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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
Research Report No. RC-1346

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By

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Disclaimer

Table of Contents

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.

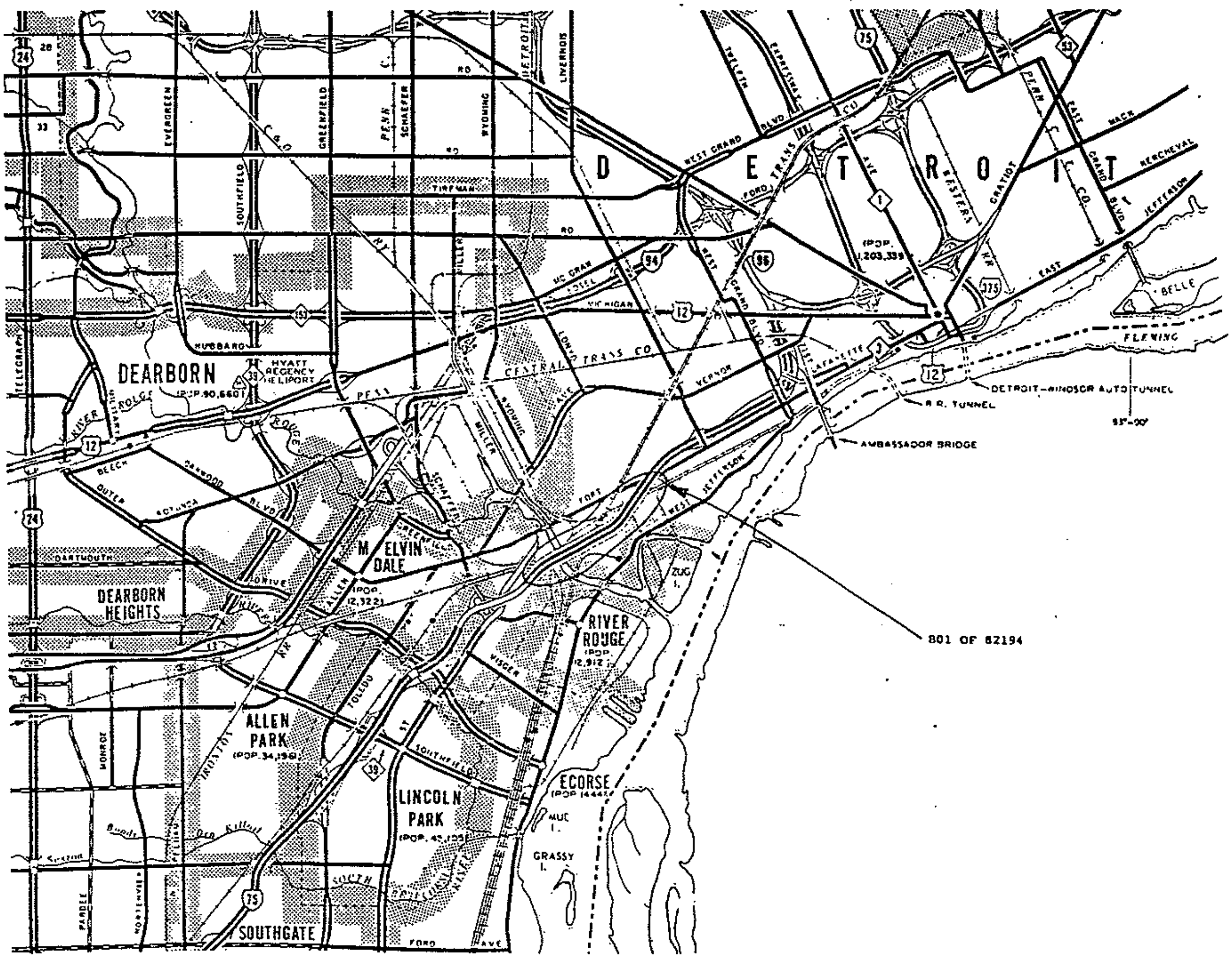


Figure 1. General location of bridge site (from Parsons Brinkerhoff, Inc. project plans).

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.

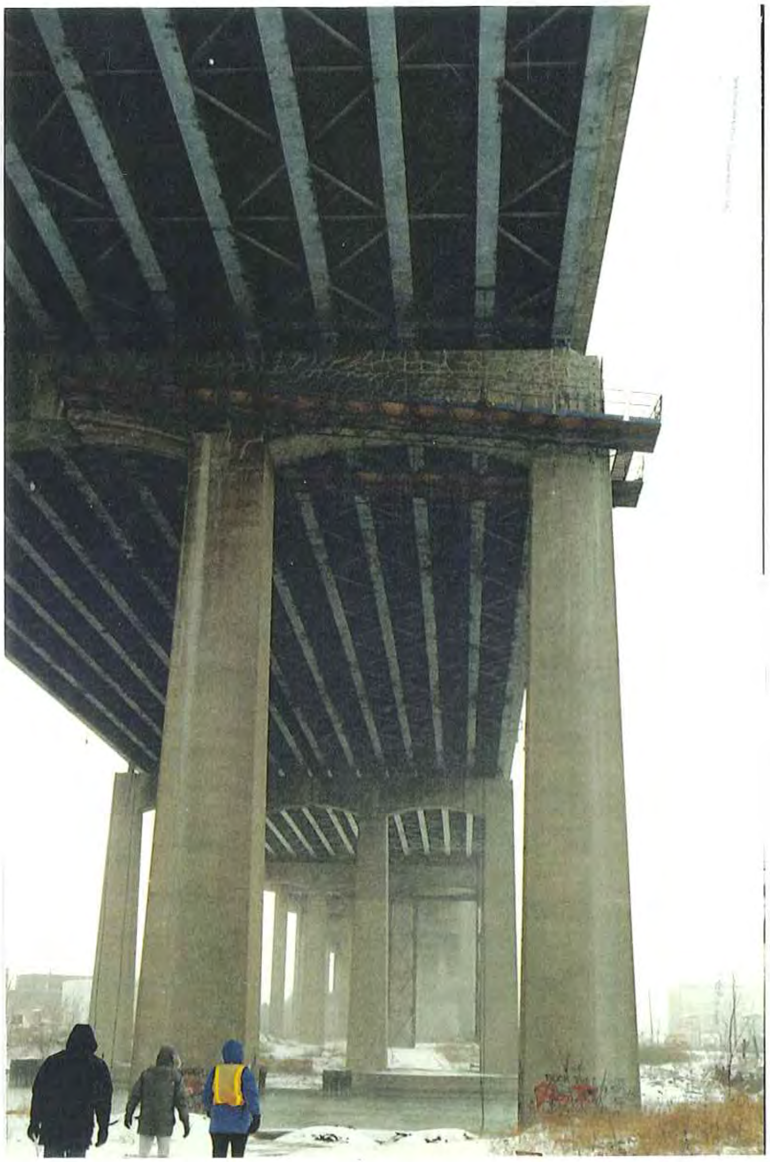


Figure 2. *Overview of the main span of the bridge looking south, showing epoxy injected Pier Cap 39 in the foreground, and Piers 38 and 37 in the background.*

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.*

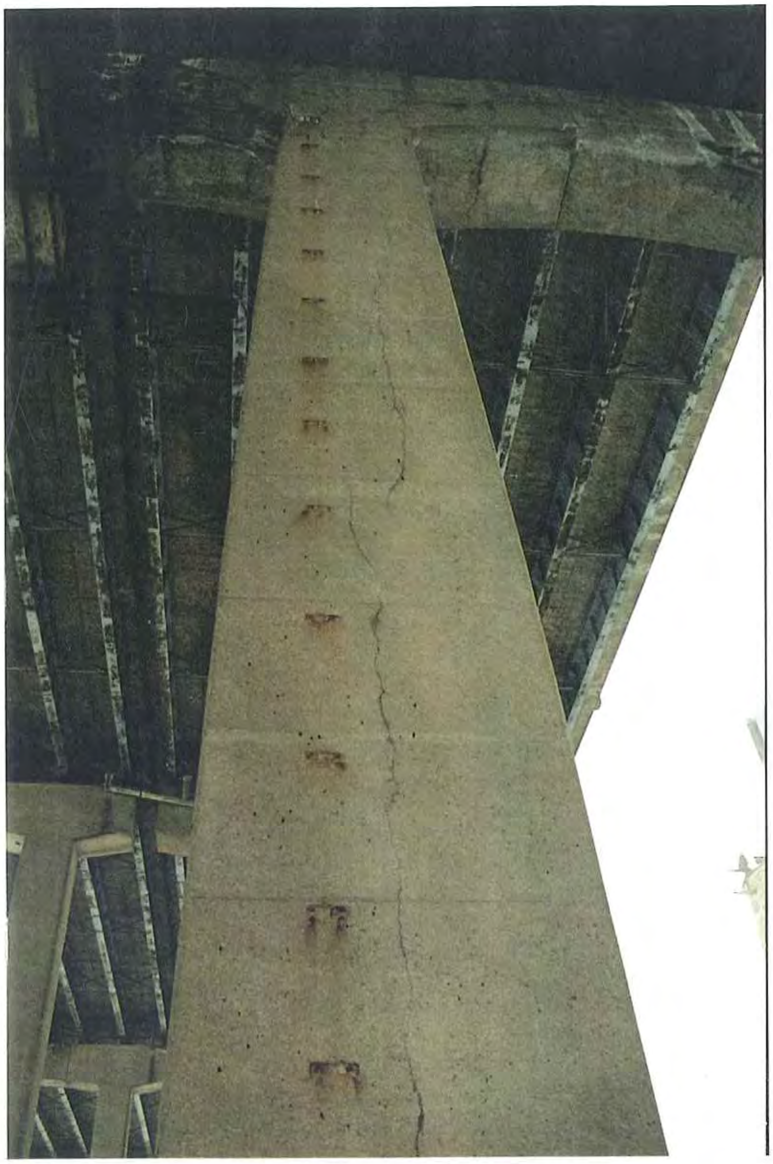


Figure 5. *Vertical cracks approximately centered in the width of the column were typical of the crack pattern in the columns. This column supports Pier 40 north-bound.*

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.

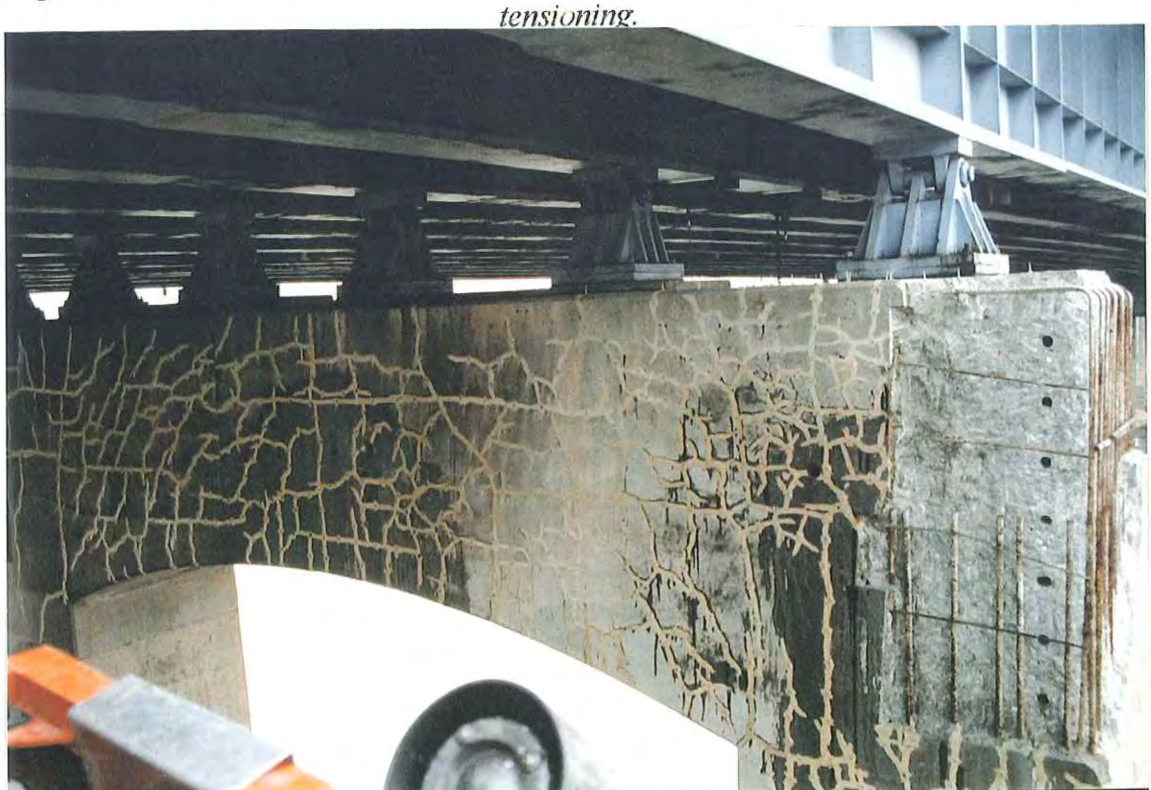


Figure 9. North face of Pier 39 south-bound. The locations of Cores 3, 2, and 1 (left to right) are shown, along with a closer view of the area prepared for post-tensioning.

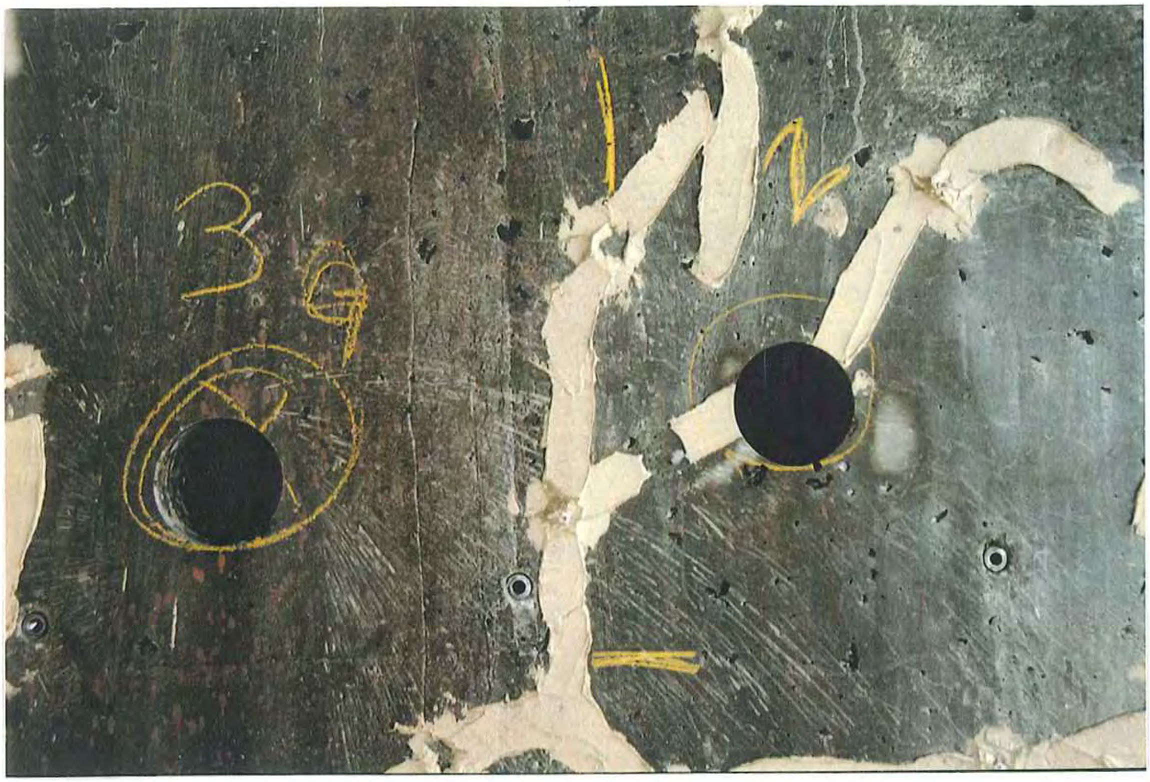


Figure 10. *Close-up of area in Figure 9 showing the locations of Cores 2 and 3.*

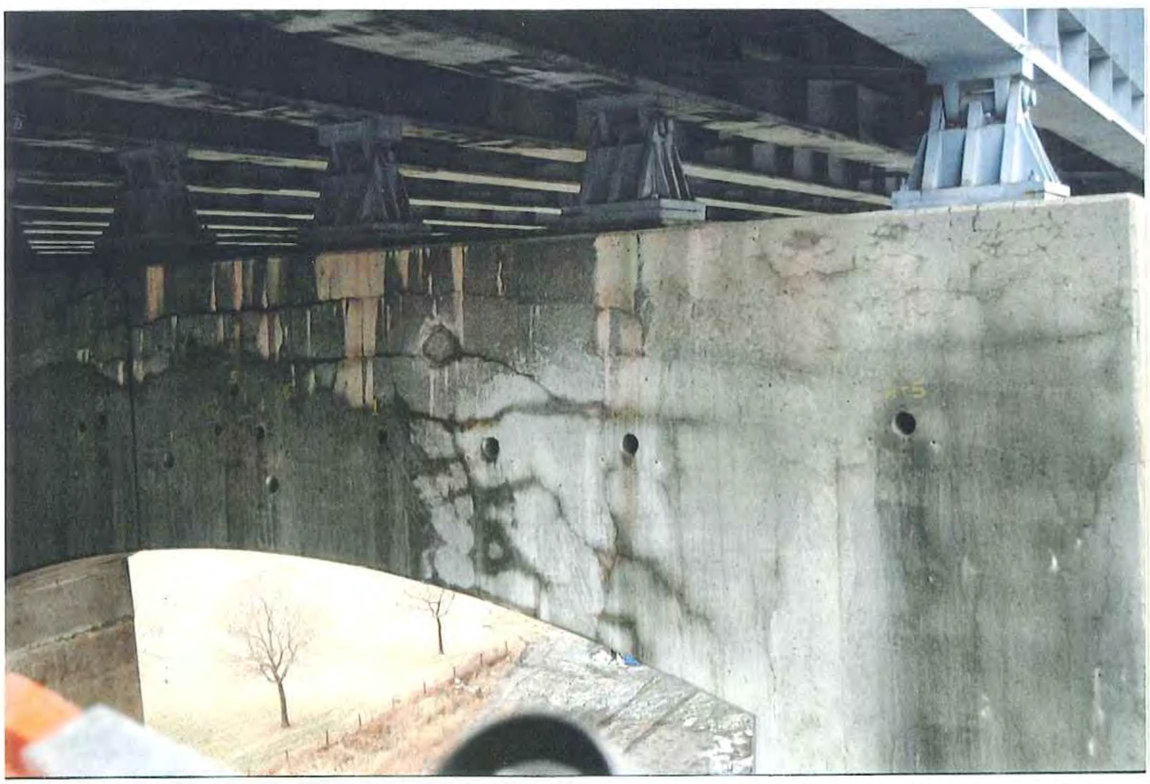


Figure 11. *South face of Pier 39 north-bound showing locations of cores. One additional core was taken in the column at the right side of the photo.*

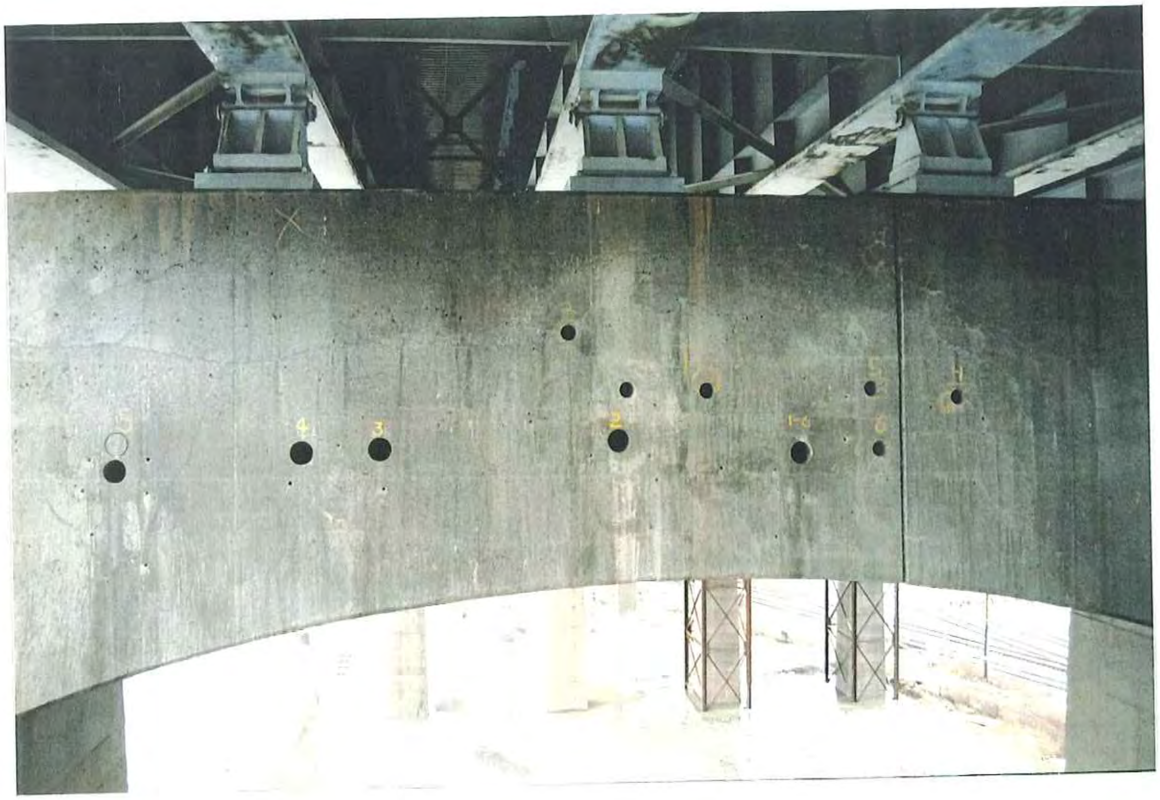


Figure 12. North face of Pier 38 north-bound showing core locations. One additional core was taken from the column at the left of the photo.

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

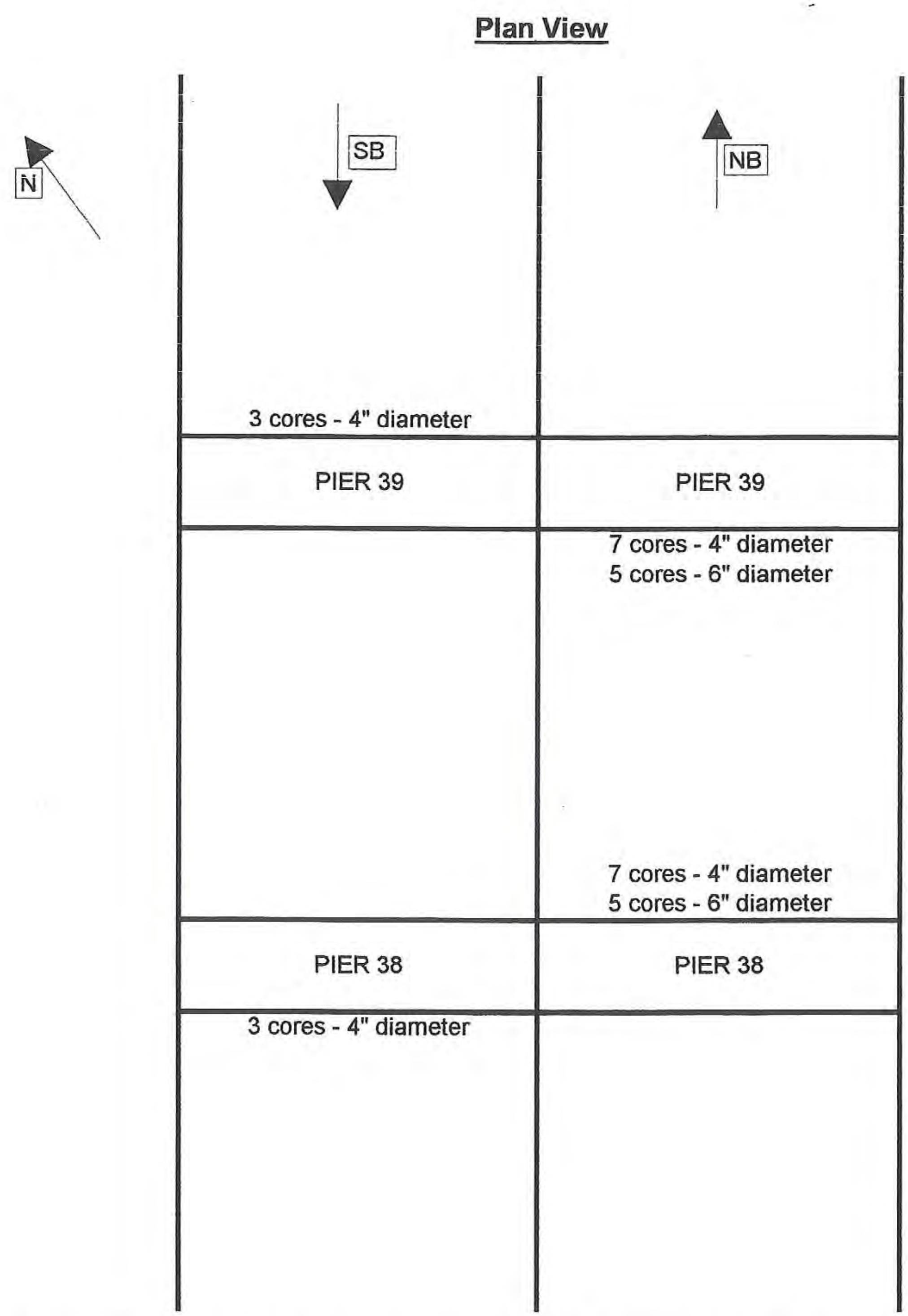
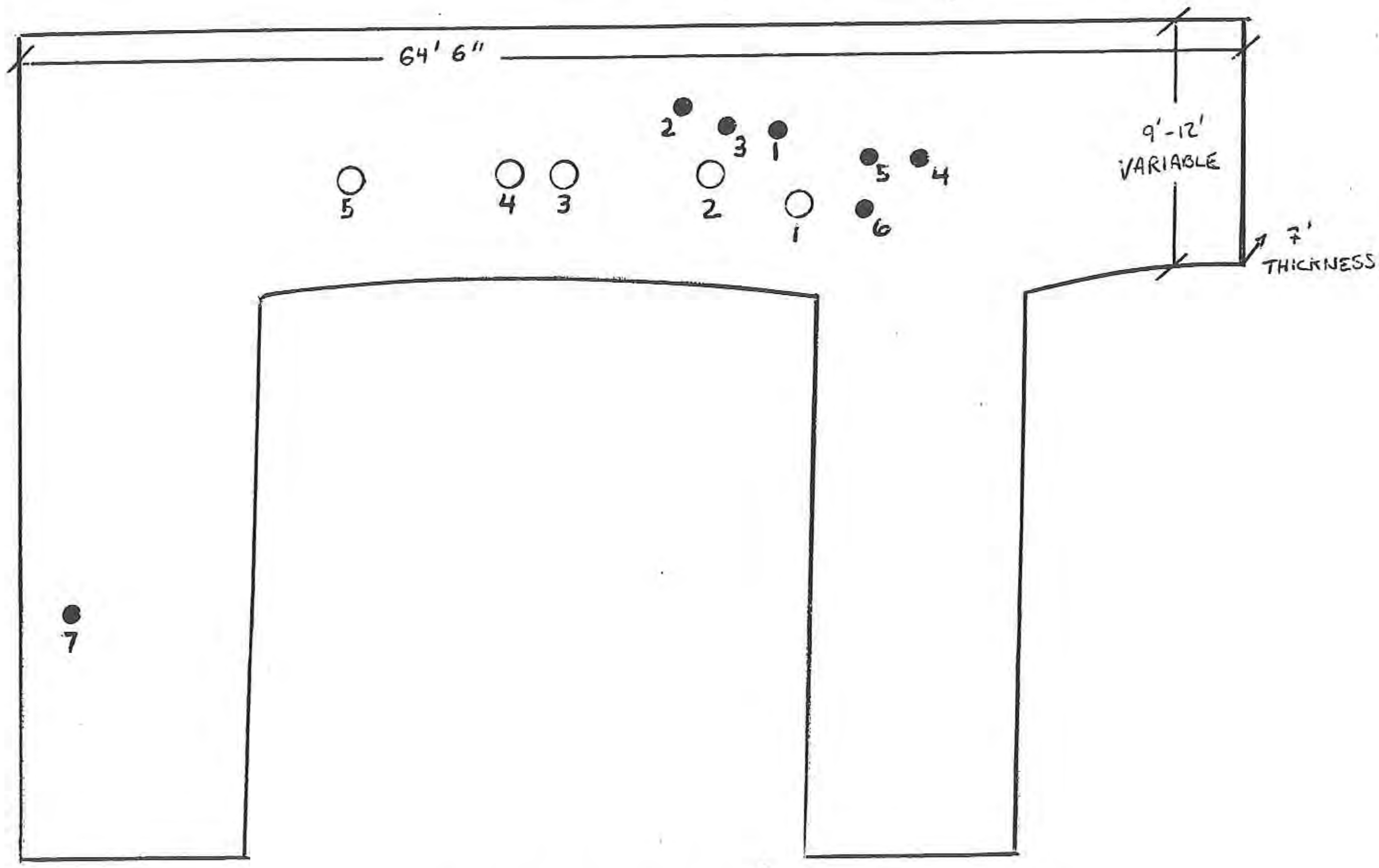


Figure 14 Plan view of coring locations for Pier Caps 38 and 39 NB and SB.

Pier 38 North-Bound North-Face



Pier 38 South-Bound South-Face

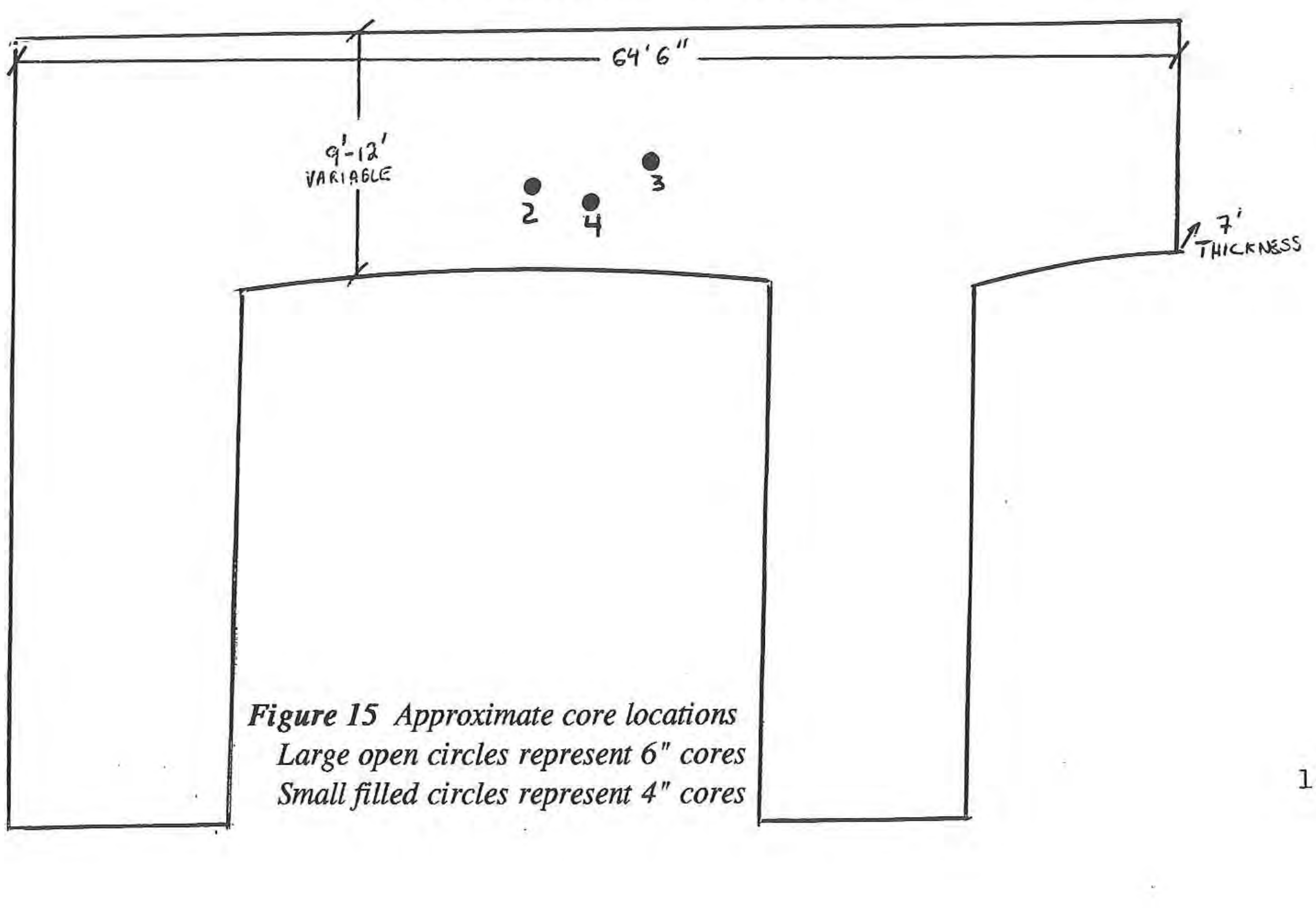
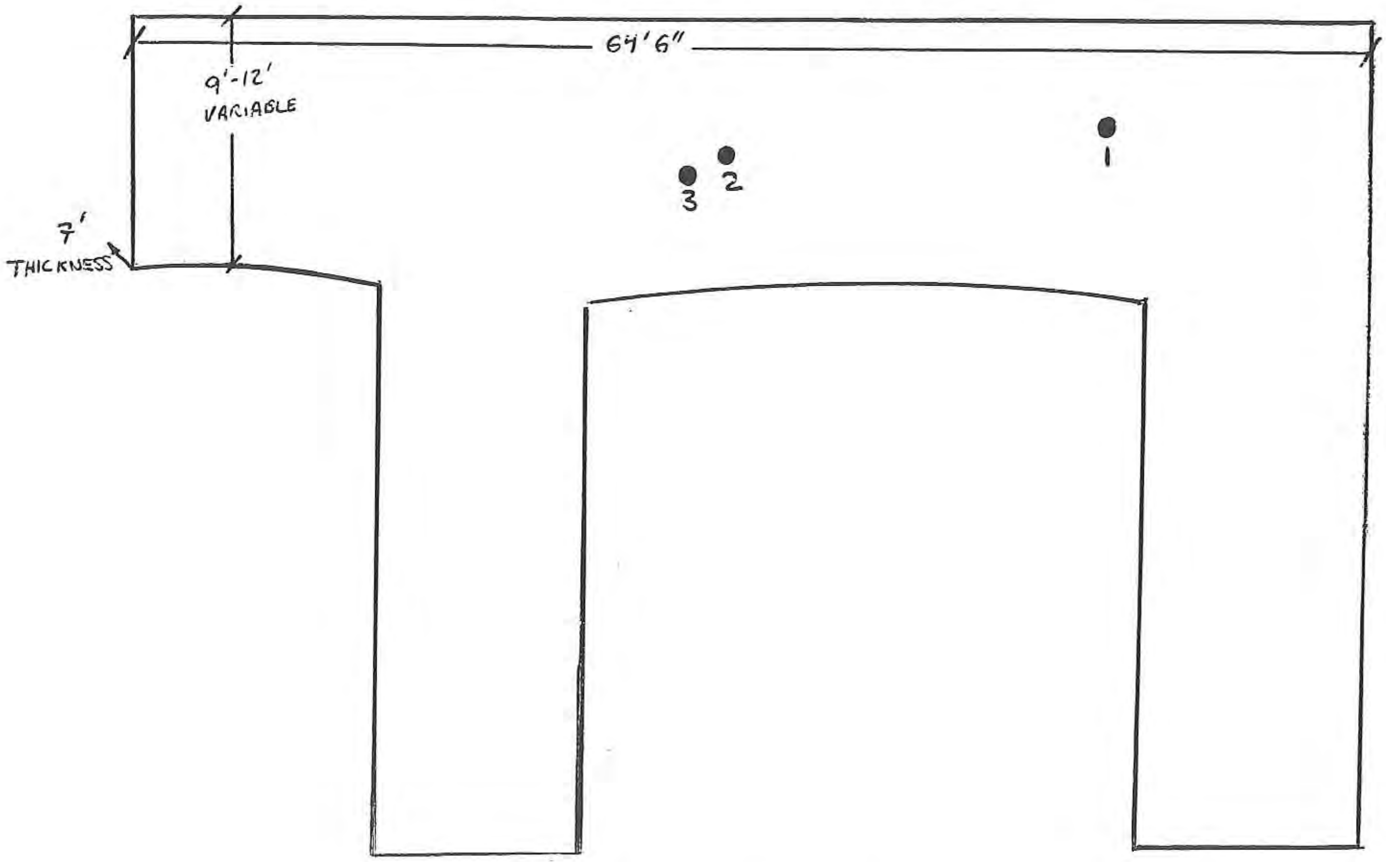


Figure 15 *Approximate core locations*
Large open circles represent 6" cores
Small filled circles represent 4" cores

Pier 39 South-Bound North-Face



Pier 39 North-Bound South-Face

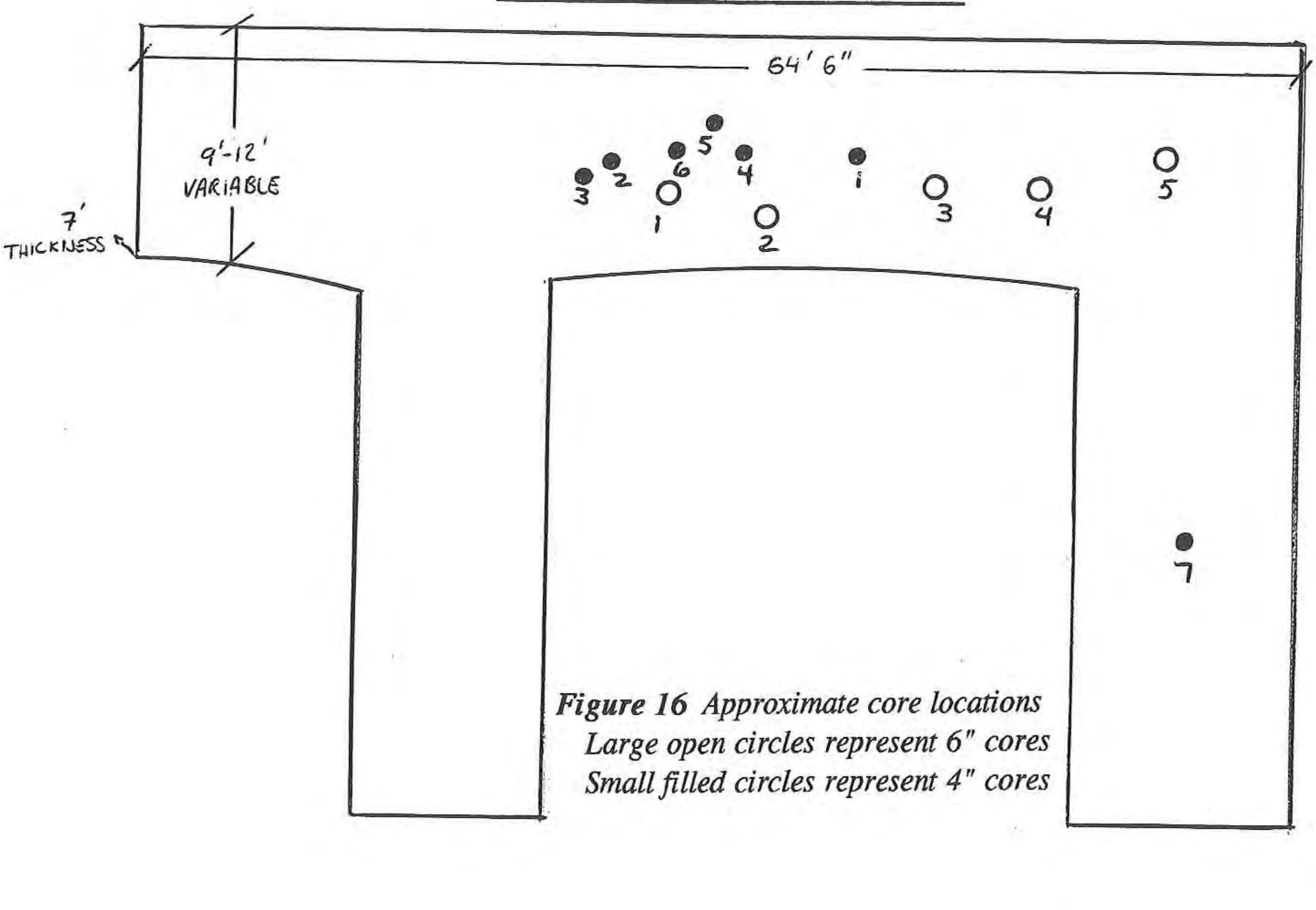


Figure 16 *Approximate core locations*
Large open circles represent 6" cores
Small filled circles represent 4" cores

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.

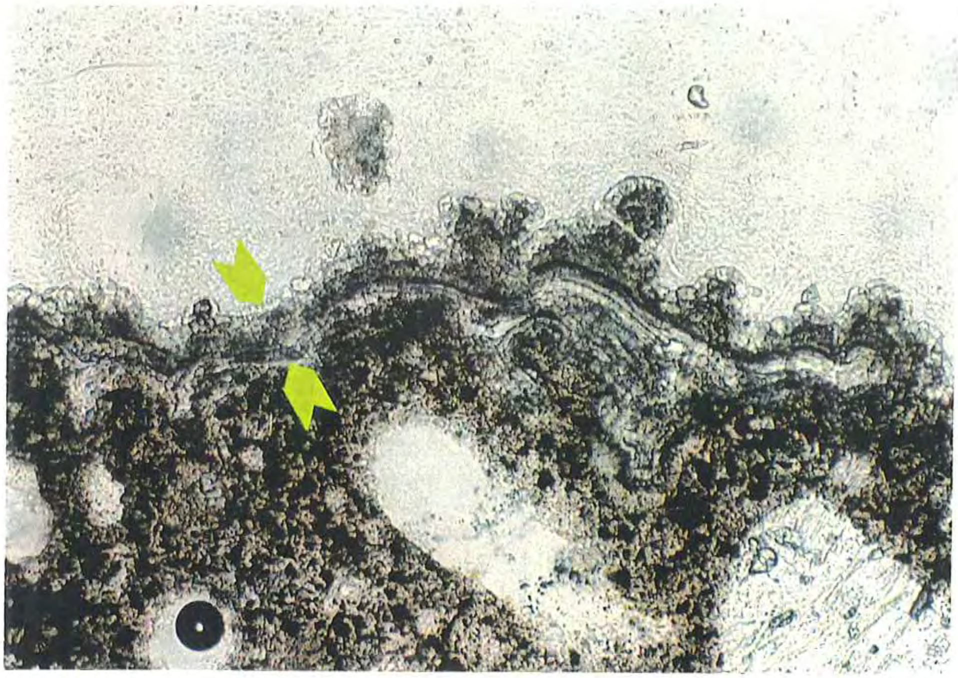


Figure 17. *Thin section photomicrograph of core #2 from Pier 38 north face, north-bound, showing inter-layered alkali-silica gel and calcium carbonate deposits (between arrows) on crack wall. Plane-polarized light. Width of field is approximately 350 μm.*

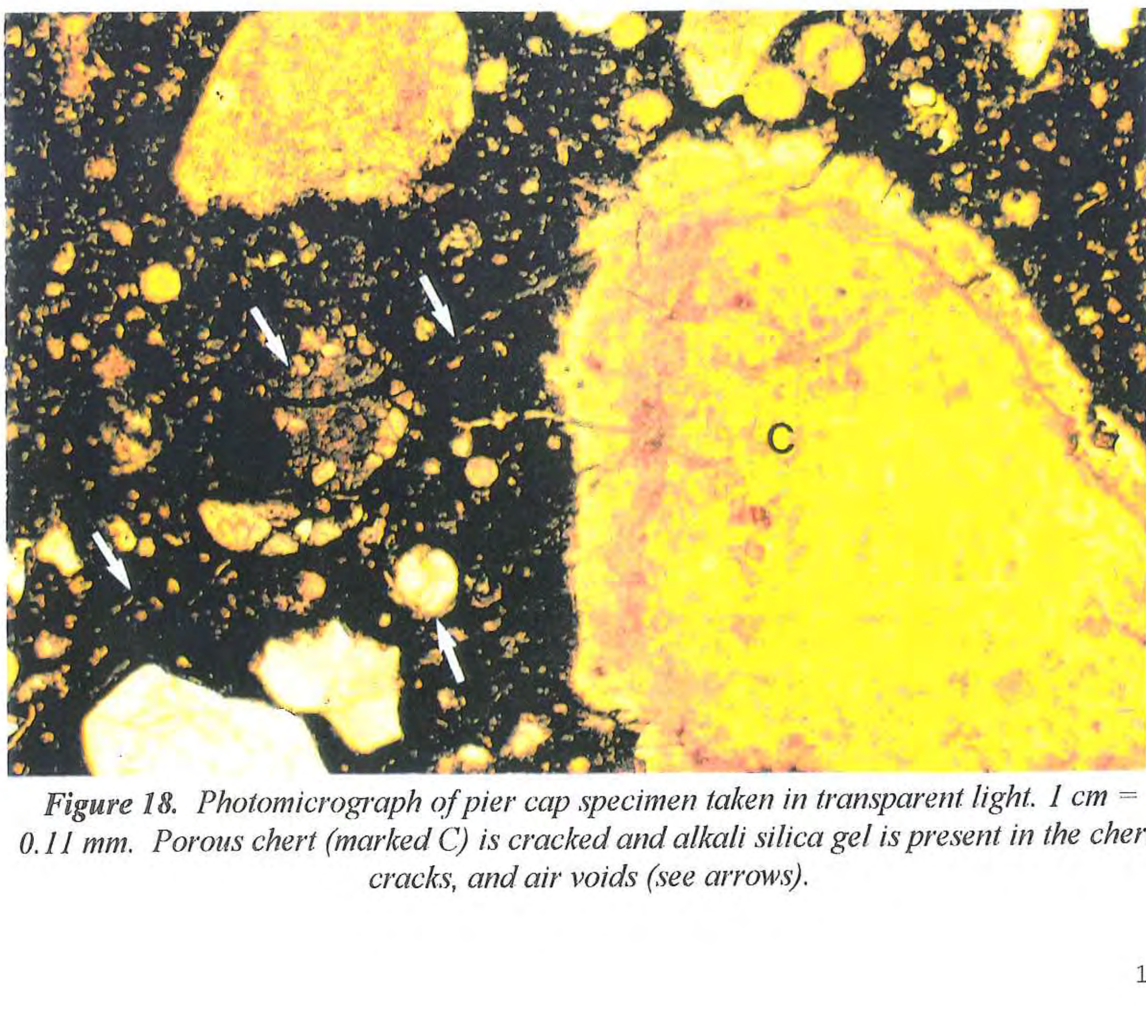


Figure 18. *Photomicrograph of pier cap specimen taken in transparent light. 1 cm = 0.11 mm. Porous chert (marked C) is cracked and alkali silica gel is present in the chert, cracks, and air voids (see arrows).*

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.

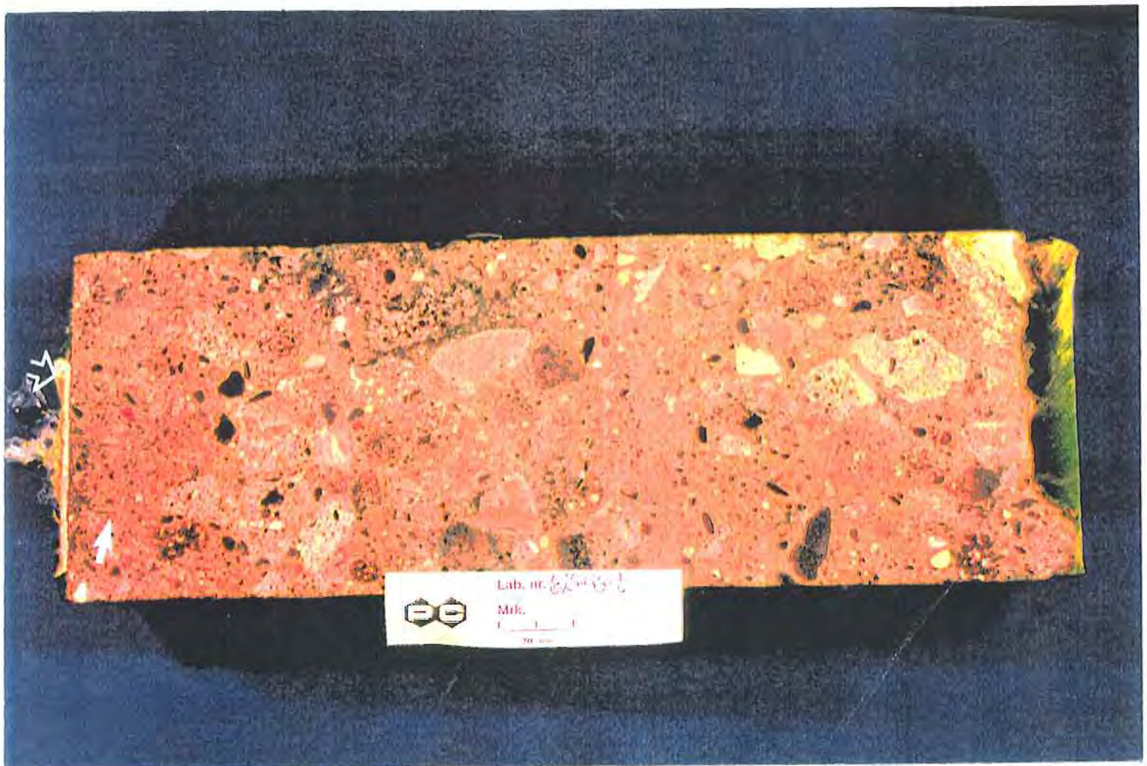


Figure 19. *Photo of Pier 39 sample with epoxy layer present at the surface (see open arrow). The layer does not penetrate the concrete. A surface crack is present (see closed arrow).*

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	--	$3.04\text{x}10^6$ psi$^+$
Pier 38 #2 NB	--	$2.76\text{x}10^6$ psi$^+$
Pier 38 #3 NB	--	$3.34\text{x}10^6$ psi$^+$
Pier 38 #4 NB	--	$2.69\text{x}10^6$ psi$^+$
Pier 38 #3 NB (4")	8072 psi*	--
Pier 38 #4 NB (4")	7748 psi*	--
Pier 38 Average	**7910 psi**	**$2.96\text{x}10^6$ psi**
Pier 39 #1 NB	--	$2.30\text{x}10^6$ psi$^+$
Pier 39 #2 NB	5089 psi	$2.42\text{x}10^6$ psi
Pier 39 #3 NB	--	$3.01\text{x}10^6$ psi$^+$
Pier 39 #4 NB	5721 psi	$2.66\text{x}10^6$ psi
Pier 39 Average	**5405 psi**	**$2.60\text{x}10^6$ 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	--	$4.60x10^6$ psi$^+$
Reference #3	4644 psi	$4.74x10^6$ psi
Reference #4	4782 psi	$4.22x10^6$ psi
Reference #5	--	$4.60x10^6$ psi$^+$
Reference #6	4750 psi	$4.01x10^6$ psi
Reference #7	--	$4.01x10^6$ psi
Average	**4725 psi**	**$4.36x10^6$ 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	$355x10^{-6}$	$413x10^{-6}$	$733x10^{-6}$	$1022x10^{-6}$
Pier 39 Average	$460x10^{-6}$	$535x10^{-6}$	$867x10^{-6}$	$1208x10^{-6}$
Reference Batch Avg.	$265x10^{-6}$*	$325x10^{-6}$	$663x10^{-6}$*	$985x10^{-6}$

** 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	$0.518x10^{-6}$	$0.722x10^{-6}$
Pier 39 Average	$0.614x10^{-6}$	$0.854x10^{-6}$
Reference Batch Avg.	$0.535x10^{-6}$*	$0.796x10^{-6}$

** At 155 days*

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

3.6 <u>Potential for Future Expansion Testing</u>

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 Surface Sealing

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.

<u>Surface Preparation</u>

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.

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Appendix A.

Petrographic Reports

CTL Structural/Architectural Engineering,
Testing and Materials Technology

5420 Old Orchard Road, Skokie, Illinois 60077-1030
847/ 965-7500 **800/ 522-2CTL** Fax: 847/ 965-6541

PETROGRAPHIC SERVICES

CTL Project No. 050860 Date: September 26, 1996

RE: Addendum to Petrographic Report for Michigan Department of Transportation

The following comments are provided to address questions raised in response to our previous report on the cracking of the concrete pier caps, I-75 bridge over the River Rouge in Detroit, Michigan.

1. Results of all ASTM C 457 linear traverse data were previously forwarded. Two full C 457 analyses were performed. Partial analyses, conducted over traverse lengths of 20 to 25 in. were performed only to confirm the petrographer's estimates of air content. It is not our practice to save the computer files for these partial analyses.

2. Photographs included in the August 7 report show representative lapped surfaces of longitudinally cut concrete cores treated with uranyl acetate. The method is used to screen cores for evidence of potential alkali-silica reaction products (gel). The arrows on the photographs show bright fluorescence attributed to alkali-silica gel. The amount of gel detected is very small.

3. Residual portland cement clinker grains probably do not constitute a significant source of additional alkalies. The residual grains are composed of C_2S crystals and intergrown C_3A and C_4AF. These formulae are given in cement chemist's notation. These phases do not incorporate significant alkalies. Generally, the alkalies present in cement are readily soluble alkali sulfates precipitated in the clinker pore spaces. Grinding clinker to make cement exposes most of these pores, thus the readily soluble alkalies are released early during cement hydration.

4. Potential alkali reactivity of the slag coarse aggregate appears to be negligible based on the service record, i.e. the slag has not reacted after 30 years in service. The report "Iron Blast-Furnace Slag Production, Processing, Properties, and Uses", by Josephson et al. does not provide insight into expressed concerns for reactivity. The report discusses the agricultural applications of slag as a fertilizer, and the treatment of soil to neutralize acidity. The report refers to 'soluble silica in slag' (referenced, 1900) and 'easily liberated silicic acid' (reference

not provided, but before 1949) as being beneficial to plants. Differences in the behavior of ground slag in an aerated soil with organic constituents and that of slag coarse aggregates embedded in concrete would be anticipated. Blast-furnace slag is alkaline and is stable in the highly alkaline environment provided by portland cement paste. The pH of slag, typically between 9 and 10, is essentially the same as the pH of carbonated paste, which can be as low as 9.0 to 9.5. ASTM tests designed to identify alkali-reactive aggregates have not identified blast-furnace slag as reactive. Unless the slag has an unusual composition, not typical of the tested slags, it is presumed nonreactive.

L. J. Powers-Couche

L. J. Powers-Couche
Senior Petrographer
Petrographic Services

050860

ACCUMULATED LINEAR TRAVERSE DATA

Originator- R. Detwiler
Operator--- R. Sturm
Date------- 06/05/96
File------- 050860-1
Measured Paste Content = 29.4%
Field:4 x 3.25 in.

Sample ID- Pier 38 #5 NB
Project #- 050860

Total Travel Executed	95.0	in
Total Area Covered	12.5	Sq. in
Total Void Length	6.6	in
Total Number of Voids	814	

Void Size Breakdown (increments of 0.0001 inches)

Voids less than 10	64	(7.86%)	[0.05%]
Voids 10 to 20	116	(14.25%)	[0.18%]
Voids 21 to 30	105	(12.90%)	[0.28%]
Voids 31 to 40	96	(11.79%)	[0.35%]
Voids 41 to 50	76	(9.34%)	[0.36%]
Voids 51 to 60	78	(9.58%)	[0.45%]
Voids 61 to 70	68	(8.35%)	[0.47%]
Voids 71 to 80	38	(4.67%)	[0.30%]
Voids 81 to 90	38	(4.67%)	[0.34%]
Voids 91 to 100	18	(2.21%)	[0.18%]
Voids 101 to 200	73	(8.97%)	[1.05%]
Voids 201 to 393.6	19	(2.33%)	[0.52%]
Voids 393.7 (1mm) and greater	25	(3.07%)	[2.47%]

LINEAR TRAVERSE CALCULATIONS

Average Chord Intercept	0.0082	in
Voids per Inch	8.57	
Specific Surface (1/in)	490.2	
Paste to Air Ratio	4.20	
Air Content	6.99	%
Spacing Factor	0.0086	in

Values in "( )" next to "Void Size" columns show void "Count" distribution relative to "Total Number of Voids".
Values in "[ ]" next to "Void Size" columns show void "Length" distribution relative to total "Air Content".

ACCUMULATED LINEAR TRAVERSE DATA

Originator- R. Detwiler
Operator--- R. Sturm
Date------- 06/05/96
File------- 050860-2
Measured Paste Content = 25.9%
Field: 4 x 3.25 in.

Sample ID- Pier 39 #7 NB
Project #- 050860

Total Travel Executed	95.0	in
Total Area Covered	12.5	Sq. in
Total Void Length	6.0	in
Total Number of Voids	633	

Void Size Breakdown (increments of 0.0001 inches)

Voids less than 10	47	(7.42%)	[0.03%]
Voids 10 to 20	97	(15.32%)	[0.15%]
Voids 21 to 30	92	(14.53%)	[0.24%]
Voids 31 to 40	69	(10.90%)	[0.25%]
Voids 41 to 50	69	(10.90%)	[0.32%]
Voids 51 to 60	53	(8.37%)	[0.31%]
Voids 61 to 70	44	(6.95%)	[0.30%]
Voids 71 to 80	22	(3.48%)	[0.17%]
Voids 81 to 90	19	(3.00%)	[0.17%]
Voids 91 to 100	8	(1.26%)	[0.08%]
Voids 101 to 200	59	(9.32%)	[0.84%]
Voids 201 to 393.6	29	(4.58%)	[0.86%]
Voids 393.7 (1mm) and greater	25	(3.95%)	[2.54%]

LINEAR TRAVERSE CALCULATIONS

Average Chord Intercept	0.0094	in
Voids per Inch	6.66	
Specific Surface (1/in)	425.3	
Paste to Air Ratio	4.14	
Air Content	6.27	%
Spacing Factor	0.0097	in

Values in "( )" next to "Void Size" columns show void "Count" distribution relative to "Total Number of Voids".
Values in "[ ]" next to "Void Size" columns show void "Length" distribution relative to total "Air Content".

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7 August 1996

Mr. Phil Mohr
Department of Civil and Environmental Engineering
1326 G.G. Brown Building
2350 Hayward
Ann Arbor, Michigan 48109-2125
Phone: (313) 763-6824

MDOT Rouge River Bridge Piers 38 and 39: Petrographic Report

Dear Phil:

Here is our petrographer's report detailing her examination of the cores by the uranyl acetate test. Together with her reports date 3 May and 8 July, this is the information that should be included in Section 2.4 of the final report. If you would like me to summarize these reports in a format and style consistent with the other sections I submitted to you earlier, just let me know and I will get you something as soon as possible. (You may want to do it yourself so that you can choose the photographs you want to include and get the layout the way you want.) I assume that the original reports would appear as an appendix to the final report.

If there is anything else you would like me to help you with, feel free to contact me. I would like to review the final report when you have it ready. I plan to be in Skokie all this month, but have several trips planned for September.

Best regards,

Rachel

Rachel J. Detwiler, Ph.D.
Senior Engineer

Construction Technology Laboratories, Inc. **Chicago/Skokie** Denver Seattle/Tacoma

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PETROGRAPHIC SERVICES REPORT

CTL Project No.: 050860-A *Date:* August 7, 1996

Re: Petrographic Screening of Samples for Evidence of Alkali-Silica Reaction

The sawcut surfaces of concrete core samples previously examined petrographically for CTL Project 050860 were treated with uranyl acetate solution in accordance with the procedures described in SHRP-C/FR-91-101, 'Handbook for the Identification of Alkali-Silica Reactivity in Highway Structures.' The methodology is used to rapidly screen samples for evidence of alkali-silica reaction. Treated samples yield a brilliant yellow-green fluorescence where alkali-silica reaction product (gel) is present; however, confirmation of gel by microscopical methods is required because fluorescence may arise from other sources. Findings from this work are presented below.

FINDINGS

Treated samples exhibit only a limited occurrence of alkali-silica gel associated with the major cracks (see attached figures). Small amounts of gel line air voids near reactive chert sand grains. 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, reaction of the slag aggregate is not implicated as the cause of cracking. Alkali-silica reaction involving the fine aggregate does not appear to be extensive enough to account for the severity of cracking in the concrete structure.

L. J. Powers-Couche

L. J. Powers-Couche
Senior Petrographer
Petrographic Services

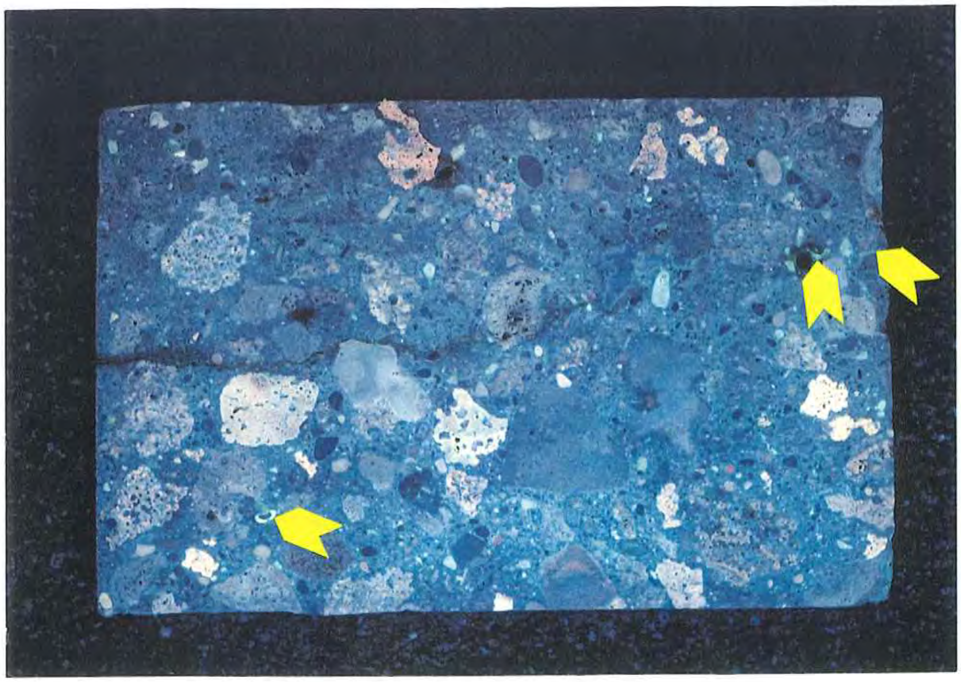
LJPC

050860-A

Attachments

CTL

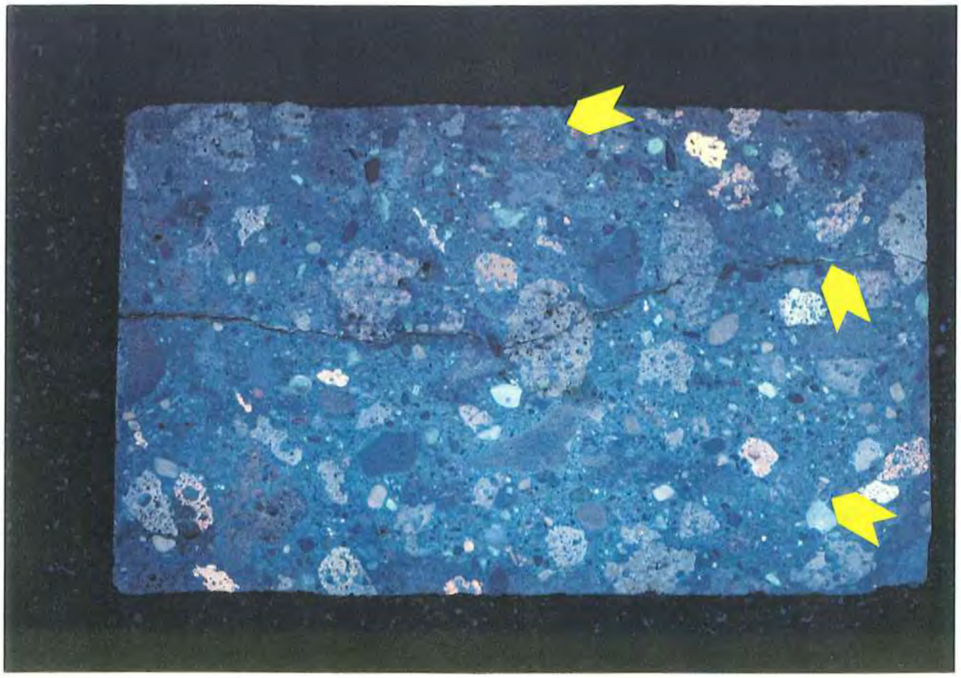
1a. Surface Photographed in Ordinary Light



1b. Surface Photographed in Ultraviolet Light

FIG. 1 PIER 39 #2 NB TREATED SAMPLE.

2a. Surface Photographed in Ordinary Light



2b. Surface Photographed in Ultraviolet Light

FIG. 2 PIER 39 #2 NB TREATED SAMPLE.

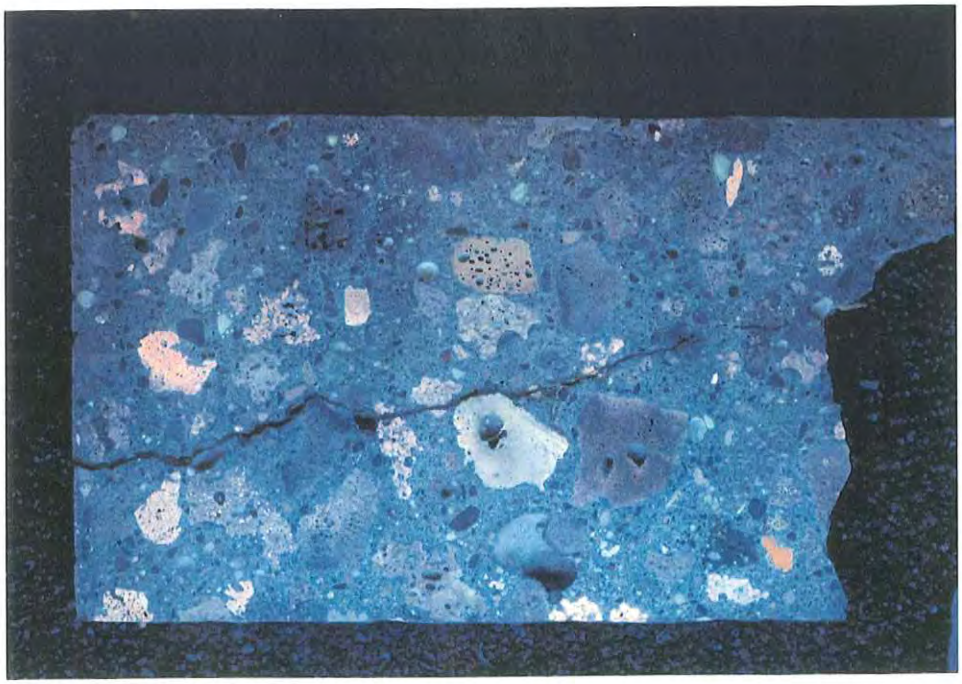
CTL

3a. Surface Photographed in Ordinary Light

3b. Surface Photographed in Ultraviolet Light

FIG. 3 PIER 38 #3 TREATED SAMPLE.

4a. Surface Photographed in Ordinary Light



4b. Surface Photographed in Ultraviolet Light

FIG. 4 PIER 39 S.F. 1C NB.

CTL Structural/Architectural Engineering, Testing and Materials Technology

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July 8, 1996

RE: MDOT Bridge Pier Samples

Sawcut cores, lapped surfaces and broken fragments of samples from the piers were screened for alkali-silica gel using the uranyl acetate method described in SHRP-C/FR-91-101. This method is considered to be a field test. Positive results require laboratory verification of the presence of gel. Negative results indicate ASR is not present.

Sample screening revealed the following:
1. Reaction rims and radial cracks filled with alkali-silica gel are not associated with the slag coarse aggregate. Many slag fragments fluoresce red, orange, and yellow. Gel fluorescence is bright yellow-green.

2. Characteristic yellow-green fluorescence is observed surrounding some chert particles (fine aggregate). Radial cracks with gel linings are not common and where they do occur, the cracks do not extend far from the reactive particle.

3. Gel linings occur in several air voids and gel is observed in portions of narrow microcracks; however, the occurrence of limited quantities of alkali-silica reaction products associated with fine aggregate particles does not fully explain the major cracks in the cores examined.

Laura J. Powers-Couche
Senior Petrographer

Project 050860
Copy to RJD
Files

CTL Structural/Architectural Engineering, Testing and Materials Technology

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8 May 1996

Dr. Will Hansen
Associate Professor
Department of Civil and Environmental Engineering
2340 G.G. Brown Building
2350 Hayward
Ann Arbor, Michigan 48109-2125

Evaluation of Pier Cap Nos. 38 and 39 of the I-75 Bridge over Rouge River in Detroit, Michigan

Dear Will:

Here are the chloride analysis and the petrographer's report on the cores you sent us from the I-75 Bridge. Note that at the level of the steel, the chloride content is much the same as the background chloride content, consistent with our visual observations in the field.

As noted in her report, our petrographer found signs of alkali-silica reaction but does not believe it to be the principal cause of distress.

If you would like to discuss these findings, please do not hesitate to call me.

Best regards,

Rachel

Rachel J. Detwiler
Senior Engineer
Materials Research and Consulting

050860

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PETROGRAPHIC SERVICES REPORT

CTL Project No.: 050860 *Date:* May 3, 1996

Re: Petrographic Examination of Concrete Cores from Pier Caps, I-75 Bridge over the Rouge River, Michigan

Eighteen cores, labeled Pier 38 South Face #2 and #3, South Bound, and Pier 38 North Face #1 through #7, North Bound, and Pier 39 South Face #1 and #1C, #2 through #7, North Bound, and Pier 39 North Face #3, South Bound (Figs. 1 through 9) were received on January 26, 1996 from Dr. Rachel Detwiler, CTL Senior Engineer. The cores were submitted on behalf of Mr. Phil Mohr, University of Michigan, Ann Arbor, Michigan. The cores were taken from deteriorated pier caps of the above specified structure in Detroit, Michigan, in order to address the cause of cracking. The six cores listed below were chosen for detailed petrographic examination (ASTM C 856):

Pier 38 North Face #5 North Bound
Pier 38 North Face #2 North Bound
Pier 38 South Face #3 South Bound
Pier 39 South Face #7 North Bound
Pier 39 South Face #1 North Bound
Pier 39 South Face #2 North Bound

FINDINGS AND CONCLUSIONS

The results of the petrographic examinations do not fully reveal the cause of concrete cracking. Based on the widespread occurrence of small amounts of alkali-silica gel, expansion caused by alkali-silica reaction (ASR) may have contributed to the deterioration; however, it does not appear to be the principal cause of distress. Alkali-silica reaction, mainly involving chert in the fine aggregate, may influence the long-term durability of the concrete. Observations from the petrographic examinations are presented in Table 1. Additional findings are presented below.

1. The six cores examined represent similar concrete composed of manufactured slag (predominantly crystalline) coarse aggregate and natural sand fine aggregate in a portland cement paste.

   a. Aggregates are well graded to 3/4 in. top size.

   b. The fine aggregate consists of quartz, quartzite, limestone, chert, feldspar, granite, schist, graywacke, and garnet (in approximate order of decreasing abundance).

   c. The paste is hard and dense with a subvitreous luster (except near cracks). Paste color ranges from mottled brown-gray to buff and dark green. Dark green paste occurs adjacent to some slag aggregates.

   d. Interpreted water-cement ratio is low, less than 0.40 in cores from Pier 38 and 0.38 to 0.43 in cores from Pier 39, based on microscopical observations (thin-section study) and macroscopical paste properties.

   e. The concrete is air entrained. Pier 39 South Face #77 North Bound contains an estimated 3 to 5% entrained air voids. The estimated air content in the remaining cores is 5 to 7%. Air voids are mostly small and nonuniformly distributed (clustered) A substantial amount of entrapped air is observed; however, these air voids are widely scattered and typically smaller than 1/2 in.

2. Major longitudinal cracks are relatively old based on the presence of substantial paste carbonation along the crack walls. Water infiltration along cracks has locally leached calcium hydroxide from the paste and deposited ettringite in available space. 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 interlayered gel and calcite coatings on the walls of major cracks (Fig. 10) and branching, gel-filled microcracks extending from reactive aggregates into the paste (Fig. 11).

TABLE 1: SUMMARY OF SELECTED PETROGRAPHIC OBSERVATIONS

Core Sample	Cracks	Microcracks	Secondary Deposits/Alkali-Silica Reaction
Pier 38			
North Face #5 North Bound	None	Many in exterior 3 in.; some associated with ASR.	Minor. Gel-soaked paste around chert; gel in microcracks.
North Face #2 North Bound	Longitudinal Transverse	Few; passing through aggregates.	Ettringite and calcium hydroxide in air voids. Minor ASR. Gel and calcite line portions of major crack.
South Face #3 South Bound	Longitudinal Transverse	Few; passing through aggregates.	Minor ettringite in voids. Gel-soaked paste around chert, graywacke, and schist.
Pier 39			
South Face #7 North Bound	None	None	Minor ettringite in air voids. Gel-soaked paste around some chert particles.
South Face #1 North Bound	Longitudinal Transverse	Few; some microcracks radiating from reactive chert.	Abundant ettringite, minor calcium hydroxide and calcite in air voids. Alkali-silica gel in cracks.
South Face #2 North Bound	Longitudinal	Many parallel to major crack; many associated with ASR.	Abundant ettringite in air voids. Alkali-silica gel in microcracks and voids. Gel-soaked paste around chert.

METHODS OF TEST

Petrographic examination of the cores was performed in accordance with ASTM C 856-83 (reapproved 1988), "Standard Practice for Petrographic Examination of Hardened Concrete."

The cores were photographed and visually examined. A 0.7-in.-thick slice was cut longitudinally from the cores and one side of each was lapped. Lapped and freshly broken surfaces were studied with a stereomicroscope at magnifications up to 45X. Secondary deposits were removed using a fine tungsten probe and placed in refractive index media on glass microscope slides. The mounted samples were studied using a polarized-light microscope at magnifications up to 1000X. A rectangular block, measuring about 2 in. long, 1 in. wide, and 0.5 in. thick, was cut from each core and placed on separate glass microscope slides with epoxy resin. The mounted samples were reduced to a thickness of about 0.0008 in. (20 µm) and examined using a polarized-light microscope at magnifications up to 400X, to determine aggregate and paste mineralogy and microstructure.

Laura J. Powers-Couche

L. J. Powers-Couche
Senior Petrographer
Petrographic Services

LPC/djp

050860

Attachments

CTL

FIG. 1 SIDE VIEWS OF CORES AS RECEIVED FOR EXAMINATION.

FIG. 2 SIDE VIEW OF CORE, AS RECEIVED.

FIG. 3 TWO SIDE VIEWS OF CORE, AS RECEIVED. ARROWS SHOW FRESH ALKALI-SILICA GEL AROUND CHERT AGGREGATES.

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FIG. 4 SIDE VIEWS OF CORES, AS RECEIVED.

FIG. 5 SIDE VIEWS OF CORES, AS RECEIVED.

FIG. 6 SIDE VIEWS OF CORES, AS RECEIVED.

FIG. 7 SIDE VIEWS OF CORES, AS RECEIVED.

FIG. 7 SIDE VIEWS OF CORES, AS RECEIVED.

FIG. 8 SIDE VIEWS OF CORES, AS RECEIVED.

FIG. 9 SIDE VIEWS OF CORES, AS RECEIVED.

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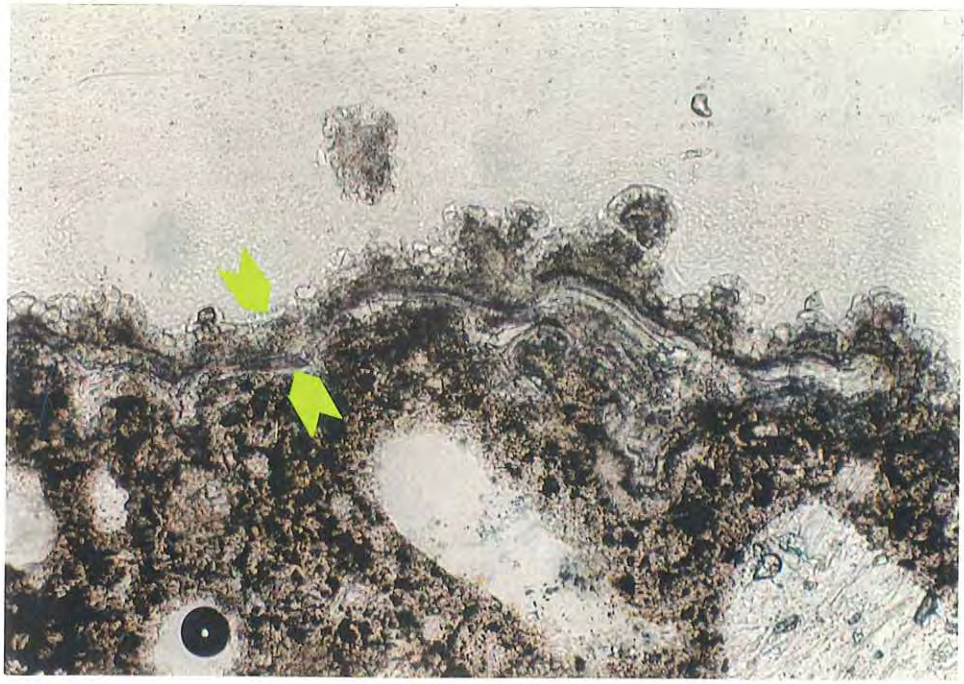

FIG. 10 THIN-SECTION PHOTOMICROGRAPH OF CORE PIER 38 NF #2NB SHOWING INTERLAYERED ALKALI-SILICA GEL AND CALCIUM CARBONATE DEPOSITS (BETWEEN ARROWS) ON CRACK WALL. PLANE-POLARIZED LIGHT. WIDTH OF FIELD IS ABOUT 350 μM.

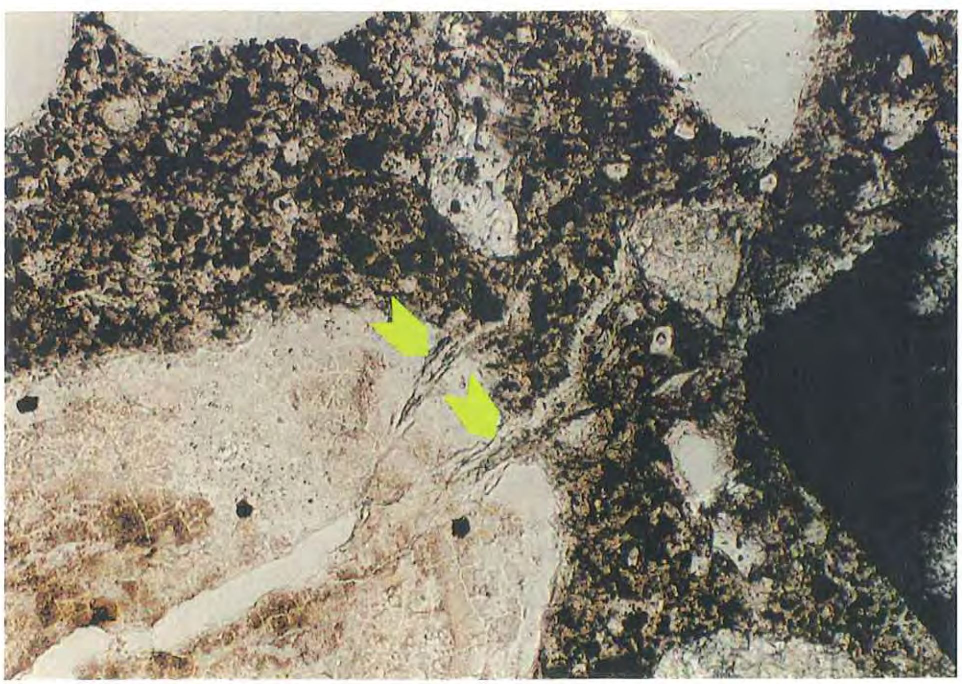

FIG. 11 THIN-SECTION PHOTOMICROGRAPH OF CORE PIER 39 SF #2NB SHOWING GEL-FILLED MICROCRACKS (ARROWS) EMANATING FROM REACTIVE CHERT PARTICLES. PLANE-POLARIZED LIGHT. WIDTH OF FIELD IS ABOUT 700 μM.

CTL

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE, ASTM C 856

CTL PROJECT NO.: 050860

CLIENT: Michigan Department of Transportation

STRUCTURE: Bridge Pier Caps

LOCATION: I-75 Bridge
Rouge River, Detroit, Michigan

DATE: May 3, 1996

REPORTED PROBLEM: Cracking

EXAMINED BY: L. Powers-Couche

SAMPLE

Identification: Pier 38 North Face #5 North Bound.

Dimensions: Diameter = 3.9 in. Length = 10.5 in.

Exterior Surface: Even, paste-rich, formed surface with scattered, small irregular air voids (bugholes).

Interior Surface: Irregular surface broken through aggregates.

Cracks, Joints, Large Voids: No significant cracks are present. No joints are observed. A few large irregular air voids up to 0.6 in. long are present.

Reinforcement: None present in core.

AGGREGATES (A)

Coarse (C): Manufactured, mostly crystalline slag.

Fine (F): Natural sand composed of siliceous and calcareous rocks and minerals. In approximate order of decreasing abundance: quartz, quartzite, limestone, chert, feldspar, granite, schist, graywacke, and garnet.

Gradation & Top Size: Well graded to 0.8 in. top size.

Shape & Distribution: CA - angular to subrounded, blocky to oblong, and uniformly distributed. FA - rounded to angular, equant to oblong, and uniformly distributed.

PASTE

Color: Mottled shades of brown-gray.

Hardness: Hard to moderately hard.

Luster: Subvitreous to dull locally.

Depth of Carbonation: The paste is carbonated to a depth of 0.2 to 0.3 in. from the exterior surface.

Air Content: Air entrained with 5 to 7% small, spherical air voids.

Paste-Aggregate Bond: Tight. Lab-induced fractures pass through the coarse aggregates.

Calcium Hydroxide*: 5 to 7% uniformly distributed tabular crystals.

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Unhydrated Portland Cement Clinker Particles (UPC's)*: 15 to 22% somewhat nonuniformly distributed UPC's and partly hydrated clinker particles.

Pozzolans*: None observed. A small amount of slag is present, presumably fragments derived from the coarse aggregate.

Secondary Deposits: Calcium hydroxide occurs in a few air voids. Most voids are empty. Alkali-silica gel lines several air voids and fills microcracks in a reactive chert particle.

MICROCRACKING: A few random microcracks and many microcracks oriented parallel to the exterior end of the core occur in the outer 3 in. Most microcracks pass through aggregates.

ESTIMATED WATER-CEMENT RATIO: <0.40.

MISCELLANEOUS: Dark, gel-soaked paste occurs around a few chert particles; however, distress caused by ASR is limited to microcracks.

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*percent by volume of paste

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE, ASTM C 856

CTL PROJECT NO.: 050860

CLIENT: Michigan Department of Transportation

STRUCTURE: Bridge Pier Caps

LOCATION: I-75 Bridge
Rouge River, Detroit, Michigan

DATE: May 3, 1996

REPORTED PROBLEM: Cracking

EXAMINED BY: L. Powers-Couche

SAMPLE

Identification: Pier 38 North Face #2 North Bound.

Dimensions: Diameter = 3.7 in. Length = 10.5 to 11.5 in.

Exterior Surface: Even, paste-rich, formed surface with scattered irregular air voids (bugholes) and two irregular cracks across the surface.

Interior Surface: Irregular surface broken through aggregates.

Cracks, Joints, Large Voids: One longitudinal crack extends the length of the core. One longitudinal crack extends from the exterior surface to a depth of 3 in. and another extends from the 6 in. transverse crack to the 9 in. transverse crack. Transverse cracks occur at depths of 6 in. and 9 in. The latter extends part way through the core.

Reinforcement: None present in core.

AGGREGATES (A)

Coarse (C): Manufactured, mostly crystalline slag.

Fine (F): Natural sand composed of siliceous and calcareous rocks and minerals. In approximate order of decreasing abundance: quartz, quartzite, limestone, chert, feldspar, granite, schist, graywacke, and garnet.

Gradation & Top Size: Well graded to 0.8 in. top size.

Shape & Distribution: CA - angular to subrounded, blocky to oblong, and uniformly distributed. FA - rounded to angular, equant to oblong, and uniformly distributed.

PASTE

Color: Mottled shades of brown-gray.

Hardness: Hard to moderately hard.

Luster: Subvitreous to dull locally.

Depth of Carbonation: Unevenly carbonated to a depth of 0.1 to 0.4 in. from the exterior surface; thin layer of paste carbonation along major longitudinal crack.

Air Content: Air entrained with 5 to 7% small, spherical air voids. Air voids are clustered; nonuniformly distributed.

Paste-Aggregate Bond: Tight. Lab-induced fractures pass through the coarse aggregates.

Calcium Hydroxide*: 5 to 7% tabular and patchy calcium hydroxide; nonuniform distribution.

Unhydrated Portland Cement Clinker Particles (UPC's)*: 15 to 22% nonuniformly distributed UPC's.

Pozzolans*: None observed. Slag particles observed in the paste are large and resemble the crystalline slag coarse aggregate.

Secondary Deposits: Ettringite and bladed calcium hydroxide occur in air voids in moderate amounts. Portions on the major longitudinal crack are coated with layered deposits of alkali-silica gel and calcium carbonate.

MICROCRACKING: A few random microcracks passing through aggregates are observed.

ESTIMATED WATER-CEMENT RATIO: <0.40.

MISCELLANEOUS: Dark, gel-soaked paste occurs around a few chert particles, however, distress caused by ASR is limited to microcracks.

*percent by volume of paste

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE, ASTM C 856

CTL PROJECT NO.: 050860

CLIENT: Michigan Department of Transportation

STRUCTURE: Bridge Pier Caps

LOCATION: I-75 Bridge
Rouge River, Detroit, Michigan

DATE: May 3, 1996

REPORTED PROBLEM: Cracking

EXAMINED BY: L. Powers-Couche

SAMPLE

Identification: Pier 38 South Face #3 South Bound.

Dimensions: Diameter = 3.9 in. Length = 9.0 in.

Exterior Surface: Even, paste-rich, formed surface with scattered, irregular air voids (bugholes) and two intersecting cracks up to 0.05 in. wide across surface.

Interior Surface: Irregular surface broken through aggregates.

Cracks, Joints, Large Voids: One major longitudinal crack passes around the coarse aggregates and extends through the core. Transverse cracks at 3.0 and 3.7 in. also pass around aggregates (offset by longitudinal crack). Minor transverse cracks branch from the major cracks.

Reinforcement: None present in core.

AGGREGATES (A)

Coarse (C): Manufactured, mostly crystalline slag.

Fine (F): Natural sand composed of siliceous and calcareous rocks and minerals. In approximate order of decreasing abundance: quartz, quartzite, limestone, chert, feldspar, granite, schist, graywacke, and garnet.

Gradation & Top Size: Well graded to 0.8 in. top size.

Shape & Distribution: CA - angular to subrounded, blocky to oblong, and uniformly distributed. FA - rounded to angular, equant to oblong, and uniformly distributed.

PASTE

Color: Mottled shades of brown-gray with green staining adjacent to some aggregates.

Hardness: Hard to moderately hard.

Luster: Subvitreous to dull locally.

Depth of Carbonation: The paste is carbonated from the exterior surface to a depth of 0.2 in.; carbonation layer is present on the walls of the longitudinal crack.

Air Content: Air entrained with 5 to 7% small, spherical air voids. Air void distribution is nonuniform with many minute voids tightly clustered.

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Paste-Aggregate Bond: Tight. Lab-induced fractures pass through the coarse aggregates.

Calcium Hydroxide*: 4 to 6% nonuniformly distributed irregular patches and tabular crystals.

Unhydrated Portland Cement Clinker Particles (UPC's)*: 15 to 22% nonuniformly distributed UPC's. Partial rims of tightly packed UPC's occur around some slag aggregates.

Pozzolans*: A small amount of residual finely ground slag is observed.

Secondary Deposits: A small amount of ettringite is present in a few air voids. Voids and cracks are mostly empty.

MICROCRACKING: A few random tight microcracks, some passing through aggregates; generally oriented parallel to outside end of core.

ESTIMATED WATER-CEMENT RATIO: <0.40.

MISCELLANEOUS: Dark rims occur around many chert particles and some graywacke and schist particles.

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*percent by volume of paste

CTL

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE, ASTM C 856

CTL PROJECT NO.: 050860

CLIENT: Michigan Department of Transportation

STRUCTURE: Bridge Pier Caps

LOCATION: I-75 Bridge
Rouge River, Detroit, Michigan

DATE: May 3, 1996

REPORTED PROBLEM: Cracking

EXAMINED BY: L. Powers-Couche

SAMPLE

Identification: Pier 39 South Face #7 North Bound.

Dimensions: Diameter = 3.9 in. Length = 11.8 in.

Exterior Surface: Somewhat rough abraded or eroded surface with a few scattered shallow irregular air voids (bugholes).

Interior Surface: Irregular surface broken through aggregates.

Cracks, Joints, Large Voids: No joints, major cracks, or large air voids are observed.

Reinforcement: None present in core.

AGGREGATES (A)

Coarse (C): Manufactured, mostly crystalline slag.

Fine (F): Natural sand composed of siliceous and calcareous rocks and minerals. In approximate order of decreasing abundance: quartz, quartzite, limestone, chert, feldspar, granite, schist, graywacke, and garnet.

Gradation & Top Size: Well graded to 0.8 in. top size.

Shape & Distribution: CA - angular to subrounded, blocky to oblong, and uniformly distributed. FA - rounded to angular, equant to oblong, and uniformly distributed.

PASTE

Color: Mottled shades of brown-gray with green rims on some aggregates.

Hardness: Hard to moderately hard.

Luster: Subvitreous to dull locally.

Depth of Carbonation: The paste is carbonated to a depth of 0.4 to 0.6 in. from the exterior surface.

Air Content: Air entrained with 3 to 5% small, spherical, nonuniformly distributed air voids.

Paste-Aggregate Bond: Tight. Lab-induced fractures pass through the coarse aggregates.

Calcium Hydroxide*: 5 to 7% uniformly distributed small crystals and larger crystals in paste-aggregate gaps.

Unhydrated Portland Cement Clinker Particles (UPC's)*: 12 to 18% nonuniformly distributed UPC's and partly hydrated clinker particles. Concentrations of residual cement occur around some aggregates.

Pozzolans*: A small amount of residual slag is present.

Secondary Deposits: Most air voids are empty or contain clumps of small ettringite needles. Alkali-silica gel is observed adjacent to reactive chert aggregates. A few air voids are lined with ettringite.

MICROCRACKING: None observed.

ESTIMATED WATER-CEMENTITIOUS RATIO: 0.38 to 0.43.

MISCELLANEOUS: Gel-soaked paste surrounds some chert aggregates.

*percent by volume of paste

CTL

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE, ASTM C 856

CTL PROJECT NO.: 050860

CLIENT: Michigan Department of Transportation

STRUCTURE: Bridge Pier Caps

LOCATION: I-75 Bridge
Rouge River, Detroit, Michigan

DATE: May 3, 1996

REPORTED PROBLEM: Cracking

EXAMINED BY: L. Powers-Couche

SAMPLE

Identification: Pier 39 South Face #1 North Bound.

Dimensions: Diameter = 3.9 in. Length = 11.0 in.

Exterior Surface: Even, paste-rich, formed surface with scattered irregular air voids (bugholes) with one major crack, 0.04 in. wide, extending across the surface.

Interior Surface: Irregular surface broken through aggregates.

Cracks, Joints, Large Voids: One longitudinal crack extends to a depth of 6 in. One transverse crack, through the core, occurs at a depth of 6 to 7 in. Both cracks pass mainly around aggregates. No joints are present. The concrete contains a few entrapped air voids with diameters up to 0.6 in.

Reinforcement: None present in core.

AGGREGATES (A)

Coarse (C): Manufactured, mostly crystalline slag.

Fine (F): Natural sand composed of siliceous and calcareous rocks and minerals. In approximate order of decreasing abundance: quartz, quartzite, limestone, chert, feldspar, granite, schist, graywacke, and garnet.

Gradation & Top Size: Well graded to 0.8 in. top size.

Shape & Distribution: CA - angular to subrounded, blocky to oblong, and uniformly distributed. FA - rounded to angular, equant to oblong, and uniformly distributed.

PASTE

Color: Mottled shades of brown-gray.

Hardness: Hard to moderately hard.

Luster: Subvitreous to dull locally.

Depth of Carbonation: The paste is unevenly carbonated from the exterior surface to a depth of 0.2 to 0.5 in.; carbonation layer occurs along major crack.

Air Content: Air entrained with 5 to 7% small, spherical, nonuniformly distributed air voids.

Paste-Aggregate Bond: Tight. Lab-induced fractures pass through the coarse aggregates.

Calcium Hydroxide*: 5 to 7% somewhat nonuniformly distributed small crystals, larger tabular crystals and irregular patches.

Unhydrated Portland Cement Clinker Particles (UPC's)*: 15 to 22% nonuniformly distributed UPC's.

Pozzolans*: None observed other than particles presumably derived from the coarse aggregate.

Secondary Deposits: Abundant ettringite partly filling air voids. Calcium hydroxide and secondary calcite occur in a few voids. Alkali-silica gel fills microcracks in and near chert particles. Interlayered gel and carbonate line the longitudinal crack.

MICROCRACKING: A few short microcracks radiating from reactive chert into the paste. Elsewhere a few narrow, random microcracks.

ESTIMATED WATER-CEMENT RATIO: 0.38 to 0.43.

MISCELLANEOUS:

1. Many chert particles show incipient ASR.
2. A few chert particles have gel-lined microcracks and cracks radiating a short distance into the paste.
3. Calcium hydroxide is leached from the paste adjacent to the longitudinal crack.
4. Paste at the exterior end of the core exhibits characteristics of late hydration of cement, i.e. ferrite phase hydration.

*percent by volume of paste

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PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE, ASTM C 856

CTL PROJECT NO.: 050860

CLIENT: Michigan Department of Transportation

STRUCTURE: Bridge Pier Caps

LOCATION: I-75 Bridge
Rouge River, Detroit, Michigan

DATE: May 3, 1996

REPORTED PROBLEM: Cracking

EXAMINED BY: L. Powers-Couche

SAMPLE

Identification: Pier 39 South Face #2 North Bound.

Dimensions: Diameter = 3.9 in. Length = 12 in.

Exterior Surface: Even, paste-rich, formed surface with scattered, irregular air voids (bugholes) and a narrow crack (0.01 in. wide) extending across the surface.

Interior Surface: Irregular surface broken through aggregates.

Cracks, Joints, Large Voids: One major crack, tightening with depth, extends to a depth of 9 in. The crack passes through several coarse aggregates. No joints are present. The concrete contains scattered entrapped air voids with diameters up to 0.3 in.

Reinforcement: None present in core.

AGGREGATES (A)

Coarse (C): Manufactured, mostly crystalline slag.

Fine (F): Natural sand composed of siliceous and calcareous rocks and minerals. In approximate order of decreasing abundance: quartz, quartzite, limestone, chert, feldspar, granite, schist, graywacke, and garnet.

Gradation & Top Size: Well graded to 0.8 in. top size.

Shape & Distribution: CA - angular to subrounded, blocky to oblong, and uniformly distributed. FA - rounded to angular, equant to oblong, and uniformly distributed.

PASTE

Color: Mottled shades of brown-gray.

Hardness: Hard to moderately hard.

Luster: Subvitreous to dull locally.

Depth of Carbonation: The paste is carbonated to a depth of 0.1 to 0.3 in. from the exterior surface and adjacent to the longitudinal crack in the outer 1 in. of the core.

Air Content: Air entrained with 5 to 7% small, spherical, nonuniformly distributed air voids.

Paste-Aggregate Bond: Tight. Lab-induced fractures pass through the coarse aggregates.

Calcium Hydroxide*: 5 to 7% somewhat nonuniformly distributed small crystals, larger tabular crystals and irregular patches.

Unhydrated Portland Cement Clinker Particles (UPC's)*: 15 to 22% nonuniformly distributed UPC's.

Pozzolans*: Small amounts of slag presumably derived from the coarse aggregate. These particles do not exhibit the "fineness" or optical properties of GBFS.

Secondary Deposits: Abundant ettringite in air voids and alkali-silica gel in microcracks.

MICROCRACKING: Random microcracks associated with ASR and passing through fine aggregates are abundant. A network of microcracks, parallel to the major longitudinal crack, occurs on both sides of the crack.

ESTIMATED WATER-CEMENT RATIO: 0.38 to 0.43.

MISCELLANEOUS: Chert sand particles may be somewhat more abundant in this sample and most have reacted or show incipient reaction. Gel-soaked paste occurs around many aggregates, voids, and cracks.

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*percent by volume of paste

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The University of Michigan
2340 Brown Building
An Arbor
Michigan 48109-2125
USA

Att. Dr. Will Hansen

18. June 1996

Ref. 9624will.rap
Case: 625-96

Dear Will

Subject: Slag concrete cores from I-75 Rouge River Bridge, Detroit

We have now made petrographic analysis of the 2 concrete cores received from you and have the following conclusions:

1. The concrete is distressed due to alkalisilica reactions. Alkalisilica gel is present in airvoids and along cracks.

2. The reactive particles are identified as different variants of chert. (Opaline chert, porous chalcedony). The chert has a grain size of approximately 1 to 4 mm.

3. The slag aggregates do not show any sign of reaction and are therefore determined as innocuous in this concept.

4. The petrographic analysis of slag aggregates indicates that slag particles are different in macro porosity. The slag particles are in general build up of crystalline phases probably with Mullite ($3Al_2O_3 \cdot SiO_2$) as main mineral.

5. The concrete contains at least 2 generations of crack evidence. Coarse cracks are present with smooth appearance and are interpreted as developed in early age. Fine cracks and microcracks in connection with reactive chert have a sharp appearance and cut aggregate particles. These cracks are interpreted to be a result of alkalisilica reactions.

The alkalisilica reaction present in this concrete is very similar to the reaction type present in Denmark as the reactive particles are chert minerals. In Danish concrete specification the maximum amount of reactive chert is 2 vol. % determinate with petrographic analysis of 0/4 A37

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mm sand. The analysed concrete has presumably more than 2 % reactive chert particles in size fraction 0/4, but no analysis has been carried out.

Yours Sincerely

PC Laboratoriet A/S

Ole Yde

Analysis

The 2 concrete samples have undertaken fluorescence epoxy impregnation before preparation of plan section, fluorescence macro plan sections and thin sections for petrographic analysis. The petrographic analysis is divided into a macroanalysis (plan sections and fluorescence plan section) and microanalysis (thin sections).

Core sample 625-96-1

Macroanalysis

The core sample has a length of approximately 177 mm with a diameter of approximately 97 mm and consist of mainly subangular aggregates with grain size up to approximately 30 mm in a grey cement paste. The larger aggregates are present as slag particles with different macro porosity. The smaller aggregate (grain size less than 5 mm) are present as rounded mainly white and grey grains.

The concrete core is penetrated by one major crack running perpendicular to the surface though sample section, see photo 1. The crack has a wide of approximately 0.5 mm at the upper approximately 50 mm and has a smooth run and do not penetrate aggregates. At lower level the cracks are narrower and aggregate grains are cut. In the fluorescence plan section the penetrating crack has a crack wide of approximately 0.5 mm through the hole section, see photo 3. In areas in upper section (marked with open arrows in photo 4) separation cracks/elongated airvoids are present. Partly cracked white particles are present (marked with closed arrows) and cracks are penetrating into the cementpaste. Coarse and fine cracks are detected in adhesion zone between aggregate and cementpaste (marked with small closed arrow). In areas slag particles are penetrated by cracks as well.

Microanalysis

The sand aggregates are well distributed and dominated by rounded quartz and feltspatic minerals in fraction size below 2 mm. Chert and calcareous rock minerals are dominated in fraction size 2 - 4 mm. The chert is present as porous opaline together with porous and dense chalcedony. In places chert is mixed with calcareous substance. Several of the chert particles show sign of intense cracking and alkalisilica gel are present in cracks and airvoids close to chert particles, see photo 7, 8, 9, 10. The is a relative high amount of reactive chert particles in the 2 - 4 mm fraction size.

Slag particles are present with different appearing and different macroporosity. Some slagparticles are macrocrystalline with elongate minerals (presumably Mullite) together with more brown particles. Other slag particles are more microcrystalline containing minerals with needle shape. It can not be excluded that slag aggregate with amorphous part are present. In general the slag particles do not show any sign of cracking, but slag particles are cut by cracks. These cracks are in general running from cementpaste into the slag particles.

The cementpaste is inhomogeneous and unhydrated Portland cementminerals are present. At surface cementpaste is carbonated to a depth of approximately 7 mm, but in connection with coarse crack the cementpaste is carbonated to a depth of approximately 30 mm. The water/cement-ratio is relative low and estimated to approximately 0.35 - 0.50. The cementpaste contain a relative moderate amount of airvoids, see enclosed airvoid analysis. In airvoids secondary precipitation's of fibrous minerals and alkalisilica gels are present.

The cementpaste contain a high amount of cracks and the cracks can be dived into at least two generation. At surface coarse crack is present with smooth event. The crack do not cut aggregate particles in upper part, but aggregates (slag) are cut in lower part of section. Fine cracks cutting aggregate and steams of microcracks are present in relation to reactive chert aggregates, see photo 7, 8, 9, 10.

<u>Core sample 625-96-2</u>

Macroanalysis

The core sample has a length of approximately 270 mm with a diameter of approximately 97 mm and consist of mainly subangular aggregates with grain size up to approximately 30 mm in a grey cement paste. The larger aggregates are present as slag particles with different macro porosity. The smaller aggregate (grain size less than 5 mm) are present as rounded mainly white, black and grey grains.

The aggregates are inhomogeneous distributed as an area of approximately 30 x 80 mm with out coarse aggregate is present in a depth of approximately 155 mm. The area is cracked with cracks running parallel to surface, see photo 5. Cracks are present in connection with reactive chert grain, see photo 6.

At upper surface a light approximately 2 mm thick layer is present. The layer is only present at surface and do not enter cracks at surface, see photo 2.

At concrete surface two cracks running perpendicular to the surface to a depth of approximately 40 - 50 mm, see photo 2. The cracks have a wide of approximately 0.2 mm and do cut some aggregates. In the fluorescence plan section only one crack at surface is detected, see photo 5. Fine cracks are detected in cement paste and in adhesion zone between aggregate and cementpaste.

Microanalysis

The upper layer (presumably epoxy layer) is build up of an amorphous matrix mixed with up to approximately 0.2 mm subangular quartz grains. Good adhesion is present between underlying cementpaste and layer, but the layer do not penetrate into cracks. A40

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The sand aggregates are well distributed and dominated by rounded quartz, feltspatic and calcareous minerals. Chert and calcareous rock minerals are dominated in fraction size 2 - 4 mm. The chert is present as porous opaline together with porous and dense chalcedony. Chert particles show sign of cracking and alkalisilica gel are present in cracks and airvoids close to chert particles as described in sample 625-96-1. The amount of reaction is not as intensive as detected in sample 625-96-1.

Slag particles are present with same appearing as previous described. In general the slag particles do not show any sign of cracking, but slag particles are cut by cracks.

The cementpaste is inhomogeneous and unhydrated Portland cementminerals are present. At surface cementpaste is carbonated to a depth of approximately 3 mm, but in connection with coarse crack the cementpaste is carbonated to a depth of approximately 30 mm. The water/cement-ratio is relative low and estimated to approximately 0.35 - 0.50. The cementpaste contain a relative moderate amount of airvoids. In airvoids secondary precipitation's of fibrous minerals and alkalisilica gels are present.

The cementpaste contain a high amount of cracks and the cracks can be dived into at least two generation. At surface coarse crack is present with smooth event. The crack do cut some aggregate particles. Fine cracks cutting aggregate and steams of microcracks are present in relation to reactive chert aggregates.

Discussion

The concrete is cracked and contains at least 2 generations of crack evidence. Coarse cracks are present with smooth appearance and are interpreted as developed in early age. In part the coarse cracks cut slag aggregates, but there is no evidence of chemical reaction in connection to slag aggregates.

Fine cracks and microcracks in connection with chert have a sharp appearance and cut aggregate particles. These cracks are interpreted to be a result of alkalisilica reactions as alkalisilica gel is present in connection to chert aggregates, in airvoids and along cracks.

The reactive particles are identified as different variants of chert. (Opaline chert, porous Chalcedony). The chert has a grain size of approximately 1 to 4 mm, but is dominated in size 2 to 4 mm.

The slag aggregates do not show any sign of reaction and are therefore determined as innocuous in this concept. Due to high degree of macroporosity slag particles may have lower or same strength as cementpaste. It is there for very possible that cracks generated in the concrete will have a tendency to cut slag aggregates. The petrographic analysis of slag aggregates indicates that slag particles have different macro porosity. The slag particles are in general build up of crystalline phases probably with Mullite ($3Al_2O_3 \cdot SiO_2$) as main mineral, but more microcrystalline to amorphous slag particles are present. A41

The crack structures indicate, that some of the cracks have been developed in a very early state of the concrete age. These macrocracks have presumably developed further in time due to external condition such as freeze/thaw.

The present of cracks, alkalimetals and high humidity have then later given rise to alkalisilica reaction and there by intense cracking.

The mortar rich area observed in sample 625-96-2 is presumeably a product of non prober workman ship. The high amount of mortar has probaly resulted in the cracking parallel to surface, but alkalisilica reactions are present in this area as well.

In sample 625-96-2 a relative high amount of potential reactive chert aggregates have not reacted and further alkalisilica reaction are therefor possible.

The alkalisilica reaction present in this concrete is very similar to the reaction type present in Denmark as the reactive particles are chert minerals. In Danish concrete specification the maximum amount of reactive chert is 2 vol. % determinate with petrographic analysis of 0-4 mm sand. The analysed concrete has presumably more than 2 % reactive chert particles in size fraction 0-4, but no analysis has been carried out.

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Photo 1. Macrofoto of sample 625-96-1. The concrete contains homogenous distributed slag particles in Portland cement paste. A macro crack penetrates the core perpendicular to surface.

Photo 2. Macrofoto of sample 625-96-2. At surface grey epoxy layer is present. The layer do not penetrate the concrete. Surface crack is present, see arrow.

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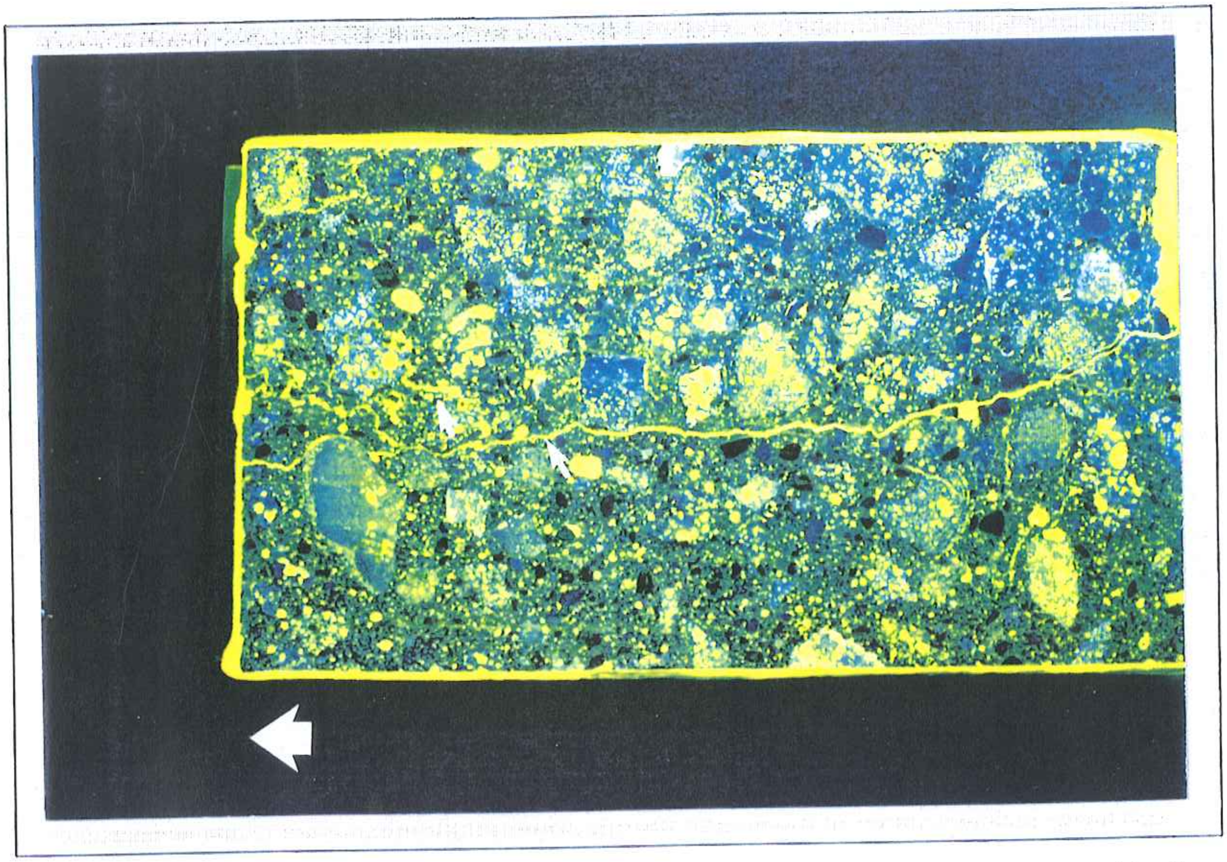
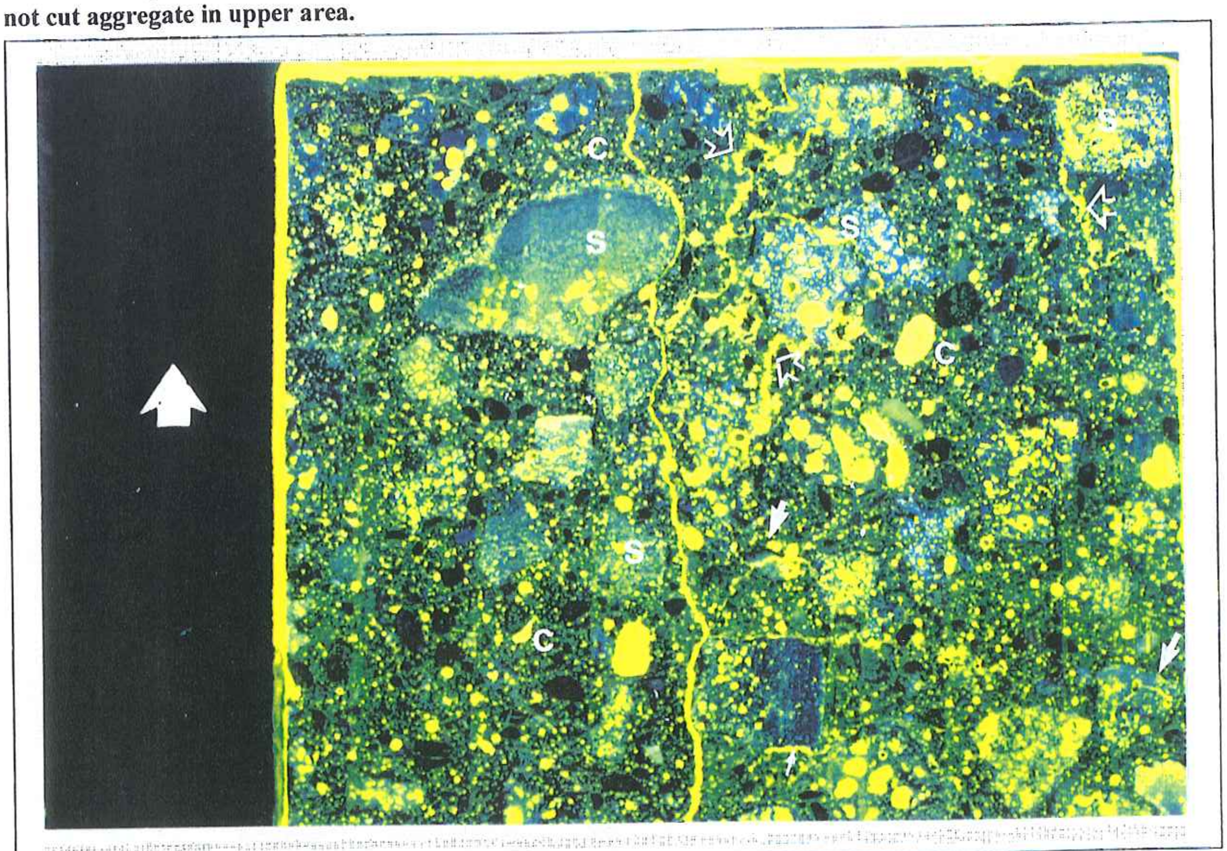


Photo 3. Macrofoto of fluorescence impregnated sample 625-96-1. The total length is represented in the fluorescence photo and cracks are high lighted, see arrows. The macro cracks have a smooth run and do not cut aggregate in upper area.



A44

Photo 4. Close up macrofoto of fluorescence impregnated sample 625-96-1 in upper area. The large arrow indicate surface. Open arrows indicates separation cracks/elongated air voids. Closed arrows indicates cracked white aggregates.

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MÅGEVEJ 7 DK-9690 FJERRITSLEV TLF. +45 98 21 32 00 FAX +45 98 21 34 54
ISLANDS BRYGGE 22 DK-2300 KØBENHAVN S. TLF. +45 31 57 14 22 FAX +45 31 57 48 96

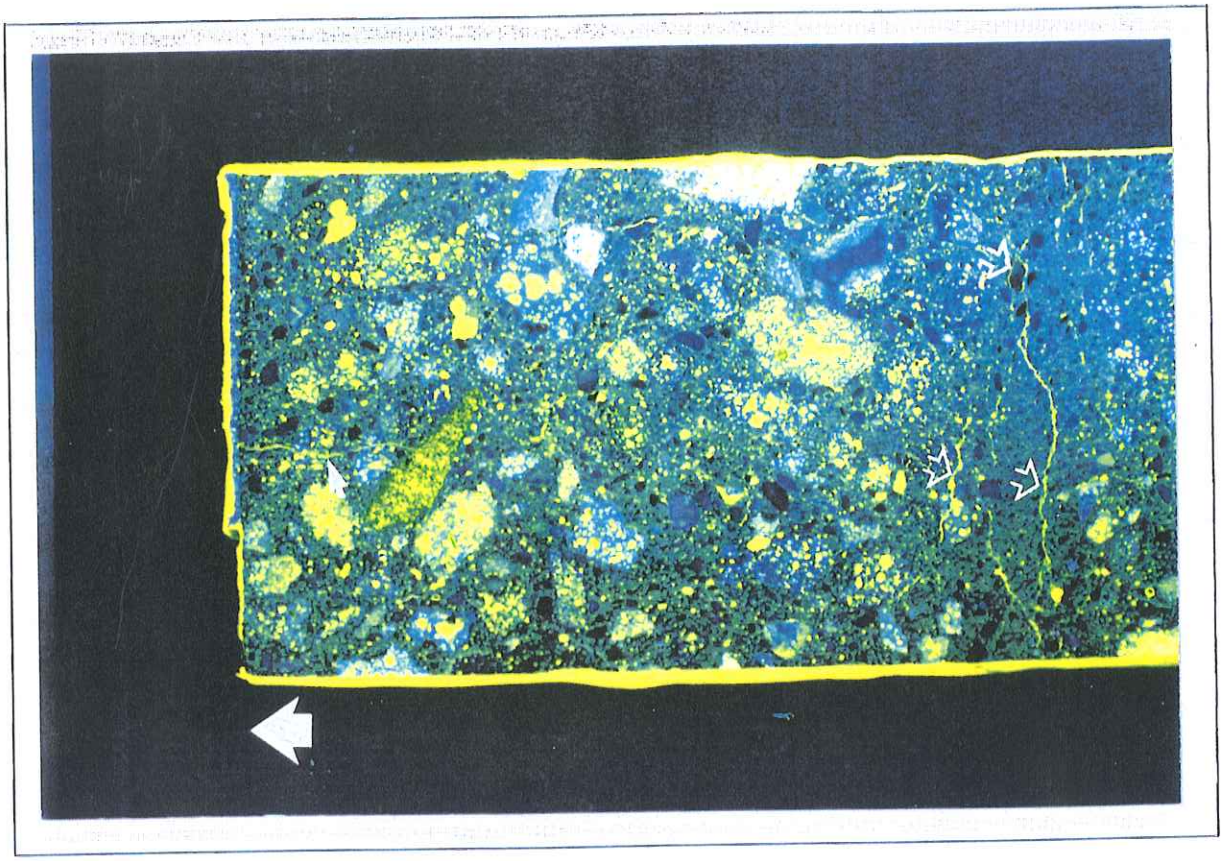

Photo 5. Macrofoto of fluorescence impregnated sample 625-96-2. Macro crack is present at surface, see arrow. In area macro cracks are present orientated parallel to surface, see open arrows.

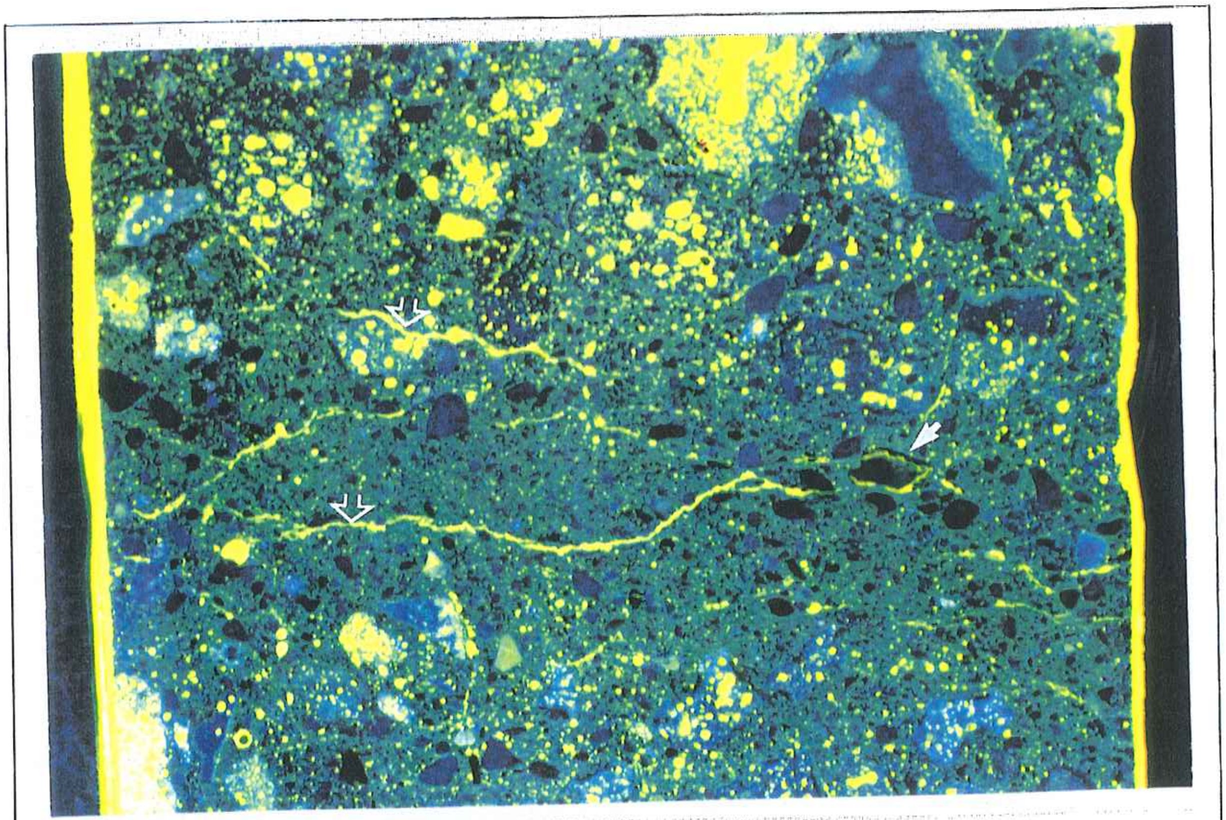

Photo 6. Close up macrofoto of fluorescence impregnated sample 625-96-1 of area with cracks parallel to surface. The cracks are present in mortar rich part. Closed arrows indicates cracked white aggregates. Other cracks have a more smooth run and do not cut aggregates, open arrows.

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ISLANDS BRYGGE 22 DK-2300 KØBENHAVN S. TLF. +45 31 57 14 22 FAX +45 31 57 48 96

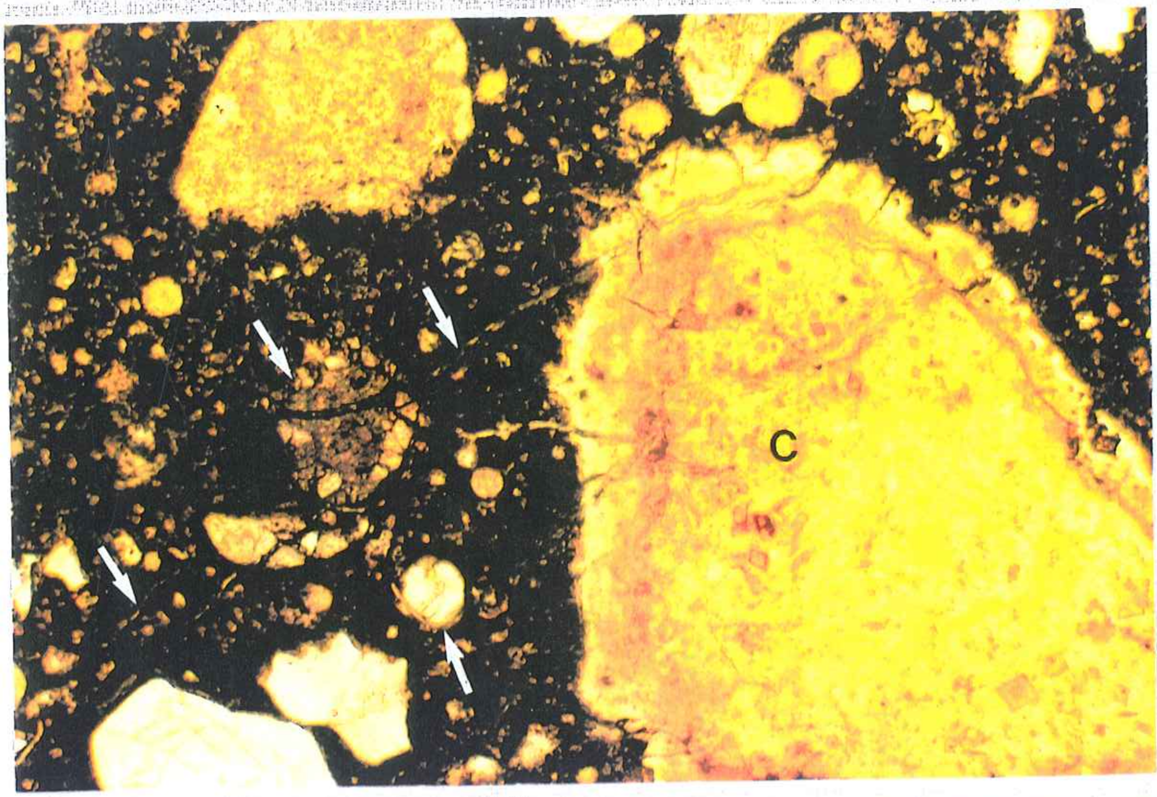

Photo 7. Microphoto sample 625-96-1. Transparent light, 1 cm = 0.26 mm. Porous chert (C) is cracked and alkali silica gel is present in chert, cracks and airvoids, see arrows.

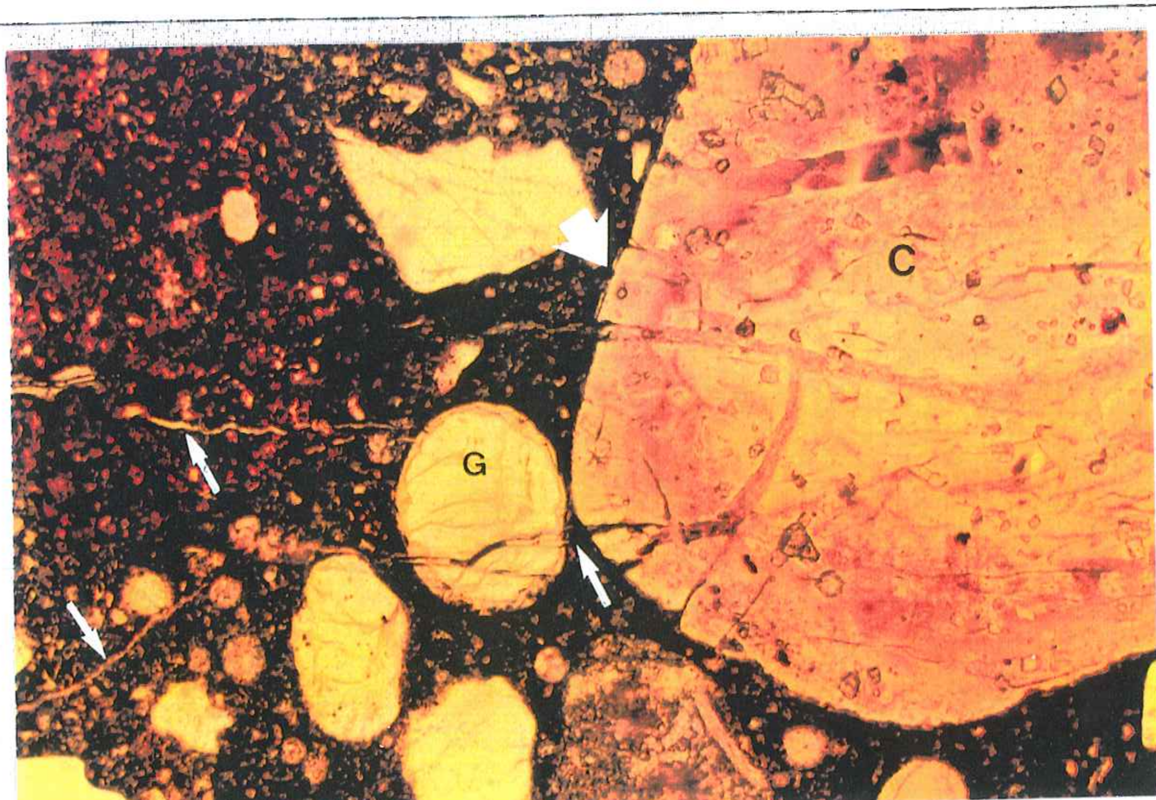

Photo 8. Microphoto sample 625-96-1. Transparent light, 1 cm = 0.26 mm. Porous chert (C) is cracked and alkali silica gel is present in chert, cracks, see arrows. Alkali silica gel is present in airvoids (G).

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ISLANDS BRYGGE 22 DK-2300 KØBENHAVN S. TLF. +45 31 57 14 22 FAX +45 31 57 48 96

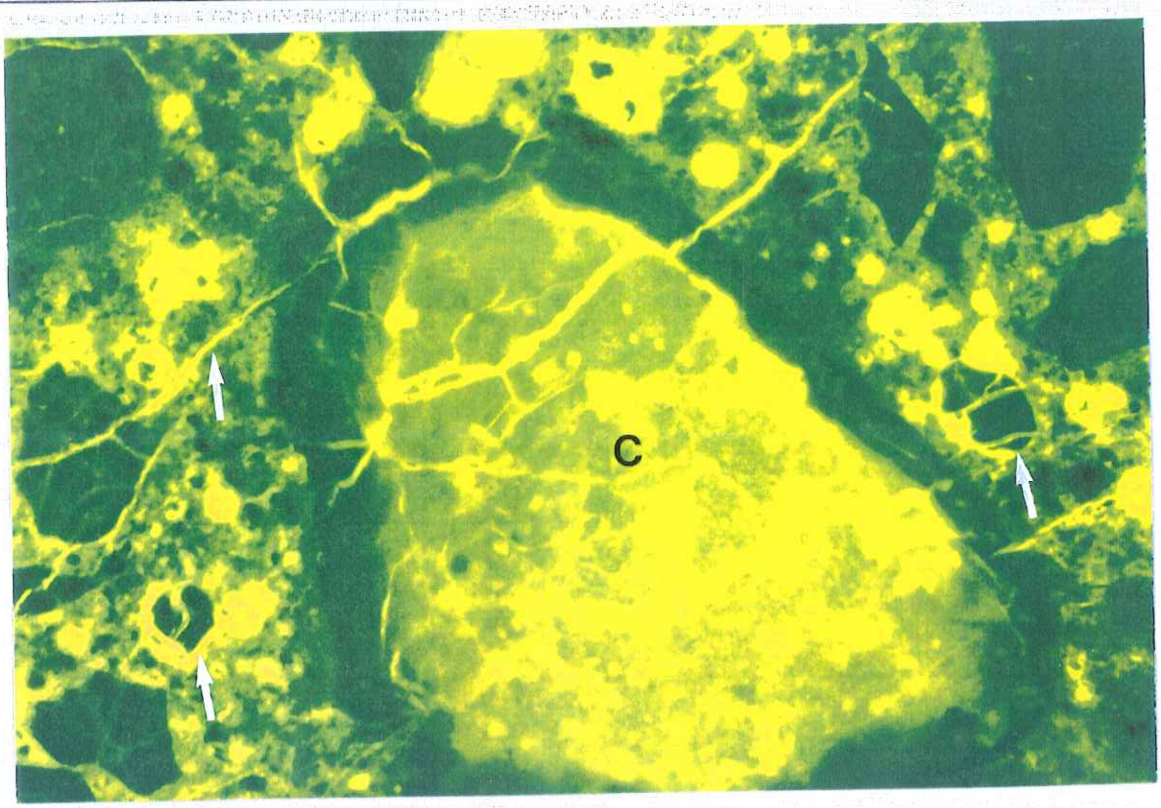


Photo 7. Microphoto sample 625-96-1. Transparent light, 1 cm = 0.11 mm. Porous chert (C) is cracked and alkali silica gel is present in chert, cracks and airvoids, see arrows.

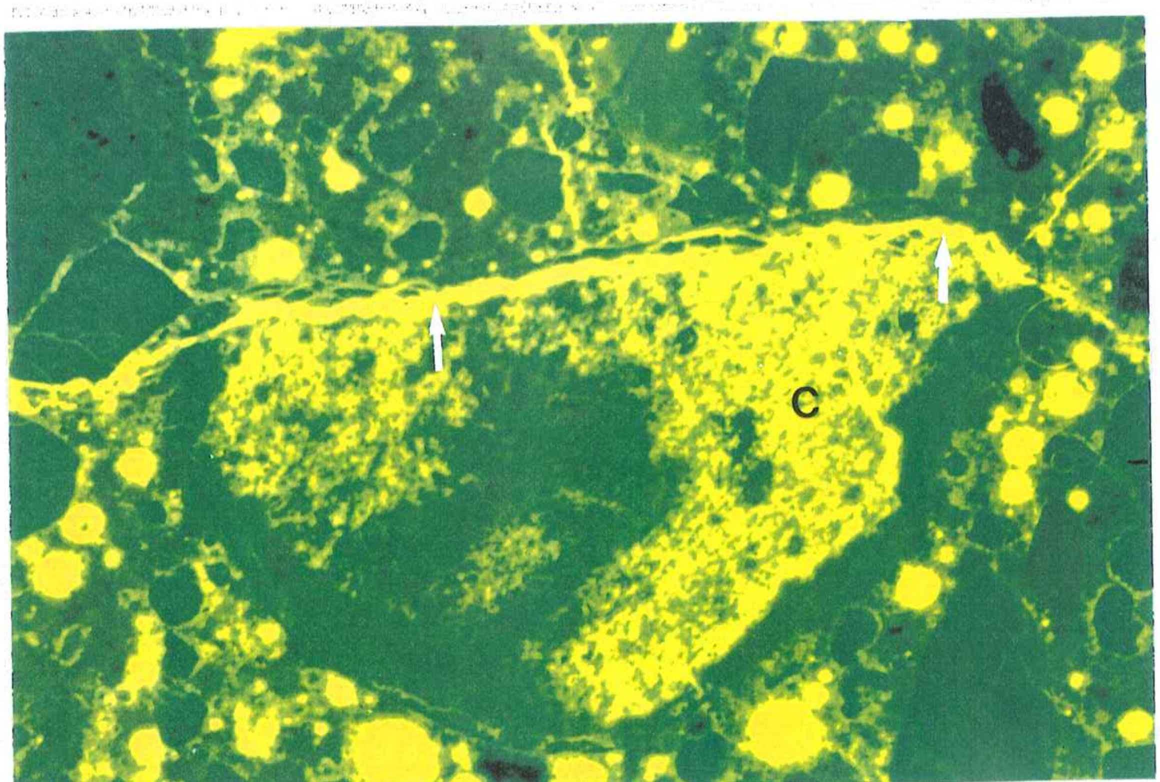


Photo 8. Microphoto sample 625-96-1. Transparent light, 1 cm = 0.26 mm. Porous chert (C) is cracked and alkali silica gel is present in chert, cracks, see arrows.

PC LABORATORIET A/S
MÅGEVEJ 7 9690 FJERRITSLEV
TLF.NR. 98 21 32 00

The University of Michigan
2340 Brown Building
An Arbor
Michigan 48109-2125

Date: 13/06/96

Report No.: 618-96

Lab. No.: 507-96

DANAK
Reg.nr. 179

TEST REPORT

Client	The University of Michigan
Sample identification	Slag concrete cores from I-75 Rouge River Bridge, Detroit.
Methods, results	Measurement af air voids in hardened concrete TI-B 4.

Kind regards

PC LABORATORIET A/S

Signature

Elo Yde

PC LABORATORIET A/S
MÅGEVEJ 7 9690 FJERRITSLEV

AUTORISATIONSREG. NR. 179
DATO 13/06/96

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE
LAB. NO.: 507-96-1
REPORT NO. : 618-96
SAMPLE LABEL: I-75 ROUGE RIVER BRIDGE

AIR-VOID ANALYSIS

Perf. by FS

TOTAL AIR CONTENT	(vol%):	3.6
AIR IN VOIDS > 2 mm	(vol%):	0.3
SPECIFIC SURFACE	(mm-1):	39
SPACING FACTOR	(mm):	0.14
AIR IN BINDER	(vol%):	11.6
PASTE VOLUME	(vol%):	27.5

Comments:
POROUS SLAG PARTICLES ARE PAINTED OVER THEREBY NOT INCLUDED IN ANALYSIS.

Appendix B.

Concrete Strength and Elastic Modulus

Static Modulus of Elasticity of Rouge River Bridge Concrete.

(Drilled Cores)

Specimen: **Pier 39 S.F. #4**

Diameter: 5.9 in — Corr.Factor: 0.980

Capped Length: 10.31 in — X-Section Area: 27.34 in2

l/d Ratio: 1.75

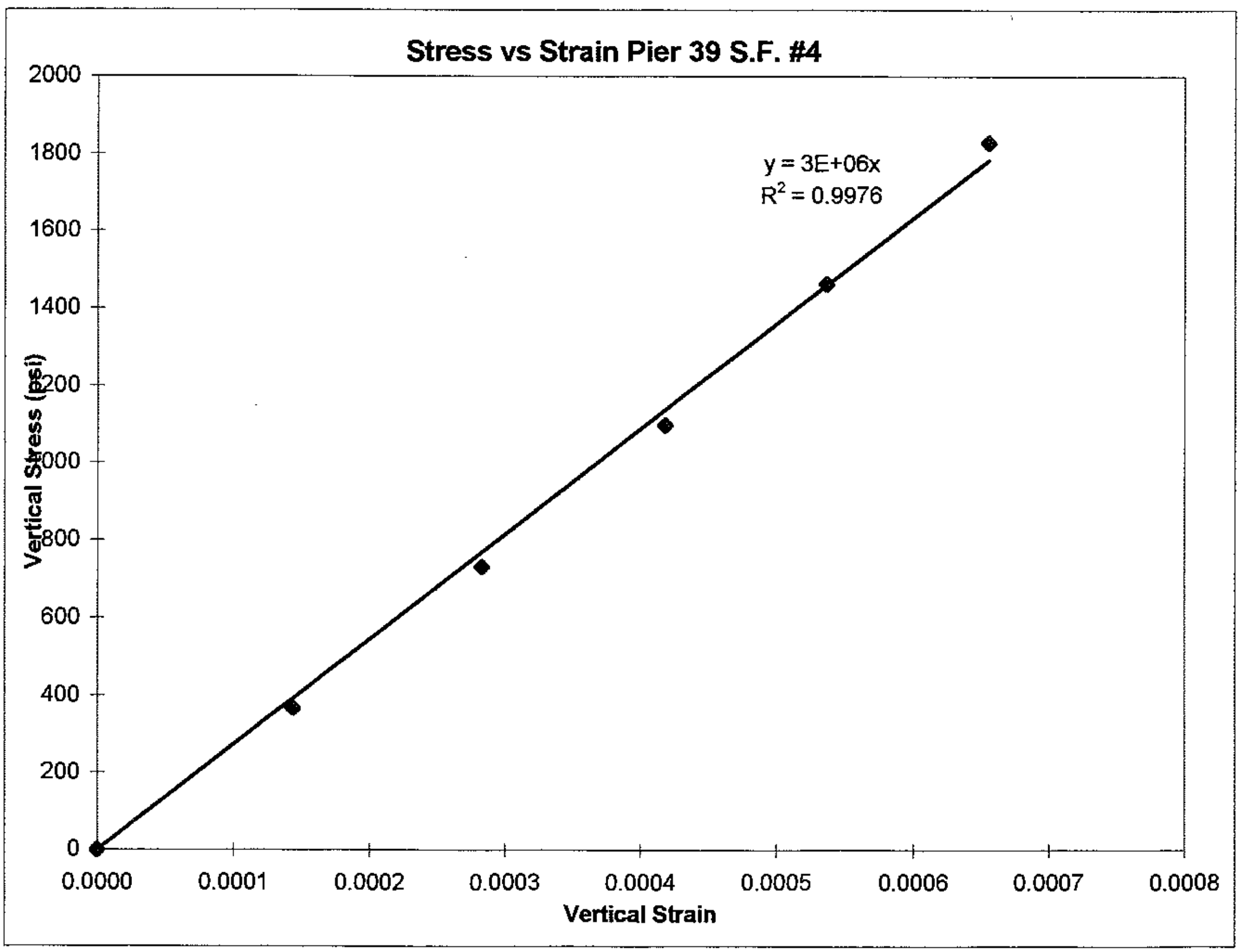
Dist. between points of control: 6.1 in

Load lb	Vert.Gage(in) Trial 1	 Trial 2	 Average	Vert.Defl. dv (in)	Vert.Strn. (--)	Vert.Stress psi	E Static psi
0	0.0000	0.0000	0.0000	0.00000	0.00000	0.00	
10000	0.0017	0.0018	0.0018	0.00088	0.00014	365.77	2.55E+06
20000	0.0033	0.0036	0.0035	0.00173	0.00028	731.54	2.59E+06
30000	0.0049	0.0053	0.0051	0.00255	0.00042	1097.30	2.62E+06
40000	0.0064	0.0067	0.0066	0.00328	0.00054	1463.07	2.73E+06
50000	0.0078	0.0082	0.0080	0.00400	0.00066	1828.84	2.79E+06
						Average Elastic Mod:	2.66E+06

Ult. Load (kip): 159.6

Ult. Strength (psi): 5838

Corr Strength (psi) 5721



Static Modulus of Elasticity of Rouge River Bridge Concrete.

(Drilled Cores)

Specimen: **Pier 39 S.F. #2**

Diameter: 5.92 in — Corr.Factor: 0.990

Capped Length: 11 in — X-Section Area: 27.53 in2

l/d Ratio: 1.86

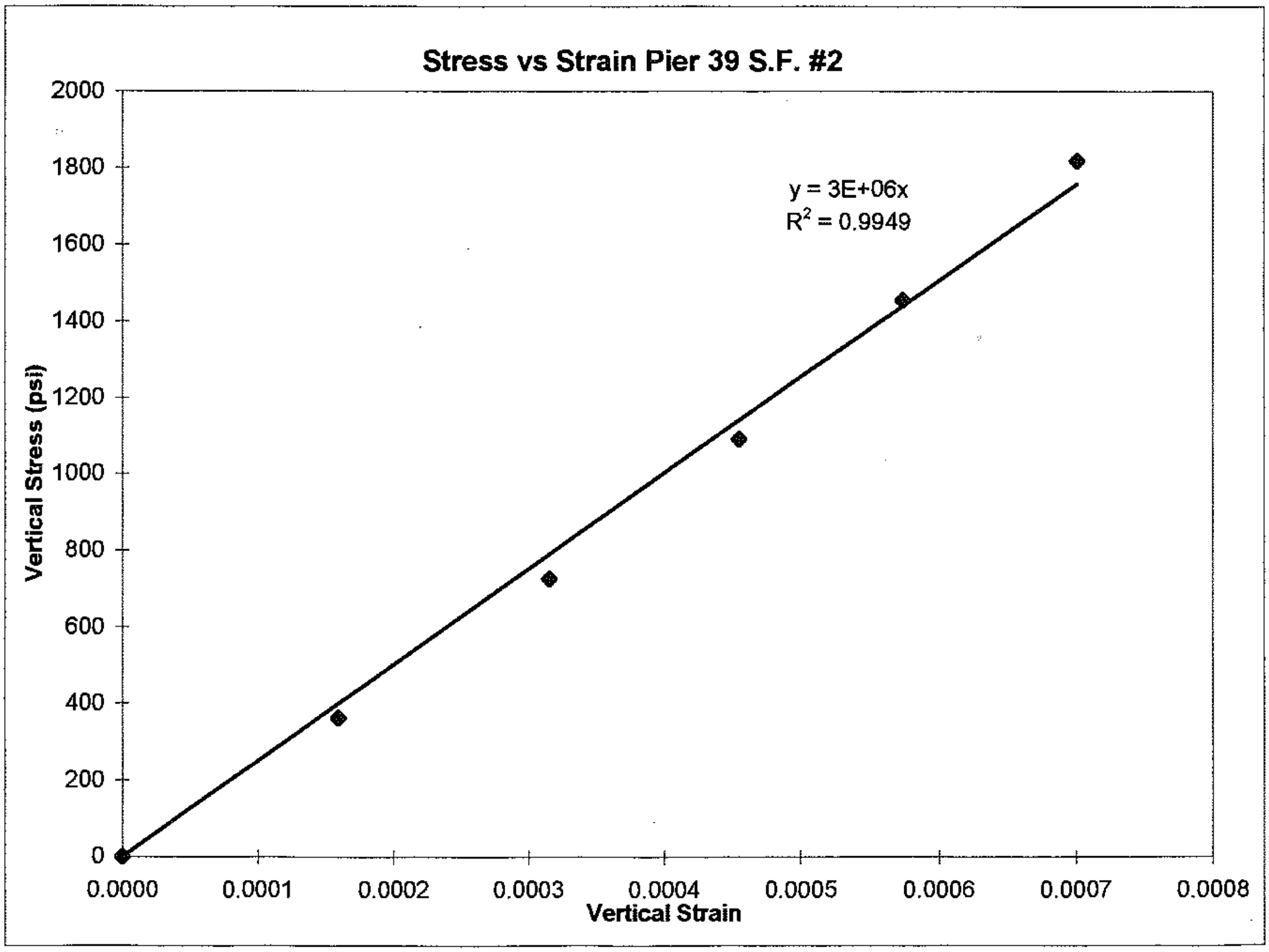
Dist. between points of control: 6.1 in

Load	Vert.Gage(in)			Vert.Defl.	Vert.Strn.	Vert.Stress	E Static
lb	Trial 1	Trial 2	Average	dv (in)	(--)	psi	psi
0	0.0000	0.0000	0.0000	0.00000	0.00000	0.00	
10000	0.0018	0.0021	0.0020	0.00098	0.00016	363.30	2.27E+06
20000	0.0039	0.0038	0.0039	0.00193	0.00032	726.60	2.30E+06
30000	0.0056	0.0055	0.0056	0.00278	0.00045	1089.90	2.40E+06
40000	0.0070	0.0070	0.0070	0.00350	0.00057	1453.20	2.53E+06
50000	0.0086	0.0085	0.0086	0.00428	0.00070	1816.51	2.59E+06
					Average Elastic Mod:		2.42E+06

Ult. Load (kip): 141.5

Ult. Strength (psi): 5141

Corr Strength (psi) 5089



Static Modulus of Elasticity of Reference Slag Concrete.

(Reference Batch Specimens - Levy Pit Slag Pit 82-22)

Specimen: **Reference #3**

Diameter:	6 in	Corr.Factor:	1.000
Capped Length:	12.2 in	X-Section Area:	28.27 in2
l/d Ratio:	2.03		

Dist. between points of control: 6.1 in

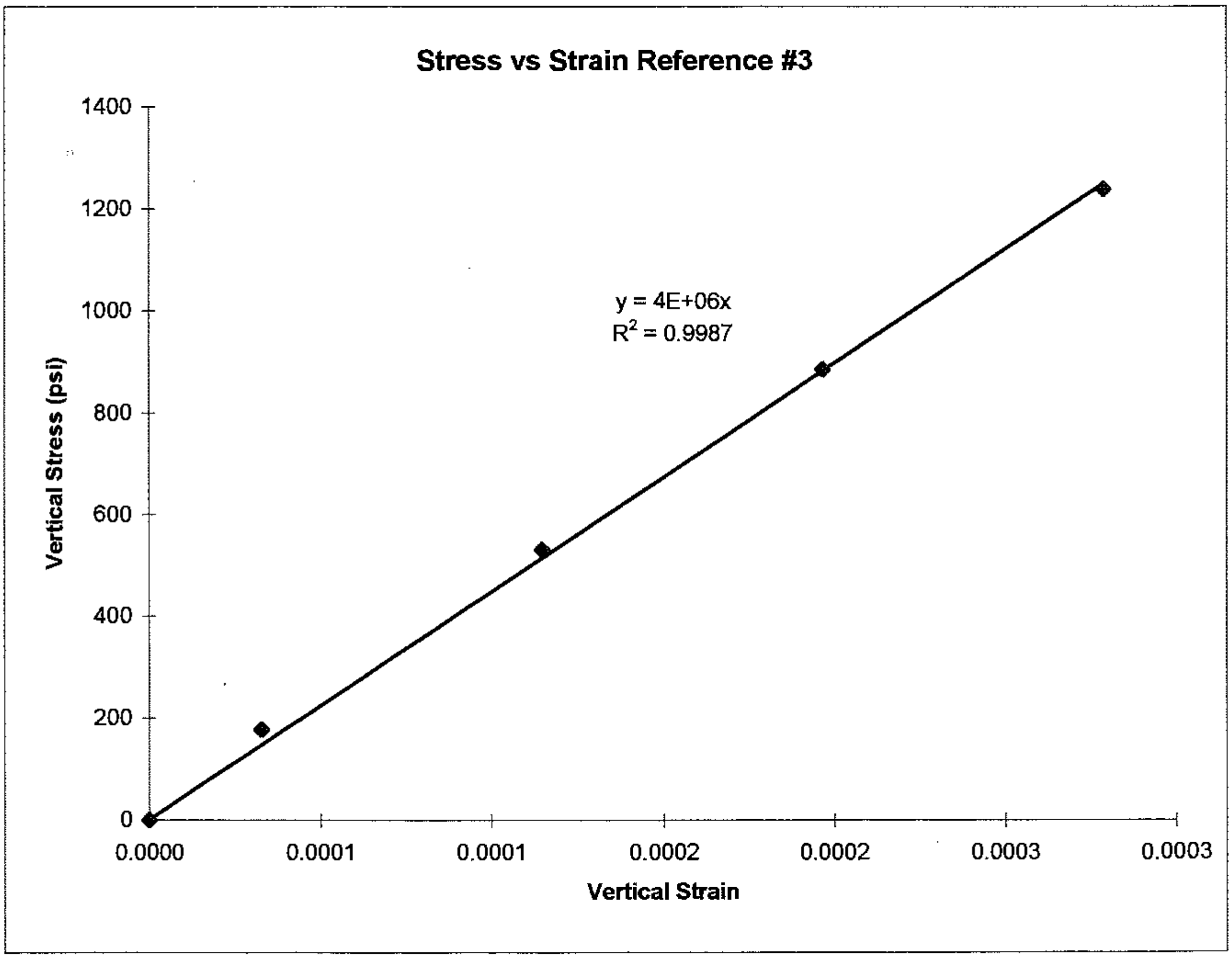
Load	Vert.Gage(in)			Vert.Defl.	Vert.Strn.	Vert.Stress	E Static
lb	Trial 1	Trial 2	Average	dv (in)	(--)	psi	psi
0	0.0000	0.0000	0.0000	0.00000	0.00000	0.00	
5000	0.0004	0.0004	0.0004	0.00020	0.00003	176.84	5.39E+06
15000	0.0014	0.0014	0.0014	0.00070	0.00011	530.52	4.62E+06
25000	0.0024	0.0024	0.0024	0.00120	0.00020	884.19	4.49E+06
35000	0.0034	0.0034	0.0034	0.00170	0.00028	1237.87	4.44E+06

Ult. Load (kip): 131.3

Ult. Strength (psi): 4644

Corr Strength (psi) 4644

Average Elastic Mod: 4.74E+06



Static Modulus of Elasticity of Reference Slag Concrete.

(Reference Batch Specimens - Levy Pit Slag Pit 82-22)

Specimen: **Reference #4**

Diameter:	6 in		Corr.Factor:	1.000
Capped Length:	12.2 in		X-Section Area:	28.27 in2
l/d Ratio:	2.03			

Dist. between points of control: 6.1 in

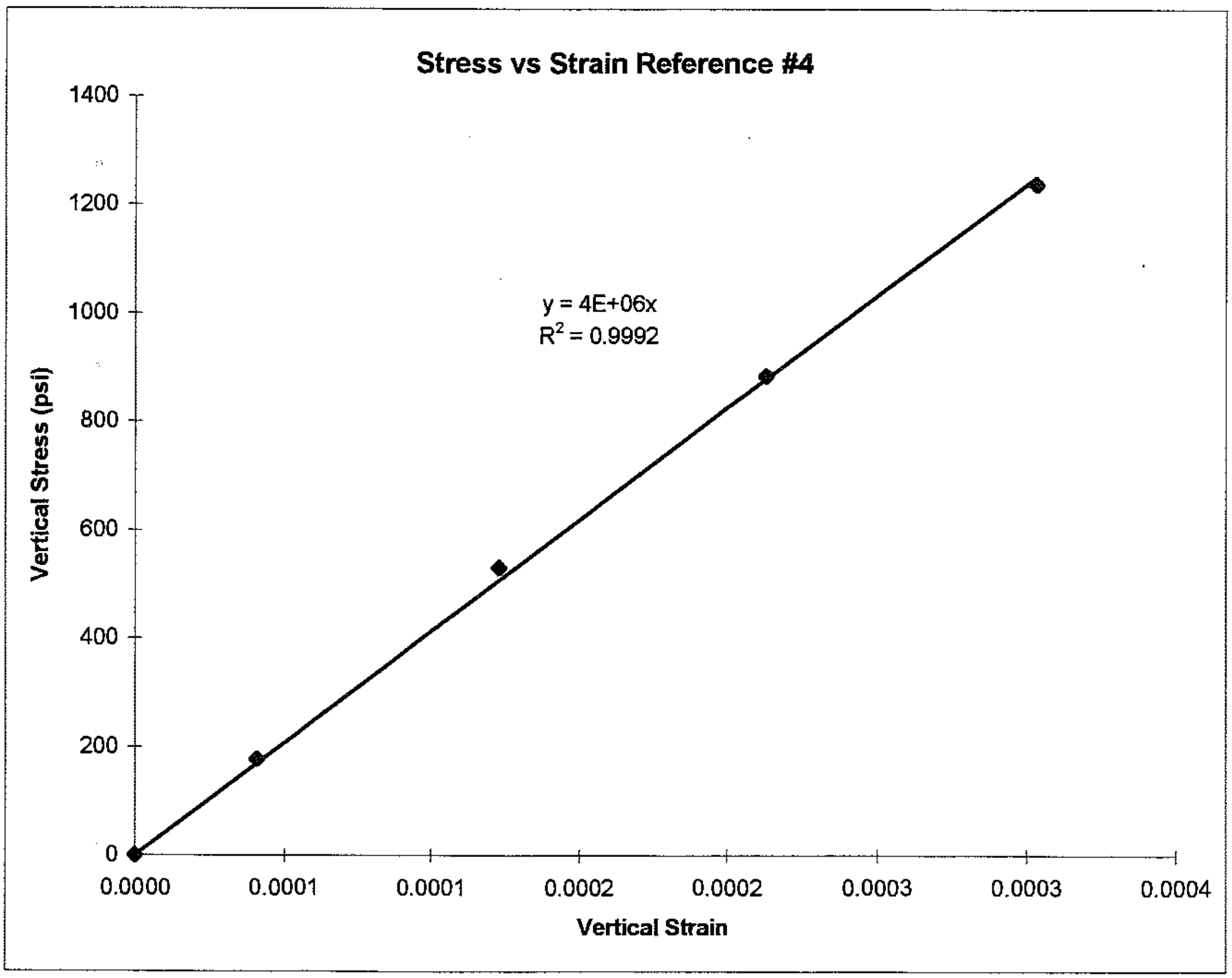
Load	Vert.Gage(in)			Vert.Defl.	Vert.Strn.	Vert.Stress	E Static
lb	Trial 1	Trial 2	Average	dv (in)	(--)	psi	psi
0	0.0000	0.0000	0.0000	0.00000	0.00000	0.00	
5000	0.0005	0.0005	0.0005	0.00025	0.00004	176.84	4.31E+06
15000	0.0015	0.0015	0.0015	0.00075	0.00012	530.52	4.31E+06
25000	0.0026	0.0026	0.0026	0.00130	0.00021	884.19	4.15E+06
35000	0.0037	0.0037	0.0037	0.00185	0.00030	1237.87	4.08E+06

Average Elastic Mod: 4.22E+06

Ult. Load (kip): 135.2

Ult. Strength (psi): 4782

Corr Strength (psi) 4782



Static Modulus of Elasticity of Reference Slag Concrete.

(Reference Batch Specimens - Levy Pit Slag Pit 82-22)

Specimen: **Reference #6**

Diameter:	6 in	Corr.Factor:	1.000
Capped Length:	12.2 in	X-Section Area:	28.27 in2
l/d Ratio:	2.03		

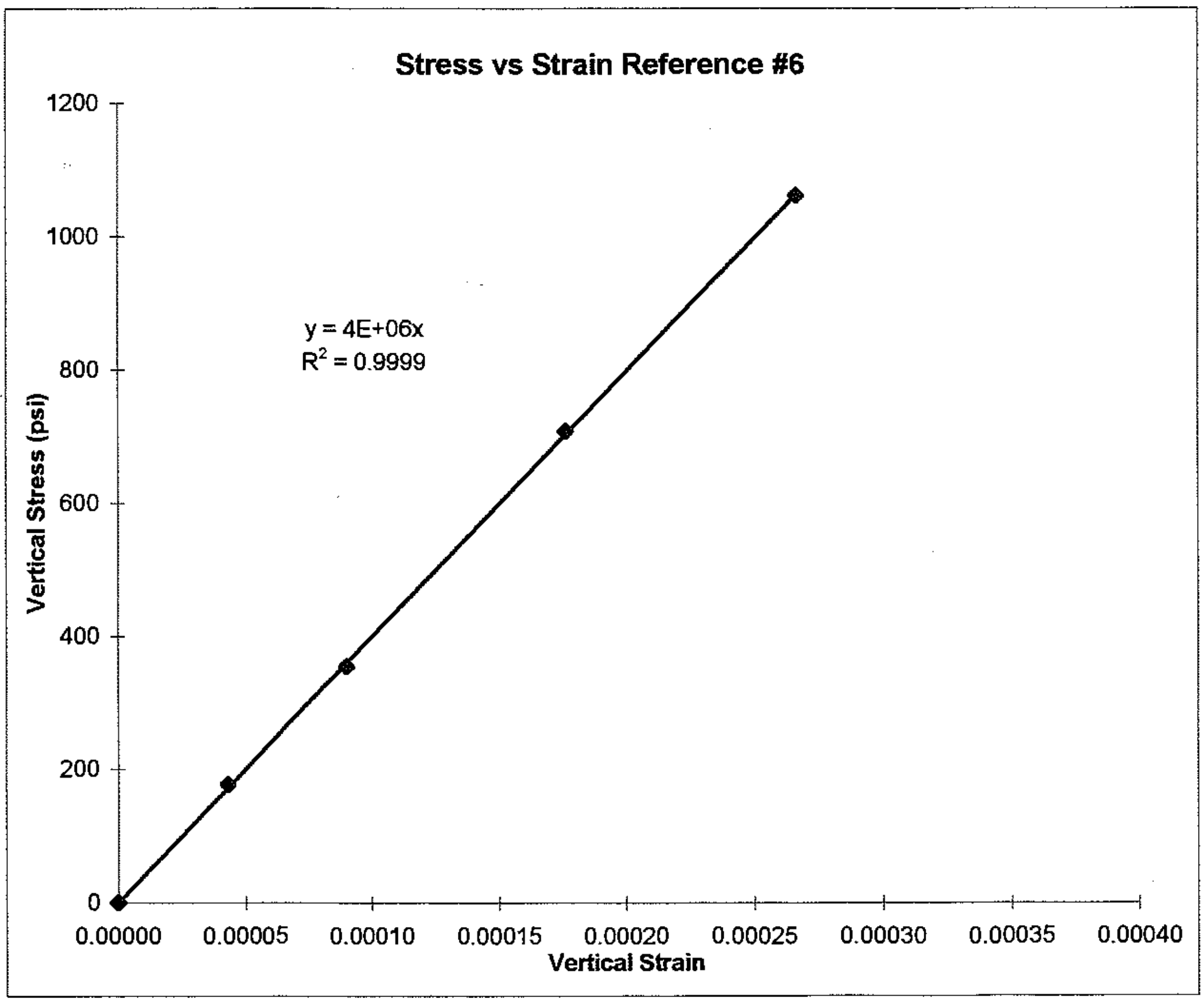
Dist. between points of control: 6.1 in

Load	Vert.Gage(in)			Vert.Defl.	Vert.Strn.	Vert.Stress	E Static
lb	Trial 1	Trial 2	Average	dv (in)	(--)	psi	psi
0	0.0000	0.0000	0.0000	0.00000	0.00000	0.00	
5000	0.0005	0.0006	0.0005	0.00026	0.00004	176.84	4.11E+06
10000	0.0011	0.0011	0.0011	0.00055	0.00009	353.68	3.92E+06
20000	0.0022	0.0021	0.0022	0.00108	0.00018	707.36	4.01E+06
30000	0.0032	0.0033	0.0033	0.00163	0.00027	1061.03	3.98E+06
					Average Elastic Mod:		4.01E+06

Ult. Load (kip): 134.3

Ult. Strength (psi): 4750

Corr Strength (psi) 4750



Client: University of Michigan
Project: I-75 Bridge Over Rouge River, Detroit
Contact: Dr. Will Hansen
Submitter: R. Detwiler

CTL Project No.: 050860
CTL Project Mgr.: R. Detwiler
Technician: P. Brindise
Approved: W. Morrison
Date: July 16, 1996

Test Results of ASTM C 42-90 Standard Test Method for
Compressive Strength of Drilled Cores of Concrete

Core Identification	Pier 38, N.F. #4	Pier 38, N.F. #3		
Nominal Maximum Aggregate Size (in.)	1	1		
Concrete Age at Test (approximate years)	Not stated	Not stated		
Moisture Condition at Test	As received-dry	As received-dry		
Orientation of Core Axis in Structure	Not stated	Not stated		
Diameter 1, (in.)	3.91	3.91		
Diameter 2, (in.)	3.90	3.90		
Average Diameter (in.)	3.90	3.91		
Cross-Sectional Area (sq in.)	11.97	11.98		
Length Trimmed (in.)	7.6	7.6		
Length Capped (in.)	7.8	7.8		
Weight in Air (lb)	7.1	7.2		
Weight in Water (lb)	3.8	3.9		
Calculated Unit Weight (pcf)	135.5	136.2		
Loading Rate, psi/sec	35	35		
Maximum Load (lb)	99,300	104,000		
Uncorrected Compressive Strength (psi)	8,290	8,680		
Ratio of Capped Length to Diameter (L/D)	2.00	2.01		
Correction Factor (ASTM C42)	1.000	1.000		
Corrected Compressive Strength (psi)	**8,290**	**8,680**		
Fracture Pattern	cone and split	shear		
Notes:				

Appendix C.

Chloride Analysis

CTL Structural/Architectural Engineering, Consulting, & Materials Technology

5420 Old Orchard Road, Skokie, Illinois 60077-1030

708/965-7500 **800/522-2CTL** Fax: 708/965-6541

Client:	**University of Michigan**	CTL Project No:	**050860**
Project:	**Concrete Evaluation Piers 38 and 39**	CTL Project Mgr.:	**Laura Powers-Couche**
		Analyst:	**Yea Yoon**
Contact:	**Dr. Will Hansen**	Approved:	
Submitter:	**Laura Powers-Couche, CTL**	Date Reported:	**8-Mar-96**
Date Rec'd:	**16-Feb-96**	Date Analyzed:	**6-Mar-96**

REPORT of CHLORIDE ANALYSIS

Sample Identification			Determined Water-Soluble Chloride
CTL ID	Client ID	Description	(wt% of sample)
918729	Pier 39 #3 NB 1 in.		0.015
919003	Pier 39 #3 NB 3 in.		0.007
918730	Pier 39 #3 NB 4 in.		0.007
918731	Pier 39 #3 NB 8 in.		0.005
918732	Pier 39 #3 SB 1 in.		0.023
919004	Pier 39 #3 SB 3 in.		0.005
918733	Pier 39 #3 SB 4 in.		0.006
918734	Pier 39 #3 SB 8 in.		0.003
918735	Pier 39 #5 NB 1 in.		0.018
919005	Pier 39 #5 NB 3 in.		0.003
918736	Pier 39 #5 NB 4 in.		0.007
918737	Pier 39 #5 NB 8 in.		0.003
918738	Pier 39 #5 SB 1 in.		0.006
919006	Pier 39 #5 SB 3 in.		0.006
918739	Pier 39 #5 SB 4 in.		0.006
918740	Pier 39 #5 SB 8 in		0.006

Notes:
1. This analysis represents specifically the samples submitted on a dry (105° C) basis.
2. This report may not be reproduced except in its entirety.
3. Analysis by potentiometric titration with silver nitrate. (ASTM C1218).

CTL Structural/Architectural Engineering, Testing and Materials Technology

5420 Old Orchard Road, Skokie, Illinois 60077-1030
847/ 965-7500 **800/ 522-2CTL** Fax: 847/ 965-6541

Client:	**University of Michigan**	CTL Project No:	**050860**
Project:	**Chloride Analysis**	CTL Project Mgr.:	**Rachel J. Detwiller**
	MDOT / University of Michigan	Analyst:	**M. Bharucha** MB
Contact:	**Dr. Will Hansen**	Approved:	DrBl
Submitter:	**Rachel J. Detwiller, CTL**	Date Reported:	**19-Jul-96**
Date Rec'd:	**11-Jul-96**	Date Analyzed:	**19-Jul-96**

REPORT of CHLORIDE ANALYSIS

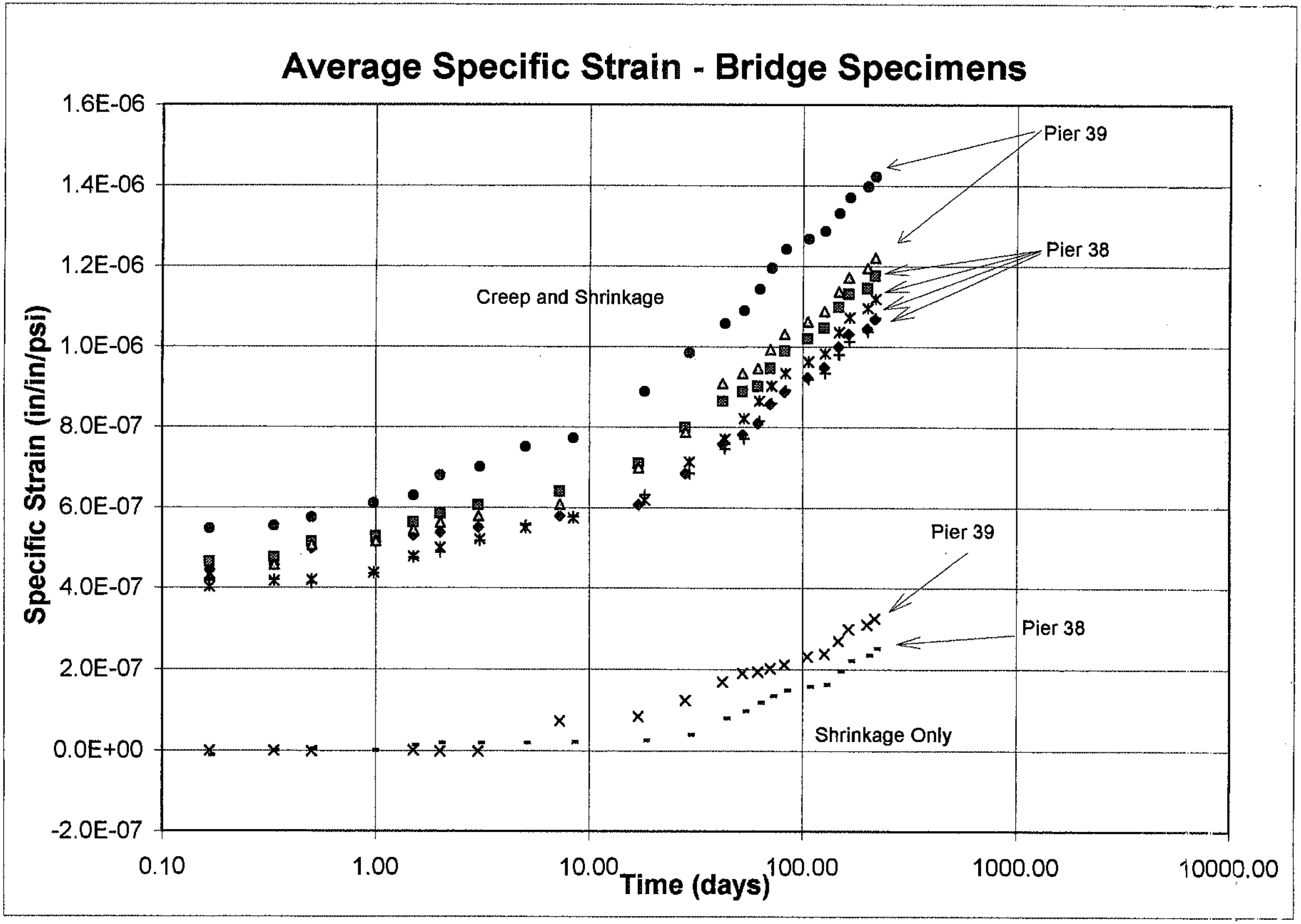
Sample Identification			Determined Water-Soluble Chloride
CTL ID	Client ID	Description	(wt% of sample)
920832	Pier 38 SF #2 1 in.		0.071
920833	Pier 38 SF #2 3 in.		0.072
920834	Pier 38 SF #2 4 in.		0.055
920835	Pier 38 SF #2 8 in.		0.066
920836	Pier 38 NF #1 1 in.		0.070
920837	Pier 38 NF #1 3 in.		0.063
920838	Pier 38 NF #1 4 in.		0.054
920839	Pier 38 NF #1 8 in.		0.042

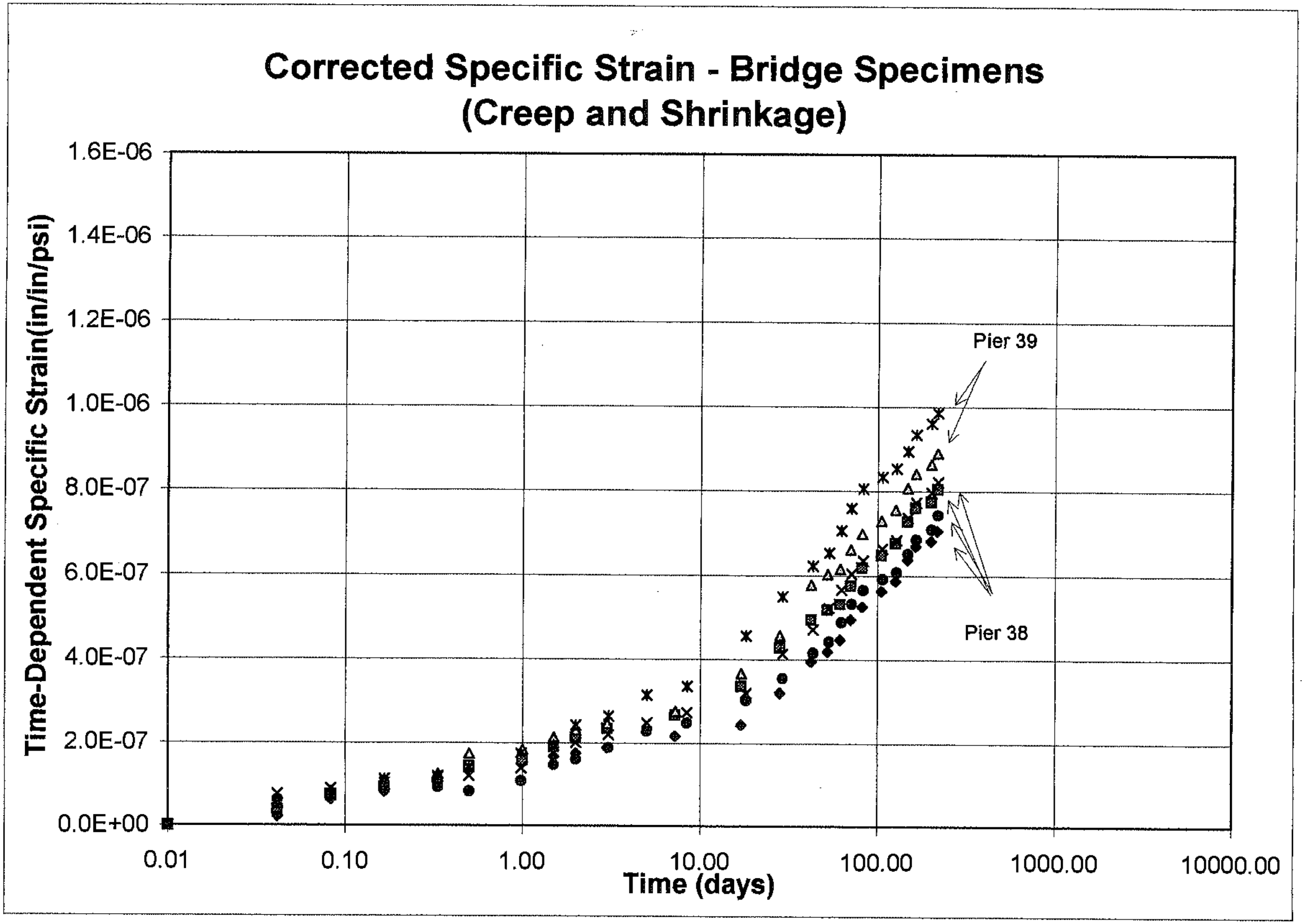
Notes:
1. This analysis represents specifically the samples submitted on a dry (105° C) basis.
2. This report may not be reproduced except in its entirety.
3. Analysis by potentiometric titration with silver nitrate. (ASTM C1218).

Appendix D.

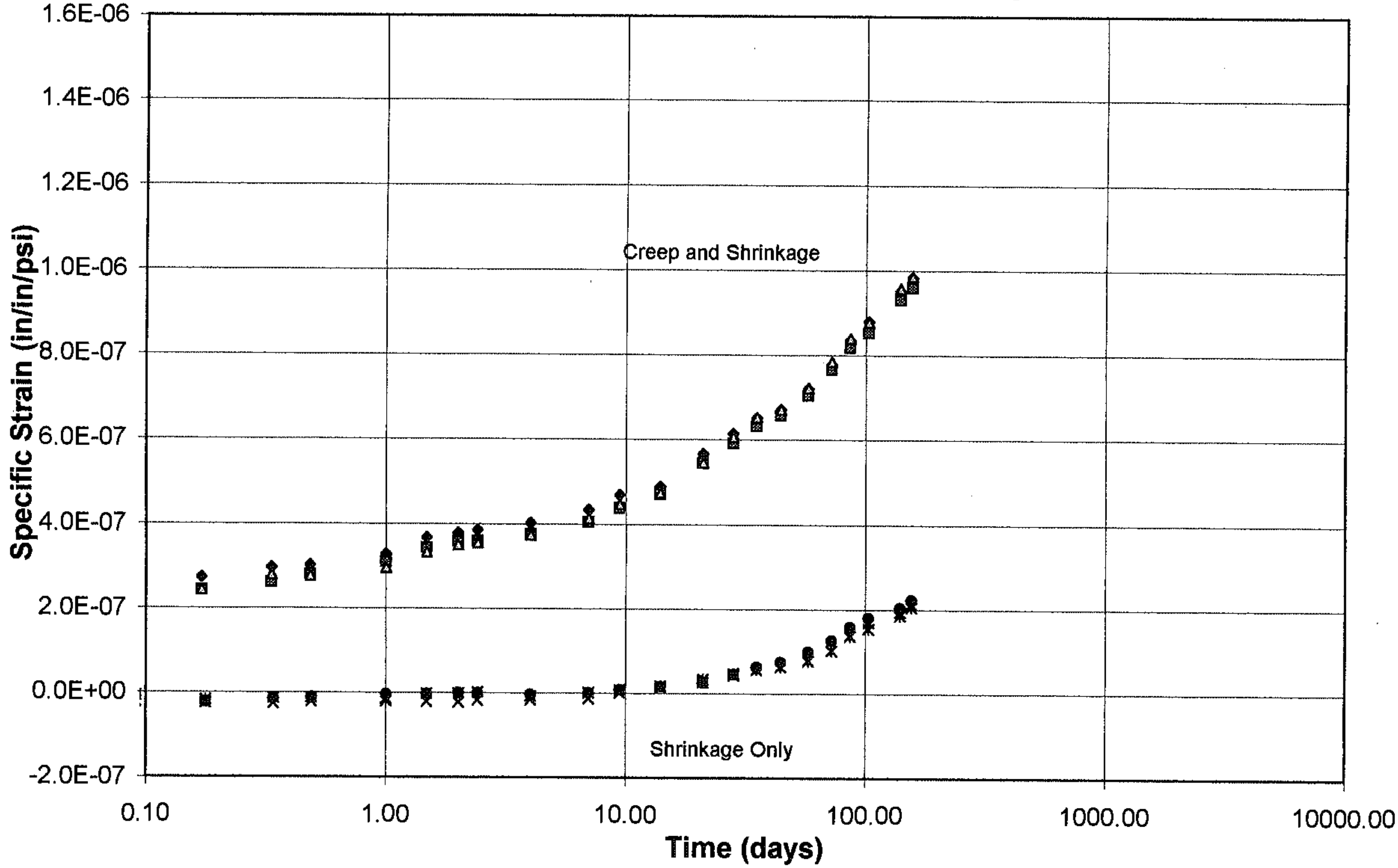
Creep Testing

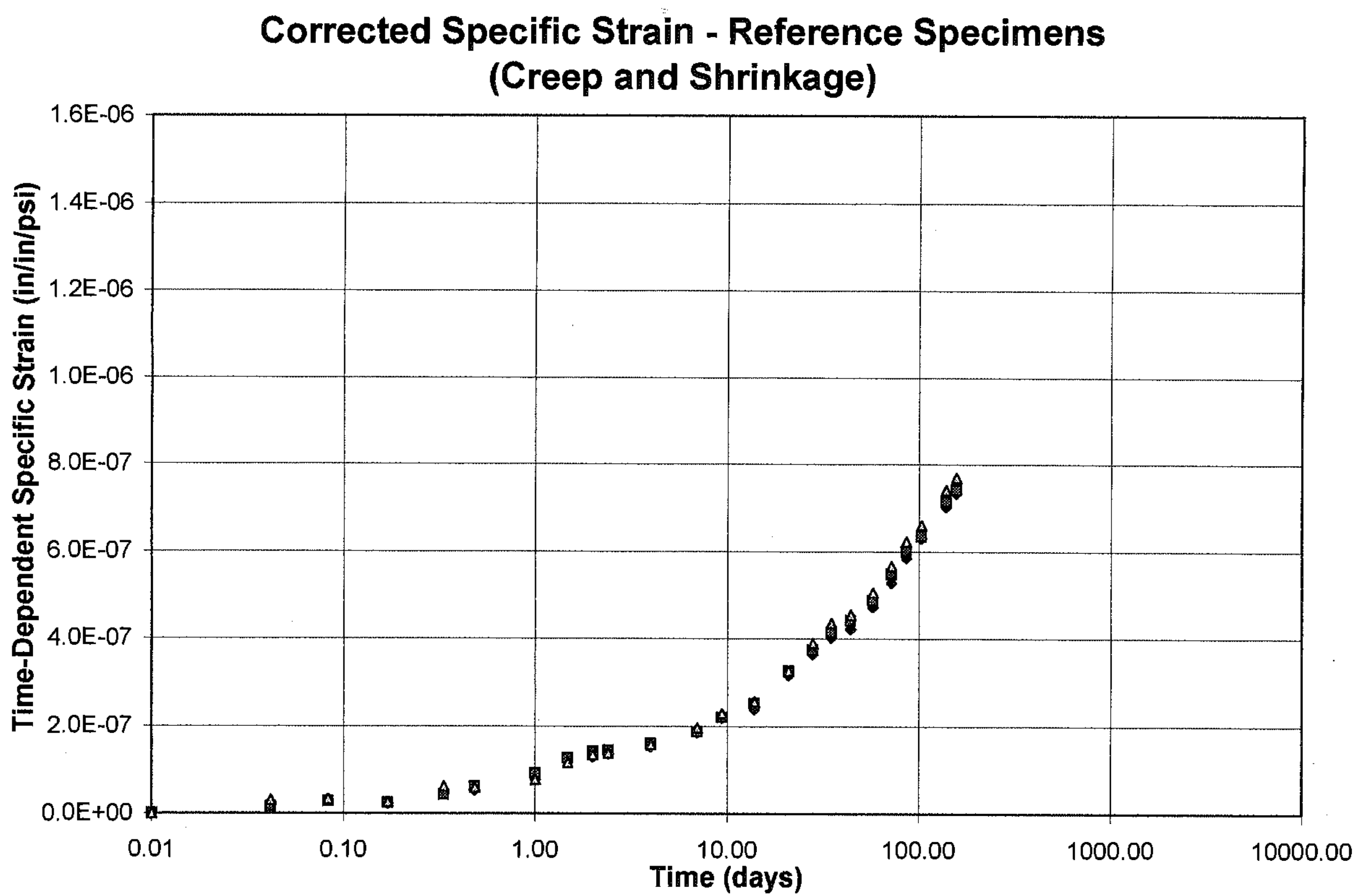
The creep testing procedure is described on pages D47 and following.


Average Specific Strain - Bridge Specimens
Specific Strain (in/in/psi)
1.6E-06
1.4E-06
1.2E-06
1.0E-06
8.0E-07
6.0E-07
4.0E-07
2.0E-07
0.0E+00
-2.0E-07
0.10
1.00
10.00
100.00
1000.00
10000.00
Time (days)
Creep and Shrinkage
Shrinkage Only
Pier 39
Pier 38
Pier 39
Pier 38


Corrected Specific Strain - Bridge Specimens
(Creep and Shrinkage)
Time-Dependent Specific Strain(in/in/psi)
1.6E-06
1.4E-06
1.2E-06
1.0E-06
8.0E-07
6.0E-07
4.0E-07
2.0E-07
0.0E+00
0.01
0.10
1.00
10.00
100.00
1000.00
10000.00
Time (days)
Pier 39
Pier 38

Average Specific Strain - Reference Specimens




Corrected Specific Strain - Reference Specimens
(Creep and Shrinkage)
Time-Dependent Specific Strain (in/in/psi)
1.6E-06
1.4E-06
1.2E-06
1.0E-06
8.0E-07
6.0E-07
4.0E-07
2.0E-07
0.0E+00
0.01
0.10
1.00
10.00
100.00
1000.00
10000.00
Time (days)

			Initial		After Time Elapsed in Testiing (Test Time t)							Ultimate Values			
Specimen	Type	Test Time t (Days)	Initial Strain (in/in)	Specific Initial Strain (in/in/psi)	Shrinkage (in/in)	Specific Shrinkage (in/in/psi)	Creep+Shrinkage (in/in)	Specific Creep +Shrinkage (in/in/psi)	Creep (in/in)	Specific Creep (in/in/psi)	V-t	V-ult	Ult Creep Strain (in/in)	Ult Specific Creep Strn. (in/in/psi)	Ult Shrink Strain (in/in)
Pier 38 #1	Creep+Shrink	218	465.E-06	.328E-06			001.E-03	.744E-06							
Pier 38 #2	Creep+Shrink	218	513.E-06	.363E-06			997.E-06	.705E-06							
Pier 38 #3	Creep+Shrink	218	424.E-06	.300E-06			1166E-06	.821E-06							
Pier 38 #4	Creep+Shrink	218	526.E-06	.372E-06			1134E-06	.804E-06							
Avg Creep 38	Creep+Shrink		467.E-06	.341E-06			1088E-06	.769E-06							
#5 38	Shrink only	218			355.E-06	.251E-06									
Avg Shrink 38	Shrink only				355.E-06	.251E-06									
Avg. Pier 38									733.E-06	.518E-06	1.57	2.19	1022E-06	.722E-06	413E-06
Pier 39 #1	Creep+Shrink	218	616.E-06	.436E-06			1394E-06	.987E-06							
Pier 39 #3	Creep+Shrink	218	470.E-06	.332E-06			1260E-06	.890E-06							
Avg Creep 39	Creep+Shrink		543.E-06	.384E-06			1327E-06	.939E-06							
Pier 39 #5	Shrink only	218			460.E-06	.325E-06									
Avg Shrink 39	Shrink only				460.E-06	.325E-06									
Avg Pier 39									867.E-06	.614E-06	1.60	2.23	1208E-06	.854E-06	535E-06
Ref #1	Creep+Shrink	155	269.E-06	.217E-06			951.E-06	.769E-06							
Ref #5	Creep+Shrink	155	269.E-06	.217E-06			921.E-06	.742E-06							
Ref #7	Creep+Shrink	155	309.E-06	.250E-06			911.E-06	.734E-06							
Avg Creep REF	Creep+Shrink		282.E-06	.228E-06			928.E-06	.748E-06							
Ref #2	Shrink only	155			255.E-06	.206E-06									
Ref #8	Shrink only	155			276.E-06	.223E-06									
Ref #9	Shrink only	155			263.E-06	.212E-06									
Avg Shrink REF	Shrink only				265.E-06	.214E-06									
Avg Reference									663.E-06	.535E-06	2.35	3.49	985E-06	.796E-06	325E-06

SPECIMEN #1 38 A											
Date	Time	Load (kips)	Time (days)	A					Avg.	Diff.	Strain #1 38 A
				1	2	3	4	5			
2/22/96	9:30 AM	0	0.00	0.02735	0.02728	0.02723	0.02726	0.02734	0.02729	*****	*****
2/22/96	10:00 AM	40	0.01	0.02335	0.02320	0.02330	0.02325	0.02320	0.02326	0.00403	504E-06
2/22/96	11:00 AM	40	0.04	0.02240	0.02238	0.02235	0.02230	0.02239	0.02236	0.00493	616E-06
2/22/96	12:00 PM	40	0.08	0.02245	0.02251	0.02252	0.02240	0.02250	0.02248	0.00482	602E-06
2/22/96	2:00 PM	40	0.17	0.02236	0.02232	0.02235	0.02232	0.02233	0.02234	0.00496	620E-06
2/22/96	6:00 PM	40	0.33	0.02250	0.02254	0.02258	0.02259	0.02251	0.02254	0.00475	594E-06
2/22/96	10:00 PM	40	0.50	0.02261	0.02258	0.02258	0.02255	0.02250	0.02256	0.00473	591E-06
2/23/96	9:30 AM	40	0.98	0.02249	0.02245	0.02246	0.02248	0.02250	0.02248	0.00482	602E-06
2/23/96	10:00 PM	40	1.50	0.02193	0.02203	0.02195	0.02199	0.02200	0.02198	0.00531	664E-06
2/24/96	10:00 AM	40	2.00	0.02205	0.02199	0.02187	0.02198	0.02195	0.02197	0.00532	666E-06
2/25/96	11:30 AM	40	3.06	0.02179	0.02180	0.02180	0.02178	0.02182	0.02180	0.00549	687E-06
2/27/96	10:30 AM	40	5.02	0.02135	0.02140	0.02134	0.02135	0.02136	0.02136	0.00593	742E-06
3/1/96	7:35 PM	40	8.40	0.02126	0.02130	0.02121	0.02128	0.02122	0.02125	0.00604	755E-06
3/11/96	3:00 PM	40	18.21	0.02113	0.02118	0.02111	0.02120	0.02116	0.02116	0.00614	767E-06
3/22/96	4:45 PM	40	29.28	0.02061	0.02068	0.02068	0.02066	0.02069	0.02066	0.00663	829E-06
4/5/96	3:45 PM	40*	43.24	0.01971	0.01981	0.01975	0.01971	0.01966	0.01973	0.00756	946E-06
4/15/96	2:00 PM	40	53.17	0.01959	0.01965	0.01959	0.01965	0.01961	0.01962	0.00767	959E-06
4/24/96	9:10 PM	40	62.47	0.01905	0.01910	0.01908	0.01905	0.01910	0.01908	0.00822	1.03E-03
5/3/96	10:00 PM	40	71.50	0.01865	0.01861	0.01861	0.01862	0.01859	0.01862	0.00868	1.08E-03
5/15/96	1:45 PM	40	83.16	0.01828	0.01818	0.01828	0.01828	0.01831	0.01827	0.00903	1.13E-03
6/7/96	4:20 PM	40	106.26	0.01795	0.01791	0.01795	0.01792	0.01790	0.01793	0.00937	1.17E-03
6/28/96	2:50 PM	40	127.20	0.01769	0.01775	0.01781	0.01772	0.01772	0.01774	0.00955	1.19E-03
7/19/96	11:45 AM	40	148.07	0.01731	0.01725	0.01719	0.01721	0.01715	0.01722	0.01007	1.26E-03
8/5/96	11:10 AM	40	165.05	0.01690	0.01689	0.01687	0.01687	0.01687	0.01688	0.01041	1.30E-03
9/10/96	11:25 AM	40	201.06	0.01658	0.01655	0.01661	0.01660	0.01661	0.01659	0.01070	1.34E-03
9/27/96	2:10 PM	40	218.17	0.01621	0.01621	0.01620	0.01615	0.01618	0.01619	0.01110	1.39E-03

SPECIMEN #1 38 B

Date	Time	Load (kips)	Time (days)	B 1	B 2	B 3	B 4	B 5	Avg.	Diff.	Strain #1 38 B
2/22/96	9:30 AM	0	0.00	0.03086	0.03082	0.03092	0.03095	0.03098	0.03091	*****	*****
2/22/96	10:00 AM	40	0.01	0.02750	0.02745	0.02758	0.02747	0.02752	0.02750	0.00340	425E-06
2/22/96	11:00 AM	40	0.04	0.02693	0.02717	0.02700	0.02690	0.02697	0.02699	0.00391	489E-06
2/22/96	12:00 PM	40	0.08	0.02655	0.02670	0.02671	0.02664	0.02672	0.02666	0.00424	530E-06
2/22/96	2:00 PM	40	0.17	0.02636	0.02633	0.02620	0.02634	0.02628	0.02630	0.00460	576E-06
2/22/96	6:00 PM	40	0.33	0.02621	0.02606	0.02604	0.02618	0.02621	0.02614	0.00477	596E-06
2/22/96	10:00 PM	40	0.50	0.02643	0.02625	0.02620	0.02642	0.02640	0.02634	0.00457	571E-06
2/23/96	9:30 AM	40	0.98	0.02581	0.02595	0.02592	0.02581	0.02580	0.02586	0.00505	631E-06
2/23/96	10:00 PM	40	1.50	0.02552	0.02543	0.02544	0.02552	0.02545	0.02547	0.00543	679E-06
2/24/96	10:00 AM	40	2.00	0.02525	0.02521	0.02514	0.02511	0.02516	0.02517	0.00573	717E-06
2/25/96	11:30 AM	40	3.06	0.02470	0.02469	0.02472	0.02475	0.02470	0.02471	0.00619	774E-06
2/27/96	10:30 AM	40	5.02	0.02426	0.02423	0.02425	0.02422	0.02426	0.02424	0.00666	833E-06
3/1/96	7:35 PM	40	8.40	0.02390	0.02381	0.02389	0.02385	0.02391	0.02387	0.00703	879E-06
3/11/96	3:00 PM	40	18.21	0.02270	0.02280	0.02279	0.02274	0.02279	0.02276	0.00814	1.02E-03
3/22/96	4:45 PM	40	29.28	0.02196	0.02211	0.02209	0.02210	0.02202	0.02206	0.00885	1.11E-03
4/5/96	3:45 PM	40*	43.24	0.02159	0.02158	0.02165	0.02166	0.02162	0.02162	0.00929	1.16E-03
4/15/96	2:00 PM	40	53.17	0.02109	0.02118	0.02119	0.02119	0.02105	0.02114	0.00977	1.22E-03
4/24/96	9:10 PM	40	62.47	0.02069	0.02061	0.02065	0.02071	0.02060	0.02065	0.01025	1.28E-03
5/3/96	10:00 PM	40	71.50	0.02015	0.02010	0.02009	0.02010	0.02009	0.02011	0.01080	1.35E-03
5/15/96	1:45 PM	40	83.16	0.01975	0.01969	0.01969	0.01971	0.01969	0.01971	0.01120	1.40E-03
6/7/96	4:20 PM	40	106.26	0.01939	0.01945	0.01941	0.01949	0.01945	0.01944	0.01147	1.43E-03
6/28/96	2:50 PM	40	127.20	0.01925	0.01931	0.01922	0.01928	0.01931	0.01927	0.01163	1.45E-03
7/19/96	11:45 AM	40	148.07	0.01881	0.01881	0.01875	0.01881	0.01879	0.01879	0.01211	1.51E-03
8/5/96	11:10 AM	40	165.05	0.01832	0.01842	0.01840	0.01839	0.01838	0.01838	0.01252	1.57E-03
9/10/96	11:25 AM	40	201.06	0.01810	0.01811	0.01812	0.01811	0.01810	0.01811	0.01280	1.60E-03
9/27/96	2:10 PM	40	218.17	0.01775	0.01771	0.01775	0.01778	0.01771	0.01774	0.01317	1.65E-03

SPECIMEN #1 38				Avg Strain #1 38	Specific Strain #1 38	Corrected Spec Strain #1 38
Date	Time	Load (kips)	Time (days)			
2/22/96	9:30 AM	0	0.00			
2/22/96	10:00 AM	40	0.01	465E-06	.328E-06	.000E+00
2/22/96	11:00 AM	40	0.04	553E-06	.390E-06	.621E-07
2/22/96	12:00 PM	40	0.08	566E-06	.400E-06	.717E-07
2/22/96	2:00 PM	40	0.17	598E-06	.422E-06	.939E-07
2/22/96	6:00 PM	40	0.33	595E-06	.420E-06	.919E-07
2/22/96	10:00 PM	40	0.50	581E-06	.411E-06	.822E-07
2/23/96	9:30 AM	40	0.98	617E-06	.436E-06	.107E-06
2/23/96	10:00 PM	40	1.50	672E-06	.475E-06	.146E-06
2/24/96	10:00 AM	40	2.00	691E-06	.488E-06	.160E-06
2/25/96	11:30 AM	40	3.06	731E-06	.516E-06	.188E-06
2/27/96	10:30 AM	40	5.02	787E-06	.556E-06	.228E-06
3/1/96	7:35 PM	40	8.40	817E-06	.577E-06	.249E-06
3/11/96	3:00 PM	40	18.21	892E-06	.631E-06	.302E-06
3/22/96	4:45 PM	40	29.28	967E-06	.684E-06	.355E-06
4/5/96	3:45 PM	40*	43.24	1.05E-03	.744E-06	.416E-06
4/15/96	2:00 PM	40	53.17	1.09E-03	.770E-06	.442E-06
4/24/96	9:10 PM	40	62.47	1.15E-03	.816E-06	.487E-06
5/3/96	10:00 PM	40	71.50	1.22E-03	.860E-06	.532E-06
5/15/96	1:45 PM	40	83.16	1.26E-03	.893E-06	.565E-06
6/7/96	4:20 PM	40	106.26	1.30E-03	.920E-06	.592E-06
6/28/96	2:50 PM	40	127.20	1.32E-03	.936E-06	.607E-06
7/19/96	11:45 AM	40	148.07	1.39E-03	.980E-06	.651E-06
8/5/96	11:10 AM	40	165.05	1.43E-03	1.01E-06	.685E-06
9/10/96	11:25 AM	40	201.06	1.47E-03	1.04E-06	.710E-06
9/27/96	2:10 PM	40	218.17	1.52E-03	1.07E-06	.744E-06

SPECIMEN #2 38 A

Date	Time	Load (kips)	Time (days)	A 1	2	3	4	5	Avg.	Diff.	Strain #2 38 A
2/23/96	12:00 PM	0	0.00	0.03030	0.03040	0.03050	0.03045	0.03042	0.03041	*****	*****
2/23/96	12:25 PM	20	0.00	0.02888	0.02881	0.02883	0.02875	0.02872	0.02880	*****	*****
2/23/96	12:45 PM	40	0.01	0.02615	0.02610	0.02625	0.02609	0.02618	0.02615	0.00426	533E-06
2/23/96	1:45 PM	40	0.04	0.02558	0.02566	0.02570	0.02580	0.02571	0.02569	0.00472	591E-06
2/23/96	2:45 PM	40	0.08	0.02523	0.02529	0.02519	0.02532	0.02531	0.02527	0.00515	643E-06
2/23/96	4:45 PM	40	0.17	0.02511	0.02513	0.02510	0.02513	0.02521	0.02514	0.00528	660E-06
2/23/96	8:45 PM	40	0.33	0.02485	0.02488	0.02483	0.02489	0.02486	0.02486	0.00555	694E-06
2/24/96	12:45 AM	40	0.50	0.02463	0.02461	0.02462	0.02451	0.02463	0.02460	0.00581	727E-06
2/24/96	12:45 PM	40	1.00	0.02436	0.02442	0.02438	0.02437	0.02439	0.02438	0.00603	754E-06
2/25/96	12:45 AM	40	1.50	0.02428	0.02423	0.02422	0.02426	0.02424	0.02425	0.00617	771E-06
2/25/96	12:45 PM	40	2.00	0.02430	0.02426	0.02415	0.02426	0.02427	0.02425	0.00617	771E-06
2/26/96	1:00 PM	40	3.01	0.02410	0.02412	0.02414	0.02413	0.02412	0.02412	0.00629	787E-06
3/1/96	7:05 PM	40	7.26	0.02384	0.02391	0.02391	0.02390	0.02388	0.02389	0.00653	816E-06
3/11/96	1:45 PM	40	17.04	0.02362	0.02360	0.02360	0.02360	0.02364	0.02361	0.00680	850E-06
3/22/96	5:15 PM	40	28.19	0.02275	0.02278	0.02275	0.02271	0.02270	0.02274	0.00768	960E-06
4/5/96	4:30 PM	40	42.16	0.02175	0.02162	0.02192	0.02178	0.02178	0.02177	0.00864	1.08E-03
4/15/96	4:15 PM	40	52.15	0.02145	0.02149	0.02139	0.02149	0.02142	0.02145	0.00897	1.12E-03
4/25/96	12:30 AM	40	61.49	0.02095	0.02095	0.02099	0.02105	0.02105	0.02100	0.00942	1.18E-03
5/3/96	10:30 PM	40	70.41	0.02045	0.02042	0.02055	0.02045	0.02050	0.02047	0.00994	1.24E-03
5/15/96	2:20 PM	40	82.07	0.02010	0.02005	0.02000	0.02008	0.02010	0.02007	0.01035	1.29E-03
6/7/96	5:40 PM	40	105.20	0.01968	0.01962	0.01961	0.01965	0.01960	0.01963	0.01078	1.35E-03
6/28/96	3:10 PM	40	126.10	0.01931	0.01935	0.01929	0.01931	0.01931	0.01931	0.01110	1.39E-03
7/19/96	12:45 PM	40	147.00	0.01898	0.01899	0.01895	0.01895	0.01892	0.01896	0.01146	1.43E-03
8/5/96	12:10 PM	40	163.98	0.01850	0.01849	0.01848	0.01842	0.01845	0.01847	0.01195	1.49E-03
9/10/96	11:50 AM	40	199.96	0.01820	0.01818	0.01819	0.01819	0.01821	0.01819	0.01222	1.53E-03
9/27/96	2:40 PM	40	217.08	0.01790	0.01795	0.01788	0.01791	0.01788	0.01790	0.01251	1.56E-03

SPECIMEN #2 38 B

Date	Time	Load (kips)	Time (days)	B 1	B 2	B 3	B 4	B 5	Avg.	Diff.	Strain #2 38 B
2/23/96	12:00 PM	0	0.00	0.03000	0.03007	0.03001	0.03010	0.03006	0.03005	*****	*****
2/23/96	12:25 PM	20	0.00	0.02839	0.02821	0.02829	0.02819	0.02823	0.02826	*****	*****
2/23/96	12:45 PM	40	0.01	0.02611	0.02611	0.02618	0.02599	0.02610	0.02610	0.00395	494E-06
2/23/96	1:45 PM	40	0.04	0.02609	0.02600	0.02599	0.02617	0.02611	0.02607	0.00398	497E-06
2/23/96	2:45 PM	40	0.08	0.02550	0.02558	0.02561	0.02555	0.02557	0.02556	0.00449	561E-06
2/23/96	4:45 PM	40	0.17	0.02524	0.02522	0.02531	0.02529	0.02534	0.02528	0.00477	596E-06
2/23/96	8:45 PM	40	0.33	0.02515	0.02522	0.02533	0.02531	0.02527	0.02526	0.00479	599E-06
2/24/96	12:45 AM	40	0.50	0.02455	0.02450	0.02460	0.02461	0.02467	0.02459	0.00546	683E-06
2/24/96	12:45 PM	40	1.00	0.02450	0.02442	0.02442	0.02444	0.02437	0.02443	0.00562	702E-06
2/25/96	12:45 AM	40	1.50	0.02425	0.02420	0.02423	0.02421	0.02427	0.02423	0.00582	727E-06
2/25/96	12:45 PM	40	2.00	0.02408	0.02405	0.02400	0.02402	0.02408	0.02405	0.00600	750E-06
2/26/96	1:00 PM	40	3.01	0.02380	0.02389	0.02390	0.02388	0.02391	0.02388	0.00617	772E-06
3/1/96	7:05 PM	40	7.26	0.02351	0.02344	0.02343	0.02345	0.02349	0.02346	0.00658	823E-06
3/11/96	1:45 PM	40	17.04	0.02315	0.02306	0.02310	0.02312	0.02311	0.02311	0.00694	868E-06
3/22/96	5:15 PM	40	28.19	0.02223	0.02230	0.02221	0.02228	0.02225	0.02225	0.00779	974E-06
4/5/96	4:30 PM	40	42.16	0.02155	0.02155	0.02158	0.02148	0.02158	0.02155	0.00850	1.06E-03
4/15/96	4:15 PM	40	52.15	0.02135	0.02132	0.02129	0.02135	0.02132	0.02133	0.00872	1.09E-03
4/25/96	12:30 AM	40	61.49	0.02115	0.02118	0.02118	0.02115	0.02110	0.02115	0.00890	1.11E-03
5/3/96	10:30 PM	40	70.41	0.02055	0.02058	0.02050	0.02065	0.02058	0.02057	0.00948	1.18E-03
5/15/96	2:20 PM	40	82.07	0.02030	0.02029	0.02029	0.02030	0.02029	0.02029	0.00975	1.22E-03
6/7/96	5:40 PM	40	105.20	0.01985	0.01989	0.01989	0.01989	0.01991	0.01989	0.01016	1.27E-03
6/28/96	3:10 PM	40	126.10	0.01960	0.01965	0.01968	0.01969	0.01961	0.01965	0.01040	1.30E-03
7/19/96	12:45 PM	40	147.00	0.01888	0.01885	0.01880	0.01889	0.01888	0.01886	0.01119	1.40E-03
8/5/96	12:10 PM	40	163.98	0.01860	0.01865	0.01868	0.01865	0.01860	0.01864	0.01141	1.43E-03
9/10/96	11:50 AM	40	199.96	0.01869	0.01861	0.01861	0.01861	0.01860	0.01862	0.01142	1.43E-03
9/27/96	2:40 PM	40	217.08	0.01840	0.01839	0.01841	0.01835	0.01835	0.01838	0.01167	1.46E-03

SPECIMEN #2 38				Avg Strain #2 38	Specific Strain #2 38	Corrected Spec Strain #2 38
Date	Time	Load (kips)	Time (days)			
2/23/96	12:00 PM	0	0.00			
2/23/96	12:25 PM	20	0.00			
2/23/96	12:45 PM	40	0.01	513E-06	.363E-06	0.00E+00
2/23/96	1:45 PM	40	0.04	544E-06	.384E-06	.216E-07
2/23/96	2:45 PM	40	0.08	602E-06	.425E-06	.628E-07
2/23/96	4:45 PM	40	0.17	628E-06	.444E-06	.811E-07
2/23/96	8:45 PM	40	0.33	647E-06	.457E-06	.943E-07
2/24/96	12:45 AM	40	0.50	705E-06	.498E-06	.135E-06
2/24/96	12:45 PM	40	1.00	728E-06	.515E-06	.152E-06
2/25/96	12:45 AM	40	1.50	749E-06	.529E-06	.167E-06
2/25/96	12:45 PM	40	2.00	761E-06	.537E-06	.175E-06
2/26/96	1:00 PM	40	3.01	779E-06	.551E-06	.188E-06
3/1/96	7:05 PM	40	7.26	819E-06	.579E-06	.216E-06
3/11/96	1:45 PM	40	17.04	859E-06	.607E-06	.244E-06
3/22/96	5:15 PM	40	28.19	967E-06	.683E-06	.321E-06
4/5/96	4:30 PM	40	42.16	1.07E-03	.757E-06	.395E-06
4/15/96	4:15 PM	40	52.15	1.11E-03	.781E-06	.419E-06
4/25/96	12:30 AM	40	61.49	1.14E-03	.809E-06	.446E-06
5/3/96	10:30 PM	40	70.41	1.21E-03	.858E-06	.495E-06
5/15/96	2:20 PM	40	82.07	1.26E-03	.888E-06	.525E-06
6/7/96	5:40 PM	40	105.20	1.31E-03	.925E-06	.562E-06
6/28/96	3:10 PM	40	126.10	1.34E-03	.950E-06	.587E-06
7/19/96	12:45 PM	40	147.00	1.42E-03	1.00E-06	.638E-06
8/5/96	12:10 PM	40	163.98	1.46E-03	1.03E-06	.669E-06
9/10/96	11:50 AM	40	199.96	1.48E-03	1.04E-06	.682E-06
9/27/96	2:40 PM	40	217.08	1.51E-03	1.07E-06	.705E-06

SPECIMEN #3 38 A

Date	Time	Load (kips)	Time (Days)	A 1	A 2	A 3	A 4	A 5	Avg.	Diff.	Strain #3 38 A
2/22/96	9:30 AM	0	0.00	0.02858	0.02866	0.02852	0.02847	0.02851	0.02855	*****	*****
2/22/96	10:00 AM	40	0.01	0.02542	0.02556	0.02540	0.02538	0.02551	0.02545	0.00309	387E-06
2/22/96	11:00 AM	40	0.04	0.02492	0.02505	0.02498	0.02501	0.02490	0.02497	0.00358	447E-06
2/22/96	12:00 PM	40	0.08	0.02481	0.02480	0.02483	0.02485	0.02479	0.02482	0.00373	467E-06
2/22/96	2:00 PM	40	0.17	0.02463	0.02463	0.02465	0.02456	0.02461	0.02462	0.00393	492E-06
2/22/96	6:00 PM	40	0.33	0.02429	0.02422	0.02423	0.02421	0.02414	0.02422	0.00433	541E-06
2/22/96	10:00 PM	40	0.50	0.02417	0.02413	0.02418	0.02416	0.02417	0.02416	0.00439	548E-06
2/23/96	9:30 AM	40	0.98	0.02410	0.02411	0.02403	0.02406	0.02409	0.02408	0.00447	559E-06
2/23/96	10:00 PM	40	1.50	0.02349	0.02358	0.02351	0.02352	0.02360	0.02354	0.00501	626E-06
2/24/96	10:00 AM	40	2.00	0.02330	0.02334	0.02332	0.02333	0.02332	0.02332	0.00523	653E-06
2/25/96	11:30 AM	40	3.06	0.02304	0.02309	0.02313	0.02315	0.02313	0.02311	0.00544	680E-06
2/27/96	10:30 AM	40	5.02	0.02297	0.02293	0.02296	0.02295	0.02299	0.02296	0.00559	699E-06
3/1/96	7:35 PM	40	8.40	0.02275	0.02278	0.02270	0.02269	0.02268	0.02272	0.00583	729E-06
3/11/96	3:00 PM	40	18.21	0.02209	0.02208	0.02206	0.02209	0.02209	0.02208	0.00647	808E-06
3/22/96	4:45 PM	40	29.28	0.02095	0.02097	0.02091	0.02098	0.02098	0.02096	0.00759	949E-06
4/5/96	3:45 PM	40*	43.24	0.03011	0.03028	0.03026	0.03018	0.03022	0.03021	0.00834	1.04E-03
4/15/96	2:00 PM	40	53.17	0.02931	0.02941	0.02935	0.02929	0.02945	0.02936	0.00919	1.15E-03
4/24/96	9:10 PM	40	62.47	0.02870	0.02869	0.02869	0.02865	0.02871	0.02869	0.00986	1.23E-03
5/3/96	10:00 PM	40	71.50	0.02831	0.02835	0.02831	0.02829	0.02829	0.02831	0.01024	1.28E-03
5/15/96	1:45 PM	40	83.16	0.02791	0.02795	0.02791	0.02798	0.02796	0.02794	0.01061	1.33E-03
6/7/96	4:20 PM	40	106.26	0.02761	0.02765	0.02761	0.02769	0.02761	0.02763	0.01091	1.36E-03
6/28/96	2:50 PM	40	127.20	0.02741	0.02740	0.02739	0.02739	0.02739	0.02740	0.01115	1.39E-03
7/19/96	11:45 AM	40	148.07	0.02685	0.02682	0.02685	0.02685	0.02682	0.02684	0.01171	1.46E-03
8/5/96	11:10 AM	40	165.05	0.02649	0.02645	0.02639	0.02641	0.02642	0.02643	0.01212	1.51E-03
9/10/96	11:25 AM	40	201.06	0.02611	0.02610	0.02612	0.02610	0.02609	0.02610	0.01244	1.56E-03
9/27/96	2:10 PM	40	218.17	0.02588	0.02590	0.02588	0.02588	0.02588	0.02588	0.01266	1.58E-03

SPECIMEN #3 38 B

Date	Time	Load (kips)	Time (days)	B 1	B 2	B 3	B 4	B 5	Avg.	Diff.	Strain #3 38 B
2/22/96	9:30 AM	0	0.00	0.03131	0.03123	0.03134	0.03133	0.03133	0.03131	*****	*****
2/22/96	10:00 AM	40	0.01	0.02771	0.02769	0.02753	0.02759	0.02755	0.02761	0.00369	462E-06
2/22/96	11:00 AM	40	0.04	0.02637	0.02640	0.02638	0.02639	0.02635	0.02638	0.00493	616E-06
2/22/96	12:00 PM	40	0.08	0.02629	0.02618	0.02611	0.02624	0.02630	0.02622	0.00508	635E-06
2/22/96	2:00 PM	40	0.17	0.02622	0.02609	0.02618	0.02615	0.02603	0.02613	0.00517	647E-06
2/22/96	6:00 PM	40	0.33	0.02620	0.02624	0.02621	0.02619	0.02620	0.02621	0.00510	638E-06
2/22/96	10:00 PM	40	0.50	0.02619	0.02620	0.02620	0.02613	0.02621	0.02619	0.00512	640E-06
2/23/96	9:30 AM	40	0.98	0.02580	0.02590	0.02580	0.02598	0.02589	0.02587	0.00543	679E-06
2/23/96	10:00 PM	40	1.50	0.02552	0.02551	0.02549	0.02550	0.02548	0.02550	0.00581	726E-06
2/24/96	10:00 AM	40	2.00	0.02519	0.02520	0.02519	0.02524	0.02521	0.02521	0.00610	763E-06
2/25/96	11:30 AM	40	3.06	0.02498	0.02494	0.02492	0.02500	0.02497	0.02496	0.00635	793E-06
2/27/96	10:30 AM	40	5.02	0.02444	0.02448	0.02448	0.02445	0.02449	0.02447	0.00684	855E-06
3/1/96	7:35 PM	40	8.40	0.02411	0.02415	0.02413	0.02411	0.02416	0.02413	0.00718	897E-06
3/11/96	3:00 PM	40	18.21	0.02374	0.02378	0.02374	0.02373	0.02378	0.02375	0.00755	944E-06
3/22/96	4:45 PM	40	29.28	0.02271	0.02278	0.02280	0.02274	0.02277	0.02276	0.00855	1.07E-03
4/5/96	3:45 PM	40*	43.24	0.02222	0.02200	0.02240	0.02217	0.02227	0.02221	0.00910	1.14E-03
4/15/96	2:00 PM	40	53.17	0.02189	0.02189	0.02185	0.02199	0.02185	0.02189	0.00941	1.18E-03
4/24/96	9:10 PM	40	62.47	0.02155	0.02152	0.02158	0.02155	0.02158	0.02156	0.00975	1.22E-03
5/3/96	10:00 PM	40	71.50	0.02118	0.02108	0.02105	0.02105	0.02105	0.02108	0.01023	1.28E-03
5/15/96	1:45 PM	40	83.16	0.02078	0.02069	0.02075	0.02078	0.02075	0.02075	0.01056	1.32E-03
6/7/96	4:20 PM	40	106.26	0.02041	0.02039	0.02045	0.02039	0.02039	0.02041	0.01090	1.36E-03
6/28/96	2:50 PM	40	127.20	0.02020	0.02021	0.02020	0.02021	0.02015	0.02019	0.01112	1.39E-03
7/19/96	11:45 AM	40	148.07	0.01955	0.01958	0.01952	0.01959	0.01955	0.01956	0.01175	1.47E-03
8/5/96	11:10 AM	40	165.05	0.01911	0.01912	0.01918	0.01910	0.01918	0.01914	0.01217	1.52E-03
9/10/96	11:25 AM	40	201.06	0.01895	0.01890	0.01900	0.01891	0.01895	0.01894	0.01237	1.55E-03
9/27/96	2:10 PM	40	218.17	0.01861	0.01860	0.01861	0.01859	0.01861	0.01860	0.01270	1.59E-03

SPECIMEN #3 38				Avg Strain #3 38	Specific Strain #3 38	Corrected Spec Strain #3 38
Date	Time	Load (kips)	Time (days)			
2/22/96	9:30 AM	0	0.00			
2/22/96	10:00 AM	40	0.01	424E-06	.300E-06	0.00E+00
2/22/96	11:00 AM	40	0.04	532E-06	.376E-06	.759E-07
2/22/96	12:00 PM	40	0.08	551E-06	.389E-06	.896E-07
2/22/96	2:00 PM	40	0.17	569E-06	.402E-06	.102E-06
2/22/96	6:00 PM	40	0.33	589E-06	.417E-06	.117E-06
2/22/96	10:00 PM	40	0.50	594E-06	.420E-06	.120E-06
2/23/96	9:30 AM	40	0.98	619E-06	.437E-06	.138E-06
2/23/96	10:00 PM	40	1.50	676E-06	.478E-06	.178E-06
2/24/96	10:00 AM	40	2.00	708E-06	.500E-06	.201E-06
2/25/96	11:30 AM	40	3.06	737E-06	.521E-06	.221E-06
2/27/96	10:30 AM	40	5.02	777E-06	.549E-06	.249E-06
3/1/96	7:35 PM	40	8.40	813E-06	.574E-06	.275E-06
3/11/96	3:00 PM	40	18.21	876E-06	.619E-06	.319E-06
3/22/96	4:45 PM	40	29.28	1.01E-03	.713E-06	.413E-06
4/5/96	3:45 PM	40*	43.24	1.09E-03	.770E-06	.470E-06
4/15/96	2:00 PM	40	53.17	1.16E-03	.822E-06	.522E-06
4/24/96	9:10 PM	40	62.47	1.23E-03	.866E-06	.566E-06
5/3/96	10:00 PM	40	71.50	1.28E-03	.904E-06	.604E-06
5/15/96	1:45 PM	40	83.16	1.32E-03	.935E-06	.635E-06
6/7/96	4:20 PM	40	106.26	1.36E-03	.964E-06	.664E-06
6/28/96	2:50 PM	40	127.20	1.39E-03	.984E-06	.684E-06
7/19/96	11:45 AM	40	148.07	1.47E-03	1.04E-06	.736E-06
8/5/96	11:10 AM	40	165.05	1.52E-03	1.07E-06	.773E-06
9/10/96	11:25 AM	40	201.06	1.55E-03	1.10E-06	.796E-06
9/27/96	2:10 PM	40	218.17	1.59E-03	1.12E-06	.821E-06

SPECIMEN #4 38 A											
Date	Time	Load (kips)	Time (days)	A					Avg.	Diff.	Strain #4 38 A
				1	2	3	4	5			
2/23/96	12:00 PM	0	0.00	0.02970	0.02961	0.02964	0.02952	0.02961	0.02962	*****	*****
2/23/96	12:25 PM	20	0.00	0.02861	0.02845	0.02842	0.02847	0.02840	0.02847	*****	*****
2/23/96	12:45 PM	40	0.01	0.02625	0.02620	0.02621	0.02621	0.02623	0.02622	0.00340	425E-06
2/23/96	1:45 PM	40	0.04	0.02612	0.02605	0.02603	0.02612	0.02595	0.02605	0.00357	446E-06
2/23/96	2:45 PM	40	0.08	0.02552	0.02553	0.02550	0.02565	0.02545	0.02553	0.00409	511E-06
2/23/96	4:45 PM	40	0.17	0.02543	0.02540	0.02539	0.02542	0.02542	0.02541	0.00420	526E-06
2/23/96	8:45 PM	40	0.33	0.02540	0.02532	0.02539	0.02542	0.02531	0.02537	0.00425	531E-06
2/24/96	12:45 AM	40	0.50	0.02480	0.02489	0.02481	0.02476	0.02478	0.02481	0.00481	601E-06
2/24/96	12:45 PM	40	1.00	0.02468	0.02466	0.02459	0.02464	0.02461	0.02464	0.00498	623E-06
2/25/96	12:45 AM	40	1.50	0.02422	0.02421	0.02422	0.02420	0.02421	0.02421	0.00540	676E-06
2/25/96	12:45 PM	40	2.00	0.02408	0.02412	0.02406	0.02407	0.02410	0.02409	0.00553	691E-06
2/26/96	1:00 PM	40	3.01	0.02398	0.02394	0.02395	0.02396	0.02394	0.02395	0.00566	708E-06
3/1/96	7:05 PM	40	7.26	0.02347	0.02346	0.02345	0.02347	0.02344	0.02346	0.00616	770E-06
3/11/96	1:45 PM	40	17.04	0.02288	0.02290	0.02291	0.02291	0.02290	0.02290	0.00672	840E-06
3/22/96	5:15 PM	40	28.19	0.02202	0.02201	0.02202	0.02200	0.02205	0.02202	0.00760	950E-06
4/5/96	4:30 PM	40	42.16	0.02105	0.02102	0.02112	0.02115	0.02108	0.02108	0.00853	1.07E-03
4/15/96	4:15 PM	40	52.15	0.02075	0.02079	0.02072	0.02078	0.02069	0.02075	0.00887	1.11E-03
4/25/96	12:30 AM	40	61.49	0.02065	0.02059	0.02059	0.02065	0.02058	0.02061	0.00900	1.13E-03
5/3/96	10:30 PM	40	70.41	0.02008	0.02005	0.02005	0.01995	0.01995	0.02002	0.00960	1.20E-03
5/15/96	2:20 PM	40	82.07	0.01960	0.01959	0.01960	0.01959	0.01958	0.01959	0.01002	1.25E-03
6/7/96	5:40 PM	40	105.20	0.01935	0.01930	0.01929	0.01935	0.01931	0.01932	0.01030	1.29E-03
6/28/96	3:10 PM	40	126.10	0.01910	0.01918	0.01910	0.01918	0.01915	0.01914	0.01047	1.31E-03
7/19/96	12:45 PM	40	147.00	0.01855	0.01865	0.01858	0.01850	0.01851	0.01856	0.01106	1.38E-03
8/5/96	12:10 PM	40	163.98	0.01829	0.01821	0.01821	0.01828	0.01828	0.01825	0.01136	1.42E-03
9/10/96	11:50 AM	40	199.96	0.01811	0.01810	0.01811	0.01812	0.01811	0.01811	0.01151	1.44E-03
9/27/96	2:40 PM	40	217.08	0.01771	0.01775	0.01771	0.01770	0.01772	0.01772	0.01190	1.49E-03

SPECIMEN #4 38 B

Date	Time	Load (kips)	Time (days)	B 1	B 2	B 3	B 4	B 5	Avg.	Diff.	Strain #4 38 B
2/23/96	12:00 PM	0	0.00	0.02920	0.02922	0.02915	0.02919	0.02913	0.02918	*****	*****
2/23/96	12:25 PM	20	0.00	0.02791	0.02794	0.02786	0.02789	0.02782	0.02788	*****	*****
2/23/96	12:45 PM	40	0.01	0.02420	0.02412	0.02416	0.02413	0.02417	0.02416	0.00502	628E-06
2/23/96	1:45 PM	40	0.04	0.02347	0.02345	0.02350	0.02342	0.02347	0.02346	0.00572	714E-06
2/23/96	2:45 PM	40	0.08	0.02318	0.02311	0.02312	0.02325	0.02318	0.02317	0.00601	751E-06
2/23/96	4:45 PM	40	0.17	0.02285	0.02287	0.02289	0.02290	0.02288	0.02288	0.00630	788E-06
2/23/96	8:45 PM	40	0.33	0.02271	0.02268	0.02265	0.02263	0.02264	0.02266	0.00652	815E-06
2/24/96	12:45 AM	40	0.50	0.02231	0.02237	0.02239	0.02229	0.02231	0.02233	0.00684	856E-06
2/24/96	12:45 PM	40	1.00	0.02223	0.02225	0.02221	0.02218	0.02215	0.02220	0.00697	872E-06
2/25/96	12:45 AM	40	1.50	0.02180	0.02190	0.02185	0.02186	0.02190	0.02186	0.00732	915E-06
2/25/96	12:45 PM	40	2.00	0.02145	0.02148	0.02148	0.02147	0.02145	0.02147	0.00771	964E-06
2/26/96	1:00 PM	40	3.01	0.02111	0.02113	0.02115	0.02112	0.02111	0.02112	0.00805	1.01E-03
3/1/96	7:05 PM	40	7.26	0.02087	0.02089	0.02086	0.02087	0.02087	0.02087	0.00831	1.04E-03
3/11/96	1:45 PM	40	17.04	0.01988	0.01990	0.01981	0.01989	0.01984	0.01986	0.00931	1.16E-03
3/22/96	5:15 PM	40	28.19	0.01870	0.01869	0.01868	0.01870	0.01870	0.01869	0.01048	1.31E-03
4/5/96	4:30 PM	40	42.16	0.01821	0.01805	0.01812	0.01810	0.01819	0.01813	0.01104	1.38E-03
4/15/96	4:15 PM	40	52.15	0.01792	0.01789	0.01795	0.01787	0.01790	0.01791	0.01127	1.41E-03
4/25/96	12:30 AM	40	61.49	0.01775	0.01772	0.01775	0.01772	0.01778	0.01774	0.01143	1.43E-03
5/3/96	10:30 PM	40	70.41	0.01739	0.01731	0.01735	0.01728	0.01730	0.01733	0.01185	1.48E-03
5/15/96	2:20 PM	40	82.07	0.01680	0.01680	0.01675	0.01679	0.01681	0.01679	0.01239	1.55E-03
6/7/96	5:40 PM	40	105.20	0.01635	0.01645	0.01636	0.01639	0.01639	0.01639	0.01279	1.60E-03
6/28/96	3:10 PM	40	126.10	0.01591	0.01590	0.01589	0.01600	0.01600	0.01594	0.01324	1.65E-03
7/19/96	12:45 PM	40	147.00	0.01535	0.01538	0.01536	0.01535	0.01535	0.01536	0.01382	1.73E-03
8/5/96	12:10 PM	40	163.98	0.01492	0.01495	0.01489	0.01490	0.01491	0.01491	0.01426	1.78E-03
9/10/96	11:50 AM	40	199.96	0.01472	0.01471	0.01475	0.01472	0.01475	0.01473	0.01445	1.81E-03
9/27/96	2:40 PM	40	217.08	0.01445	0.01445	0.01448	0.01442	0.01445	0.01445	0.01473	1.84E-03

SPECIMEN #4 38				Avg Strain #4 38	Specific Strain #4 38	Corrected Spec Strain #4 38 B
Date	Time	Load (kips)	Time (days)			
2/23/96	12:00 PM	0	0.00			
2/23/96	12:25 PM	20	0.00			
2/23/96	12:45 PM	40	0.01	526E-06	.372E-06	.000E+00
2/23/96	1:45 PM	40	0.04	580E-06	.410E-06	.382E-07
2/23/96	2:45 PM	40	0.08	631E-06	.446E-06	.741E-07
2/23/96	4:45 PM	40	0.17	657E-06	.464E-06	.921E-07
2/23/96	8:45 PM	40	0.33	673E-06	.475E-06	.104E-06
2/24/96	12:45 AM	40	0.50	728E-06	.515E-06	.143E-06
2/24/96	12:45 PM	40	1.00	747E-06	.528E-06	.156E-06
2/25/96	12:45 AM	40	1.50	795E-06	.562E-06	.190E-06
2/25/96	12:45 PM	40	2.00	828E-06	.585E-06	.213E-06
2/26/96	1:00 PM	40	3.01	857E-06	.606E-06	.234E-06
3/1/96	7:05 PM	40	7.26	904E-06	.639E-06	.267E-06
3/11/96	1:45 PM	40	17.04	1.00E-03	.708E-06	.336E-06
3/22/96	5:15 PM	40	28.19	1.13E-03	.799E-06	.427E-06
4/5/96	4:30 PM	40	42.16	1.22E-03	.865E-06	.493E-06
4/15/96	4:15 PM	40	52.15	1.26E-03	.890E-06	.518E-06
4/25/96	12:30 AM	40	61.49	1.28E-03	.903E-06	.531E-06
5/3/96	10:30 PM	40	70.41	1.34E-03	.948E-06	.576E-06
5/15/96	2:20 PM	40	82.07	1.40E-03	.990E-06	.618E-06
6/7/96	5:40 PM	40	105.20	1.44E-03	.102E-05	.648E-06
6/28/96	3:10 PM	40	126.10	1.48E-03	.105E-05	.676E-06
7/19/96	12:45 PM	40	147.00	1.55E-03	.110E-05	.727E-06
8/5/96	12:10 PM	40	163.98	1.60E-03	.113E-05	.760E-06
9/10/96	11:50 AM	40	199.96	1.62E-03	1.15E-06	.775E-06
9/27/96	2:40 PM	40	217.08	1.66E-03	1.18E-06	.804E-06

SPECIMEN #5 38 A

Date	Time	Load (kips)	Time (days)	A 1	A 2	A 3	A 4	A 5	Avg.	Diff.	Strain #5 38 A
2/22/96	9:30 AM	0	0.00	0.03180	0.03196	0.03185	0.03187	0.03185	0.03187	*****	*****
2/22/96	10:00 AM	0	0.01	0.03208	0.03190	0.03186	0.03193	0.03200	0.03195	-0.00009	-011E-06
2/22/96	11:00 AM	0	0.04	0.03192	0.03190	0.03197	0.03182	0.03191	0.03190	-0.00004	-005E-06
2/22/96	12:00 PM	0	0.08	0.03221	0.03213	0.03230	0.03219	0.03220	0.03221	-0.00034	-042E-06
2/22/96	2:00 PM	0	0.17	0.03206	0.03210	0.03200	0.03199	0.03202	0.03203	-0.00017	-021E-06
2/22/96	6:00 PM	0	0.33	0.03208	0.03219	0.03217	0.03195	0.03209	0.03210	-0.00023	-029E-06
2/22/96	10:00 PM	0	0.50	0.03170	0.03170	0.03182	0.03175	0.03181	0.03176	0.00011	014E-06
2/23/96	9:30 AM	0	0.98	0.03162	0.03171	0.03172	0.03160	0.03159	0.03165	0.00022	027E-06
2/23/96	10:00 PM	0	1.50	0.03154	0.03168	0.03169	0.03170	0.03168	0.03166	0.00021	026E-06
2/24/96	10:00 AM	0	2.00	0.03165	0.03169	0.03164	0.03170	0.03169	0.03167	0.00019	024E-06
2/25/96	11:30 AM	0	3.06	0.03175	0.03171	0.03170	0.03171	0.03165	0.03170	0.00016	020E-06
2/27/96	10:30 AM	0	5.02	0.03168	0.03170	0.03162	0.03173	0.03170	0.03169	0.00018	023E-06
3/1/96	7:35 PM	0	8.40	0.03167	0.03169	0.03169	0.03165	0.03168	0.03168	0.00019	024E-06
3/11/96	3:00 PM	0	18.21	0.03165	0.03168	0.03169	0.03162	0.03166	0.03166	0.00021	026E-06
3/22/96	4:45 PM	0	29.28	0.03161	0.03165	0.03163	0.03159	0.03150	0.03160	0.00027	034E-06
4/5/96	3:45 PM	0	43.24	0.03078	0.03077	0.03081	0.03071	0.03078	0.03077	0.00110	137E-06
4/15/96	2:00 PM	0	53.17	0.03069	0.03070	0.03069	0.03061	0.03070	0.03068	0.00119	149E-06
4/24/96	9:10 PM	0	62.47	0.03039	0.03045	0.03048	0.03045	0.03045	0.03044	0.00142	178E-06
5/3/96	10:00 PM	0	71.50	0.03025	0.03029	0.03029	0.03035	0.03029	0.03029	0.00157	197E-06
5/15/96	1:45 PM	0	83.16	0.03011	0.03005	0.03009	0.03010	0.03000	0.03007	0.00180	225E-06
6/7/96	4:20 PM	0	106.26	0.03000	0.02985	0.02991	0.02995	0.02990	0.02992	0.00194	243E-06
6/28/96	2:50 PM	0	127.20	0.02985	0.02990	0.02975	0.02980	0.02985	0.02983	0.00204	255E-06
7/19/96	11:45 AM	0	148.07	0.02955	0.02952	0.02945	0.02952	0.02955	0.02952	0.00235	294E-06
8/5/96	11:10 AM	0	165.05	0.02921	0.02919	0.02915	0.02915	0.02911	0.02916	0.00270	338E-06
9/10/96	11:25 AM	0	201.06	0.02905	0.02895	0.02901	0.02892	0.02890	0.02897	0.00290	363E-06
9/27/96	2:10 PM	0	218.17	0.02878	0.02881	0.02871	0.02869	0.02872	0.02874	0.00312	391E-06

SPECIMEN #5 38 B

Date	Time	Load (kips)	Time (days)	B 1	B 2	B 3	B 4	B 5	Avg.	Diff.	Strain #5 38 B
2/22/96	9:30 AM	0	0.00	0.02958	0.02950	0.02948	0.02962	0.02964	0.02956	*****	*****
2/22/96	10:00 AM	0	0.01	0.02955	0.02961	0.02963	0.02964	0.02959	0.02960	-0.00004	-5.00E-06
2/22/96	11:00 AM	0	0.04	0.02963	0.02959	0.02965	0.02968	0.02960	0.02963	-0.00007	-8.25E-06
2/22/96	12:00 PM	0	0.08	0.02975	0.02965	0.02958	0.02961	0.02959	0.02964	-0.00007	-009E-06
2/22/96	2:00 PM	0	0.17	0.02971	0.02972	0.02960	0.02961	0.02970	0.02967	-0.00010	-013E-06
2/22/96	6:00 PM	0*	0.33	0.02669	0.02668	0.02660	0.02668	0.02651	0.02663	0.00023	029E-06
2/22/96	10:00 PM	0	0.50	0.02698	0.02689	0.02688	0.02685	0.02682	0.02688	-0.00002	-003E-06
2/23/96	9:30 AM	0	0.98	0.02704	0.02712	0.02707	0.02705	0.02708	0.02707	-0.00021	-026E-06
2/23/96	10:00 PM	0	1.50	0.02670	0.02682	0.02680	0.02682	0.02682	0.02679	0.00007	009E-06
2/24/96	10:00 AM	0	2.00	0.02664	0.02663	0.02665	0.02661	0.02665	0.02664	0.00023	029E-06
2/25/96	11:30 AM	0	3.06	0.02661	0.02662	0.02660	0.02658	0.02659	0.02660	0.00026	033E-06
2/27/96	10:30 AM	0	5.02	0.02660	0.02659	0.02665	0.02658	0.02668	0.02662	0.00024	031E-06
3/1/96	7:35 PM	0	8.40	0.02660	0.02652	0.02655	0.02656	0.02661	0.02657	0.00030	037E-06
3/11/96	3:00 PM	0	18.21	0.02650	0.02651	0.02659	0.02645	0.02648	0.02651	0.00036	045E-06
3/22/96	4:45 PM	0	29.28	0.02628	0.02623	0.02626	0.02624	0.02624	0.02625	0.00061	7.67E-05
4/5/96	3:45 PM	0	43.24	0.02610	0.02613	0.02612	0.02628	0.02625	0.02618	0.00069	086E-06
4/15/96	2:00 PM	0	53.17	0.02589	0.02590	0.02591	0.02581	0.02582	0.02587	0.00100	125E-06
4/24/96	9:10 PM	0	62.47	0.02559	0.02555	0.02559	0.02557	0.02580	0.02562	0.00124	156E-06
5/3/96	10:00 PM	0	71.50	0.02545	0.02541	0.02531	0.02545	0.02541	0.02541	0.00146	182E-06
5/15/96	1:45 PM	0	83.16	0.02529	0.02525	0.02529	0.02535	0.02539	0.02531	0.00155	194E-06
6/7/96	4:20 PM	0	106.26	0.02521	0.02530	0.02521	0.02525	0.02521	0.02524	0.00163	204E-06
6/28/96	2:50 PM	0	127.20	0.02531	0.02515	0.02518	0.02525	0.02522	0.02522	0.00164	205E-06
7/19/96	11:45 AM	0	148.07	0.02481	0.02484	0.02479	0.02475	0.02480	0.02480	0.00207	258E-06
8/5/96	11:10 AM	0	165.05	0.02455	0.02451	0.02458	0.02460	0.02458	0.02456	0.00230	288E-06
9/10/96	11:25 AM	0	201.06	0.02445	0.02450	0.02445	0.02448	0.02448	0.02447	0.00239	299E-06
9/27/96	2:10 PM	0	218.17	0.02430	0.02435	0.02430	0.02432	0.02428	0.02431	0.00255	319E-06

SPECIMEN #5 38				Avg Strain #5 38
Date	Time	Load (kips)	Time (days)	
2/22/96	9:30 AM	0	0.00	
2/22/96	10:00 AM	0	0.01	-008E-06
2/22/96	11:00 AM	0	0.04	-006E-06
2/22/96	12:00 PM	0	0.08	-026E-06
2/22/96	2:00 PM	0	0.17	-017E-06
2/22/96	6:00 PM	0*	0.33	125E-09
2/22/96	10:00 PM	0	0.50	006E-06
2/23/96	9:30 AM	0	0.98	625E-09
2/23/96	10:00 PM	0	1.50	018E-06
2/24/96	10:00 AM	0	2.00	026E-06
2/25/96	11:30 AM	0	3.06	027E-06
2/27/96	10:30 AM	0	5.02	027E-06
3/1/96	7:35 PM	0	8.40	030E-06
3/11/96	3:00 PM	0	18.21	035E-06
3/22/96	4:45 PM	0	29.28	055E-06
4/5/96	3:45 PM	0	43.24	112E-06
4/15/96	2:00 PM	0	53.17	137E-06
4/24/96	9:10 PM	0	62.47	167E-06
5/3/96	10:00 PM	0	71.50	189E-06
5/15/96	1:45 PM	0	83.16	209E-06
6/7/96	4:20 PM	0	106.26	223E-06
6/28/96	2:50 PM	0	127.20	230E-06
7/19/96	11:45 AM	0	148.07	276E-06
8/5/96	11:10 AM	0	165.05	313E-06
9/10/96	11:25 AM	0	201.06	331E-06
9/27/96	2:10 PM	0	218.17	355E-06

SPECIMEN #1 39 A

Date	Time	Load (kips)	Time (days)	A 1	A 2	A 3	A 4	A 5	Avg.	Diff.	Strain #1 39 A
2/22/96	9:30 AM	0	0.00	0.02762	0.02752	0.02764	0.02752	0.02764	0.02759	*****	*****
2/22/96	10:00 AM	40	0.01	0.02113	0.02129	0.02130	0.02125	0.02124	0.02124	0.00635	793E-06
2/22/96	11:00 AM	40	0.04	0.02070	0.02061	0.02064	0.02068	0.02071	0.02067	0.00692	865E-06
2/22/96	12:00 PM	40	0.08	0.02048	0.02045	0.02044	0.02050	0.02046	0.02047	0.00712	890E-06
2/22/96	2:00 PM	40	0.17	0.02020	0.02005	0.02009	0.02013	0.02010	0.02011	0.00747	934E-06
2/22/96	6:00 PM	40	0.33	0.02003	0.02002	0.02000	0.01998	0.02002	0.02001	0.00758	947E-06
2/22/96	10:00 PM	40	0.50	0.01975	0.01981	0.01983	0.01980	0.01970	0.01978	0.00781	976E-06
2/23/96	9:30 AM	40	0.98	0.01935	0.01940	0.01939	0.01934	0.01941	0.01938	0.00821	1.03E-03
2/23/96	10:00 PM	40	1.50	0.01920	0.01920	0.01921	0.01920	0.01913	0.01919	0.00840	1.05E-03
2/24/96	10:00 AM	40	2.00	0.01861	0.01867	0.01868	0.01862	0.01861	0.01864	0.00895	1.12E-03
2/25/96	11:30 AM	40	3.06	0.01837	0.01832	0.01832	0.01833	0.01835	0.01834	0.00925	1.16E-03
2/27/96	10:30 AM	40	5.02	0.01783	0.01779	0.01786	0.01786	0.01781	0.01783	0.00976	1.22E-03
3/1/96	7:35 PM	40	8.40	0.01759	0.01756	0.01756	0.01755	0.01752	0.01756	0.01003	1.25E-03
3/11/96	3:00 PM	40	18.21	0.01624	0.01630	0.01628	0.01630	0.01631	0.01629	0.01130	1.41E-03
3/22/96	4:45 PM	40	29.28	0.01493	0.01500	0.01498	0.01498	0.01495	0.01497	0.01262	1.58E-03
4/5/96	3:45 PM	40*	43.24	0.01391	0.01398	0.01394	0.01400	0.01399	0.01396	0.01362	1.70E-03
4/15/96	2:00 PM	40	53.17	0.01361	0.01359	0.01365	0.01369	0.01370	0.01365	0.01394	1.74E-03
4/24/96	9:10 PM	40	62.47	0.01335	0.01339	0.01345	0.01339	0.01345	0.01341	0.01418	1.77E-03
5/3/96	10:00 PM	40	71.50	0.01278	0.01280	0.01281	0.01282	0.01281	0.01280	0.01478	1.85E-03
5/15/96	1:45 PM	40	83.16	0.01225	0.01221	0.01221	0.01225	0.01225	0.01223	0.01535	1.92E-03
6/7/96	4:20 PM	40	106.26	0.01200	0.01190	0.01200	0.01195	0.01200	0.01197	0.01562	1.95E-03
6/28/96	2:50 PM	40	127.20	0.01170	0.01178	0.01169	0.01178	0.01175	0.01174	0.01585	1.98E-03
7/19/96	11:45 AM	40	148.07	0.01119	0.01129	0.01122	0.01122	0.01125	0.01123	0.01635	2.04E-03
8/5/96	11:10 AM	40	165.05	0.01081	0.01071	0.01075	0.01065	0.01069	0.01072	0.01687	2.11E-03
9/10/96	11:25 AM	40	201.06	0.01045	0.01045	0.01055	0.01042	0.01045	0.01046	0.01712	2.14E-03
9/27/96	2:10 PM	40	218.17	0.01020	0.01018	0.01018	0.01015	0.01018	0.01018	0.01741	2.18E-03

SPECIMEN #1 39 B											
Date	Time	Load (kips)	Time (days)	B					Avg.	Diff.	Strain #1 39 B
				1	2	3	4	5			
2/22/96	9:30 AM	0	0.00	0.02500	0.02507	0.02509	0.02514	0.02502	0.02506	*****	*****
2/22/96	10:00 AM	40	0.01	0.02159	0.02144	0.02158	0.02160	0.02153	0.02155	0.00352	439E-06
2/22/96	11:00 AM	40	0.04	0.02095	0.02097	0.02089	0.02095	0.02099	0.02095	0.00411	514E-06
2/22/96	12:00 PM	40	0.08	0.02070	0.02075	0.02076	0.02065	0.02071	0.02071	0.00435	544E-06
2/22/96	2:00 PM	40	0.17	0.02005	0.02013	0.02018	0.02020	0.02019	0.02015	0.00491	614E-06
2/22/96	6:00 PM	40	0.33	0.02021	0.02016	0.01994	0.02011	0.02008	0.02010	0.00496	621E-06
2/22/96	10:00 PM	40	0.50	0.01975	0.01993	0.01988	0.01984	0.01990	0.01986	0.00520	651E-06
2/23/96	9:30 AM	40	0.98	0.01945	0.01950	0.01949	0.01946	0.01938	0.01946	0.00561	701E-06
2/23/96	10:00 PM	40	1.50	0.01928	0.01921	0.01922	0.01920	0.01913	0.01921	0.00586	732E-06
2/24/96	10:00 AM	40	2.00	0.01859	0.01866	0.01867	0.01861	0.01867	0.01864	0.00642	803E-06
2/25/96	11:30 AM	40	3.06	0.01851	0.01841	0.01845	0.01843	0.01847	0.01845	0.00661	826E-06
2/27/96	10:30 AM	40	5.02	0.01785	0.01782	0.01783	0.01782	0.01785	0.01783	0.00723	904E-06
3/1/96	7:35 PM	40	8.40	0.01762	0.01768	0.01764	0.01762	0.01761	0.01763	0.00743	929E-06
3/11/96	3:00 PM	40	18.21	0.01628	0.01621	0.01619	0.01621	0.01623	0.01622	0.00884	1.11E-03
3/22/96	4:45 PM	40	29.28	0.01533	0.01546	0.01542	0.01534	0.01541	0.01539	0.00967	1.21E-03
4/5/96	3:45 PM	40*	43.24	0.01480	0.01460	0.01478	0.01471	0.01479	0.01474	0.01033	1.29E-03
4/15/96	2:00 PM	40	53.17	0.01435	0.01439	0.01435	0.01429	0.01435	0.01435	0.01072	1.34E-03
4/24/96	9:10 PM	40	62.47	0.01331	0.01335	0.01335	0.01339	0.01345	0.01337	0.01169	1.46E-03
5/3/96	10:00 PM	40	71.50	0.01285	0.01275	0.01280	0.01280	0.01278	0.01280	0.01227	1.53E-03
5/15/96	1:45 PM	40	83.16	0.01235	0.01232	0.01232	0.01235	0.01231	0.01233	0.01273	1.59E-03
6/7/96	4:20 PM	40	106.26	0.01195	0.01199	0.01200	0.01195	0.01198	0.01197	0.01309	1.64E-03
6/28/96	2:50 PM	40	127.20	0.01178	0.01175	0.01169	0.01172	0.01175	0.01174	0.01333	1.67E-03
7/19/96	11:45 AM	40	148.07	0.01129	0.01128	0.01129	0.01131	0.01135	0.01130	0.01376	1.72E-03
8/5/96	11:10 AM	40	165.05	0.01095	0.01092	0.01090	0.01090	0.01091	0.01092	0.01415	1.77E-03
9/10/96	11:25 AM	40	201.06	0.01055	0.01052	0.01055	0.01055	0.01055	0.01054	0.01452	1.82E-03
9/27/96	2:10 PM	40	218.17	0.01030	0.01028	0.01025	0.01020	0.01028	0.01026	0.01480	1.85E-03

SPECIMEN #1 39				Avg Strain #1 39	Specific Strain #1 39	Corrected Spec Strain #1 39
Date	Time	Load (kips)	Time (days)			
2/22/96	9:30 AM	0	0.00			
2/22/96	10:00 AM	40	0.01	616E-06	.436E-06	.000E+00
2/22/96	11:00 AM	40	0.04	690E-06	.487E-06	.518E-07
2/22/96	12:00 PM	40	0.08	717E-06	.507E-06	.711E-07
2/22/96	2:00 PM	40	0.17	774E-06	.547E-06	.112E-06
2/22/96	6:00 PM	40	0.33	784E-06	.554E-06	.118E-06
2/22/96	10:00 PM	40	0.50	813E-06	.575E-06	.139E-06
2/23/96	9:30 AM	40	0.98	864E-06	.610E-06	.175E-06
2/23/96	10:00 PM	40	1.50	891E-06	.630E-06	.194E-06
2/24/96	10:00 AM	40	2.00	961E-06	.679E-06	.243E-06
2/25/96	11:30 AM	40	3.06	991E-06	.701E-06	.265E-06
2/27/96	10:30 AM	40	5.02	1.06E-03	.750E-06	.315E-06
3/1/96	7:35 PM	40	8.40	1.09E-03	.771E-06	.336E-06
3/11/96	3:00 PM	40	18.21	1.26E-03	.890E-06	.454E-06
3/22/96	4:45 PM	40	29.28	1.39E-03	.985E-06	.549E-06
4/5/96	3:45 PM	40*	43.24	1.50E-03	1.06E-06	.622E-06
4/15/96	2:00 PM	40	53.17	1.54E-03	1.09E-06	.654E-06
4/24/96	9:10 PM	40	62.47	1.62E-03	1.14E-06	.707E-06
5/3/96	10:00 PM	40	71.50	1.69E-03	1.19E-06	.759E-06
5/15/96	1:45 PM	40	83.16	1.76E-03	1.24E-06	.805E-06
6/7/96	4:20 PM	40	106.26	1.79E-03	1.27E-06	.832E-06
6/28/96	2:50 PM	40	127.20	1.82E-03	1.29E-06	.853E-06
7/19/96	11:45 AM	40	148.07	1.88E-03	1.33E-06	.895E-06
8/5/96	11:10 AM	40	165.05	1.94E-03	1.37E-06	.934E-06
9/10/96	11:25 AM	40	201.06	1.98E-03	1.40E-06	.962E-06
9/27/96	2:10 PM	40	218.17	2.01E-03	1.42E-06	.987E-06

SPECIMEN #3 39 A

Date	Time	Load (kips)	Time (days)	A 1	A 2	A 3	A 4	A 5	Avg.	Diff.	Strain #3 39 A
2/23/96	12:00 PM	0	0.00	0.02361	0.02371	0.02378	0.02369	0.02370	0.02370	*****	*****
2/23/96	12:25 PM	20	0.00	0.02248	0.02231	0.02229	0.02230	0.02235	0.02235	*****	*****
2/23/96	12:45 PM	40	0.01	0.02128	0.02113	0.02124	0.02122	0.02115	0.02120	0.00249	312E-06
2/23/96	1:45 PM	40	0.04	0.02021	0.02032	0.02026	0.02029	0.02026	0.02027	0.00343	429E-06
2/23/96	2:45 PM	40	0.08	0.01977	0.01980	0.01972	0.01986	0.01976	0.01978	0.00392	490E-06
2/23/96	4:45 PM	40	0.17	0.01980	0.01981	0.01983	0.01986	0.01990	0.01984	0.00386	482E-06
2/23/96	8:45 PM	40	0.33	0.01952	0.01962	0.01953	0.01964	0.01957	0.01958	0.00412	515E-06
2/24/96	12:45 AM	40	0.50	0.01898	0.01901	0.01890	0.01888	0.01894	0.01894	0.00476	595E-06
2/24/96	12:45 PM	40	1.00	0.01890	0.01891	0.01876	0.01889	0.01885	0.01886	0.00484	605E-06
2/25/96	12:45 AM	40	1.50	0.01862	0.01867	0.01861	0.01861	0.01864	0.01863	0.00507	634E-06
2/25/96	12:45 PM	40	2.00	0.01860	0.01861	0.01857	0.01857	0.01855	0.01858	0.00512	640E-06
2/26/96	1:00 PM	40	3.01	0.01853	0.01851	0.01855	0.01857	0.01856	0.01854	0.00515	644E-06
3/1/96	7:05 PM	40	7.26	0.01822	0.01815	0.01819	0.01813	0.01812	0.01816	0.00554	692E-06
3/11/96	1:45 PM	40	17.04	0.01772	0.01780	0.01768	0.01765	0.01767	0.01770	0.00599	749E-06
3/22/96	5:15 PM	40	28.19	0.01680	0.01688	0.01681	0.01682	0.01685	0.01683	0.00687	858E-06
4/5/96	4:30 PM	40	42.16	0.01568	0.01568	0.01562	0.01569	0.01562	0.01566	0.00804	1.01E-03
4/15/96	4:15 PM	40	52.15	0.01540	0.01545	0.01542	0.01549	0.01545	0.01544	0.00826	1.03E-03
4/25/96	12:30 AM	40	61.49	0.01531	0.01535	0.01535	0.01530	0.01531	0.01532	0.00837	1.05E-03
5/3/96	10:30 PM	40	70.41	0.01480	0.01485	0.01480	0.01479	0.01488	0.01482	0.00887	1.11E-03
5/15/96	2:20 PM	40	82.07	0.01449	0.01445	0.01441	0.01445	0.01442	0.01444	0.00925	1.16E-03
6/7/96	5:40 PM	40	105.20	0.01419	0.01425	0.01429	0.01419	0.01425	0.01423	0.00946	1.18E-03
6/28/96	3:10 PM	40	126.10	0.01415	0.01411	0.01409	0.01390	0.01395	0.01404	0.00966	1.21E-03
7/19/96	12:45 PM	40	147.00	0.01365	0.01359	0.01360	0.01355	0.01361	0.01360	0.01010	1.26E-03
8/5/96	12:10 PM	40	163.98	0.01325	0.01329	0.01319	0.01319	0.01321	0.01323	0.01047	1.31E-03
9/10/96	11:50 AM	40	199.96	0.01301	0.01300	0.01300	0.01305	0.01302	0.01302	0.01068	1.34E-03
9/27/96	2:40 PM	40	217.08	0.01278	0.01282	0.01278	0.01281	0.01278	0.01279	0.01090	1.36E-03

SPECIMEN #3 39 B

Date	Time	Load (kips)	Time (days)	B 1	B 2	B 3	B 4	B 5	Avg.	Diff.	Strain #3 39 B
2/23/96	12:00 PM	0	0.00	0.03448	0.03449	0.03458	0.03451	0.03444	0.03450	*****	*****
2/23/96	12:25 PM	20	0.00	0.03228	0.03230	0.03231	0.03225	0.03218	0.03226	*****	*****
2/23/96	12:45 PM	40	0.01	0.02950	0.02941	0.02951	0.02948	0.02950	0.02948	0.00502	628E-06
2/23/96	1:45 PM	40	0.04	0.02923	0.02917	0.02920	0.02921	0.02921	0.02920	0.00530	662E-06
2/23/96	2:45 PM	40	0.08	0.02905	0.02910	0.02903	0.02907	0.02908	0.02907	0.00543	679E-06
2/23/96	4:45 PM	40	0.17	0.02868	0.02870	0.02865	0.02868	0.02858	0.02866	0.00584	730E-06
2/23/96	8:45 PM	40	0.33	0.02832	0.02831	0.02829	0.02826	0.02831	0.02830	0.00620	775E-06
2/24/96	12:45 AM	40	0.50	0.02781	0.02781	0.02780	0.02778	0.02783	0.02781	0.00669	837E-06
2/24/96	12:45 PM	40	1.00	0.02768	0.02762	0.02769	0.02763	0.02768	0.02766	0.00684	855E-06
2/25/96	12:45 AM	40	1.50	0.02718	0.02728	0.02723	0.02724	0.02723	0.02723	0.00727	909E-06
2/25/96	12:45 PM	40	2.00	0.02682	0.02691	0.02690	0.02688	0.02690	0.02688	0.00762	952E-06
2/26/96	1:00 PM	40	3.01	0.02650	0.02655	0.02656	0.02659	0.02653	0.02655	0.00795	994E-06
3/1/96	7:05 PM	40	7.26	0.02628	0.02627	0.02628	0.02625	0.02620	0.02626	0.00824	1.03E-03
3/11/96	1:45 PM	40	17.04	0.02470	0.02471	0.02468	0.02467	0.02469	0.02469	0.00981	1.23E-03
3/22/96	5:15 PM	40	28.19	0.02357	0.02350	0.02351	0.02358	0.02357	0.02355	0.01095	1.37E-03
4/5/96	4:30 PM	40	42.16	0.02199	0.02185	0.02199	0.02190	0.02199	0.02194	0.01256	1.57E-03
4/15/96	4:15 PM	40	52.15	0.02165	0.02155	0.02162	0.02155	0.02160	0.02159	0.01291	1.61E-03
4/25/96	12:30 AM	40	61.49	0.02140	0.02145	0.02139	0.02145	0.02148	0.02143	0.01307	1.63E-03
5/3/96	10:30 PM	40	70.41	0.02090	0.02089	0.02090	0.02085	0.02089	0.02089	0.01361	1.70E-03
5/15/96	2:20 PM	40	82.07	0.02045	0.02040	0.02045	0.02035	0.02039	0.02041	0.01409	1.76E-03
6/7/96	5:40 PM	40	105.20	0.01995	0.01991	0.01990	0.01991	0.01995	0.01992	0.01458	1.82E-03
6/28/96	3:10 PM	40	126.10	0.01955	0.01950	0.01950	0.01952	0.01951	0.01952	0.01498	1.87E-03
7/19/96	12:45 PM	40	147.00	0.01880	0.01885	0.01885	0.01885	0.01881	0.01883	0.01567	1.96E-03
8/5/96	12:10 PM	40	163.98	0.01842	0.01845	0.01842	0.01841	0.01841	0.01842	0.01608	2.01E-03
9/10/96	11:50 AM	40	199.96	0.01812	0.01811	0.01815	0.01810	0.01815	0.01813	0.01637	2.05E-03
9/27/96	2:40 PM	40	217.08	0.01775	0.01781	0.01775	0.01772	0.01772	0.01775	0.01675	2.09E-03

SPECIMEN #3 39				Avg	Specific	Corrected
Date	Time	Load (kips)	Time (days)	Strain #3 39	Strain #3 39	Spec Stain #3 39
2/23/96	12:00 PM	0	0.00			
2/23/96	12:25 PM	20	0.00			
2/23/96	12:45 PM	40	0.01	470E-06	.332E-06	.000E+00
2/23/96	1:45 PM	40	0.04	545E-06	.385E-06	.535E-07
2/23/96	2:45 PM	40	0.08	584E-06	.413E-06	.811E-07
2/23/96	4:45 PM	40	0.17	606E-06	.428E-06	.966E-07
2/23/96	8:45 PM	40	0.33	645E-06	.456E-06	.124E-06
2/24/96	12:45 AM	40	0.50	716E-06	.506E-06	.174E-06
2/24/96	12:45 PM	40	1.00	730E-06	.516E-06	.184E-06
2/25/96	12:45 AM	40	1.50	771E-06	.545E-06	.213E-06
2/25/96	12:45 PM	40	2.00	796E-06	.563E-06	.231E-06
2/26/96	1:00 PM	40	3.01	819E-06	.579E-06	.247E-06
3/1/96	7:05 PM	40	7.26	861E-06	.609E-06	.277E-06
3/11/96	1:45 PM	40	17.04	988E-06	.698E-06	.366E-06
3/22/96	5:15 PM	40	28.19	1.11E-03	.787E-06	.455E-06
4/5/96	4:30 PM	40	42.16	1.29E-03	.910E-06	.578E-06
4/15/96	4:15 PM	40	52.15	1.32E-03	.935E-06	.603E-06
4/25/96	12:30 AM	40	61.49	1.34E-03	.947E-06	.615E-06
5/3/96	10:30 PM	40	70.41	1.41E-03	.993E-06	.661E-06
5/15/96	2:20 PM	40	82.07	1.46E-03	.103E-05	.699E-06
6/7/96	5:40 PM	40	105.20	1.50E-03	.106E-05	.730E-06
6/28/96	3:10 PM	40	126.10	1.54E-03	.109E-05	.757E-06
7/19/96	12:45 PM	40	147.00	1.61E-03	.114E-05	.806E-06
8/5/96	12:10 PM	40	163.98	1.66E-03	.117E-05	.841E-06
9/10/96	11:50 AM	40	199.96	1.69E-03	1.20E-06	.863E-06
9/27/96	2:40 PM	40	217.08	1.73E-03	1.22E-06	.890E-06

SPECIMEN #5 39 A											
Date	Time	Load (kips)	Time (days)	A					Avg.	Diff.	Strain #5 39 A
				1	2	3	4	5			
2/23/96	12:00 PM	0	0.00	0.03086	0.03080	0.03069	0.03080	0.03086	0.03080	*****	*****
2/23/96	12:25 PM	0	0.00	0.03086	0.03080	0.03069	0.03080	0.03086	0.03080	*****	*****
2/23/96	12:45 PM	0	0.01	0.03086	0.03080	0.03069	0.03080	0.03086	0.03080	0.00000	0.00E+00
2/23/96	1:45 PM	0	0.04	0.03113	0.03110	0.03112	0.03102	0.03112	0.03110	-0.00030	-037E-06
2/23/96	2:45 PM	0	0.08	0.03089	0.03098	0.03081	0.03080	0.03097	0.03089	-0.00009	-011E-06
2/23/96	4:45 PM	0	0.17	0.03078	0.03082	0.03072	0.03086	0.03085	0.03081	0.00000	-500E-09
2/23/96	8:45 PM	0	0.33	0.03079	0.03082	0.03086	0.03080	0.03079	0.03081	-0.00001	-001E-06
2/24/96	12:45 AM	0	0.50	0.03080	0.03083	0.03086	0.03085	0.03082	0.03083	-0.00003	-004E-06
2/24/96	12:45 PM	0	1.00	0.03078	0.03080	0.03075	0.03082	0.03077	0.03078	0.00002	002E-06
2/25/96	12:45 AM	0	1.50	0.03081	0.03080	0.03090	0.03087	0.03082	0.03084	-0.00004	-005E-06
2/25/96	12:45 PM	0	2.00	0.03089	0.03080	0.03087	0.03088	0.03081	0.03085	-0.00005	-006E-06
2/26/96	1:00 PM	0	3.01	0.03076	0.03080	0.03071	0.03080	0.03081	0.03078	0.00003	003E-06
3/1/96	7:05 PM	0	7.26	0.02920	0.02910	0.02913	0.02911	0.02912	0.02913	0.00167	209E-06
3/11/96	1:45 PM	0	17.04	0.02885	0.02879	0.02882	0.02887	0.02890	0.02885	0.00196	245E-06
3/22/96	5:15 PM	0	28.19	0.02811	0.02818	0.02819	0.02815	0.02816	0.02816	0.00264	331E-06
4/5/96	4:30 PM	0	42.16	0.02768	0.02760	0.02762	0.02765	0.02759	0.02763	0.00317	397E-06
4/15/96	4:15 PM	0	52.15	0.02745	0.02745	0.02749	0.02740	0.02742	0.02744	0.00336	420E-06
4/25/96	12:30 AM	0	61.49	0.02745	0.02740	0.02742	0.02745	0.02742	0.02743	0.00337	422E-06
5/3/96	10:30 PM	0	70.41	0.02741	0.02739	0.02741	0.02739	0.02739	0.02740	0.00340	426E-06
5/15/96	2:20 PM	0	82.07	0.02729	0.02725	0.02729	0.02731	0.02729	0.02729	0.00352	440E-06
6/7/96	5:40 PM	0	105.20	0.02705	0.02702	0.02709	0.02705	0.02710	0.02706	0.00374	468E-06
6/28/96	3:10 PM	0	126.10	0.02691	0.02695	0.02685	0.02685	0.02690	0.02689	0.00391	489E-06
7/19/96	12:45 PM	0	147.00	0.02655	0.02661	0.02651	0.02655	0.02661	0.02657	0.00424	530E-06
8/5/96	12:10 PM	0	163.98	0.02621	0.02629	0.02625	0.02619	0.02611	0.02621	0.00459	574E-06
9/10/96	11:50 AM	0	199.96	0.02610	0.02609	0.02605	0.02605	0.02605	0.02607	0.00473	592E-06
9/27/96	2:40 PM	0	217.08	0.02585	0.02588	0.02580	0.02588	0.02578	0.02584	0.00496	621E-06

SPECIMEN #5 39 B

Date	Time	Load (kips)	Time (days)	B 1	B 2	B 3	B 4	B 5	Avg.	Diff.	Strain #5 39 B
2/23/96	12:00 PM	0	0.00							*****	*****
2/23/96	12:25 PM	0	0.00							*****	*****
2/23/96	12:45 PM	0	0.01							0.00000	0.00E+00
2/23/96	1:45 PM	0	0.04	0.03227	0.03231	0.03232	0.03236	0.03230	0.03231	-0.03231	-4.04E-03
2/23/96	2:45 PM	0	0.08	Stud Loose!						0.00000	0.00E+00
2/23/96	4:45 PM	0	0.17							0.00000	0.00E+00
2/23/96	8:45 PM	0	0.33							0.00000	0.00E+00
2/24/96	12:45 AM	0	0.50							0.00000	0.00E+00
2/24/96	12:45 PM	0	1.00	0.02712	0.02716	0.02720	0.02714	0.02721	0.02717	-0.02717	-3.40E-03
2/25/96	12:45 AM	0	1.50	0.02698	0.02713	0.02710	0.02717	0.02711	0.02710	0.00007	009E-06
2/25/96	12:45 PM	0	2.00	0.02719	0.02720	0.02709	0.02715	0.02715	0.02716	0.00001	001E-06
2/26/96	1:00 PM	0	3.01	0.02715	0.02720	0.02722	0.02728	0.02728	0.02723	-0.00006	-007E-06
3/1/96	7:05 PM	0	7.26	0.02712	0.02720	0.02725	0.02713	0.02722	0.02718	-0.00002	-002E-06
3/11/96	1:45 PM	0	17.04	0.02723	0.02719	0.02729	0.02726	0.02721	0.02724	-0.00007	-009E-06
3/22/96	5:15 PM	0	28.19	0.02700	0.02705	0.02698	0.02700	0.02700	0.02701	0.00016	020E-06
4/5/96	4:30 PM	0	42.16	0.02658	0.02651	0.02650	0.02650	0.02648	0.02651	0.00065	082E-06
4/15/96	4:15 PM	0	52.15	0.02625	0.02629	0.02625	0.02615	0.02615	0.02622	0.00095	119E-06
4/25/96	12:30 AM	0	61.49	0.02615	0.02619	0.02612	0.02612	0.02615	0.02615	0.00102	128E-06
5/3/96	10:30 PM	0	70.41	0.02599	0.02600	0.02601	0.02600	0.02595	0.02599	0.00118	147E-06
5/15/96	2:20 PM	0	82.07	0.02585	0.02595	0.02590	0.02589	0.02581	0.02588	0.00129	161E-06
6/7/96	5:40 PM	0	105.20	0.02572	0.02565	0.02571	0.02561	0.02565	0.02567	0.00150	187E-06
6/28/96	3:10 PM	0	126.10	0.02570	0.02569	0.02569	0.02572	0.02571	0.02570	0.00146	183E-06
7/19/96	12:45 PM	0	147.00	0.02530	0.02531	0.02530	0.02525	0.02535	0.02530	0.00186	233E-06
8/5/96	12:10 PM	0	163.98	0.02505	0.02498	0.02501	0.02502	0.02495	0.02500	0.00216	271E-06
9/10/96	11:50 AM	0	199.96	0.02491	0.02490	0.02491	0.02490	0.02485	0.02489	0.00227	284E-06
9/27/96	2:40 PM	0	217.08	0.02475	0.02472	0.02478	0.02478	0.02481	0.02477	0.00240	300E-06

SPECIMEN #5 39				Avg Strain #5 39
Date	Time	Load (kips)	Time (days)	
2/23/96	12:00 PM	0	0.00	
2/23/96	12:25 PM	0	0.00	
2/23/96	12:45 PM	0	0.01	0.00E+00
2/23/96	1:45 PM	0	0.04	-2.04E-03
2/23/96	2:45 PM	0	0.08	-5.50E-06
2/23/96	4:45 PM	0	0.17	-2.50E-07
2/23/96	8:45 PM	0	0.33	-6.25E-07
2/24/96	12:45 AM	0	0.50	-1.88E-06
2/24/96	12:45 PM	0	1.00	-1.70E-03
2/25/96	12:45 AM	0	1.50	002E-06
2/25/96	12:45 PM	0	2.00	-002E-06
2/26/96	1:00 PM	0	3.01	-002E-06
3/1/96	7:05 PM	0	7.26	103E-06
3/11/96	1:45 PM	0	17.04	118E-06
3/22/96	5:15 PM	0	28.19	175E-06
4/5/96	4:30 PM	0	42.16	239E-06
4/15/96	4:15 PM	0	52.15	269E-06
4/25/96	12:30 AM	0	61.49	275E-06
5/3/96	10:30 PM	0	70.41	286E-06
5/15/96	2:20 PM	0	82.07	300E-06
6/7/96	5:40 PM	0	105.20	327E-06
6/28/96	3:10 PM	0	126.10	336E-06
7/19/96	12:45 PM	0	147.00	381E-06
8/5/96	12:10 PM	0	163.98	422E-06
9/10/96	11:50 AM	0	199.96	438E-06
9/27/96	2:40 PM	0	217.08	460E-06

REFERENCE SPECIMEN #1 - LOADED - A

Date	Time	Load (kips)	Time (days)	A 1	A 2	A 3	A 4	A 5	Avg.	Diff.	Strain Ref #1-A
4/24/96	1:50 PM	0	0.00	0.02475	0.02478	0.02475	0.02468	0.02478	0.02475	*****	*****
4/24/96	2:10 PM	35	0.01	0.02235	0.02232	0.02235	0.02235	0.02235	0.02234	0.00240	301E-06
4/24/96	3:10 PM	35	0.04	0.02225	0.02228	0.02225	0.02228	0.02229	0.02227	0.00248	310E-06
4/24/96	4:10 PM	35	0.08	0.02229	0.02225	0.02231	0.02225	0.02231	0.02228	0.00247	308E-06
4/24/96	6:15 PM	35	0.17	0.02238	0.02235	0.02238	0.02231	0.02230	0.02234	0.00240	301E-06
4/24/96	10:10 PM	35	0.33	0.02195	0.02191	0.02198	0.02192	0.02198	0.02195	0.00280	350E-06
4/25/96	1:45 AM	35	0.48	0.02195	0.02190	0.02199	0.02198	0.02198	0.02196	0.00279	349E-06
4/25/96	2:10 PM	35	1.00	0.02185	0.02189	0.02190	0.02191	0.02185	0.02188	0.00287	359E-06
4/26/96	1:40 AM	35	1.48	0.02150	0.02145	0.02148	0.02148	0.02151	0.02148	0.00326	408E-06
4/26/96	2:10 PM	35	2.00	0.02135	0.02131	0.02129	0.02131	0.02125	0.02130	0.00345	431E-06
4/27/96	12:00 AM	35	2.41	0.02125	0.02129	0.02125	0.02131	0.02121	0.02126	0.00349	436E-06
4/28/96	2:10 PM	35	4.00	0.02100	0.02105	0.02111	0.02105	0.02111	0.02106	0.00368	461E-06
5/1/96	2:10 PM	35	7.00	0.02048	0.02045	0.02048	0.02041	0.02045	0.02045	0.00429	537E-06
5/4/96	12:10 AM	35	9.42	0.02010	0.02008	0.02008	0.02005	0.02011	0.02008	0.00466	583E-06
5/8/96	12:10 PM	35	13.92	0.01980	0.01975	0.01970	0.01970	0.01969	0.01973	0.00502	628E-06
5/15/96	12:40 PM	35	20.94	0.01895	0.01889	0.01889	0.01885	0.01888	0.01889	0.00586	732E-06
5/22/96	3:00 PM	35	28.03	0.01821	0.01819	0.01821	0.01820	0.01825	0.01821	0.00654	817E-06
5/29/96	1:45 PM	35	34.98	0.01775	0.01771	0.01769	0.01771	0.01772	0.01772	0.00703	879E-06
6/7/96	3:10 PM	35	44.04	0.01755	0.01756	0.01758	0.01752	0.01755	0.01755	0.00720	900E-06
6/21/96	12:30 AM	35	57.43	0.01715	0.01712	0.01710	0.01705	0.01712	0.01711	0.00764	955E-06
7/5/96	10:45 AM	35	71.86	0.01651	0.01655	0.01661	0.01652	0.01658	0.01655	0.00819	1.02E-03
7/19/96	11:00 AM	35	85.87	0.01595	0.01595	0.01592	0.01591	0.01595	0.01594	0.00881	1.10E-03
8/5/96	9:50 AM	35	102.82	0.01565	0.01559	0.01561	0.01559	0.01551	0.01559	0.00916	1.14E-03
9/10/96	10:10 AM	35	138.83	0.01481	0.01475	0.01481	0.01478	0.01475	0.01478	0.00997	1.25E-03
9/27/96	1:10 PM	35	155.96	0.01448	0.01449	0.01445	0.01448	0.01450	0.01448	0.01027	1.28E-03

REFERENCE SPECIMEN #1 - LOADED - B

Date	Time	Load (kips)	Time (days)	B 1	B 2	B 3	B 4	B 5	Avg.	Diff.	Strain Ref #1-B
4/24/96	1:50 PM	0	0.00	0.02635	0.02630	0.02635	0.02632	0.02640	0.02634	*****	*****
4/24/96	2:10 PM	35	0.01	0.02445	0.02445	0.02441	0.02449	0.02442	0.02444	0.00190	238E-06
4/24/96	3:10 PM	35	0.04	0.02390	0.02389	0.02395	0.02385	0.02389	0.02390	0.00245	306E-06
4/24/96	4:10 PM	35	0.08	0.02382	0.02389	0.02385	0.02390	0.02382	0.02386	0.00249	311E-06
4/24/96	6:15 PM	35	0.17	0.02389	0.02395	0.02399	0.02391	0.02389	0.02393	0.00242	302E-06
4/24/96	10:10 PM	35	0.33	0.02361	0.02359	0.02359	0.02362	0.02365	0.02361	0.00273	342E-06
4/25/96	1:45 AM	35	0.48	0.02365	0.02365	0.02368	0.02368	0.02361	0.02365	0.00269	336E-06
4/25/96	2:10 PM	35	1.00	0.02335	0.02341	0.02333	0.02336	0.02340	0.02337	0.00297	372E-06
4/26/96	1:40 AM	35	1.48	0.02299	0.02295	0.02300	0.02305	0.02306	0.02301	0.00333	417E-06
4/26/96	2:10 PM	35	2.00	0.02285	0.02289	0.02282	0.02281	0.02289	0.02285	0.00349	437E-06
4/27/96	12:00 AM	35	2.41	0.02281	0.02275	0.02278	0.02281	0.02280	0.02279	0.00355	444E-06
4/28/96	2:10 PM	35	4.00	0.02261	0.02265	0.02259	0.02260	0.02265	0.02262	0.00372	466E-06
5/1/96	2:10 PM	35	7.00	0.02250	0.02245	0.02248	0.02245	0.02240	0.02246	0.00389	486E-06
5/4/96	12:10 AM	35	9.42	0.02219	0.02215	0.02221	0.02211	0.02215	0.02216	0.00418	523E-06
5/8/96	12:10 PM	35	13.92	0.02195	0.02205	0.02202	0.02195	0.02191	0.02198	0.00437	546E-06
5/15/96	12:40 PM	35	20.94	0.02141	0.02142	0.02139	0.02141	0.02140	0.02141	0.00494	617E-06
5/22/96	3:00 PM	35	28.03	0.02089	0.02085	0.02089	0.02091	0.02082	0.02087	0.00547	684E-06
5/29/96	1:45 PM	35	34.98	0.02045	0.02045	0.02049	0.02042	0.02039	0.02044	0.00590	738E-06
6/7/96	3:10 PM	35	44.04	0.02025	0.02021	0.02025	0.02021	0.02026	0.02024	0.00611	764E-06
6/21/96	12:30 AM	35	57.43	0.01969	0.01970	0.01971	0.01965	0.01964	0.01968	0.00667	833E-06
7/5/96	10:45 AM	35	71.86	0.01905	0.01901	0.01899	0.01900	0.01898	0.01901	0.00734	917E-06
7/19/96	11:00 AM	35	85.87	0.01851	0.01852	0.01851	0.01855	0.01852	0.01852	0.00782	978E-06
8/5/96	9:50 AM	35	102.82	0.01818	0.01815	0.01811	0.01811	0.01809	0.01813	0.00822	1.03E-03
9/10/96	10:10 AM	35	138.83	0.01732	0.01731	0.01735	0.01731	0.01735	0.01733	0.00902	1.13E-03
9/27/96	1:10 PM	35	155.96	0.01708	0.01705	0.01708	0.01705	0.01711	0.01707	0.00927	1.16E-03

REF SPECIMEN #1 - LOADED				Avg	Specific	Corrected
Date	Time	Load (kips)	Time (days)	Strain Ref #1	Strain Ref #1	Spec Strain Ref #1
4/24/96	1:50 PM	0	0.00			
4/24/96	2:10 PM	35	0.01	269E-06	.217E-06	.000E+00
4/24/96	3:10 PM	35	0.04	308E-06	.249E-06	.314E-07
4/24/96	4:10 PM	35	0.08	310E-06	.250E-06	.328E-07
4/24/96	6:15 PM	35	0.17	301E-06	.243E-06	.261E-07
4/24/96	10:10 PM	35	0.33	346E-06	.279E-06	.620E-07
4/25/96	1:45 AM	35	0.48	342E-06	.277E-06	.593E-07
4/25/96	2:10 PM	35	1.00	365E-06	.295E-06	.776E-07
4/26/96	1:40 AM	35	1.48	412E-06	.333E-06	.116E-06
4/26/96	2:10 PM	35	2.00	434E-06	.350E-06	.133E-06
4/27/96	12:00 AM	35	2.41	440E-06	.355E-06	.138E-06
4/28/96	2:10 PM	35	4.00	463E-06	.374E-06	.157E-06
5/1/96	2:10 PM	35	7.00	511E-06	.413E-06	.196E-06
5/4/96	12:10 AM	35	9.42	553E-06	.447E-06	.229E-06
5/8/96	12:10 PM	35	13.92	587E-06	.474E-06	.257E-06
5/15/96	12:40 PM	35	20.94	675E-06	.545E-06	.328E-06
5/22/96	3:00 PM	35	28.03	751E-06	.606E-06	.389E-06
5/29/96	1:45 PM	35	34.98	809E-06	.653E-06	.436E-06
6/7/96	3:10 PM	35	44.04	832E-06	.672E-06	.454E-06
6/21/96	12:30 AM	35	57.43	894E-06	.722E-06	.505E-06
7/5/96	10:45 AM	35	71.86	971E-06	.784E-06	.567E-06
7/19/96	11:00 AM	35	85.87	1.04E-03	.840E-06	.622E-06
8/5/96	9:50 AM	35	102.82	1.09E-03	.877E-06	.660E-06
9/10/96	10:10 AM	35	138.83	1.19E-03	9.58E-07	7.41E-07
9/27/96	1:10 PM	35	155.96	1.22E-03	9.86E-07	7.69E-07

REFERENCE SPECIMEN #5 - LOADED - A											
Date	Time	Load (kips)	Time (days)	A					Avg.	Diff.	Strain Ref #5-A
				1	2	3	4	5			
4/24/96	1:50 PM	0	0.00	0.02785	0.02781	0.02789	0.02780	0.02779	0.02783	*****	*****
4/24/96	2:10 PM	35	0.01	0.02550	0.02555	0.02558	0.02555	0.02558	0.02555	0.00228	285E-06
4/24/96	3:10 PM	35	0.04	0.02559	0.02561	0.02555	0.02552	0.02552	0.02556	0.00227	284E-06
4/24/96	4:10 PM	35	0.08	0.02539	0.02541	0.02545	0.02539	0.02539	0.02541	0.00242	303E-06
4/24/96	6:15 PM	35	0.17	0.02541	0.02548	0.02541	0.02545	0.02545	0.02544	0.00239	299E-06
4/24/96	10:10 PM	35	0.33	0.02515	0.02519	0.02515	0.02515	0.02512	0.02515	0.00268	335E-06
4/25/96	1:45 AM	35	0.48	0.02492	0.02495	0.02501	0.02499	0.02500	0.02497	0.00285	357E-06
4/25/96	2:10 PM	35	1.00	0.02480	0.02475	0.02478	0.02475	0.02478	0.02477	0.00306	382E-06
4/26/96	1:40 AM	35	1.48	0.02438	0.02435	0.02438	0.02441	0.02438	0.02438	0.00345	431E-06
4/26/96	2:10 PM	35	2.00	0.02418	0.02415	0.02415	0.02415	0.02412	0.02415	0.00368	460E-06
4/27/96	12:00 AM	35	2.41	0.02409	0.02415	0.02418	0.02405	0.02411	0.02412	0.00371	464E-06
4/28/96	2:10 PM	35	4.00	0.02395	0.02398	0.02390	0.02391	0.02394	0.02394	0.00389	487E-06
5/1/96	2:10 PM	35	7.00	0.02350	0.02351	0.02345	0.02351	0.02349	0.02349	0.00434	542E-06
5/4/96	12:10 AM	35	9.42	0.02309	0.02310	0.02311	0.02309	0.02305	0.02309	0.00474	593E-06
5/8/96	12:10 PM	35	13.92	0.02275	0.02270	0.02271	0.02271	0.02275	0.02272	0.00510	638E-06
5/15/96	12:40 PM	35	20.94	0.02195	0.02198	0.02191	0.02195	0.02192	0.02194	0.00589	736E-06
5/22/96	3:00 PM	35	28.03	0.02155	0.02150	0.02158	0.02150	0.02151	0.02153	0.00630	788E-06
5/29/96	1:45 PM	35	34.98	0.02115	0.02111	0.02111	0.02110	0.02109	0.02111	0.00672	840E-06
6/7/96	3:10 PM	35	44.04	0.02095	0.02095	0.02089	0.02096	0.02086	0.02092	0.00691	863E-06
6/21/96	12:30 AM	35	57.43	0.02052	0.02050	0.02051	0.02049	0.02049	0.02050	0.00733	916E-06
7/5/96	10:45 AM	35	71.86	0.01990	0.01995	0.01989	0.01999	0.01989	0.01992	0.00790	988E-06
7/19/96	11:00 AM	35	85.87	0.01948	0.01942	0.01945	0.01948	0.01945	0.01946	0.00837	1.05E-03
8/5/96	9:50 AM	35	102.82	0.01921	0.01925	0.01922	0.01920	0.01920	0.01922	0.00861	1.08E-03
9/10/96	10:10 AM	35	138.83	0.01848	0.01841	0.01842	0.01841	0.01839	0.01842	0.00941	1.18E-03
9/27/96	1:10 PM	35	155.96	0.01810	0.01813	0.01815	0.01809	0.01811	0.01812	0.00971	1.21E-03

REFERENCE SPECIMEN #5 - LOADED - B

Date	Time	Load (kips)	Time (days)	B 1	B 2	B 3	B 4	B 5	Avg.	Diff.	Strain Ref #5-B
4/24/96	1:50 PM	0	0.00	0.02559	0.02561	0.02558	0.02555	0.02558	0.02558	*****	*****
4/24/96	2:10 PM	35	0.01	0.02351	0.02352	0.02355	0.02359	0.02359	0.02355	0.00203	254E-06
4/24/96	3:10 PM	35	0.04	0.02329	0.02328	0.02325	0.02319	0.02321	0.02324	0.00234	292E-06
4/24/96	4:10 PM	35	0.08	0.02318	0.02319	0.02312	0.02315	0.02312	0.02315	0.00243	304E-06
4/24/96	6:15 PM	35	0.17	0.02315	0.02321	0.02320	0.02320	0.02312	0.02318	0.00241	301E-06
4/24/96	10:10 PM	35	0.33	0.02309	0.02311	0.02308	0.02310	0.02309	0.02309	0.00249	311E-06
4/25/96	1:45 AM	35	0.48	0.02291	0.02291	0.02295	0.02295	0.02290	0.02292	0.00266	332E-06
4/25/96	2:10 PM	35	1.00	0.02252	0.02251	0.02255	0.02250	0.02251	0.02252	0.00306	383E-06
4/26/96	1:40 AM	35	1.48	0.02225	0.02221	0.02221	0.02228	0.02225	0.02224	0.00334	418E-06
4/26/96	2:10 PM	35	2.00	0.02215	0.02219	0.02215	0.02215	0.02219	0.02217	0.00342	427E-06
4/27/96	12:00 AM	35	2.41	0.02215	0.02215	0.02211	0.02218	0.02220	0.02216	0.00342	428E-06
4/28/96	2:10 PM	35	4.00	0.02208	0.02200	0.02201	0.02205	0.02201	0.02203	0.00355	444E-06
5/1/96	2:10 PM	35	7.00	0.02198	0.02185	0.02191	0.02195	0.02191	0.02192	0.00366	458E-06
5/4/96	12:10 AM	35	9.42	0.02165	0.02169	0.02165	0.02158	0.02169	0.02165	0.00393	491E-06
5/8/96	12:10 PM	35	13.92	0.02135	0.02145	0.02138	0.02135	0.02140	0.02139	0.00420	525E-06
5/15/96	12:40 PM	35	20.94	0.02075	0.02069	0.02071	0.02069	0.02068	0.02070	0.00488	610E-06
5/22/96	3:00 PM	35	28.03	0.02025	0.02015	0.02019	0.02018	0.02015	0.02018	0.00540	675E-06
5/29/96	1:45 PM	35	34.98	0.01979	0.01975	0.01975	0.01978	0.01980	0.01977	0.00581	726E-06
6/7/96	3:10 PM	35	44.04	0.01945	0.01945	0.01951	0.01942	0.01942	0.01945	0.00613	767E-06
6/21/96	12:30 AM	35	57.43	0.01895	0.01889	0.01898	0.01899	0.01892	0.01895	0.00664	830E-06
7/5/96	10:45 AM	35	71.86	0.01831	0.01839	0.01835	0.01829	0.01829	0.01833	0.00726	907E-06
7/19/96	11:00 AM	35	85.87	0.01775	0.01781	0.01772	0.01772	0.01781	0.01776	0.00782	978E-06
8/5/96	9:50 AM	35	102.82	0.01735	0.01730	0.01731	0.01730	0.01730	0.01731	0.00827	1.03E-03
9/10/96	10:10 AM	35	138.83	0.01650	0.01651	0.01655	0.01658	0.01658	0.01654	0.00904	1.13E-03
9/27/96	1:10 PM	35	155.96	0.01632	0.01628	0.01625	0.01630	0.01629	0.01629	0.00929	1.16E-03

REF SPECIMEN #5 - LOADED				Avg	Specific	Corrected
Date	Time	Load (kips)	Time (days)	Strain Ref #5	Strain Ref #5	Spec Strain Ref #5
4/24/96	1:50 PM	0	0.00			
4/24/96	2:10 PM	35	0.01	269.1E-06	.217E-06	.000E+00
4/24/96	3:10 PM	35	0.04	288.0E-06	.233E-06	.152E-07
4/24/96	4:10 PM	35	0.08	303.3E-06	.245E-06	.276E-07
4/24/96	6:15 PM	35	0.17	299.6E-06	.242E-06	.246E-07
4/24/96	10:10 PM	35	0.33	322.8E-06	.261E-06	.433E-07
4/25/96	1:45 AM	35	0.48	344.5E-06	.278E-06	.609E-07
4/25/96	2:10 PM	35	1.00	382.5E-06	.309E-06	.916E-07
4/26/96	1:40 AM	35	1.48	424.4E-06	.343E-06	.125E-06
4/26/96	2:10 PM	35	2.00	443.4E-06	.358E-06	.141E-06
4/27/96	12:00 AM	35	2.41	446.0E-06	.360E-06	.143E-06
4/28/96	2:10 PM	35	4.00	465.3E-06	.376E-06	.158E-06
5/1/96	2:10 PM	35	7.00	499.9E-06	.404E-06	.186E-06
5/4/96	12:10 AM	35	9.42	541.9E-06	.438E-06	.220E-06
5/8/96	12:10 PM	35	13.92	581.3E-06	.469E-06	.252E-06
5/15/96	12:40 PM	35	20.94	672.8E-06	.543E-06	.326E-06
5/22/96	3:00 PM	35	28.03	731.1E-06	.591E-06	.373E-06
5/29/96	1:45 PM	35	34.98	782.8E-06	.632E-06	.415E-06
6/7/96	3:10 PM	35	44.04	814.9E-06	.658E-06	.441E-06
6/21/96	12:30 AM	35	57.43	872.6E-06	.705E-06	.487E-06
7/5/96	10:45 AM	35	71.86	947.5E-06	.765E-06	.548E-06
7/19/96	11:00 AM	35	85.87	1.01E-03	.817E-06	.600E-06
8/5/96	9:50 AM	35	102.82	1.06E-03	.852E-06	.635E-06
9/10/96	10:10 AM	35	138.83	1.15E-03	9.31E-07	7.14E-07
9/27/96	1:10 PM	35	155.96	1.19E-03	9.59E-07	7.42E-07

REFERENCE SPECIMEN #7 - LOADED - A

Date	Time	Load (kips)	Time (Days)	A					Avg.	Diff.	Strain Ref #7-A
				1	2	3	4	5			
4/24/96	1:50 PM	0	0.00	0.02908	0.02905	0.02907	0.02909	0.02911	0.02908	*****	*****
4/24/96	2:10 PM	35	0.01	0.02690	0.02685	0.02685	0.02680	0.02682	0.02684	0.002236	280E-06
4/24/96	3:10 PM	35	0.04	0.02650	0.02652	0.02652	0.02651	0.02655	0.02652	0.00256	320E-06
4/24/96	4:10 PM	35	0.08	0.02648	0.02645	0.02642	0.02648	0.02645	0.02646	0.002624	328E-06
4/24/96	6:15 PM	35	0.17	0.02659	0.02655	0.02658	0.02658	0.02652	0.02656	0.002516	315E-06
4/24/96	10:10 PM	35	0.33	0.02632	0.02631	0.02631	0.02630	0.02635	0.02632	0.002762	345E-06
4/25/96	1:45 AM	35	0.48	0.02622	0.02621	0.02625	0.02621	0.02620	0.02622	0.002862	358E-06
4/25/96	2:10 PM	35	1.00	0.02605	0.02601	0.02605	0.02605	0.02609	0.02605	0.00303	379E-06
4/26/96	1:40 AM	35	1.48	0.02558	0.02561	0.02565	0.02564	0.02560	0.02562	0.003464	433E-06
4/26/96	2:10 PM	35	2.00	0.02549	0.02550	0.02549	0.02551	0.02555	0.02551	0.003572	447E-06
4/27/96	12:00 AM	35	2.41	0.02545	0.02549	0.02549	0.02542	0.02541	0.02545	0.003628	454E-06
4/28/96	2:10 PM	35	4.00	0.02531	0.02525	0.02535	0.02529	0.02530	0.02530	0.00378	473E-06
5/1/96	2:10 PM	35	7.00	0.02495	0.02500	0.02505	0.02495	0.02498	0.02499	0.004094	512E-06
5/4/96	12:10 AM	35	9.42	0.02465	0.02460	0.02465	0.02460	0.02465	0.02463	0.00445	556E-06
5/8/96	12:10 PM	35	13.92	0.02440	0.02439	0.02435	0.02439	0.02439	0.02438	0.004696	587E-06
5/15/96	12:40 PM	35	20.94	0.02345	0.02340	0.02341	0.02335	0.02335	0.02339	0.005688	711E-06
5/22/96	3:00 PM	35	28.03	0.02280	0.02279	0.02275	0.02279	0.02275	0.02278	0.006304	788E-06
5/29/96	1:45 PM	35	34.98	0.02232	0.02239	0.02240	0.02233	0.02238	0.02236	0.006716	840E-06
6/7/96	3:10 PM	35	44.04	0.02225	0.02221	0.02225	0.02220	0.02221	0.02222	0.006856	857E-06
6/21/96	12:30 AM	35	57.43	0.02172	0.02175	0.02171	0.02172	0.02175	0.02173	0.00735	919E-06
7/5/96	10:45 AM	35	71.86	0.02135	0.02129	0.02130	0.02129	0.02131	0.02131	0.007772	972E-06
7/19/96	11:00 AM	35	85.87	0.02085	0.02081	0.02082	0.02085	0.02081	0.02083	0.008252	1.03E-03
8/5/96	9:50 AM	35	102.82	0.02059	0.02060	0.02060	0.02059	0.02060	0.02060	0.008484	1.06E-03
9/10/96	10:10 AM	35	138.83	0.02001	0.01998	0.01995	0.01996	0.01989	0.01996	0.009122	1.14E-03
9/27/96	1:10 PM	35	155.96	0.01968	0.01969	0.01970	0.01958	0.01961	0.01965	0.009428	1.18E-03

REFERENCE SPECIMEN #7 - LOADED - B

Date	Time	Load (kips)	Time (days)	B 1	B 2	B 3	B 4	B 5	Avg.	Diff.	Strain Ref #7-B
4/24/96	1:50 PM	0	0.00	0.02769	0.02769	0.02765	0.02771	0.02770	0.02769	*****	*****
4/24/96	2:10 PM	35	0.01	0.02499	0.02500	0.02495	0.02501	0.02495	0.02498	0.00271	338E-06
4/24/96	3:10 PM	35	0.04	0.02481	0.02475	0.02485	0.02480	0.02480	0.02480	0.00289	361E-06
4/24/96	4:10 PM	35	0.08	0.02479	0.02475	0.02471	0.02469	0.02478	0.02474	0.00294	368E-06
4/24/96	6:15 PM	35	0.17	0.02478	0.02480	0.02481	0.02482	0.02481	0.02480	0.00288	361E-06
4/24/96	10:10 PM	35	0.33	0.02465	0.02455	0.02459	0.02454	0.02455	0.02458	0.00311	389E-06
4/25/96	1:45 AM	35	0.48	0.02458	0.02458	0.02455	0.02454	0.02455	0.02456	0.00313	391E-06
4/25/96	2:10 PM	35	1.00	0.02425	0.02421	0.02419	0.02425	0.02425	0.02423	0.00346	432E-06
4/26/96	1:40 AM	35	1.48	0.02389	0.02385	0.02385	0.02385	0.02389	0.02387	0.00382	478E-06
4/26/96	2:10 PM	35	2.00	0.02371	0.02375	0.02378	0.02375	0.02378	0.02375	0.00393	492E-06
4/27/96	12:00 AM	35	2.41	0.02368	0.02365	0.02368	0.02369	0.02371	0.02368	0.00401	501E-06
4/28/96	2:10 PM	35	4.00	0.02351	0.02349	0.02350	0.02351	0.02351	0.02350	0.00418	523E-06
5/1/96	2:10 PM	35	7.00	0.02315	0.02320	0.02315	0.02320	0.02329	0.02320	0.00449	561E-06
5/4/96	12:10 AM	35	9.42	0.02285	0.02285	0.02280	0.02285	0.02289	0.02285	0.00484	605E-06
5/8/96	12:10 PM	35	13.92	0.02269	0.02270	0.02270	0.02271	0.02269	0.02270	0.00499	624E-06
5/15/96	12:40 PM	35	20.94	0.02219	0.02215	0.02211	0.02215	0.02216	0.02215	0.00554	692E-06
5/22/96	3:00 PM	35	28.03	0.02181	0.02185	0.02181	0.02185	0.02182	0.02183	0.00586	732E-06
5/29/96	1:45 PM	35	34.98	0.02152	0.02149	0.02145	0.02151	0.02145	0.02148	0.00620	776E-06
6/7/96	3:10 PM	35	44.04	0.02125	0.02128	0.02121	0.02125	0.02122	0.02124	0.00645	806E-06
6/21/96	12:30 AM	35	57.43	0.02078	0.02075	0.02081	0.02071	0.02075	0.02076	0.00693	866E-06
7/5/96	10:45 AM	35	71.86	0.02010	0.02011	0.02015	0.02000	0.01995	0.02006	0.00763	953E-06
7/19/96	11:00 AM	35	85.87	0.01941	0.01939	0.01939	0.01941	0.01939	0.01940	0.00829	1.04E-03
8/5/96	9:50 AM	35	102.82	0.01879	0.01878	0.01875	0.01871	0.01870	0.01875	0.00894	1.12E-03
9/10/96	10:10 AM	35	138.83	0.01795	0.01790	0.01791	0.01791	0.01801	0.01794	0.00975	1.22E-03
9/27/96	1:10 PM	35	155.96	0.01765	0.01762	0.01762	0.01761	0.01762	0.01762	0.01006	1.26E-03

REF SPECIMEN #7 - LOADED				Avg	Specific	Corrected
Date	Time	Load (kips)	Time (days)	Strain Ref #7	Strain Ref #7	Spec Strain Ref #7
4/24/96	1:50 PM	0	0.00			
4/24/96	2:10 PM	35	0.01	309E-06	.250E-06	.000E+00
4/24/96	3:10 PM	35	0.04	340E-06	.275E-06	.253E-07
4/24/96	4:10 PM	35	0.08	348E-06	.281E-06	.315E-07
4/24/96	6:15 PM	35	0.17	338E-06	.273E-06	.230E-07
4/24/96	10:10 PM	35	0.33	367E-06	.297E-06	.469E-07
4/25/96	1:45 AM	35	0.48	374E-06	.302E-06	.528E-07
4/25/96	2:10 PM	35	1.00	406E-06	.328E-06	.779E-07
4/26/96	1:40 AM	35	1.48	455E-06	.368E-06	.118E-06
4/26/96	2:10 PM	35	2.00	469E-06	.379E-06	.129E-06
4/27/96	12:00 AM	35	2.41	477E-06	.385E-06	.136E-06
4/28/96	2:10 PM	35	4.00	498E-06	.402E-06	.152E-06
5/1/96	2:10 PM	35	7.00	537E-06	.433E-06	.184E-06
5/4/96	12:10 AM	35	9.42	581E-06	.469E-06	.219E-06
5/8/96	12:10 PM	35	13.92	605E-06	.489E-06	.239E-06
5/15/96	12:40 PM	35	20.94	702E-06	.567E-06	.317E-06
5/22/96	3:00 PM	35	28.03	760E-06	.614E-06	.364E-06
5/29/96	1:45 PM	35	34.98	808E-06	.652E-06	.403E-06
6/7/96	3:10 PM	35	44.04	831E-06	.672E-06	.422E-06
6/21/96	12:30 AM	35	57.43	892E-06	.721E-06	.471E-06
7/5/96	10:45 AM	35	71.86	962E-06	.777E-06	.528E-06
7/19/96	11:00 AM	35	85.87	1.03E-03	.835E-06	.585E-06
8/5/96	9:50 AM	35	102.82	1.09E-03	.880E-06	.630E-06
9/10/96	10:10 AM	35	138.83	1.18E-03	9.53E-07	7.03E-07
9/27/96	1:10 PM	35	155.96	1.22E-03	9.84E-07	7.34E-07

REFERENCE SPECIMEN #2 - UNLOADED - A											
Date	Time	Load (kips)	Time (days)	A					Avg.	Diff.	Strain #2-UL-A
				1	2	3	4	5			
4/24/96	10:15 AM	0	0.00	0.03018	0.03020	0.03018	0.03019	0.03021	0.03019	*****	*****
4/24/96	2:30 PM	0	0.01	0.03009	0.03009	0.03015	0.03009	0.03010	0.03010	0.00009	011E-06
4/24/96	3:35 PM	0	0.05	0.03015	0.03019	0.03121	0.03021	0.03021	0.03039	-0.00020	-025E-06
4/24/96	4:40 PM	0	0.09	0.03019	0.03020	0.03025	0.03021	0.03021	0.03021	-0.00002	-003E-06
4/24/96	6:45 PM	0	0.18	0.03028	0.03020	0.03025	0.03034	0.03032	0.03028	-0.00009	-011E-06
4/24/96	10:40 PM	0	0.34	0.03015	0.03019	0.03019	0.03020	0.03019	0.03018	0.00001	001E-06
4/25/96	2:15 AM	0	0.49	0.03031	0.03019	0.03031	0.03019	0.03025	0.03025	-0.00006	-007E-06
4/25/96	2:30 PM	0	1.00	0.03035	0.03032	0.03035	0.03038	0.03038	0.03036	-0.00016	-021E-06
4/26/96	2:00 AM	0	1.48	0.03015	0.03021	0.03025	0.03019	0.03021	0.03020	-0.00001	-001E-06
4/26/96	2:40 PM	0	2.01	0.03019	0.03020	0.03021	0.03015	0.03018	0.03019	0.00001	750E-09
4/27/96	12:20 AM	0	2.41	0.03019	0.03021	0.03015	0.03009	0.03008	0.03014	0.00005	006E-06
4/28/96	2:30 PM	0	4.00	0.03025	0.03021	0.03025	0.03019	0.03015	0.03021	-0.00002	-002E-06
5/1/96	2:40 PM	0	7.01	0.03015	0.03019	0.03019	0.03011	0.03008	0.03014	0.00005	006E-06
5/4/96	12:35 AM	0	9.42	0.03008	0.03010	0.03010	0.03010	0.03011	0.03010	0.00009	012E-06
5/8/96	12:35 PM	0	13.92	0.02998	0.02995	0.03005	0.03005	0.03002	0.03001	0.00018	023E-06
5/15/96	1:05 PM	0	20.94	0.02985	0.02989	0.02985	0.02979	0.02989	0.02985	0.00034	042E-06
5/22/96	3:40 PM	0	28.05	0.02969	0.02969	0.02971	0.02971	0.02970	0.02970	0.00049	061E-06
5/29/96	2:10 PM	0	34.99	0.02959	0.02961	0.02961	0.02955	0.02958	0.02959	0.00060	076E-06
6/7/96	3:45 PM	0	44.05	0.02955	0.02959	0.02951	0.02955	0.02949	0.02954	0.00065	082E-06
6/21/96	1:00 AM	0	57.44	0.02940	0.02945	0.02942	0.02945	0.02947	0.02944	0.00075	094E-06
7/5/96	11:15 AM	0	71.86	0.02919	0.02915	0.02925	0.02921	0.02929	0.02922	0.00097	122E-06
7/19/96	11:20 AM	0	85.87	0.02885	0.02872	0.02878	0.02882	0.02882	0.02880	0.00139	174E-06
8/5/96	10:29 AM	0	102.83	0.02865	0.02861	0.02865	0.02865	0.02861	0.02863	0.00156	195E-06
9/10/96	10:35 AM	0	138.84	0.02831	0.02838	0.02832	0.02829	0.02832	0.02832	0.00187	234E-06
9/27/96	1:40 AM	0	155.47	0.02825	0.02819	0.02815	0.02821	0.02821	0.02820	0.00199	249E-06

REFERENCE SPECIMEN #2 - UNLOADED - B											
Date	Time	Load (kips)	Time (days)	B					Avg.	Diff.	Strain #2-UL-B
				1	2	3	4	5			
4/24/96	10:15 AM	0	0.00	0.03155	0.03145	0.03150	0.03145	0.03148	0.03149	*****	*****
4/24/96	2:30 PM	0	0.01	0.03141	0.03139	0.03145	0.03142	0.03140	0.03141	0.00007	009E-06
4/24/96	3:35 PM	0	0.05	0.03150	0.03155	0.03152	0.03158	0.03155	0.03154	-0.00005	-007E-06
4/24/96	4:40 PM	0	0.09	0.03161	0.03165	0.03159	0.03161	0.03165	0.03162	-0.00014	-017E-06
4/24/96	6:45 PM	0	0.18	0.03175	0.03178	0.03172	0.03172	0.03178	0.03175	-0.00026	-033E-06
4/24/96	10:40 PM	0	0.34	0.03175	0.03178	0.03175	0.03176	0.03175	0.03176	-0.00027	-034E-06
4/25/96	2:15 AM	0	0.49	0.03170	0.03168	0.03162	0.03165	0.03169	0.03167	-0.00018	-023E-06
4/25/96	2:30 PM	0	1.00	0.03158	0.03165	0.03161	0.03165	0.03160	0.03162	-0.00013	-017E-06
4/26/96	2:00 AM	0	1.48	0.03155	0.03155	0.03151	0.03158	0.03151	0.03154	-0.00005	-007E-06
4/26/96	2:40 PM	0	2.01	0.03151	0.03159	0.03149	0.03149	0.03150	0.03152	-0.00003	-004E-06
4/27/96	12:20 AM	0	2.41	0.03159	0.03150	0.03149	0.03155	0.03149	0.03152	-0.00004	-005E-06
4/28/96	2:30 PM	0	4.00	0.03159	0.03160	0.03165	0.03159	0.03161	0.03161	-0.00012	-015E-06
5/1/96	2:40 PM	0	7.01	0.03151	0.03145	0.03159	0.03152	0.03151	0.03152	-0.00003	-004E-06
5/4/96	12:35 AM	0	9.42	0.03141	0.03145	0.03149	0.03139	0.03141	0.03143	0.00006	007E-06
5/8/96	12:35 PM	0	13.92	0.03129	0.03129	0.03130	0.03139	0.03135	0.03132	0.00016	020E-06
5/15/96	1:05 PM	0	20.94	0.03119	0.03115	0.03111	0.03115	0.03115	0.03115	0.00034	042E-06
5/22/96	3:40 PM	0	28.05	0.03100	0.03105	0.03105	0.03101	0.03105	0.03103	0.00045	057E-06
5/29/96	2:10 PM	0	34.99	0.03095	0.03098	0.03095	0.03092	0.03095	0.03095	0.00054	067E-06
6/7/96	3:45 PM	0	44.05	0.03089	0.03091	0.03085	0.03085	0.03091	0.03088	0.00060	076E-06
6/21/96	1:00 AM	0	57.44	0.03065	0.03068	0.03072	0.03065	0.03071	0.03068	0.00080	101E-06
7/5/96	11:15 AM	0	71.86	0.03045	0.03039	0.03040	0.03045	0.03045	0.03043	0.00106	132E-06
7/19/96	11:20 AM	0	85.87	0.03024	0.03021	0.03020	0.03015	0.03018	0.03020	0.00129	161E-06
8/5/96	10:29 AM	0	102.83	0.03001	0.02999	0.03000	0.02995	0.03002	0.02999	0.00149	187E-06
9/10/96	10:35 AM	0	138.84	0.02968	0.02971	0.02965	0.02965	0.02969	0.02968	0.00181	226E-06
9/27/96	1:40 AM	0	155.47	0.02945	0.02938	0.02935	0.02939	0.02941	0.02940	0.00209	261E-06

REF SPECIMEN #2-UNLOADED				Avg Strain #2-UL
Date	Time	Load (kips)	Time (days)	
4/24/96	10:15 AM	0	0.00	
4/24/96	2:30 PM	0	0.01	010E-06
4/24/96	3:35 PM	0	0.05	-016E-06
4/24/96	4:40 PM	0	0.09	-010E-06
4/24/96	6:45 PM	0	0.18	-022E-06
4/24/96	10:40 PM	0	0.34	-016E-06
4/25/96	2:15 AM	0	0.49	-015E-06
4/25/96	2:30 PM	0	1.00	-019E-06
4/26/96	2:00 AM	0	1.48	-004E-06
4/26/96	2:40 PM	0	2.01	-002E-06
4/27/96	12:20 AM	0	2.41	625E-09
4/28/96	2:30 PM	0	4.00	-009E-06
5/1/96	2:40 PM	0	7.01	001E-06
5/4/96	12:35 AM	0	9.42	009E-06
5/8/96	12:35 PM	0	13.92	022E-06
5/15/96	1:05 PM	0	20.94	042E-06
5/22/96	3:40 PM	0	28.05	059E-06
5/29/96	2:10 PM	0	34.99	071E-06
6/7/96	3:45 PM	0	44.05	079E-06
6/21/96	1:00 AM	0	57.44	097E-06
7/5/96	11:15 AM	0	71.86	127E-06
7/19/96	11:20 AM	0	85.87	168E-06
8/5/96	10:29 AM	0	102.83	191E-06
9/10/96	10:35 AM	0	138.84	230E-06
9/27/96	1:40 AM	0	155.47	255E-06

REFERENCE SPECIMEN #8 - UNLOADED - A

Date	Time	Load (kips)	Time (days)	A 1	A 2	A 3	A 4	A 5	Avg.	Diff.	Strain #8-UL-A
4/24/96	10:15 AM	0	0.00	0.02940	0.02948	0.02942	0.02949	0.02942	0.02944	*****	*****
4/24/96	2:30 PM	0	0.01	0.02939	0.02941	0.02939	0.02945	0.02941	0.02941	0.00003	004E-06
4/24/96	3:35 PM	0	0.05	0.02961	0.02965	0.02965	0.02965	0.02959	0.02963	-0.00019	-023E-06
4/24/96	4:40 PM	0	0.09	0.02960	0.02961	0.02961	0.02965	0.02965	0.02962	-0.00018	-023E-06
4/24/96	6:45 PM	0	0.18	0.02965	0.02971	0.02970	0.02971	0.02975	0.02970	-0.00026	-033E-06
4/24/96	10:40 PM	0	0.34	0.02955	0.02949	0.02951	0.02958	0.02958	0.02954	-0.00010	-012E-06
4/25/96	2:15 AM	0	0.49	0.02950	0.02955	0.02950	0.02958	0.02949	0.02952	-0.00008	-010E-06
4/25/96	2:30 PM	0	1.00	0.02955	0.02950	0.02955	0.02958	0.02958	0.02955	-0.00011	-014E-06
4/26/96	2:00 AM	0	1.48	0.02945	0.02945	0.02941	0.02951	0.02950	0.02946	-0.00002	-003E-06
4/26/96	2:40 PM	0	2.01	0.02948	0.02950	0.02949	0.02950	0.02945	0.02948	-0.00004	-005E-06
4/27/96	12:20 AM	0	2.41	0.02945	0.02939	0.02945	0.02939	0.02940	0.02942	0.00003	003E-06
4/28/96	2:30 PM	0	4.00	0.02942	0.02945	0.02950	0.02951	0.02942	0.02946	-0.00002	-002E-06
5/1/96	2:40 PM	0	7.01	0.02948	0.02942	0.02951	0.02942	0.02945	0.02946	-0.00001	-002E-06
5/4/96	12:35 AM	0	9.42	0.02945	0.02939	0.02941	0.02941	0.02939	0.02941	0.00003	004E-06
5/8/96	12:35 PM	0	13.92	0.02935	0.02938	0.02931	0.02941	0.02939	0.02937	0.00007	009E-06
5/15/96	1:05 PM	0	20.94	0.02930	0.02935	0.02925	0.02919	0.02920	0.02926	0.00018	023E-06
5/22/96	3:40 PM	0	28.05	0.02905	0.02909	0.02910	0.02905	0.02909	0.02908	0.00037	046E-06
5/29/96	2:10 PM	0	34.99	0.02885	0.02885	0.02884	0.02891	0.02890	0.02887	0.00057	072E-06
6/7/96	3:45 PM	0	44.05	0.02878	0.02872	0.02875	0.02872	0.02875	0.02874	0.00070	087E-06
6/21/96	1:00 AM	0	57.44	0.02850	0.02845	0.02848	0.02851	0.02850	0.02849	0.00095	119E-06
7/5/96	11:15 AM	0	71.86	0.02825	0.02821	0.02819	0.02821	0.02818	0.02821	0.00123	154E-06
7/19/96	11:20 AM	0	85.87	0.02785	0.02775	0.02778	0.02778	0.02779	0.02779	0.00165	207E-06
8/5/96	10:29 AM	0	102.83	0.02765	0.02762	0.02765	0.02759	0.02759	0.02762	0.00182	228E-06
9/10/96	10:35 AM	0	138.84	0.02730	0.02725	0.02731	0.02730	0.02726	0.02728	0.00216	270E-06
9/27/96	1:40 AM	0	155.47	0.02710	0.02709	0.02710	0.02711	0.02709	0.02710	0.00234	293E-06

REFERENCE SPECIMEN #8 - UNLOADED - B											
Date	Time	Load (kips)	Time (days)	B					Avg.		Strain
				1	2	3	4	5		Diff.	#8-UL-B
4/24/96	10:15 AM	0	0.00	0.02280	0.02275	0.02275	0.02270	0.02272	0.02274	*****	*****
4/24/96	2:30 PM	0	0.01	0.02255	0.02259	0.02252	0.02253	0.02255	0.02255	0.00020	025E-06
4/24/96	3:35 PM	0	0.05	0.02265	0.02275	0.02269	0.02271	0.02269	0.02270	0.00005	006E-06
4/24/96	4:40 PM	0	0.09	0.02275	0.02275	0.02268	0.02269	0.02269	0.02271	0.00003	004E-06
4/24/96	6:45 PM	0	0.18	0.02295	0.02299	0.02295	0.02299	0.02289	0.02295	-0.00021	-026E-06
4/24/96	10:40 PM	0	0.34	0.02295	0.02295	0.02301	0.02295	0.02298	0.02297	-0.00022	-028E-06
4/25/96	2:15 AM	0	0.49	0.02290	0.02289	0.02289	0.02295	0.02290	0.02291	-0.00016	-020E-06
4/25/96	2:30 PM	0	1.00	0.02280	0.02275	0.02272	0.02275	0.02272	0.02275	0.00000	-500E-09
4/26/96	2:00 AM	0	1.48	0.02275	0.02281	0.02280	0.02281	0.02281	0.02280	-0.00005	-007E-06
4/26/96	2:40 PM	0	2.01	0.02275	0.02271	0.02279	0.02279	0.02269	0.02275	0.00000	-250E-09
4/27/96	12:20 AM	0	2.41	0.02279	0.02280	0.02279	0.02279	0.02279	0.02279	-0.00005	-006E-06
4/28/96	2:30 PM	0	4.00	0.02281	0.02278	0.02285	0.02289	0.02284	0.02283	-0.00009	-011E-06
5/1/96	2:40 PM	0	7.01	0.02275	0.02281	0.02272	0.02269	0.02278	0.02275	-0.00001	-750E-09
5/4/96	12:35 AM	0	9.42	0.02269	0.02265	0.02262	0.02259	0.02260	0.02263	0.00011	014E-06
5/8/96	12:35 PM	0	13.92	0.02259	0.02255	0.02251	0.02250	0.02251	0.02253	0.00021	026E-06
5/15/96	1:05 PM	0	20.94	0.02245	0.02250	0.02245	0.02245	0.02239	0.02245	0.00030	037E-06
5/22/96	3:40 PM	0	28.05	0.02220	0.02225	0.02221	0.02221	0.02225	0.02222	0.00052	065E-06
5/29/96	2:10 PM	0	34.99	0.02205	0.02211	0.02210	0.02205	0.02204	0.02207	0.00067	084E-06
6/7/96	3:45 PM	0	44.05	0.02195	0.02195	0.02200	0.02199	0.02195	0.02197	0.00078	097E-06
6/21/96	1:00 AM	0	57.44	0.02169	0.02175	0.02171	0.02175	0.02179	0.02174	0.00101	126E-06
7/5/96	11:15 AM	0	71.86	0.02145	0.02141	0.02155	0.02145	0.02151	0.02147	0.00127	159E-06
7/19/96	11:20 AM	0	85.87	0.02128	0.02125	0.02122	0.02122	0.02125	0.02124	0.00150	188E-06
8/5/96	10:29 AM	0	102.83	0.02101	0.02105	0.02100	0.02095	0.02102	0.02101	0.00174	217E-06
9/10/96	10:35 AM	0	138.84	0.02085	0.02081	0.02084	0.02091	0.02088	0.02086	0.00189	236E-06
9/27/96	1:40 AM	0	155.47	0.02068	0.02075	0.02065	0.02068	0.02062	0.02068	0.00207	259E-06

REF SPECIMEN #8-UNLOADED				Avg
Date	Time	Load (kips)	Time (days)	Strain #8-UL
4/24/96	10:15 AM	0	0.00	
4/24/96	2:30 PM	0	0.01	014E-06
4/24/96	3:35 PM	0	0.05	-009E-06
4/24/96	4:40 PM	0	0.09	-009E-06
4/24/96	6:45 PM	0	0.18	-029E-06
4/24/96	10:40 PM	0	0.34	-020E-06
4/25/96	2:15 AM	0	0.49	-015E-06
4/25/96	2:30 PM	0	1.00	-007E-06
4/26/96	2:00 AM	0	1.48	-005E-06
4/26/96	2:40 PM	0	2.01	-003E-06
4/27/96	12:20 AM	0	2.41	-001E-06
4/28/96	2:30 PM	0	4.00	-007E-06
5/1/96	2:40 PM	0	7.01	-001E-06
5/4/96	12:35 AM	0	9.42	009E-06
5/8/96	12:35 PM	0	13.92	018E-06
5/15/96	1:05 PM	0	20.94	030E-06
5/22/96	3:40 PM	0	28.05	055E-06
5/29/96	2:10 PM	0	34.99	078E-06
6/7/96	3:45 PM	0	44.05	092E-06
6/21/96	1:00 AM	0	57.44	123E-06
7/5/96	11:15 AM	0	71.86	157E-06
7/19/96	11:20 AM	0	85.87	197E-06
8/5/96	10:29 AM	0	102.83	223E-06
9/10/96	10:35 AM	0	138.84	253E-06
9/27/96	1:40 AM	0	155.47	276E-06

REFERENCE SPECIMEN #9 - UNLOADED - A											
Date	Time	Load (kips)	Time (days)	A							Strain
				1	2	3	4	5	Avg.	Diff.	#9-UL-A
4/24/96	10:15 AM	0	0.00	0.02420	0.02421	0.02418	0.02414	0.02415	0.02418	*****	*****
4/24/96	2:30 PM	0	0.01	0.02415	0.02409	0.02409	0.02411	0.02411	0.02411	0.00007	008E-06
4/24/96	3:35 PM	0	0.05	0.02419	0.02420	0.02411	0.02421	0.02420	0.02418	-0.00001	-750E-09
4/24/96	4:40 PM	0	0.09	0.02449	0.02450	0.02450	0.02449	0.02448	0.02449	-0.00032	-040E-06
4/24/96	6:45 PM	0	0.18	0.02470	0.02465	0.02458	0.02471	0.02465	0.02466	-0.00048	-060E-06
4/24/96	10:40 PM	0	0.34	0.02449	0.02451	0.02448	0.02448	0.02451	0.02449	-0.00032	-040E-06
4/25/96	2:15 AM	0	0.49	0.02445	0.02445	0.02448	0.02452	0.02448	0.02448	-0.00030	-038E-06
4/25/96	2:30 PM	0	1.00	0.02441	0.02445	0.02440	0.02448	0.02448	0.02444	-0.00027	-034E-06
4/26/96	2:00 AM	0	1.48	0.02445	0.02445	0.02448	0.02448	0.02445	0.02446	-0.00029	-036E-06
4/26/96	2:40 PM	0	2.01	0.02449	0.02450	0.02445	0.02450	0.02449	0.02449	-0.00031	-039E-06
4/27/96	12:20 AM	0	2.41	0.02439	0.02445	0.02442	0.02448	0.02441	0.02443	-0.00025	-032E-06
4/28/96	2:30 PM	0	4.00	0.02435	0.02439	0.02445	0.02435	0.02441	0.02439	-0.00021	-027E-06
5/1/96	2:40 PM	0	7.01	0.02445	0.02441	0.02445	0.02435	0.02445	0.02442	-0.00025	-031E-06
5/4/96	12:35 AM	0	9.42	0.02419	0.02425	0.02420	0.02425	0.02419	0.02422	-0.00004	-005E-06
5/8/96	12:35 PM	0	13.92	0.02411	0.02405	0.02400	0.02405	0.02401	0.02404	0.00013	016E-06
5/15/96	1:05 PM	0	20.94	0.02400	0.02395	0.02395	0.02389	0.02389	0.02394	0.00024	030E-06
5/22/96	3:40 PM	0	28.05	0.02390	0.02375	0.02371	0.02379	0.02370	0.02377	0.00041	051E-06
5/29/96	2:10 PM	0	34.99	0.02365	0.02370	0.02369	0.02375	0.02368	0.02369	0.00048	060E-06
6/7/96	3:45 PM	0	44.05	0.02361	0.02355	0.02359	0.02359	0.02361	0.02359	0.00059	073E-06
6/21/96	1:00 AM	0	57.44	0.02345	0.02349	0.02350	0.02349	0.02342	0.02347	0.00071	088E-06
7/5/96	11:15 AM	0	71.86	0.02321	0.02325	0.02328	0.02325	0.02321	0.02324	0.00094	117E-06
7/19/96	11:20 AM	0	85.87	0.02295	0.02292	0.02292	0.02291	0.02292	0.02292	0.00125	157E-06
8/5/96	10:29 AM	0	102.83	0.02241	0.02235	0.02239	0.02241	0.02240	0.02239	0.00178	223E-06
9/10/96	10:35 AM	0	138.84	0.02221	0.02219	0.02215	0.02218	0.02215	0.02218	0.00200	250E-06
9/27/96	1:40 AM	0	155.47	0.02205	0.02209	0.02205	0.02209	0.02201	0.02206	0.00212	265E-06

REFERENCE SPECIMEN #9 - UNLOADED - B

Date	Time	Load (kips)	Time (days)	B 1	B 2	B 3	B 4	B 5	Avg.	Diff.	Strain #9-UL-B
4/24/96	10:15 AM	0	0.00	0.02860	0.02858	0.02860	0.02862	0.02858	0.02860	*****	*****
4/24/96	2:30 PM	0	0.01	0.02849	0.02840	0.02843	0.02841	0.02845	0.02844	0.00016	020E-06
4/24/96	3:35 PM	0	0.05	0.02865	0.02859	0.02860	0.02861	0.02865	0.02862	-0.00002	-003E-06
4/24/96	4:40 PM	0	0.09	0.02872	0.02875	0.02875	0.02870	0.02871	0.02873	-0.00013	-016E-06
4/24/96	6:45 PM	0	0.18	0.02885	0.02878	0.02790	0.02880	0.02879	0.02862	-0.00003	-004E-06
4/24/96	10:40 PM	0	0.34	0.02875	0.02881	..02879	0.02885	0.02878	0.02880	-0.00020	-025E-06
4/25/96	2:15 AM	0	0.49	0.02875	0.02872	0.02872	0.02871	0.02871	0.02872	-0.00013	-016E-06
4/25/96	2:30 PM	0	1.00	0.02875	0.02872	0.02868	0.02870	0.02878	0.02873	-0.00013	-016E-06
4/26/96	2:00 AM	0	1.48	0.02870	0.02872	0.02875	0.02875	0.02875	0.02873	-0.00014	-017E-06
4/26/96	2:40 PM	0	2.01	0.02875	0.02872	0.02875	0.02872	0.02875	0.02874	-0.00014	-018E-06
4/27/96	12:20 AM	0	2.41	0.02875	0.02870	0.02869	0.02871	0.02865	0.02870	-0.00010	-013E-06
4/28/96	2:30 PM	0	4.00	0.02869	0.02871	0.02870	0.02871	0.02875	0.02871	-0.00012	-014E-06
5/1/96	2:40 PM	0	7.01	0.02860	0.02859	0.02859	0.02861	0.02865	0.02861	-0.00001	-002E-06
5/4/96	12:35 AM	0	9.42	0.02858	0.02851	0.02855	0.02855	0.02850	0.02854	0.00006	007E-06
5/8/96	12:35 PM	0	13.92	0.02845	0.02848	0.02850	0.02851	0.02852	0.02849	0.00010	013E-06
5/15/96	1:05 PM	0	20.94	0.02835	0.02830	0.02829	0.02831	0.02830	0.02831	0.00029	036E-06
5/22/96	3:40 PM	0	28.05	0.02810	0.02820	0.02815	0.02815	0.02819	0.02816	0.00044	055E-06
5/29/96	2:10 PM	0	34.99	0.02799	0.02795	0.02792	0.02795	0.02799	0.02796	0.00064	079E-06
6/7/96	3:45 PM	0	44.05	0.02791	0.02795	0.02789	0.02791	0.02791	0.02791	0.00068	085E-06
6/21/96	1:00 AM	0	57.44	0.02775	0.02771	0.02771	0.02778	0.02771	0.02773	0.00086	108E-06
7/5/96	11:15 AM	0	71.86	0.02745	0.02748	0.02751	0.02749	0.02751	0.02749	0.00111	139E-06
7/19/96	11:20 AM	0	85.87	0.02720	0.02712	0.02715	0.02711	0.02712	0.02714	0.00146	182E-06
8/5/96	10:29 AM	0	102.83	0.02700	0.02695	0.02701	0.02699	0.02710	0.02701	0.00159	198E-06
9/10/96	10:35 AM	0	138.84	0.02670	0.02675	0.02678	0.02671	0.02678	0.02674	0.00185	232E-06
9/27/96	1:40 AM	0	155.47	0.02649	0.02650	0.02655	0.02649	0.02649	0.02650	0.00209	262E-06

REF SPECIMEN #9-UNLOADED				Avg
Date	Time	Load (kips)	Time (days)	Strain #9-UL
4/24/96	10:15 AM	0	0.00	
4/24/96	2:30 PM	0	0.01	014E-06
4/24/96	3:35 PM	0	0.05	-002E-06
4/24/96	4:40 PM	0	0.09	-028E-06
4/24/96	6:45 PM	0	0.18	-032E-06
4/24/96	10:40 PM	0	0.34	-032E-06
4/25/96	2:15 AM	0	0.49	-027E-06
4/25/96	2:30 PM	0	1.00	-025E-06
4/26/96	2:00 AM	0	1.48	-027E-06
4/26/96	2:40 PM	0	2.01	-028E-06
4/27/96	12:20 AM	0	2.41	-022E-06
4/28/96	2:30 PM	0	4.00	-021E-06
5/1/96	2:40 PM	0	7.01	-016E-06
5/4/96	12:35 AM	0	9.42	001E-06
5/8/96	12:35 PM	0	13.92	015E-06
5/15/96	1:05 PM	0	20.94	033E-06
5/22/96	3:40 PM	0	28.05	053E-06
5/29/96	2:10 PM	0	34.99	070E-06
6/7/96	3:45 PM	0	44.05	079E-06
6/21/96	1:00 AM	0	57.44	098E-06
7/5/96	11:15 AM	0	71.86	128E-06
7/19/96	11:20 AM	0	85.87	169E-06
8/5/96	10:29 AM	0	102.83	211E-06
9/10/96	10:35 AM	0	138.84	241E-06
9/27/96	1:40 AM	0	155.47	263E-06

1. GENERAL

To facilitate the investigation of creep characteristics in accordance with ASTM C512 and other specifications, Soiltest has designed the CT-180 Concrete Creep Tester for subjecting three standard 6 in. diameter cylinders to the necessary sustained compressive force. The unit consists of a simple loading frame, spring operated to maintain the applied force despite dimensional changes in the specimens, and a hydraulic ram and hand pump for the initial application of the desired load.

With the apparatus assembled as shown in the enclosed drawing and the lower ram bearing plate resting freely upon the specimen stack, unrestrained by the (loosened) nuts below the plate, the required load is applied with the hand pump, thus compressing the heavy springs in the lower portion of the frame. By tightening the nuts both above and below the plate, the force in the compressed springs will now be maintained by the reacting tension in the supporting tie rods of the stressed frame.

The loading frame is 78 in. high and 14 in. in diameter at the base. Its daylight opening is adjustable from approximately 39 in. to 44-3/4 in. All plates are 1 in. thick. The bearing blocks are 6 in. in diameter, and their surfaces are plane within .001 in. To insure proper leveling, the lower bearing block is seated on a 1-1/4 in. diameter ball. The 6 in. diameter springs are coiled from 1-1/2 in. diameter bars and will compress 1-3/4 in. at approximately 25,400 lb. each, or 76,200 lb. total. All parts are plated for protection.

The single acting cylinder has a 3 in. bore and stroke, and is spring returned. It is activated by a 10,000 PSI hand pump which displaces 0.294 cubic in. per stroke, and is set at the maximum load by an internal adjustable relief valve. The reservoir has an 80 cu. in. capacity.

Calibrated with U.S. Bureau of Standards certified proving rings, the 8-1/2 in. diameter, 60,000 lb. gauge is accurate to within 1 percent of the indicated load from 10 percent to full capacity. Recalibration may be effected without removing the movement from its case as the linkage and zero adjusting screw are easily accessible through ports located at the side and back of the gauge. The linkage has a micro-adjusting screw for delicate calibration changes. The gauge may be conveniently bled through a port located on the rear of the gauge. A six ft. flexible hose, equipped with quick disconnects is provided for coupling the pump and gauge to the cylinder.

2. SPECIMEN PREPARATION

In general, nine specimens are required for testing purposes, three each for compressive strength, and creep deformation, and three for controls to indicate deformations due to causes other than loading. The cylinders are fabricated from standard molds modified for the attachment of the necessary strain gauge studs and inserts.

In addition to the specimens, dummy cylinders 3 in. long are required between the specimens and bearing plates to prevent unnatural stress distributions at the specimens ends. Consult ASTM 512 for dimensional details of the specimens, curing and storage instructions.

3. STRAIN MEASUREMENT

The longitudinal strain in the specimen should be measured to the nearest 10 microin. (0.000010 in.) per in. The prime requirement of the strain-measuring device is that it shall be capable of measuring strains for at least one year without change in calibration. Systems in which the varying strains are compared with a constant length standard bar are considered most reliable. See Section 6, Accessories.

4. PROCEDURE

Immediately before loading the creep specimens, determine the compressive strength of the strength specimens.

With the ram removed from the frame, adjust the ram plates to their approximate positions and stack the specimens, dummy cylinders and bearing blocks in their respective positions within the frame. Drop the lower ram bearing plate to the stack and tighten the nuts above the plates to hold the stack securely. Adjust the stack into proper alignment to prevent eccentric loading. The ram may now be placed between the bearing plates and, with the nuts beneath the lower bearing plate loosened 1 in. or so from the plate, the load may now be applied. Load the specimens at a intensity of not more than 40 percent of the compressive strength at the age of loading.

Take strain readings immediately before and after loading, 6 hours later, then daily for one week, weekly for one month, and monthly until the end of one year. Strain readings on the control specimens should be observed on the same schedule. After the load has been applied, tighten the nuts above the lower ram bearing plate. The ram may now be removed for loading other frames, if desired.

5. CALCULATIONS

Calculate the total strain per PSI (or kilopascals) at any time as the difference between the average strain values of the loaded and control specimens divided by the average stress to determine creep strain per PSI (or Kilopascals) for any age, subtract from the total strain per PSI (or

Kilopascals) at that age the strain per PSI (or kilopascals) immediately after loading. If desired, plot total strains per PSI (or kilopascals) on semilog coordinate paper, on which the logarithmic axis represents time, to determine the constants 1/E and F(K) for the following equation:

$$e = (1/E) + F(K) \log_e (t+1)$$

where:

e = total strain per PSI (or kP_a),

E = instantaneous modulus of elasticity, in PSI (or kP_a),

F(K) = creep rate, calculated as the slope of a straight line representing the creep curve on the semilog plot, and

t = time after loading, in days.

The quantity 1/E is the initial elastic strain per PSI (or kilopascals) and is determined from the strain readings taken immediately before and after loading the specimen.

If loading is not accomplished expeditiously, some creep may occur before the after loading strain is observed, in which event extrapolation to zero time by the method of least squares may be used to determine this quantity. It should not be implied from the use of the logarithmic expression that the creep strain time relationship is necessarily an exact logarithmic function; however, for the period of one year, the expression approximates normal creep behavior with sufficient accuracy to make possible the calculation of parameters that are useful for the purpose of comparing concretes.

6. ACCESSORIES

Soiltest manufactures an extensive variety of equipment for fabricating the necessary specimens. Attention is called to the concrete section in our latest catalog.

The CT-171 Multi-Position Strain Gauge is ideally suited to the requirements mentioned in Section 3, Strain Measurement. It will measure relative displacement between gauge points of 2, 4, 6, 8, or 10 in. It is equipped with a dial indicator of 0.2 in. range graduated in 0.0001 in. divisions (the equivalent of 10 microin. over a gauge length of 10 in. on creep specimens). The unit is provided with eight brass inserts, two reusable contact seats which screw into the inserts, and two contact points which mount on the strain gauge.

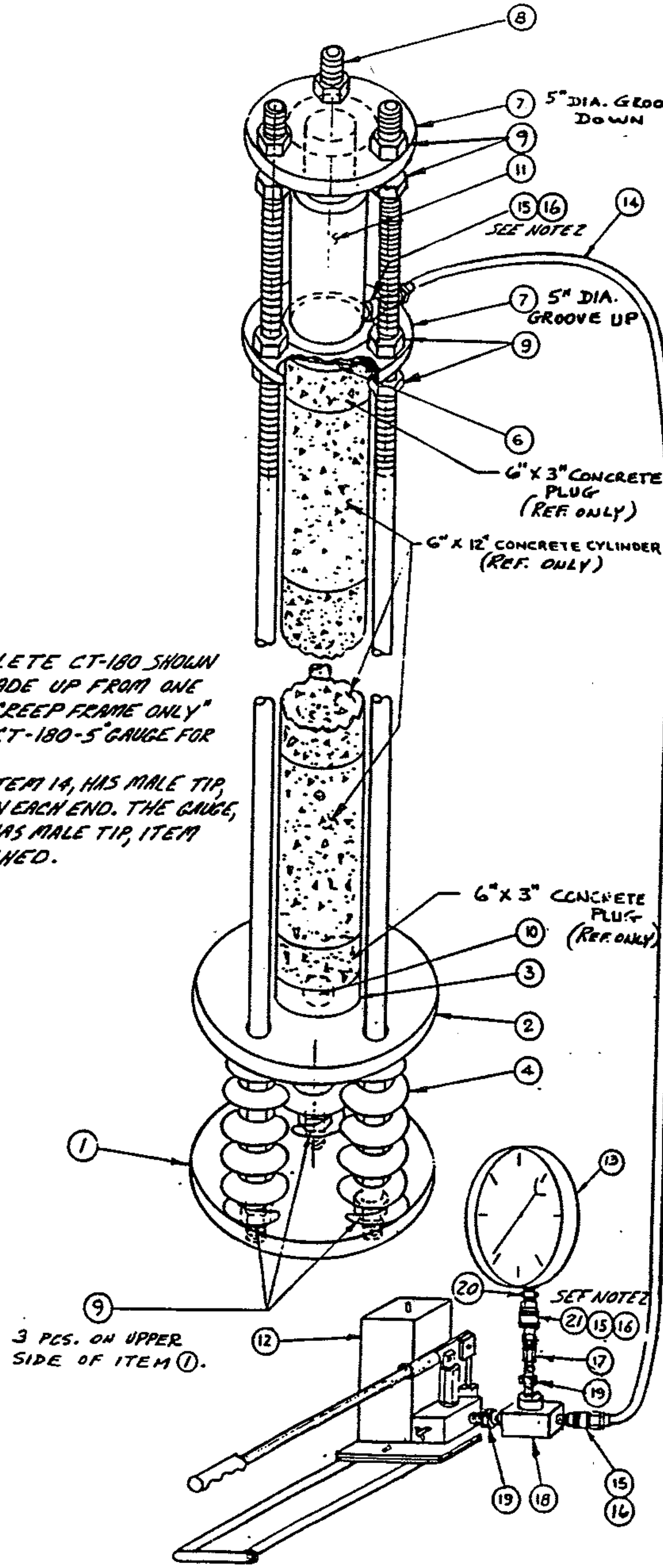
Accessory items which may be obtained include the CT-176 Punch Bar, used to establish the gauge distances for locating the contact points in the specimen, and the CT-175 invar Master Bar. This bar has a mean coefficient of thermal expansion of 0.5×10^{-6} in/in/°F for the range of 0–350°F. Over a 10 in. gauge length, a 10° temperature difference would change the master bar setting only 0.00005, of less than one dial indicator division.

Inquiries for quantity lots of either the standard or custom frames to your individual specifications will be given special consideration.

ADDITION

CT-180 Assembly Drawing

SOILTEST



NOTE:

1. THE COMPLETE CT-180 SHOWN MAY BE MADE UP FROM ONE CT-180Y, "CREEP FRAME ONLY" PLUS ONE CT-180-5" GAUGE FOR CT-180."
2. THE HOSE, ITEM 14, HAS MALE TIP, ITEM 16, ON EACH END. THE GAUGE, ITEM 13 HAS MALE TIP, ITEM 16, ATTACHED.

Appendix E.

Potential for Future ASR Expansion

Interim Interpretation of MiDOT Core Expansions

Based on the 5 month expansion of the MiDOT pier cap cores, the following <u>interim</u> 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 is available for further reaction if sufficient alkali is present.

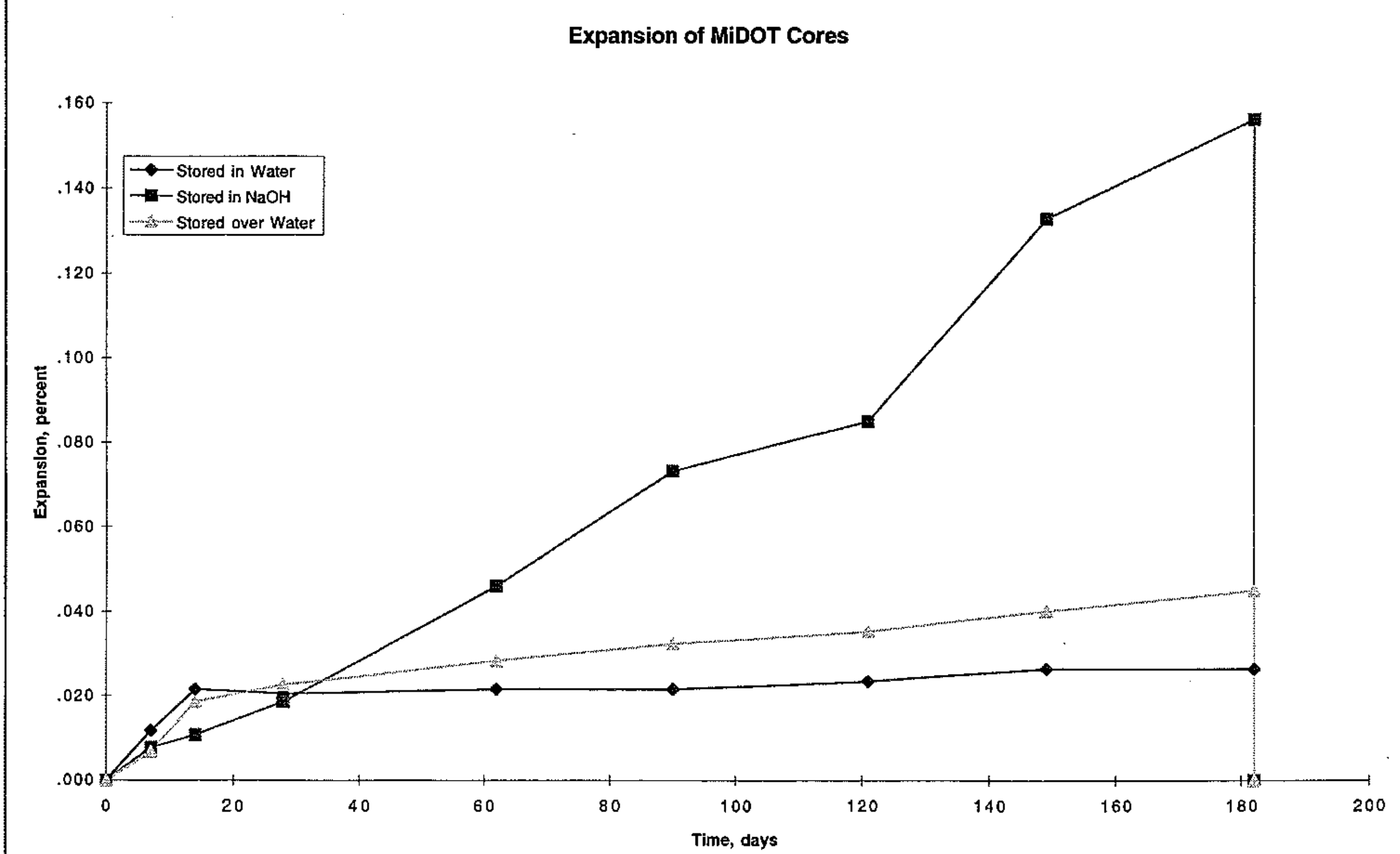
4. There appears to be small potential for further in the concrete pier cap provided that adequate moisture is available.

In summary, the cores are not yet at equilibrium with their storage environment. Therefore, conclusions at this point are speculate, and may not be representative of the final test results. Based on data obtained to date, the ASR reaction appears to have exhausted its moisture and alkali supply. Potentially reactive aggregate is available for further reaction if sufficient alkalis become available.

John Gajda
Materials Engineer
7-19-96

	In Water		In NaOH		Over Water	
Date	Length	Weight	Length	Weight	Length	Weight
2/21/96	.2107	4130	.1101	4137	.1821	4146
2/28/96	.2119	4148	.1109	4154	.1828	4144
3/6/96	.2129	4155	.1112	4162	.1840	4151
3/20/96	.2128	4161	.1120	4170	.1844	4163
4/23/96	.2129	4166	.1148	4181	.1850	4172
5/21/96	.2129	4169	.1176	4185	.1854	4174
6/21/96	.2131	4171	.1188	4188	.1857	4175
7/19/96	.2134	4171	.1237	4192	.1862	4177
8/21/96	.2134	4174	.1261	4196	.1867	4180
11/21/96						
2/21/97						

	In Water		In NaOH		Over Water	
Elapsed Days	% Length Change	Weight Change	% Length Change	Weight Change	% Length Change	Weight Change
0	.000		.000		.000	
7	.012	18	.008	17	.007	-2
14	.021	7	.011	8	.019	7
28	.021	6	.019	8	.022	12
62	.021	5	.046	11	.028	9
90	.021	3	.073	4	.032	2
121	.023	2	.085	3	.035	1
149	.026	0	.133	4	.040	2
182	.026	3	.156	4	.045	3
182						
182						



E3

CTL

Alkali-Silica Reactivity Testing

The objective of this test is to determine if there is any potential for alkali-silica reactivity (ASR) in concrete and to determine future maximum expansions. This test is usually performed on concrete that already exhibits ASR.

This ASR test consists of the determination of relative weight and length changes of "companion" samples cut from concrete cores when immersed in 1M NaOH solution (40 grams NaOH/liter of water), in water, and over water in a sealed container, all at 100°F. These weight and length changes are interpreted to indicate the potential for possible or further alkali-silica reactivity. Companion samples for testing are adjacent cores Core samples are fitted at each end with gauge points. Their weights and lengths are measured and one sample is immersed in the NaOH solution, another in water, and a third over water in a sealed container. Immersion in water is used to determine length and weight change caused by rewetting of partially dried concrete. Moisture equilibrium is identified when the samples reach constant weight ± 2 grams. This point is taken as a reference when assessing the potential for expansion from additional alkali-silica reactivity in the companion samples. Immersion of samples in NaOH solution is intended to force any potential expansive alkali-silica reactivity. Differences in expansion between cores stored in NaOH solution and in water can indicate the presence of potentially reactive aggregates. Comparison of the expansion of the cores stored over water with those stored in water can indicate the presence of sufficient alkali to sustain expansive ASR. Expansion of the samples stored in water, after weight equilibrium is reached, indicates swelling of existing alkali-silica gel. In order to establish meaningful trends, time duration of these tests is, at a minimum, six to nine months. It is suggested by CTL that testing be continued to 1 year.

Appendix F.

Historical Record

Date: August 9, 1977

To: K.A. Allemeir - Engineer of Testing Research

T.R. Wiseman - Engineer of Maintenance

From: M.G. Brown and D.J. Kanellitsas

Subject: Cracking in Pier Girders of I-75 - Rouge River Bridge

Research Project 77 TI-416

The following is a summary of a field inspection of the subject piers made by D.J. Kanellitsas and M.G. Brown on July 19, 1977. This was as per verbal request of G.J. McCarthy subsequent to an article in the July 17th Detroit News by J.F. Nehman. This article contained a picture of the south face of northbound pier 38, which is the south river pier. Our detailed inspection concerned the four largest piers at the river, nos. 37 to 40, and also those adjacently to these four.

Mr. McCarthy's remark in the article that the cracking in the pier caps is "source disintegration only" and "not load related" is correct. We believe the cracks to be nothing more than the common shrinkage and temperature related cracks associated with hot weather curing conditions and massive volumes of concrete generating great amounts of heat while curing. The largest piers, nos. 38 and 39 had girder sections 8.5 ft wide and 9 ft to 12 ft deep. Piers 37 and 40 had girders of the same depth, but 7 ft wide. This is evidenced by the random pattern of the cracking (map cracking) very typical of this type of concrete behavior, as opposed to the very definite pattern of cracks which would be caused by stress, whether shear or tensile. The fact that these cracks appear only in the largest piers is not evidence that they are stress cracks, as the newspaper article hints, but rather substantiates our position that they are due to massive concrete pours and hot weather.

A search of the project records furnished additional information which adds support to the hot weather - thermal cracking theory. The eight pier girder pours for nos. 37 to 40 were placed in hot weather form May 7 to July 29, 1965. In addition, a 7.5 sack cement per cubic yard mix was used " to obtain 70 % design strength earlier." This rich mix would have greatly increased thermal buildup and resultant surface cracking. In contrast, all pier girder pours other than the above, used the normal 6 sack of cement per cubic yard.

Shrinkage cracks on pier columns and caps usually are no more than 2" to 3" deep, remain in this condition for a long time, perhaps 20 to 30 years. Eventually if the concrete is not protected, and under wet conditions with many freeze-thaw cycles may occur. At that time we proceed with repairs. In this particular case there is no cause for alarm because:

1. The previously leaking expansion joints over piers 37 to 40 have been replaced with new joints that are relatively leakproof.

2. With the aid of "snooper," the tops of the pier caps and all the bearing rockers were inspected. No cracks were found, and the top of the piers is very sound. Therefore, the leaching of the cracks seen from below is due to water entering the top and exiting from the bottom after traveling through the pier cap. It is rather the result of water seeping through the crack itself, when water ran down the side of the pier through leaking expansion joints, or in some cases, from the overflowing downspouts on the ?? of ?? girder.

3. In order to provide added protection, now that the deck has been resurfaced, and new expansion joints have been installed, we proposed to let a contract whereby some of the most prominent cracks will be pressure grouted, and the entire cap will be sealed with protective coating.

4. D.J. Kanellitsas recalled that when these particular piers were designed (at that time he worked on drawing plans for the caisson foundation), it was not the stresses, but rather the geometry or aesthetics that controlled the size of the piers, and therefore, they were greatly over designed.

The newspaper's contention that the Department was unaware of the existence of cracks is inaccurate. Our inspection reports dated back to 1969 mentions these particular cracks in piers 37 to 40.

Neither of the "two possible causes" for the cracking offered by Dr. Almeida exists. There is no evidence of honeycomb, nor flow of water through the pier cap as explained previously. He should also be aware of the Department's annual, systematic bridge inspection program which surpasses even Federal requirements.

Finally, the newspaper write-up mentions cracks through the 8 1/2" deck. The existence of these has been for some time and usually occurs in the longer continuos spans such as the three at the river. That problem, or course, has been taken care of recently complete latex concrete overlay on the southbound spans in 1975 and the northbound this year.

FIL 77 TI-41[illegible]

OFFICE MEM[illegible]R[illegible]UM

Roll 121
Start @ 145

[illegible] 1977

K. A. Allemeier - Engineer of Testing & [illegible]
T. [illegible] Wisdom - Engineer [illegible] M[illegible]

M. [illegible] and D. J. Kagelitsas

[illegible] Rouge River [illegible] 30! of 82(94)

[illegible] field inspection of the subject piers made by D. J. [illegible] 19, 1977. This was at per verbal request of [illegible] article in the July 17th Detroit News by J. F. [illegible] The article contained a picture of the south face of northbound pier 38, [illegible] Our detailed inspection concerned the four largest [illegible] river, nos. 37 to 40, and also those adjacent to these four.

[illegible]'s remark in the article that the cracking in the pier caps is "[illegible] [illegible] only" and "not load related" is correct. We believe the cracks to be [illegible] common shrinkage and temperature related [illegible] associated [illegible] girder [illegible] 7' wide [illegible] of the same depth, [illegible] pattern [illegible] map crack [illegible] concrete [illegible] as opposed to the very definite pattern [illegible] caused by stress, whether shear or tensile. The fact that [illegible] appear only in the larger piers is not evidence that they are stress cracks, [illegible] article [illegible] substantiates our position that they are due [illegible] pours and hot weather.

[illegible] project records [illegible] additional information which adds support to [illegible] thermal cracking theory. The eight pier girder pours for nos. 37 to [illegible] in hot weather from May 7 to July 29, 1965. In addition, a 7.5 sack [illegible] to obtain 70% design strength earlier." This [illegible] thermal buildup and resultant surface cracking. [illegible] pier girders [illegible] than the above, used the normal 6 sacks of cement [illegible]

[illegible] on pier [illegible] usually are no [illegible] 2" to 3" [illegible] perhaps 20 to 30 years. [illegible] with squares [illegible] this particular case [illegible]

[illegible] The previously leaking expansion [illegible] over piers 37 and 40 have been [illegible] with new joints that are relatively leakproof.

[illegible], the tops of the pier caps and all the bearing rockers were inspected. No cracks of any consequence were found, and the top of the piers is very sound. Therefore, the leaching of the cracks seen from below is not due to water entering the top and exiting from the bottom after travelling through the pier [illegible] of [illegible] water seeping through the crack itself, [illegible] the [illegible] rough a leaking [illegible] joint, or in some cases, [illegible] downspouts on the [illegible] girder.

[illegible] added protection, now that the deck has been resurfaced, and [illegible] expansion joints installed, we propose to let a contract whereby some of the [illegible] cracks will be pressure grouted, and the entire pier cap will be [illegible] a protective coating.

4. D. J. Kaseillitos recalled that when these particular piers were designed (at that time he worked on drawing the plans for the caisson foundations), it was not the stresses, but rather the geometry or aesthetics that controlled the size of these piers, and, therefore, they are greatly overdesigned.

The newspaper's contention that the Department was unaware of the existence of cracks is [illegible] inspection reports [illegible] to 1969 [illegible] there [illegible]

[illegible] the cracking offered [illegible] flow of water through the [illegible] or explains [illegible] He should also be made aware of the Department's annual, systematic bridge inspection program which surpasses even Federal requirements.

Finally, the newspaper write-up mentions cracks through the 8-1/2" deck. The existance of these has been known for some time and usually occurs in the longer continuous spans such as the three at the river. That problem, of course, has been taken care of by recently [illegible] latex concrete overlay on the southbound spans in 1975 and the northbound this year.

M. G. Brown
Acting Supervising Engineer
Materials Research Unit-Research Laboratory

D. J. Kaseillitos
Assistant Engineer of Structures Maintenance

MGB:DJK:[illegible]

cc: J. P. [illegible]
C. J. [illegible]
M. N. [illegible]
W. J. [illegible]
H. D. LaFrance
L. T. Oehler ← THIS COPY FOR

1181-166

[illegible]

MICHIGAN STATE HIGHWAY DEPARTMENT

[illegible] OF TESTING AND RESEARCH

BRIDGE CONCRETE PROPORTIONING

PROJ. NO. [illegible]

ROUTE [illegible] CROSSING [illegible] River

CONTRACTOR R. [illegible] Butler & Co. (Jutton-Kelly Co.)

REPORT NO 69 DATE 5-21-65

Name and Location of Concrete Proportioning Plant [illegible] Co. - Miller Rd. Plant

Source of Cement Peerless Type II Silo # ____ Car # ____ Truck # ____

Source of Aggregate: Fine Amer. Agg. Green Oak CA: E.C. Levy-Trenton 6B

Plan Volume of Pour 236.0 Cu. Yds. Grade A [illegible] Chart No. [illegible]-177 (Mod.) [illegible]-177 (Remodified) Consistency: ☐ Medium ☐ Dry

PROPORTIONS

Time of Test	Wt. per cu. ft (Bone Dry) Coarse Aggregate, Lbs.	Quantities per sack of Cement from chart: Sand, Pounds	Coarse Aggregate, Pounds	Water, Pounds	Moisture Percent: Sand	Coarse Aggregate	Total Computed Mix: Cement, Sacks	Sand, Pounds	Coarse Aggregate, Pounds	Water, Pounds	Computed Water to be added at mixer, gallons	Actual water added at mixer, gallons	Relative Water Content	Slump, Inches	Estimated sacks of cement per cu. yd. of concrete
[illegible]	74	163.0	186.0	33.6	4.3	2.9	7.5	1291	1436	302.8	36.3	32.0	1.25	4	7.5
[illegible]	74	[illegible]	[illegible]	[illegible]	4.3	2.9	7.5	1242	1471	304.6	36.6	32.0	1.25	3½	7.5
														3	

Weather: A.M. Clear, Cool

P.M. Clear, [illegible]

Pour [illegible]

Time, from [illegible] to [illegible]

Number of Test Beams [illegible] Series [illegible]

Date water gauge checked 5/26/64 Result OK

Date batch scale checked 4/9/65 Result OK

Auto. Controls checked 5/17/65 Result OK

(A) Plan Volume of Pour 236.0 cyds

(B) Actual Quantity Placed by Batch Count 238.0 cyds

(C) Difference between A & B 2.0 cyds*

(D) Overrun or ~~Underrun~~ (C/A) 0.9%

(E) Total Concrete Batched 240.0 cyds

(F) Excess (E minus B) 2.0 cyds*

(G) Total Cement Batched 450.0 bbls

*Explain under Remarks

RECEIVED

By ____ FIELD TESTING DIV.

TEMPERATURES

Time	Atmosphere	Water	Fine Aggregate	Coarse Aggregate	Concrete	Housing	Air Content %
[illegible]		65°	62°	60°			
					72°		7.1 Chace
[illegible]					75°		6.7 Rollameter
[illegible]					76°		[illegible].3 Rollameter
					73°		

REMARKS [illegible] cement. [illegible] electric vibrators were used [illegible] the concrete. Concrete to be cured by [illegible] forms in place and top rolled [illegible] 2.0 cyds excess concrete wasted. [illegible] used to obtain [illegible] [illegible] [illegible] was [illegible]

Prepared by [illegible] Inspector Checked by [illegible] Project Engineer

F7

One gallon of water is equivalent to 8.33 pounds.

Reports are to be filled out completely and mailed not later than the day following the cast.

Original to District Materials Supervisor, Copy to Br. Const. Engr., Copy to Dist. Br. Engr., Retain one copy.

1431-169

Form 1000
Rev. 10/60

MICHIGAN STATE HIGHWAY DEPARTMENT
OFFICE OF TESTING AND RESEARCH
Lansing, Michigan

Project [illegible]
F. A. No. I-75-1(64)41 Part I
Series No. On175 (7 day)
Date May 28, 1965

REPORT ON MODULUS OF RUPTURE

Chart No. 65MV-177 (Modified)
Date Molded May 21, 1965
Station No.
Time of Day 6:00 PM
Temperature 65 degrees F.
Grade of Concrete A(6AA)

Concrete used in Pours D & E, Girder, Pr. [illegible]
Cement Factor 7.5 Sacks per cu. yd.
Slump in. 3
Curing Method of Structure Forms left in place and curing
Curing Method of Beam Placed under wetted sand
Consistency

Source of Cement Peerless Type II
Fine Aggregate Amer. Agg. Co. - Green Oaks
Coarse Aggregate E.C. Levy - Trenton [illegible] (6AA)
Coarse Aggregate (6A)

DATE OF TESTS
Beam A May 25, 1965
Beam B May 28, 1965
Beam C
Beam D

MACHINE NO. 31 JACK NO. 327142

Beam	Test No.	Load Applied Pounds	Modulus of Rupture, Lbs. per sq. in.	Average	Length of Beam Broken Off, In.	Average Temperature	Breadth of Beam	Depth of Beam	FACTOR
A	1	2225	726	727	12 3/4	60°	6	6-1/16"	0.3265
7-days	2	2250	727		13	60°	6-1/16"	6-1/16"	0.3232
B	1	2100	693	718	12	60°	6-1/16"	6"	0.3299
7-days	2	2300	743		11 1/2	60°	6-1/16"	6-1/16"	0.3232
C	1								
14-days	2								
D	1								
14-days	2								
C	1								
28-days	2								
D	1								
28-days	2								

RECEIVED
JUN 3 1965
FIELD TESTING DIV.
DISTRICT 10

REMARKS

Molded by J. Kelsch
Mix Inspector

Tested by T. Ledzer
Proportioning Plant Inspector

Reported by [signature]
Project Engineer Jose J. Sanchez

Repeat 7-day test results on 14-day or 28-day reports.
Copies: Original to District Materials Supervisor
[illegible] Engineer (On Bridge Construction [illegible])
[illegible] District Bridge Engineer).
Construction Engineer. (On Road Construction).
Bridge Engineer. (On Bridge Construction).
Retain one copy for Project Engineer's file.

NOTE:
Reports to be filled out completely and mailed not later than one day following the test.

F8

Form 1910
Rev. 12/60

1181-469

I-75 over Rouge River

MICHIGAN STATE HIGHWAY DEPARTMENT
OFFICE OF TESTING AND RESEARCH
Lansing, Michigan

Project: 82 101 or 82192 B3
P. A. No. I-75-1(64)43 Part X
Series No. 0-195 (4 day)
Date: May 25, 1965

REPORT ON MODULUS OF RUPTURE

Chart No. 65W-177 Modified
Date Molded: 2:00 May 21, 1965
Station No.
Time of Day: 2:00 PM
Temperature: 65 degrees F.
Grade of Concrete: A(6AA) High Early

Concrete used in: Piers D & E, Girder, pier 38 EB
Cement Factor: 7.5 Sacks per cu. yd.
Slump in.: 3
Curing Method of Structure: Forms in place & curing com...
Curing Method of Beam: Placed under wetted sand
Consistency

Source of Cement: Peerless Type II
Fine Aggregate: Amer. Agg. Co. - Green Oaks
Coarse Aggregate: E. C. Levy - Trenton (6AA)
Coarse Aggregate: (10A)

DATE OF TESTS
Beam A: May 25, 1965
Beam B:
Beam C:
Beam D:

MACHINE NO. 163   JACK NO. 327-53

Beam	Test No.	Load Applied Pounds	Modulus of Rupture, Lbs. per sq. in.	Average	Length of Beam Broken Off, In.	Average Temperature	Breadth of Beam	Depth of Beam	FACTOR
A 4 day ~~7-days~~	1	2225	726	727	12 3/4	60°	6"	6-1/16"	0.3265
	2	2250	727		13	60°	6-1/16"	6-1/16"	0.3232
B	1								
7-days	2								
C	1								
14-days	2								
D	1								
...-days	2								
C	1								
28-days	2								
D	1								
28 days	2								

RECEIVED
MAY 27 1965
By [signature]
FIELD ... ENGR.
DISTRICT 10

REMARKS: Tested at 4 days for the purpose of determining percentage of designed strength.

Molded by: ... Welsch
Mix Inspector

Tested by: T. ...edger
Proportioning Plant Inspector

Reported by: [signature]
Project Engineer ... J. Sanc...

Repeat 7-day test results on 14-day or 28-day reports.
Copies: Original to District Materials Supervisor
... Engineer (On Bridge Construction This Copy
to Division Bridge Engineer)

Form [illegible]
(Rev. [illegible])

1181-168

MICHIGAN STATE HIGHWAY DEPARTMENT
OFFICE OF TESTING AND RESEARCH
Lansing, Michigan

I-75 over [illegible]

Project: EX E01 of 82194C, C6
F. A. No. I-75-1(6)61 Part I
Series No. G-196 (28 day)
Date: June 18, 1965

REPORT ON MODULUS OF RUPTURE

Chart No. 6507-177 Mod.
Date Molded: May 21, 1965
Station No.
Time of Day: 6:00 PM
Temperature: 65 degrees F.
Grade of Concrete: A(6AA)

Concrete used in: Girder, Pr. 38 HB, [illegible]
Cement Factor: 7.5
Slump in.: 3
Curing Method of Structure: Forms left in place & curing
Curing Method of Beams: Placed in sand & wetted
Consistency:

RECEIVED
JUN 25 1965
By
FIELD TESTING DIV.
DISTRICT 10

Source of Cement: Peerless Type II
Fine Aggregate: Amer. Agg. Green Oaks
Coarse Aggregate: E.C.Levy - Trenton [illegible] (6AA)
Coarse Aggregate: (10A)

DATE OF TESTS
Beam A: May 25, 1965
Beam B: May 28, 1965
Beam C: June 18, 1965
Beam D:

MACHINE NO. 31
JACK NO. 327142

Beam	Test No.	Load Applied Pounds	Modulus of Rupture, Lbs. per sq. in.	Average	Length of Beam Broken Off, In.	Average Temperature	Breadth of Beam	Depth of Beam	FACTOR
A	1	2225	726	727	12 3/4	60°	6"	6-1/16"	0.3265
4-days	2	2250	727		13	60°	6-1/16"	6-1/16"	0.3232
B	1	2100	693	718	12	60°	6-1/16"	6"	[illegible]
7-days	2	2300	743		11½	60°	6-1/16"	6-1/16"	0.3232
C	1								
14-days	2								
D	1								
-days	2								
C	1	2650	847	836	12	66°	6-1/8"	6-1/16"	0.3198
28-days	2	2550	824		13	66°	6-1/16"	6-1/16"	0.3232
D	1								
28-days	2								

REMARKS: 4 day and 7 day Rupture Reports on this beam erroneously marked G-195; should have been marked G-196.

Molded by: J. Kelsch
Mix Inspector

Tested by: L. Minier
Proportioning Plant Inspector

Reported by: [signature]
Project Engineer J. E. Rice

[illegible] test results on 14-day or 28-day reports.

Copies: Original to District Materials Supervisor, District Engineer. (On Bridge Construction Then Copy to District Bridge Engineer). Construction Engineer. (On Road Construction). Bridge Engineer. (On Bridge Construction). Retain one copy for Project Engineer's file.

NOTE: Reports to be filled out completely and mailed not later than one day following [illegible]

F10

1181-160

[illegible]HIGAN STATE HIGHWAY DEPARTMENT

[illegible] OF TESTING AN[illegible]

BRIDGE CONCRETE PROPORTIONING

ROUTE [illegible] CROSSING [illegible] River

CONTRACTOR R. E. Dailey & Co. (Sutton-Kelly Co[illegible]

REPORT NO 70 DATE 6-4-65

Name and Location of Concrete Proportioning Plant Cooper Supply Co. - Miller Rd. Plant - Detroi[illegible]

Source of Cement Peerless Type II Silo # ________ Car # ________ Truck # ________

Source of Aggregate: Fine Amer. Agg Grosse 6AA E.C. Levy - Trenton 6B

[illegible] Volume of Pour 236.0 Cu. Yds. Grade A(6AA) Chart No. 55MV-177(Mod.) Consistency: ☐ N[illegible] ☐ D[illegible]

PROPORTIONS

Time of Test	Wt. per cu. ft. (Bone Dry) Coarse Aggregate, Lbs.	Quantities per sack of Cement from chart: Sand, Pounds	Coarse Aggregate, Pounds	Water, Pounds	Moisture Percent: Sand	Coarse Aggregate	Total Computed Mix: Cement, Sacks	Sand, Pounds	Coarse Aggregate, Pounds	Water, Pounds	Computed Water to be added at mixer, gallons	Actual water added at mixer, gallons	Relative Water Content	[illegible]
[illegible]AM	74	158.0	56.0	53.6	5.0	4.7	7.5	1244	1461	277.1	33.3	29.7	1.25	3½
[illegible]AM														3
[illegible]AM														3½

Weather A.M. Clear, Cool

P.M. Clear, Cool

Pour Pier 36 38 Girder, Pour 2 & 3

Time from [illegible] AM to [illegible] PM

Number of Test Beams 3 Series 3-14

Date water gauge checked 1/26/64 Result OK

Date batch scale checked 4/9/65 Result OK

Automatic Controls Checked 6/3/65 Result ok

A. Plan Volume of Pour 236.0

B. Actual Quantity Placed by Batch Count 235.0

C. Difference between A & B 1.0

D. Overrun or Underrun (C/A) 0.4

E. Total Concrete Batched 238.0

F. Excess [illegible] 3.0

G. Total Cement Batched 446.5

*Explain under Remarks

RECEIVED

TEMPERATURES

Time	Atmosphere	Water	Fine Aggregate	Coarse Aggregate	Concrete	Housing	Air Conten[t]
[illegible] AM		65°	56°	64°			
[illegible] AM					65°		6.9 (Chac[e])
[illegible] AM					70°		7.3 (Cha[illegible])
[illegible] AM					70°		6.0 Roll-[illegible]

REMARKS: [illegible] oz. Darex added [illegible] sack of cement. Two Stowe electric vibrators were [illegible] [illegible] vibrate the concrete. Concrete to be cured by leaving forms in place and top roll[illegible] [illegible] curing compound. [illegible] Snace from [illegible] performed air checks on the job. (F) Exc[illegible] concrete wasted. [illegible] forms being smaller than plan. See Revision [illegible] [illegible] High Early [illegible] used to obtain [illegible]% of Strength earlier.

[illegible] Checked by [illegible] J. E. Fipe

[illegible]

[illegible] are to be [illegible] complete [illegible] the day following the pour.

Original to District Materials Supervisor, Copy to Br. Cement. Engr., Copy to Dist. Br. Engr., Retain one copy.

1181469

Form 1932 (Rev. 12/55)

MICHIGAN STATE HIGHWAY DEPARTMENT
OFFICE OF TESTING AND RESEARCH
Lansing, Michigan

Project BI 3C1 of 821949, [illegible]
F. A. No. I-75-1(64)[illegible] part 1
Series No. 6-198
Date 6-8-65

REPORT ON MODULUS OF RUPTURE

Chart No. 65MV-177 (Mod.)
Date Molded 6-4-65
Station No.
Time of Day 1:00 PM
Temperature 69 degrees F.
Grade of Concrete A(6AA)

Concrete used in Pier 38 S.B. Girder, Pour B A B
Cement Factor 7.5 Sacks per cu. yd.
Slump in. [illegible]
Curing Method of Structure Top: Membrane curing Compound Side: Forms left in Place
Curing Method of Beam Buried in wet sand
Consistency

Source of Cement Peerless Type II
Fine Aggregate Amer. Agg. Greenoak[illegible]
Coarse Aggregate E.C. Levy Co. Trenton 6AA
Coarse Aggregate (6A)

DATE OF TESTS
Beam A 6-8-65
Beam B
Beam C
Beam D

MACHINE NO. 31

JACK NO. 327142

Beam	Test No.	Load Applied Pounds	Modulus of Rupture, Lbs. per sq. in.	Average	Length of Beam Broken Off In.	Average Temperature	Breadth of Beam	Depth of Beam	FACTOR
A	1	2100	671	6[illegible]5	12½	66	6 1/8	6 1/16	0.3198
4 Days	2	2100	679		13	66	6 1/16	6 1/16	0.3232
B	1								
7-days	2								
C	1								
14-days	2								
D	1								
14-days	2								
C	1								
28-days	2								
D	1								
28-days	2								

RECEIVED

By

FIELD TESTING DIV.
DISTRICT [illegible]

REMARKS Tested at [illegible] to determine percentage of [illegible] strength [illegible]
[illegible]

Molded by [illegible]
Mix Inspector

Tested by C. Haunz
Proportioning Plant Inspector

Reported by J. A. Rice
Project Engineer

F12

Repeat [illegible] test results on 14-day or 28-day reports.

Copies: Original to District Materials Supervisor
District Engineer. (On Bridge Construction This Copy [illegible] District Bridge Engineer).
Construction Engineer. (On Road Construction).
Bridge Engineer. (On Bridge Construction).
Retain one copy for Project Engineer's file.

NOTE:
Reports to be filled out completely and mailed not later than one day following the two

Form 1912
(Rev. 12/63)

1181-168

MICHIGAN STATE HIGHWAY DEPARTMENT
OFFICE OF TESTING AND RESEARCH

Lansing, Michigan

Project	[illegible]
P. A. No.	I-75-1(54)43 Part I
Series No.	C-198 (11 day)
Date	June 15, 1965

REPORT ON MODULUS OF RUPTURE

Chart No.	6917-177 Mod.	Concrete used in	Girder, Pr. 38 S8, beams D & E
Date Molded	June 4, 1965	Cement Factor	7.5 sacks per cu. yd
Station No.		Slump in.	3-1/4
Time of Day	3:00 PM	Curing Method of Structure	Top: membrane curing compound; Sides: forms left in place
Temperature	71 degrees F.	Curing Method of Beam	Placed in sand & wetted
Grade of Concrete	A(6AA)	Consistency	

Source of Cement: Peerless Type II

Fine Aggregate: Amer. Agg. Co. - Green Oaks

Coarse Aggregate: E.C. Levy - Trenton (6AA)

Coarse Aggregate: (10A)

DATE OF TESTS

Beam A: June 15, 1965

Beam B:

Beam C:

Beam D:

MACHINE NO. 31 JACK NO. 327142

Beam	Test No.	Load Applied Pounds	Modulus of Rupture, Lbs. per sq. in.	Average	Length of Beam Broken Off, In.	Average Temperature	Breadth of Beam	Depth of Beam	FACTOR
A	1	2200	711	711	12 3/4	65°	6-1/16"	6-1/16"	0.3232
11 11-days	2	2200	711		12	65°	6-1/16"	6-1/16"	0.3232
B	1								
7-days	2								
C	1								
14-days	2								
D	1								
14-days	2								
C	1								
28-days	2								
D	1								
28-days	2								

RECEIVED
JUN 21 1965
By
FIELD TESTING DIV.
DISTRICT 10

REMARKS

Molded by: J. Olszewski Jr. & P. Fay, Mix Inspector

Tested by: L. Minier, Prestressing Plant Inspector

Reported by: Project Engineer J. F. [illegible]

Repeat 7-day test results on 14-day or 28-day reports.

Copies: Original to District Materials Supervisor
District Engineer. (On Bridge Construction This Copy to District Bridge Engineer).
Construction Engineer. (On Road Construction).
Bridge Engineer. (On Bridge Construction).
Retain one copy for Project Engineer's file.

NOTE:
Reports to be filled out completely and mailed not later than one day [illegible]

181-168

Form 1602
Rev. 10/60

I-75 [illegible]

MICHIGAN STATE HIGHWAY DEPARTMENT
OFFICE OF TESTING AND RESEARCH
Lansing, Michigan

Project: [illegible] of [illegible], [illegible]
F. A. No.: I-75-1(64)43 Part 1
Series No.: C-198 (28 day)
Date: 7/2/65

REPORT ON MODULUS OF RUPTURE

Chart No.: 65MF-177 (Modified)
Date Molded: 6/4/65
Station No.:
Time of Day: 3:00 PM
Temperature: 71 degrees F.
Grade of Concrete: A(6AA)

Concrete used in: Girder Pier 38 SB, Pour D & E
Cement Factor: 7.5 Sacks per cu. yd.
Slump in.: 3½
Curing Method of Structure: Top: Membrane curing compound; Sides: Forms left in Place
Curing Method of Beam: Placed under wetted sand
Consistency:

Source of Cement: Peerless Type II
Fine Aggregate: Amer. Agg. Co. Greencastle
Coarse Aggregate: E.C.Levy Co. Trenton (4A) (6A) A
Coarse Aggregate: (10A)

DATE OF TESTS
Beam A: 6-8-65
Beam B: 6-15-65
Beam C: 7-2-65
Beam D:

MACHINE NO.: 31
JACK NO.: 327 42

Beam	Test No.	Load Applied Pounds	Modulus of Rupture, Lbs per sq. in.	Average	Length of Beam Broken Off In.	Average Temperature	Breadth of Beam	Depth of Beam	FACTOR
A	1	2100	671	675	12½	66°	6 1/8	6 1/16	0.3198
4 3-days	2	2100	679		13	66°	6 1/16	6 1/16	0.3232
B	1	2200	711	711	12 3/4	65°	6 1/16	6 1/16	0.3232
11 7-days	2	2200	711		12	65°	6 1/16	6 1/16	0.3232
C	1								
14-days	2								
D	1								
14-days	2								
C	1	2300	751	772	12½	65°	6	6 1/16	0.3265
28-days	2	2450	792		12½	65°	6 1/16	6 1/16	0.3232
D	1								
28-days	2								

RECEIVED

REMARKS

[illegible] 1965

Molded by: W. Wiszewski & F. Fay
Mix Inspector

Tested by: F. Ledger
Proportioning Plant Inspector

[illegible] by: G. E. Rice
Project Engineer

Report 7-day test results on 14-day or 28-day reports

By

FIELD TESTING DE.
DISTRICT 10

Copies: Original to District Materials Supervisor
District Engineer. (On Bridge Construction This Copy to District Bridge Engineer).
Construction Engineer. (On Road Construction).
Bridge Engineer. (On Bridge Construction).
Retain one copy for Project Engineer's file.

Reports to be filled out completely and mailed not later than one day following the test.

F14

1181-182

H. E. MacDougall
P. A. Nordgren
A. J. Sinelli

J. J. Vrscan
J. Rice
file

MICHIGAN STATE HIGHWAY DEPARTMENT
OFFICE OF TESTING & RESEARCH
Lansing, Michigan

Control Section: 82 201 of 82194C, C1

F.A. No.: I-75-1 (64) 43 Part 1

Date: 5-4-65 Report No.: 197

REPORT OF CONCRETE INSPECTION

Length ____ Miles. Width ____
Cross Section ____ Reinforcement: steel bars
Location of plant and bridge site: Fisher Freeway over Rouge River

Contractor: Jutton - Kelly
Project Engineer: J. Rice
Plant Inspector: T. Ledger
Site Inspector: D. Faye

EQUIPMENT

Transit Mix Supplier: Cooper Supply Company
Address: Miller Road - Detroit
Mixers ____
Proportioning Plant ____
Finishing Machine ____
Longitudinal Float ____
Transportation ____

Scales checked date: 4-2-65
Water meter checked date: 4-2-65
Mixer Performance: Satisfactory
Condition of Forms: Satisfactory
Curing Method: Forms Condition: Satisfactory
Brand Name ____ Lot No. ____
Condition of Stockpiles: Satisfactory
Load Transfer Device Mfg. ____

MATERIALS

Cement: Peerless - Detroit Type I
Fine Agg.: Am. Agg. Green Oak (47-3) Spec. 2NS
Coarse Agg.: E. C. Levy Co. (Trenton) Spec. 6A
Coarse Agg. ____ Spec. ____
Grade of Concrete: A

Mixture Proportions

Mix Design No.: 65 MV-177 Modified
Wt. per cu. ft. of loose C.A. (bone dry): 74

Material	Quan. Per Sack of Cement	Moisture Content of Agg. %	Computed Batch, lb.
A: 2NS	2.2	2.0	1244
A: 6AA	3.2	1.7	1461
A:			
Cement	94	7.5 sacks per batch	705
Water	53.5		33.3 gals.

Water actually used in mix: 33.0 gals.
Slump: 2 1/2 in. Water Withheld: 0.3 gals.

Pour: "D and E" of Pier 38 (Girder, Southbound)
Average Air Content: 6.0 % A.E. Additive: Darex 2 5/8 oz. per sack
Location of Test ____

Average Air Content ____ % A.E. Additive ____ oz. per sack
Location of Test ____

Cylinders ____ At ____ Days Last average 2.9 date 5-21-65
Temperature of Air: 54 °F Concrete: 70 °F Time of Test: 11:10 A.M.

Remarks ____

W. Drake
James E. Fergus
Assistant District
Materials Supervisor
jes

Testing and Research Engineer

F15

1181-159

Form 340
(Rev. 4-63)

MICHIGAN STATE HIGHWAY DEPARTMENT
OFFICE OF TESTING AND RESEARCH

BRIDGE CONCRETE PROPORTIONING

PROJ. NO [illegible]
ROUTE [illegible] CROSSING Rouge River
CONTRACTOR [illegible]
REPORT NO 71 DATE 7-10-65

Name and Location of Concrete Proportioning Plant Cooper Supply Co. Miller Rd. Plant-Det.
Source of Cement Peerless Type [illegible] Silo # ______ Car # ______ Truck # ______
Source of Aggregate: Fine Amer. Agg. Grosse 6AA E. C. Levy Co. Trenton 6B
Plan Volume of Pour 236.0 Cu. Yds. Grade A(6AA) Chart No. 65MV-177 Consistency: ☐ Medium ☐ Dry

PROPORTIONS

Time of Test	Wt per cu. ft. Bone Dry Coarse Aggregate, lbs	Quantities per sack of Cement from chart: Sand, Pounds	Coarse Aggregate, Pounds	Water, Pounds		Moisture Percent: Sand	Coarse Aggregate	Total Computed Mix: Cement, Sacks	Sand, Pounds	Coarse Aggregate, Pounds	Water, Pounds	Computed Water to be added at mixer, gallons	Actual water added at mixer, gallons	Relative Water Content	Slump-inches	Estimated sacks of cement per cu. yd. of concrete
									1238	1405	170.4	24.5	19.6	1.25	1½	7.5
[illegible]	[illegible]	150.[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]								
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]		3	
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	4952	[illegible]	115.[illegible]	[illegible]	116.4		3	
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	3714	4395	530.[illegible]	100.[illegible]	[illegible]			

Weather: A.M. [illegible]
P.M. [illegible]
Pour [illegible] Girder [illegible]
Time from [illegible] A.M. to [illegible]
Number of Test Beams [illegible] Series [illegible]
Date water gauge checked 7-1-65 Result OK
Date batch scale checked 4-[illegible]-65 Result OK
Date automatic Controls checked [illegible] Result OK

A Plan Volume of Pour 236.0 cyds
B Actual Quantity Placed by Batch Count 239.5 cyds
C Difference between A & B 3.5 cyds*
D [illegible] 1.5 %
E Total Concrete Batched 240.0 cyds
(F) Excess (E minus B) 0.5 cyds*
(G) Total Cement Batched 450.0 bbls
By [illegible]
*Explain under Remarks

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FIELD TESTING DIV
DISTRICT 10
1965

TEMPERATURES

Time	Atmosphere	Water	Fine Aggregate	Coarse Aggregate	Concrete	Housing	Air Content %
7:3[illegible] A.M.		6[illegible]°	[illegible]°	55°			
[illegible] A.M.					[illegible]°		7.5 (Chace)
[illegible]					[illegible]°		
[illegible]							[illegible] (Chace)
[illegible]							[illegible] (Chace)

REMARKS [illegible] oz. [illegible] added per sack of cement. [illegible] Stowe electric vibrators were used to [illegible] roller [illegible] [illegible] used to obtain [illegible]

F16

1181-158

I-75 Monroe [illegible] River

Project: BI 801 of 821940, 91

F. A. No. I-75-1(64)45 Part 1

Series No. G-203 (4 day)

Date 7/4/65

[illegible] 1962
[illegible] 12/63

MICHIGAN STATE HIGHWAY DEPARTMENT
OFFICE OF TESTING AND RESEARCH
Lansing, Michigan

REPORT ON MODULUS OF RUPTURE

Chart No. 65MV-177 Modified

Date Molded 7/10/65

Station No.

Time of Day 5:00 PM

Temperature 81 degrees F.

Grade of Concrete A(6AA)

Concrete used in Girder Pier 39 NB, Pour B & E

Cement Factor 7.5 Sacks per cu. yd

Slump in. 2½

Curing Method of Structure Top: Membrane curing compound Sides: forms left in place

Curing Method of Beam Placed under wetted sand

Consistency

Source of Cement Peerless Type II

Fine Aggregate Amer. Agg. Co. Greencake

Coarse Aggregate E. G. Levy Co. Trenton (6A)

Coarse Aggregate (2A)

DATE OF TESTS

Beam A 7/14/65

Beam B

Beam C

Beam D

MACHINE NO. 31

JACK NO. 327142

Beam	Test No.	Load Applied Pounds	Modulus of Rupture, Lbs. per sq. in.	Average	Length of Beam Broken Off, In.	Average Temperature	Breadth of Beam	Depth of Beam	FACTOR
A	1	2550	785	780	11½	64°	6 1/8	6 3/16	0.3070
[illegible]-days	2	2500	776		12½	64°	6 3/16	6 1/8	0.3102
B	1								
7-days	2								
C	1								
14-days	2								
D	1								
14-days	2								
C	1								
28-days	2								
D	1								
28-days	2								

RECEIVED
JUL 20 1965
By
FIELD TESTING D..
DISTRICT 10

REMARKS

[illegible]
Mix Inspector

Tested by I. Ledrer F17
Proportioning Plant Inspector

Reported by J. E. Rice
Project Engineer

Repeat [illegible] test results on 14-day or 28-day reports.

Copies: [illegible] Original to District Materials Supervisor
[illegible] District Engineer. (On Bridge Construction This Copy [illegible]
[illegible] District B[illegible] Engineer)
[illegible] Engineer. (On Road Construction)

NOTE: [illegible] filled out completely and mailed not later than one day following the two [illegible]

(Rev. 12/60)

MICHIGAN STATE HIGHWAY DEPARTMENT
OFFICE OF TESTING AND RESEARCH
Lansing, Michigan

I-75 over [illegible]

Project: [illegible]

F. A. No. I-75-1([illegible])[illegible] Part I

Series No. G-203 (10 day)

Date: July 20, 1965

REPORT ON MODULUS OF RUPTURE

Chart No. 65 MW-17B Modified

Date Molded: July 10, 1965

Station No.

Time of Day: 2:00 PM

Temperature: 81 degrees F.

Grade of Concrete: A(6AA)

Concrete used in: Girder, Pr. 39 [illegible], [illegible] D & E

Cement Factor: 7.5 Sacks per cu. yd.

Slump in: 2½

Curing Method of Structure: Top: Membrane curing compou[nd] Sides: forms left in place

Curing Method of Beam: Placed under wetted sand

Consistency

Source of Cement: Peerless Type II

Fine Aggregate: Amer. Agg. Co. - Green Oaks

Coarse Aggregate: E. C. Levy - Trenton (6A.A)

Coarse Aggregate: (10A)

DATE OF TESTS

Beam A: July 14, 1965

Beam B: July 20, 1965

Beam C:

Beam D:

MACHINE NO. 31 JACK NO. 27142

Beam	Test No.	Load Applied Pounds	Modulus of Rupture, Lbs. per sq. in.	Average	Length of Beam Broken Off, In.	Average Temperature	Breadth of Beam	Depth of Beam	FACTOR
A	1	2550	783	780	11½	64°	6-1/8"	6-3/16"	0.3070
4 7-days	2	2500	776		12½	64°	6-3/16"	6-1/8"	0.3102
B	1	2900	891	913	11 3/4	67°	6-1/4"	6-1/8"	0.3071
10 8-days	2	2950	934		12	67°	6-3/16"	6-1/16"	0.3166
C	1								
14-days	2								
D	1								
14-days	2								
D	1								
28-days	2								
D	1								
28-days	2								

RECEIVED
JUL 22 1965
By [signature]
FIELD TESTING DIV.
DISTRICT 10

REMARKS

Made by: J. Kelsch
Mix Inspector

Tested by: T. Ledzer
Proportioning Plant Inspector F18

Reported by: [signature]
Project Engineer J. [illegible]

Repeat lines test results on 14-day or 28-day reports

Copies: Original to District Materials Supervisor
District Engineer. (On Bridge Construction This Copy to District Bridge Engineer)
Construction Engineer. (On Road Construction)
Bridge Engineer. (On Bridge Construction)

NOTE:
Reports to be filled out completely and mailed not later than one day following th[e]

MICHIGAN STATE HIGHWAY DEPARTMENT
OFFICE OF TESTING AND RESEARCH
Lansing, Michigan

Project: [illegible]
F. A. No. I 75-1(64)43 Part I
Series No. G-203 (31 day)
Date 8-10-65

REPORT ON MODULUS OF RUPTURE

Chart No. 69MV-177 Modified
Date Molded 7-10-65
Station No.
Time of Day 3:00 PM
Temperature 81 degrees F
Grade of Concrete [illegible](AA)

Concrete used in Girder, Pier 39 NB, pours D & E
Cement Factor 7.5 Sacks per cu. yd.
Slump in. 2½
Curing Method of Structure Top: membrane curing compound Sides: forms left in place
Curing Method of Beam Placed under wetted sand
Remarks

RECEIVED
AUG 13 1965
FIELD TESTING DIV.

DATE OF TESTS
Beam A 7-14-65
Beam B 7-20-65
Beam C 8-10-65
Beam D

Source of Cement Peerless Type II By
Fine Aggregate Amer. Agg. Corp. - Green Oaks
Coarse Aggregate E. C. Levy - Trenton 6AA
Coarse Aggregate 10A

MACHINE NO. 21
JACK NO. 527142

Beam	Test No.	Load Applied Pounds	Modulus of Rupture, Lbs per sq. in.	Average	Length of Beam Broken Off, In.	Average Temperature	Breadth of Beam	Depth of Beam	FACTOR
A	1	2550	783	780	12½	64°	6-1/8"	6-3/16"	0.3070
[illegible]-days	2	2500	776		12½	64°	6-3/16"	6-1/8"	0.3102
B	1	2900	891	913	13-3/4	67°	6-1/4"	6-1/8"	0.3071
1[illegible]-days	2	2950	934		12	67°	6-3/16"	6-1/16"	0.3166
C	1								
6-days	2								
D	1								
14-days	2								
[illegible]	1	2950	906	906	12-3/4	64°	6-1/4"	6-1/8"	0.3071
28-days	2	2800	905		12½	64°	6-3/16"	6- 6"	0.3232
[illegible]	1								
28-days	2								

REMARKS

Molded by [illegible] Welsch
Mix Inspector

Tested by [illegible] Ledner
Proportioning Plant Inspector

Reported by [signature]
Project Engineer J. E. Rice

Report 7-day test results on 14-day or 28-day reports.
[illegible] District Materials Supervisor
[illegible] Engineer (On Bridge Construction [illegible])
[illegible] Engineer
Construction Engineer (On Road Construction)
Bridge Engineer (On Bridge Construction)
Return one copy for Project Engineer's file.

NOTE
Reports to be filled out completely and mailed not later than one day following the test.

1181-159

MICHIGAN STATE HIGHWAY DEPARTMENT

OFFICE OF TESTING AND RESEARCH

BRIDGE CONCRETE PROPORTIONING

PROJ. NO. [illegible] 01 of 821940, C1

ROUTE I-75 CROSSING Rouge River

CONTRACTOR R. E. Dailey & Co. (Sub: Jutton-Kelly

REPORT NO 72 DATE 7-29-65

Name and Location of Concrete Proportioning Plant Cooper Supply Co. - Miller Rd. Plant - Detroit

Source of Cement Peerless Type III Bin # ______ Car # ______ Truck # ______

Source of Aggregate: Fine Amer. Agg. - Greenwood; CA R.C.Lacy - Trenton 6B

Plan Volume of Pour 236.0 Cu. Yds. Grade A(64A) Chart No. 65MV-178 Mod. Consistency: ☐ Medium ☐ Dry

PROPORTIONS

Time of Test	Wt. per cu. ft. (Bone Dry) Coarse Aggregate, Lbs.	Quantities per sack of Cement from chart: Sand, Pounds	Coarse Aggregate, Pounds	Water, Pounds	Moisture Percent: Sand	Coarse Aggregate	Total Computed Mix: Cement, Sacks	Sand, Pounds	Coarse Aggregate, Pounds	Water, Pounds	Computed Water to be added at mixer, gallons	Actual water added at mixer, gallons	[illegible]	[illegible]	[illegible]
[illegible]:30 AM	74	158.0	186.0	22.[illegible]	4.1	4.2	7.5	1244	1426	322.7	36.7	35.2	[illegible]	3½	7.[illegible]
[illegible]:30 PM	74	158.0	186.0	53.6	5.0	4.5	7.5	1244	1458	280.0	33.6	28.0	1.25	3½	7.[illegible]
7 - 9 cyd loads batched in 4½ cyd batches							33.75	5553	6417	1452.1	174.2	158.4		3½	33.[illegible]
[illegible] - 8 cyd loads batched in 4 cyd batches							30.0	4936	5704	1290.8	154.8	140.8		3½	30.[illegible]
[illegible] - [illegible] cyd load batched in 1½ cyd batch							11.25	1566	2187	420.0	50.4	42.0			[illegible]
[illegible] - 8 cyd load batched in 4 cyd batches							30.0	4976	5832	1120.0	134.4	112.0			30.[illegible]

Weather: A.M.

AM & P.M. Partly cloudy & cool

Pours D & E, Pier 39 SB, Girder

Time, from 7:25 AM to [illegible]:30 PM

Number of Test Beams 3 Series 2-205

Date water gauge checked 7-1-65 Result OK

Date batch scale checked 4-9-65 Result OK

Auto. Controls Checked 7-1-65 Result OK

(A) Plan Volume of Pour 236.0 cyds

(B) Actual Quantity Placed by Batch Count 241.5 cyds

(C) Difference between A & B 5.5 cyds

(D) Overrun or Underrun (C/A) 2.3%

(E) Total Concrete Batched 250.5 cyds

(F) Excess (E minus B) 9.0 cyds

(G) Total Cement Batched 469.7 bbls

*Explain under Remarks

RECEIVED [illegible] 1965

By ______

FIELD TESTING DIV. DISTRICT 10

TEMPERATURES

Time	Atmosphere	Water	Fine Aggregate	Coarse Aggregate	Concrete	Housing	Air Content %
[illegible]:30 AM		65°	66°	64°			
[illegible]:40 AM							6.5 Chace
[illegible]:00 AM					74°		
[illegible]:50 AM							7.0 Chace
[illegible]:30 AM							7.0 Roll[illegible]

REMARKS 2-5/8 oz. Darex added per sack of cement. Two Stowe electric vibrators were [illegible] to vibrate the concrete. Concrete cured by leaving forms in place and top rolled with curing compound. High Early Concrete was used to obtain 70% strength earlier. (F) Cha[illegible] cyd load rejected because too wet. J. Kibler of T & R took Rollameter air check.

Prepared by J. Olszewski Jr. & J. Kelsch (Conc.) Inspector L. Minier (Plant)

Checked by [signature] Project Engineer J. E. Rice

F20

[illegible]

Reports are to be filled out completely and mailed not later than the day following the cast.

Original to District Materials Supervisor, Copy to Br. Constr. Engr., Copy to Dist. Br. Engr., Retain one copy.

MICHIGAN STATE HIGHWAY DEPARTMENT
OFFICE OF TESTING AND RESEARCH
Lansing, Michigan

1181-168

Project: [illegible]
F. A. No. I-75-1(64)83 Part I
Series No. C-205 (4 day)
Date: August 2, 1965

REPORT ON MODULUS OF RUPTURE

Chart No. 65W-177 (Modified)
Date Molded: July 29, 1965
Station No.
Time of Day: 3:00 PM
Temperature: 70 degrees F
Grade of Concrete: A(6AA)

Concrete used in: Girder, Pier 39 [illegible]
Cement Factor: 7.5 Sacks per cu. yd.
Slump in.: 3½
Curing Method of Structure: Top: membrane curing compo Sides: forms left in place
Curing Method of Beam: Placed under wetted sand
Consistency:

Source of Cement: Peerless Type II
Fine Aggregate: Amer. Agg. Co. - Greenoaks
Coarse Aggregate: E.C. Levy - Trenton (6AA)
Coarse Aggregate: (10A)

DATE OF TESTS
Beam A: 8-2-65
Beam B:
Beam C:
Beam D:

MACHINE NO. 31 JACK NO. 327140

Beam	Test No.	Load Applied Pounds	Modulus of Rupture, Lbs. per sq. in.	Average	Length of Beam Broken Off, In.	Average Temperature	Breadth of Beam	Depth of Beam	FACTOR
A	1	2500	816	767	12	67°	6-1/8"	6"	0.3265
[illegible]-days	2	2200	718		13	67°	6-1/8"	6"	0.3265
B	1								
7-days	2								
C	1								
14-days	2								
D	1								
14-days	2								
C	1								
28-days	2								
D	1								
28-days	2								

RECEIVED
AUG 5 1965
By:
FIELD TESTING DIV.
DISTRICT 10

REMARKS

Molded by: [illegible] Mix Inspector
Tested by: [illegible] Ledger, Proportioning Plant Inspector
Reported by: [signature] Project Engineer [illegible]

[illegible] test results on 14-day or 28-day reports [illegible] to District Materials Engineer [illegible] to District Bridge Engineer. Construction Engineer. (On Road Construction). Bridge Engineer. (On Bridge Construction). Retain one copy for Project Engineer's file.

NOTE: Reports to be filled out completely and mailed not later than one day following

MICHIGAN STATE HIGHWAY DEPARTMENT
OFFICE OF TESTING AND RESEARCH
Lansing, Michigan

1181-162

P. A. No.	I-75-1(64)43
Series No.	9-205 (7 day)
Date	8-5-65

REPORT ON MODULUS OF RUPTURE

[illegible] No. 65MV-177 (Modified)

Concrete used in Girder, Pier 59 S.B., Beams B & [illegible]

Molded 7-29-65

Cement Factor 7.5 Sacks per cu. [illegible]

[illegible] No.

Slump in. 3½

[illegible] of Day 3:00 PM

Curing Method of Structure Top: Membrane curing compound [illegible] Sides: Forms left in place

[illegible]rature 70 degrees F.

Curing Method of Beam Placed under wetted sand

[illegible] of Concrete A(6AA)

Consistency

[illegible] of [illegible]ent Peerless Type II

[illegible] Aggregate Amer. Agg. Co. - Greenoaks

[illegible] Aggregate E.C. Levy Co. Trenton (6A) 6A A

[illegible] Aggregate (10A)

DATE OF TESTS

Beam A 8-2-65

Beam B 8-5-65

Beam C

Beam D

[illegible]CHINE NO. 31

JACK NO. 327142

[illegible]am	Test No.	Load Applied Pounds	Modulus of Rupture, Lbs. per sq. in.	Average	Length of Beam Broken Off In.	Average Temperature	Breadth of Beam	Depth [illegible] Beam	FACTOR
A 4	1	2500	816	767	12	67°	6 1/8	6	0.3265
Days	2	2200	718		13	67°	6 1/8	6	0.3265
B 7	1	2350	783	784	12½	67°	6	6	0.3333
Days	2	2400	784		10½	67°	6	6 1/16	0.3265
C 14	1								
Days	2								
C 14	1								
Days	2								
C	1								
Days	2								
D	1								
Days	2								

RECEIVED
AUG 10 1965
By
FIELD TESTING DIV.
DISTRICT 10

[illegible]ARKS

[illegible]ed by V. Kelsch
Mill Inspector

Tested by T. Ledger
Proportioning Plant Inspector

Reported by J. E. Rice
Project Engineer

[illegible] 7-day test results on 14-day or 28-day reports.
[illegible] Original to District Materials Supervisor
District Engineer. (On Bridge Construction This Copy to District Bridge Engineer)
Construction Engineer. (On Road Construction)
Bridge Engineer. (On Bridge Construction)
Retain the Copy for Project Engineer's File.

NOTE:
Reports to be filled out completely and mailed not later than one day following the [illegible]

F22

[illegible]

MICHIGAN STATE HIGHWAY DEPARTMENT
OFFICE OF TESTING AND RESEARCH
Lansing, Michigan

Project [illegible]
F. A. No. I-75-1(65)4? [illegible]
Series No. S-205 (28 day)
Date 8-26-65

REPORT ON MODULUS OF RUPTURE

Chart No. 6500-177 (Modified)
Date Molded 7-29-65
Station No.
Time of Day 3:00 PM
Temperature 70 degrees F.
Grade of Concrete X(6AA)

Concrete used in Girder, Pr. 39 [illegible]
Cement Factor 7.5
Slump in 3½
Curing Method of Structure Top: membrane curing [illegible] Sides: forms left in [illegible]
Curing Method of Beam Covered with wet sand
Consistency

Source of Cement Peerless Type II
Fine Aggregate Amer. Agg. Co. - Greenoaks
Coarse Aggregate 6A
Coarse Aggregate [illegible]

DATE OF TESTS
Beam A 8-2-65
Beam B 8-5-65
Beam C 8-26-65
Beam D

MACHINE NO. 1 JACK NO. 327142

Beam	Test No.	Load Applied Pounds	Modulus of Rupture Lbs per sq. in.	Average	Length of Beam Broken Off. In.	Average Temperature	Breadth of Beam	Depth of Beam	FACTOR
[illegible]	1	[illegible]	[illegible]	[illegible]	12	[illegible]	6-?/8"	6"	0.3265
[illegible]	2	[illegible]	[illegible]		12	57°	[illegible]	6"	0.3265
[illegible]	1	2350	789	79[illegible]	12½	67°	6"	6"	[illegible]
?-days	2	2400	784		10½	57°	6"	6-1/16"	0.3265
C	1								
14-days	2								
D	1								
[illegible]	2								
[illegible]	1	[illegible]	[illegible]	[illegible]	12		6-?/8"	6-1/8"	0.3133
[illegible]	2	[illegible]	[illegible]		[illegible]		6"	6-1/8"	0.3199
28-days	2								

RECEIVED

REMARKS

F23

[illegible]
Inspector

Tested by A. [illegible]
Proportioning Plant Inspector

Reported by [illegible]
[illegible] Engineer [illegible]

A. S. Macdougall
R. A. Nordgren
A. Sinelli
Vracan
J. Rice
File

MICHIGAN STATE HIGHWAY DEPARTMENT
OFFICE OF TESTING & RESEARCH
Lansing, Michigan

Form 407
(Rev. 10-64)

Project: BI 801 of 821940, C1
F.A. No. I-75-1 (64) 43 Part 1
Date: 7-29-65 Report No. 202

REPORT OF CONCRETE INSPECTION

Length ______ Miles. Width ______
Cross section ______ Reinforcement: steel bars
Location of [redacted] bridge site: Fisher Expressway over Rouge River

Contractor: Dutton - Kelly Company
Project Engineer: J. Rice
Plant Inspector: L. Minier
Site Inspector: J. Kelch

EQUIPMENT

Transit Mix Supplier: Cooper Supply Company
Address: Miller Road - Detroit
Mixers ______
Proportioning Plant ______
Finishing Machine ______
Longitudinal Float ______
Transportation ______
Vibrator ______

MATERIALS

Cement: Peerless - Brennan Street Type ______
Fine Agg.: American Agg. (47.3) Spec. ______
Coarse Agg.: Levy Slag (Trenton) Spec. ______
Coarse Agg. ______ Spec. ______
Grade of Concrete: A High-Early-Strength

Mixture Proportions

Mix Design No.: 65 MV-177 Modified
Wt. per cu. ft. of loose C.A. (bone dry): 74

Materials	Quantity Per Sack of Cement lb.	Moisture Content of Agg. %	Computed Batch, lb.
FA 2NS	158.0	4.1	1236
CA 6AA	186.0	2.2	1426
CA			
Cement	94	7.5 sacks per batch	705
Water	53.6		38.7

Water actually used in mix: 34.0
Slump: 3 in. [redacted]: 3.3

Batch Scales Checked (date): 4-9-65
Water Gauge Checked (date): 4-7-65
Mixer Performance: Satisfactory
Condition of Finish: Satisfactory
Curing Method: forms & wetted burlap Condition: Satisfactory
Brand Name: ---- Lot No. ----
Condition of Stockpiles: Satisfactory
Load Transfer Device Mfg.: ----

Pour: D and E of Pier 39 Southbound Girder
Average Air Content: 7.0 % A.E. Additive: Darex 2 5/8 oz. per ______
Location of Test ______

Average Air Content ______ % A.E. Additive ______ oz. per ______
Location of Test ______

Field Test Result ______ of ______ % air. Last [redacted] overrun 0.7 % date 6-26-65
Temperature Air 68 °F Concrete 84 °F Time of Test 11:30 A.M.

REMARKS ______

EM

______ Assistant District Materials Supervisor

Testing and Research Engineer

Typist: ibg Date: 8-9-65 Proof read by ______

Station		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	Average
SOUTH HAVEN EXP FARM	MAX	74	72	73	69	72	73	75	74	78	75	59	67	63		78	76	62	76	64	77		77	67	83	79	80	74	53	51	60	75	71.5
	MIN	45	48	58	42	50	58	[illegible]	56	62	54	46	48	43		60	54	44	42	46	41		51	47	55	65	58	51	43	42	38	48	50.3
* * *																																	
SOUTH CENTRAL LOWER 09																																	
BATTLE CREEK WBCK	MAX	82	84	81	67	70	73	83	84	83	76	65	76	71	78	74	77	57	81	71	72	81	76	64	77	83	81	68	55	57	65	77	73.8
	MIN	48	53	60	50	48	56	59	60	64	53	46	46	45	43	56	53	48	42	49	45	44	62	48	51	64	60	47	38	39	34	44	50.2
CHARLOTTE	MAX	81	85	82	78	71	75	85	85	85	79	67	78	74	79	76	78	63	84	77	77	84	80	73	77	85	82	69	57	59	68	77	76.3
	MIN	45	44	59	50	47	53	53	57	60	60	45	39	38	49	51	53	48	36	45	42	40	59	50	50	62	59	55	35	42	31	39	48.6
COLDWATER STATE SCHOOL	MAX	83	82	81	69	68	75	84	84	82	77	65	76	71	78	77	78	58	81	72	71	82	79	64	79	85	82	69	62	57	66	76	74.6
	MIN	48	56	61	50	48	55	57	58	62	53	47	47	45	44	55	54	44	43	49	46	45	61	54	53	63	62	48	40	39	36	44	50.5
EAST LANSING HORT FARM	MAX	76	83	82	81	70	76	85	85	85	81	72	78	76	78	79	79	78	82	82	70	81	81	73	76	84	81	73	64	56	66	74	77.0
	MIN	44	45	50	49	46	53	55	61	64	63	50	44	42	40	50	64	49	38	52	43	42	64	49	51	60	64	63	39	40	34	39	50.2
HASTINGS FISHERIES	MAX	81	84	82	73	74	75	88	85	84	76	65	78	75	81	80	77	59	85	73	76	85	83	78	80	85	81	72	54	58	69	78	76.6
	MIN	50	47	62	50	44	56	57	60	65	64	50	44	38	44	56	51	48	38	48	39	46	61	49	53	65	62	46	39	38	31	38	49.6
HILLSDALE	MAX	82	83	81	70	68	73	83	83	82	76	65	76	74	78	77	78	65	81	75	73	82	81	70	78	85	86	68	63	56	67	75	75.3
	MIN	47	57	60	54	48	54	57	59	61	61	45	45	44	45	51	61	51	41	50	43	42	63	48	52	61	60	53	37	36	36	41	50.4
IONIA 5 NW	MAX	78	82	80	75	71	77	87	80	80	78	58	66	72	76	74	74	59	81	72	71	82	82	74	76	83	79	72	50	56	64	72	73.9
	MIN	47	45	50	49	41	54	57	60	60	60	49	34	47	42	53	59	44	39	47	42	42	60	41	52	64	61	50	37	35	32	39	48.5
JACKSON FAA AIRPORT	MAX	82	84	82	69	66	75	84	84	83	77	64	76	70	75	78	79	56	81	70	72	82	75	63	75	83	82	70	60	55	66	74	73.9
	MIN	47	51	62	50	47	55	57	62	65	52	46	44	41	43	52	56	46	40	47	44	41	58	52	53	62	64	48	37	37	37	43	49.6
LANSING WB AIRPORT	MAX	80	84	84	64	71	77	85	85	86	77	54	79	68	77	76	79	58	83	72	73	78	73	58	76	86	76	59	49	57	68	74	73.7
	MIN	43	42	60	47	45	55	52	63	66	54	47	43	39	44	54	55	42	39	50	40	46	56	48	53	63	65	48	40	39	31	37	48.6
OWOSSO SEWAGE PLANT	MAX	70	79	80	68	69	75	84	83	83	77	65	77	67	74	76	77	64	78	69	70	79	73	64	76	85	81	68	55	56	65	71	72.8
	MIN	43	43	61	45	42	55	56	65	64	60	51	43	37	42	54	64	46	39	51	41	39	61	47	51	59	63	52	40	40	35	40	49.3
SAINT JOHNS	MAX	74	84	82	81	70	77	85	87	84	81	70	78	78	78	78	78	67	81	82	72	81	81	73	76	86	85	76	64	65	67	74	76.9
	MIN	45	45	61	46	43	45	55	62	64	63	51	46	40	41	55	64	45	41	50	40	44	62	45	51	62	63	58	40	39	36	39	49.7
THREE RIVERS	MAX	85	84	83	82	69	72	84	85	84	78	69	78	76	80	80	78	66	83	77	75	84	84	80	81	84	80	70	59	58	68	79	77.3
	MIN	49	54	59	56	52	59	55	60	65	62	46	43	48	45	59	65	51	41	51	42	41	62	54	55	65	61	51	37	37	35	44	51.7
* * *																																	
SOUTHEAST LOWER 10							P. 37 SB							P. 37 NB							P. 38 NB											
ADRIAN 2 NNE	MAX	78	84	82	67	67	73	83	86	86	77	65	77	73	77	82	83	82	64	74	71	83	82	62	75	86	85	70	67	56	68	77	75.3
	MIN	44	52	57	48	46	51	60	57	62	55	47	40	44	39	46	63	47	40	49	44	40	55	51	56	59	63	50	40	38	42	44	49.5
ANN ARBOR UNIV OF MICH	MAX	81	78	85	85	66	77	86	86	86	73	71	76	75	76	80	81	75	81	80	70	81	81	72	75	84	82	68	65	52	63	74	76.3
	MIN	51	45	43	48	46	54	60	61	66	45	48	48	46	46	52	62	53	46	55	46	46	64	48	51	58	64	64	41	40	44	48	51.9
DEARBORN	MAX	81		84	82	65	74	82	87	86	83	76	79	79	72	85		84	85	77	72	82	82		74	87	86	77	70				79.6
	MIN	47		59	47	45	54	58	56	65	64	49	49	45	41	48		54	48	43	47	42	63		49	56	61	62	39				51.6
DETROIT WBAP CITY	MAX	70	79	85	58	67	73	81	86	86	80	68	81	70	73	84	85	66	77	74	73	81	75	62	72	85	84	73	66	54	57	71	74.6
	MIN	44	50	57	46	45	52	57	57	68	60	52	51	45	46	54	65	52	46	58	48	45	54	49	53	58	66	50	44	42	45	47	51.7
DETROIT WBAP W WAYNE C	MAX	71	80	84	68	65	75	79	86	85	79	70	79	72	73	85	85	65	75	74	73	82	78	62	76	88	86	73	68	56	70	74	75.4
	MIN	43	48	58	46	46	53	56	54	62	58	47	44	44	43	48	62	48	41	50	47	43	53	50	54	57	62	51	41	38	42	42	49.4
DETROIT WBAP WILLOW RN	MAX	68	83	81	64	64	74	82	83	82	76	65	76	69	71	82	81	62	76	71	71	81	75	59	75	84	83	71	65	53	66	74	73.1
	MIN	40	44	51	47	45	51	58	53	61	56	45	43	43	39	43	61	50	38	49	42	35	49	45	50	56	61	50	40	37	41	42	47.3
FLINT WB AIRPORT	MAX	73	82	83	60	71	78	86	85	84	78	66	77	68	74	81	79	60	77	71	71	80	73	62	77	88	84	70	60	57	68	73	74.1
	MIN	44	46	57	48	48	56	60	63	64	55	48	43	40	41	53	56	47	39	49	42	40	52	49	52	60	64	50	40	39	37	40	49.1
GROSSE POINTE FARMS	MAX	58	60	84	83	59	63	66	84	85	84	79	77	78	66	82	83	83	72	72	68	75	79	66	65	80	85	76	74	66	69	69	73.9
	MIN	42	46	53	45	43	48	51	52	63	60	49	46	43		52	61	55	45	57	48	48	61	50	51	55	62	64	45	41	43	46	50.8
LAPEER	MAX	70	78	83	78	70	71	80	84	84	78	75	77	77	73	82	79	61	80	80	75	81	81	69	75	88	85	71	64	54	67	73	75.6
	MIN	41	45	55	45	45	51	56	59	64	62	48	44	43	38	51	51	46	37	54	40	48	59	46	47	57	53	64	41	39	37	42	49.0
MILFORD GM PROVING GRN	MAX	69	81	81	66	67	76	82	83	82	75	63	75	68	72	77	76	55	75	68	68	78	70	57	73	82	78	67	57	51	61	70	71.1
	MIN	45	47	60	46	47	53	57	62	65	53	45	44	42	44	53	55	43	38	48	44	44	51	49	51	57	62	47	38	38	38	39	48.5
MONROE SEWAGE PLANT	MAX	72	68	83	84	58	70	73	86	88	80	78	75	77	88	85	87	84	67	73	71	73	81	71	65	87	89	75	72	65	63	70	75.4
	MIN	43	50	57	47	47	50	53	57	62	64	48	49	47	45	55	64	56	48	59	50	45	62	52	55	58	64	65	45	39	48	48	52.6
MOUNT CLEMENS AF BASE	MAX	63	64	83	54	61	66	68	85	84	77	63	76	64	68	80	80	61	69	70	68	77	69	57	69	83	83	71	63	51	66	68	69.7
	MIN	40	45	49	43	45	48	51	51	61	56	48	41	40	37	50	59	45	39	49	42	39	45	42	48	53	65	50	41	39	43	44	46.7
PONTIAC STATE HOSPITAL	MAX	67	79	83	82	68	74	80	85	84	79	75	77	76	72	81	80	75	76	76	72	80	80	70	75	83	82	71	68	56	68		75.8
	MIN	43	46	58	45	47	50	56	55	65	64	48	45	44	44	48	63	50	42	56	45	44	61	48	49	56	63	64	42	40	41		50.7
PORT HURON SEWAGE PL	MAX	62	71	85	84	61	69	75	86	85	79	80	77	77	73	83	83	81	75	75	64	81	81	56	71	84	85	73	72	60	67	67	74.9
	MIN	38	40	57	39	40	48	47	52	60	58	49	46	42	39	49	59	48	43	50	43	44	52	45	46	52	64	64	44	41	42	49	48.1
WILLIS 5 SSW	MAX	69	85	83	64	64	78	84	85	85	77	66	76	72	73	85	82	61	77	73	70	82	78	61	73	86	84	70	66	55	67	75	74.4
	MIN	41	47	57	48	45	50	58	62	64	60	47	44	44	38	45	61	54	39	52	45	39	52	48	52	57	62	52	42	37	41	42	49.2

9 MAXIMUM AND MINIMUM TEMPERATURES ARE THE HIGHEST AND LOWEST HOURLY READINGS FOR EACH DAY.

From CLIMATOLOGICA DATA 1965
US Dpt of Commerce
Weather Bureau

DAILY TEMPERATURES

MICHIGAN
JUNE 1965

Continued

Station		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	Average
SOUTH HAVEN EXP FARM	MAX	80	78	70	76	86	84	76	78	69	66	68	71	70	65	74	70	67	63	78	83	75	86	84	70	72	81	90	87	80	65		75.4
	MIN	57	58	45	49	54	61	59	61	56	59	49	50	44	50	48	49	49	44	50	65	57	58	63	51	50	53	66	69	61	58		54.8
* * *																																	
SOUTH CENTRAL LOWER 09																																	
BATTLE CREEK WBCK	MAX	81	65	67	72	79	84	74	83	78	79	79	81	71	69	70	70	73	76	81	85	77	88	80	74	75	81	91	91	85	73		77.7
	MIN	59	49	42	43	49	66	62	59	59	58	50	52	45	48	48	48	51	50	51	61	53	52	63	51	49	49	63	69	58	55		55.7
CHARLOTTE	MAX	83	67	67	73	78	85	76	85	78	79	81	84	79	69	70	70	73	78	84	87	81	90	86	75	75	81	90	92	83	75		79.1
	MIN	54	52	40	45	47	64	62	57	56	51	50	45	40	47	46	47	50	44	44	52	61	45	64	45	46	50	57	72	55	52		51.3
COLDWATER STATE SCHOOL	MAX	82	67	66	72	82	85	74	84	79	82	81	83	72	71	72	74	74	80	83	86	79	90	84	76	75	83	90	93	85	74		79.3
	MIN	57	49	43	47	50	65	60	57	59	52	47	47	43	46	46	48	50	44	50	56	51	49	65	50	48	49	62	68	58	54		52.3
EAST LANSING HORT FARM	MAX	82	79	66	71	76	84	79	82	76	77	79	84	80	70	70	69	71	77	82	80	85	89	88	75	74	82	89	88	87	74		78.8
	MIN	54	53	40	44	47	65	63	59	59	57	48	53	46	48	45	48	51	50	52		63	48	66	51	51	49	61	74	61	57		53.9
HASTINGS FISHERIES	MAX	84	66	70	75	78	85	77	82	79	79	80	83	75	74	73	72	75	80	83	87	77	89	85	76	76	81	90	90	84	76		79.4
	MIN	54	57	40	43	50	66	62	60	58	57	47	46	40	46	44	46	47	46	47	59	61	50	63	48	46	49	60	74	60	54		52.7
HILLSDALE	MAX	81	67	69	73	81	85	73	83	78	81	81	82	75	71	73	74	73	76	82	85	80	90	84	74	76	82	89	89	84	74		78.8
	MIN	41	55	42	45	50	65	61	56	61	56	49	47	40	44	44	47	49	43	45	53	60	46	65	48	45	40	60	70	60	53		51.3
IONIA 5 NW	MAX	81	64	68	73	78	85	80	82	76	78	80	83	74	72	69	66	74	80	82	87	78	86	76	76	76	79	89	88	82	75		77.9
	MIN	50	52	41	43	49	61	62	58	60	59	49	53	39	47	46	45	47	50	52	60	60	49	67	47	49	52	59	76	57	54		53.1
JACKSON FAA AIRPORT	MAX	81	66	67	71	78	85	73	84	78	77	79	80	70	70	71	72	70	74	80	85	77	88	84	74	75	81	92	93	87	75		77.9
	MIN	57	48	40	47	51	67	60	60	61	56	48	47	42	44	44	47	50	47	51	59	52	48	64	49	46	49	59	69	59	53		52.5
LANSING WB AIRPORT	MAX	83	63	68	73	76	85	74	82	77	76	81	85	73	71	72	70	73	79	84	90	79	88	82	74	76	82	91	93	83	76		78.6
	MIN	55	46	39	42	52	67	63	62	58	55	46	47	42	45	43	47	49	46	47	55	53	50	58	48	49	49	62	71	56	49		51.7
OWOSSO SEWAGE PLANT	MAX	81	62	66	70	77	84	76	82	76	76	79	81	68	68	69	62	70	75	83	85	77	85	77	71	72	79	89	90	81	72		76.0
	MIN	50	52	36	41	49	64	64	60	60	56	48	51	42	44	43	47	49	47	49	58	63	51	67	50	48	46	64	75	59	54		52.9
SAINT JOHNS	MAX	82	83	67	72	78	87	79	82	79	77	80	83	79	72	70	69	72	78	83	87	78	86	88	77	76	80	89	91	91	79		79.8
	MIN	51	52	40	43	49	63	63	61	61	56	51	53	42	45	45	46	48	48	53	60	58	54	66	51	48	48	63	74	61	54		53.6
THREE RIVERS	MAX	83	71	68	75	84	85	74	86	80	83	82	83	79	74	74	73	76	78	84	87	85	90	88	77	77	84	90	91	82	75		80.6
	MIN	55	56	46	44	48	68	63	57	59	55	48	46	46	49	48	49	50	46	45	57	65	49	66	52	51	47	61	68	61	54		53.6
* * *					Rec 38 SB																				Rec 40 NB								
SOUTHEAST LOWER 10																																	
ADRIAN 2 NNE	MAX	83	70	64	70	82	90	74	85	81	82	83	85	73	70	72	74	74	78	84	89	83	90	85	75	74	79	92	92	87	74		79.8
	MIN	54	51	46	47	46	60	60	54	63	60	51	53	44	46	51	52	52	48	51	58	55	48	65	54	47	47	56	67	62	51		55.3
ANN ARBOR UNIV OF MICH	MAX	82	79	65	72	80	89	73	84	83	78	81	84	81	68	68	72	70	76	81	87	87	89	86	84	75	81	91	92	90	75		80.1
	MIN	56	54	46	48	53	63	64	59	64	62	51	58	48	40	49	52	51	52	56	64	66	56	68	56	52	53	63	72	66	57		56.6
DEARBORN	MAX	82	82	66	70	79		83	78	77	75			80		67	68	67	79	80		81	85	83	77	69	79		87	83	81		77.4
	MIN		54	43	41	41		64	60	64	61				43		51	50	50	54		61	52	66	54	49	46		56	66	57		53.8
DETROIT WBAP CITY	MAX	84	66	67	69	79	87	75	81	80	80	79	85	71	66	72	73	72	79	85	88	81	85	86	73	75	80	90	92	88	76		78.8
	MIN	58	49	45	48	53	65	66	63	65	60	52	54	48	48	51	54	52	54	56	64	66	55	67	55	50	54	63	71	64	55		56.8
DETROIT WBAP M WAYNE C	MAX	82	68	67	72	78	89	75	82	80	80	81	86	73	69	74	74	73	81	85	90	84	88	87	77	75	83	92	94	90	76		80.2
	MIN	55	50	44	45	50	62	62	57	62	57	51	53	45	50	51	54	50	47	51	55	56	45	65	51	46	49	56	68	61	55		53.4
DETROIT WBAP WILLOW RN	MAX	80	66	65	70	76	89	72	82	78	78	73	83	70	66	71	73	71	78	83	89	81	88	85	76	75	80	91	93	87	76		78.3
	MIN	51	45	40	39	45	60	59	57	62	55	47	51	43	45	46	51	49	48	51	58	58	47	61	52	46	44	53	67	61	48		51.3
FLINT WB AIRPORT	MAX	83	64	67	72	80	88	73	84	76	77	80	84	70	70	71	71	73	78	85	88	79	88	83	74	75	80	90	94	86	76		78.6
	MIN	54	48	40	40	50	64	60	59	58	53	48	50	43	45	47	51	50	45	49	56	56	50	59	51	47	45	63	72	60	50		52.1
GROSSE POINTE FARMS	MAX	81	87	63	64	76	89	87	83	83	77	78	85	85	66	69	72	71	77	84	88	88	86	87	85	73	75	91	94	93	88		80.8
	MIN	55	54	48	47	54	61	62	60	63	58	55	57	45	46	52	53	49	50	53	60	65	54	62	52	50	50	62	69	65	57		55.6
LAPEER	MAX	82	72	65	70	80	89	86	83	83	76	79	82	77	68	68	66	70	78	84	88	85	87	86	75	75	79	90	92	89	76		79.3
	MIN	52	55	39	40	45	60	64	57	60	56	45	49	42	42	43	47	46	43	46	55	64	48	65	48	42	42	59	73	62	53		51.4
MILFORD GM PROVING GRN	MAX	79	60	63	67	72	83	70	80	74	74	78	81	67	65	68	68	69	76	81	83	78	85	81	71	72	78	88	90	84	72		75.2
	MIN	53	45	40	44	51	64	61	59	59	55	51	52	44	46	45	49	49	49	52	56	60	49	61	52	47	52	63	69	61	51		53.0
MONROE SEWAGE PLANT	MAX	80	82	60	63	73	86	86	82	83	82	83	86	86	70	66	68	70	78	85	90	90	83	86	87	74	73	88	94	95	90		80.6
	MIN	60	58	45	49	53	64	64	60	63	63	54	59	48	50	53	56	53	52	56	58	66	53	68	60	52	53	65	68	67	59		57.6
ϕMOUNT CLEMENS AF BASE	MAX	81	61	63	65	73	84	74	78	74	72	75	82	66	62	68	70	69	76	81	87	78	83	84	69	71	78	87	92	85	73		75.4
	MIN	56	46	41	40	44	61	61	57	61	50	46	47	42	43	46	51	48	47	52	56	55	52	62	48	46	46	61	71	63	54		51.8
PONTIAC STATE HOSPITAL	MAX	82	81	68	69	77	87	80	82	81	78	80	83	80	67	69	71	71	77	84	86	84	87	87	75	75	80	90	92	87	79		79.0
	MIN	54	52	44	44	49	63	63	60	62	57	50	55	46	45	49	50	49	51	54	60	65	53	65	52	49	50	57	70	64	57		54.6
PORT HURON SEWAGE PL	MAX	76	62	59	69	81	89	88	82	81	78	77	82	60	56	63	62	66	71	84	88	88	86	85	81	65	77	89	93	93	67		76.6
	MIN	51	49	45	45	48	63	63	55	64	56	50	56	46	48	49	53	52	49	54	59	62	55	67	54	49	50	60	72	66	57		54.9
WILLIS 5 SSW	MAX	81	67	64	69	78	89	74	84	80	80	82	84	73	68	71	74	74	79	83	88	82	89	86	75	73	77	91	91	87	76		79.0
	MIN	59	52	43	46	46	60	63	55	61	57	49	50	41	48	49	54	51	46	47	53	59	45	61	51	46	47	56	67	60	55		52.9

ϕ MAXIMUM AND MINIMUM TEMPERATURES ARE THE HIGHEST AND LOWEST HOURLY READINGS FOR EACH DAY.

E26

Continued

DAILY TEMPERATURES

Station		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	Average
SOUTH HAVEN EXP FARM	MAX	76	77	75	80	78	81	79	81	79	76	78	79	84	82	74	78	76	75	76	78	77	84	89	85	76	75	77	76	74	73	70	[illegible]
	MIN	47	62	57	57	62	50	63	63	69	54	53	57	65	68	64	58	64	59	56	52	51	69	72	71	61	65	66	54	49	49	62	[illegible]
* * *																																	
SOUTH CENTRAL LOWER 09																																	
BATTLE CREEK WBCK	MAX	77	80	76	84	80	78	70	84	81	79	82	85	90	85	80	83	83	80	76	78	84	90	96	90	83	86	83	76	71	78	79	[illegible]
	MIN	47	57	58	51	61	54	61	60	63	54	57	55	59	64	59	53	58	59	57	48	46	62	70	67	60	62	63	53	50	46	62	[illegible]
CHARLOTTE	MAX	78	84	80	86	80	78	74	87	84	81	84	89	93	89	83	85	81	83	77	78	85	93	96	93	89	89	87	80	74	79	80	[illegible]
	MIN	41	51	53	46	58	47	60	56	63	48	54	50	57	67	48	46	53	51	55	43	42	61	60	71	52	52	60	44	40	40	52	[illegible]
COLDWATER STATE SCHOOL	MAX	77	78	77	83	83	79	73	85	81	78	80	84	90	84	81	83	84	81	75	75	83	89	93	91	83	84	83	77	73	77	79	[illegible]
	MIN	43	52	54	50	55	50	61	60	62	54	55	55	58	61	57	49	58	61	56	48	46	61	69	65	59	60	62	50	49	43	60	[illegible]
EAST LANSING HORT FARM	MAX	78	80	80	83	81	79	78	86	86	80	80	86	91	90	84	81	81	79	77	76	82	89	92	91	89	85	81	80	71	77	79	[illegible]
	MIN	41	54	59	49	61		61	59	64	50	58	52	62	70	58	51	57	57	56	43	46	64	65	69	60	56	64	53	50	43	61	[illegible]
HASTINGS FISHERIES	MAX	79	82	77	84	83	79	75	85	84	80	83	88	91	90	80	82	86	82	80	81	87	89	96	95	86	86	83	81	76	83	81	[illegible]
	MIN	44	55	57	50	63	53	60	62	67	49	53	53	63	69	56	53	57	57	55	45	48	65	64	68	57	55	63	49	47	44	60	[illegible]
HILLSDALE	MAX	78	75	77	80	83	77	72	85	80	78	81	84	88	84	80	82	85	80	76	75	82	87	92	94	82	83	82	76	73	77	77	[illegible]
	MIN	46	46	57	48	56	46	60	59	58	51	52	53	58	68	53	67	56	61	55	43	44	60	68	67	56	57	64	45	45	44	58	[illegible]
IONIA 5 NW	MAX	78	75	76	82	77	79	76	87	82	79	82	88	90	86	78	79	77	79	75	75	84	88	94	90	83	83	82	78	73	80	79	[illegible]
	MIN	41	60	60	56	52	45	59	57	71	45	51	50	69	74	51	56	53	53	52	43	43	75	61	65	57	54	66	49	47	41	58	[illegible]
JACKSON FAA AIRPORT	MAX	79	77	78	84	81	77	72	85	83	79	82	84	89	85	80	83	82	80	76	76	82	91	93	92	82	83	84	75	72	77	82	[illegible]
	MIN	43	54	58	53	57	51	60	60	61	53	56	56	64	61	56	50	57	60	56	45	45	64	66	69	59	60	59	50	47	45	60	[illegible]
LANSING WB AIRPORT	MAX	80	82	82	85	76	80	73	87	87	80	82	87	94	87	82	83	79	81	77	78	84	92	94	92	85	87	85	77	73	78	80	[illegible]
	MIN	40	59	55	48	58	46	61	57	59	46	56	51	65	60	53	54	60	57	56	39	44	66	60	67	55	53	56	49	45	41	63	[illegible]
OWOSSO SEWAGE PLANT	MAX	77	77	78	84	74	78	74	86	85	78	79	85	90	85	78	80	75	78	72	75	81	88	88	90	84	84	82	81	68	75	77	[illegible]
	MIN	41	58	58	51	62	45	60	58	67	57	54	52	64	71	55	55	57	54	50	40	48	65	61	69	56	53	58	50	47	44	61	[illegible]
SAINT JOHNS	MAX	79	80	79	85	85	79	79	86	87	84	81	86	91	93	85	80	80	80	79	77	90	94	93	91	86	85	85	83	73	79	79	[illegible]
	MIN	43	57	57	50	62	46	60	59	67	49	54	53	65	69	55	57	55	56	51	42	45	62	69	57	55	59	58	53	48	43	62	[illegible]
THREE RIVERS	MAX	78	80	80	83	82	79	77	85	84	79	82	85	89	87	80	84	83	82	79	76	81	89	92	92	82	84	82	79	77	80	78	[illegible]
	MIN	46	56	59	51	58	54	63	60	66	55	59	53	60	69	55	50	59	61	56	49	47	63	70	69	59	58	65	54	50	46	62	[illegible]
* * *																																	
SOUTHEAST LOWER 10			R. 40 SB								P. 39 NB																			Pier 39 SB			
ADRIAN 2 NNE	MAX	79	76	81	82	83	75	73	86	83	83	82	82	89	83	81	84	81	81	74	74	81	88	93	94	83	84	84	78	71	78	80	[illegible]
	MIN	42	50	57	57	57	48	62	59	65	55	54	53	55	64	54	52	59	59	52	44	45	57	68	69	60	58	59	52	50	46	53	[illegible]
ANN ARBOR UNIV OF MICH	MAX	78	77	78	81	80	79	73	85	85	83	84	86	90	90	86	83	82	79	79	72	78	87	90	88	81	81	78	82	70	77	78	[illegible]
	MIN	49	55	62	58	63	50	61	61	66	56	61	60	62	72	61	60	62	60	57	48	50	60	66	68	59	61	54	56	49	49	58	[illegible]
DEARBORN	MAX	76	75				77	74	84	85	80		80	85	85	82	78	78		73	68	73	88	89	95	83	85	86	85	71	76	80	[illegible]
	MIN	45	53				46	62	60	67	56		56	58	56	57	62	62		57	46	50	59	66	52	62	62	67	57	53	49	57	[illegible]
DETROIT WBAP CITY	MAX	78	79	82	81	80	75	74	86	86	78	77	82	88	86	78	84	77	80	71	73	79	87	85	95	85	81	86	78	71	76	78	[illegible]
	MIN	47	60	64	59	57	50	64	62	69	60	61	58	61	70	59	61	64	63	57	50	55	63	68	70	63	62	65	59	57	51	59	[illegible]
DETROIT WBAP M WAYNE C	MAX	78	80	84	82	84	78	74	86	86	80	80	81	90	85	82	84	79	79	71	72	79	91	90	96	85	83	85	78	72	78	80	[illegible]
	MIN	41	50	59	54	56	50	61	58	67	58	56	55	56	65	54	54	58	63	55	48	51	58	69	68	62	61	63	54	50	49	55	[illegible]
DETROIT WBAP WILLOW RN	MAX	77	79	81	82	83	76	71	84	84	78	80	80	88	84	81	85	80	80	71	72	78	88	90	93	83	82	83	77	70	76	80	[illegible]
	MIN	42	52	59	51	51	43	61	56	64	53	54	52	56	64	53	54	58	58	51	44	46	56	66	66	61	59	62	54	50	46	54	[illegible]
FLINT WB AIRPORT	MAX	79	81	80	84	77	78	76	86	87	78	82	86	93	88	81	82	76	80	75	75	82	89	90	93	88	86	85	78	66	76	78	[illegible]
	MIN	40	55	60	52	53	45	61	59	60	51	56	53	65	64	55	53	59	57	50	41	50	66	63	69	56	58	57	50	48	44	56	[illegible]
GROSSE POINTE FARMS	MAX	74	79	83	82	82	72	74	81	86	85	76	78	87	87	83	84	81	76	77	69	74	83	81	94	82	80	84	83	71	71	79	[illegible]
	MIN	48	59	60	57	62	49	62	60	66	51	59	52	59	72	52	58	61	61	56	50	53	62	65	70	60	63	64	55	54	49	59	[illegible]
LAPEER	MAX	79	79	79	84	82	78	75	84	84	81	79	85	92	90	86	80	79	78	76	74	82	89	89	90	90	85	84	78	67	75	79	[illegible]
	MIN	36	54	58	58	60	41	59	58	65	49	53	50	65	72	53	50	55	57	44	37	44	59	59	71	54	53	54	50	49	40	50	[illegible]
MILFORD GM PROVING GRN	MAX	76	76	75	79	75	75	70	82	83	76	80	81	88	84	77	80	75	75	70	71	77	87	87	86	81	81	81	74	64	74	74	[illegible]
	MIN	42	60	60	53	54	47	60	59	63	54	59	56	65	63	55	54	59	59	51	46	49	64	64	66	59	59	63	55	50	46	61	[illegible]
MONROE SEWAGE PLANT	MAX	76	80	82	82	84	73	75	86	87	84	77	79	89	90	82	87	87	79	79	73	77	91	90	97	84	83	85	83	71	75	80	[illegible]
	MIN	50	60	62	58	62	50	64	60	68	59	58	60	61	70	59	59	61	62	58	50	54	64	70	69	64	61	65	59	52	51	62	[illegible]
9 MOUNT CLEMENS AF BASE	MAX	73	79	81	82	76	72	73	81	84	75	76	79	88	85	76	83	75	76	69	70	75	83	83	92	82	78	82	73	67	72	77	[illegible]
	MIN	40	61	60	51	50	43	64	61	62	53	57	51	56	60	55	54	60	59	50	42	46	63	64	65	59	59	57	56	50	46	56	[illegible]
PONTIAC STATE HOSPITAL	MAX	78	78	80	85	81	77	75	85	85	84	81	82	88	89	86	84	81	79	77	73	78	89	89	94	85	85	85	80	70	77	77	[illegible]
	MIN	45	58	59	53	63	46	59	59	68	53	58	56	62	72	56	56	61	60	55	46	50	62	65	71	60	59	62	54	52	48	55	[illegible]
PORT HURON SEWAGE PL	MAX	79	79	80	83	81	77	74	78	84	84	75	79	87	87	83	82	80	71	68	69	77	86	86	94	85	81	82	76	70	70	78	[illegible]
	MIN	50	55	60	55	59	48	61	62	67	56	58	55	61	72	60	62	62	63	54	48	50	62	65	70	60	64	62	55	55	51	57	[illegible]
WILLIS 5 SSW	MAX	77	76	80	82	83	75	72	86	83	79	80	81	88	82	81	83	89	79	73	72	77	87	89	93	84	83	82	77	71	78	78	[illegible]
	MIN	38	46	59	50	54	44	61	57	65	53	53	51	55	62	52	50	56	61	52	43	43	54	65	68	59	55	63	50	46	42	49	[illegible]

9 MAXIMUM AND MINIMUM TEMPERATURES ARE THE HIGHEST AND LOWEST HOURLY READINGS FOR EACH DAY.

F27

March 18, 1965

American Aggregates Corporation
Brighton, Michigan

Attention: D. D. Steinhiler, Plant Manager

Gentlemen:

Tests have been completed on the sample of coarse aggregate taken at your Green Oak plant, Pit No. 47-3 on March 4, 1965 to determine compliance with the requirements of the Supplemental Specifications for 31AA and 31A Aggregate Special coarse aggregates.

The results of the total carbonate content determination indicates that the material, as represented by the sample submitted, meets the specification carbonate content requirement for 31AA and 31A Aggregate Special.

Nothing in this letter should be construed as acceptance of any material produced from this source. Each shipment must be inspected for compliance with the specification requirements.

Very truly yours,

OFFICE OF TESTING AND RESEARCH

J. C. Brehler
Engineer of Materials
Field Testing Division

JCB:pc

cc: J. E. Meyer
C. B. Laird
C. J. Olsen
J. A. DeKamp

F28

(Coarse Aggregate)

MICHIGAN
STATE HIGHWAY DEPARTMENT
OFFICE OF TESTING AND RESEARCH
TESTING LABORATORY DIVISION
UNIVERSITY OF MICHIGAN
ANN ARBOR

REPORT OF TEST
Sheet 1 of 2

Project: General
Laboratory No.: 65A-786
Date: March 15, 1965

Report on sample of: COARSE AGGREGATE (Gravel)
Date sampled: March 4, 1965 Date received: March 4, 1965
Source of Material: American Aggregates Corporation (Green Oak) Pit No. 47-3
Sampled from: Source Quantity represented: --
Submitted by: D. Pennington, Materials Inspector
Intended use: Bituminous concrete aggregate Specification: 1963 R&B Supp.

TEST RESULTS

Laboratory Number		
Passing, per cent by weight:		
3 inch sieve		
2½ inch sieve		
2 inch sieve		
1½ inch sieve		
1 inch sieve		
¾ inch sieve		
½ inch sieve		
⅜ inch sieve	100	
No. 4 sieve	47	
No. 10 sieve	8.3	
No. 40 sieve		
No. 200 sieve		
Loss by Washing, per cent	1.5	
Liquid Limit		
Plasticity Index		
Specific Gravity, dry		
Absorption, per cent		

1. Soft, xxxxxxxxxxxxxxxxxxxxxx Particles, per cent	1.3	
2. Chert Particles, per cent		
3. Hard Absorbent Particles, per cent	1.2	
4. Sum of 1, 2, and 3, per cent		
5. Coke, per cent		
6. Iron, per cent		
7.		
8.		
9. Thin or Elongated Pieces, per cent	8.6	
10. Incrusted Particles (greater than ⅓ surface area), per cent		
11. Incrusted Particles (⅓ surface area or less), per cent		
12. Crushed Material in Sample, per cent	100	
13. Deval Abrasion, per cent of wear		
Crushed Material in Abrasion, per cent		
Soft and Non-durable in Abrasion, per cent		
14. Los Angeles Abrasion, per cent of wear		
15. Gravel in Bank Run Material, per cent		

REMARKS:

See Sheet 2.

cc:
File
J.C.Brehler
P.Serafin
G.Gallup (3)
J.DeKamp
A.H.Lawrence

CF

E29

Signed W.W. McLaughlin
Testing and Research Engineer

MICHIGAN
STATE HIGHWAY DEPARTMENT

OFFICE OF TESTING AND RESEARCH
TESTING LABORATORY DIVISION

UNIVERSITY OF MICHIGAN
ANN ARBOR

REPORT OF TEST

Sheet 2 of 2

Project General

Laboratory No. 65A-786

Date March 15, 1965

Report on sample of COARSE AGGREGATE (Gravel)

Date sampled ______ Date received ______

Source of material ______

Sampled from ______ Quantity represented ______

Submitted by ______

Intended use ______ Specification ______

TEST RESULTS

Carbonate Content
(Per Cent by Weight)

Limestone	12.8
Dolomitic Limestone	1.4
Dolomite	20.7
Total Carbonate Content	34.9

REMARKS: Tested for information

CF

F30

Signed W. W. McLaughlin
Testing and Research Engineer

PETROGRAPHIC ANALYSIS OF COARSE AGGREGATE
AMERICAN AGGREGATE CORP. (GREEN OAK) PIT NO. 47-3
LABORATORY NO. 65A-786

SUMMARY REPORT

Sample:

On March 4, 1965 a sample of coarse aggregate was received in the Laboratory. Information accompany the sample stated that the material was obtained by D. Pennington on March 4, 1965 from the American Aggregate Corp. (Green Oak) Pit No. 47-3, located approximately 4 miles east and 2 miles south of Brighton, Livingston County. The material used in this analysis was a portion of the sample submitted to the Laboratory for a carbonate rock content determination.

Summary:

Petrographic examination has been completed on the coarse aggregate. The sample is composed of approximately 26 per cent igneous rock, 26 per cent metamorphic rock and 48 per cent sedimentary rock particles. The igneous and metamorphic rock particles are hard, fresh and dense. The sedimentary rock particles vary from very hard to soft, from fresh to moderately weathered and vary from slightly porous to very finely porous.

The gradation of the material, as determined by the Aggregate Unit, is as follows:

Sieve Size	Amount Passing (Per cent by weight)	Amount Retained (Per cent by weight)
3/8	100	0
No. 4	47	53
No. 10	8.3	38.7

The composition of this material, tabulated on the attached sheets, has been computed on the basis that 100 per cent of the sample is coarser than the No. 10 sieve. Material finer than the No. 10 sieve has not been examined.

DETAILED PETROGRAPHY

Test Procedures:

Representative portions - 500 particles - of each sieve fraction of the coarse aggregate were examined and identified megascopically by acid testing and a scratch test for hardness and microscopically with a stereomicroscope.

Composition:

Sedimentary rock particles constitute the largest fraction of the coarse aggregate with igneous and metamorphic rock particles being present in approximately equal amounts. The largest individual constituents are dolomite, quartzite and basic igneous detritus. The igneous and metamorphic rock particles are hard, fresh and dense. The carbonate rock particles of the sedimentary rock fraction vary from moderately hard to soft, vary from fresh to moderately weathered and vary from slightly porous to porous. The sandstone particles vary from moderately soft to soft, are moderately fresh and are porous. The siltstone particles are soft, moderately weathered and porous. The chert and cherty limestone particles are very hard, fresh and very finely porous.

Igneous Rocks:

Due to the small size of the material, valid separation of the igneous rocks into individual categories is not possible. The igneous rock particles have been separated into the following groups: acidic igneous detritus - particles derived from granites; intermediate igneous detritus - particles derived from diorites; and basic igneous detritus - particles derived from gabbros and basalts. The pink, buff, mottled white and pink, the mottled pink or red and black, the mottled white, pink and black colored acidic igneous detritus particles are hard, fresh, dense and vary from medium to fine grained These particles are angular in shape with dull to subvitreous lustered, ridged surfaces. The mottled buff, white and black and the mottled white and black colored intermediate igneous detritus particles are hard, fresh, dense and medium to very fine grained. These particles are angular in shape with dull to subvitreous lustered, ridged surfaces. The mottled whitish and dark green, the black and green and the medium to very dark green colored basic igneous detritus particles are hard, fresh, dense and fine grained to microcrystalline textured. These particles are angular in shape with dull, ridged surfaces.

Metamorphic Rocks:

Quartzite particles are one of the largest constituents of the sample. The colorless, white, pink, tan, light green and light gray colored quartzite particles are hard, fresh, dense and are fine grained to microcrystalline textured. These particles are angular in shape with dull to vitreous lustered, ridged surfaces. The light to medium green and reddish-brown colored metasediment particles are hard, fresh, dense and very fine grained to microcrystalline textured. These particles are angular in shape with dull, ridged surfaces. The light to medium green colored tillite particles are hard, fresh, dense and have a porphyritic type texture. The tillite particles are angular in shape with dull, ridged surfaces.

Sedimentary Rocks:

The whitish, tan, light gray and light brown colored limestone particles are moderately soft, fresh, slightly porous and very fine grained to microcrystalline textured. These particles are angular in shape with dull ridged surfaces. A few limestone particles are fossiliferous. The tan and light brown colored dolomitic limestone particles are soft, moderately weathered, porous and very fine grained to microcrystalline textured. These particles are angular in shape with dull, ridged surfaces. The whitish, buff, tan, light brown and light gray colored dolomite particles vary from moderately hard to moderately soft, are fresh, vary from slightly porous to porous and are fine to very fine grained. The dolomite particles are angular in shape with dull, ridged surfaces. The light gray and yellowish-brown colored sandstone particles vary from moderately soft to soft, are moderately fresh, porous and are fine to very fine grained. The particles are subangular in shape with dull, rough, dented surfaces. The yellowish-brown colored siltstone particles are soft, moderately weathered, porous and are very fine grained to microcrystalline textured. These particles are subangular in shape with dull, rough, dented surfaces. The whitish, tan, buff, and light to medium brown colored chert and cherty limestone particles are very hard, fresh, very finely porous and microcrystalline textured. The particles are angular in shape with dull to subvitreous lustered, ridged surfaces.

G. H. Gallup

G. H. Gallup
Geologist IIA

GHG:pl
102565
Attachments

TABLE I

CALCULATIONS OF RESULTS OF PARTICLE COUNTS

Constituents	Composition of Fractions Retained on Sieves Shown Below			
	No. 4		No. 10	
	Number of Particles	Per Cent	Number of Particles	Per Cent
Acidic Igneous Detritus	43	8.6	48	9.6
Intermediate Igneous Detritus	8	1.6	19	3.8
Basic Igneous Detritus	68	13.6	71	14.2
Quartzite	97	19.4	81	16.2
Metasediments	33	6.6	21	4.2
Tillite	17	3.4	9	1.8
Limestone	56	11.2	68	13.6
Dolomitic Limestone	5	1.0	5	1.0
Dolomite	131	26.2	138	27.6
Sandstone	14	2.8	12	2.4
Siltstone	3	0.6	--	--
Chert	14	2.8	22	4.4
Cherty Limestone	11	2.2	6	1.2
Totals	500	100.0	500	100.0
Gradation-Per Cent Retained on Individual Sieves		57.8		42.2

	Weighted Percentages of Constituents in Each Sieve Fraction		Weighted Composition of Sample
	No. 4	No. 10	
Acidic Igneous Detritus	5.0	4.1	9.1
Intermediate Igneous Detritus	0.9	1.6	2.5
Basic Igneous Detritus	7.9	6.0	13.9
Quartzite	11.2	6.8	18.0
Metasediments	3.8	1.8	5.6
Tillite	2.0	0.8	2.8
Limestone	6.5	5.7	12.2
Dolomitic Limestone	0.6	0.4	1.0
Dolomite	15.1	11.6	26.7
Sandstone	1.6	1.0	2.6
Siltstone	0.3	--	0.3
Chert	1.6	1.9	3.5
Cherty Limestone	1.3	0.5	1.8
Total in Sieve Fraction	57.8	42.2	100.0

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TABLE II

COMPOSITION OF SAMPLE

Constituents	Amount, as number of particles in per cent		
	In Fractions Retained on Sieves Shown Below[a]		In Whole Sample[b]
	No. 4	No.10	Total
Acidic Igneous Detritus	8.6	9.6	9.1
Intermediate Igneous Detritus	1.6	3.8	2.5
Basic Igneous Detritus	13.6	14.2	13.9
Quartzite	19.4	16.2	18.0
Metasediments	6.6	4.2	5.6
Tillite	3.4	1.8	2.8
Limestone	11.2	13.6	12.2
Dolomitic Limestone	1.0	1.0	1.0
Dolomite	26.2	27.6	26.7
Sandstone	2.8	2.4	2.6
Siltstone	0.6	--	0.3
Chert	2.8	4.4	3.5
Cherty Limestone	2.2	1.2	1.8
Totals	100.0	100.0	100.0

(a) Based on count of 500 particles in each sieve fraction.

(b) Based on gradation of sample as received disregarding portion passing No. 10 sieve and the distribution of constituents by sieve fractions shown at left.

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OFFICE MEMORANDUM

MICHIGAN
STATE HIGHWAY DEPARTMENT
JOHN C. MACKIE, COMMISSIONER

July 20, 1964

To: Paul J. Serafin, Bituminous Engineer
Testing Laboratory Division

From: G. H. Gallup

Subject: Petrographic Analyses of Bituminous Concrete Coarse Aggregates

Attached are petrographic analysis reports on coarse aggregates from the American Aggregate Corp. (Green Oak) Pit No. 47-3 submitted to the writer on April 4, 1964.

The materials utilized for petrographic analyses were portions of samples submitted to the National Crushed Stone Association.

OFFICE OF TESTING AND RESEARCH

G. H. Gallup

G. H. Gallup, Geologist
Testing Laboratory Division

GHG:mm

Attachments

cc: C.J. Olsen
R.H. Vogler

OFFICE MEMORANDUM

MICHIGAN
STATE HIGHWAY DEPARTMENT
JOHN C. MACKIE, COMMISSIONER

July 24, 1964

To: W. W. McLaughlin, Testing and Research Engineer
Office of Testing and Research

From: C. J. Olsen

Subject: Petrographic Analyses of Bituminous Concrete Coarse Aggregates

Attached are petrographic analyses of coarse aggregates from the American Aggregate Corp. (Green Oak) Pit No. 47-3, prepared by G.H. Gallup, Geologist.

OFFICE OF TESTING AND RESEARCH

C. J. Olsen, Director
Testing Laboratory Division

CJO:mm

Attachments

cc: E.A. Finney
J.C. Brehler

M.S.H.D.
OFFICE OF
TESTING AND RESEARCH
JUL 27 1964
RECEIVED
ADMINISTRATIVE UNIT

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F37

PETROGRAPHIC ANALYSIS OF COARSE AGGREGATE 31A
AMERICAN AGGREGATE CORP. (GREEN OAK) PIT NO. 47-3
LABORATORY NO. 64A-2196

SUMMARY REPORT

Sample:

On April 4, 1964, a sample of crushed natural gravel was obtained from Mr. Paul J. Serafin. Information accompanying the sample stated that the material was obtained from the American Aggregate Corp. (Green Oak) Pit No. 47-3, produced to meet current specification requirements for Coarse Aggregate 31A for use on a cooperative research program by the National Crushed Stone Association.

Summary:

Petrographic examination has been made on the crushed natural gravel of 3/8-inch maximum size. The sample is composed of approximately 25 per cent igneous, 34 per cent metamorphic and 41 per cent sedimentary rock particles. All rock particles, with the exception of the sandstones, some cherts, and shales, appear to be fresh and dense.

The gradation of the material as reported by the Aggregate Unit is as follows:

Sieve Size	Amount Passing (Per cent by weight)	Amount Retained (Per cent by weight)
3/8	100	0
No. 4	61	39
No. 10	12.6	48.4
Loss by Washing	1.5	

The composition of this material, tabulated on the attached sheets, has been computed on the basis that 100 per cent of the sample is coarser than the No. 10 sieve. Material in the sample finer than this size has not been analyzed.

DETAILED PETROGRAPHY

Test Procedure:

Representative portions - 500 particles - of each sieve fraction of the crushed natural gravel were examined and identified megascopically by acid testing and a scratch test for hardness and microscopically with a petrographic microscope and a stereomicroscope.

Composition:

Sedimentary rocks constitute the largest fraction of the crushed natural gravel, metamorphic rocks the next largest constituent and igneous rocks the smallest constituent. The largest individual constituents of the whole sample are quartzite, dolomite and limestone. Nearly all the igneous and metamorphic rock particles are dense, tough and predominantly unweathered. The carbonate rock constituents of the sedimentary fraction appear fresh and moderately dense on fractured faces, but the sandstone particles are predominantly soft, porous, and weakly bonded. The composition of the gravel is shown on the attached tables and the constituents are described below.

Igneous Rocks:

Igneous rocks present are granites, granodiorites, diorites, gabbros, basalts and felsites. The acidic and intermediate igneous rocks - granites, granodiorites and diorites - are dense, tough and unweathered. Colors range from pink to red and whitish to dark gray with a few buff colored particles. Granitoid textures are predominant in these rocks in both macro- and microcrystalline grain size particles. A few particles have gneissic (banded) textures. The basic igneous rocks are gabbros and basalts. The gabbros are medium to dark mottled green, fine grained, dense and tough. Some unfractured faces are slightly to moderately weathered. The basalts are dark green to black, very fine grained to microcrystalline textured, dense and tough. A few basalt particles are amygdaloidal. The remaining igneous rock constituents are the light to medium green and gray, dense, tough and microcrystalline to porphyritic textured felsites and small amount of acidic and intermediate igneous rock detritus.

Metamorphic Rocks:

Quartzites of various colors are the dominant metamorphic rock type present and constitutes the largest single rock type in the sample. All of these particles are unweathered, tough, dense and have fine grained or microcrystalline textures. The metasediments and tillites present are dense, tough, unweathered, green and gray with fine grained to microcrystalline and porphyritic type textures.

Sedimentary Rocks:

A large percentage of the sedimentary rock fraction is composed of carbonate rocks. The limestones and dolomites are similar in most physical characteristics such as: the whitish, buff and light to medium gray color; the fresh appearance on fractured faces and the slight to moderate weathering on unfractured faces; moderate hardness and porosity; and the fine grain to microcrystalline textures. The dolomites are predominantly light to dark gray, slightly harder than and have larger grain sizes than the other carbonate rocks. Chert particles found in this sample are whitish and light to dark gray, dull to vitreous lustered, dense to very porous, hard and unweathered. Many of these cherts contain enough calcium carbonate to be considered as cherty limestones but for convenience are classed strictly as chert. The sandstone particles are predominantly soft to moderately soft, porous, fine grained, whitish to buff and light gray in color and cemented predominantly with calcium carbonate cement. Shale is a minor constituent of the sample and is dark gray, very fine grained, soft, porous and moderately weathered.

George H. Gallup

George H. Gallup,
Geologist II

071664
GHG:mm

TABLE I

CALCULATIONS OF RESULTS OF PARTICLE COUNTS

Constituents	Composition of Fractions Retained on Sieves Shown Below			
	No. 4		No. 10	
	Number of Particles	Per Cent	Number of Particles	Per Cent
Granite	19	3.8	--	--
Granodiorite	9	1.8	--	--
Diorites	13	2.6	4	0.8
Gabbro	15	3.0	18	3.6
Basalt	41	8.2	47	9.4
Felsites	16	3.2	16	3.2
Acid-Igneous Detritus	10	2.0	16	3.2
Intermediate Igneous Detritus	5	1.0	20	4.0
Quartzite	112	22.4	117	23.4
Metasediments	38	7.6	45	9.0
Tillite	8	1.6	14	2.8
Limestone	75	15.0	77	15.4
Dolomitic Limestone	27	5.4	17	3.4
Dolomite	75	15.0	86	17.2
Sandstone	27	5.4	17	3.4
Chert	10	2.0	4	0.8
Shale	--	--	2	0.4
Totals	500	100.0	500	100.0
Individual Per Cent Retained on Sieve		44.7		55.3

	Weighted Percentages of Constituents in Each Sieve Fraction		Weighted Composition of Sample
	No. 4	No. 10	
Granite	1.7	--	1.7
Granodiorite	0.8	--	0.8
Diorites	1.2	0.4	1.6
Gabbro	1.3	2.0	3.3
Basalt	3.7	5.2	8.9
Felsites	1.4	1.8	3.2
Acid-Igneous Detritus	0.9	1.8	2.7
Intermediate Igneous Detritus	0.5	2.2	2.7
Quartzite	10.0	12.9	22.9
Metasediments	3.4	5.1	8.5
Tillite	0.7	1.5	2.2
Limestone	6.7	8.5	15.2
Dolomitic Limestone	2.4	1.9	4.3
Dolomite	6.7	9.5	16.2
Sandstone	2.4	1.9	4.3
Chert	0.9	0.4	1.3
Shale		0.2	0.2
Total in Sieve Fraction	44.7	55.3	100.0

TABLE II

COMPOSITION OF SAMPLE

Constituents	Amount, as number of particles in per cent		
	In Fractions Retained On Sieves Shown Below[a]		In Whole Sample[b]
	No. 4	No. 10	Total
Granite	3.8	--	1.7
Grandiorite	1.8	--	0.8
Diorite	2.6	0.8	1.6
Gabbro	3.0	3.6	3.3
Basalt	8.2	9.4	8.9
Felsites	3.2	3.2	3.2
Acid Igneous Detritus	2.0	3.2	2.7
Intermediate Igneous Detritus	1.0	4.0	2.7
Quartzite	22.4	23.4	22.9
Metasediments	7.6	9.0	8.5
Tillite	1.6	2.8	2.2
Limestone	15.0	15.4	15.2
Dolomitic Limestone	5.4	3.4	4.3
Dolomite	15.0	17.2	16.2
Sandstone	5.4	3.4	4.3
Chert	2.0	0.8	1.3
Shale	--	0.4	0.2
Totals	100.0	100.0	100.0

(a) Based on count of 500 particles in each sieve fraction.

(b) Based on gradation of sample as received disregarding portion passing No. 10 sieve and the distribution of constituents by sieve fractions shown at left.

PETROGRAPHIC ANALYSIS OF COARSE AGGREGATE 25A
AMERICAN AGGREGATE CORP. (GREEN OAK) PIT NO. 47-3
LABORATORY NO. 64A-2197

SUMMARY REPORT

Sample:

On April 4, 1964, a sample of crushed natural gravel was obtained from Mr. Paul J. Serafin. Information accompanying the sample stated that the material was obtained from the American Aggregate Corp. (Green Oak) Pit No. 47-3, produced to meet current specification requirements for Coarse Aggregate 25A for use on a cooperative research program by the National Crushed Stone Association.

Summary:

Petrographic examination has been made on the crushed natural gravel of 5/8-inch maximum size. The sample is composed of approximately 29 per cent igneous, 31 per cent metamorphic and 40 per cent sedimentary rock particles. All rock types, with the exception of sandstone and shale particles, appear generally to be fresh and dense. Uncrushed faces of some basic igneous rock types, predominantly gabbros, exhibit slight to moderate weathering.

The gradation of the material as reported by the Aggregate Unit is as follows:

Sieve Size	Amount Passing (Per cent by weight)	Amount Retained (Per cent by weight)
5/8	100	0
1/2	95	5
3/8	59	36
No. 4	20	39
No. 10	7.4	12.6
Loss by Washing	1.1	

The composition of this material, tabulated on the attached sheets, has been computed on the basis that 100 per cent of the sample is coarser than the No. 10 sieve. Material finer than this size has not been analyzed.

DETAILED PETROGRAPHY

Test Procedure:

Representative portions - 500 particles - of each sieve fraction of the crushed natural gravel were examined and identified megascopically by acid testing and a scratch test for hardness and microscopically with a petrographic microscope and a stereomicroscope.

Composition:

The material is a crushed natural gravel composed of nearly equal portions of igneous, metamorphic and sedimentary rock types. The largest individual constituents are quartzite, limestone and dolomite. Nearly all the igneous and metamorphic rock particles are dense, tough and predominantly unweathered. The carbonate rock constituents of the sedimentary fraction appear fresh and moderately dense on fractured faces but the sandstone particles are predominantly soft, porous and weakly bonded. The composition of the gravel is shown on the attached tables and the constituents are described below.

Igneous Rocks:

Igneous rocks present are granites, granodiorites, diorites, gabbros, basalts and felsites. The acidic and intermediate igneous rocks - granites, granodiorites and diorites - are dense, tough and unweathered. Colors range from pink to red and whitish to dark gray with a few buff colored particles. Granitoid textures are predominant in these rocks in both macro- and microcrystalline grain size particles. A few particles have gneissic (banded) textures. The basic igneous rocks are gabbro and basalts in nearly equal proportions. The gabbros are medium to dark, mottled green, fine grained, dense, and tough. Some unfractured faces of these particles are slightly to moderately weathered. The basalts are dark green to black, very fine grained to microcrystalline textured, dense and tough. A few basalts are amygdaloidal. The remaining igneous rock constituents are the light to medium green and gray, dense, tough, microcrystalline to porphyritic textured felsites and a minor amount of acidic and intermediate igneous rock detritus.

Metamorphic Rocks:

Quartzites of various colors are the dominant metamorphic rock particles present and constitute the largest rock type present in the sample. All these particles are unweathered, dense and tough and have fine grained or microcrystalline textures. The metasediments and tillites present are dense, tough, unweathered, green and gray with fine grained to microcrystalline and porphyritic type textures.

Sedimentary Rocks:

A large percentage of the sedimentary rocks present is composed of limestone, dolomitic limestone and dolomite. The limestones and dolomitic limestones are similar in most physical characteristics such as: the whitish, buff and light to medium gray color; the fresh appearance on fractured faces and the slight to moderate weathering on unfractured faces; moderate hardness and porosity; and the fine grain to microcrystalline textures. The dolomites are predominantly light to dark gray, slightly harder than and have larger grain sizes than the other carbonate rocks. Uncrushed surfaces on the carbonate rock fraction appear slightly to moderately weathered. Chert particles found in this sample are whitish and light to dark gray, dull to vitreous lustered, dense to very porous, hard and unweathered. Many of these cherts contain enough calcium carbonate to be considered as cherty limestones but for convenience are classed strictly as chert. The sandstone particles are predominantly soft to moderately soft, porous, fine grained, whitish to buff and light gray in color and cemented predominantly with calcium carbonate cement. Shale is a minor constituent of the sample and is dark gray, very fine grained, soft, porous and moderately weathered.

George H. Gallup

George H. Gallup,
Geologist II

071764
GHG:mm

TABLE I

CALCULATIONS OF RESULTS OF PARTICLE COUNTS

Constituents	Composition of Fractions Retained on Sieves Shown Below							
	1/2-in.		3/8-in.		No. 4		No. 10	
	Number of Particles	Per Cent	Number of Particles	Per Cent	Number of Particles	Per Cent	Number of Particles	Per Cent
Granite	38	7.6	39	7.8	28	5.6	6	1.2
Granodiorite	17	3.4	11	2.2	6	1.2	--	--
Diorites	24	4.8	23	4.6	25	5.0	4	0.8
Gabbro	23	4.6	37	7.4	31	6.2	23	4.6
Basalt	35	7.0	36	7.2	24	4.8	27	5.4
Felsites	23	4.6	21	4.2	15	3.0	6	1.2
Acid-Igneous Detritus	--	--	--	--	--	--	28	5.6
Intermediate Igneous Detritus	--	--	--	--	--	--	15	3.0
Quartzite	108	21.6	90	18.0	117	23.4	145	29.0
Metasediments	44	8.8	32	6.4	23	4.6	24	4.8
Tillite	20	4.0	25	5.0	11	2.2	9	1.8
Limestone	60	12.0	63	12.6	83	16.6	70	14.0
Dolomitic Limestone	32	6.4	38	7.6	40	8.0	42	8.4
Dolomite	61	12.2	63	12.6	64	12.8	46	9.2
Sandstone	10	2.0	12	2.4	21	4.2	34	6.8
Chert	5	1.0	10	2.0	10	2.0	16	3.2
Shale	--	--	--	--	2	0.4	5	1.0
Totals	500	100.0	500	100.0	500	100.0	500	100.0
Individual Per Cent Retained on Sieve		5.4		38.9		42.1		13.6

Constituents	Weighted Percentages of Constituents in Each Sieve Fraction				Weighted Composition of Sample
	1/2-in.	3/8-in.	No. 4	No. 10	
Granite	0.4	3.0	2.4	0.2	6.0
Granodiorite	0.2	0.9	0.5	--	1.6
Diorite	0.3	1.8	2.1	0.1	4.3
Gabbro	0.2	2.9	2.6	0.6	6.3
Basalt	0.4	2.8	2.0	0.7	5.9
Felsites	0.2	1.6	1.3	0.2	3.3
Acid Igneous Detritus	--	--	--	0.8	0.8
Intermediate Igneous Detritus	--	--	--	0.4	0.4
Quartzite	1.2	7.0	9.9	4.0	22.1
Metasediments	0.5	2.5	1.9	0.7	5.6
Tillite	0.2	1.9	0.9	0.2	3.2
Limestone	0.6	4.9	7.0	1.9	14.4
Dolomitic Limestone	0.3	3.0	3.4	1.1	7.8
Dolomite	0.7	4.9	5.4	1.3	12.3
Sandstone	0.1	0.9	1.7	0.9	3.6
Chert	0.1	0.8	0.8	0.4	2.1
Shale	--	--	0.2	0.1	0.3
Total in Sieve Fraction	5.4	38.9	42.1	13.6	100.0

TABLE II

COMPOSITION OF SAMPLE

Constituents	Amount, as number of particles in per cent				
	In Fraction Retained On Sieves Shown Below[a]				In Whole Sample[b]
	1/2-in.	3/8-in.	No. 4	No. 10	Total
Granite	7.6	7.8	5.6	1.2	6.0
Granodiorite	3.4	2.2	1.2	--	1.6
Diorite	4.8	4.6	5.0	0.8	4.3
Gabbro	4.6	7.4	6.2	4.6	6.3
Basalt	7.0	7.2	4.8	5.4	5.9
Felsites	4.6	4.2	3.0	1.2	3.3
Acid Igneous Detritus	--	--	--	5.6	0.8
Intermediate Igneous Detritus	--	--	--	3.0	0.4
Quartzite	21.6	18.0	23.4	29.0	22.1
Metasediments	8.8	6.4	4.6	4.8	5.6
Tillite	4.0	5.0	2.2	1.8	3.2
Limestone	12.0	12.6	16.6	14.0	14.4
Dolomitic Limestone	6.4	7.6	8.0	8.4	7.8
Dolomite	12.2	12.6	12.8	9.2	12.3
Sandstone	2.0	2.4	4.2	6.8	3.6
Chert	1.0	2.0	2.0	3.2	2.1
Shale	--	--	0.4	1.0	0.3
Totals	100.0	100.0	100.0	100.0	100.0

(a) Based on count of 500 particles in each sieve fraction.

(b) Based on gradation of sample as received disregarding portion passing No. 10 sieve and the distribution of constituents by sieve fractions shown at left.

Appendix G.

Literature Review

This appendix includes the literature review as revised on June 7, 1996 in response to MDOT 's comments.

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

Literature Review

Submitted to

Michigan Department of Transportation

By

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CAUSE OF CONCRETE PIER CAP DETERIORATION ON THE I-75 BRIDGE OVER RIVER ROUGE IN DETROIT, AND EFFECTIVENESS OF REPAIR METHODS

Revised Literature Review

1. INTRODUCTION

1.1 Background

The I-75 bridge over the Rouge River in Detroit was built in 1965 with an HS20 design live load. Its main span is a high level, 3 span, continuous, steel girder bridge with a composite concrete deck. The substructure is entirely reinforced concrete. The adjacent spans are also steel girder spans with composite concrete decks.

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 were not. These deck joints above piers 37 and 40 have allowed water and chlorides from the bridge deck above to enter the pier cap concrete. These two piers are more severely deteriorated than are caps 38 and 39. Cores taken from piers caps 37 and 40 show high chloride content, but adequate compressive strength. Petrographic examination indicated the presence of alkali silica reactivity (ASR) associated with the fine aggregates. It was concluded that concrete overstressing and chloride damage led to the deterioration of caps 37 and 40. Pier caps 38 and 39 show deterioration similar to that in caps 37 and 40, though to a lesser extent. Petrographically examined fragments from these piers yielded similar findings to the cores from caps 37 and 40.

A repair method was established, including epoxy injection of all cracks, concrete chipping and replacement, 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. The remaining caps were injected and surface treated as planned. The cracking proved so extensive though, that contract funds ran out before much of the work could be completed.

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. The proposed plan of injecting and sealing pier caps 38 and 39, along with post tensioning is currently under investigation.

In this project, mainly the pier caps are considered. Variation in the paste color of the concrete surfaces appear to indicate that several mix designs were used for construction of the piers. In general, a relatively buff colored paste appeared to be present at most columns, while a relatively gray paste appeared to be present at most pier caps. Cracking is much more severe in the pier caps (where 7.5 sacks of cement per cubic yard of concrete were used) than in the columns (where 6 sacks of cement per cubic yard of concrete were used). To some extent, the reduced cracking in the columns may also be associated with the longitudinal confinement due to loading in the columns.

Cracking in the columns is variable. In general a relatively fine crack is observed in the columns, approximately centered in the width of the column, and associated with the buff colored concrete. Some more severe cracking is evidenced in the columns, for instance at the top of one column in Pier 38 north-bound, where the paste color is gray. Extensive pattern cracking is observed in all pier caps of Piers 37 to 40.

1.2 Objective

The objective of this project is to determine the extent of deterioration in pier caps 38 and 39, to determine the impact of ASR on the structure, and to determine the feasibility of the proposed repair methods. In this investigation, specific emphasis is placed on the role of the use of blast-furnace slag as coarse aggregate. Specifically, the effects of slag on ASR and long-term durability are being investigated. The creep properties of ASR affected concrete are of importance in evaluating the proposed post-tensioning. This literature review summarizes the state of the art information on topics related specifically to the needs of this project.

2. BLAST-FURNACE SLAG AS COARSE AGGREGATE

Blast-furnace slag is a byproduct of iron production. It is formed when iron ore (a mixture of iron oxides, silica, and alumina) is reduced through chemical reactions to iron. The silica and alumina compounds combine with calcium, added to the furnace in the form of limestone or dolomite, to form the slag. The properties of slag are highly dependent on its chemical composition, as well as the conditions under which it is cooled. There are three basic types of slag; air cooled slag, foamed or expanded slag, and granulated slag. Air cooled slag develops a crystalline structure similar to natural igneous rocks. This type of slag produces a strong aggregate frequently used in concrete. Foamed slag is produced by injecting water into the molten slag. The occluded gasses and steam produced in the process form the expanded light-weight structure of the slag. The result is a strong light-weight aggregate suitable for concrete applications. Finally, granulated slag is a rapidly cooled slag, where crystals are not allowed to form. This type of slag is often ground and used as an additive to portland cement, as it has hydraulic setting properties. (Lee, 1974)

Though the use of slag as aggregate in concrete has achieved wide acceptance, the use of slag is not without inherent difficulties, and should be monitored closely. Foamed slags, used as lightweight aggregate, tend to have highly angular particle shapes, which can have harmful effects on workability, finishing, and bleeding of the concrete. The use of special mix designs containing increased air entrainment, increased cement content, or the use of mineral admixtures can help protect against these effects. The typically high absorption capacity of slags requires careful monitoring during mixing and proportioning to ensure a proper consistency concrete. Finally, segregation is possible with foamed slag aggregates, as excessive vibration or improper placing practice may cause the slag to rise upward in the concrete. In general, the same precautions that would be taken with any lightweight aggregate should be taken with foamed slag. (Popovics, 1979)

The chemical composition of blast-furnace slag may be somewhat variable. Typically, though, the composition falls into a fairly well defined range, as shown in Table #1.

Chemical Constituent	% by Mass
Lime (CaO)	36-43%
Silica (SiO_2)	28-36%
Alumina (Al_2O3)	12-22%
Magnesia (MgO)	4-11%
Total Sulfur (as S)	1-2%
Total Iron (as FeO or Fe_2O_3)	0.3-1.7%

***Table #1** Chemical constituents of blast-furnace slag (by mass). (After Lee, 1974)*

Concrete made with slag aggregate is often more permeable than standard concretes, allowing a greater infiltration of chlorides and other pollutants into the concrete. It can be noted from Table #1, though, that slag contains a certain percentage of Fe_2O_3, which is the same oxide that forms as a passivating layer on reinforcing steel. In this manner, the use of slag as aggregate can actually inhibit the corrosion of reinforcing steel in the concrete, even in the presence of high chloride concentrations. (Lee, 1974)

The high silica content of blast-furnace slags makes them potentially susceptible to ASR reactivity. Just as highly siliceous volcanic glasses have been found to be reactive, blast-furnace slag may also be reactive, requiring special testing and acceptance criteria, as described later in this report.

As was discussed earlier, the properties of slag can be greatly affected by the method of cooling. During slag formation, dicalcium silicate compounds can occur in at least three different crystalline forms. When the slag is cooled slowly, to form a dense aggregate, a phase change in the dicalcium silicate, which can occur at atmospheric temperatures, can lead to harmful expansive volume change. This unstable dicalcium silicate on its own, or combined with slag's alkaline reactivity may lead to harmful deterioration of the concrete. It should be noted that the unstable phase of dicalcium silicate does not form when the slag is cooled rapidly, as in foamed slag and granulated slag. Furthermore, it should be noted that a careful monitoring of slag composition can avoid the harmful properties described here. (Lee, 1974)

In fact, extensive research has shown that ground granulated blast-furnace slag is beneficial in concrete for many of the same reasons slag as coarse aggregate may be harmful. For example, ground granulated blast-furnace slag may increase workability, reduce bleeding and segregation, and reduce concrete permeability. Furthermore, when ground into a fine powder, the siliceous slag reacts so rapidly with the hydroxide ions in the cement that harmful levels of hydroxide ions are not allowed to build up, and the expansive ASR reaction may never occur. (Ozyildirim, 1990)

The reduction in permeability is brought forth by the fineness of the slag particles. In addition, the slag acts as an additional binding agent in the paste as it has some cementitious value. The reduced permeability in turn inhibits the ingress of external water and alkalis, which could accelerate or induce ASR. If there was to be any reaction of the slag, its uniform distribution through the system would prevent the buildup of alkali-silica gel rich zones where damage from expansion would be likely to occur. In aggregates, the harmful expansion is typically associated with gel-rich zones surrounding and penetrating the aggregate, also known as the reaction rim, and cracks emanating outward from reactive particles. Petrography has shown that the slag aggregate from this bridge shows no evidence of reactivity.

3. LONG-TERM DURABILITY AND CREEP PROPERTIES OF ASR AFFECTED STRUCTURES

3.1 Creep and Drying Shrinkage

The effects of creep in structures are complicated by drying shrinkage that occurs simultaneously. During the early stages of drying of a concrete member, the center remains moist as the outside edges dry. This puts the center of the member in compression, and the edges in tension as they try to shrink from moisture loss. The occurrence of ASR, which takes place in the moist center, further increases the compression in the center and the tension at the surfaces of the member. In this way, the effects of drying shrinkage can be more severe in ASR affected concretes than in nonreactive concretes.

Over the long term, stresses in the concrete are relaxed by creep of the cement paste. The process of creep is complex and has two components, drying creep and basic creep. Basic creep is the amount of creep movement under constant relative humidity conditions. This type of creep is decreased with decreasing relative humidity. Drying creep on the other hand is the creep effect associated with the drying of the specimen. Total creep during drying can be much larger than basic creep. The basic creep component decreases as the specimen dries. The extent of ASR expansion in affected concrete is determined in part by the concrete's creep properties. Because ASR expansion is inhibited by external and internal restraint, any reduction in such restraint can lead to increased expansion. Creep allows for such continued ASR expansion. (Helmuth, 1993)

3.2 Restraint From Expansion

Reacting aggregate particles are restrained from expanding by internal and external forces. The cohesive strength of concrete portions that are not expansive restrain the ASR affected particles. Furthermore, other forces such as the weight of the structure, reinforcing steel, prestressing or post-tensioning can restrain the expansion.

It should be noted that pressure cannot stop the alkali-silica reaction. On the contrary, the solubility of the reactive silica is increased with increasing pressure. Thus, increasing pressure, caused either by expansive swelling of the ASR reaction or by external means, can actually accelerate the further dissolution of reactive silicas. Because the dissolution of silicas produces a negative volume change, this in turn reduces the pressures caused by the expansive gel formation. If an external pressure is applied, this reaction cycle may continue until all available reactive silicas are consumed, assuming the availability of moisture and alkalis. (Helmuth, 1993) It should also be noted that ASR gel can diffuse into the paste under confining pressure.

At the same time, significant changes are evident in the expansion behavior through the application of external loading. Even small loads have been shown to have significant effects on the expansion caused by ASR. This is true both for loads applied initially to the specimen, as well as loads applied after some expansion has occurred. (Helmuth, 1993)

When concrete that is subjected to ASR is unconfined, the resulting distress is often manifested as a random distribution of cracks, typically referred to as map cracking. The map cracking usually consists of a pattern of larger cracks that bound a network of barely visible finer cracks. When the expansive forces of the ASR are restrained, for example by reinforcing steel, the cracks tend to run parallel to the reinforcing steel. In general, the cracks tend to run in the direction in which the restraint is applied. (Building Research, 1982)

3.3 Combined Effects of Chlorides and ASR

Concrete that is affected by ASR, and contains chlorides from de-icing salts, marine environments or other contaminating environments such as heavy industrial applications, suffers from increased susceptibility to expansion in a twofold manner.

First, the inclusion of salts accelerates the ASR reaction and expansion process. When salts enter into the concrete, typically in solution, they can provide a semi-infinite supply of alkali to the reaction (as well as moisture). In this way the reaction is only limited by the available reactive silicas in the aggregate. (Helmuth, 1993)

Second, the presence of ASR has been shown to increase the corrosion rate of embedded steel in the concrete. The occurrence of ASR in NaCl contaminated mortars raises the Cl^-/OH^- ratio in the mortar's pore solutions. In addition, ASR increases the corrosion rate of embedded steel in mortars of a given Cl^-/OH^- ratio. This indicates that increased corrosion rates in mortars containing reactive aggregates can be attributed both to an increase in the Cl^-/OH^- ratio, and to a microstructural change in the mortar, caused by the ASR. It can further be concluded that salt contaminated concretes are at greater risk of chloride induced corrosion of reinforcing steel if the concrete is affected by ASR. (Kawamura, 1989)

4. AVAILABLE REPAIR METHODS

The following is a discussion of available repair methodologies that may be useful, in whole or in part, in accomplishing repairs to the deteriorated piers. Selection and design of an appropriate comprehensive repair approach must account for existing deterioration and structural requirements, as well as anticipated construction costs and longevity of repairs. Some modification of the following discussion will likely occur as additional information becomes available. The final project report will provide a more detailed analysis of prospective repair techniques.

4.1 Post-Tensioning

Research has shown that ASR-related deterioration can be significantly reduced if affected concrete is adequately restrained through inducing confining stresses via application of post-tensioning. Available data indicate that confining stresses on the order of 450 psi in biaxial directions or 300 psi in triaxial directions are effective. Uniaxial restraint would not be considered of significant benefit with respect to reduction of effects of ASR. (Stark, 1993) Post-tensioning can also be effective in increasing pier cap capacity beyond the original design strength, and could possibly be accomplished without shoring of bridge decks. Issues to be considered during design of such repairs would include the following:

1. A comprehensive repair approach would likely require restoration of severely deteriorated concrete in conjunction with post-tensioning.

2. Biaxial or triaxial post-tensioning will be difficult to accomplish. Adequate confinement of ongoing ASR would likely require drilling through the entire thickness of the pier caps in a gridwork pattern for their full length. It is possible that embedded reinforcing steel could be cut or damaged in the process. The joint between north- and south-bound lanes would inhibit post-tensioning along the length of the pier caps, although adequate anchorages could likely be designed, or grouting of this joint and post-tensioning both piers as a single unit may be feasible.

3. Member shortening and post-tensioning losses must be accommodated in the repair design. Pier columns and bridge bearings will resist effects of post-tensioning to some extent.

4. Several sequences of post-tensioning may be required to maintain prestress, since creep losses are expected to be significant within ASR-affected concrete.

5. In general, significantly less cracking was observed in columns than in pier caps. As previously stated, this may be due to differing concrete mixes used in various portions of the structure. However, it may also be related to confinement effects of longitudinal column compression combined with transverse restraint provided by reinforcing steel.

6. In order to adequately restrain the deteriorating concrete from continued expansion, it may be necessary to use steel jacketing in conjunction with post-tensioning. The jacketing could cover all or portions of the exposed pier cap surface, and would provide additional support in critical areas.

4.2 Epoxy Injection

Pressure injection of epoxy resin into cracks in pier caps can be used to restore significant structural capacity to damaged concrete. However, a number of issues must be considered prior to selection of epoxy injection as a repair method. These would include the following:

1. Epoxy injection of cracked surfaces will not stop ASR within affected members.

2. Epoxy injection will likely induce random water-impermeable planes within the pier caps. Hence, water and vapor transmission through injected pier cap concrete may be significantly less than through non-injected concrete. If sources of water are available, this could lead to critical saturation of concrete and increased freeze-thaw- and ASR-related deterioration.

3. Epoxy injection may not be fully effective in restoring the strength of deteriorated concrete. This is due to the likelihood that epoxy will not fully penetrate fine cracks (that may or may not be interconnected with surface cracks), deep cracks, and cracks containing various potential contaminants. Hence, an assessment should be made of the effectiveness of the injection process through confirmation coring and testing, and repairs should be designed accordingly.

4. Epoxy injection may be most effective if used in conjunction with surface sealers. In order for a sealer to be effective, it must be able to prevent ingress of water and salts into the concrete, while allowing water vapors to escape. Trapped water can create vapor pressures sufficient to cause the sealant to spall off. In addition the

sealer should be able to span minor cracks without losing integrity if there is crack movement.

4.3 Partial and Full Pier Cap Replacement

An alternative to crack injection that could be used for repairs to severely cracked concrete would be partial-depth concrete removal and replacement. Typically, ASR-related cracks widen significantly near unconfined concrete surfaces. Partial depth concrete removal could be used to remove severely cracked concrete, facilitate placement of post-tensioning hardware if necessary, and add supplemental reinforcing steel if required for pier cap strengthening. Cross-sectional dimensions of repaired pier caps could also be increased for additional strengthening. If continued ASR is judged likely, post-tensioning could be used for ASR confinement.

In the event that deterioration due to ASR or other factors is very severe, and continued deterioration is likely regardless of what repair method is used, it may be necessary to perform a complete pier cap replacement. Because this option would require a temporary support structure, this is considered the least favored repair method.

5. TEST METHODS

5.1 Tests for Slag to Be Used As Aggregate

ASTM C 33 gives the general provisions for materials to be used as aggregates for concrete. These include the size gradation and allowable content of various deleterious materials for the given exposure conditions. Crushed air-cooled blast-furnace slag is excluded from the abrasion requirements of ASTM C 33. A preliminary estimate of the soundness of the aggregate is to be given according to ASTM C 88. ASTM C 127 and ASTM C 128 describe the methods for determining the specific gravity and absorption of coarse and fine aggregate, respectively. This information is necessary in the proportioning of concrete mixtures.

There are a number of different tests which may be used in evaluating the potential alkali reactivity of an aggregate. This section briefly summarizes several standard test methods for use in qualifying aggregates for use in concrete. The recommended course of action basically follows Portland Cement Association's *Guide Specification for Concrete Subject to Alkali-Silica Reactions* (1995). According to the PCA *Guide Specification*, the best method for evaluating the susceptibility of an aggregate to ASR is its previous performance under field conditions. In order to be able to rely on past performance to predict future performance, the previous conditions of exposure must have been at least as severe as those expected in the proposed structure or pavement, the alkali content and water/cement ratio of the existing concrete at least as high as in the proposed concrete, the dosage of pozzolan(s) in the existing concrete less than in the proposed concrete, the existing concrete at least 15 years old, and the aggregate representative of that used in the existing concrete. In general it is almost impossible to meet all of these requirements; thus one must resort to the use of accelerated laboratory tests. Such tests by their nature are less than satisfactory: they may be biased to give either false positives or false negatives, or they may cause a different mechanism to take place in the test than occurs in the field. The best approach is to use several test methods in concert and employ sound engineering judgment in evaluating the results. The tests discussed below may be performed in any order. Note that even if an aggregate is determined to be reactive, it can still be used in concrete provided appropriate measures are taken to control the reactivity. The reader is referred to the PCA *Guide Specification* for recommendations on control measures and how to evaluate their adequacy.

Certain mineralogical constituents are known to react with alkalis in cement and concrete. ASTM C 295 provides a guide for the petrographic examination of aggregates to determine the quantities of these minerals present. PCA's *Guide Specification* recommends limits on these constituents as described in section 5.2.2. In addition, X-ray fluorescence can provide useful information about the chemistry of the aggregate. Results are reported as oxide contents. Analysis of the available alkalis by ASTM C 114 will indicate the presence of alkalis that may become available over time. ASTM C 114 also gives procedures for determining the contents of sulfate and sulfide sulfur, which can combine with oxygen to form sulfate. Sulfate in aggregates can cause an internal form of sulfate attack. Such sulfide compounds as pyrite (FeS_2) can be expansive under certain conditions. The methods for analyzing for alkalis, sulfate, and sulfide sulfur are all wet chemical methods.

Aggregates should also be evaluated by ASTM C 1260 (formerly P 214) for potential alkali reactivity. It is based on an accelerated test method developed in South Africa. In this test, aggregates to be used as fine aggregates are sieved and recombined to achieve the prescribed grading requirements with a minimum of crushing. Coarse aggregates are crushed to the appropriate sizes. Mortar bars made from the aggregate are exposed to a 1N NaOH solution at 80°C for 14 days. According to the PCA *Guide Specification*, a 14-day expansion exceeding 0.10% indicates a potentially reactive aggregate. Note that this test should not be considered as a "go-no go" test; that is, an aggregate having an expansion of just under 0.10% should not automatically be considered "safe". Oberholster (1993), one of the developers of the South African version of the test, recommends a graded approach, with expansions of more than 0.05% warranting some control measures and expansions exceeding 0.10% warranting more stringent control measures.

Coarse aggregate determined by either ASTM C 295 or ASTM C 1260 to be potentially reactive may be further evaluated by ASTM C 1293 (adopted from CSA-A23.2-14A). In this test method, coarse aggregate is sieved and recombined to a specified grading. Concrete is made from this aggregate and a cement having an alkali content of approximately 1.0%. NaOH is used to bring the total alkali content of the cement to 1.25% as a means of moderately accelerating the test. Concrete prisms are exposed to a 100% R.H. environment at 38°C (100°F) for one year. According to the PCA *Guide*

Specification, a mean expansion greater than 0.04% indicates a potentially reactive aggregate. For additional information, the prisms can be cut open and examined petrographically for signs of incipient ASR. The test can also be continued beyond one year; normal Canadian practice is to continue the test for two years.

In sorting out the results from these three tests, any field data indicating alkali reactivity of the aggregate in service takes precedence over all else: the aggregate is clearly reactive. ASTM C 1260 is generally used as a screening test; if an aggregate does not appear reactive on this test, it should be considered innocuous (taking into account Oberholster's recommendation to take some precautions for aggregates having expansions of 0.05-0.10%). The CSA prism test (ASTM C 1293) is considered to be more definitive. Although it takes longer than ASTM C 1260, it has been extensively correlated with field performance in Canada.

Other test methods include ASTM C 227 and ASTM C 289. ASTM C 227, which is designed primarily to evaluate specific combinations of cement, pozzolan (if used), and aggregate, involves exposing mortar bars to a 100% R.H. environment at 100°F (38°C) for up to 12 months and measuring the expansion at intervals. Thus the time required is as long as for the CSA test, but the result is less definitive because it involves mortar rather than concrete. ASTM C 1260, which is designed specifically to screen for reactive aggregates, provides results in 14 days which are as good as or better than ASTM C 227. Also, the expansion found by ASTM C 227 could be the result of some mechanism(s) other than ASR, while ASTM C 1260 is designed to force ASR.

ASTM C 289 is a wet chemical method for determining the potential alkali reactivity of an aggregate. In this test the aggregate is crushed, combined with a 1N solution of NaOH, and then stored at 80°C for 24 hours. The reduction in alkalinity and quantity of dissolved silica indicate the potential for alkali reactivity of the aggregate. However, the presence of such minerals as carbonates or certain magnesium silicates may render the results inaccurate. ASTM C 295, which examines the lithology and mineralogy rather than the chemistry, will provide more accurate and complete information. The reactivity of an aggregate is not determined so much by the chemistry as by the mineralogy. Silica can be close to inert in the form of large, well-ordered crystals; somewhat reactive in microcrystalline, highly fractured, or cryptocrystalline form; and highly reactive if amorphous. A chemical test cannot easily distinguish these differences. The petrographic examination will also show the distribution of the various minerals in the aggregate, which will also affect its reactivity in concrete.

5.2 ASR Test Methods

5.2.1 Field Examination

In the field, ASR manifests itself by a characteristic "map cracking" or "pattern cracking". However, as Stark (1991) points out, restraint due to abutting concrete or embedded steel reinforcement may affect the development of the crack pattern. For example, a highway pavement may exhibit more cracking in the longitudinal direction due to greater restraint in the longitudinal direction. Palmer et al. (1988) observe that combinations of poor workmanship, weathering, shrinkage, or differential stresses due to general deterioration or damage can result in cracking, and that these mechanisms are not always easily distinguished from ASR. Thus in a condition survey of a deteriorated structure it is essential to keep an open mind and consider all possible causes of cracking. Stark (1991) gives the example of ASR cracking that may be confused with D-cracking due to freeze/thaw cycling. Cracking that is more severe along pavement joints may appear to be D-cracking, although it could be due to ASR. The two may be distinguished by noting that D-cracking is roughly parallel to the joint, while cracking associated with ASR has a more random orientation. One should also keep in mind that more than one mechanism could have caused the deterioration observed.

Palmer et al. (1988) provide an excellent general discussion of how to conduct a field inspection. Specific observations of certain features during the condition survey may point to ASR; these observations must then be followed up by a laboratory investigation to confirm and supplement the findings. Features that may point to ASR include:

(a) **Cracking.** Record the position, disposition, and pattern of cracking. Photography may be used for general documentation, but major cracks should also be documented on scale drawings showing length, width, apparent depth, continuity, surface displacement, path (around or across aggregate particles), and obvious association with reinforcement, stress orientation, or surface discoloration. Typically, in the absence of restraint or loading, the crack pattern can be described as a three-armed star.

(b) **Discoloration.** Surface discoloration is sometimes a feature of ASR and may occur along cracks or as surface patches. Staining due to rust indicates corrosion of the reinforcement. Note that ASR is not the only possible cause of light-colored stains.

(c) **Efflorescence and exudations.** Since gel is the product of ASR, the presence of either fresh or dried gel, especially along cracks, may indicate ASR. Samples of any exudation should be collected for later analysis.

(d) **Pop-outs**. Expansion of an aggregate particle or of gel close to the surface may cause detachment of a conical portion of the surface, leaving a small pit. Note that pop-outs can also be caused by frost damage.

Stark (1991) discusses the uranyl acetate test, which can be used on concrete in either the field or the laboratory to identify ASR gel. This test works best on newly exposed surfaces such as fresh fractures, cores, or ground or sawn surfaces. *Note that performing this test requires the use of certain safety gear, which is not discussed in the following general description of the method.* The surface should first be rinsed with tapwater. Then the uranyl acetate solution is applied. After 3-5 minutes (to allow the solution to react with any ASR gel that may be present), rinse the surface with tapwater. Under UV light the ASR gel appears bright yellowish-green. Deposits are generally localized in cracks, voids, and aggregate particles. Broad films of gel on sawn or cored surfaces may indicate smearing during the cutting process. With experience, the bright fluorescence of ASR gel can be easily distinguished from the weaker fluorescence of carbonated paste and calcite deposits, and the natural fluorescence of some minerals.

The uranyl acetate test was originally developed by Naitesayer and Hover (1988). They found that analyzing the concrete within one hour of treatment with the uranyl acetate solution was necessary; otherwise the background flourescence would increase dramatically. Stark et al. (SHRP-C-343) state that the uranyl acetate test "requires discrimination between broad, faint areas of flourescence which reflect, for example, pretreatment locations of alkali absorbed on cement hydration products, and ASR gel deposits, which usually occur within the periphery of reacted aggregate particles." Both of these potential errors can contribute to false positive results.

Core samples should be taken from several areas of the structure for detailed study in the laboratory. It is important to obtain representative samples from areas of the structure that are in different stages of deterioration. One petrographer uses the rule "the good, the bad, and the ugly" in selecting her samples, that is, some concrete that is in good condition, some that has started to deteriorate, and some that is severely deteriorated. Palmer et al. (1988) outline the documentation required for each core: location, diameter, length, and orientation of the core; fractures, joints, and disintegration; number of pieces (if not intact); size, position, and condition of any reinforcement present; and other features such as damp patches and fresh gel. Photographs should also be taken of the core location, and the condition of the concrete inside the core hole should be noted.

5.2.2 Laboratory Examination

Even if the uranyl acetate test has been performed in the field, it may be worthwhile to perform it again under the more controlled conditions of the laboratory for confirmation. A standard petrographic examination (ASTM C 856) will provide such evidence as reaction rims and/or gel around aggregate particles and allow the identification and quantification of reactive constituents in the aggregate. PCA's *Guide Specification for Concrete Subject to Alkali-Silica Reactions* (1995), while directed at the prevention rather than the diagnosis of ASR, provides useful limits on the quantities of the various reactive minerals that may be present in the aggregate:

- Optically strained, microfractured, or microcrystalline quartz exceeding 5.0%
- Chert or chalcedony exceeding 3.0%
- Tridymite or cristobalite exceeding 1.0%
- Opal exceeding 0.5%
- Natural volcanic glass in volcanic rocks exceeding 3.0%

The petrographer can also identify any potentially reactive constituents that have not yet reacted. It is important that the petrographer approach the examination with an open mind, as other destructive mechanisms may be implicated instead of or in addition to ASR.

If necessary, the petrographic examination can be supplemented by scanning electron microscopy and/or chemical analyses of portions of the specimen. For example, the alkali content of the concrete can be estimated. If good records of the concrete mix design are available, the cement alkali content can also be estimated. Records of the field performance of the aggregates can also be useful.

In order to estimate the potential for further expansion due to ASR, one each of a set of three cores can be exposed at 100°F for 12 months to (1) immersion in water, (2) immersion in 1 N NaOH solution, or (3) storage over water in a sealed container. Expansion of the core stored in NaOH solution of more than 0.030 percentage points greater than for the companion core stored in water is considered indicative of expansive ASR. Note that this test is not a standard test; thus the results are not directly comparable from one laboratory to another, and it is not known how many replicates are required to produce a reliable result.

5.3 Chloride Test Methods

5.3.1 ASTM C 1152

The chloride ion content can be determined for either acid- or water-soluble chlorides. Generally the acid-soluble chloride ion content is considered the "total" chloride ion content, while the water-soluble chloride ion content is a better indication of the chloride ions actually available to the steel.

ASTM C 1152 provides for two methods of sampling the concrete to be tested. Ideally, a core will be taken from the structure, with samples sliced from the core and pulverized for analysis. Alternatively, a rotary impact drill may be used to take powder samples directly. ASTM C 1152 cautions that the latter method may produce unrepresentative samples. In addition, it is difficult to control conditions in the field sufficiently to prevent contamination or loss of some of the specimens, while intact cores are less sensitive to errors in handling. Once the samples are obtained and pulverized, they are analyzed for acid-soluble chlorides by the method described in ASTM C 114. In this method, the sample is first dissolved in dilute nitric acid. Methyl orange is used to indicate when the solution is sufficiently acidic for the test. The solution is heated to boiling, then filtered. The filtrate is allowed to cool to room temperature and is then titrated with silver nitrate solution to determine the chloride ion concentration. It should be noted that some aggregates contain chlorides that will show up as acid soluble chlorides, but most likely will not effect corrosion.

A more precise sampling method is currently being developed at the University of Toronto and at CTL. (Detwiler et al., 1996) In this method a nominal 4-inch core is milled off in layers of approximately 1 mm thickness. Each layer is collected as a separate sample, which is analyzed according to ASTM C 114 for acid-soluble chlorides. The chloride concentration is plotted as a function of the distance from the surface. Depending on the conditions of exposure, the data may be used to compute the apparent diffusion coefficient. Since the conditions of Fick's Second Law generally are not obtained in real structures, this is generally not appropriate for field concrete without a certain amount of extrapolation. However, conditions approaching those assumed in Fick's Law may be imposed in the laboratory setting so that an apparent diffusion coefficient can be calculated. The apparent diffusion coefficient can then be used *with caution* to give some indication of the future service life. Even without an apparent diffusion coefficient, the data from these detailed chloride profiles can be instructive, however.

5.3.2 "Rapid Chloride" Test

Another convenient and commonly-used test is ASTM C 1202 (similar to AASHTO T 277). In this test a 4-inch concrete cylinder or nominal 4-inch core is sliced to a thickness of 2 inches, vacuum saturated, and placed in a cell with NaCl solution on one side and NaOH solution on the other. A potential difference of 60 V direct current is maintained across the cell. The test measures the total charge passed in six hours, with the values divided into broad categories indicating the chloride ion penetrability.

Essentially the test measures the electrical conductivity of the concrete, which can be roughly correlated with the concrete's resistance to chloride ion penetration. Buenfeld and Newman (1987) point out that while the diffusion coefficient of an ion through concrete depends primarily on the microstructure of the concrete, conductivity depends on both microstructure and pore solution chemistry. Concretes can differ significantly in pore solution chemistry, particularly when they contain different amounts of supplementary cementing materials. Thus the comparison of concretes containing different amounts of supplementary cementing materials -- let alone different materials -- is dubious. Zhang and Gjørv (1991) point out that the relation between electrical conductivity and diffusion may vary with the mechanism and type of diffusion. For example, ions diffuse through cement paste by volume diffusion, but the presence of cracks or other voids will result in surface diffusion. Thus small defects will affect the accuracy of measurement. They also state that the measurement of electrical conductivity according to ASTM C 1202 is based on the rate of ion penetration before steady state is reached.

Hooton (1989) emphasizes the need for caution in interpreting the test results. Lower-quality concrete is difficult to evaluate properly because the temperature of the specimens rises when the current is applied, artificially increasing the rate of ion diffusion and necessitating discontinuation of the test. Even concretes that would normally be expected to perform well may exhibit such temperature increases. On the other hand, good-quality concretes are difficult to distinguish because the total charge passed in six hours is so low. The variability of the test results is another reason for caution. Hooton observes that the test results have been abused by those who try to attach some significance to minor differences between concretes in order to show that one is superior to another.

Broadly speaking, however, ASTM C 1202 does provide comparable results to other test methods. Detwiler and Fapohunda (1993) compared the results with those obtained from a similar test method in which most of the concerns outlined above had been minimized or eliminated and found that the two test methods resulted in comparable rankings of eighteen different concretes containing different supplementary cementing materials. Detwiler et al. (1996) found that ASTM C 1202 gave similar rankings of high-quality concretes with and without silica fume to the detailed chloride profile tests. ASTM C 1202 has the additional advantages of being a quick, simple, and relatively inexpensive test. Since it is also a standard test, it is easy to obtain comparable results from one laboratory to another, and the precision of the test is established. So long as one does not read more into the results than are warranted, it is a good indication of the general quality of the concrete and its ability to resist the ingress of chloride ions.

5.3.3 Corrosion Potential Survey

In addition to removal of samples for determination of the chloride ion content and the resistance to chloride ion penetration, a corrosion potential survey is often included as a nondestructive test method in the structural evaluation of a concrete element. The corrosion potential survey is used to determine the probability of corrosion taking place on the reinforcing steel.

ASTM C 876, "Half Cell Potentials of Uncoated Reinforcing Steel in Concrete," outlines the procedures and materials required to perform this test. A copper-copper sulfate half-cell is used in combination with a high-impedance digital voltmeter to measure electrical potentials of the embedded reinforcing steel within concrete.

Typically, a connection to reinforcing steel is made by drilling into the steel reinforcement and fitting a small knurled steel pin into the steel. An alligator clamp is then connected to the steel pin which is connected to the negative terminal of the high-impedance digital voltmeter. The positive terminal of the high-impedance digital voltmeter is connected to the copper-copper sulfate half-cell. Electrical potential measurements are taken by prewetting the concrete surface, placing a small water-soaked cellulose sponge at a specific test location, and placing the half-cell in contact with the sponge.

Potential measurements are usually recorded on a 4 ft (or smaller) grid. If the concrete surface is sealed with paint or a sealer, the coating must be removed by grinding to expose the concrete surface. Potential measurements are valid from 32 to 120°F. A temperature correction of the potential measurement is required when the temperature is outside of the 72±10°F range.

According to ASTM C 876-91, laboratory testing of reinforced concrete specimens indicates the following regarding significance of the corrected numerical value of the potentials measured relative to the copper-copper sulfate (CSE) half-cell.

(a) If potentials over an area are numerically less than -0.20 Volts CSE, there is a greater than 90% probability that no reinforcing steel corrosion is occurring in that area at the time of measurement.

(b) If potentials over an area are in the range of -0.20 to -0.35 Volts CSE, corrosion activity of the reinforcing steel in that area is uncertain.

(c) If potentials over an area are numerically greater than -0.35 Volts CSE, there is a greater than 90% probability that reinforcing steel corrosion is occurring in that area at the time of measurement.

Results can be presented in tabular form or in the preferred graphical method presented in Figure #1. Note that the potentials are presented in 50 millivolt increments.

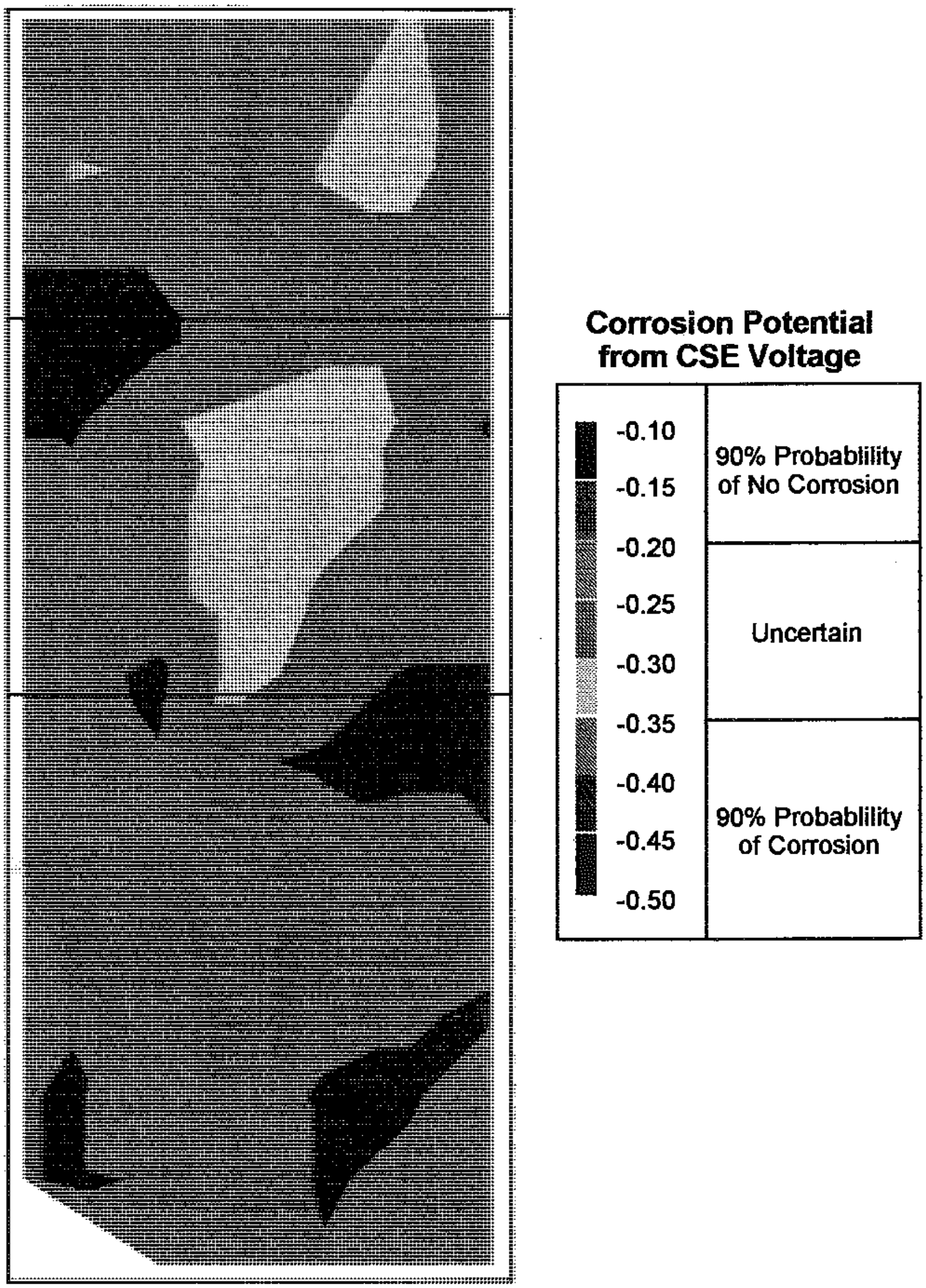

Figure #1 *Corrosion Potential of Reinforcement.*

6. ALKALI SILICA REACTIVITY, CAUSES, MECHANISMS AND PREVENTION

6.1 Causes and Mechanisms and Manifestations of ASR in Bridge Structures

Alkali-Silica Reactivity (ASR) is one of several types of harmful Alkali-Aggregate Reactions (AAR) that occur in concrete. The two fundamental processes of ASR are the swelling and/or dissolution of soluble silica and the formation of alkali-silica gels. The alkali-silica gels are formed by the reaction of the soluble silicas with calcium ions supplied by cement hydration reactions. These gels are expansive, creating internal forces in the concrete that may lead to cracking and crack opening. (Helmuth, 1993)

6.1.1 ASR Reaction Chemistry

ASR is a heterogeneous chemical reaction which occurs in siliceous aggregate particles. The reaction occurs between the alkaline pore solution of the cement paste and the silica particles in the fine or coarse aggregates. The reaction takes place when large amounts of hydroxyl ions are present in the pore solution due to high alkali concentrations. These hydroxyl ions in turn cause the silica in the aggregates to enter into solution. The combination of alkalis and silicas in solution precipitates out alkali silicates. The alkali silicates swell when combined with water, creating the expansive ASR gel. (Johansen, 1995)

The composition of ASR susceptible aggregate has a significant influence on the reaction process. Porous silica-containing aggregates react rapidly, with ASR gel forming first at the aggregate surface, and moving inward. Tensile stresses that build up during the reaction can cause both the aggregate and surrounding paste to crack. In denser polycrystalline aggregates, the reaction occurs primarily on the surface and at grain boundaries. The formation of expansive gel is usually limited and cracking is typically confined to the grain boundary regions. (Johansen, 1995)

6.1.2 Manifestations of ASR in Bridge Structures

ASR can be found in any concrete element of a bridge, including the bridge deck, parapet walls, substructure, and abutments. The cracking patterns exhibited tend to run parallel to the direction of loading in members subjected to restraint or loading. In unrestrained members, the crack pattern is more random, as will be described later. On bridge decks, the predominant pattern of cracks is longitudinal along the deck, with some curvature of the cracks toward the transverse direction at the ends of the deck. In the substructure, horizontal crack patterns extending along the length of pier caps, and long vertical cracks in columns can be indicative of ASR. Furthermore, the presence of a white or brownish deposit in and around cracks is frequently associated with ASR. It should be noted that ASR deterioration may, in some cases, be easily confused with other types of distress such as freeze-thaw deterioration and cracking due to corrosion of embedded steel. The use of microscopic evaluation and/or other test methods is required to ensure proper identification of the cause of distress. (Stark, 1991)

6.1.3 Moisture and Temperature Conditions

It has been shown that the moisture condition of the concrete is very important in the occurrence of ASR. A relative humidity of roughly 80% must be maintained in order to sustain expansive alkali-silica reactions. The moisture condition of a concrete element varies highly non-linearly through its thickness, and is dependent on its exposure to environmental conditions. Furthermore, climate region and seasonal variation play a significant role in the moisture state within the element. In cold moist climates such as the one in Michigan, it is not uncommon for the moisture state within the center of a concrete element to remain above the 80% relative humidity threshold year round. This promotes an increased susceptibility to ASR. (Helmuth, 1993)

In addition to the moisture state of the concrete, temperature plays a significant role in ASR development. Research conducted in Canada (Alasali, 1989), showed that specimens stored at high temperatures (38°C) expanded more rapidly than specimens stored at room temperature (23°C). Greatly increasing the temperature (to 80°C) led to massive increases in expansion of the specimens. While such high temperature is not experienced in the field, it is an effective method of accelerating ASR in laboratory studies.

While lower temperatures have been shown to inhibit ASR development, it is not recommended to place concrete at cold temperatures as a method of inhibiting ASR reactivity. Because ASR acts over a span of years, it is most beneficial to ensure the highest quality concrete at the onset by avoiding cold weather concreting. Furthermore, pozzolanic additives, which are used to inhibit ASR, tend to retard the hydration reaction. The combined effects of cold temperature and the use of pozzolans can lead to permeability problems in the hardened concrete. (Mohr, 1993)

6.2 ASR Prevention

There are a number of effective methods of limiting and preventing the expansive alkali-silica reaction. The three primary approaches to reducing reactivity are to reduce the content of reactive alkalis, to avoid use of reactive silica bearing aggregates, and to inhibit movement of the reactants through the concrete by densifying the mix with pozzolanic additives. Other methods, such as the use of ASR inhibiting admixtures has also been successful.

6.2.1 Avoidance of Reactive Silica-Bearing Aggregates

Petrographic evaluation of aggregates in accordance with ASTM C295 is used to determine the constituents of concrete aggregates. It has been found that the aggregates that are susceptible to ASR, tend to contain silica constituents in quantities shown in Table # 2.

Reactive Constituent	Maximum Tolerable Amount	
	Kosmatka	Corps. of Engineers
Opal	0.5%	0%
Chert and Chalcedony	3%	--
Chert with any Chalcedony	--	5%
Tridymite and Cristobalite	1%	1%
Strained or Microcrystalline Quartz	5%*	--
Highly Strained Quartz	--	20%
Volcanic Glasses (>55% Silica)	0%	3%

* *as found in granites, granite gneiss, graywackes, argillites, phyllites, siltstones, and natural sands and gravels*

***Table #2** Constituents of aggregates prone to ASR (after Lane, 1992)*

Aggregates that are found to be susceptible to ASR based on the above table, should be evaluated further for ASR using ASTM C227 or ASTM C 1260. These test methods measure expansion of mortar bars cast using the aggregate in question. The amount of expansion measured after a set period of time determines the acceptance/rejection criteria for that aggregate. For ASTM C227, a 0.05% expansion after 3 months, or a 0.10% expansion after 6 months leads to rejection of the aggregate. ASTM C 1260 specifies that a mean expansion after 14 days of 0.10% indicates unacceptable expansion levels. (Kosmatka, 1991)

6.2.2 Use of Low Alkali Cements

The use of low alkali cements is a viable option where large portions of available aggregates are reactive. In a study conducted in Virginia it was found that a significant increase in ASR expansion occurs if the cement alkali content is greater than 0.40%. Furthermore, the study shows that expansion of cement paste mortars is roughly linear with increasing alkali content at ages of two months or more (it takes roughly one month for the alkali concentration of the cement paste pore solutions to stabilize, especially in high alkali cements). This indicates that expansion of cement mortars is primarily a function of the alkali content of the cement, if aggregate type is kept constant. (Lane, 1995)

Traditionally a portland cement alkali content of 0.60% or less has been considered acceptable in providing protection against ASR expansion. It has been found though, that cements with alkali contents exceeding 0.40% can yield excessive expansions. Furthermore, it has been found that expansions exceeding 0.10% are typically associated with the onset of damage in mortar bars. Thus, the use of 0.10% expansion as an acceptance criteria for cements is recommended. (Lane, 1995)

6.2.3 Use of Pozzolans to Inhibit Reactivity

Pozzolanic materials such as fly ash, silica fume, and ground granulated blast-furnace slag, are frequently added to concrete as a means of inhibiting expansion due to ASR. For effective protection against ASR, the type and amount of pozzolan used, as well as the method by which it is introduced into the mix can be important. For example, data by Lane and Ozyildirim (1995) shows that fly ash, slag, and silica fume each have varying degrees of effectiveness, based on the level at which they are added. Typically pozzolans are added as a cement replacement, with replacement being made by mass of total cementitious material. For fly ash (class F), a 15% replacement is sufficient to protect mortar bars which contain cement with a 0.6% alkali content from ASR expansion. At 35% fly ash replacement, mortar bars with 0.92% cement alkali content are held to acceptable expansion levels (less than 0.1%). Similar trends are seen for slag and silica fume additions. It was shown that a 35% fly ash replacement, a 50% slag replacement, or a 7% silica fume replacement can sufficiently reduce expansion even in mortar bars made with high alkali cement.

Pozzolans influence the ASR reaction in several ways. First, their addition as a cement replacement reduces the amount of alkalis available, as the cement content has been reduced. Second, pozzolans densify the mix, reducing the permeability of the concrete. This restrains the movement of free alkalis, and inhibits the reaction process. Finally, and most importantly, pozzolans are composed primarily of glassy silica or silicates which react in the presence of hydroxide ions in a manner similar to the way reactive aggregates react. Because of the extremely fine nature of these pozzolans as compared with aggregates, they react much more quickly. In this way, the reaction actually uses up available hydroxide ions so rapidly that harmful concentrations may never form. The product of this reaction, commonly referred to as a pozzolanic reaction, is cementitious in nature, and contributes to the concrete strength while reducing concrete permeability. At the same time, there are no longer enough hydroxide ions available to force siliceous aggregates into solution, which would lead to the expansive alkali-silicate gel. (Lane, 1992)

There has been some controversy over the effectiveness of fly ash in reducing the harmful effects of ASR. After extensive research in the 1980's it was concluded that the class C fly ash requires a much larger cement replacement than does class F fly ash in order to protect against ASR expansion. Furthermore, when used in low replacement amounts, class C fly ash has actually been found to contribute to expansion in some cases, rather than inhibiting it. Class F fly ash is thus recommended for use with ASR susceptible materials. (Lane, 1992)

6.2.4 Use of ASR Inhibiting Admixtures

Lithium based admixtures have been found to inhibit ASR expansion when added in small dosages to concrete containing highly reactive aggregates. Lithium compounds come in both liquid and powder forms. Lithium fluoride and lithium carbonate salts are powders, while lithium hydroxide solution is available as a liquid or as a powder. Lithium carbonate is effective in preventing expansion at a dosage of 1% by weight of cement. Lithium fluoride is effective in dosages of 0.5% to 1% by weight of cement. (Concrete Technology Today, 1993)

Lithium based admixtures react with siliceous aggregates in a manner similar to alkalis in cement. In normal cement pastes, the KOH or NaOH in solution reacts with the silicas in the aggregate to form the ASR gel. The addition of lithium changes this reaction process. The lithium in solution reacts with the silicas, creating a gel similar to the ASR gel formed by normal cement pastes. The difference with the gel formed by the lithium reaction is that it is not expansive, when used in conjunction with a low alkali cement. When added to alkali bearing cements, lithium appears to be effective only if it has been added above a threshold amount, where the effects of the alkalis can be counteracted by the lithium. The use of lithium as an ASR inhibitor is still a relatively young area of research, with further study ongoing to better understand the reaction processes. (Diamond, 1992)

Lithium-based compounds have also been used in surface applications to inhibit further degradation due to ASR in hardened concrete. These applications are currently an experimental technology. This approach was first studied at CTL in 1991. In these initial studies, ASTM C 227 mortar bars exhibiting expansions due to ASR were soaked in saturated solutions of various lithium compounds. Continued expansion due to ASR was virtually eliminated by soaking mortar bars in a saturated LiOH solution for a period of 2 days. (Stark, 1993) Other lithium compounds were not as effective as the LiOH in suppressing further expansion. A similar experiment was performed by spraying an ASR affected highway pavement section with a saturated LiOH solution. Results indicated that surface cracks transmitted the lithium ions into the concrete; however, diffusion of the lithium was too slow to be effective in the bulk of the concrete. (Stark, 1993)

6.2.5 Use of Surface Sealers and Injections to Inhibit ASR

Surface sealers and injection materials present a significant advantage of being able to be applied either at time of construction or to existing deteriorating structures. In this way they may be effective in both prevention and repair applications. Numerous papers have been published on the effects of various sealers and injections, and their respective effectiveness in mitigating ASR. While the conclusions are not necessarily in agreement as to which types of sealants and injections are most effective, the following guidelines appear to be appropriate.

Surface barrier coatings must fulfill three basic requirements in order to be effective in preventing ASR deterioration. First, they must prevent the ingress of aggressive agents such as water and chlorides into the concrete. Second, they must permit evaporation of moisture trapped within the concrete. Third, they should have the ability to bridge minor cracks while maintaining the continuity and integrity of the coating. In this way, contaminants can be kept out of the concrete, and entrapped water will be allowed to escape. (Swamy, 1992) Several types of coatings are used for ASR prevention. The more common among these are monomers, polymers, epoxies, acrylic rubbers, and silanes.

Injection or penetrant materials have the function of filling existing cracks, and preventing free movement of contaminants through the cracks. The type of penetrant to be used depends on the crack width. It is also recommended to use penetrants in conjunction with sealers, as penetrants alone do not stop the ingress of contaminants. If the reaction process in already taking place, the repair method may not manifest immediate results. Instead, the water trapped in the concrete may continue the expansion process for some time. (Moriya, 1989)

7. CHLORIDE ATTACK - CAUSES, MECHANISMS AND PREVENTION

7.1 Causes and Mechanisms of Chloride Attack

The introduction of chlorides into concrete containing reinforcing steel can lead to the corrosion of the steel, and damage to the concrete. The pH of non-carbonated concrete is on the order of 12 to 13. In the high-alkaline environment of concrete, a cohesive oxide film forms on the surface of the reinforcing steel, protecting it from further corrosion. The introduction of chloride ions destroys the passivity of the steel, allowing for corrosion to take place if sufficient moisture and oxygen are present. The corrosion of the steel leads to loss of cross-section as well as the formation of a corrosion product that takes up greater volume than the uncorroded steel. This expansive corrosion reaction can lead to internal forces building up and causing cracking in the concrete. (Hope, 1987)

Nielsen (1979) attributes the ability of chloride ions to break down the passivating layer to their tendency to polarize, as illustrated in Figure #2. The coating of Fe_2O_3 on the iron surface has a positive charge, which creates an electric double layer of ions (A). The negative chloride ions are attracted to the electric double layer. Near the surface, the chloride ions become polarized and elongated (B). The chloride ions then become part of the electric double layer, dissolving ferrous (Fe^{+2}) ions (C). The cycle continues as more chloride ions are attracted to the site. Since the chloride ions are not consumed in any of the corrosion reactions, the effect is intensified, resulting in pitting corrosion (D).

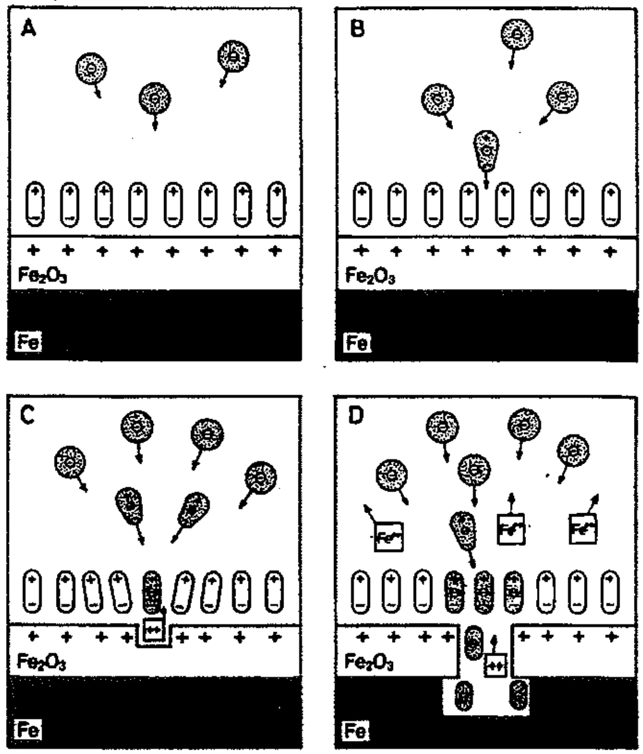

Figure #2 Chloride ion attack of Reinforcement (Nielsen, 1979)

The maximum chloride ion concentration that can be tolerated before corrosion initiates can vary from 0.03-0.4% by mass of concrete. Nielsen (1979) explains that not all chloride ions are free to attack the steel, since they can be chemically bound or physically absorbed by the cement gel. The activity of chloride ions also depends on the concentrations of other ions in the pore solution, particularly Na^{+}, Ca^{+2}, and OH^{-}. In addition, the rate of corrosion will depend on the availability of oxygen and water at the steel surface as well as the electrical conductivity of the concrete.

Chlorides may be introduced into the concrete by several means. At the time of mixing, they may be added in chloride bearing admixtures, or they may be present in chloride bearing aggregates. Once the concrete has hardened, chlorides can permeate the concrete in de-icing salt solutions. It is not the total chloride content in a concrete that is of importance for corrosion, but rather the amount of free chlorides. Some chlorides are bound in aggregates or through the formation of chloroaluminate. These chlorides are not harmful to reinforcing steel. The free chlorides on the other hand, when present in sufficient concentration in the vicinity of the steel can cause serious and rapid deterioration of the steel. (Hope, 1987)

7.2 Protection Against Chloride Attack

Corrosion of steel occurs when the chloride ion content in the vicinity of the reinforcing steel is on the order of 1.2 lb/yd^3. To reduce the susceptibility of a concrete to chloride intrusion from de-icing salts, low permeability mix designs are sometimes used. This has become common practice for bridge decks where chloride corrosion of reinforcement has long been identified as a leading concern. By reducing the chloride ion permeability of a concrete, the infiltration and free movement of chloride ions in the concrete is reduced, protecting it from the buildup of chloride ion concentrations around the reinforcing steel. In bridge decks, pozzolanic additives and polymeric additives are frequently used to reduce concrete permeability. (Sprinkel, 1993)

Surface sealers such those mentioned in section 6.2.5 are also used to protect against chloride intrusion. It has been shown that the use of sealers can significantly reduce chloride intrusion when the sealers are applied at the time of construction. Sealers can protect the concrete for 10 years or more. At the same time, experimental data shows that sealers are not as effective in preventing corrosion as are pozzolanic and polymeric additives. Furthermore, it is unknown whether later application of sealers, or reapplication of sealers will yield positive results. (Sprinkel, 1993)

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