIE is most applicable for structures exposed in aqueous solution (ACI, 1996). More information on chloride ion extraction and re-alkalization of concrete can be found in the Beaudette (2002a, 2002b).

Partial depth beam repair is an environment modifying approach that can be used as a preventive maintenance measure for beam ends with potential deterioration. However, this technique is most applicable to repair projects and is therefore discussed in greater detail later in this chapter.

2.4.2.2.4 Electrical Control Methods

There are several different methods that are appropriate as preventative maintenance in the control of electrons for reinforcement corrosion prevention in concrete structures. These include surface applied corrosion inhibitors; surface applied sacrificial anodes, and internally applied sacrificial anodes. Discussion on sacrificial anodes is included in the current approaches section on preventive maintenance, as state departments of transportation are currently using them.

Corrosion-inhibiting admixtures chemically arrest the corrosion reaction (Kosmatka and Panarese, 1988). Corrosion inhibitors may be added to the portland cement concrete during beam production as an admixture or be applied to hardened concrete as a spray applied surface treatment. This section will consider only surface applied corrosion inhibitors, however, it should be noted that corrosion inhibitors were once incorporated directly into the concrete used for prestressed concrete I-beams in Michigan (MDOT, 1992). Surface applied products are available from at least two manufacturers in the United States. Penetrating inhibitors have been shown to reduce the corrosion of reinforcing steel in bridge decks. The method has been tested extensively as described in SHRP-S-666 and may involve heating the concrete above the boiling point of water and then soaking the surface with an aqueous calcium nitrite solution (Al-Qadi et al, 1993). The best results have been found when the contaminated surface layer is either removed or scarified prior to application of inhibitors.

Some penetrating inhibitors that have been shown to be effective with overlays are Alox 901 and Cortec VCI-1337 (MCI 2020). Alox 901 may be applied to un-dried surfaces and has the potential to double the service life of an overlay (or repair material). MCI 2020 also boasts increased overlay service life and has shown nearly a 100% reduction in corrosion rates in laboratory specimens (Al-Qadi et al, 1993). MCI 2020 does not perform as well on un-dried specimens (Al-Qadi et al, 1993), and requires the overlay to contain the inhibiting admixture MCI 2000. Alox 901 must not be overlaid with an admixture-modified concrete (Weyers et al, 1993). Both inhibitors are easily applied with a typical commercial garden sprayer but require a light sandblasting or shotblasting to achieve adequate bond strength with the overlay (Al-Qadi et al, 1993).

Partial depth beam repair is an electrical control approach that can be used as a preventive maintenance measure for beam ends with potential deterioration. However, this technique is most applicable to repair projects and is therefore discussed in greater detail later in the following section of this chapter.

2.4.2.2.5 Analytical Tools

Analytical tools are available to predict when a member may be affected by corrosion-induced distress. Enright and Frangopol (2000) present and summarize methods for damage modeling and predicting service life of concrete beams, specifically with respect to corrosion-initiated distress. According to Frangopol and Enright, the location of damage in a member is related to the type of member, its position in the structure, and the damage source. In other words, the source of the damage can be linked to a damage pattern. Their study included a sampling of bridges in the Midwest and Colorado. A conclusion was that the most common form of prestressed concrete I-beam distress for these bridges is at the transverse deck joints. Further, this damage is corrosion related and due to chloride ion ingress. To this end, Enright and Frangopol modeled resistance and steel area loss over time, for both shear and flexural reinforcement. This may be a valuable modeling tool for determining whether or not strengthening or replacement of a superstructure is required.

Others have also used analytical tools to model future performance. Research by Cairns and Millard predicted the residual lifetime of the structure with no remediation (Canis and Millard, 1999). The qualitative and quantitative effects on residual structural capacity concrete and steel section loss, longitudinal cracking and loss of bond between reinforcement and concrete were investigated by this team.

How a structure behaves over time (i.e. from construction to some point in time) is useful in understanding what structural effects have changed from the original design assumptions. Mari and Valdes considered continuous concrete box beam bridges composed of precast reinforced and prestressed concrete beams with a U cross section and a cast-in-place top slab (Mari and Valdez, 2000). A 1:2 scale model of a two-span continuous bridge was tested to study its behavior during the construction process and under permanent loads. Time-dependent concrete properties, support reactions, deflections, and strains in the concrete and steel were measured for

500 days. Time-dependent redistributions of stresses and internal forces throughout the bridge were also measured. The test results were compared with analytical predictions obtained by means of a numerical model developed for the non-linear and time-dependent analysis of segmentally erected reinforced and prestressed concrete structures. Generally good agreement was obtained, showing the adequacy of the model to reproduce the structural effects of complex time-dependent phenomena.

A probabilistic model has been developed that predicts bridge specific deterioration based on random field simulations (Sterritt and Chryssanthopoulos, 1999). For these simulations, reinforced concrete bridges were targeted, with emphasis placed on structural members for whom field data was collected. As a result of the work by Sterritt and Chryssanthopoulos, models were developed for chloride ingress, spatial corrosion initiation, spatial delamination and corrosion propagation. Nogueira has also investigated employing probabilistic deterioration models (Nogueira, 1999).

Zemajtis and Weyers have also proposed a corrosion deterioration model for bridge structures on the general representation of deterioration versus time relationship (Zemajtis and Weyers, 1998). The Zemajtis and Weyers work considers diffusion period, corrosion stage, bar size, spacing, concrete cover, and the deterioration stages in the modeling.

2.4.3 Repair Techniques

Repair techniques are considered by some researchers to be those that restore a bridge component to an acceptable level of service (Weyers et al, 1993). Repair of prestressed concrete I-beams with end deterioration is cost effective compared to beam replacement, as evidenced by a recent MDOT project that found repairs to cost 35 to 69 percent of full-replacement cost (Needham, 2000).

Repair strategies can be developed with several or no levels of redundancy to assure durable repairs (Emmons, 1994). Different repair scenarios can be used, depending on the degree of exposure and damage (Xanthakos, 1996). Monitoring of repairs is recommended to improve damage inspection and assessment techniques (Shanafelt and Horn, 1980). The ability to perform maintenance or replacement of repair systems should also be considered (Xanthakos, 1996).

Assumptions made by the engineer designing the repairs should be studied carefully. Parameters indicated on the original project documents may not be representative of actual conditions. Tilly cites studies in which long term prestressing losses were 36 percent of the applied stress, compared to the 21 percent loss expected by design (Tilly, 1987). An accurate understanding of prestress loss is important because of the need to compute allowable shear and flexural capacities in beams with more advanced levels of deterioration.

2.4.3.1 *Current Approaches*

2.4.3.1.1 Structure Modification Methods

Beam replacement is known to be an option for prestressed concrete I-beam end deterioration that has been exercised by Michigan and neighboring states. Researchers have also cited complete replacement of damaged members as an option (Shanafelt and Horn, 1985). Removed beams are commonly lifted up and out of a bridge, although some states have developed alternate methods of replacement, such as lateral removal. However, in-place repair techniques were found to be desirable compared to replacement due to faster completion times and less overall inconvenience to users.

Repairs to bridge decks, support members, and framing members all constitute structure modification methods (see Appendix J). In order for repairs to be successful, the cause of the corrosion induced concrete deterioration should be remedied. Often, improperly functioning bridge decks are associated with end deterioration. Techniques such as joint replacement, deck overlays, and new decks can be used to prolong the life of repairs made directly to the beam-ends.

Support member modifications are also options for the repair of prestressed concrete I-beams. One such retrofit is the addition of a haunch to the pier or abutment to establish a new, sound bearing area (Emmons, 1994). Replacement or repair of bearings is another option that may increase the life of repairs made directly to the beam-ends.

Beam replacement is an option that has been pursued by Minnesota, Illinois, and Michigan. Various methods exist for beam removal including selective vertical removal, selective horizontal removal, or total bridge replacement. A less aggressive approach would be to install a supplemental primary framing beam that would carry a portion of the distressed beams load.

Primary framing members may also be repaired through the use of external beam strengthening methods. Concrete encasement, external post-tensioning, and externally bonded reinforcement could be a means of accomplishing external beam strengthening.

Perhaps the repair technique that holds the most promise for distressed I-beams ends is partial depth repair (PDR). In general, PDR's are those that do not extend the full thickness of the member. PDR's can be performed using relatively simple approaches or with more advanced methods. Summaries of these approaches follow.

Traditional removal and replacement of deteriorated concrete has been applied by Michigan and neighboring states for the distressed prestressed concrete I-beam ends (Xanthakos, 1996; Needham, 2000). In general, concrete repairs must provide protection for the substrate, have an acceptable appearance, and have the ability to carry the required loads (Emmons, 1994).

Needham drafted a repair procedure for prestressed concrete I-beams with end distress (Needham, 1999). Needham's work involved two separate repair and testing scenarios, exposing 12-inches of prestressing and shear reinforcement at the beam ends. From review of Figure 5 in Needham's report, the entire beam cross section was removed to the first stirrup (approximately 3-inches). Concrete was removed at the flange tips to a distance of approximately 12-inches from the "original" end of the member. Removal was tapered back from the flange tips, along two roughly diagonal lines, in plan, to the limits of web removal. Concrete was patched with MDOT Grade D polymer (latex) modified concrete.

Using the described removal and patching procedure, Needham found that through full-scale laboratory testing, there was no stress loss in prestressing strands (Needham, 1999). Needham did conclude that because of a high load hit on the beam that was used for his experiment, some loss in prestressing did occur from the hit. Premature shear cracking and the angles at which the shear cracks formed evidenced this. Bond or tensile testing of the completed patch material to the concrete substrate was compromised when using a 15-pound jackhammer for concrete removal. Other removal methods, such as hydro-demolition, were suggested. As a result of

Needham's research, MDOT plans were prepared detailing an end repair method for prestressed concrete I-beams with and without end blocks.

The suggested prestressed concrete I-beam end repair technique was executed in 1999 in Lower Michigan (Needham, 2000). Although numerous problems were encountered in the field repairs, they appeared to be attributed to contractor/engineer miscommunication and not necessarily the design details. Additionally, prestressed concrete I-beams repaired using the technique were found to be economically attractive compared to complete superstructure replacements (35 to 69 percent of replacement cost) and were estimated to have a life of 30 to 40 years.

Whether to repair or replace a member is an early critical decision that an engineer will need to make. General criteria are stated within NCHRP Report No. 226 for whether to replace or repair prestressed concrete beams having damaged concrete (Shanafelt and Horn, 1980). These criteria appear to have been developed from the findings of their multi-survey.

Load relief for the structure must be provided when performing repairs in compression zones to allow surface repairs to carry load (Emmons, 1994). Others have cited this need as well and refer to it as preloading (Shanafelt and Horn, 1980; Keating and Fisher, 1987; Xanthakos, 1996). Depending on the location of the concrete patching along the span and height of an element, pre-loading may be essential to keep repairs from experiencing in-service tensile stresses (Keating and Fisher, 1987; Xanthakos, 1996).

Dimensionally stable repair materials are required to have the repaired concrete carry load after removal of the load relief provisions (Emmons, 1994). Standard procedures exist in Michigan for shoring of superstructure beams (MDOT, 2001a). Shoring may be required to safely make repairs to beams.

Weyers et al suggest two pairs of techniques to identify concrete that is to be removed (Weyers et al, 1993). They are visual observation and hammer sounding (for low-effort removal over defined areas), and coring and half-cell measurement (for high-effort removal over undefined areas). Removal methods must be selective, preserve the substrate, and provide a quality bonding surface. Weyers et al suggest defining limits of removal by a four-category classification that includes surface, concrete cover, matrix, and core concrete removal.

One of the benefits of performing a partial depth repair is that the process permits unsound or poor quality concrete to be replaced. The limits of removal and quality of concrete can be easily seen during removal operations. Once removal has been performed, other protective strategies, such as the application of a corrosion inhibitor or reinforcement coatings, can be applied (Al-Qadi, 1993). Lastly, the removal of contaminated concrete and subsequent replacement with fresh concrete is an effective means of reducing reinforcing steel corrosion (Al-Qadi, 1993).

Different removal techniques and equipment are used depending on the depth location of the concrete to be removed. Each technique has strengths and weaknesses (Weyers et al, 1993). Hydrodemolition is a technique in which high pressure water is used (around 20 to 40-ksi) to remove concrete of any condition. Hydrodemolition is an attractive removal option because it cleans the reinforcing bars as it removes surrounding concrete without causing damage to the remaining concrete or steel (Weyers et al, 1993). Due to the size and configuration of equipment currently available, this technique has been successful in large, horizontal applications such as parking structures and bridge decks (Weyers et al, 1993). However, the research team has yet to learn that hydrodemolition can effectively be used in difficult access, vertical applications. With hydrodemolition, variability in the depth of concrete removal may be encountered if the quality of the concrete changes in depth or area (Weyers et al, 1993).

Other concrete removal methods include the use of pneumatic and electric breakers and rotary hammers. These processes are flexible, but also the most labor intensive and production rates are slow (Weyers et al, 1993). In addition, concrete and reinforcing steel in the removal area can be damaged in the process (Weyers et al, 1993). In work conducted for this project, electric rotary hammers were found to be a feasible means of removing deteriorated concrete, after it had first been scored with saw cuts. The rough, angular resulting surface profile was similar to that achieved by pneumatic breakers, and favorable for bonding repair materials (Weyers et al, 1993).

As with concrete removal processes, several options exist for preparing the roughened concrete surface. Depending on the requirements of the repair material, surface preparation methods consisting of compressed air blasting, detergent washing, water blasting, grit blasting, sand blasting, scabbling, or mechanical abrasion may be appropriate. One researcher has found that using sandblasting techniques has increased the bond strength of overlays when specific contact inhibitors are used (Al-Qadi, 1993).

Bond of the repair material to the substrate is of concern. A layer of bonding material is often placed at the transition zone (between the repair material and the substrate) to promote adhesion to the substrate. This material may consist of a thin layer of the repair material or a different material compatible with both the substrate the repair material. The Illinois Department of Transportation has used patches that were mechanically anchored to the concrete using pins and welded wire fabric reinforcement (Xanthakos, 1996).

Protection of reinforcement is considered important to the success of a patch. Methods to prepare exposed reinforcement in a repair are generally similar to those used for concrete. In addition, supplemental measures may be taken to retard corrosion of the reinforcement while the repair is in-service. Some of these methods include placing a highly alkaline material (concrete mortar) around the perimeter of the bar, installing a passive or impressed current cathodic protection system, applying a penetrating corrosion inhibitor, or providing electrical insulation protection (epoxy or zinc-based coatings). Penetrating corrosion inhibitors and cathodic protection are discussed elsewhere in this chapter.

In a study by Whiting et al (1998), coatings were applied to reinforcement in prestressed concrete elements prior to repairs being made. Exposed reinforcement in the patches was coated with either a zinc-rich paint or an epoxy. As a result of the study, epoxy coated reinforcement was found not to perform was well as reinforcement coated with zinc-rich paint.

Conditions in the field may warrant that internal strengthening measures be taken. These conditions may consist of internal reinforcement that has been corroded to a point of significant section loss. If necessary, replacement or supplement of this reinforcement could be performed in accordance with applicable design codes during a PDR.

Repair materials that may be feasible for partial depth repairs include, but are not limited to portland cement concrete, portland cement mortar, magnesium phosphate cement and shotcrete (Emmons, 1994; Kosmatka et al, 2001). Specific selection of the binder, aggregate, fillers, and polymer modifiers, further depends on desired performance of the material and the overall material type selected (e.g.: portland cement mortar vs. shotcrete). More information on the selection and proportioning of repair materials can be found in Emmons (1994) and Kosmatka et al (2001).

The ability for a surface repair to accommodate the imposed stresses is important (Emmons, 1994). Stresses form due to volume changes of the new and existing concrete as well as from service loads being carried by the repair. Nearly a dozen unique forces exist for the engineer to consider when designing surface repairs (Emmons, 1994).

Portland cement concrete and shotcrete repairs have been investigated by SHRP Report S-360. Weyers et al stress that neither of these techniques will be effective for longer than 15 years if the cause of the distress is not addressed. Additionally, all chloride-contaminated concrete should be removed from the areas surrounding repairs in order for the repair to be effective. Weyers et al also identify concrete and shotcrete containing corrosion inhibitors as repair materials having promise for superstructure repair. Procedures employing corrosion inhibitors in the repair material and as a surface applied treatment for the substrate were investigated. Products were manufactured by W.R. Grace and Company and Cortec Corporation in the Weyers et al study (1993).

For their research on the performance of repairs to prestressed concrete bridge components, Whiting et al selected conventional portland cement concrete, latex modified concrete, and silica fume concrete with corrosion inhibitor repair materials (Whiting et al, 1998). Whiting monitored the performance of the repairs during laboratory induced wetting-drying cycles by the use of half-cell potential, chloride ion, and visual measurements. Prestressed concrete slabs, beams, and piles were included in the research. For the beam specimens, half-cell potential measurements within the patch were more negative than –350mv (copper-copper sulfate half-cell), even after four wetting and drying cycles. These results were discounted as being representative of the steel corroding outside of the patch area. Chloride ion measurements were found to both vary widely (even considering a uniform application of deicer) as well as not be representative of active corrosion, upon visual examination. Silica fume portland cement concrete with an inorganic corrosion inhibitor was found to have the best overall performance on the beam specimens, compared to the conventional portland cement concrete patch and latex modified concrete patch. The overall performance rankings generally correlated with the half-cell potential testing, dissection observation, and surface observation trends. Upon dissection of the patches, corrosion of the central strand was commonly observed. Whiting et al postulated that the corrosion of this strand was caused by a lateral migration of chloride from the surrounding contaminated concrete.

The repair material chosen will largely govern placement and formwork options. Some options include form-and-pump, form and cast-in-place, hand application, low-pressure spraying and others (Emmons, 1994).

Options for curing repair materials primarily consist of moist or membrane curing methods. Moist curing methods (wet burlap covered with polyethylene sheeting) are often viewed as being more efficient than membranes (curing compounds), but application in vertical, difficult access areas, such as beam-ends, may be difficult.

External post tensioning is a technique to restore strength and durability (Keating and Fisher, 1987; Shanafelt and Horn, 1980; Xanthakos, 1996). External post-tensioning may be an option to consider for beam-ends that have distress extending a considerable distance from the end of the beam (Keating and Fisher, 1987).

Post-tensioning, metal sleeve splices, and internal splicing are methods recognized for restoring prestressing (Xanthakos, 1996). Post-tensioning and metal sleeve splices may be applicable to prestressed concrete I-beam end deterioration. Internal splicing does not appear practical due to the absence of a second fixed end for the prestressing strand. All three methods are generally considered when remedying serious damage.

External post-tensioning, internal splices, and sleeve splices are recognized to restore strength and durability to prestressed concrete I-beams (Shanafelt and Horn, 1985). Sleeve splices were also noted to be applicable to I-beams that may have experienced environmental damage.

Preston et al (1987) investigated and looked to restore flexural capacity to distressed prestressed concrete box beam bridges in Pennsylvania. As described in Preston et al, the repair techniques of NCHRP Report 280 were considered and implemented. Repair procedures for the box beam bridge included retrofit with epoxy coated reinforcing bars (not cited in NCHRP Report 280) and post-tensioned, epoxy coated tendons. Preston et al suggest investigating remaining capacity in a distressed beam before making repairs, as earlier designs were likely conservative compared to current design requirements. This conclusion may apply to both the box beams used in Preston's project and the prestressed concrete I-beams included in this research.

Lanyi summarized two projects in Alberta, Canada, where distressed prestressed concrete channel beams were rehabilitated (Lanyi, 1994). Damage to these bridges consisted of a high load hit and extensive corrosion of an exterior bottom channel leg. Repair techniques largely consisted of removing concrete, splicing in new prestressing strands, and patching with conventional concrete.

Fiber reinforced polymer (FRP) laminates appears to be gaining acceptance in the bridge repair arena as evidenced by a recent New York State DOT project (Hag-Elsafi and Alampalli, 2000). To design the FRP retrofit, existing condition data was first input into a computer program. The program was used to determine remaining capacity and design the FRP retrofit (Hag-Elsafi and Alampalli, 2000).

2.4.3.1.2 Electrical Control Methods

The United States is generally recognized as a leader in using cathodic protection as a preventive maintenance and repair technique for concrete bridge elements (Tilly, 1987). In the 1980's, impressed-current cathodic protection systems were preferred for large or important structures and often used in bridge deck applications (Arner and Panganiban, 1986; Tilly, 1987; Kennedy, 1991). Cathodic protection has limitations. Excessive or discontinuous protection may result in hydrogen embrittlement or active corrosion (ACI, 1996).

The success of the protection system used in Kennedy's project was based on whether a 100-mV potential shift could be measured between the reinforcement and a reference half-cell. Impressed current systems appear to be useful in applications where the system can be closely monitored. For the purposes of protecting prestressed concrete I-beam ends, it appears that galvanic or sacrificial anode cathodic protection systems are desirable over impressed current systems.

Response from the multi-state survey discussed in Chapter 5 did not indicate impressed-current cathodic protection is being used, however sacrificial anode systems are receiving attention.

One sacrificial anode available for use in repairs is the Galvashield XP, manufactured by Norcure (Beaudette, 2001b). The anode is constructed of a zinc core surrounded by a cement matrix. The Galvashield XP anode has been used in repairs to prestressed concrete I-beams in Iowa to provide protection against the accelerated corrosion of reinforcing steel. (Beaudette, 2001d).

The Galvashield XP anode is roughly the shape and size of a hockey puck and is to be installed within the original cross-section of the member during patching. Wire leads from the anode are provided to permit an electrical connection to the reinforcing steel (Beaudette, 2001b). Norcure specifies a maximum anode spacing of 30-inches for the XP anode. The Galvashield XP anodes are reported to have a service life of 10 to 20 years under normal conditions.

According to Daniel Burns of Vector Corrosion Technologies, the performance of the Norcure anodes is reportedly in accordance with non-published Vector recommendations (Burns, 2001).

2.4.3.2 *New Approaches*

Protection of concrete elements can be equated to controlling the cause of deterioration according to Emmons. Protection to eliminate future causes of new deterioration must be provided to prohibit return to a deteriorated state (Emmons, 1994). The high frequency of beam end deterioration with failed transverse deck joints suggests the proper joint maintenance is required (Whiting et al, 1998).

Because of the established link between the leaking transverse deck joints and beam end deterioration, some researchers have suggested modifying the deck drainage and possibly reducing the number and location of joints in a structure (Whiting et al, 1998). A deck overlay may be a means to eliminate the joints, however consideration should be given to providing a high quality overlay that will not foster deterioration of the substrate or support beams.

Bearing modifications are a potential solution to bearing region cracking in prestressed concrete I-beams. Supporting documentation for the need to modify bearings to mitigate distress has not been found.

2.5 MDOT Practice

2.5.1 Use and Evolution of Prestressed Concrete in Michigan

According to a survey submitted for NCHRP Report 90, the first prestressed concrete bridge in Michigan was constructed in 1951 (Moore et al, 1970). Little is known about this first bridge, however, a substantial number of documents are available to document progress since.

Review of MDOT Design Division Informational Memoranda (IM) has provided insight to the changes in MDOT practice in the last 20 years (Till, 2001a). Changes summarized by the IM show progress in the MDOT design philosophy. Probably the most significant changes in the design practices as related to the problem of end-deterioration is the use of CLL design (MDOT, 1989). CLL design is the practice of designing a continuous deck to carry the bridge live load over simply supported beams intended to carry the structure dead load and live load. This practice effectively eliminates deck joints over beam-ends, where a joint would otherwise be placed to accommodate the seasonal movements of the superstructure. Several authors have recognized that the majority of prestressed concrete I-beam end distress occurs at bridge deck joints (Tilly, 1987; Whiting et al, 1998; Needham, 1999; Enright and Frangopol, 2000).

Other important MDOT design changes over the past three decades include use of epoxy-coated shear connectors (IM 320-B), re-designing the placement of bond breakers for strand ends (IM 332-B), omitting corrosion inhibitors from the mix design (IM 447-B), using larger diameter strand (IM 484-B), and sealing of certain width cracks (IM 332-B).

To potentially minimize deck and beam deterioration, shear connector "D" bars were specified to be epoxy coated by IM 320-B (MDOT, 1984a). "D" bars provide horizontal shear resistance between the beam and the deck slab. Since 1984, this is the only epoxy-coated reinforcement used in Michigan prestressed concrete I-beams.

Repair or maintenance of the bridge deck could have a positive, adverse, or no effect on prestressed concrete I-beams. For deck repairs, the use of high-early strength concrete was recommended by MDOT in 1991 (IM 402-B). The required cement content (846 pounds per cubic yard) is the only mix design requirement contained in IM 402-B. High *w/c* concrete could be prone to shrinkage not experienced by lower *w/c* mixes (Kosmatka and Panarese, 1994). Cracks created by the volume change may be of sufficient width to permit intrusion of chloride-laden solutions through the deck and to the beam (Whiting et al, 1998).

Calcium nitrate was used in the casting of prestressed concrete I-beams until 1992 (MDOT, 1992). It is not known when the use of calcium nitrate was initiated. When properly used, calcium nitrate is a corrosion inhibitor effective in reinforcing and stabilizing the passive oxide film on the steel reinforcement (Kosmatka and Panarese, 1994). According to IM 447-B, the use of calcium nitrate was discontinued because there was limited documentation of severe corrosion to prestressed concrete I-beams. MDOT has now recognized that these problems do exist, but only in the end regions near transverse deck joints (Jadun, 1990; Needham, 1999).

Designers recognized the benefits of larger diameter strand with the release of IM 484 (MDOT, 1997a). Permissible prestressing strand diameter changed with this IM from 0.5-inch to 0.6-inch. This change is significant from a design and end deterioration standpoint because material loss or degradation has a greater effect on a 0.5-inch diameter strand compared to a 0.6-inch diameter strand.

Sealing longitudinal cracks greater than 10-mils in width was recommended by IM 332-B to prevent strand corrosion (MDOT, 1984b). Sealing cracks is a positive step in preventing member deterioration, however cracks as small as 4-mils have been reported to be sufficient to permit localized corrosion of reinforcement (Moore et al, 1970).

Table 2-3 summarizes the design evolution of prestressed concrete I-beams in Michigan.

2.5.2 Current Design

Several MDOT documents are available for the use in the design of new prestressed concrete I-beam bridges. The Bridge Design Manual is intended to be a single source reference for MDOT design engineers and consultants assigned the responsibility of producing bridge plans (MDOT, 2001a). Much of the information in the MDOT Bridge Design Manual originated from Bridge Squad Leaders' Notes.

The types of documents available to the bridge designer to assist in producing plans include maintenance reports, scoping reports, and other inspection data (MDOT, 2001a). The level of detail included in these reports and data is of interest to the project herein. The level of detail potentially indicates how accurate or reliable the data is. Work by other researchers has identified that the greatest amount of distress occurs in an otherwise concealed region behind the diaphragm (Ahlborn et al, 2001). Whether or not field inspectors are getting behind the diaphragms to observe the entire beam end is of interest. From a review of the Scoping Checklist in Appendix 2.02.19 A.3 of the Bridge Design Manual, observations related to beam end repairs are to be performed (MDOT, 2001a).

While past MDOT research projects have example plans for restoration or repair projects (Needham, 2000), the Bridge Design Manual does not. This lack of a sample set of restoration or repair drawings may lead to inconsistencies or omissions between different projects, thereby increasing project costs.

The MDOT Bridge Design Guides serve as an aid for consistently designing and detailing bridges (MDOT, 2001b). In this sense, they serve a similar function to the example plan sheets of the Bridge Design Manual. Of particular interest to this project are the current details that are included in Chapter Six – Superstructure. Section 6.60 contains several current standard design details for prestressed concrete I-beams. In particular, design sheet 6.60.13 contains prestressed concrete I-beam connection details being used at piers and abutments.

Table 2-3. Summary of Changes in MDOT Practice

Changes Occurred in	Bridge Elements	Materials or Section Properties	Years 1958	1959	1964	1975	1989	1997
Materials	Beam	f's (psi)	250,000	250,000	250,000	270,000	270,000	270,000
		f'c (psi)	5,000	5,000	5,000	5,000	5,000	5,000
	Deck	f'c (psi)	3,000	3,000	3,000	3,500	3,500	3,500
Design	Beam	Beam Types	AASHTO I-IV	AASHTO I-IV	AASHTO I-IV	AASHTO I-IV WI 70-inch Beam*	AASHTO I-IV	AASHTO I-IV MI 1800 Beam
		E deck / E beam	0.77	0.77	0.77	0.83	0.80	0.80
		Cover on Top (in)	3.00	3.00	3.00	3.00	3.00	3.00
		Cover on Bottom (in)	2.00	2.00	2.00	2.00	2.00	2.00
		Beam Spacing (ft)	5 to 7	4 to 7	5 to 9	6 to 10	Info n/a	6 to 11
		Span Length (ft)	30 to 80	60 to 100	30 to 90	20 to 100	Info n/a	90-150 (MI1800)
		Grade of P. Steel	250	250	250	270	270	270
		Nom. diam of P. Steel (in)	7/16"	7/16"	1/2"	1/2"	1/2"	3/5"
		Draped Strands	No	Yes	Yes	Info n/a	Info n/a	Yes
		Coated reinforcement	No	No	No	No	Epoxied D-Bars	Epoxied D-Bars
		Bond Breaker Used	No	No	No	No	Yes	Yes
	Deck	Thickness (in)	8" constant	8" constant	7" to 9" varies with beam spacing	8" to 9 1/2" varies with beam spacing	Info n/a	Info n/a
		Sections	Precast	Precast	Precast	Precast	Precast	Precast
		Span	Simple	Simple	Simple	Simple	Simple	Simple
		Strands	Pretensioned	Pretensioned	Pretensioned	Pretensioned	Pretensioned	Pretensioned
		Coating	Uncoated	Uncoated	Uncoated	Uncoated	Uncoated	Uncoated
		Calcium Nitrate Added	No	No	No	No	Yes	No
Manufacturing & Construction	Deck	Sections	Cast in Place	Cast in Place	Cast in Place	Cast in Place	Cast in Place	Cast in Place
		Support Conditions	Simple	Simple	Simple	Simple	Continuous for LL	Continuous for LL

*The use of the WI-70 beam began in 1977.

Table 2-3 was compiled with information from the following sources:

Documents
1. General Instructions for Designing Precast Prestressed Concrete I-Beams (6-1957)
2. Prestressed Concrete Beams. AASHO Standard Sections (3-6-1958, Rev. 3-27-1958)
3. Prestressed Concrete Beams. AASHO PCI Standard I Sections with Draped Strands (6.60a, Issued 11-1-1959)
4. Prestressed Concrete I-Beams. AASHO-PCI Standard I Sections (6.60.03, September 1963)
5. Prestressed Concrete I-Beams. AASHO-PCI Standard I Sections (6.60.01, July 1964)
6. Historical Information from Squad Leader Notes, p. BB3a (6-5-1963)
7. Historical Information from Squad Leader Notes, p. BB1 (6-12-1974)
8. Prestressed Concrete I-Beams (6.60.01, April, 1975)
9. MDOT Design Division Informational Memorandum 285B (June 8, 1981)
10. MDOT Design Division Informational Memorandum 318-B (January 4, 1984)
11. MDOT Design Division Informational Memorandum 320-B (February 2, 1984)
12. MDOT Design Division Informational Memorandum 321-B and 268-R (February 2, 1984)
13. MDOT Design Division Informational Memorandum 332-B (July 3, 1984)
14. MDOT Design Division Informational Memorandum 336-B (May 5, 1986)
15. MDOT Design Division Informational Memorandum 348-B (April 8, 1985)
16. MDOT Design Division Informational Memorandum 361-B (December 3, 1985)
17. MDOT Design Division Informational Memorandum 366-B (May 5, 1986)
18. MDOT Design Division Informational Memorandum 368-B and 311-R (June 24, 1986)
19. MDOT Design Division Informational Memorandum 378-B (March 17, 1987)
20. MDOT Design Division Informational Memorandum 385-B, Revised IM 378-B (September 30, 1987)
21. MDOT Design Division Informational Memorandum 402-B (July 1, 1991)
22. MDOT Design Division Informational Memorandum 411-B (August 29, 1989)
23. MDOT Design Division Informational Memorandum 446-B (July 15, 1992)
24. MDOT Design Division Informational Memorandum 447-B (August 3, 1992)
25. MDOT Design Division Informational Memorandum 447-B (August 3, 1992)
26. MDOT Design Division Informational Memorandum 458-B (June 8, 1995)
27. MDOT Design Division Informational Memorandum 484-B (December 5, 1997)

Plans and Drawings

28. Stirrup from Superior Product Company (02-10-1976)
29. Prestressed Concrete I-Beam Details. PC-1B. (7-25-1979)
30. Prestressed Concrete I-Beam Details. PC-1F. (5-16-1985)
31. 70" Prestressed Concrete I-Beam Details. PC-2E. (5-16-1985)
32. Bearing Details for Prestressed Concrete I-Beams (4-10-1990)
33. Bearing at Abutments with Prestressed I-Beams (4-10-1979, 8-9-1971, 11-2-1970, 5-21-1962)

2.5.3 Bridge Management

The thousands of bridges in Michigan require a concerted effort to be maintained in a level safe for the traveling public. Successful efforts are realized in an effective bridge management program. The foundation of a bridge management program is the field inspection. Though it is possible for a field inspection to be performed for any number of reasons, bridge inspections are typically performed as part of a Pontis or NBIS inspection program.

The Pontis inspection is performed to provide estimated future cost information for the maintenance of bridges. Pontis inspections commonly document construction quantities on a bridge, so that repair and replacement estimates can be developed. On the other hand, NBIS inspections are meant to assess the structure and the overall safety of the structure. The NBIS generally apply to all bridges on public roads with a span greater than 20-feet (CFR Title 23, 1999). The NBIS require an inspection organization to be established by each state. Also required of the states is the preparation and maintenance of their bridge inventory. According to the NBIS, a maximum bridge inspection frequency of two years is required. A random review of bridge inspection reports from Michigan has shown conformance with the inspection frequency requirements established by the NBIS.

Inspection procedures to be used to satisfy NBIS requirements are presented in the Manual for Condition Evaluation of Bridges (AASHTO, 2000) and the Interim Revisions to the Manual (AASHTO, 2001). The Manual for Condition Evaluation of Bridges also includes sections for determining the load rating of a bridge. Suggestions for distress conditions to observe in prestressed concrete beams, bridge drainage structures, and bearings are included in the Manual for Condition Evaluation of Bridges. The bearing region condition of prestressed concrete beams is to be checked per the Manual for Condition Evaluation of Bridges. Various types of material tests for the bridge inspector to consider are also included in the Manual. Tests for concrete include strength, sonic, ultrasonic, magnetic, electrical, nuclear, thermography, radar, and radiography based techniques (AASHTO, 2000).

3.0 Field Investigation (Task 2)

3.1 Introduction

The focus of Task 2 was to conduct an extensive field investigation of twenty PC I-beam bridges to observe the distress at the beam-ends. This chapter is divided into seven sections. First, a field visit to a precast concrete plant is described. Next, the process of how the field specimens were selected is documented. Then the inspection preparation and protocol are clearly defined. The research team's field experiences are summaries. A thorough review of each bridge's inspection data is given. Lastly, the organization of the inspection data is explained. This chapter provides a comprehensive outlook on the current sate of distress in PC I-beam ends in Michigan.

3.2 Precast Plant

On June 19, 2001, the research team visited Premarc, a prestressed concrete plant. Premarc is located on Chicago Drive SW of Grand Rapids in Michigan. The Premarc Corporation, is a closely held, family owned Michigan manufacturing concern, founded in 1925. Since then, Premarc has supplied the construction industry with concrete products such as prestressed concrete I-beams for highways and bridges. The manager of the plant showed the research team the products made at Premarc and led them through the process of manufacturing prestressed concrete.

3.2.1 Concrete Plant

The plant has its own facility for concrete mixture design and manufacturing. Commercial software is utilized for the design of concrete mixture. The computerized equipment is used to produce a desired strength from the concrete.

3.2.2 Prestressing Systems and Anchorages

The plant has several open air pretensioning beds. Buckets are used to transport concrete approximately 500-feet from the plant to the beds. Beams are cast in permanent type beds, where forms are fixed in place. A precast stressing bed of a long reinforced concrete slab is cast on the ground with vertical anchor bulkheads at its ends. Prestressing steel is pretensioned against independent anchorages prior to the placement of concrete around it. Prestressing can be accomplished by prestressing individual strands, or all the strands at one jacking operation.

One of the components of a prestressing operation is the jacking system applied, i.e., the manner in which the prestressing force is transferred to the steel tendons. Such a force is applied through the use of hydraulic jacks of different capacity; depending on whether individual tendons are being prestressed or all the tendons are being stressed simultaneously.

3.2.3 Quality Control

Precast concrete is cast into complex shapes. Plant-cast precast concrete components are not fabricated under optimum conditions of forming. For example, the process of the pretensioning and concrete placement is taking place in the open space. Thus the requirement to maintain the certain temperature is not satisfied. This may be one of the reasons why most of the prestressed concrete I-beams in stock have hairline cracks along the flange.

3.3 Selection of Field Specimens

The objective of this project is to inspect a full spectrum of bridges at different ages and conditions. Beam types were not taken into consideration, however access issues were a primary concern. For this reason, only "highway-over-highway" bridges were selected. Other access parameters such as "bridge clearance" and the presence of a shoulder or sidewalk on the "featured intersection" were reviewed on the site plan.

A total of 20 prestressed concrete I-beam bridges were selected in order to document the detailed beam end conditions and general conditions of the deck and substructure. In order to initiate the field investigation; Grand Region (23 bridges), North Region (26 bridges) and Bay Region (17 bridges) have been selected for the prestressed concrete I-beam bridge pool.

The information collected was reviewed and organized in a tabular form. The tables contain information on bridge ID, year built, geometry, characteristics and ratings/conditions, etc. (see Appendix A for Grand, Appendix B for North and Appendix C for Bay). The primary parameters for selecting the bridges to be inspected were the year built and the deck and stringer ratings/conditions. The bridges subjected to high load hits were automatically eliminated.

In Table 3-1, Table 3-2, and Table 3-3 the shaded bridges indicate that particular bridge was studied for the finite element modeling analysis in Chapter 10.

3.3.1 Grand Region

The research team selected a group of 5 bridges satisfying the criteria (see Table 3-1) out of a pool of 23 prestressed concrete I-beam bridges that are highway-over-highway (see Appendix A).

The selection was made based on the bridge rating during the most recent inspection (6-9 for deck, 6-9 for stringer, 6-8 for abutment, and 5-7 for pier) and year built (1961, 1963, 1964, 1969 and 1972). The list was reviewed with respect to inspection feasibility (traffic control and access issues).

Table 3-1. Bridges Identified for Inspection in Grand Region

NBI No.	County	Year Built	Facility Carried	Feature Intersected
41025 S07	Kent	1961	KNAPP ST	I-96
41027 S06	Kent	1963	US-131 NB	6TH AVE
41029 S16-3	Kent	1964	I-196, M-21 EB	LANE AVE
41029 S16-4	Kent	1964	I-196, M-21 WB	LANE AVE
41029 S23	Kent	1972	I-196 WB	36TH ST

3.3.2 North Region

There is a concentration of new prestressed concrete I-beam bridges in the North Region. Our goal was to inspect these bridges in order to document their condition at an early age. Out of a set of 26 prestressed concrete I-beam bridges located (see Appendix B) in this region, five bridges were selected for inspection (see Table 3-2). The most recent ratings of these bridges were 7-8 for deck, 8 for stringer, 7-8 for abutment, and 7-8 for pier. A bridge engineer verified the easy accessibility of all these bridges, including two located on a portion of US-131 that was not yet open to traffic.

Table 3-2. Bridges Identified for Inspection in North Region

NBI No.	County	Year Built	Facility Carried	Feature Intersected
67016 S09	Oceola	1984	US-131 N B	US-10
67016 S10	Oceola	1984	US-131 S B	US-10
53034 S05	Mason	1986	CHAUVEZ RD	US-31
83033 S06	Wexford	1997	NO. 36 ROAD	US-131
83033 S03	Wexford	1998	WHALEY RD	US-131 RELOC.

3.3.3 Bay Region

Based on the latest bridge rating (4-8 for deck, 4-7 for stringer, 6-7 for abutment, and 4-7 for pier) and year built (1961, 1967, 1968, and 1969, 1971) 10 bridges (see Table 3-3) out of 17 were selected by the research team for inspection (see Appendix C).

Table 3-3. Bridges Identified for Inspection in Bay Region

NBI No.	County	Year Built	Facility Carried	Feature Intersected
06111 S04	Arenac	1968	I-75 NB	M-61
06111 S05	Arenac	1968	LINCOLN RD	I-75 SB
06111 S06	Arenac	1968	LINCOLN RD	I-75 NB
06111 S11	Arenac	1968	M-33	I-75
25042 S12-8	Genesee	1967	I-69 RAMP F	I-75
25042 S12-3	Genesee	1969	I-69 EB	I-75
25042 S12-4	Genesee	1969	I-69 WB	I-75
25042 S12-7	Genesee	1969	I-69 RAMP E	I-75
25132 S34	Genesee	1971	I-475 SB	CLIO RD
29011 S03	Gratiot	1971	US-27 NB	US-27BR (POLK RD)

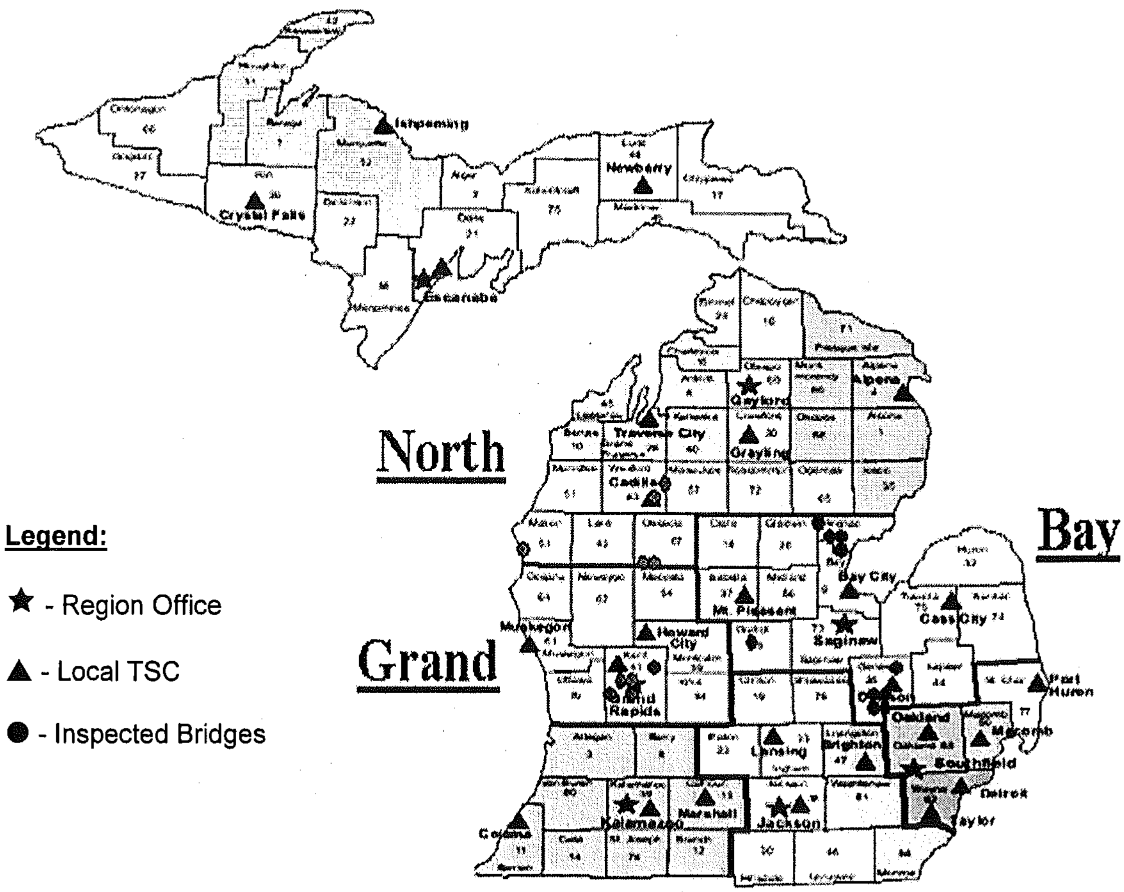

Figure 3-1. Site Location of the 20 Inspected Bridges

Figure 3-1 shows a visual summary of the 20 inspected bridges within their respective region.

3.3.4 Bridge Categories by Design and Loading

The classification of the inspected bridges according to design is based on the data obtained from MDOT, Construction and Technology Division.

Based on the data collected, twenty inspected prestressed concrete I-girder bridges are categorized by their different design characteristics, such as deck type, bearing type, girder type, diaphragm type on piers (end diaphragms) and intermediate diaphragms, diaphragm type on abutments/backwalls, as well as traffic load under and on the bridge. The information below is based on the data summarized in Table I-1 (Appendix I). Also, this information is sorted in order to provide better understanding of bridges distribution according to deck bearing types (see Table 3-4), girder diaphragm (see Table 3-5), and loading on and under bridges (see Table 3-6).

Bridge design categories are subdivided by (1) deck, (2) bearing pads on abutments and piers, (3) girder and (4) diaphragm types, and (5) loading as shown below.

(1) Deck type:

Composite: 41029S23, 41029S163, 41029S164, 41027S060, 41025S070, 67016S090, 67016S100, 53034S050, 83033S050, 25042S124, 25042S123, 25042S128, 25042S127, 06111S11, 25132S34, 29011S03, 06111S04, 06111S05, 06111S06

Continuous: 83033S060

(2) Bearing pad type:

Abutments:

Elastomeric Pad: 41029S23, 67016S090, 67016S100, 53034S05, 83033S060, 83033S050, 25042S124, 25042S123, 25042S128, 25042S127, 06111S11, 25132S34, 29011S03, 06111S04, 06111S05, 06111S06

Neoprene Pad: 41029S163, 41029S164, 41027S06, 41025S07

Piers:

Elastomeric Pad: 41029S23, 53034S05, 83033S050, 25042S124, 25042S123, 25042S128, 25042S127, 06111S11, 25132S34, 06111S04, 06111S05, 06111S06

Neoprene Pad: 41029S163, 41029S164, 41027S06, 41025S07, 29011S03

No: 67016S09, 67016S10, 83033S06

(3) Girder type:

AASHTO I, II, III: 41029S16-3, 41029S16-4, 06111S11, 25132S34

AASHTO I, II: 29011S03

AASHTO I, III: 41029S23, 06111S04

AASHTO II, III: 41027S06, 41025S07, 06111S05, 06111S06

AASHTO III: 25042S12-4, 25042S12-3, 25042S12-8, 25042S12-7

Wisconsin 70": 67016S09, 67016S10, 83033S05

Wisconsin 70", AASHTO I: 53034S05

MI 1800: 83033S06

(4) Diaphragm type:

Abutments/Backwalls:

Type A: 67016S09, 67016S10, 29011S03

Type B: 41025S07, 29011S03

No: 41029S23, 41029S16-3, 41029S16-4, 53034S05, 83033S06, 83033S05, 25042S12-4, 25042S12-3, 25042S12-8, 25042S12-7, 06111S11, 25132S34, 06111S04, 06111S05, 06111S06

End Diaphragms (On Piers):

Type A: 41027S06, 25042S12-4, 25042S12-3, 25042S12-8, 25042S12-7

Type Dm: 83033S05

Multiple (more than one present): 41029S23, 41029S16-3, 41029S16-4, 41025S07, 53034S05, 06111S11, 25132S34, 29011S03, 06111S04, 06111S05, 06111S06

No: 67016S09, 67016S10, 83033S06, 83033S05

Intermediate Diaphragms (At the middle of span):

Type A: 67016S09, 67016S10, 25132S34,

Type Am: 83033S06, 83033S05

Type G: 06111S04

Type I: 25042S12-4, 25042S12-3, 25042S12-8, 25042S12-7

Multiple: 41029S23, 41029S16-3, 41029S16-4, 41027S06, 41025S07, 53034S05, 06111S11, 29011S03, 06111S04, 06111S05, 06111S06

No: 41029S23, 67016S09, 67016S10, 83033S06, 83033S05, 25132S34, 06111S04

(5) Loading:

ADT under:

Less then 1,000: 29011S03

1,000-10,000: 41029S23, 41029S16-3, 41029S16-4, 41027S06, 67016S09, 67016S10, 25132S34, 06111S04, 06111S05, 06111S06

10,000-20,000: 06111S11

20,000-50,000: 41025S07

Over 50,000: 25042S12-4, 25042S12-3, 25042S12-7

N/A: 53034S05, 83033S06, 83033S05, 25042S12-8

ADTT under:

0 or less: 41029S23, 41029S16-3, 41029S16-4, 41027S06

0-1,000: 67016S09, 67016S10, 25132S34, 29011S03, 06111S04, 06111S05, 06111S06

1,000-10,000: 41025S07, 25042S12-4, 25042S12-3, 25042S12-7, 06111S11

N/A: 53034S05, 83033S06, 83033S05, 25042S12-8

ADT on:

Less then 1,000: 06111S05, 06111S06

1,000-10,000: 41025S07, 67016S09, 67016S10, 25042S12-7, 06111S11, 25132S34, 29011S03, 06111S04

10,000-20,000: 41029S23

20,000-50,000: 41029S16-3, 41029S16-4, 25042S12-4, 25042S12-3

over 50,000: 41027S06

N/A: 53034S05, 83033S06, 83033S05, 25042S12-8

ADTT on:

0 or less: 06111S05, 06111S06

0-1,000: 41025S07, 67016S09, 67016S10, 25042S12-7, 06111S11, 25132S34, 29011S03, 06111S04

1,000-10,000: 41029S23, 41029S16-3, 41029S16-4, 41027S06, 25042S12-4, 25042S12-3,

N/A: 53034S05, 83033S06, 83033S05, 25042S12-8

Table 3-4. Bridge Distribution According to Deck and Bearing Pad Type

	Deck Type		Bearing Pad Type					
			On Abutments			On Piers		
	Composite	Continuous	Elastomeric	Neoprene	None	Elastomeric	Neoprene	None
No. of Bridges	19	1	16	4	0	12	5	3

Table 3-5. Bridge Distribution According to Girder and Diaphragm Type

	Girder Type								Diaphragm Type[i]												
									Abutments/ BackWalls			End Diaphragms (On Piers)				Intermediate Diaphragm					
	I, II, III	I, II	I, III	II, III	III	Wisconsin 70"	Wisconsin 70", AASHTO I	MI 1800	Type A	Type B	Not present[ii]	Type A	Type Dm	Multiple[iii]	Not present	Type A	Type Am	Type G	Type I	Multiple	Not present
No. of Bridges	4	1	2	4	4	3	1	1	3	2	16	5	1	11	4	3	2	1	4	11	7

[i] Diaphragm type – see Appendix I, Table I-2, [ii] Not present – No diaphragm is present at the bridge
[iii] Multiple –Two or more different diaphragm types are presented in the bridge
Note: Total number of bridges appearing in the Table 3-5 may be more than twenty. This occurs when bridges fall into two different design categories.

Table 3-6. Bridge Distribution According to Loading On and Under Bridge

	Loading																			
	ADT_u[i]						ADTT_u[ii]				ADT_on[iii]						ADTT_on[iv]			
	<1,000	1,000-10,000	10,000-20,000	20,000-50,000	> 50,000	N/A	< 0	0-1,000	1,000-10,000	N/A	< 1,000	1,000-10,000	10,000-20,000	20,000-50,000	> 50,000	N/A	< 0	0 - 1,000	1,000-10,000	N/A
No. of Bridges	1	10	1	1	3	4	4	7	5	4	2	8	1	4	1	4	2	8	6	4

[i] Average Daily Traffic Under Bridge, [ii] Average Daily Truck Traffic Under Bridge
[iii] Average Daily Traffic On Bridge, [iv] Average Daily Truck Traffic On Bridge

3.4 *Inspection Preparation and Training*

Prior to the actual fieldwork, the preparation for inspection consisted of planning the inspection, a review of field documentation and inspection procedures, and understanding safety and traffic control issues. A list of the tools used during the inspection is given in Appendix D. Also, the shoulder closure plan and traffic control equipment is given in Appendix E and F respectively.

3.4.1 Planning the Inspection

The principal inspection goal was to document the condition of the prestressed I-beam ends, diaphragms, and general structure of the bridges. The documentation included mapping the beam-end conditions on a template, notes on diaphragm conditions and bearings, and brief notes on the deck, barriers, abutments and piers conditions. Notes also included the structural system such as beam-end restraints, deck system and expansion joints. Also, it was important to note the condition of drainage systems, joints, embankments, and utility lines.

3.4.2 Inspection Schedule

At this stage contacts with the regional MDOT offices were made to help correlate the trips scheduled for bridge inspection. The final schedule for the inspection was established as shown in Table 3-7 and Table 3-8.

Table 3-7. Bridge Inspection Schedule

Region	No. of Inspection Days	Inspection Dates	No. of Bridges	Total No. of Beams Inspected
Grand	4	07/12/01-07/15/01	5	132
North	3	07/18/01-07/20/01	5	53
Bay	3	07/23/01-07/25/01	5	130
	3	08/27/01-08/29/01	5	99

Table 3-8. Prestressed Concrete I-beam Bridges Inspected

Region	NBI No.	County	Year Built	No. of Spans	No. of Beams per Span	Beam Type	Total No. of Beams per Bridge	Inspection Date
Grand	41141029000S230	Kent	1972	3	8	I, III	24	July 12,01
	41141029000S163	Kent	1964	3	8	I, II, III	24	July13,01
	41141029000S164	Kent	1964	3	8	I, II, III	24	July 13,01
	41141027000S060	Kent	1963	3	12	II, III	36	July 14,01
	41141025000S070	Kent	1961	4	6	II, III	24	July 15, 01
North	67167016000S090	Oceola	1984	1	6	Wisc.70	6	July 18, 01
	67167016000S100	Oceola	1984	1	7	Wisc.70	7	July 18, 01
	53153034000S050	Mason	1986	4	6	Wisc.70	24	July 19, 01
	83183033000S060	Wexford	1997	1	8	MI 1800	8	July 20, 01
	83183033000S050	Wexford	1998	2	4	Wisc.70	8	July 20, 01
Bay	25125042000S128	Genesee	1967	4	5/2, 3/2	III	16	July23-24,01
	25125042000S123	Genesee	1969	4	7/2, 4/2	III	22	July23-24,01
	25125042000S124	Genesee	1969	4	7/2, 4/2	III	22	July23-24,01
	25125042000S127	Genesee	1969	4	5/2, 3/2	III	16	July23-24,01
	06106111000S110	Arenac	1968	6	9	I, II, III	54	July 25, 01
	25125132000S340	Genesee	1971	4	6	I, II, III	24	Aug. 27, 01
	29129011000S030	Gratiot	1961	3	9	I, II	27	Aug.27-28, 01
	06106111000S040	Arenac	1968	3	6	I, III	18	Aug. 28, 01
	06106111000S050	Arenac	1968	3	5	II, III	15	Aug. 29, 01
	06106111000S060	Arenac	1968	3	5	II, III	15	Aug. 29, 01

3.4.3 Field Documentation and Inspection Procedures

Based on the general plan of the structure, the team prepared templates that contained a beam plan, beam face templates and room for survey notes and annotations. The main goal was to collect detailed information on beam-end and diaphragm conditions. For use in the field, template plans of beams, diaphragm location and orientation, the beam and diaphragm elevations were prepared for each bridge. The templates were to be completed in the field by the individual inspector looking at a specific beam face.

Prior the inspection trips, the team reviewed procedures and documentation according to the Bridge Inspector Training Manual (Hartle et al, 1995). These procedures included the following:

a. Report all visible cracks, recording their width, length, location and orientation (horizontal, vertical, or diagonal). Check beam flange surfaces for longitudinal cracks. Inspect the beam webs for structural cracks.
b. Note any corrosion and efflorescence stains.
c. Record area, location, depth, and general characteristics of concrete scaling.
d. Document concrete surfaces delamination using sketches showing the location and pertinent dimensions.
e. Sketch spalling and the presence of exposed reinforcing steel.
f. Examine the areas near the bearings and the cast-in-place end diaphragms for spalling concrete, bearing condition and sole plate corrosion.
g. Inspect the end diaphragms for cracking and spalling.
h. Document high load hits on beams.
i. Document any repairs that have been made previously.

3.4.4 Safety & Traffic Control Issues

The fundamental safety practices the inspector should be aware of, according to Manual 90 (Hartle et al, 1995), were reviewed in preparation for the inspection. The most relevant practices pertaining to the particular project were discussed with the inspection team prior to each trip.

A traffic control plan was prepared and presented to everyone on the team (see Appendix E: Traffic Control Plan for a Shoulder Closure and Appendix F: Traffic Control Equipment). The traffic control was limited to a shoulder closure on most inspection sites. The four bridges in Genesee County with featured intersection on I-75 required lane closures. Genesee County provided the traffic control for the inspection team.

3.5 Field Inspection Protocol

The site inspections were performed to visually and physically examine each bridge component to document its condition in detail. The structural components that were inspected were beams, diaphragms, deck, joint, drainage system, piers, and abutments. The details of the inspection procedure for each component are described in the following sub-sections.

3.5.1 Beams & Diaphragms

The first purpose of the beam inspection was to document the condition of the beam-end and to identify the cause of distress. The second purpose was to collect data in order to develop an inspection procedure for identifying corrosion prone beam-ends. Using the templates of the beams and the diaphragms (see Figure 4-6. Example of Field Investigation Template Form) the research team documented the elements' details and documented all their findings. The team also took specific field measurements, such as crack widths and delaminated and spalled areas, which detailed the level of deterioration in elements.

During the time of actual field investigation the research team performed the following tasks:

1. Measurement of cracks (location, orientation, length, width, and coordinates), delaminations and spalls on beam-ends and diaphragms using a crack gage and hammers where necessary. Beam elevation sketches have been used as templates to document the information.
2. Noted the location, length, and corrosion level of any exposed rebar or tendons.
3. Noted the presence of rust stains and efflorescence and location.
4. Took photos of selected beams and diaphragms.
5. Checked and recorded type and condition of sole plates and bearings.

3.5.2 Deck, Joints & Drainage System

During the inspection of the deck the conditions of the joints, deck, wearing surface, and drainage systems were documented. The research team performed the following tasks:

1. Documented the estimates of deck deterioration. Noted how many cracks and orientation in each slab and the amount and location of spalling. Indicated asphalt overlay or other repair if any.
2. Recorded locations of expansion and control joints on the deck.
3. Noted if there was any sidewalk and type and condition of barrier.
4. Took photos depicting the deck condition.

Joints were inspected to document any leakage and presence of debris. The joints are particularly critical because they prevent the leakage of runoff and deicing chemicals to the substructure elements below the deck. Inspection of the bridge surface flow drainage system is performed to document the condition of pipes or channels of the drainage system.

3.5.3 Substructure

Concrete substructure components were inspected for cracking, spalling, delamination, and exposed reinforcement. Pier caps were examined for a buildup of moisture and excessive spalling. The research team performed the following tasks:

1. Documented cracks and spalls on piers and abutments and their location.
2. Identified the cause of any deterioration of substructure elements.
3. Took pictures of piers and abutments.

3.6 Review of Inspection Process and Field Experience

3.6.1 Inspection Process

The inspection of each bridge varied with bridge type, equipment and people available. The following steps provide a common basis of the process for each particular bridge.

Coordination. Prior to the inspections coordination meetings took place between all of the research team members. All arrived at the site with a clear understanding of what was to be accomplished and how to do it safely throughout the completion of the inspection.

Inform Authorities. The local MDOT office was informed prior to beginning the inspection.

Inspection Notes. The research team used a set of bridge plans and sketches as templates. The team supervisor reviewed the notes at the end of each day to make sure that the information was complete. The team took pictures of the structure and structural details, recording the roll and picture number. A description, location, and direction of the picture was also recorded. A digital camera was also utilized primarily to document the beam-ends.

General Course of Action. The research team looked over the entire bridge. It was helpful in order to get a feel of what problems may exist and how to use access equipment in specific areas. Once the plan of inspection was formulated, the inspection was started, generally at one end and worked across the bridge.

One inspector climbed around the bridge taking notes on abutment, pier, and deck conditions. Other members inspected the beam-ends, primarily using ladders. In one case, MDOT "reach all" equipment was utilized. In the inspection of the last five bridges a boom lift was used in addition to ladders.

Direct Physical Measurements. The condition survey of bridges included two stages of direct physical measurements:

1. Visual Inspections and Crack Measurement.

Results of the visual investigation were noted on a field investigation form. Cracks were measured and sketched on the template. The number beside each crack indicated the crack width. Photographs were taken to supplement the field notes since it was often difficult to describe the location and extent of deterioration of a bridge member solely with a written explanation.

2. Delamination Testing.

To test for delaminations, the research team used the most basic method of testing using a hammer and sounding for hollow spots. The presence of delaminations indicates that corrosion of the steel has progressed to the point where distress has occurred at the beam end. Prior training of the team provided fairly uniform documentation of this condition.

3.6.2 Field Experience

The research team took more time than estimated, because of the difficulty in accessing various portions of the bridge structure. The inspection at times could be very dangerous due to traffic and the use of tall ladders, thus making it essential to have technically and physically capable people performing the work.

In the field the research team faced the following difficulties:

- The beam was too high to access by conventional ladders.
- Fitting ladders into the narrow gap between piers and crash barriers.
- Safely placing ladders on the inclined embankments.
- The condition of embankments (deteriorated and/or covered by debris) made it difficult to climb or to place ladder safely.

In all these cases the team was trying to collect the desirable information in as safe a manner as possible.

The MDOT regional office helped the inspection of the bridge in Bay Region, Arenac County (Pontis bridge ID is 06106111000S110) by arranging a "Reach-All" and its operators. It would have been impossible to access the superstructure elements by ladders, because the superstructure was too high and the soil underneath the structure was too soft to safely set the ladders. The "Reach-All" allowed the team access from the bridge deck in order to inspect the fascia and beams.

3.7 Inspection Data Review

The following is the overview of findings from the inspection of each bridge presented in the sequence of inspection.

3.7.1 Grand Region

3.7.1.1 *NBI No: 41029 S23*

The bridge was inspected on July 12, 2001. The inspection took 8 hours and 30 minutes between 1:00 p.m. and 9:30 p.m.

General Bridge Information

The bridge is located in Kent County. Constructed in 1972 the bridge carries I-196 WB over 36th Street in Grand Rapids.

Bridge Geometry

The bridge is 116.6-ft long, 49.8-ft wide, and carries 4 lanes of traffic with 0.9-ft wide sidewalks. The bridge orientation is North-South. This is a skewed three span bridge with eight beams per span. The beam lengths are: 59.00-ft at the central span and 28.8-ft at the side spans. Side spans beams are types I and III, and center span beams are type III.

Abutments, Pier, Deck and Joints

The north abutment has several hairline cracks and minor rust and efflorescence stains. Some of the beam sole plates are corroded. South abutment, bearings and sole plates are in a good condition.

Piers have minor cracks and spalls, light rust, efflorescence and water stains.

Deck shoulders are covered with debris. Main span deck is in good condition. Side span decks have cracks and potholes with some rust and efflorescence stains. Expansion joints have leakage. Spalls are noted along construction joints. The construction joint filler is missing at the barriers. Drainage system is clean and operable. The general view of the deck is shown in Photo 3-1.

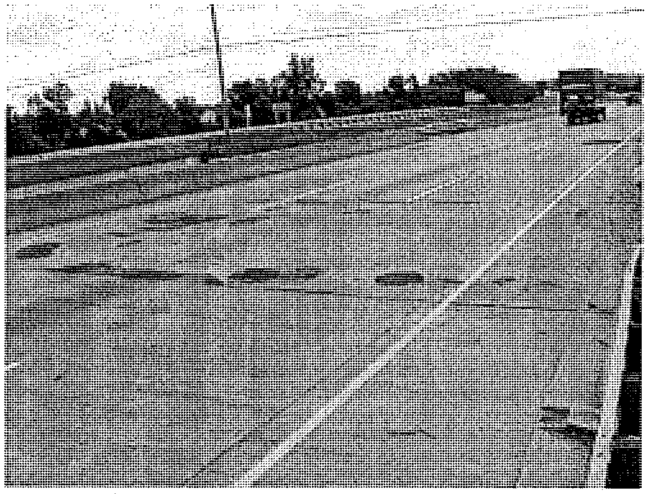

Photo 3-1. Deck Condition of 41029 S23

Stringers, Bearing and Diaphragms

Most beam-ends display cracking and minor spalling, rust and/or efflorescence stains. At some of the beam-ends shear reinforcement is exposed. Examples of the typical beam-end conditions are shown in Photo 3-2 and Photo 3-3:

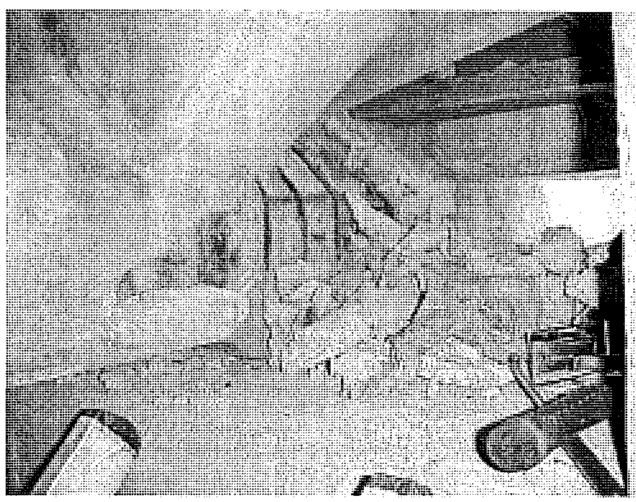

Photo 3-2. View of beam-end condition with exposed rebars

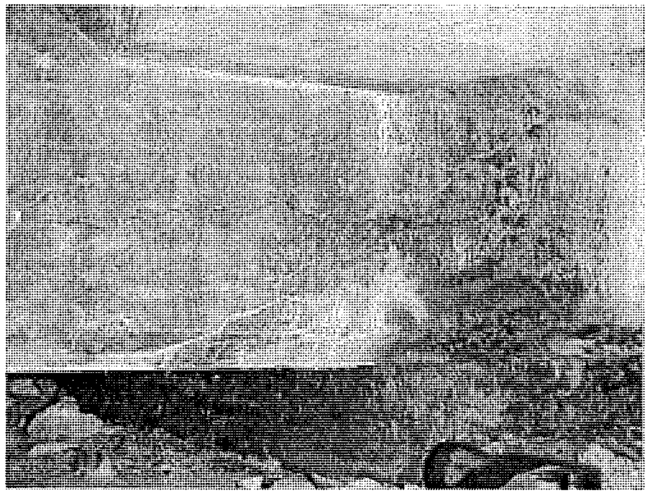

Photo 3-3. Close view of the beam-end condition - note cracking and efflorescence

Most of the sole plates on the piers are rusted. Several diaphragms on both piers are cracked and delaminated. One concrete diaphragm between beams W5 and W6 on pier S1 was partially spalled with exposed shear reinforcement (see Photo 3-4).

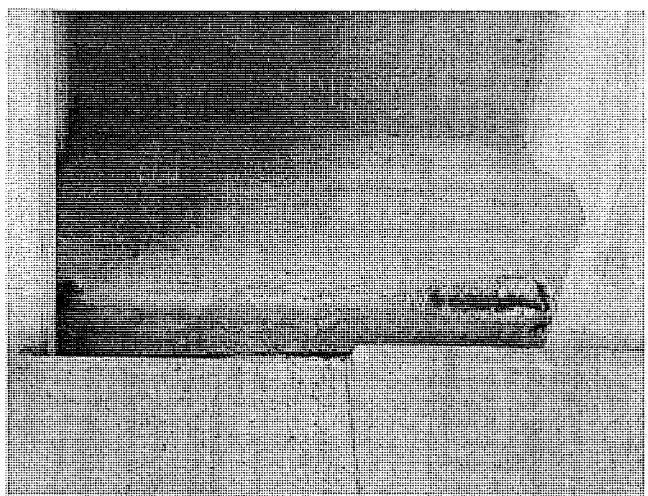

Photo 3-4. Diaphragm condition with exposed rebars

3.7.1.2 *NBI No: 41029 S163*

The bridge was inspected on July 13, 2001. The inspection took 6 hours between 7:00 a.m. and 1:00 p.m.

General Bridge Information

The bridge is located in Kent County. Constructed in 1964, the bridge carries I-196, M-21 EB over Lane Avenue in Grand Rapids.

Bridge Geometry

The bridge is 126-ft long, 45.6-ft wide and carries 2 lanes of traffic with 10.8-ft wide shoulders. The bridge orientation is North-South. This is a non-skewed three span bridge with eight beams per span. The beam lengths are: 63.00-ft at the central span and 31.5-ft at the side spans. Side span beam types are I and III, and center span beam type is II.

Abutments, Pier, Deck and Joints

Both abutments have a few vertical cracks. All piers have random cracks, spalls, delamination, and rust stains. Heavy rust stains and exposed reinforcement are observed on the east fascia of pier S1 and west fascia of pier S3. The severe pier and stringer conditions are shown in Photo 3-5.

Photo 3-5. Documenting severe deterioration of the pier and beam end

The deck has potholes with bituminous patches. The presence of construction joints packing and seals cannot be detected. Continuous cracking and some openings with patch (bituminous) along joints are noted. Barriers have some cracking, scaling and some rust staining.

Stringers, Bearing and Diaphragms

Most of the beam-ends are partially delaminated and spalled showing exposed shear reinforcement. The example of severe beam-end deterioration is shown on Photo 3-6:

Photo 3-6. Severe deterioration of beam-end

Most of the sole plates are rusted and the neoprene pads appeared to have lost flexibility.

The diaphragms on both piers are cracked, spalled, and delaminated. The concrete diaphragms on pier W1 between beams S4 and S5 and on pier W2 between beams S2 and S3 are heavily spalled with exposed shear reinforcement (see Photo 3-7).

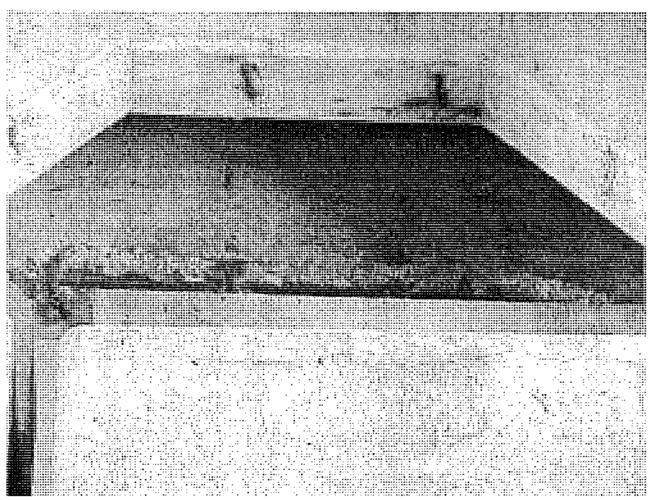

Photo 3-7. Diaphragm condition with some deterioration

3.7.1.3 *NBI No: 41029 S164*

The bridge was inspected on July 13, 2001. The inspection took 6 hours between 2:00 p.m. and 8:00 p.m.

General Bridge Information

The bridge is located in Kent County, constructed in 1964. The bridge carries I-196, M-21 WB over Lane Avenue in Grand Rapids.

Bridge Geometry

The bridge is 126-ft long, 45.6-ft wide and carries 2 lanes of traffic with 10.8-ft wide shoulders. The bridge orientation is West-East. This is a non-skewed three span bridge with eight beams per span. The beam lengths are: 63.00-ft at the central span and 31.5-ft at the side spans. Side spans beam types are I and III, and center span beam type is II.

Abutments, Pier, Deck and Joints

The abutments have a few vertical cracks and minor water and rust stains. Elastometric bearings are deformed. Piers have cracks, spalls, rust, and water stains. Shear reinforcement is exposed, and concrete has spalled at the bottom of the pier cap W1 (see Photo 3-8). Also, notice the difficulty of accessing the beam end behind the diaphragm for visual inspection.

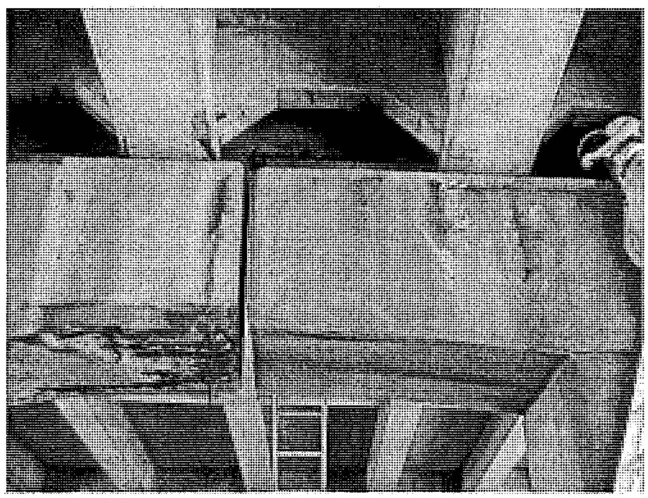

Photo 3-8. Exposed reinforcement and some spalls at piers, beam-end and diaphragms

The deck is overlaid with asphalt (see Photo 3-9)

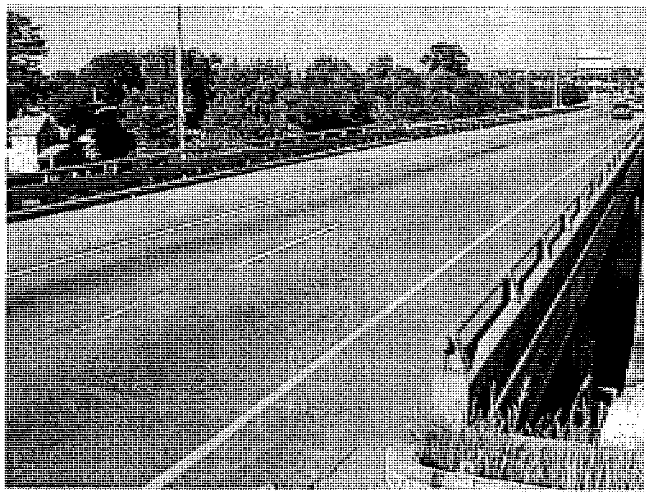

Photo 3-9. General view of the deck

Minor cracks and some spalls are noted along construction joints. Cracking, scaling and some rust stains are noted at the barriers.

Stringers, Bearing and Diaphragms

Most of the beam-ends display major cracking and spalls, rust and/or efflorescence stains, and exposed shear reinforcement. An example of the most severe deterioration of a beam-end and pier cap is shown in Photo 3-10:

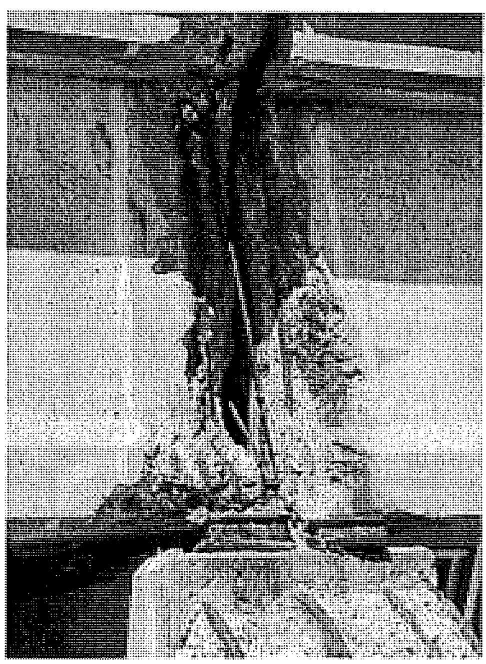

Photo 3-10. Beam-end and pier cap deterioration

Some of the sole plates are corroded and neoprene pads deformed with the ends curling. Some diaphragms have cracks, spalls and delaminations. The concrete diaphragm on pier W1 between beams S2 and S3 shows spalls with exposed shear reinforcement (see Photo 3-11).

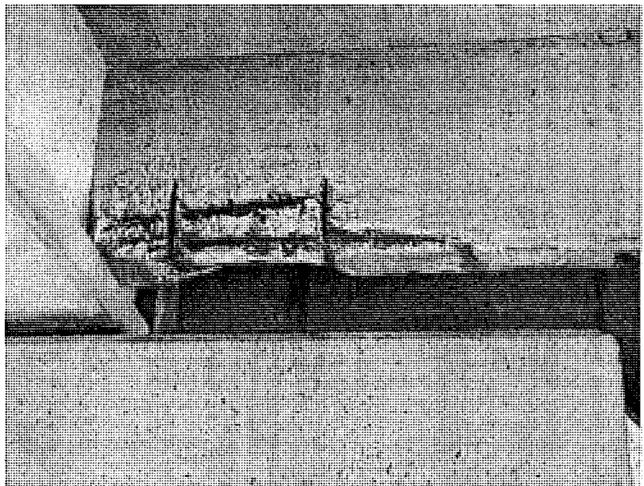

Photo 3-11. Spall with exposed reinforcement at diaphragm

3.7.1.4 *NBI No: 41027 S060*

The bridge was inspected on July 14, 2001. The inspection took 11 hours and 30 minutes between 7:00 a.m. and 6:30 p.m.

General Bridge Information

The bridge is located in Kent County. Constructed in 1963 the bridge carries US-131NB over 6th Avenue in Grand Rapids.

Bridge Geometry

The bridge is 138.7-ft long, 69.2 ft wide and carries 5 lanes of traffic with 4.6-ft wide sidewalks. The bridge orientation is North-South. This is a non-skewed three span bridge with twelve beams per span. The beam lengths are: 58.8-ft at the central span and 37.70-ft at the side spans. Side spans beam types are II, and center span beam type is III.

Abutments, Pier, Deck and Joints

The north abutment has vertical cracks through all abutment depth, spalls and rust stains. East side of the abutment has exposed reinforcement at wing wall. The south abutment has minor hairline vertical cracks.

Pier S1 has rust stain, delamination, spalls and rusty cracks. The north fascia of the pier is spalled with exposed reinforcement as shown in Photo 3-12.

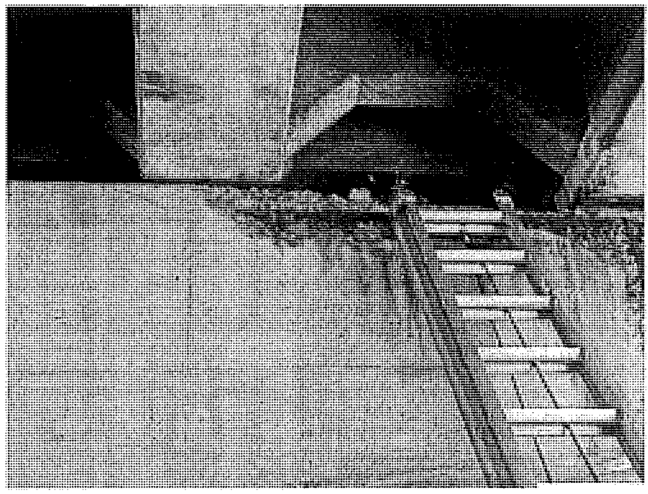

Photo 3-12. Spall at the North fascia of pier S1

Pier S2 has random cracks, spalls, rust and water stains, and delaminations. The south and north fascias of pier S2 have exposed shear reinforcement.

The deck has few patches, spalls, and a wide crack at the construction joint S1. Vertical cracks, spalls, efflorescence and rust stains are noted at both barriers.

Stringers, Bearing and Diaphragms

The beam-ends display cracks and spalls of various scales with rust and efflorescence stains.

Some of the concrete diaphragms on the South and North sides of pier S2 are cracked, delaminated and showed efflorescence and rusted stains. Concrete diaphragms on the North side of pier S1 are delaminated and exhibit efflorescence stains. The concrete diaphragms on the South side of pier S1 are in good condition.

Most of the sole plates are corroded and neoprene pads appear to have lost flexibility and deformed with ends curling.

3.7.1.5 *NBI No: 41025 S070*

The bridge was inspected on July 15, 2001. The inspection took 6 hours between 7:00 a.m. and 1:00 p.m.

General Bridge Information

The bridge is located in Kent County. Constructed in 1961 the bridge carries Knapp Street over I-96 in Grand Rapids.

Bridge Geometry

The bridge is 210.9-ft long, 32.8-ft wide and carries 2 lanes of traffic with 4.4-ft wide sidewalks. The bridge orientation is East-West. This is a non-skewed four span bridge with six beams per span. The beam lengths are 70.00-ft at the central spans and 35.00-ft at the side spans. The beam types of the two side spans are types II and III, and the beam type of the two center spans is type III.

Abutments, Pier, Deck and Joints

The west abutment has minor cracks, delamination and small areas of rust and water stains. The North side of West abutment is partially spalled with exposed reinforcement.

The east abutment has some vertical and horizontal cracks, delamination, and rust, water and efflorescence stains. The sole plates are corroded. The severe condition on the east abutment and delamination at the end of beam S3 are shown in Photo 3-13.

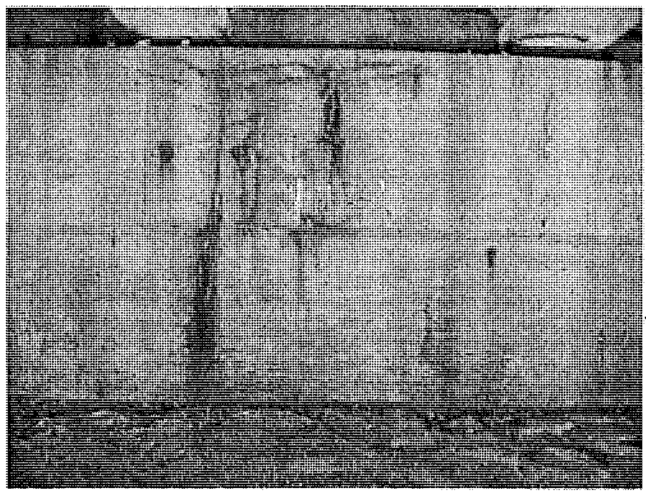

Photo 3-13. Condition of east abutment

Minor spalls, efflorescence, rust stains, and visible cracks are noted at West fascia of pier W1. The East fascia of pier cap of pier W1 is in good condition. Cracks, delamination, spalls, rust, and efflorescence stains are noted at the bottom of columns of pier W1.

Minor areas of spall, delamination, efflorescence and rust stains are noted at West fascia of the pier W2. Delamination, rust, water stains, and spalls are noted at the East fascia of pier W2. Efflorescence, rust, water stains, delaminations, spalls, and cracks are noted at both fascias of pier W3.

Cracks and potholes are noted between the deck and approaches. The deck shoulders are covered with sand and debris. Minor cracks and some spalls are noted along construction joints. Some joints are sealed. Multiple cracks, rust, and efflorescence stains are noticed at the barriers. The drainages are all clean and operable.

Stringers, Bearing and Diaphragms

Most of the beam-ends display cracks, spalls, water stains, rust and efflorescence. A few of the beams have exposed shear reinforcement. The sole plates are rusted and pads appear to have lost flexibility and are deformed with the ends curling.

The diaphragms at pier W1 are patched and have delamination, rust stains, and exposed reinforcement. The diaphragms at pier W2 show cracks, exposed reinforcement, and rust. The diaphragms at pier W3 are in good condition with minor areas of delamination at the bottom.

The example of the most severe deterioration of beam-end and diaphragm is shown in Photo 3-14.

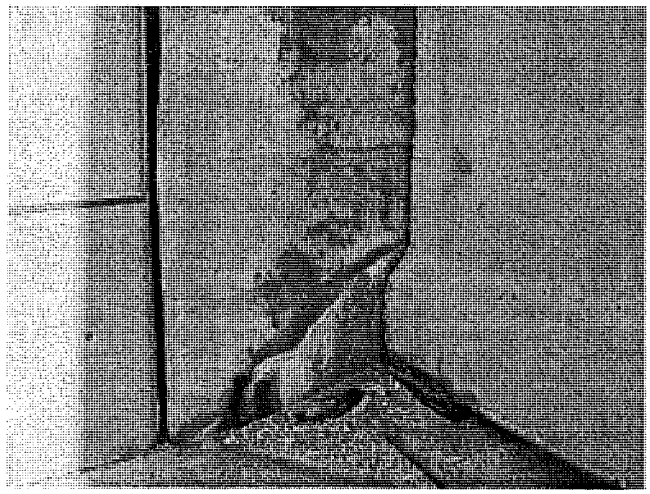

Photo 3-14. Beam-end and diaphragm condition

3.7.2 North Region

3.7.2.1 *NBI No: 67016 S090*

The bridge was inspected on July 18, 2001. The inspection took 3 hours between 9:00 a.m. and noon.

General Bridge Information

The bridge is located in Oceola County. It was constructed in 1984, and the bridge carries US-131 NB over US-10 in Reed City.

Bridge Geometry

The bridge is 111.0-ft long, 47.3-ft wide and carries 3 lanes of traffic with one 8-ft wide shoulder. The bridge orientation is North-South. This is a non-skewed one span bridge with six beams. The beam length is 111.00-ft and the beam type is Wisconsin 70" prestressed concrete I-beams.

Abutments, Deck and Joints

Both abutments have a few hairline vertical cracks through all retaining walls. Rust and efflorescence stains, spall and delamination noted. Efflorescence stain is noted at the bottom of the deck next to abutment as shown in Photo 3-15.

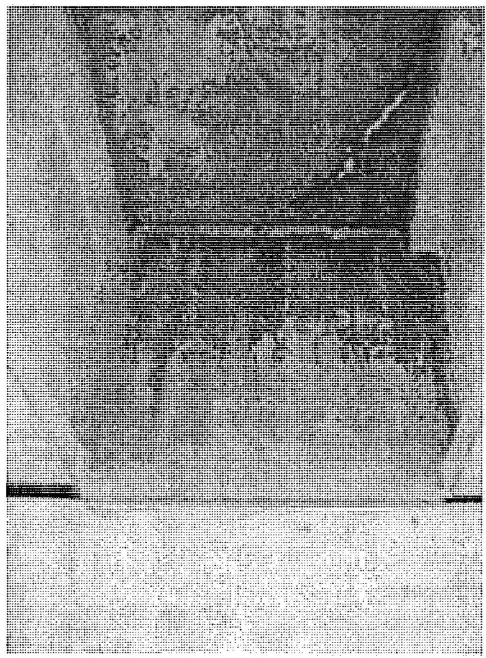

Photo 3-15. Rust and efflorescence stain on the deck

Next to the bearings there are water and rust stains. Some of the sole plates are partially rusted. Both embankments are in good condition. There is some vegetation on the slope.

The deck is new, and the drainage system is clean. Shoulders are covered with debris and some water stains are noted. The approaches are cracked and patched by asphalt. Minor cracks and some potholes are noted along construction joints.

Stringers and Diaphragms

Most of the beam-ends display minor cracking, spalls and rust stains. The common crack pattern is as shown in Photo 3-16.

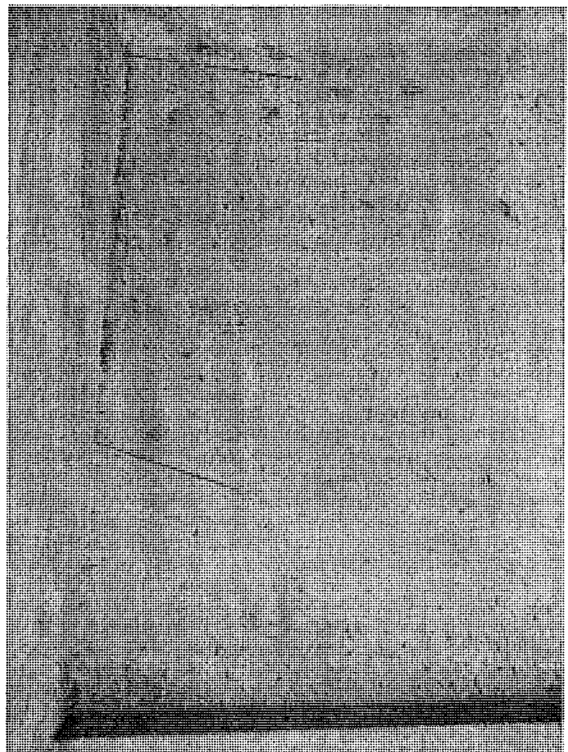

Photo 3-16. Beam-end common crack pattern (cracks outlined in blue)

The diaphragms located at the south abutment are in good condition. The team did not have equipment to check the diaphragms located at the middle of the span.

3.7.2.2 *NBI No: 67016 S100*

The bridge was inspected on July 18, 2001. The inspection took 3 hours and 30 minutes between 1:00 p.m. and 4:30 p.m.

General Bridge Information

The bridge is located in Oceola County. Constructed in 1984 the bridge carries US-131 SB over US-10 in Reed City.

Bridge Geometry

The bridge is 108-ft long, 53.1-ft wide and carries 3 lanes of traffic with 8-ft wide shoulders. The bridge orientation is North-South. This is a non-skewed one span bridge with seven beams. The beam length is 108.00-ft and the beam's type is Wisconsin 70" prestressed concrete I-beams.

Abutments, Deck and Joints

Both abutments have vertical cracks through all retaining walls, rust and water stains. Neoprene pads are in good condition. Sole plates have some rust. Both embankments are in good condition.

The deck is concrete. Expansion joints are full of sand and have rust stains and spall. Sand and water stains are noted at the shoulders. West concrete barrier has spall at South end of the deck. The approaches have some potholes and asphalt patches.

Stringers and Diaphragms

Most of the beam-ends display minor cracking, spalls, rust and water stains next to the bearings. The common crack pattern is as shown in Photo 3-17.

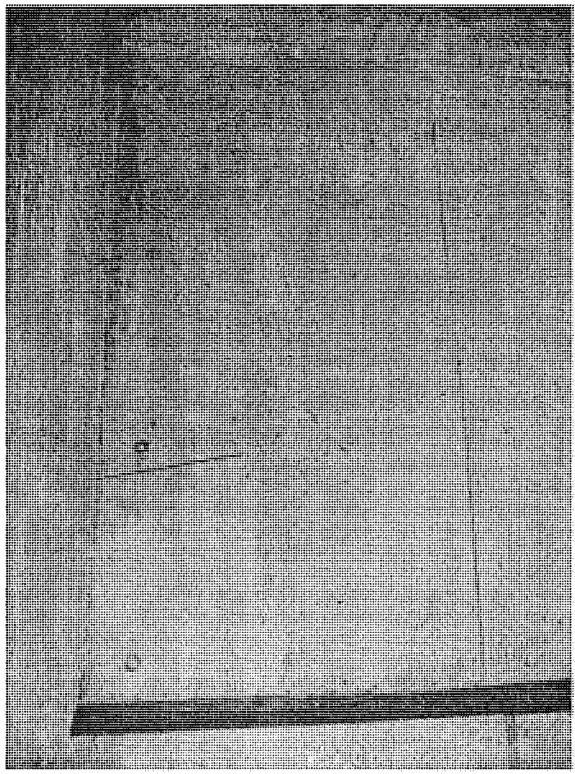

Photo 3-17. Beam-end common crack pattern (cracks outlined in blue)

The diaphragms located on the South abutment are in good condition. The team did not have equipment to check the diaphragms located at the middle of the span.

3.7.2.3 *NBI No: 53034 S050*

The bridge was inspected on July 19, 2001. The inspection took 7 hours between 7:00 a.m. and 2:00 p.m.

General Bridge Information

The bridge is located in Mason County. Constructed in 1986 the bridge carries Chauvez Road over US-31 in Ludington.

Bridge Geometry

The bridge is 305.77-ft long, 41.00-ft wide and carries 2 lanes of traffic with 7.4-ft wide shoulders. The bridge orientation is West-East. This is a skewed four span bridge with six beams per span.

The beam lengths are: 108.10-ft at two central spans W2 and W3, 37.70-ft at span W1 and 44.7-ft at span W4. The center spans beam type is Wisconsin 70" prestressed concrete I-beam. Side spans beams are 28" prestressed concrete I-beams for the interior beams and Wisconsin 70" prestressed concrete I-beams for fascia beams.

Abutments, Pier, Deck and Joints

The West abutment has a vertical hairline crack through all of the retaining wall. Both abutments have a few vertical cracks and minor rust, efflorescence and water stains. The sole plates have some rust.

Cracks, efflorescence, rust and water stains are noted at the piers caps. All pier caps are sealed with resin. The pier columns do not display any kind of deterioration. The common pier condition is shown in Photo 3-18.

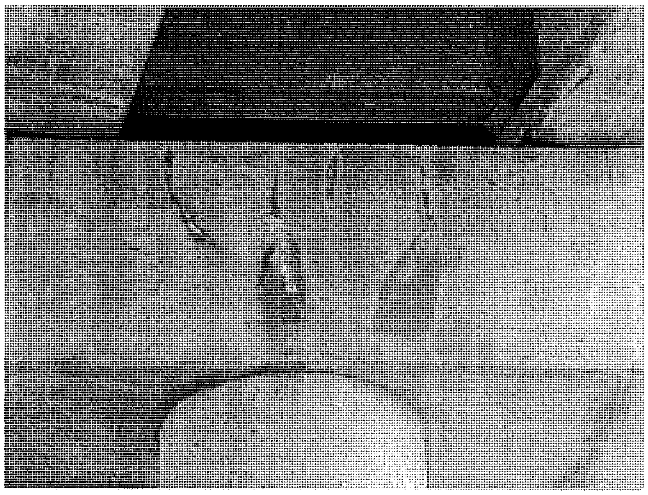

Photo 3-18. Pier condition with leaching from stress cracks

The bottom of the deck has some efflorescence staining mostly near the back wall as shown in Photo 3-19.

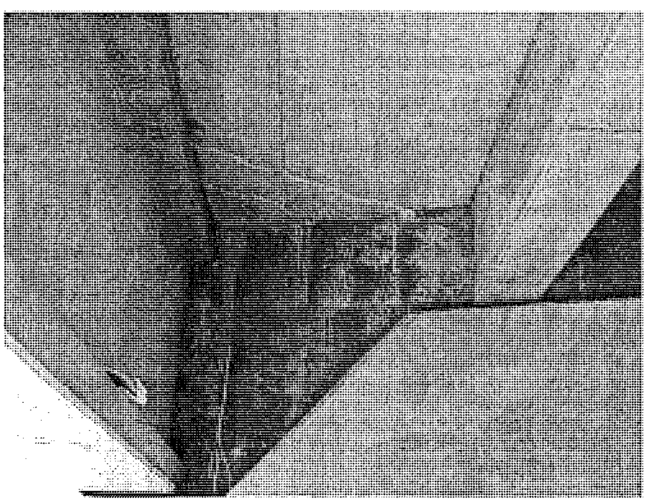

Photo 3-19. Efflorescence stain at bottom of the deck

The expansion joints are cracked and filled with sand. Sand is noted in the construction joints. The surface of deck has small potholes. The spalls are patched with epoxy. The concrete barrier has efflorescence stains and cracks.

Stringers, Bearings and Diaphragms

Most of the beam-ends display minor water and rust stains and micro and hairline cracks. The example of map cracking is shown in Photo 3-20.

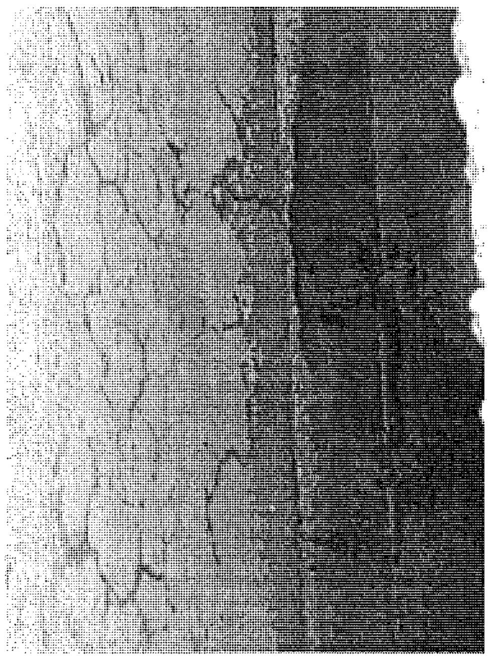

Photo 3-20. Example of map cracking at a beam-end

Some of the beam-ends are patched. Some of the sole plates are partially or completely rusted. Several neoprene pads are discolored. Hairline cracks are noted at some of the diaphragms, but most of them do not show any deterioration.

3.7.2.4 *NBI No: 83033 S060*

The bridge was inspected on July 19 and July 20, 2001. The inspection took 6 hours between 4:00 p.m. and 6:30 p.m. on July 19 and between 7:00 a.m. and 10:30 a.m. on July 20.

General Bridge Information

The bridge is located in Wexford County. Constructed in 1997 the bridge carries No. 36 Road over US-131 in Cadillac (Sec. 27 and 34 Haring TWP).

Bridge Geometry

The bridge is 146.00-ft long, 47.2-ft wide and carries 2 lanes of traffic with 10-ft wide shoulders. The bridge orientation is West-East. This is a non-skewed one span bridge with eight beams. The beam length is 142.6 ft and the beam type is Michigan 1800 beam.

Abutments, Deck and Joints

The West abutment has vertical cracks through the retaining wall. Spalls and delaminations are noted at wall top corners. The East abutment has vertical cracks through the entire retaining wall and light rust stains at both, North and South, sides.

The deck is new. A crack with efflorescence is noted at the bottom near the West abutment between beams S3 and S4. Sand and stones are accumulated on the shoulders and joints.

Stringers, Bearings and Diaphragms

Most of the beam-ends display hairline cracks. The examples of hairline cracks are shown in Photo 3-21 and Photo 3-22 (The crack is traced with purple chalk).

Photo 3-21. Example of hairline cracks

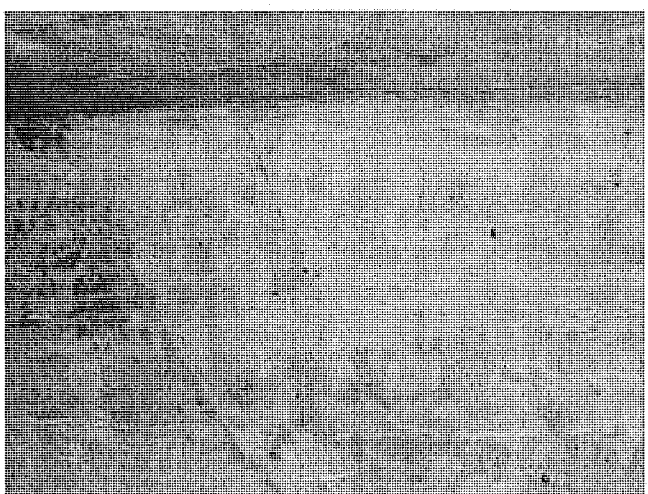

Photo 3-22. Example of hairline cracks

Some of the beam-ends are patched. Some of the sole plates are missing. Neoprene pads are deformed, dried out, or discolored.

3.7.2.5 *NBI No: 83033 S050*

The bridge was inspected on July 20, 2001. The inspection took 4 hours between 11:00 a.m. and 3:00 p.m.

General Bridge Information

The bridge is located in Wexford County. Constructed in 1998, the bridge carries Whaley Road over US-131 Reloc. in Cadillac.

Bridge Geometry

The bridge is 237.5-ft long, 47.2-ft wide and carries 2 lanes of traffic with 1.75-ft wide sidewalk. The bridge orientation is West-East. This is a skewed two span bridge with four beams per span. The beam length is 124.8-ft for the first span and 112.9-ft for second span. The beam type is Wisconsin 70" prestressed concrete I-beam.

Abutments, Pier, Deck and Joints

The West abutment has a few vertical hairline cracks through the retaining wall. The East abutment has few vertical and diagonal hairline cracks. On the back walls of both abutments vertical cracks, efflorescence, and rust stains are noted.

Water stain and few patches are noted on both the West and East fascias at the caps of the middle pier. Several pier columns have water stain (see Photo 3-23).

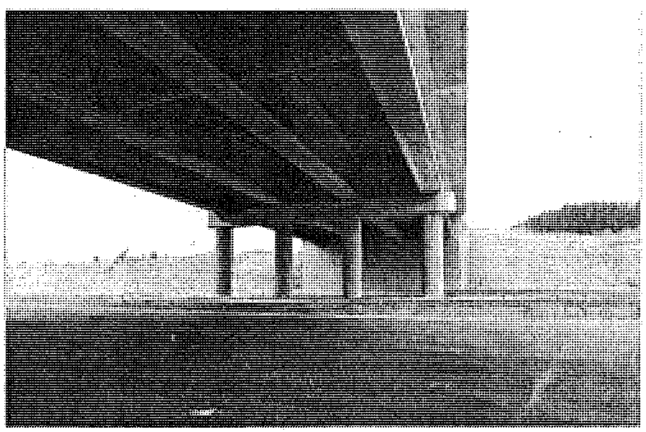

Photo 3-23. Water stain on the piers and caps

The bottom of the deck has efflorescence stain at the South corner of the West abutment and the North corner of the East abutment (see Photo 3-24).

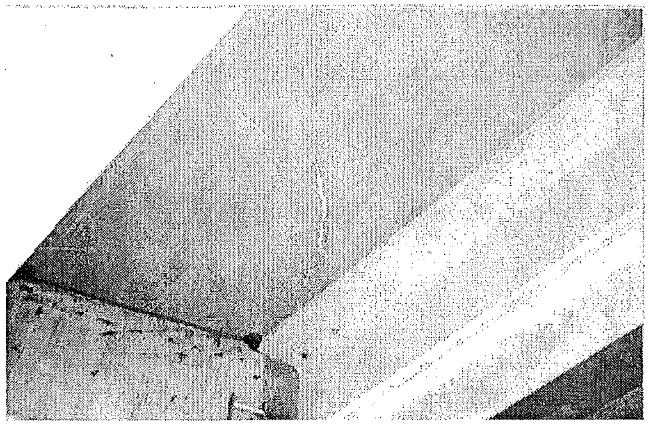

Photo 3-24. Efflorescence stain at South corner of West abutment and deck

The construction joints are cracked and filled with sand. An example of a control joint with hot-pour joint sealant is seen in Photo 3-25.

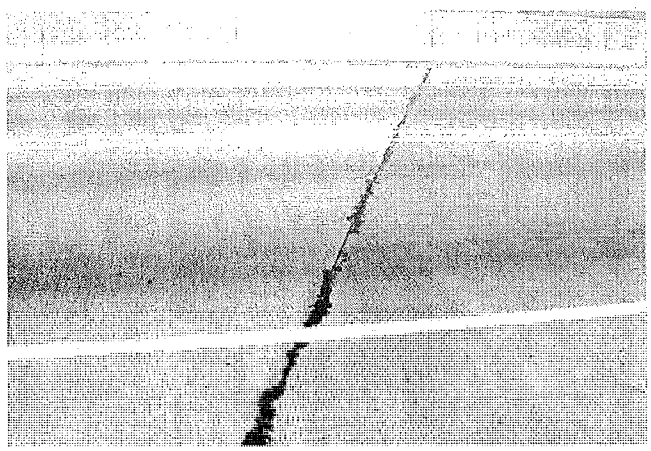

Photo 3-25. Condition of Deck Control joint

The sidewalks are covered with sand and gravel. The concrete barriers have a few cracks, rust, and efflorescence stains.

Stringers, Bearings and Diaphragms

Most of the beam-ends display microscopic and hairline cracks as shown in Photo 3-26.

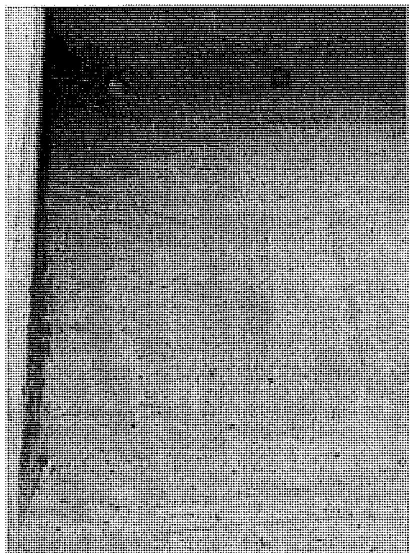
Photo 3-26. Hairline cracks at beam-end

Some of the beam-ends have incipient spalls as shown in Photo 3-27.

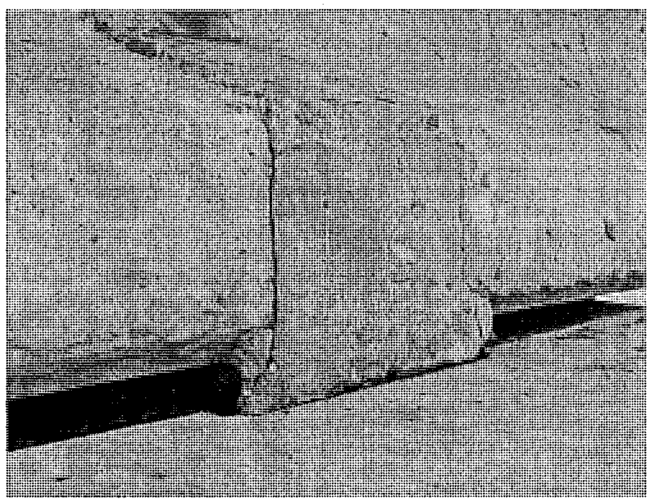
Photo 3-27. Beam-end restraint detail

Elastomeric bearings do not show any kind of deterioration. Few diaphragms at the middle pier have minor efflorescence and water stains. One diaphragm between beams S3 and S4 is patched. Diaphragm on the East abutment, North corner has crack and efflorescence stain.

3.7.3 Bay Region

3.7.3.1 *NBI No: 25042 S124*

The bridge was inspected on July 23 and July 24, 2001. The inspection took 5 hours between 7:30 a.m. and 10:00 a.m. on July 23 and July 24.

General Bridge Information

The bridge is located in Genesee County. Constructed in 1969 the bridge carries I-69 WB over I-75.

Bridge Geometry

The bridge is 210.0-ft long, 43.0-ft wide and carries 2 lanes of traffic with 18.0-ft wide shoulders and curbs. The bridge orientation is West-East. This is a skewed four span bridge with seven beams per the two center spans and four beams per the two side spans. The beam lengths are: 65.5-ft for two central spans W2 and W3, 38.5-ft for span W1 and 34.4-ft for span W4. The side and center spans beam type is III.

Abutments, Pier and Deck

The West abutment has several vertical cracks with water and rust stains (see Photo 3-28).

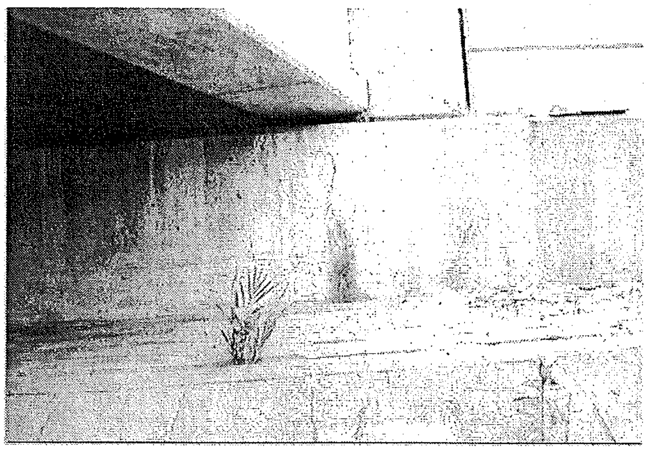

Photo 3-28. Vertical crack with moisture and rust stain at West abutment

The East abutment has a few vertical cracks, rust and water satins, and efflorescence stain at the North side of the abutment.

The pier caps have rust and water stain, and delamination. An efflorescence stain is noted at pier W3. The bottom of the cap of pier W2 is spalled and showing exposed reinforcement.

An efflorescence stain is noted at the bottom of the deck next to the East abutment. The surface of the deck was not inspected due to the time limit.

Stringers, Bearing and Diaphragms

Most of the beam-ends are delaminated, cracked, spalled and some exhibit exposed shear reinforcement as shown in Photo 3-29 and Photo 3-30 (at pier W1, beam S1; at pier W2, beams S1, S2, S4 and S6; at pier W3, beams S2 and S6).

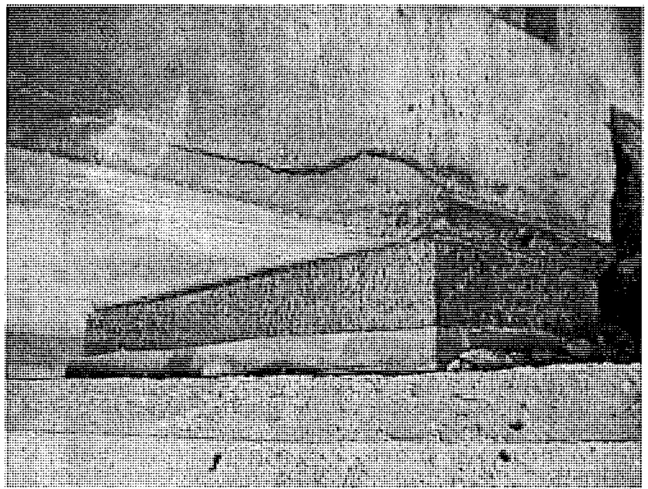

Photo 3-29. Delamination at the beam-end and bearing with sole plate

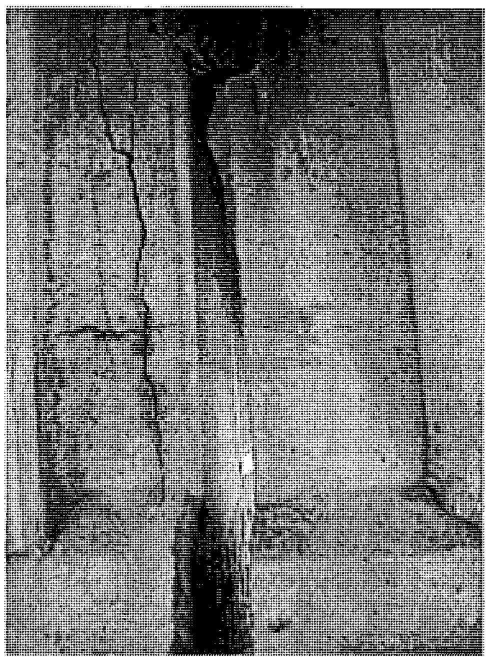

Photo 3-30. Beam-end cracking

Efflorescence stains are noted at some beam-ends. The beam-ends at the abutments have hairline cracks with some rust and water stain. Bearing plates are rusted and pads appear to have lost flexibility and deformed with ends curling.

Several diaphragms are cracked and delaminated. The diaphragms at pier W3 between beams S4 and S5 and pier W1 between beams S2 and S3 are spalled showing exposed shear reinforcement.

3.7.3.2 *NBI No: 25042 S123*

The bridge was inspected on July 23 and July 24, 2001. The inspection took 5 hours between 10:00 a.m. and 12:30 p.m. on July 23 and July 24.

General Bridge Information

The bridge is located in Genesee County. Constructed in 1969 the bridge carries I-69 EB over I-75.

Bridge Geometry

The bridge is 210.0-ft long, 43.0-ft wide and carries 2 lanes of traffic with 18.0-ft wide shoulders and curbs. The bridge orientation is West-East. This is a skewed four span bridge with seven beams per the two center spans and four beams per the two side spans. The beam lengths are: 65.5-ft for the two central spans W2 and W 3, 38.5-ft for span W1 and 34.4-ft for span W4. The side and center spans beam type is III.

Abutments, Pier and Deck

The east abutment has three vertical cracks through the entire abutment. Rust stain noted at one of the cracks located at the center of the abutment. A water stain is noted next to beam S1, and an efflorescence stain is noted next to beam S2. The West abutment has several vertical cracks with water and rust stains.

All of the pier caps have rust and water stains. Column S3 of pier W3 has multiple cracks. The cap of pier W1, N-end, has delamination and spall, and exposed reinforcement at the bottom. Pier W2, S-end, E-fascia has delamination.

An efflorescence stain is noted at the North corner at the bottom of the deck next to East abutment. The surface of the deck was not inspected due to the time limit.

Stringers, Bearing and Diaphragms

The beam-ends are cracked, delaminated, and spalled. Also, there is evidence of rust, water and efflorescence stains. The beam-ends of eight beams out of 22 are spalled with exposed shear reinforcement evident. The beam-ends at the abutment have hairline cracks with some rust and water stain. The sole plates are rusted and the pads appear to have lost flexibility and are deformed. The diaphragms between beams S5, S6 and S7 at pier W1 are delaminated. The diaphragm between beams S1 and S2 at pier W3 is delaminated at the bottom and partially spalled with exposed reinforcement. The condition at the beam-ends at the pier caps is shown in Photo 3-31.

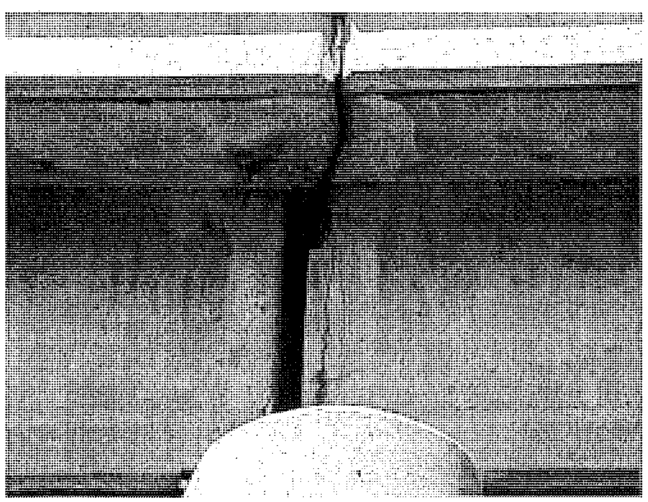

Photo 3-31. Condition at the beam-ends at the pier

3.7.3.3 *NBI No: 25042 S128*

The bridge was inspected on July 23 and July 24, 2001. The inspection took 4 hours between 12:30 p.m. and 2:30 p.m. on July 23 and July 24.

General Bridge Information

The bridge is located in Genesee County. Constructed in 1967 the bridge carries I-69 Ramp F over I-75.

Bridge Geometry

The bridge is 210.0-ft long, 27.0-ft wide and carries 1 lane of traffic with curbs. The bridge orientation is West-East. This is a skewed four span bridge with five beams per the two center spans and three beams per the two side spans. The beam lengths are: 65.5-ft for two central spans W2 and W3, 38.5-ft for span W1 and 34.4 ft for span W4. The side and center spans beam type is III.

Abutments, Pier and Deck

The East abutment has one vertical crack and a water stain all over the abutment. The West abutment has wide vertical cracks thought the entire wall at the South and North sides of the abutment. Several vertical cracks, water stain, and delamination are noted between beams S1 and S3.

All of the pier caps have water and rust stains. Delamination, cracks, and rust stains are noted at the fascia and bottom of all pier caps (see Photo 3-32).

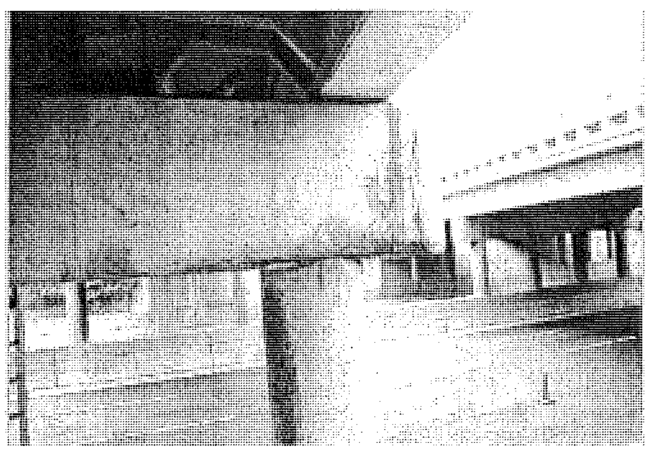

Photo 3-32. Delamination, cracks and rust stain at pier and pier cap

The bottom of pier W1 is spalled showing exposed reinforcement (see Photo 3-33).

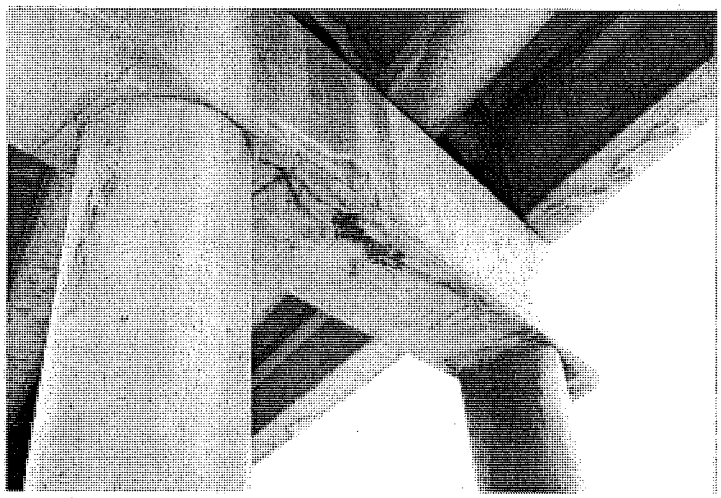

Photo 3-33. Exposed reinforcement at the bottom of the pier W1

No deterioration noted at the deck bottom. The surface of the deck was not inspected due to the time limit.

Stringers, Bearing and Diaphragms

All of the beam-ends are cracked. Many of the beam-ends have moisture leaching from the cracks. Some of the beam-ends are delaminated or spalled showing exposed shear reinforcement as shown in Photo 3-34.

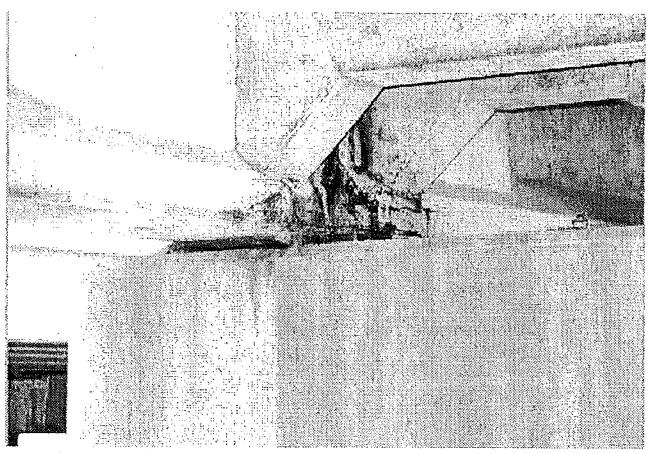

Photo 3-34. Deterioration at the beam-ends in between diaphragms

Some hairline cracks and small areas of water stain are noted at the beam-ends at the abutments. The sole plates are corroded, and the elastomeric pads are deformed. The diaphragms at pier W1 between beams S1 and S2 and S4 and S5 are cracked and delaminated and exhibit minor rust and efflorescence stains.

3.7.3.4 *NBI No: 25042 S127*

The bridge was inspected on July 23 and July 24, 2001. The inspection took 4 hours between 2:30 p.m. and 4:30 p.m. on July 23 and July 24.

General Bridge Information.

The bridge is located in Genesee County. Constructed in 1969 the bridge carries I-69 Ramp E over I-75.

Bridge Geometry.

The bridge is 210.0-ft long, 27.0-ft wide and carries 1 lane of traffic with curbs. The bridge orientation is West-East. This is a skewed four span bridge with five beams per the two center spans and three beams per the two side spans. The beam lengths are: 65.5-ft for two central spans W2 and W3, 38.5-ft for span W1, and 34.4-ft for span W4. The side and center spans beam type is III.

Abutments, Pier and Deck

The East abutment has vertical cracks through the abutment. The West abutment has vertical cracks thought the entire wall and rust and water stain.

Pier W1 has minor rust and water stain, spalls and cracks at both fascias (see Photo 3-35).

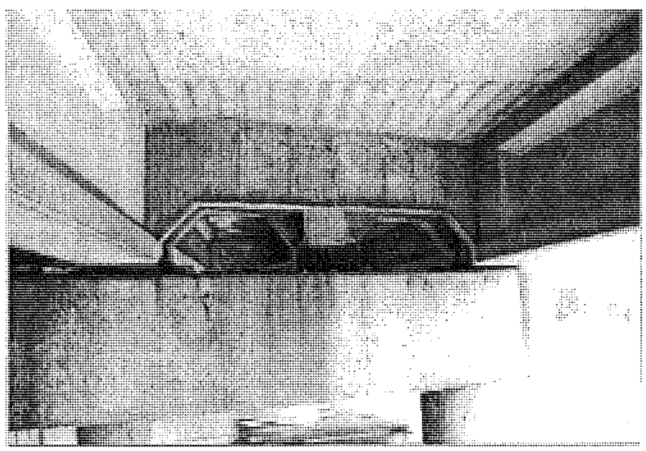

Photo 3-35. Rust and water stain, spalls and cracks at Pier W1

The East fascia of pier W1 has areas of delamination, cracks, and efflorescence stains. Piers W2 and W3 have water and rust stains. Pier W3 displays crack and delamination at the bottom of the pier cap (see Photo 3-36).

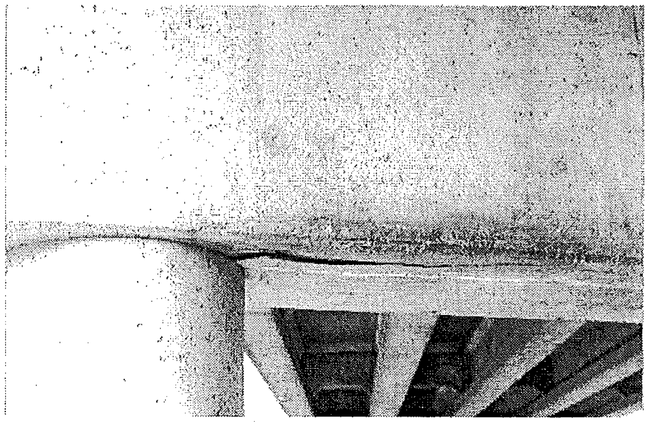

Photo 3-36. Crack and delamination at the bottom of the pier W3

Cracks and efflorescence stains are noted between piers W3 and W2 at the deck bottom. The surface of the deck was not inspected due to the time limit.

Stringers, Bearing and Diaphragms

The beam-ends at the piers are cracked, delaminated and spalled. Some of the beams have rust and efflorescence stain. Eleven beams out of twenty are spalled with exposed reinforcement. The beam-ends at the abutments have hairline cracks, water, and rust stains. The typical beam-end condition is shown in Photo 3-37.

Photo 3-37. Beam-end exhibiting significant deterioration

The exterior diaphragms at pier W1 are delaminated and display rust stains. The diaphragm between beams S2 and S3 has a diagonal crack. One exterior and two interior diaphragms at pier W3 are delaminated. The diaphragm between beams S2 and S3 exhibits spalling. The diaphragm between beams S1 and S2 has a large horizontal crack, rust, and efflorescence stain.

The sole plates are rusted, and the neoprene pads are deformed and discolored or dried out.

3.7.3.5 *NBI No: 06111 S11*

The bridge was inspected on July 25, 2001. The inspection took 11 hours and 30 minutes between 7:30 a.m. and 8:00 p.m.

General Bridge Information

The bridge is located in Arenac County. Constructed in 1968 the bridge carries M-33 over I-75 at 1.0 MI N of Alger.

Bridge Geometry

The bridge is 380.8-ft long, 48.2-ft wide and carries 4 lanes of traffic with narrow sidewalks. The bridge orientation is West-East. This is a skewed six span bridge with nine beams per span. The beam lengths are: 73.3-ft for the four central spans W2, W3, W4 and W 5, 37.0-ft for span W1 and 50.6-ft for span W6. The center spans beam type is III. Side span W1 beam types are II for fascia and I for interior. Side span W6 beam types are II for fascia and III for interior.

Abutments, Pier, Deck and Joints

The west abutment has several narrow cracks, light water stain, and minor area of spall and delamination. The East abutment has few narrow cracks at the North side and wide crack at the South side. Water and rust stains are noted along both abutments.

All pier caps have delamination, rust, and water stain. The worst condition at the piers is shown in Photo 3-38.

Photo 3-38. Condition of pier and pier cap

The corners of the pier caps at piers W4 and W5 are spalled and showing exposed reinforcement and efflorescence stains (see Photo 3-32). Some of the piers are cracked and patched. Most of the pier columns have water and rust stains. Column S4 of pier W1 has several wide vertical cracks thought all column height (see Photo 3-39).

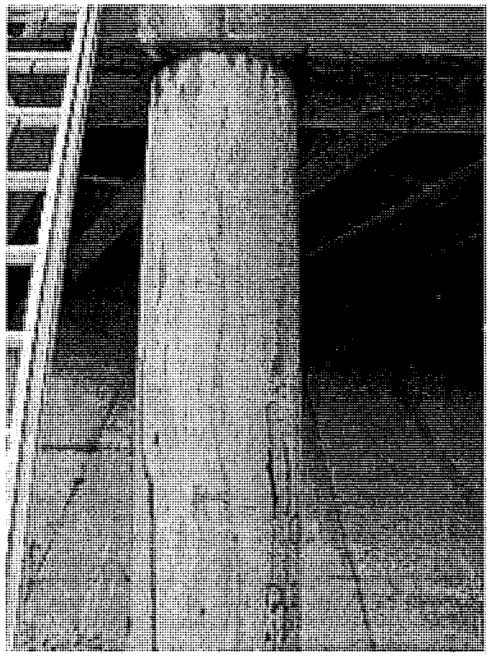

Photo 3-39. Vertical cracks at column S4 of the pier W1

The surface of the concrete deck has many areas with cracks and efflorescence. The bottom of the deck is cracked with efflorescence and water stains next to both abutments. The deck sidewalks are covered with sand and gravel. Delamination, cracks, efflorescence, and rust are noted on the barriers. Some joints are filled with the sand. Several spalls and patches are noted along the joints. One joint has a spall with exposed reinforcement visible as shown in Photo 3-40.

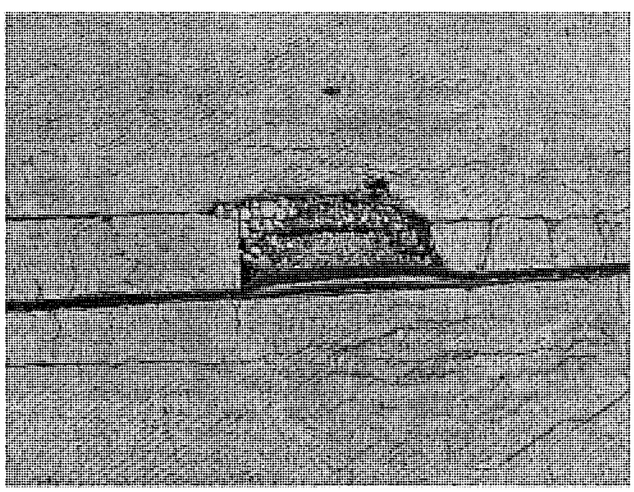

Photo 3-40. Spall with exposed reinforcement at the deck joint

Stringers, Bearing and Diaphragms

Vertical and horizontal cracks of various widths, from hairline to wide, are noted at the beam-ends (see Photo 3-41). Also, some beam-ends had been patched, but they were noted to have delaminations.

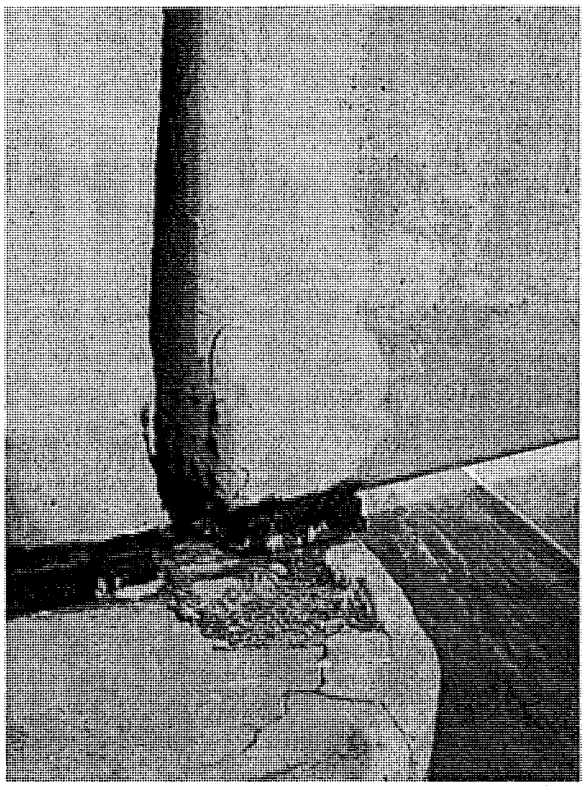

Photo 3-41. Cracks at the beam-end and pier cap

Some of the beam-ends have rust, water, and efflorescence stains and exhibit exposed shear reinforcement (see Photo 3-42). Sole plates are corroded. The diaphragms at the abutments are cracked. Most of the diaphragms at the piers are cracked and delaminated at their bottom and exhibit some rust stains and exposed reinforcement.

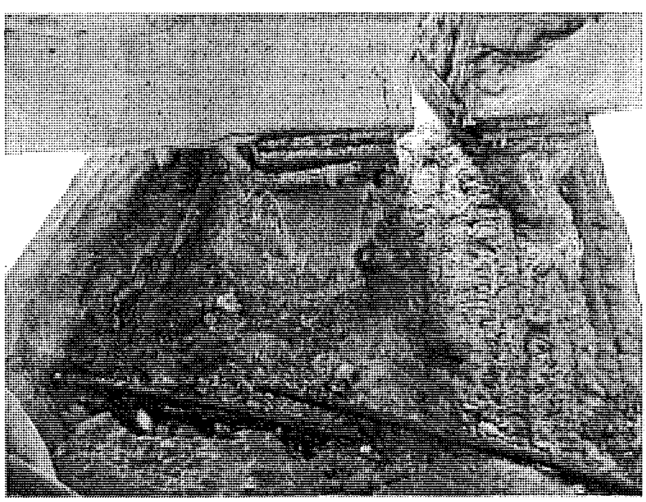

Photo 3-42. Delaminated beam-end and pier

3.7.3.6 *NBI No: 25132 S34*

The bridge was inspected on August 27, 2001. The inspection took 6 hours between 8:00 a.m. and 2:00 p.m.

General Bridge Information

The bridge is located in Genesee County. Constructed in 1971 the bridge carries I-475 over Clio Road at 1.5 MI E of N JCT I-75.

Bridge Geometry.

The bridge is 175.20-ft long, 51.51-ft wide and carries 2 lanes of traffic with variable shoulders width. The bridge orientation is West-East. This is a skewed four span bridge with six beams per span. The beam lengths are: 58.6-ft for the two central spans W2 and W3 and 29.0-ft for the two side spans W1 and W4. The two center spans beam type is III. Side span W1 and W4 beam types are II for fascia and I for interior.

Abutments, Pier, Deck and Joints.

Both abutments have several cracks through the entire wall. The West and East abutments have rust and water stains at the North and South corners. The North corner of East abutment spalled showing exposed reinforcement as shown on Photo 3-43.

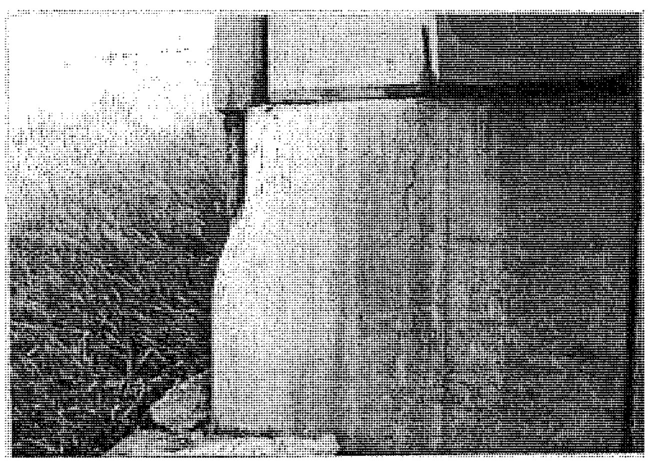

Photo 3-43. Spall at North corner of East abutment

The sole plates are partially rusted, and the neoprene pads are deformed, sometimes broken or displaced.

All piers show rust and water stains. Few cracks and some areas of delamination are noticed at the center pier W2.

The deck is under replacement. New concrete overlay is being placed on the East approach and main deck. Concrete on the West approach is removed up to the reinforcement. The shoulders are cracked. Cracks, some areas of water and efflorescence stains are noticed at the deck bottom. The concrete barriers are cracked at several locations. The hole through the entire width of the deck depth is noticed at the West approach (see Photo 3-44).

Photo 3-44. Hole at the deck West approach

Stringers, Bearing and Diaphragms

Major spall occurred at one of the beam-ends (Beam S5, Pier W3) and minor spalls are noticed at several others. Vertical and horizontal cracks are noted at the ends of beams. The typical beam-end condition is shown on Photo 3-45.

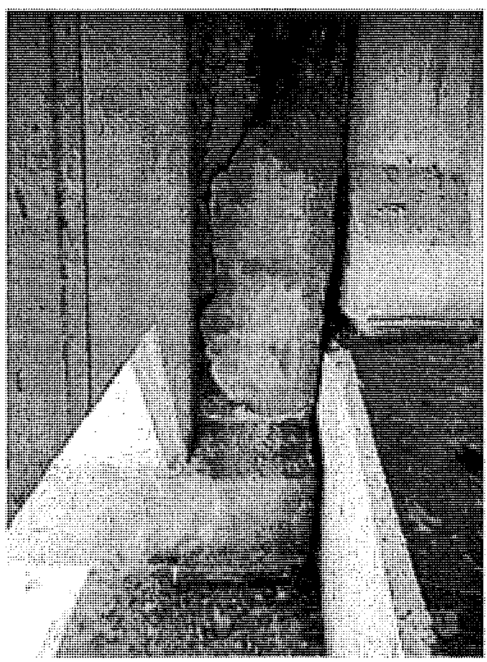

Photo 3-45. Typical beam-end condition

Spalls occurred at most of the beam-ends next to the bearings. The sole plates are rusted, and the neoprene pads are deformed and sometimes broken. An example of the beam-end condition is shown in Photo 3-46.

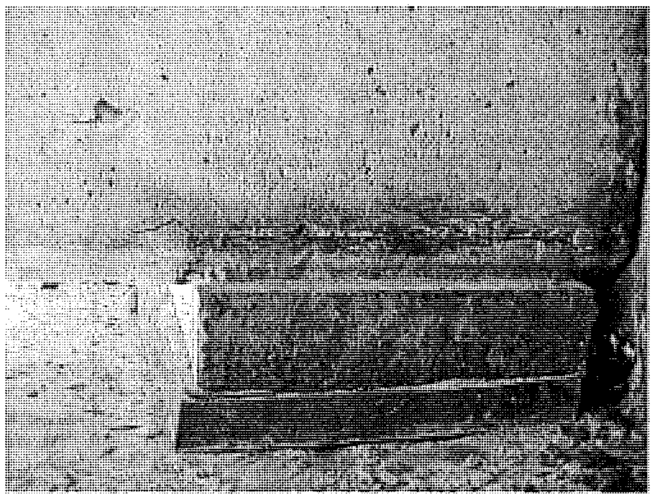

Photo 3-46. Rust crack at beam-end

The diaphragms at the abutments do not exhibit any kind of deterioration. Some of the diaphragms at the piers have cracks and small areas of delamination.

3.7.3.7 *NBI No: 29011 S03*

The bridge was inspected on August 27 and August 28, 2001. The inspection took 4 hours and 30 minutes between 4:00 p.m. and 6:30 p.m. on August 27 and between 7:00 a.m. and 9:00 a.m. on August 28.

General Bridge Information

The bridge is located in Gratiot County. Constructed in 1961 the bridge carries US-27 NB over US-27 BR (Polk Road) 2.0 MI N of Ithaca.

Bridge Geometry

The bridge is 113.9-ft long, 47.7 ft-wide and carries 2 lanes of traffic with 2.7-ft wide shoulders. The bridge orientation is North-South. This is a non-skewed three span bridge with nine beams per span. The beam lengths are: 43.9-ft for central span S2 and 35.0-ft for the two side spans S1 and S3. The center span beam type is II. The side spans S1 and S3 beam types are II for fascia and I for interior.

Abutments, Pier, Deck and Joints

The South abutment has horizontal and vertical cracks below beams W5 and W4, efflorescence, water and rust stains (see Photo 3-47).

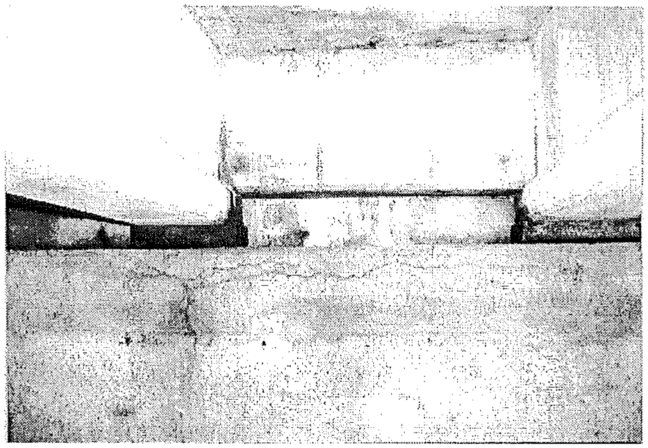

Photo 3-47. Condition of South abutment

The North abutment has water stains, heavy rust stains and areas of delamination and spall (see Photo 3-48).

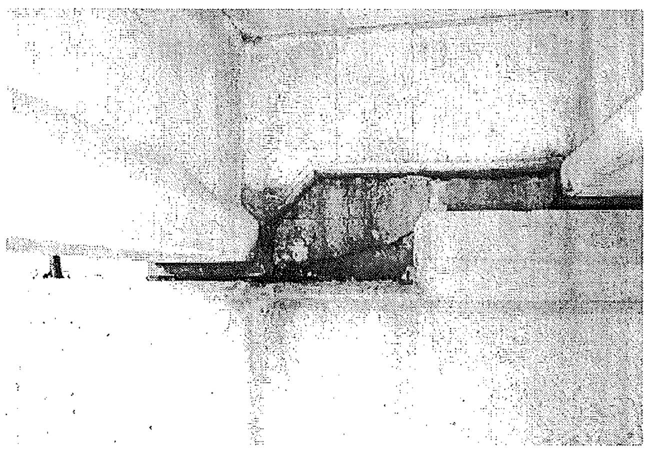

Photo 3-48. Condition of North abutment and diaphragm

Several vertical cracks are noticed thought out entire wall. Very wide diagonal crack noted at West side of the abutment. The sole plates are rusted. The elastomeric pads are in good condition. The pier caps have rust and water stains. Few cracks and some areas of delamination are noticed at pier W2 (see Photo 3-49).

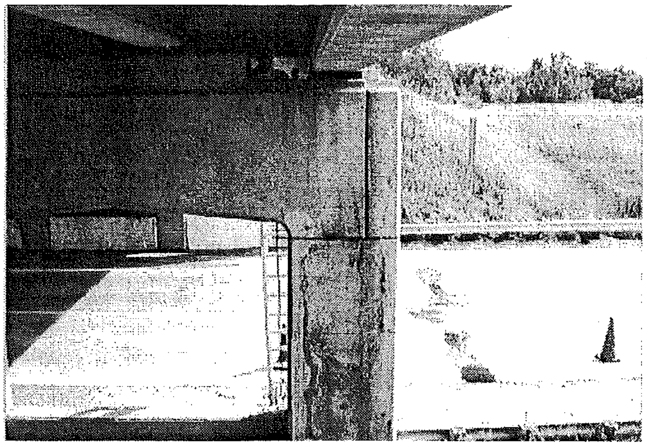
Photo 3-49. Deterioration observed at pier W2

Most of the columns are patched. Column W9, at pier S2 has cracks and water stains.

Several efflorescence and water stains are noted at the bottom of the deck. The concrete barriers have some cracks. The drains are clean and operable. Gravel and stone debris have accumulated on the shoulders. The joints are covered with an asphalt overlay. The overlay of the main deck and approach is new. The main deck has a new asphalt patch.

Stringers, Bearing and Diaphragms

Most of the beam-ends display cracks, some rusted, spalled and delaminated areas, and efflorescence and water stains (see Photo 3-50). The sole plates are discolored and deformed with ends curling. Some of the diaphragms at the piers are cracked and show rust and efflorescence stains. The diaphragms at the abutments have horizontal narrow cracks and small areas of rust and efflorescence stains.

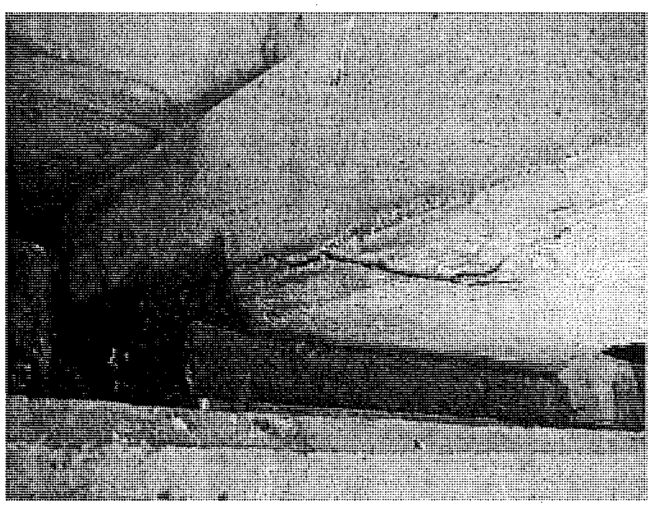
Photo 3-50. Rust and spall at beam-end

3.7.3.8 *NBI No: 06111 S04*

The bridge was inspected on August 28, 2001. The inspection took 5 hours between 10:30 a.m. and 3:30 p.m.

General Bridge Information

The bridge is located in Arenac County. Constructed in 1968 the bridge carries I-75 NB over M-61 at 4.0 MI W of Standish.

Bridge Geometry

The bridge is 111.8-ft long, 42.9-ft wide and carries 2 lanes of traffic with 9.45-ft wide shoulders. The bridge orientation is South-North. This is a non-skewed three span bridge with six beams per span. The beam lengths are: 49.0-ft at the central span S2 and 31.3-ft at the two side spans S1 and S3. The side spans beam types are III for fascia and I for interior, and center spans beam type is III.

Abutments, Pier, Deck and Joints

Both abutments have vertical crack through the center of the abutment and wet cracks with efflorescence stain at East and West corners. The North abutment has rust and water stains. The South fascia of pier S1 has heavy rust stains under beam W1. The North fascia of pier S1 has rust and water stains, delamination with spalls and exposed shear reinforcement. The worst condition observed was under beam W1 as shown in Photo 3-51.

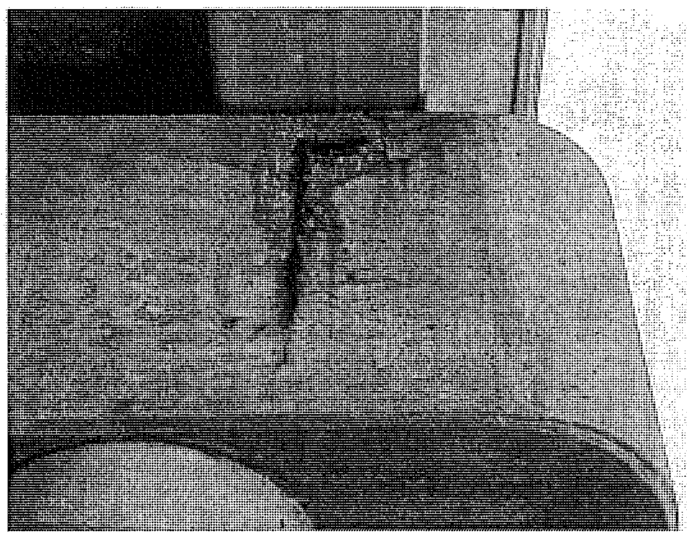

Photo 3-51. Spall with exposed reinforcement at pier S1 under beam W1

Both fascias of pier S2 have rust and water stain (see Photo 3-52).

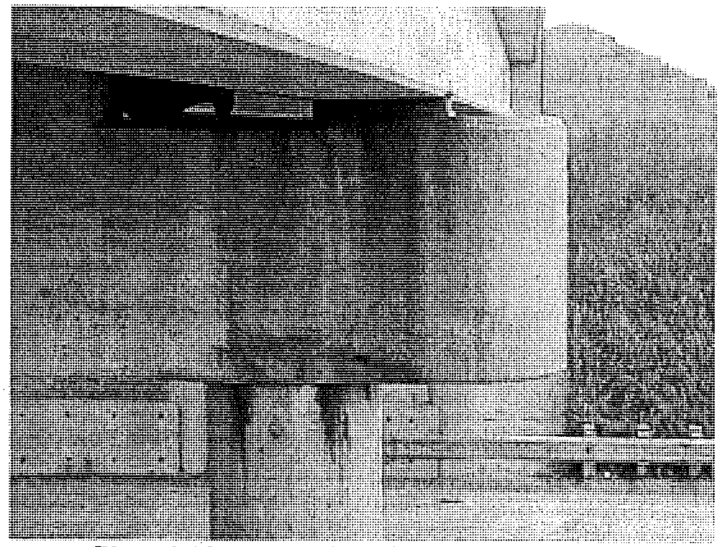
Photo 3-52. Rust stain and wet spots at pier S2

Efflorescence stains and a horizontal crack were noted under beam W6 at the pier cap, on the North fascia.

The deck has a new concrete overlay. Spalls at approaches and along joints were noted. At the South approach rusted cracks were noted. The joints are sealed with asphalt and are partially broken. The shoulders are patched and covered with sand, gravel and debris. The drainage system is clean and operable. The bottom of the deck displays multiple cracks and rust stains.

Stringers, Bearing and Diaphragms

The beam-ends at the piers display cracks, minor rust stain, and spall next to the bearings. Most of the sole plates are rusted and pads are deformed (see Photo 3-53).

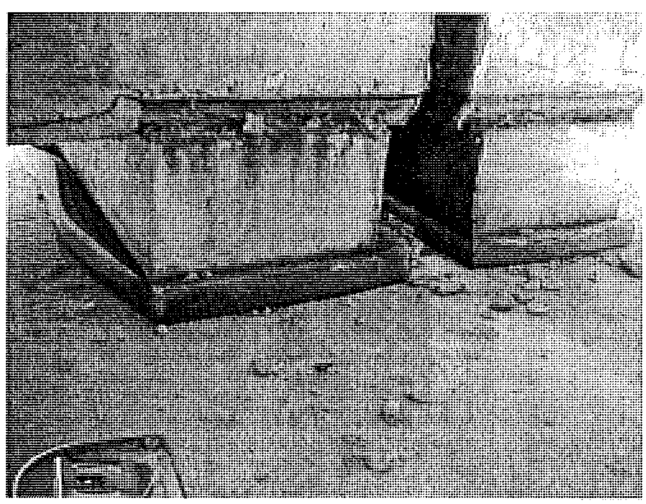
Photo 3-53. Condition of the bearings on pier

The diaphragms at the piers are in good condition. The diaphragms at the North abutment have small areas of delamination between beams W1 and W4 and exposed shear reinforcement between beams W2 and W3. The diaphragms at the South abutment between beams S1 and S2 are cracked. The sole plates at the abutments are painted and slightly rusted.

Some of the neoprene pads appear to have lost flexibility, deformed and partially lost.

3.7.3.9 *NBI No: 06111 S05*

The bridge was inspected on August 29, 2001. The inspection took 5 hours and 30 minutes between 7:00 a.m. and 12:30 p.m.

General Bridge Information

The bridge is located in Arenac County. Constructed in 1968 the bridge carries Lincoln Road over I-75 SB in 2.4 MI South of Sterling.

Bridge Geometry

The bridge is 155.8-ft long, 32.2-ft wide and carries 2 lanes of traffic with 4.1-ft wide sidewalks. The bridge orientation is West-East. This is a skewed three span bridge with five beams per span. The beam lengths are: 54.80-ft at the central span W2, 33.6-ft at span W1 and 36.8-ft at span W3. The side spans beam types are III for fascia and II for interior, and center spans beam type is III.

Abutments, Pier, Deck and Joints

The west abutment has several hairline vertical cracks. The North side of the abutment has wet and leaching cracks (see Photo 3-54).

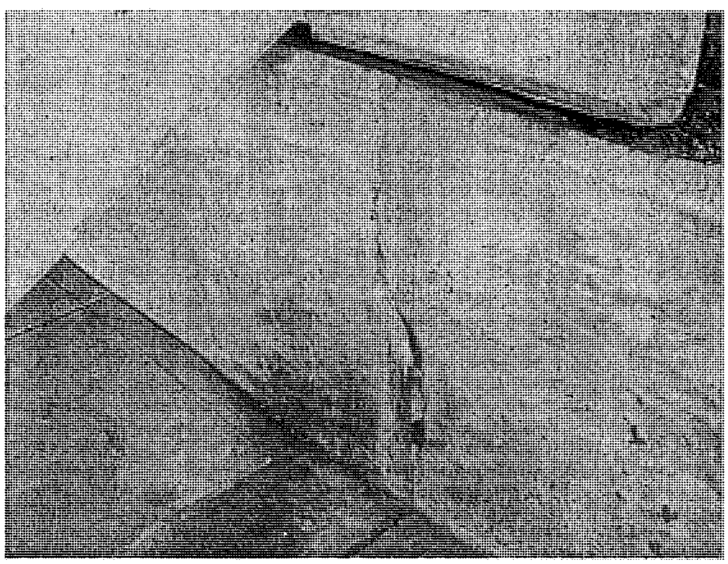

Photo 3-54. Leaching crack at the North side of West abutment

The East abutment has cracks between beams S2 and S4 and wet and leaching cracks at the South and North sides of the abutment.

Pier W1 has vertical and horizontal cracks, rust and water stains. Large area of delamination noted under beams S4 and S3 at the East fascia of pier W1. The West fascia of pier W1 exhibits heavy rust stain and exposed reinforcement. Pier W2 has rust and water stain. Wet cracks, areas of exposed reinforcement and delamination noticed. The typical condition at the piers is shown in Photo 3-55:

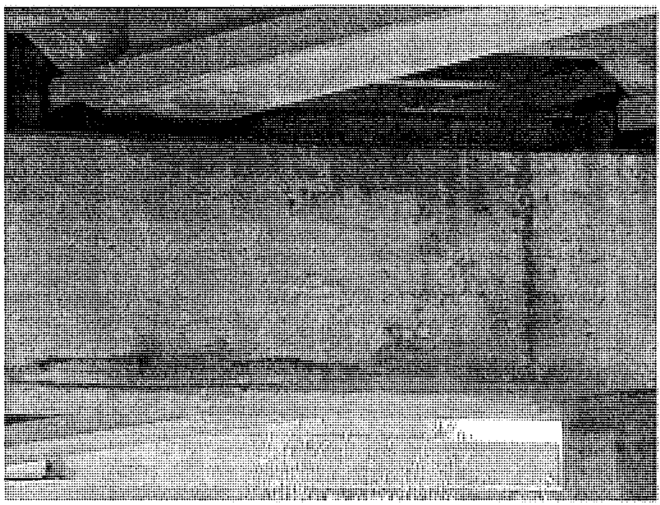

Photo 3-55. Typical pier cap beam condition

The concrete deck overlay is patched with the asphalt. The expansion and construction joints are broken and few asphalt patches noticed along the construction joints. Sand, gravel and debris accumulated along the sidewalks. The drain system is partially filled with debris. Cracks are noted at the deck bottom along East abutment and exposed reinforcement with rust stain around at bay S1. Efflorescence stain noted at bays S1 and S2 next to West abutment.

Stringers, Bearing and Diaphragms

The beam-ends display cracks, spalls, delamination, rust, water, and efflorescence stains (see Photo 3-56).

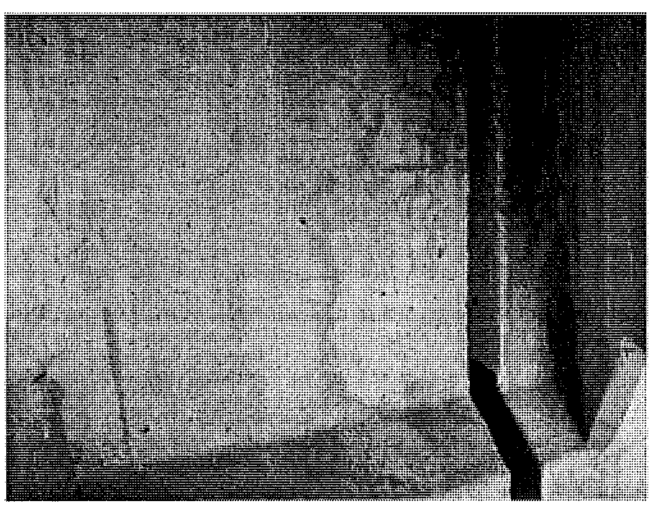

Photo 3-56. Cracks and water stain at the beam end

Some of the spalled beam-ends showed exposed reinforcement (see Photo 3-57).

Photo 3-57. Cracks, spalls and exposed reinforcement at some of the beam-ends

The sole plates at the piers are rusted and bearing pads are deformed. The beam-ends at the abutments have hairline cracks, minor water and rust stains. The bearings at the abutments are painted but partially rusted. Some elastomeric pads are deformed with ends curling. Few diaphragms are cracked and have some rust and efflorescence stains.

3.7.3.10 *NBI No: 06111 S06*

The bridge was inspected on August 29, 2001. Inspection took 4 hours between 12:30 p.m. and 4:30 p.m.

General Bridge Information

The bridge is located in Arenac County. Constructed in 1968 the bridge carries Lincoln Road over I-75 NB in 2.4 MI South of Sterling.

Bridge Geometry

The bridge is 155.8-ft long, 32.2-ft wide and carries 2 lanes of traffic with 4.1-ft wide shoulders. The bridge orientation is West-East. This is a skewed three span bridge with five beams per span. The beam lengths are: 54.30-ft at the central span W2, 36.8-ft at span W1 and 33.6-ft at span W3. Side spans beam types are III for fascia and II for interiors, and center spans beam type is III.

Abutments, Pier, Deck and Joints

Both abutments have vertical hairline cracks through the walls and wet cracks, some with efflorescence stains, at South and North corners (see Photo 3-58).

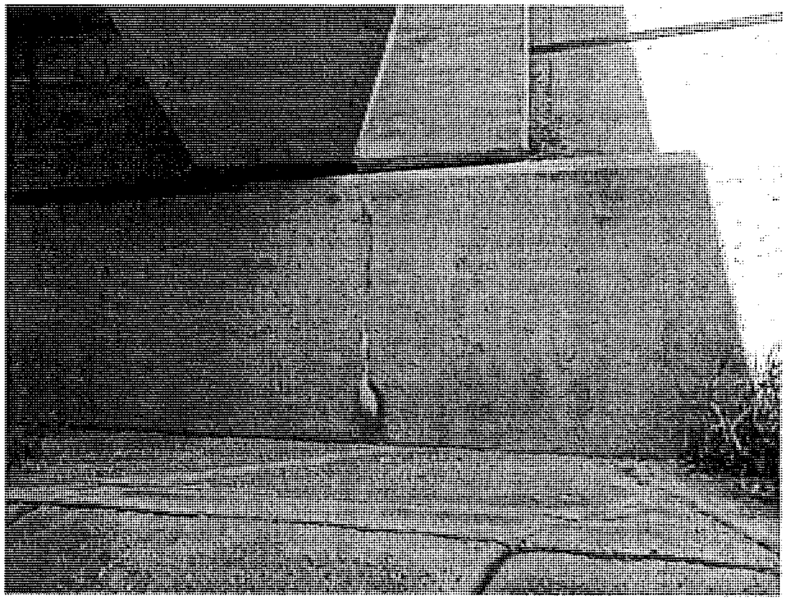

Photo 3-58. Cracks with evidence of moisture and efflorescence at the abutments

Next to the beam-ends some diaphragms at the abutments have minor spalls and water stains. The North side of the diaphragm at the East abutment is delaminated and the South side of the West abutment has exposed reinforcement and rust stains.

The piers have rust and water stain. The East fascia of pier W1 and the West fascia of pier W2 are cracked and delaminated. The West fascia of pier W2 displays exposed shear reinforcement, rust and efflorescence stains (see Photo 3-59).

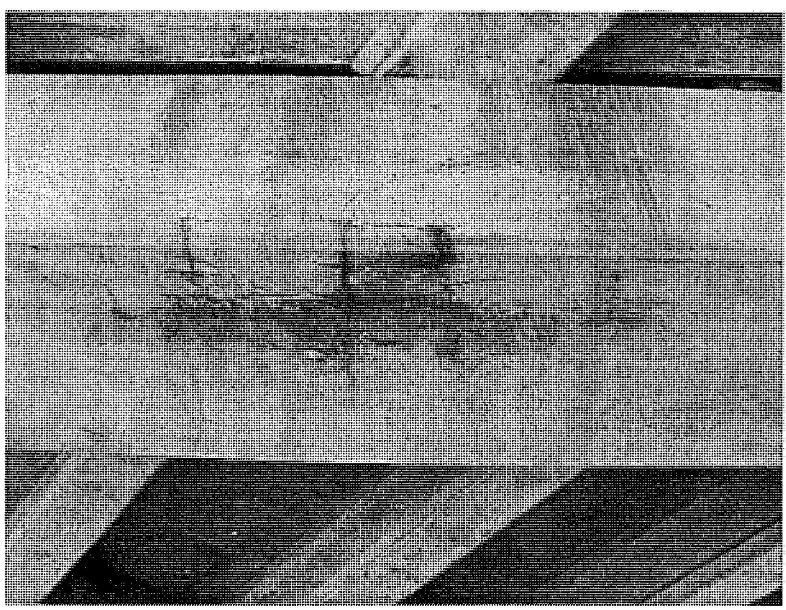

Photo 3-59. Exposed reinforcement, rust and efflorescence at W. fascia of pier cap beam W2

The concrete deck of the main span is in good condition. Both approaches have asphalt patches and potholes. The construction joints are partially broken as it shown in Photo 3-60.

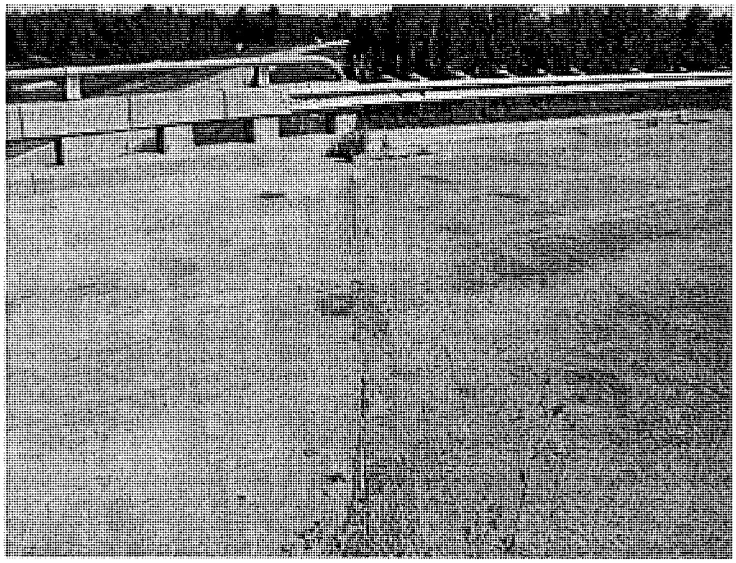

Photo 3-60. Construction joint between deck and approach

The shoulders are covered with gravel and debris. Drainage system is partially filled with debris. The bottom of the deck has cracks and efflorescence stains in the bays next to both abutments. Spalls with exposed reinforcement are noticed at bay S1 and bay S4 next to East abutment.

Stringers, Bearing and Diaphragms

The beam-ends at the piers are cracked and delaminated. At pier W2 beams S2 and S5 and at pier W1 beams S3 and S5 spalled and showing exposed shear reinforcement. The common beam-end condition is shown in Photo 3-61.

Photo 3-61. Common beam-end condition

At the abutments the beam-ends display some hairline cracks and some areas of water and rust stains.

Most of the diaphragms do not display any kind of deterioration. One diaphragm at pier W1 between beams S2 and S3 has a horizontal crack.

The sole plates are painted, some of them are slightly rusted. Some of the elastomeric pads are deformed with ends curling.

3.8 Data Organization

3.8.1 Condition Database

The data collected during the field investigation are organized in a Microsoft Access database. The organization of the database consists of several tables with links as shown in Figure 3-2. The first table of the hierarchy is the Inventory table. This table contains the inventory information on the 20 bridges that were inspected. The fields in this table are Pontis Bridge ID, County, Region, Year Built, Number of Spans, Maximum Span Length, Deck Width, Length, Clearance, Facility Carried, Feature Intersected, Deck Notes, Barrier Notes, Pier Notes, and Abutment Notes.

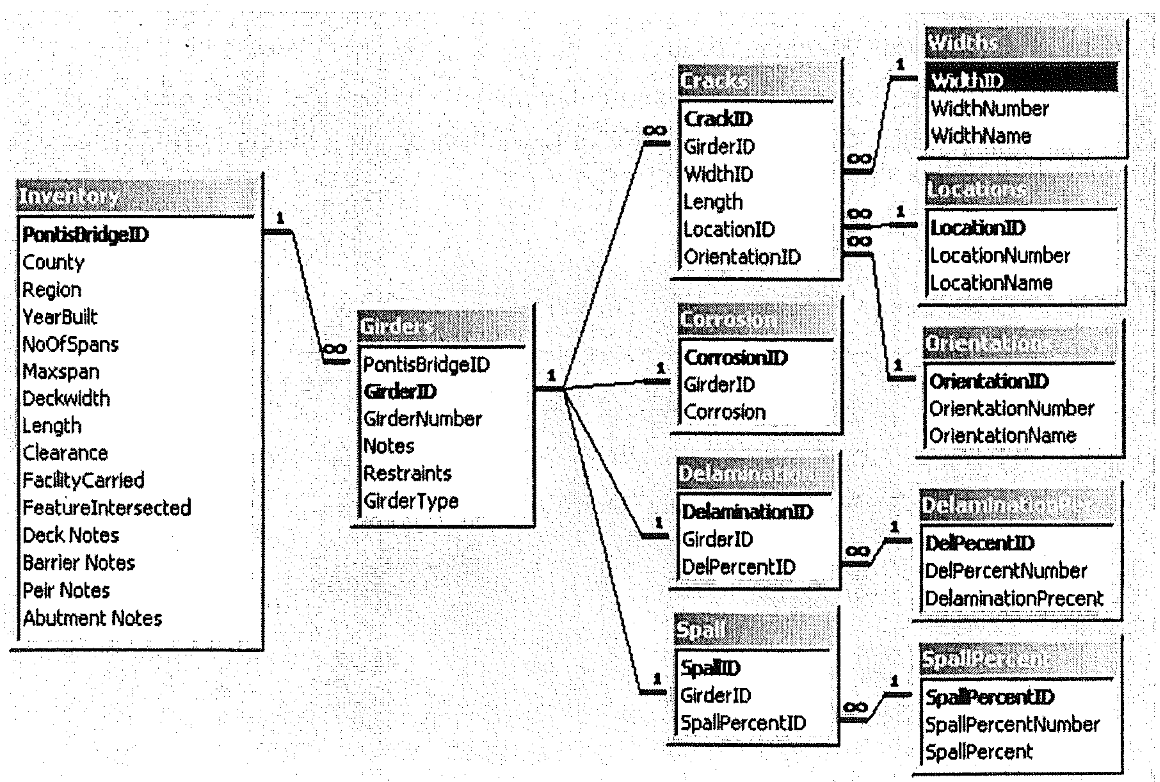

Figure 3-2. Data Base Relationships

The next table is titled Girders, and it has a "one to many" relationship with the Inventory table. "One to many" refers to the relationship where one field in a table can have many fields from another table related to that one. The fields in this table are Girder ID, Girder Number, Notes, Restraints, and Girder Type. The Girder ID field is an auto-numbering column used to create relationships to other tables. The Girder Number includes a span, a beam in the span, an end of that beam, and a face at that end. The first letter in the notation explains which direction to start counting the spans from (either S-south or E-east), and the number that follows is the span number. Next, is a letter denoting the direction for counting of the beams (either S-south or E-east), and the number that follows is the beam number. The following letter designates the end of the beam of interest (either {N-north or S-south} or {E-east or W-west}). The last letter designates one of the seven faces (either {N-north and S-south} or {E-east and W-west} U-underneath, R-rear end, B-bearing, I-interior diaphragm, X-exterior diaphragm). An example of a beam number for a bridge running north to south with three spans and seven beams in a span is S2-E5-N-W.

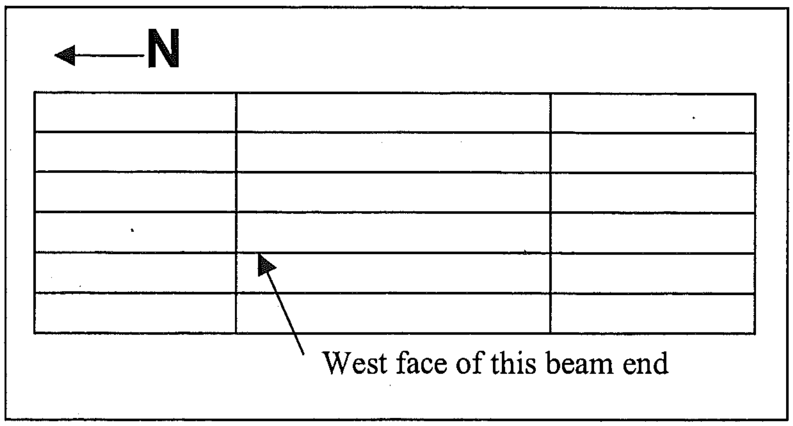

Figure 3-3. Example of How to Designate a Beam Number

This number reads: second span from the south, fifth beam from the east, north end, west face. The Notes field is for any comments on that particular Girder Number. A pictorial representation of where the girder-end is located can be seen in Figure 3-3. The Restraints field describes the beam-ends as either pinned or roller. The Girder Type only permits the four AASHTO types, Wisconsin 70" I-beam type, Michigan 1800 type, or no type (when the face is a diaphragm).

The Cracks table has a "one to many" relationship with the Girders table. The fields in this table are Crack ID, Girder ID, Width, Length, Location, and Orientation. The Crack ID and Girder ID are both auto-numbering columns used to create relationships to other tables. The Width field has an associated table named Width where the preset values for this field are kept; the relationship between Width field and Width table is a "many to one." The preset values in the Width table are Hairline, Moderate, Major. The number values designated to Hairline are ≤ .001 inch, Moderate are .002-.010 inch, and Major are > .010 inch. The Length field has the numeric values in inches of each crack length. The Location field has an associated table named Location where the preset values for this field are kept; the relationship between Location field and Location table is a "many to one." The preset values in the Location table are Top Flange, Web, Bottom Flange, Bearing, Top Flange & Web, Web & Bottom Flange, Top & Web & Bottom, and Diaphragm. The Orientation field has an associated table named Orientation where the preset values for this field are kept; the relationship between Orientation field and Orientation table is a "many to one". "Many to one" refers to the relationship where one field in a table can have many of the same entries of a preset value in another table. The preset values in the Orientation table are Horizontal, Vertical, < 90, > 90, MUC (Multiply Ultra fine Cracks), and Mapping. < 90 and > 90 refer to diagonal cracks, and the convention is shown in Figure 3-4.

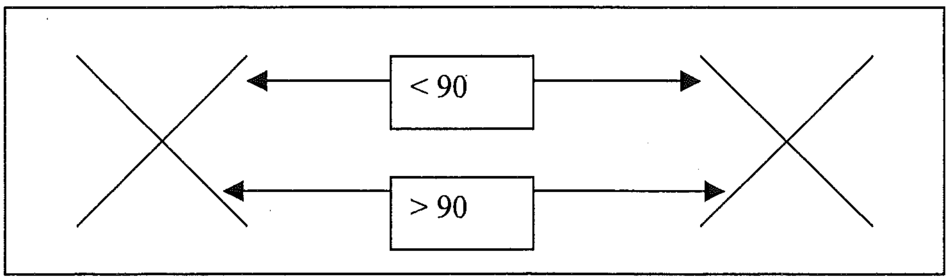

Figure 3-4. Diagonal Crack Convention

The Corrosion table has a "one to one" relationship with the Girders table; the fields in this table are Corrosion ID, Girder ID, and Corrosion. The Corrosion ID and Girder ID are both auto-numbering columns used to create relationships to other tables. The Corrosion field is a check box. If there was any sign of rust on that particular face then the box was checked.

The Delamination table has a "one to one" relationship with the Girder table; the fields in this table are Delamination ID, Girder ID, and DelPercent ID. Delamination ID and Girder ID are both auto-numbering columns used to create relationships to other tables. The DelPercent ID field has an associated table named Delamination Percent where the preset values for this field are kept; the relationship between DelPercent ID field and Delamination Percent table is a "many to one." The preset values in the Delamination Percent table are 0%, 25%, 50%, 75%, 100%. These percentages have a range meaning:

Table 3-9. Delamination Percent

Database Entry	Range
0%	0%,
25%	1%-25%,
50%	26%-50%,
75%	51%-75%
100%	76%-100%

The Spall table has the same format as the delamination table.

A data entry form was created in Microsoft Access to aid in the data entry process, which is shown in Figure 3-5.

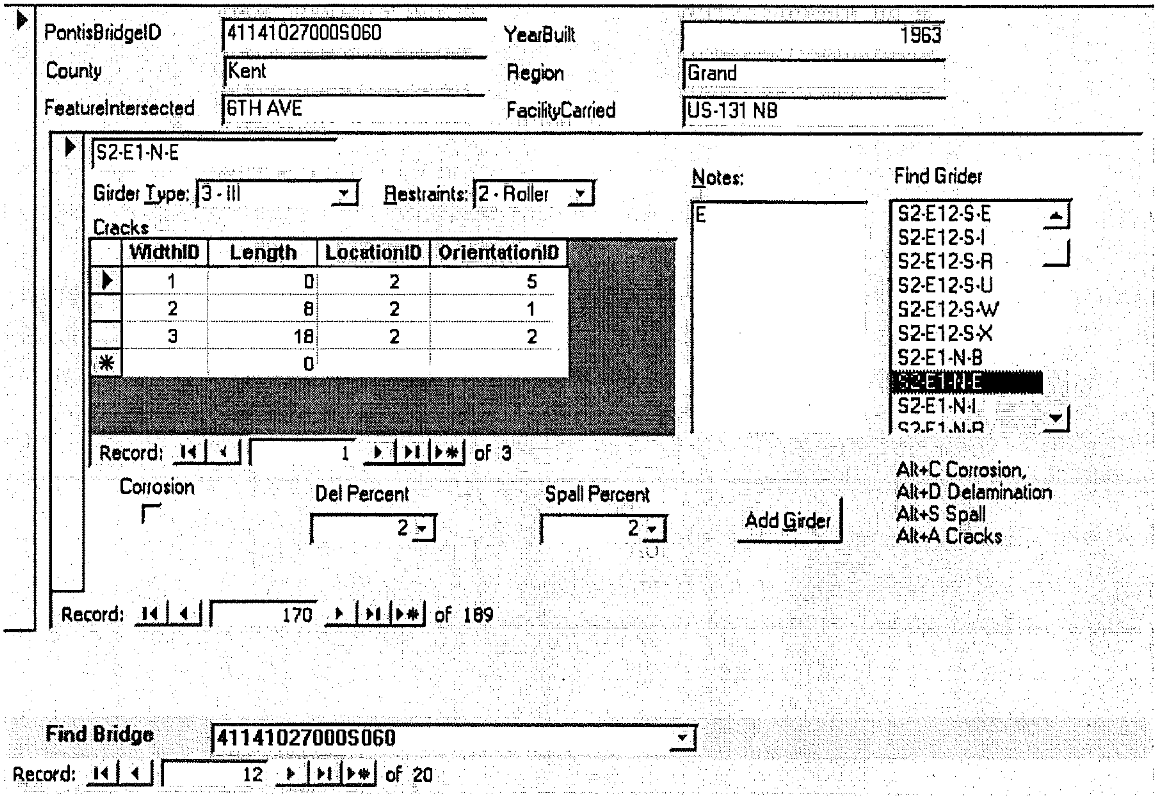

Figure 3-5. Inspection Data Entry Form

3.8.2 Inspection Photographs

The inspection photographs were organzined into a digital database that is located on the website, therefore it will be discussed in sub-section 8.3.1.

3.9 Summary

Field inspection of twenty prestressed concrete I-beam bridges was conducted. These bridges ranged from two to forty years old and were previously condition rated between four (poor) and eight (very good).

The field survey included a detailed visual inspection of the overall I-beam structure condition, including documenting the condition of each I-beam end.

As was expected the older structures exhibit more deterioration than the newer structures. The field survey shows that most of the older bridges suffer severe beam-end deterioration. Although the bridge from 1968 located at Bay region, Arenac County, NBI No. 06111 S04 was found to be in excellent shape in comparison to its peers.

Two to seven year old beam-ends were found to be in good condition, although these bridges (1984-1999) already display some beam-end problems, such as vertical and diagonal cracks, and horizontal cracks along the flange.

The data collected during the field inspection was analyzed and compared with the information available in MDOT NBI inspection reports. The condition of abutments, piers and deck observed by the team does not differ significantly from NBI inspection comments for these items.

The main goal of the team was to collect precise information on the beam-end condition and thus very detailed information on location, size, and orientation of deterioration was obtained. This information could be obtained from the field notes and it is also available on the website. The issue of beam-end deterioration was difficult to compare with MDOT practice because most of the NBI inspection reports do not rate the condition of beam-ends. Some NBI reports mentioned beam-end deterioration under the item 59 "Stringer". There is a need to include end condition assessment in the inspection procedure. This will allow for inspectors to rate the condition and properly assign a protective strategy prior to severe deterioration.

The analysis for the inspection data can be found in section 4.3.