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Cause of Concrete Pier Cap Deterioration on the I-75 Bridge over River Rouge (B01 of 82194) in Detroit, and Effectiveness of Repair Methods

Final Report

Submitted to

Michigan Department of Transportation

Department of Civil and Environmental Engineering
The University of Michigan
College of Engineering

Ann Arbor, MI 48109-2125

Research and Technology Section
Materials and Technology Division
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By

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Disclaimer

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1. Introduction and Background

1.1 Background

The main span of the Interstate 75 bridge over the Rouge River in Detroit (B01 of 82194) was built during the summer of 1965 with an HS20 design live load. The main span is a high level, three span continuous steel girder bridge with a composite concrete deck. The substructure of the main span consists of four piers, Piers 37 to 40, and is constructed entirely of reinforced concrete. The adjacent spans are also steel girder spans with composite concrete decks and reinforced concrete substructure.

The bridge, which is in a heavy industrial area, has experienced considerable deterioration over its life. A superstructure rehabilitation project was conducted in 1989. In 1994, a contract was awarded to address cracking in the pier caps of Piers 37 to 40. Piers 37 and 40 were considered separately from Piers 38 and 39, as the former are directly beneath deck joints, and the latter are not. The deck joints above Piers 37 and 40 have allowed water and salts from the bridge deck above to enter the pier cap concrete. Furthermore, misaligned deck drain pipes have allowed water and salt to spill onto Pier Caps 37 and 40 as well. These two pier caps are more severely deteriorated than are Caps 38 and 39. Cores taken from Pier Caps 37 and 40 show high chloride content (5.7 to 34.6 lb/yd^3 at 1/2 inch below the concrete exterior surface), and generally adequate but highly variable compressive strength (2680 to 7450 psi). Petrographic examination indicated the presence of alkali silica gel associated with the fine aggregates in the concrete of Caps 37 and 40. It was concluded that concrete overstressing and chloride damage led to the deterioration of Caps 37 and 40.

Pier Caps 38 and 39 showed deterioration similar to that in Caps 37 and 40, though to a lesser extent. Due to the lack of deck joints and misaligned drains, these piers were not suspected of suffering from severe chloride related distresses. Petrographic examination of a fragment from Pier 38 yielded similar findings to the cores from Caps 37 and 40, with alkali silica gel noted in the mortar.

A repair method was established, including epoxy injection of all cracks, concrete chipping and replacement where necessary, and application of a penetrating surface sealer to repel water. Temporary supports for Piers 37 and 40 were used during repair. It was found that chipping was so easy on Pier Cap 40, that one of the two pier caps for this pier was completely removed. This led to concern at MDOT as to the conditions of all four pier caps, and whether the proposed repair methods were adequate. Due to the unexpected severity of the distresses, especially in Piers 37 and 40, project funds were exhausted before repairs on Piers 37 to 40 could be completed. In the first contract the south-bound pier cap of Pier 39 was epoxy injected concurrent with repairs to Piers 37 and 40.

A second contract is planned to complete the rehabilitation. It has been decided that a total replacement of Pier Caps 37 and 40 will be carried out in the second contract. Regarding Pier

Caps 38 and 39, MDOT decided to seek a second opinion on the causes of distress, and the suitability of the proposed repair techniques to determine whether these two pier caps could be saved. The proposed plan of injecting and sealing Pier Caps 38 and 39, along with post-tensioning is the focus of investigation in this study.

1.2 Objectives

The objectives of this study have been to:

- Determine the causes and extent of distress in Pier Caps 38 and 39.
- Evaluate suitable repair techniques, and determine the feasibility of the proposed repair method.

1.3 Research Approach

In considering the possible repair methods for Pier Caps 38 and 39, there are two issues to be addressed, structural capacity and material durability. The purpose of this study is to determine whether the concrete in the pier caps is durable, and if so, whether it can be expected to continue to provide the needed structural capacity in the future. Then, based on the results of this investigation and the structural evaluation of the pier caps (performed under a separate contract by Parsons Brinkerhoff, Inc.) possible repair techniques will be evaluated.

The study has been conducted in four phases, several of which were conducted concurrently. The first phase was a field evaluation of the pier caps in question, as well as the collection of samples for laboratory study. The second phase was a literature review of the state of the art in topics relating to the use of blast-furnace slag as coarse aggregate, the properties of alkali silica reaction (ASR) affected concrete, available pier cap repair methods, and test methods appropriate to this investigation. The third phase of the study included laboratory analysis of the cored samples, and investigation into the historical records relating to the bridge's construction and evaluation. The laboratory investigation included; determination of the concrete strength, elastic modulus, and creep properties; determination of concrete chloride content; petrographic evaluation of the concrete microstructure; testing for potential future ASR expansion, and visual evaluation of the effectiveness of crack injection. The final phase of the project has involved the evaluation of proposed repair techniques based on the findings of the other project phases.

2. Field Observations

A visual review from grade of Piers 37 through 40 was performed on December 11, 1995 by the project team. A plan of the site location is shown in Figure 1. An overview of the main span of the bridge, showing the piers in question is seen in Figure 2. The team accessed portions of Pier Caps 38 and 39 north-bound from the ground, and from above using a reach-all (Figure 3) provided by the Michigan Department of Transportation (MDOT). Coring of samples from Piers 38 and 39 was conducted by MDOT and under the direction of the project team on December 11-15, 1995.

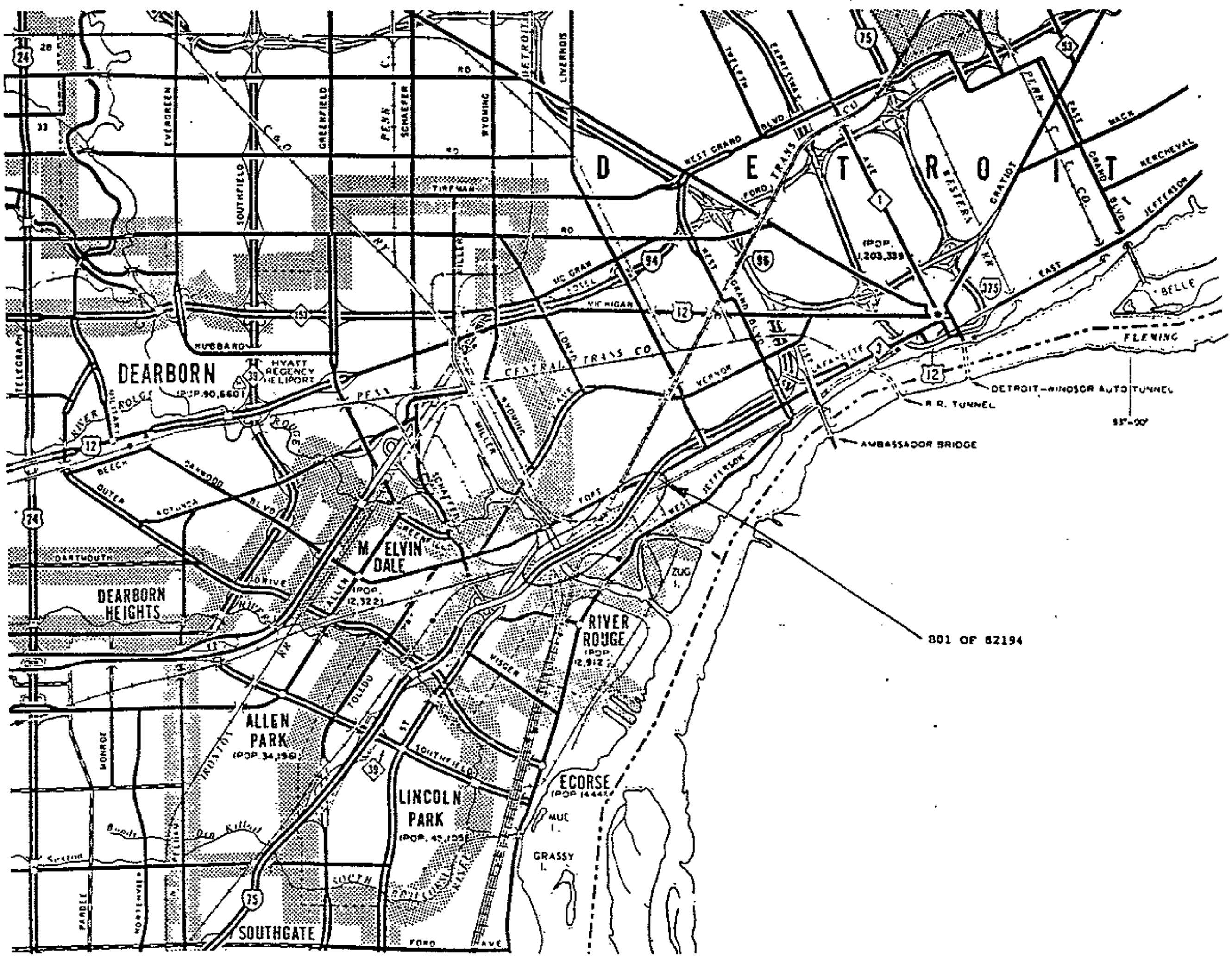


Figure 1. General location of bridge site (from Parsons Brinkerhoff, Inc. project plans).

The bridge is continuous over Piers 38 and 39. Evidence of significant ongoing leakage through the bridge deck at these piers was not observed, although evidence of previous leakage through the longitudinal joint between north- and south-bound lanes was observed. No other significant sources of water were noted. In contrast, ongoing water leakage was observed at the joints over Piers 37 and 40.

Variation in paste color of concrete surfaces (Figure 4) indicate that several different mix designs were used for construction of the piers. In general, a relatively buff colored paste appeared to be present at most pier columns, while a relatively gray paste appeared to be present at most pier caps. Columns appeared to have been constructed in three equal-height concrete placements, with pier caps placed in two separate placements. This was confirmed by the construction records, which showed that the pier caps contained 7.5 sacks of cement per cubic yard of concrete compared to the columns which had 6 sacks of cement per cubic yard of concrete.

Cracking in pier columns varied. In general, a relatively fine vertical crack was observable at pier columns, approximately centered in the width of the column. Figure 5 shows a column from Pier 40 north-bound exhibiting this type of crack pattern. However, in some areas the cracking was more extensive and pronounced. For example, in Pier 38 north-bound one column exhibited significantly more extensive cracking at the top column section than at lower sections; concrete color in the more extensively cracked section was gray while the remainder of the column was buff.

Extensive pattern cracking was observed at all of the pier caps in Piers 37 to 40. Figure 6 shows a close-up of the typical pattern cracking. Some cracks at the north face of Pier 38 north-bound measured wider than 1/8 in. In general, no delamination was detected when surfaces adjacent to cracks were sounded with a hammer. However, surfaces at the exterior (west) face of the pier cap were delaminated.

The pier cap at Pier 40 south-bound had been demolished in preparation for replacement. Reinforcing steel presumably removed from the pier cap was observed adjacent to the base of the pier, as shown in Figure 7. Corrosion observed on the steel was not considered severe since no substantial pitting or rust pack was observed and reinforcing steel deformations were generally clearly present. Reportedly, concrete was easily demolished at this pier cap.

Cracks in the pier cap at Pier 39 south-bound had been epoxy injected as part of repairs in progress, as shown in Figures 8 and 9. Figures 9 and 10 show the locations of the cores taken in the north face of the pier cap for this project.

Figures 11, 12 and 13 show the project's remaining core locations on Pier 39 north-bound, south face, Pier 38 north-bound north face, and Pier 38 south-bound south face respectively.

Finally, Figures 14, 15 and 16 summarize the general locations of all cores.

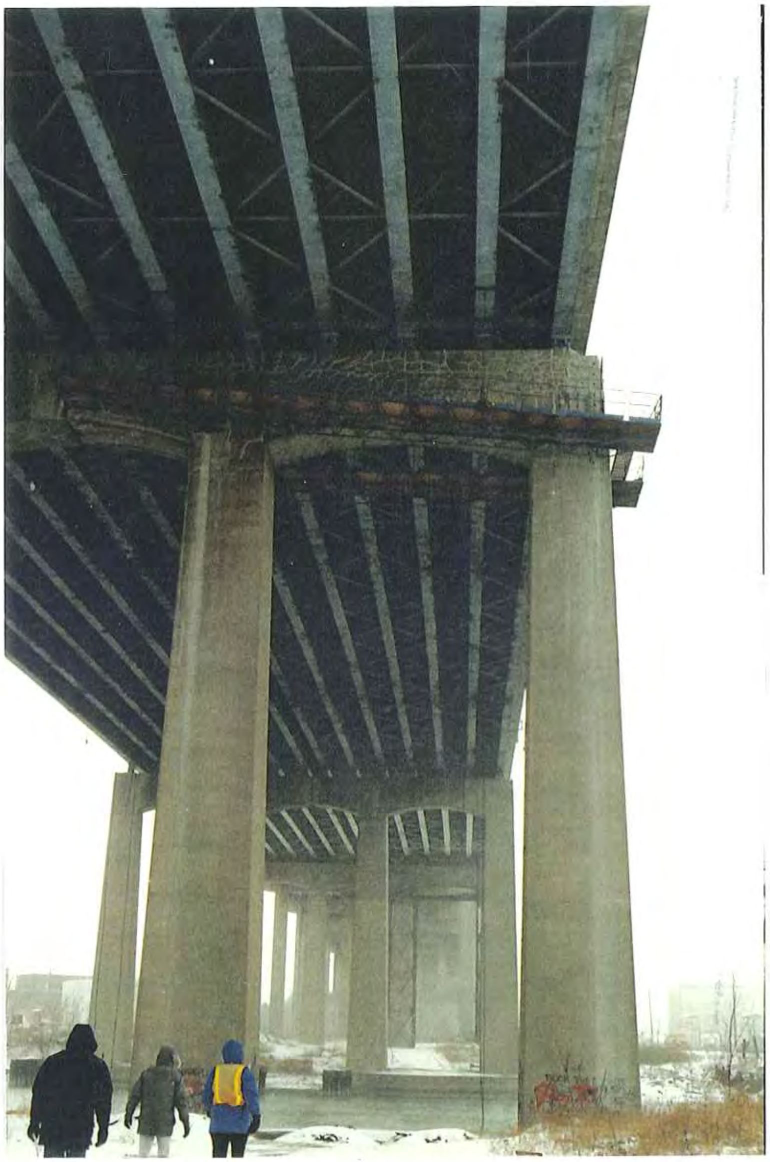


Figure 2. *Overview of the main span of the bridge looking south, showing epoxy injected Pier Cap 39 in the foreground, and Piers 38 and 37 in the background.*

Figure 3. *A reach-all provided access to the bridge piers for observation and coring.*

Figure 4. *North face of Pier 38 north-bound and outer column. Note the difference in color between the pier cap and column.*

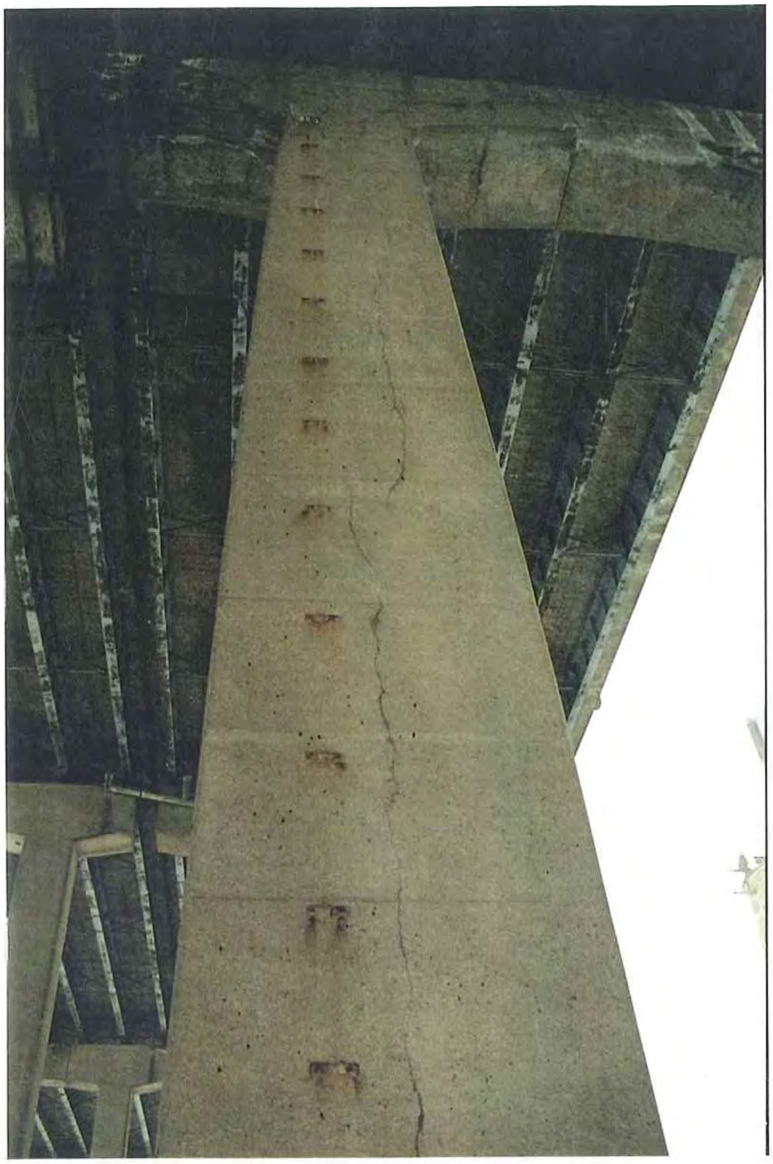


Figure 5. *Vertical cracks approximately centered in the width of the column were typical of the crack pattern in the columns. This column supports Pier 40 north-bound.*

Figure 6. Close-up of south face of Pier 39 north-bound showing the typical pattern cracking seen on the pier caps.

Figure 7. Reinforcing steel presumably taken from Pier 40 during demolition. Note that very little corrosion has taken place; the rust visible on the surface of the bars could have been present at construction or could have occurred after demolition.

Figure 8. South face of Pier 39 south-bound. The cracks have been injected with epoxy as part of the repair procedure. The left end of the pier cap has been prepared for post-tensioning.

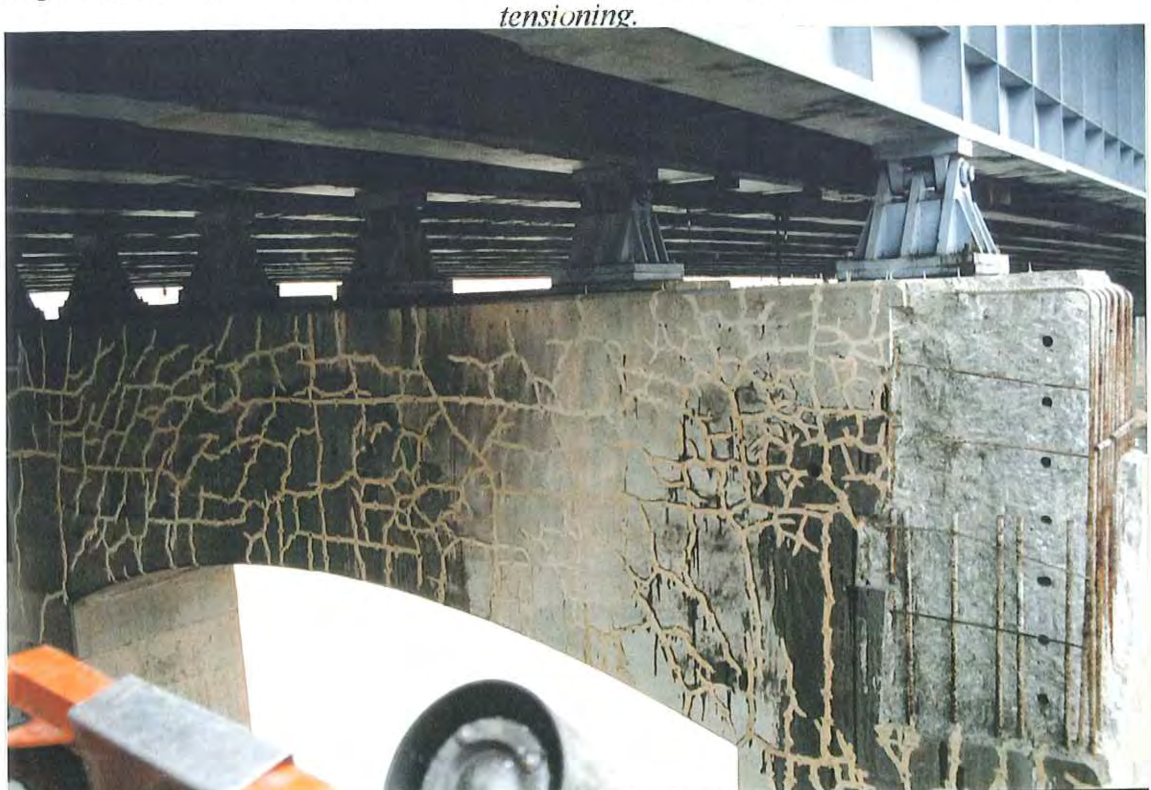


Figure 9. North face of Pier 39 south-bound. The locations of Cores 3, 2, and 1 (left to right) are shown, along with a closer view of the area prepared for post-tensioning.

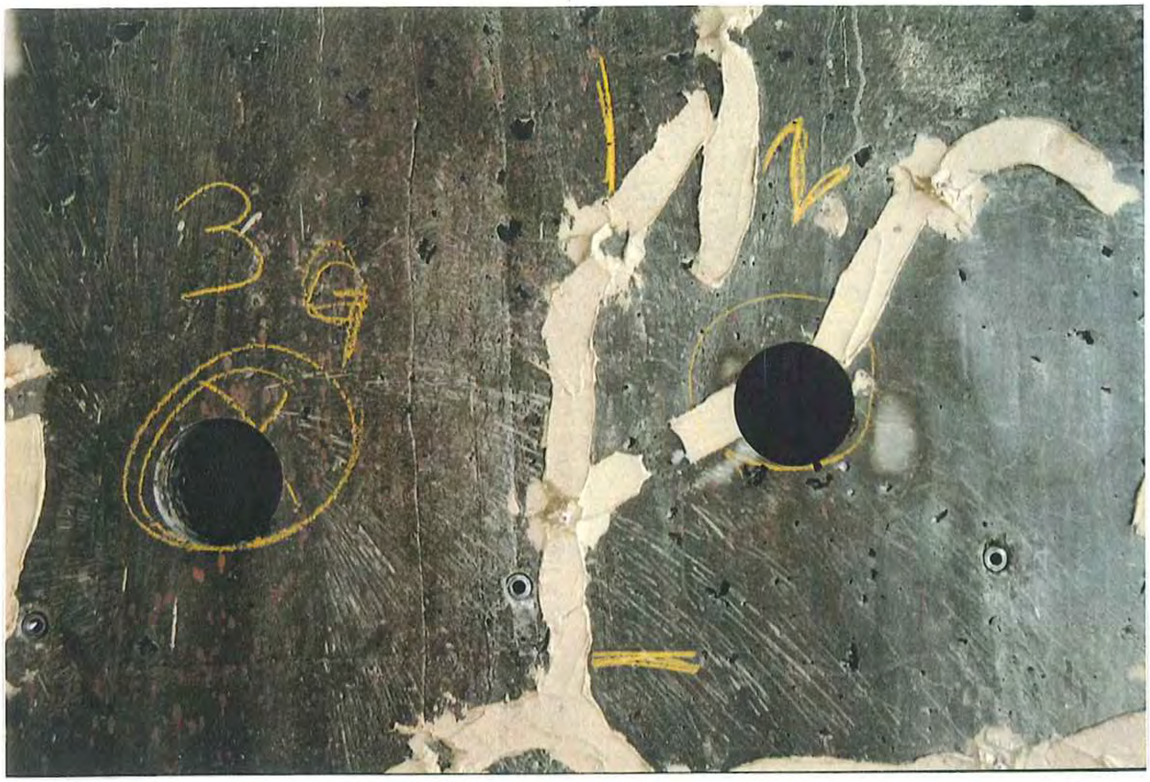


Figure 10. Close-up of area in Figure 9 showing the locations of Cores 2 and 3.

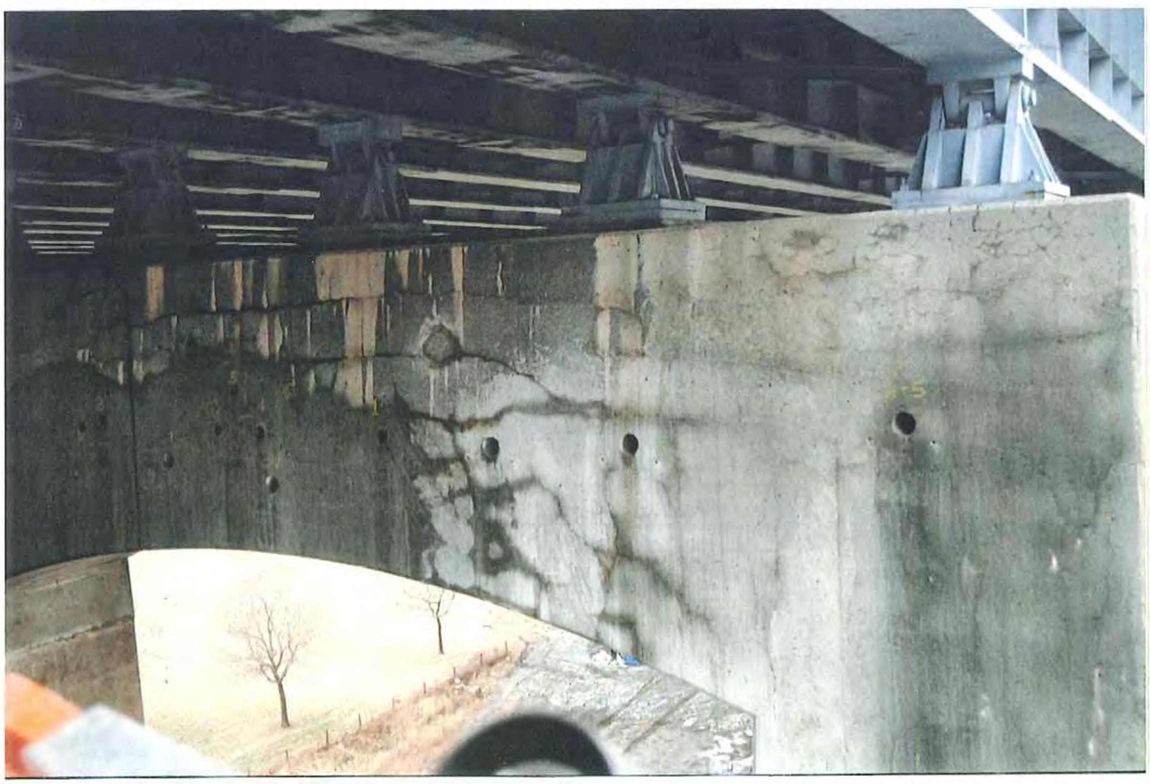


Figure 11. South face of Pier 39 north-bound showing locations of cores. One additional core was taken in the column at the right side of the photo.

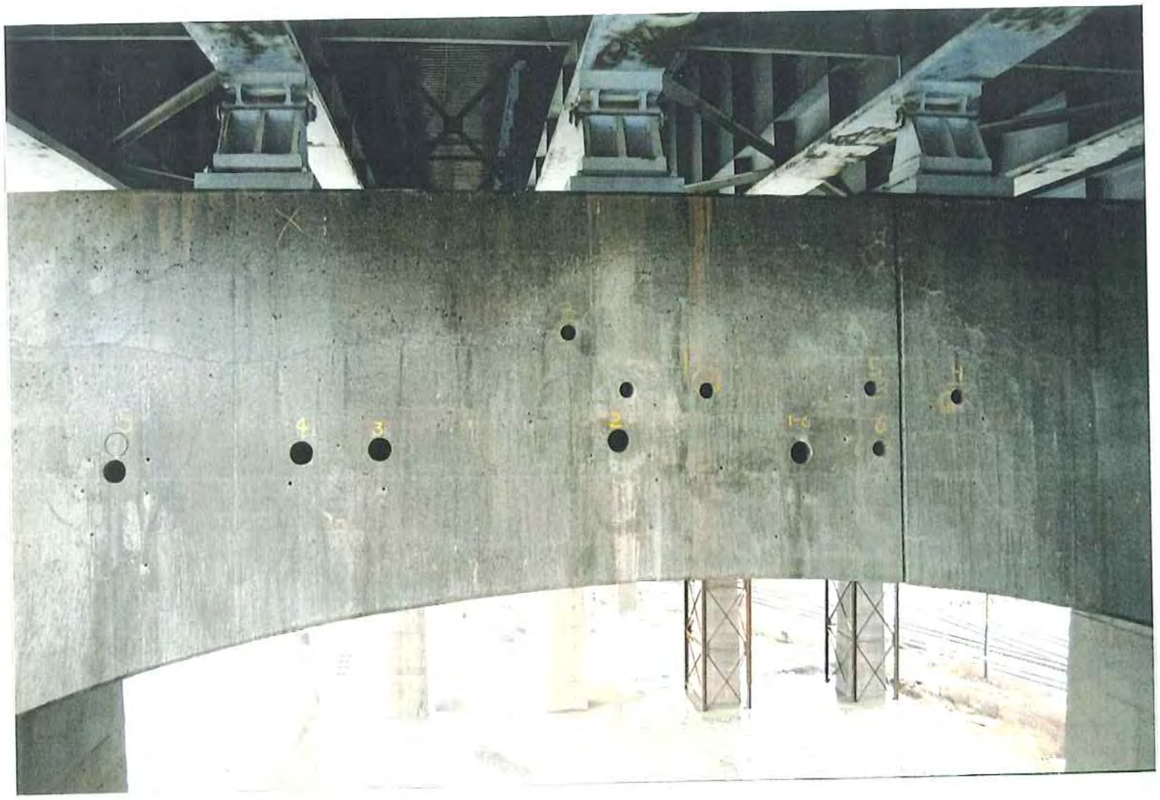


Figure 12. *North face of Pier 38 north-bound showing core locations. One additional core was taken from the column at the left of the photo.*

Figure 13. *South Face of Pier 38 south-bound, showing all three cores.*

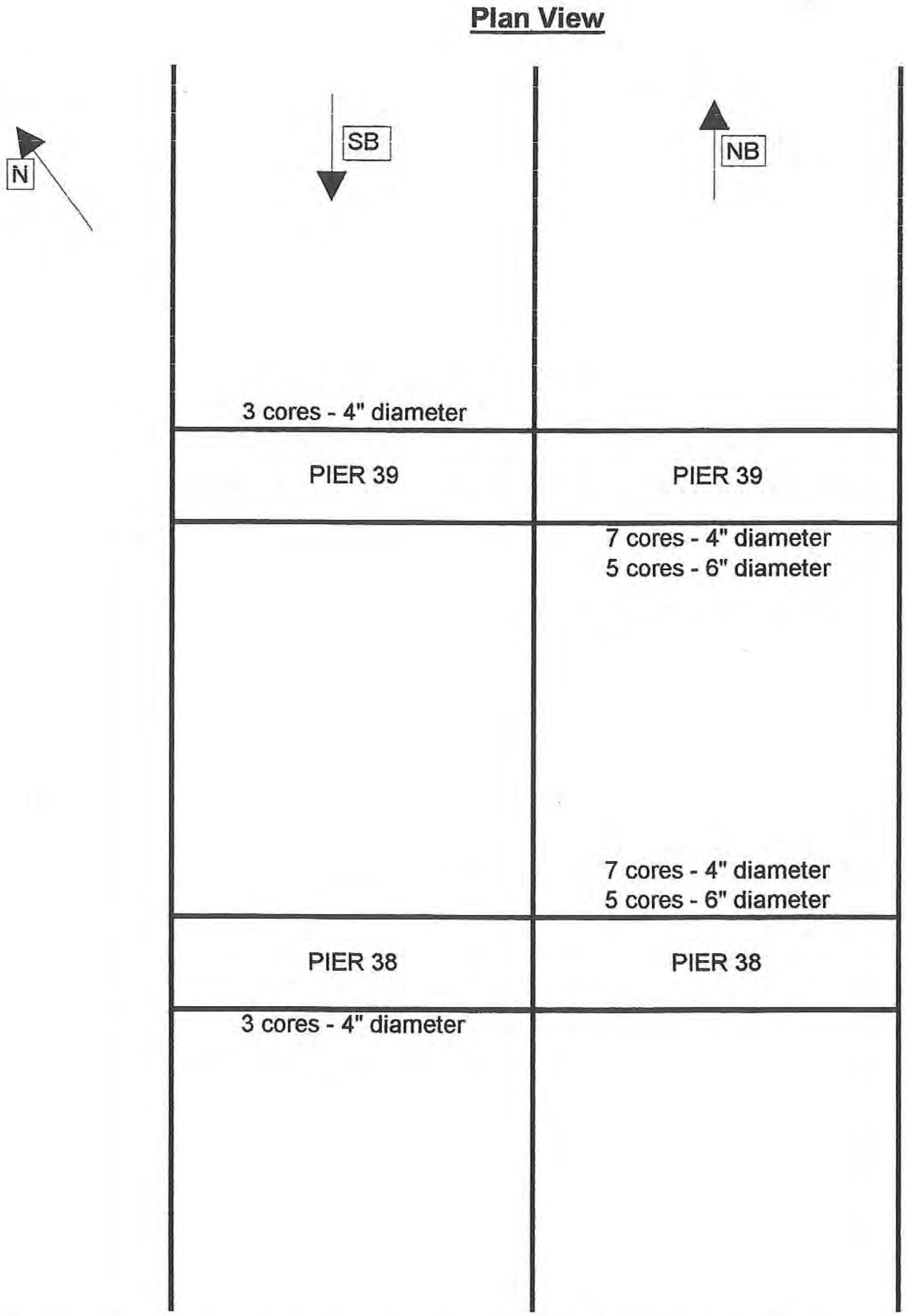
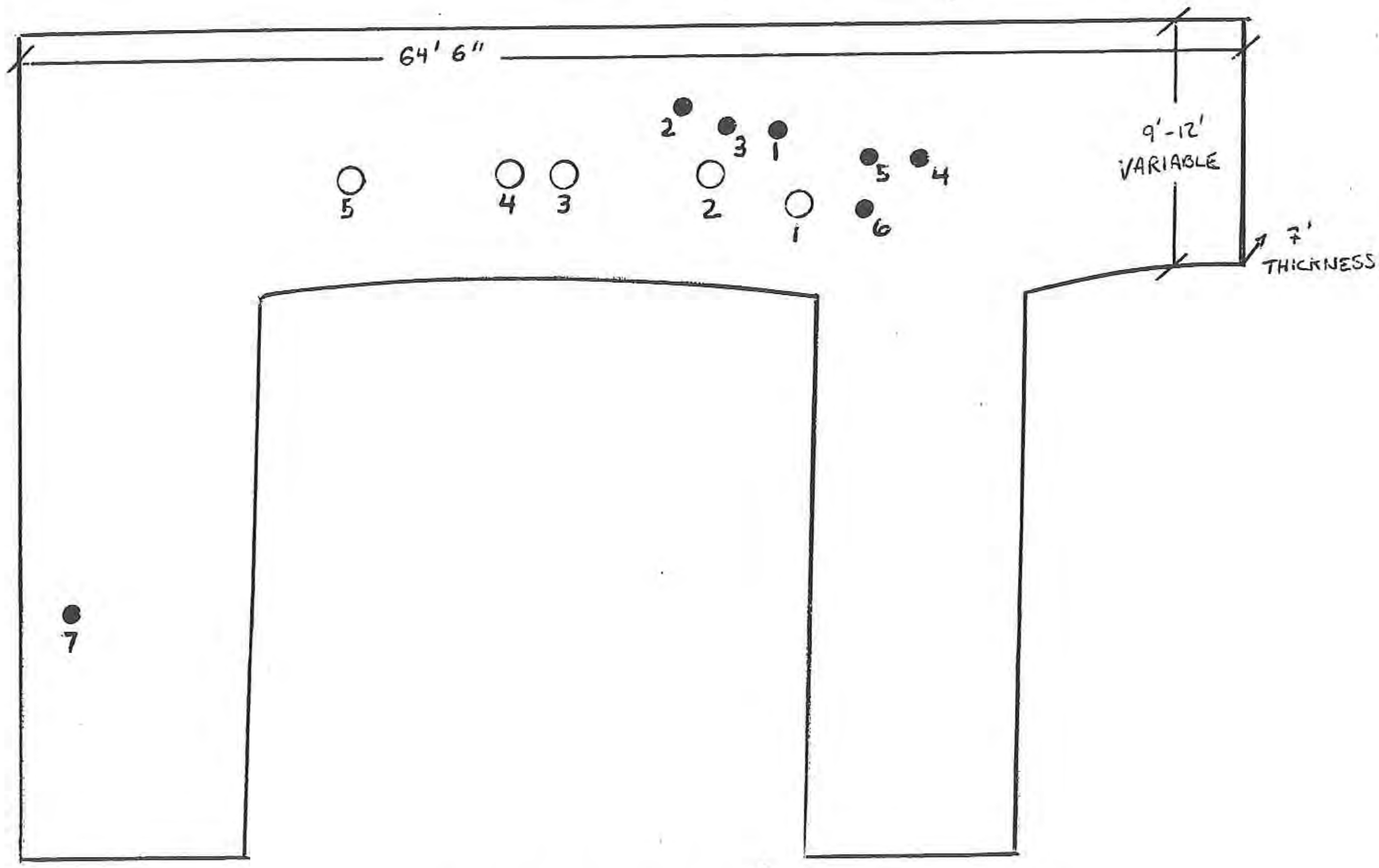


Figure 14 *Plan view of coring locations for Pier Caps 38 and 39 NB and SB.*

Pier 38 North-Bound North-Face



Pier 38 South-Bound South-Face

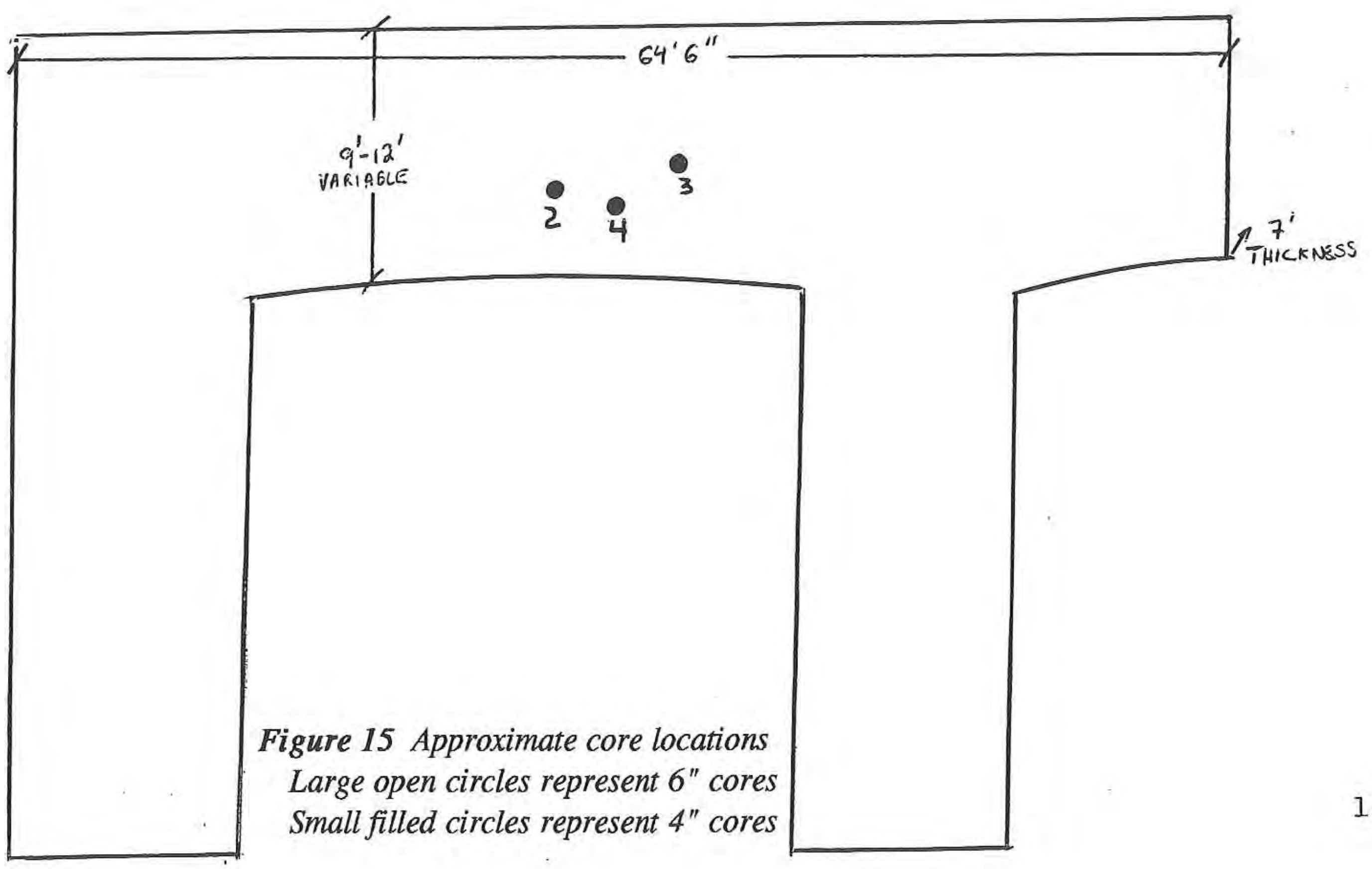
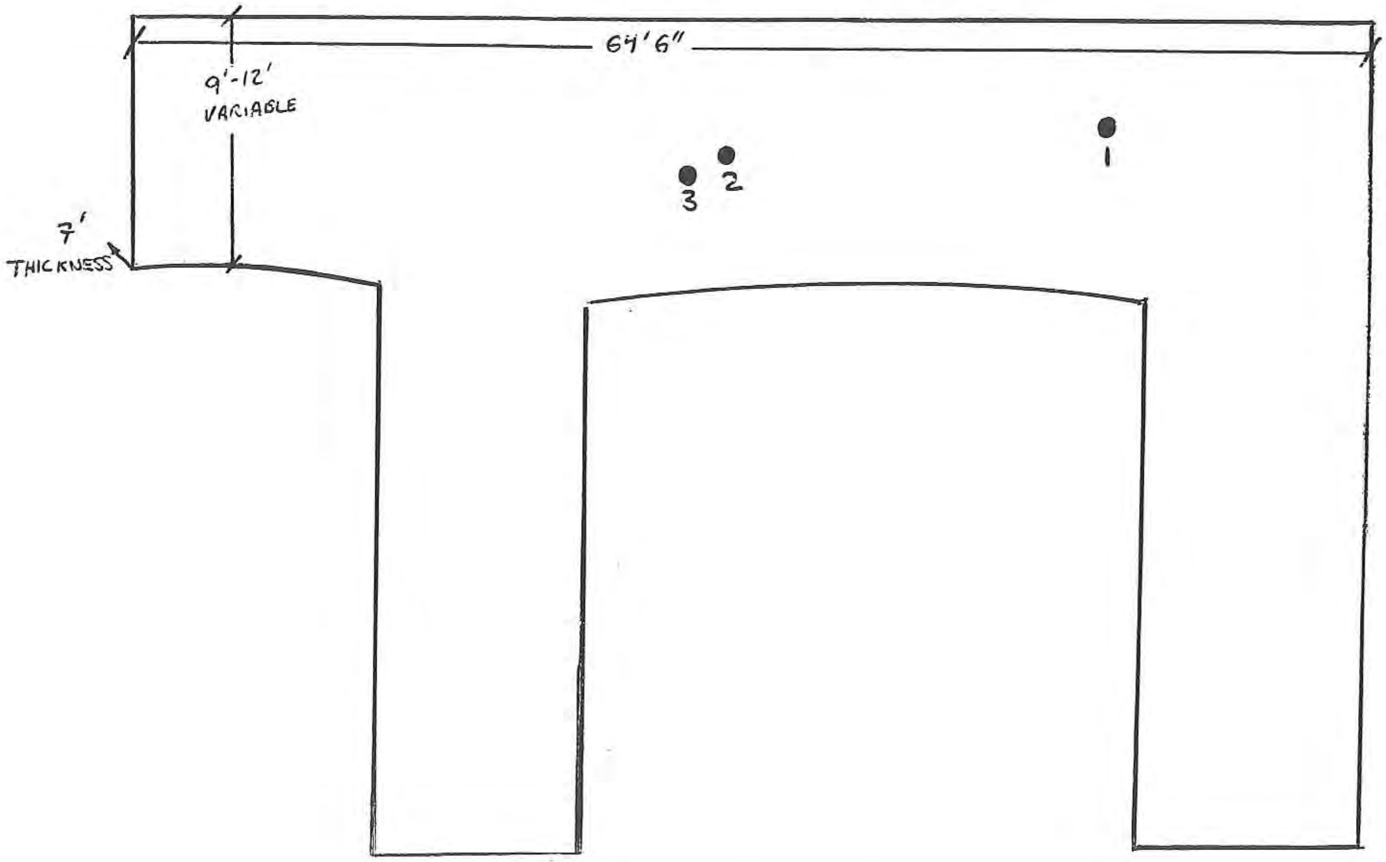


Figure 15 *Approximate core locations*
Large open circles represent 6" cores
Small filled circles represent 4" cores

Pier 39 South-Bound North-Face



Pier 39 North-Bound South-Face

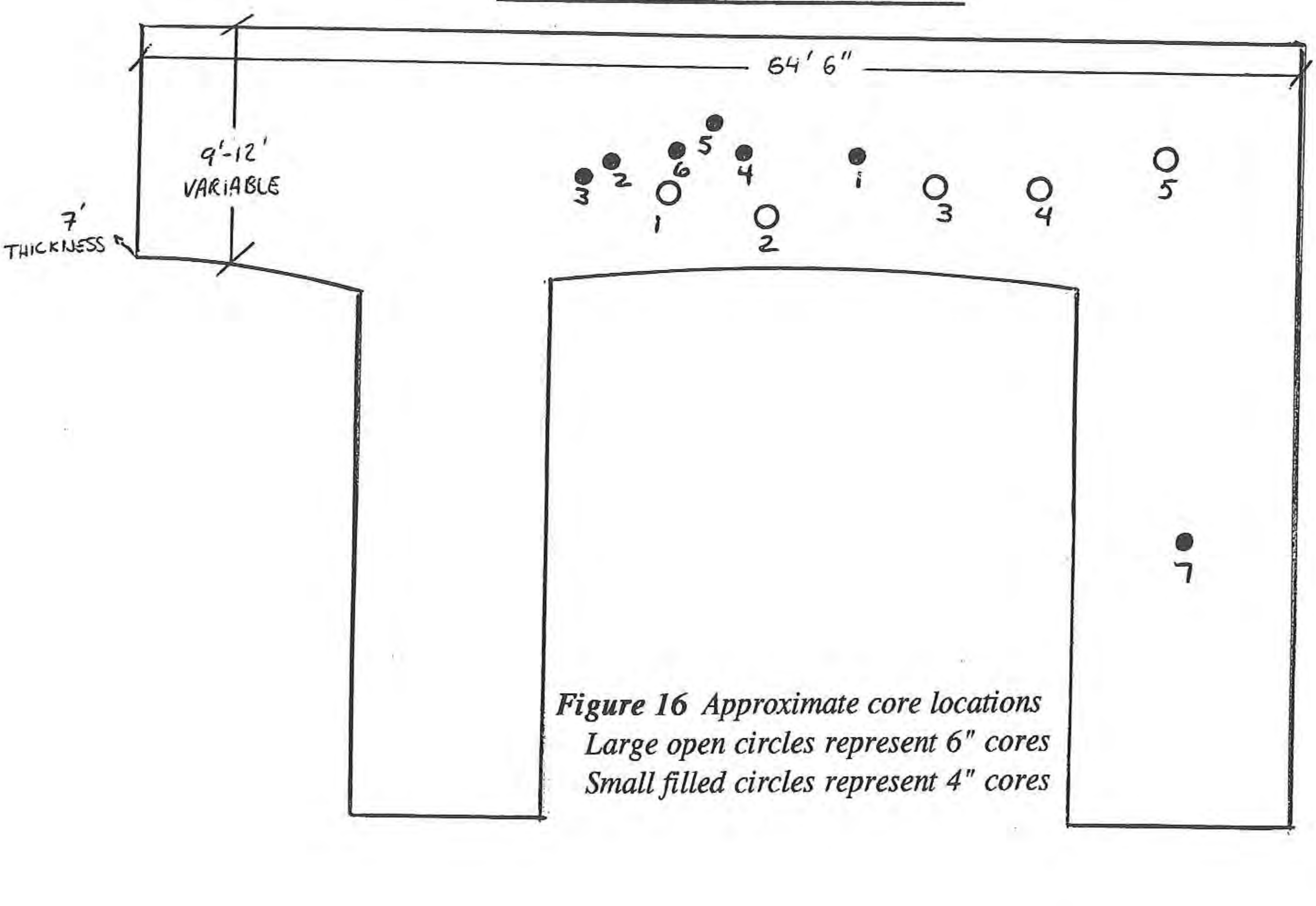


Figure 16 *Approximate core locations*
Large open circles represent 6" cores
Small filled circles represent 4" cores

3. Laboratory Analysis of Cored Samples

The following sections detail the findings of the laboratory analysis of the cores taken from the pier caps. While these sections summarize the findings, the raw data from the various tests is found in appendices A to E at the end of the report.

The first section details the findings of the petrographic analysis, which has been performed independently by two laboratories. Through these investigations the primary causes of deterioration in the pier caps are identified, and together they present a clear analysis of the distress modes. Furthermore, the findings of the petrographic investigations are verified by the other laboratory test methods and MDOT's historical records.

3.1 Petrographic Analysis (ASTM C-856)

Petrographic examinations of the concrete in Pier Caps 38 and 39, performed in accordance with ASTM C-856, were conducted by CTL in Skokie, Ilinois, and PC Laboratoriet A/S in Denmark. The findings of both laboratories are concomitant, and provide significant evidence as to the causes of distress.

The concrete contains at least two generations of crack evidence. Major cracks are present with a smooth appearance, and do not cut through aggregates. These cracks are interpreted to have developed at early ages. Substantial paste carbonation along the crack walls provides further evidence of considerable crack age. Figure 17 shows the presence of calcium carbonate on the crack surfaces.

Fine cracks and microcracks are determined to have occurred in connection with reactive chert and are interpreted to be a result of ASR. These cracks have a sharp appearance and cut through aggregate particles. The reactive particles are identified as different variants of chert (opaline chert, porous chalcedony) The chert appears in the fine aggregate with a grain size of approximately 1 to 4 mm. Figure 18 shows a photomicrograph of ASR affected porous chert.

Much of the evidence of alkali-silica reaction indicates incipient (early) ASR: gel-soaked paste around aggregates, small concentrations of gel adjacent to aggregates, accumulations in air-voids, dark rims on aggregates, peripheral microcracks and zonal degradation of the aggregate. Evidence of active ASR includes local deposits of inter-layered gel and calcite coatings on the walls of major cracks and branching, gel-filled microcracks extending from reactive aggregates into the paste.

Many of the slag coarse aggregate particles exhibit fluorescence; however, this yellow-orange or pink-orange fluorescence is natural and is not associated with deleterious reaction. Based on these observations using the uranyl acetate method, and those from thin section petrography, the slag aggregate is considered innocuous, and ASR in the slag is not implicated as a cause of cracking.

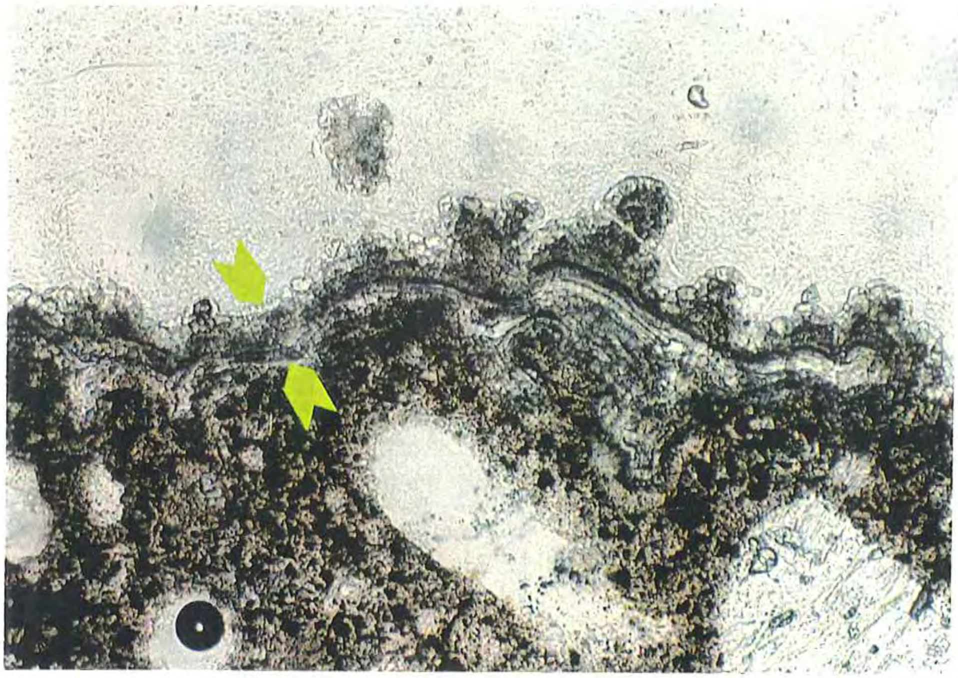


Figure 17. Thin section photomicrograph of core #2 from Pier 38 north face, north-bound, showing inter-layered alkali-silica gel and calcium carbonate deposits (between arrows) on crack wall. Plane-polarized light. Width of field is approximately 350 μm.

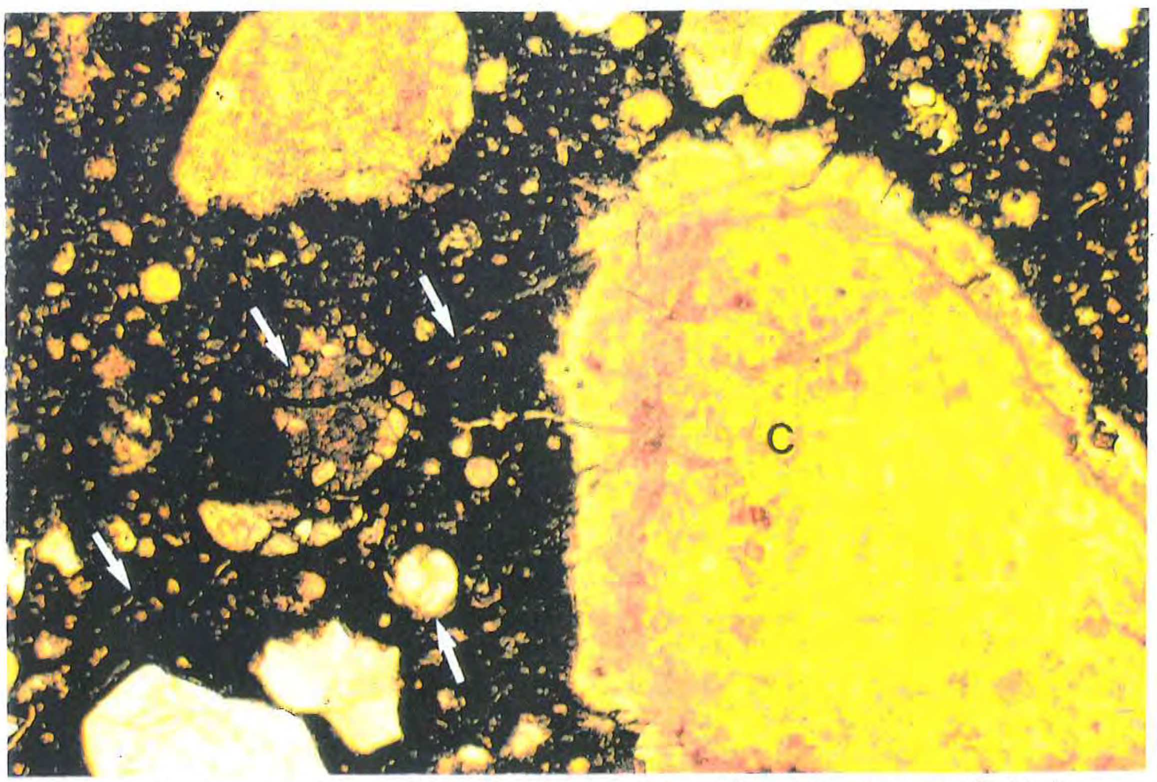


Figure 18. Photomicrograph of pier cap specimen taken in transparent light. 1 cm = 0.11 mm. Porous chert (marked C) is cracked and alkali silica gel is present in the chert, cracks, and air voids (see arrows).

ASR is present in air voids and along cracks. Small amounts of gel line air voids near reactive chert sand grains. Water infiltration has locally leached calcium hydroxide from the paste and deposited ettringite in available space. The total air void content is estimated at 3 to 7% for the various specimens (with most specimens falling in the 5-7% range). The air voids are mostly small and nonuniformly distributed. Considerable entrapped air is noted.

The epoxy in the two cores that were taken from injected areas did not penetrate the cracks. It should be noted that the cracks in the cored specimens were very small. It is not known how effective the injection was in the larger cracks. Figure 19 shows a pier cap sample from an epoxy injected region in Pier 39. The photo shows that there is little or no penetration of the epoxy into the small surface crack.

The full petrographic reports are found in Appendix A.

Figure 19. *Photo of Pier 39 sample with epoxy layer present at the surface (see open arrow). The layer does not penetrate the concrete. A surface crack is present (see closed arrow).*

3.2 Compressive Strength and Elastic Modulus Testing (ASTM C-42, C-469)

In order to quantify the structural integrity of the bridge concrete, compressive strength and elastic modulus testing was performed on concrete from areas showing no visible signs of deterioration. Two 6-inch diameter cores from Pier Cap 39 and two 4-inch diameter cores for Pier Cap 38 were tested in accordance with ASTM C42-90. The compressive strengths for all of the specimens tested were adequate. Elastic modulus testing was conducted in accordance with ASTM C 469 for these specimens. Elastic modulus was also determined from the creep specimens based on the initial deformation after loading, before time dependent creep effects set in. Table 1 shows the measured compressive strengths and elastic moduli for the bridge cores.

It should be noted that the size of the core has an influence on the measured strength (6-inch diameter cores tend to exhibit lower strengths than do 4-inch diameter cores). Six inch diameter cores are considered standard for ASTM C 42, and the strengths of the 4" diameter cores have been corrected in Table 1 to account for a different diameter (Young, 1981). Corrections were also made for length-to-diameter ratio in accordance to ASTM C 42 where appropriate. In addition, the Pier 38 cores were tested in a dry condition, which would be expected to yield much higher strength than cores tested in saturated surface dry condition.

Specimen	Compressive Strength	Elastic Modulus
Pier 38 #1 NB	--	3.04×10^6 psi$^+$
Pier 38 #2 NB	--	2.76×10^6 psi$^+$
Pier 38 #3 NB	--	3.34×10^6 psi$^+$
Pier 38 #4 NB	--	2.69×10^6 psi$^+$
Pier 38 #3 NB (4")	8072 psi*	--
Pier 38 #4 NB (4")	7748 psi*	--
Pier 38 Average	**7910 psi**	**2.96×10^6 psi**
Pier 39 #1 NB	--	2.30×10^6 psi$^+$
Pier 39 #2 NB	5089 psi	2.42×10^6 psi
Pier 39 #3 NB	--	3.01×10^6 psi$^+$
Pier 39 #4 NB	5721 psi	2.66×10^6 psi
Pier 39 Average	**5405 psi**	**2.60×10^6 psi**

* *Tested in dry condition*

-- *Not measured*

$^+$ *From creep testing*

Table 1. *Measured strength and elastic modulus values for pier cap specimens.*

Table 2 shows the strength and elastic modulus values of a reference batch of slag concrete that was made in the laboratory on March 27, 1996. The reference slag batch was made with a 35 S MDOT mix design, using slag from the Levy Pit (#82-22). This is the same slag source as was used in the bridge, though the properties of the slag have likely changed over 30 years. As can be seen, the elastic modulus values are considerably higher for the laboratory batch. This is likely due to using a different and probably denser slag in the laboratory concrete. The complete strength and modulus data is found in Appendix B. Data for the creep specimens is found in Appendix D.

Specimen	Compressive Strength	Elastic Modulus
Reference #1	--	4.60×10^6 psi+
Reference #3	4644 psi	4.74×10^6 psi
Reference #4	4782 psi	4.22×10^6 psi
Reference #5	--	4.60×10^6 psi+
Reference #6	4750 psi	4.01×10^6 psi
Reference #7	--	4.01×10^6 psi
Average	**4725 psi**	**4.36×10^6 psi**

-- Not measured

+ From creep testing

***Table 2.** Elastic moduli of reference specimens from creep testing.*

3.3 Chloride Testing

Chloride concentration testing was performed on samples from the bridge piers in order to determine whether intrusion of salts might have contributed to the distress in Piers 38 and 39. The lack of visible leakage from the bridge deck, and the unlikeliness of significant salt spray from bridge traffic indicated that chlorides were not expected to be major contributors in deterioration. This assessment could only be confirmed through chloride testing however.

Six cores were analyzed for water-soluble chloride concentration:

- Pier 38 southbound (1 core)
- Pier 38 northbound (1 core)
- Pier 39 southbound (2 cores)
- Pier 39 northbound (2 cores)

For each core, samples were taken at depths of 1, 3, 4, and 8 inches from the outside surface of the pier. The 8-inch depth represents the location of the main reinforcing steel; the other depths give an indication of how readily the chloride ions diffuse into the concrete.

A 3/4-inch diameter plug was drilled from the core at the specified depth desired. The plug was dried and ground to pass a 600 μm sieve and the ground material thoroughly blended. A 3-gram sample of each was mixed with water and boiled for 5 minutes. The sample was cooled, filtered and acidified with nitric acid. The sample was spiked with 4 mL of a sodium chloride standard solution. The samples were titrated with a 0.1 N silver nitrate solution using a silver sulfide specific ion electrode. Each run at the autotitrator includes two additional samples, the standard chloride solution and a laboratory control sample. The mL of titrant for the sodium chloride solution is subtracted from the mL for the sample and the chloride is calculated using a locked computer template.

Depth from Surface	Pier 39 #3 NB	Pier 39 #3 SB	Pier 39 #5 NB	Pier 39 #5 SB
1 inch	0.015	0.023	0.018	0.006
3 inches	0.007	0.005	0.003	0.006
4 inches	0.007	0.006	0.007	0.006
8 inches	0.005	0.003	0.003	0.006

Table 3. *Water-soluble chloride contents (% by mass of concrete) in Pier Cap 39 specimens at various depths from concrete surface.*

Depth from Surface	Pier 38 #2 SB	Pier 38 #1 NB
1 inch	0.071	0.070
3 inches	0.072	0.063
4 inches	0.055	0.054
8 inches	0.066	0.042

Table 4. *Water-soluble chloride contents (% by mass of concrete) in Pier Cap 38 specimens at various depths from concrete surface.*

As can be seen from Tables 3 and 4, the chloride concentrations in all of the cores from Pier 39 were very low, consistent with the field observations of no signs of corrosion of the reinforcing steel. The chloride concentrations in the cores from Pier 38 were significantly higher. This concrete may thus be more susceptible to future chloride-related distress. CTL is currently using a corrosion threshold of 0.025 to 0.05% total chlorides by mass of concrete, with water soluble chlorides being 70 to 90% of those values. It is possible that there are some chlorides in the fine aggregate, which would increase the measured value, but would not increase corrosion risk. Nonetheless, there is some reason for concern about the chloride levels in Pier 38.

In order to convert the chloride levels presented in Tables 3 and 4 into units consistent with MDOT's reporting system, the density of the concrete is needed. The density will be assumed here to be 4000 lb/yd^3. Tables 5 and 6 show chloride levels in lbs/yd^3. Chloride data is reported in Appendix C.

Depth from Surface	Pier 39 #3 NB	Pier 39 #3 SB	Pier 39 #5 NB	Pier 39 #5 SB
1 inch	0.61	0.93	0.72	0.24
3 inches	0.29	0.21	0.13	0.24
4 inches	0.29	0.24	0.29	0.24
8 inches	0.21	0.13	0.13	0.24

Table 5. *Chloride contents (lbs/yd^3) in Pier Cap 39 specimens at various depths from concrete surface.*

Depth from Surface	Pier 38 #2 SF	Pier 38 #1 NF
1 inch	2.85	2.80
3 inches	2.88	2.53
4 inches	2.21	2.16
8 inches	2.64	1.68

Table 6. *Chloride contents (lbs/yd^3) in Pier Cap 38 specimens at various depths from concrete surface.*

3.4 Crack Depth

The depth of cracking is important in determining an appropriate repair method. For this reason, the depth of cracking in the cracked cores taken from the bridge is described in Table 7.

Core	Depth of Cracking
Pier 38, NF, NB #1	Cracked full length (7-1/2")
Pier 38, NF, NB #2	Cracked full-length (11") Longitudinal and transverse cracks
Pier 38, NF, NB #5	Microcracking in outer 3" only
Pier 38, SF, SB #2	Cracked full length (9")
Pier 38, SF, SB #3	Longitudinal and transverse cracking
Pier 39, SF, NB #1	Longitudinal and transverse cracking
Pier 39, SF, NB #1C	5" Longitudinal and transverse cracking
Pier 39, SF, NB #2	Full-length (12") longitudinal crack
Pier 39, SF, NB #3	Full-length (11") crack

Table 7. *Depth of Cracking in the cracked cores taken from the bridge.*

3.5 Creep Testing (ASTM C-512)

Creep testing was conducted on specimens from Pier Caps 38 and 39 as well as on a reference batch made in the laboratory with slag aggregate. The specimens were loaded to roughly 25% of ultimate strength and tested for creep over a period of 6 months. The creep testing was conducted at constant temperature and humidity (21°C and 50% RH) in a controlled environmental chamber. Testing was conducted in accordance with ASTM C-512. The creep testing yielded deformation from initial loading, long-term loading, and long-term drying shrinkage. Initial deformation is used in calculating the elastic modulus, as described in section 3.2. Unloaded specimens subjected to the same environmental condition give the drying shrinkage. The loaded specimens yield a combination of drying shrinkage and creep deformation. The drying shrinkage is then subtracted out to give creep.

In order to compare different creep tests, they must be normalized by dividing each by the stress at which it was tested. This normalized creep is called specific creep. As can be seen from Tables 8 and 9, the creep properties of bridge specimens are very similar to the creep properties of laboratory made specimens. This indicates that the ASR in the fine aggregate has likely had little effect on the long-term deformation under loading of the bridge concrete. Based on this finding, the post-tensioning that is proposed as a repair method can be expected to act in a manner that is similar to the way it would with a new slag concrete. There is some difference between the two pier caps, though overall, the trends are very similar. Furthermore, the values of creep and shrinkage in these specimens is well within the expected range given by ACI 209 and the AASHTO Bridge Code.

	Shrinkage		Creep	
Specimen	**@ 218 days**	**@ Ultimate**	**@ 218 days**	**@ Ultimate**
Pier 38 Average	355×10^{-6}	413×10^{-6}	733×10^{-6}	1022×10^{-6}
Pier 39 Average	460×10^{-6}	535×10^{-6}	867×10^{-6}	1208×10^{-6}
Reference Batch Avg.	265×10^{-6}*	325×10^{-6}	663×10^{-6}*	985×10^{-6}

** At 155 days*

***Table 8.** Average creep and shrinkage values for the pier caps and the reference batch (in/in).*

	Specific Creep	
Specimen	**@ 218 days**	**@ Ultimate**
Pier 38 Average	0.518×10^{-6}	0.722×10^{-6}
Pier 39 Average	0.614×10^{-6}	0.854×10^{-6}
Reference Batch Avg.	0.535×10^{-6}*	0.796×10^{-6}

** At 155 days*

***Table 9.** Average specific creep, normalized to account for differences in applied stresses during testing (in/in/psi).*

3.6 Potential for Future Expansion Testing

In order to determine the potential for future expansion from ASR reactivity, testing was conducted by CTL in which concrete specimens from the bridge were subjected to varying environmental conditions. It should be noted that this test was only performed on one set of three specimens. As this is a non-standard test method, it is therefore not known how many tests are required to produce reliable results. In addition, this test is typically run for 12 months before final conclusions are drawn. This testing should thus not be considered alone, rather it is included because it supports the findings of the petrographic evaluations.

Three cores (cores #4, 5, and 6) were taken from adjacent locations in Pier 39 Northbound. It is important to locate cores for this test so that they represent the same concrete. Core #4 was immersed in water at 100°F to determine expansion due to uptake of moisture. This is an intrinsic property of partially dried concrete and occurs regardless of whether potentially reactive aggregate is present. Such expansion normally ceases when the core essentially reaches mass equilibrium, and is used as a baseline above which expansion due to ASR may be calculated.

Core #5 was immersed in one normal sodium hydroxide (1N NaOH) solution at 100°F to determine whether potentially reactive aggregate is present in the concrete. If so, expansive reaction will be reflected in delayed, long-term expansion after the core has reached mass equilibrium. This reveals only whether reactive aggregate is present, not whether alkali is present in sufficient concentration in pore solutions at the time of coring to sustain expansive ASR. As is well-known, available alkali becomes depleted as ASR progresses, thereby limiting the potential for expansion due to the reaction. The concentration used in this test is typically greater than would originally exist in concrete. Thus, the exposure forces the ASR reaction, and provides a conservative indication of susceptibility of the aggregate to expansive ASR.

Core #6 was stored at 100°F over water in a sealed container. This exposure determines whether sufficient alkali is still available to produce expansive ASR. Collectively, the three exposures characterize the potential for expansive ASR in the future.

The expansion criterion for ASR presence used for cores stored in NaOH solution or over water is 0.030 percentage points greater than that developed after mass equilibrium is reached by the core stored in water. This criterion was selected on the basis of microcrack development associated with ASR, as observed microscopically in concrete. Experience has indicated that this expansion differential will be reached, if at all, within 9 to 15 months after testing is initiated, depending on available alkali and type of reactive aggregate. An endpoint for the test period is indicated by lack of sustained expansion compared with that for the core immersed in water, or by expansion differentials exceeding 0.030 percentage points, even if no further expansion develops.

Processing for these tests consisted of selecting appropriate core sections, sawing each end normal to the length of the core, and cementing in gage points at the center of each end for comparator readings. These readings were taken, together with mass readings, at 7, 14, and 28 days and at 2, 3, 4, 5, and 6 months. All readings were made with cores in the dampened condition. Future readings are planned at ages of 9 and 12 months, and will be submitted at a subsequent date.

Interim test results, at 6 months, are summarized in Table 10 and in the following text. Table 10 indicates changes in mass between successive weighings during storage in water or NaOH solution, or over water. It is seen that major increases occurred during the first two months for cores stored in all storage conditions. Mass gains are currently at or near mass equilibrium, where mass changes are within ±2 grams. The significance of these periods of major uptake of moisture or solution define a "zero-point" baseline from which to calculate expansions that develop due solely to continued ASR during the test period.

Test Age	In Water		In 1N NaOH		Over Water	
	% Length Change	Mass Change (g)	% Length Change	Mass Change (g)	% Length Change	Mass Change (g)
7 days	0.012	18	0.008	17	0.007	-2
14 days	0.021	7	0.011	8	0.019	7
28 days	0.021	6	0.019	8	0.022	12
2 mos.	0.021	5	0.046	11	0.028	9
3 mos.	0.021	3	0.073	4	0.032	2
4 mos.	0.023	2	0.085	3	0.035	1
5 mos.	0.026	0	0.133	4	0.040	2
6 mos.	0.026	3	0.156	4	0.045	3

Table 10. *Core Expansion, %, and Mass Change, g.*

Table 10 also summarizes the expansion data for all cores. Data are referenced to the initial reading made prior to introduction of the core into the test environment. Expansions reported here include those due primarily to uptake of water or NaOH solution, and therefore do not isolate those due solely to ASR during the test period.

After mass equilibrium is reached in all storage conditions, core expansions will be compared to determine the potential for future expansion relevant to each storage condition. Expansion differentials exceeding 0.030 percentage points indicate the potential for future expansion relevant to the storage condition in which the expansion measurement occurred.

As discussed above, the cores are nearing, but have not yet reached mass equilibrium. Based on the 5 month expansion of the pier cap cores, the following interim conclusions are drawn:

1. The cores are nearing equilibrium with their storage environment, based on their uptake of moisture. *Experience dictates that all conclusions at this time are speculative.*
2. Testing should continue for at least the originally specified 12-month period.
3. Potentially reactive aggregate appears to be available for further reaction if sufficient alkali is present.
4. There appears to be small potential for further expansion in the concrete pier cap provided that adequate moisture is available.

It should be noted that it is difficult to assess on a percentage basis how much of the reaction has already occurred, as this is heavily dependent on availability of the reactants and water. It is also not known precisely what the original amounts of reactive aggregates and alkalis were in the concrete. For this reason, only the potential for future expansion is discussed.

4. Historical Records

A significant amount of information can be gleaned from historical records regarding the construction of the bridge. In particular, MDOT's construction records and weather records for the time of placement of the pier caps have been investigated in this study. The types of information that are sought are clues to the causes of major cracking which might be associated with construction conditions. This would further corroborate the previous findings that major cracks were caused by thermal/shrinkage related problems due to large sections and hot weather, and that microcracks occurred due to ASR in the fine aggregate.

4.1 1977 MDOT Internal Memo

When it was found through petrography that two types of cracking had occurred, MDOT located and provided the project team with a copy of a historical internal memo regarding the distress development in the bridge pier caps. This memo was written in response to a 1977 Detroit News article which identified cracking in the bridge pier caps.

In the memo, several conclusions are drawn regarding the cause and severity of cracking. It was found through inspection at that time that the cracking in the bridge piers was not related to shear or tensile stresses from applied loads. Furthermore, the bridge piers were considerably overdesigned, and geometry and aesthetics had governed pier design, not stresses.

The causes of cracking were thought to be shrinkage and/or thermal effects related to hot-weather placement of the massive pier caps. This hypothesis, the memo said, was supported by the fact that high cement content mixes (7.5 sacks per cubic yard) were used "to obtain 70% of the design strength earlier". In addition, the pier caps were placed in hot weather from May 7 to July 29, 1965. These two factors combined would have greatly increased the thermal buildup and resultant surface cracking of the massive 7 foot thick piers. All other pier cap girder placements (other than those for Pier Caps 37 to 40) used the normal 6 sacks of cement per cubic yard of concrete.

The memo went on to say that such cracking typically will not deteriorate significantly for 20 to 30 years. If the concrete is not protected, deterioration such as spalling due to freeze-thaw damage may occur, at which time one would proceed with repairs. The memo, transcribed from microfilm is presented in Appendix F.

4.2 MDOT Construction Records and Weather Records

A review of MDOT"s construction records for the bridge Piers 37 to 40 indicates that the mix design and weather information presented in the 1977 internal memo are correct. Table 11 summarizes the temperature and concrete mix information in the MDOT records that are pertinent to the observed distresses. Table 12 gives a summary of the mix design information for the pier cap concrete. Copies of the construction records and temperature records are found in Appendix F.

Pier	Placing Date	Air Temp. (Deg F)*	Concrete Temp. (Deg F)	Slump (in.)	Air Content (%)	Sacks of Cement	MOR+ at 28 days (psi)
37 NB	5/14/65	44-73	72-76	2.75-3.5	5.8-8.5	7.5	796
37 SB	5/7/65	57-81	78	3.0-3.5	5.5-7.3	7.5	698
38 NB	5/21/65	45-81	72-78	3.0-4.0	6.3-7.1	7.5	836
38 SB	6/4/65	48-69	68-70	3.0-3.25	6.0-7.3	7.5	772
39 NB	7/10/65	60-78	80-81	1.5-3.0	7.5-9.0	7.5	906
39 SB	7/29/65	57-71	74	3.5	6.5-7.0	7.5	934
40 NB	6/24/65	55-73	80-82	3.5-5.5	6.5-9.3	7.5	900
40 SB	7/2/65	60-79	76	3.5	6.0	7.5	865

**From Climatological Data Detroit, 1965, US Dept. of Commerce, Weather Bureau*

+MOR = Modulus of Rupture

***Table 11.** Temperature and mix information from construction record*

Pier	Conc. Grade	Cement		Fine Aggregate		Coarse Aggregate		Design Water (lbs)	Admixture	
		Type	Amt (lbs)	Type	Amt (lbs)	Type	Amt (lbs)		Type	Amt (oz)
37 NB	A (6AA)	Peerless II	705	Am. Agg. (47-3)	1158	Levy (Trenton)	1437	312.4	Darex AE	18.8
37 SB	A (6AA)	Huron II	705	Am. Agg. (47-3)	1204	Levy (Trenton)	1469	259.2	Darex AE	16.9
38 NB	A (6AA)	Peerless II	705	Am. Agg. (47-3)	1281	Levy (Trenton)	1436	302.8	Darex AE	19.7
38 SB	A (6AA)	Peerless II	705	Am. Agg. (47-3)	1244	Levy (Trenton)	1461	277.1	Darex AE	19.7
39 NB	A (6AA)	Peerless II	705	Am. Agg. (47-3)	1238	Levy (Trenton)	1465	278.9	Darex AE	19.7
39 SB	A (6AA)	Peerless II	705	Am. Agg. (47-3)	1234	Levy (Trenton)	1426	322.7	Darex AE	19.7
40 NB	A (6AA)	Peerless II	705	Am. Agg. (47-3)	1225	Levy (Trenton)	1413	343.6	Darex AE	19.7
40 SB	A (6AA)	Peerless II	705	Am. Agg. (47-3)	1243	Levy (Trenton)	1424	314.6	Darex AE	19.7

***Table 12.** Mix design data for Pier Caps 37 to 40.*

4.3 1965 Petrographic Report of American Aggregate Pit (#47-3)

A petrographic analysis of the American Aggregate Green Oak Plant, Pit No. 47-3, was conducted on March 15, 1965. This pit was also the source of fine aggregate for the bridge concrete. The analysis found that the chert content was 3.5% based on particle count. This is above the threshold for reactive aggregates as identified by Lane in the Literature Review (see Appendix G). Furthermore, there is a high content of quartzite (18%), which is identified in the report to be fine grained microcrystalline textured. Depending on the exact make-up of these particles, they may be reactive as well, as identified in the Literature Review. An earlier report from 1964 indicated a 2.1% chert content, but again a high microcrystalline quartzite content. The petrographic reports are found in Appendix F.

5. Summary of Causes of Distress and Concrete Properties in Pier Caps 38 and 39

Causes of the distresses observed in Pier Caps 38 and 39 are evaluated based on field and laboratory investigations, and analysis of historical records. The following summarizes the distresses in these pier caps:

- The major cracking observed in the pier caps occurred early in the life of the bridge, likely from thermal and/or shrinkage effects associated with massive sections (65' x 7' x 9 to 12'), high cement content (7.5 sacks/yd^3 versus the standard 6 sacks/yd^3) and summer placing conditions.
- Additional micro-cracking was caused by ASR expansion from reactive chert particles in the fine aggregate.
- A portion of the total entrained air voids are filled with deposits from the ASR reaction. Total air content is now less than would be recommended for adequate freeze-thaw resistance, and is estimated to range from 3 to 7% (with most cores being in the 5 to 7% range).
- The ASR appears dormant, as long as additional water and alkalis are not supplied to the reactive aggregates (preliminary based on 6 month test results).

In addition to these findings, the following was noted relating to the properties of the concrete in the pier caps:

- Concrete compressive strength is found to be well above required levels, with measured values above 5000 psi., and does not appear to have been adversely affected by ASR.
- The creep properties of the pier cap concrete are not significantly different from those of a trial batch containing slag aggregate cast in the laboratory which has not been influenced by ASR.
- Chloride levels have not been found to be high in these pier caps, indicating that corrosion is likely not an issue.
- Concrete elastic modulus in the bridge pier caps (average 2.78x10^6 psi for all specimens) is somewhat lower than is typical for regular concrete, but is not unexpected in the slag concrete (due to slag's high porosity and low bulk specific gravity of 2.31). This would

lead to an expected increase in deformation due to loading, though no adverse effects have been noted on the bridge.

6. Repair Recommendations

In determining the repair methods that are most suitable to this bridge, a number of methods are evaluated. The types of repairs chosen must address two needs: (1) preventing future deterioration, and (2) strengthening the structure to bring it up to current code requirements. Prevention of future deterioration can include epoxy injection of the cracks, possibly in conjunction with sealing of the surface, or removal and replacement of the damaged concrete. Upgrading the load capacity can be attained through post-tensioning and/or jacketing to increase the area of the section.

The repair methods originally proposed for this bridge include a combination of several of these methods. The pier caps were to be patched and epoxy injected. Additional reinforcement was to be added and encased under additional concrete cover. The new concrete was to be surface coated, and the structure post-tensioned. Based on the investigations of this study, this approach is considered appropriate.

In particular, future deterioration of the concrete can be avoided by preventing water and alkalis from entering the concrete. The threefold approach of epoxy injecting cracks, adding additional concrete cover, and surface sealing all exterior portions of the pier caps can provide a lasting deterrent for water and alkalis.

The pier cap will be strengthened by epoxy injection, addition of reinforcement, and post-tensioning. The post-tensioning will provide a compressive force along the length of the pier cap, and will be anchored through its 7 ft thickness. The prestressing creates a new stress distribution in the pier cap. This will put the cap in compression in both the longitudinal and transverse directions, but leaves it susceptible to damage from the prestressing in the vertical direction. For this reason, the additional reinforcement is needed to hold the cap together. Furthermore, the stirrups provide the needed additional shear capacity required to meet current design criteria. The added concrete cover simply protects the new reinforcement from the environment.

The following sections discuss each of the repair methods.

6.1 Crack injection

The cracks in the concrete pier caps can be repaired by either of two methods, depending on the type of cracking. Deep penetrating cracks are generally injected with an epoxy compound, while concrete with shallow cracks and delaminations is typically chipped away and replaced.

The injection method would generally be used with the deep penetrating cracks seen in the pier caps of this bridge. The injection has a dual purpose of restoring bond between the cracked concrete surfaces and preventing future ingress of water and salts (which contain chlorides and alkalis) into the cracks. The injection material is typically stronger than the concrete into which it is injected, and if properly injected, restores the concrete's load carrying capacity. The sealing of the cracks removes a major source of potential deicing salt intrusion into the concrete. The blockage of these salts (which contain alkalis in the form of sodium) from cracks is important, as any unreacted alkali-reactive aggregate particles still present are considered unlikely to react without the availability of additional alkalis.

A survey of drydocks owned by the US Navy was reported by Burke and Detwiler (1985). They found that cracks could be successfully repaired by pressure injected epoxy provided proper procedures were followed. Crack injection requires considerable skill on the part of the operator. For the injection to restore the concrete bond it must penetrate deep into the cracks. Thus strict quality control measures for inspection of epoxy injection, and stringent qualification guidelines for crack injection operators are necessary. Furthermore, the distance between injection ports should be specified in the contract or by the injection contractor to be submitted for engineer approval, to ensure effective injection technique.

Epoxy Injection Materials

Epoxy injection is effective to 9 feet into cracks down to 0.002 inches wide (Murray, 1987). The small hairline cracks, below 0.002 inches in width (about 1/2 the thickness of a human hair) will likely be closed when post-tensioning is applied, and can be bridged by a suitable surface sealer.

Using the proper viscosity injection material is vital to effective repair. The grade and class of epoxy to be used are described in ASTM C-881, and are dependent on crack size and placing temperature. Specifying too low a viscosity, especially in larger cracks, can result in a loss of injection fluid, as it can be very difficult to contain. A viscosity that is too high will make it impossible for the epoxy to penetrate the crack. Very fine cracks (less than 0.010 inch wide) should be injected with an epoxy of 500 cps (centipoises) or less. That is roughly the consistency of a light-weight oil. Higher viscosity epoxy is recommended for larger cracks (Murray, 1987).

In addition to viscosity, other important selection criteria for the epoxy include pot life, minimum curing temperature, insensitivity to moisture present in the crack, and ability to deform under load. (Murray, 1987). These properties can typically be adjusted by the manufacturer to meet project requirements.

Epoxy Injection Procedure

A six step procedure is followed in epoxy injection.

(1) The concrete surface around the crack must be cleaned, so that a sealant may bond to it effectively. The crack is flushed with high pressure water to remove loose debris and

contaminants. Acid preparation is not recommended, as it is vital to remove all of the acid to avoid future damage. Due to the difficult working conditions, it may be difficult to ensure complete acid removal, and more harm than good may result. The water flushing is followed by using compressed air to blow out the water and dry the concrete. Allowing adequate time for drying is important, as free water on the crack surfaces can interfere with the bonding capacity of the epoxy. Most epoxies will bond well to moist concrete, but may be inhibited by free water on surface.

(2) The surface of the crack is sealed with an epoxy or polyester sealer to prevent the liquid epoxy that will be injected from seeping out of the crack before it hardens. If very high injection pressures are needed, special procedures are required for bonding the sealer to the concrete surface.

(3) The entry ports for the epoxy are installed next, using one of several methods, fittings inserted into drilled holes, bonded flush fittings, or interruptions left in the sealing material. Port spacing should be such that a desired depth is penetrated before the epoxy flows out of an adjacent port. Typically, though, ports are not spaced more than 12 inches apart, For cracks less than 0.010 inch in width, ports should be spaced at a maximum of 4 to 6 inches apart.

(4) The epoxy may be mixed using one of two methods, pre-mixing or in-line mixing. Pre-mixing is done using a mechanical stirrer, and is conducted in accordance with manufacturer instructions. In-line mixing requires a special nozzle that mixes the epoxy components during pumping.

(5) The cracks are injected through the ports in a systematic manner. Vertical cracks may be filled using one of two methods: 1) filling the bottom of the crack first and moving upward, or 2) filling the broadest part of the crack first and moving to the finer areas next. Horizontal cracks are filled in a similar manner, typically starting at one end and working across the crack. The crack is filled when injection pressure can be maintained. If there is any seepage of epoxy from the crack, or draining in a vertical crack, any voids must be re-injected.

(6) After the epoxy has cured, the entry ports are removed and plugged. If aesthetics are an issue, the surface seal is removed (Murray, 1987; Trout, 1989).

There are several quality control measures for crack injection. First, the epoxy should be inspected for proper mixing. Many epoxies attain a certain color when properly mixed. Second, the impregnation of the cracks can be monitored from adjacent injection ports during placement. Finally, destructive and non-destructive testing can determine the effectiveness of injection and the presence of voids (Murray, 1987). At least a few cores should be taken to ensure adequate depth of penetration of the epoxy.

6.2 <u>Surface Sealing</u>

In addition to crack injection, sealing all surfaces of the pier caps will serve as added protection against the ingress of water and chlorides. There are many proprietary products on the market that are sold as concrete surface sealers. When choosing the specific sealer appropriate for this bridge, the following criteria should be met. The surface sealing material should be waterproof to prevent the ingress of liquid water and salts. The sealant should be able to bridge hairline cracks and maintain that seal while allowing movement of these cracks. Furthermore, the sealant ***must be breathable***, allowing water vapor to escape. This will prevent it from spalling off due to the buildup of water vapor pressure behind the sealant. The sealant should also contain no alkalis, and should be permanent when cured. Finally, the sealer should be resistant to ultraviolet light. Many sealers deteriorate under prolonged exposure (Kubanick, 1990).

The sealant industry is seeking ways to reduce the toxicity and content of volatile organic compounds (VOC's) in its sealers, and some such sealers are now on the market. To avoid VOC's and toxic formulations, it is recommended to use water-borne, high-solids, or 100% solids coatings (Kubanick, 1990).

Several suitable types of sealers and coatings are available, including monomers, polymers, epoxies, and acrylic rubbers. Individual manufacturers should be consulted regarding the properties of their specific products, based on the recommended characteristics described above. It should be noted that lithium based compounds are not recommended for this repair, as the use of lithium compounds is as yet an experimental technology. Particularly for massive structures such as these pier caps, the depth of penetration of the compound cannot be guaranteed. Furthermore, the rate of diffusion may be too slow to be effective.

Surface Preparation

Before applying a surface sealer, the concrete surface must be prepared. It should be noted that there are numerous proprietary surface sealing systems with different surface requirements. Thus, manufacturer instructions should be followed. The following, though, are the general considerations for surface preparation.

(1) Surface uniformity is needed for many sealers to be effective. For some products, protrusions, holes, and cracks should be removed or filled prior to sealing. Typically decorative coatings have more stringent requirements in this area.

(2) The surface should be clean of all foreign matter which could act as debonding agents, including dust, oils, curing compounds and the like. All unsound or crumbling concrete should also be removed.

(3) While the required surface moisture condition varies for different sealing compounds, typically free surface water should be avoided. A saturated surface dry or dryer condition is often required for the sealer to bond effectively.

(4) Laitance (layer of high water-cement ratio gel which often comes to the surface during placement) may need to be removed. Laitance can usually be removed by brushing vigorously with a stiff broom or wire brush.

(5) The concrete surface should be tested for sufficient strength to resist any shrinkage of the sealer as it cures.

If any doubt remains as to the adequacy of the cleaning method, a small test patch should be sealed to determine whether proper adhesion has been obtained. This test area should be placed under the same moisture and temperature conditions as the actual application. Some manufacturers recommend specific test methods for their products, though unfortunately no standard test method exists (Gaul, 1981). In addition to these recommendations, excellent guidelines for preparing concrete surfaces for sealing have been published by several trade associations including the American Concrete Institute (ACI), The National Association of Corrosion Engineers (NACE), and the Steel Structures Painting Council (SSPC). Several ASTM standards outline standard procedures for preparing concrete and masonry for coating, including ASTM D-4258 through D-4263 (Kubanick, 1990).

6.3 Chipping and Replacing of Small Areas of Damaged Concrete

No surface spalling or delaminations were noted in the pier caps during the field evaluation, and no such delamination cracking was seen in the cores taken from the bridge. However, some cores were cracked in the transverse direction (roughly perpendicular to the axis of the core). Where such delaminations are encountered, the damaged concrete should be removed with a small chipping hammer until intact material is exposed. The chipped surface should be cleaned thoroughly to remove any loose material and debris. The chipped area should then be repaired with cast-in-place concrete for larger sections. Shotcrete should not be used in this application due to the difficult accessibility of the pier caps, high operator dependency, and difficulty in quality control.

Chipping and Surface Preparation

Special caution should be observed when chipping to avoid confusing the relatively "soft" slag aggregate with damaged concrete. During testing of the cores in the laboratory it was noted that the slag concrete was very easy to cut. A small pneumatic chipping hammer (30 lbs or less) should be used to avoid removing excessive amounts of intact material or damaging reinforcing steel. A 15 lb hammer is light enough to use on vertical and overhead surfaces, and is thus recommended for this project. Electric and hydraulic hammers may also be used (Emmons, 1993).

Choosing the proper jack-hammer tool can also speed repairs and improve repair quality. The most commonly used tool is the standard moil, which is used for breaking up the concrete. For soft concrete, as is encountered in this bridge, a 3-inch chisel may be more efficient. In order to roughen the surface of the intact repair face, a brushing tool may be specified. Roughening the surface of the existing concrete will facilitate a good bond of the repair material (Aberdeen's, 1989).

Once the damaged material has been removed, and the surface roughened, the substrate should be reinspected to ensure no damaged areas remain. Next the pore structure should be opened using shotblasting, hydroblasting, or vacuuming. An open pore structure will provide capillary suction of the repair material, and facilitate a strong bond. After blasting, any debris should be removed. Finally, the moisture level of the repair surface can influence the success of the repair. A dry substrate may absorb too much water from the repair material, while excess water in the substrate may clog pores and reduce bonding. Typically, a saturated surface dry condition is considered to be a good solution (Emmons, 1993).

Replacement of the Concrete

The important consideration with regard to the repair concrete is that it be compatible with the existing concrete. In particular, it should respond to loading and temperature changes to the same degree as the existing concrete to avoid delamination of the repair (Vaysburd, 1996). Furthermore, shrinkage should be considered, as drying shrinkage can cause delamination of the repair. Shrinkage can be influenced by the cement content in the mix; a high cement content leads to high shrinkage. In addition, the repair concrete must be properly air entrained to ensure frost durability (Emmons, 1993).

6.4 Post Tensioning

From a material durability standpoint, post-tensioning is expected to provide little benefit in preventing future ASR reaction, as tri-axial post tensioning would be required to adequately control reactivity. Tri-axial post-tensioning is not feasible due to space constraints in the vertical direction. The limited laboratory data from this study indicates that ASR is not ongoing and will not progress provided that additional alkalis are not allowed to penetrate into the concrete. For this reason, it is recommended that crack injection, new concrete cover, and surface sealing be used to prevent future deterioration. These methods are expected to provide sufficient protection to the concrete, as future deterioration is not expected without the ingress of additional salts.

From a structural point of view, bi-axial post-tensioning (along the pier cap length and through the thickness) and/or increasing the concrete section area can be effective for increasing the load carrying capacity to meet current code requirements (Vejvoda, 1992; Nilsson, 1996). If post-tensioning is chosen, it should be applied externally, using either threaded rods or prestressing strand. External post-tensioning will help to reduce cost, avoid interfering with the existing reinforcing components in the pier caps, and facilitate future inspection.

Two techniques are common for attaching the external tendons. (1) They may be anchored in bearing plates that are anchored to the ends of the member, or (2) they may be clamped to pre-loaded end bolts that pass through the member (Manning, 1988). If increased flexural resistance is needed, the tendons may be deflected at the midspan using a saddle clamp. Due to the confined space at the center of the bridge, where the north-bound and south-bound pier caps come together, it is more feasible to use the second approach in this application. Furthermore, the second approach allows for pre-tensioning of the members passing through the pier cap, providing bi-axial compression. This approach should be used in conjunction with additional reinforcement in this bridge to prevent damage to the pier cap in the vertical direction due to the post-tensioning.

In order to avoid eccentricity during repair construction, the prestressing tendons should be added and tensioned in pairs simultaneously, one on either side of the pier cap. Using this approach will also ensure proper alignment of the bearing plates or end bolts. Furthermore, the final tensioning of the prestressing strands should be done sequentially to avoid eccentric loading.

Protecting the prestressing strands from the relatively aggressive environment may be considered. This protection typically includes covering all tendons from end to end in a waterproof enclosure, and filling the space around the tendons with a corrosion resistant grease. The alternative approach is to leave the tendons uncovered, allowing for condition monitoring. This will allow easier determination of future corrosion damage, but will not afford any protection to the strands (Freyermuth, 1991). Due to the relative inaccessibility of the pier caps for future monitoring, the approach of taking measures to protect against the environment is considered prudent.

When post-tensioning to repair the pier cap, the same procedures and equipment will be used that are used in conventional pre-stressing/post-tensioning projects. The repairs will be governed by ACI 318 "Building Code Requirements for Reinforced Concrete" sections 18.4 and 18.5 (Greve, 1987), and the AASHTO Bridge Design Code.

6.5 Jacketing and Increasing Section Area

Increasing the section by casting additional concrete around the pier caps may also yield the desired strengthening of the pier caps, but may also be the most difficult repair method to perform economically. This method would involve one of two approaches; (1) chipping the existing concrete surface to expose existing reinforcement and create a rough bonding surface for the repair concrete to bond to, or (2) adding a concrete shell and transferring load to the shell by post-tensioning the shell to and through the existing cap.

In the first method, additional reinforcing steel would be tied into the existing steel, so as to facilitate transfer of loads. Additional reinforcement would be such that current design codes are met. The chipped concrete would be cleaned of debris prior to the placement of the new concrete as described in section 6.3 above. Epoxy injection of the cracks in the existing structure prior to increasing the section should be performed, so as to restore the capacity of that concrete as well. The additional concrete should be compatible with the existing concrete in strength and deflection under loading to avoid delamination of the repair. The surface of the new concrete should be treated with a surface sealer to prevent future ingress of contaminants.

A potential pitfall in selecting the repair concrete mix design is to mandate high strength and/or low permeability with the mistaken belief that high-quality concrete is paramount. However, it is essential to successful repairs that the old and new concrete work together; otherwise the purpose of adding to the section thickness is defeated. If the repair material thus chosen has a higher permeability than would be desirable for durability, the section can be made thicker for added cover.

It is essential to provide a positive connection between the repair concrete and the substrate with steel ties or dowels to ensure that they work together in resisting applied loads; the bond between them is not adequate by itself.

The second method, using post tensioning to hold the shell in place would be less dependent on material compatibility. The success of this type of repair lies in the design of the post-tensioning, which will transfer the vertical loads on the pier to horizontal loads in the jacket. The jacket's ability to resist this type of loading must be assessed under current conditions, and under the possibility of further loss of capacity of the existing concrete (Pierce, 1996). In this approach, the shell is not directly tied to the substrate material, nor does it have to be bonded to it. Rather, the shell is used to hold the substrate in place and provide additional capacity.

Jacketing with other materials such as steel, reinforced plastics, and rubber is also common. This approach is typically used to provide needed durability to the structure, and can also be used effectively to increase structural capacity. In addition to choosing an appropriate material, the type of anchorage to be used (adhesive, anchorage bolts, wrap-around stays) must be determined. Since jacketing is expensive, it is often not used unless it is absolutely necessary.

In this repair, a jacketing system as such is not recommended. The additional concrete cover that will be provided has the purpose of providing cover to the new reinforcing steel, and will not add significant structural benefits. At the same time, the first approach to jacketing, which considers the need for a good bond to be established between the existing and new concrete, should be followed with this repair.

7. <u>References</u>

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