CMX

GORDONS CORNER ROAD OVER ROUTE 9
SUPPLEMENT TO "PHASE 1 REPORT - BRIDGE DECK REPLACEMENT STUDY" DATED JULY 9, 2008

Prepared by: Gregory D. Bitsko, P.E., P.P.
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PROJECT NUMBER: 070804502

Introduction

During our presentation of the Phase 1 Report -Bridge Deck Replacement Study to the NJDOT SME's, it was suggested that an additional alternative be considered which would provide for the replacement of the superstructure using conventional steel girders with a cast-in-place concrete deck to provide a basis of comparison for the Inverset alternatives. Additionally, it was noted during the presentation that the Scope of Services for CMX did not include performing a hands-on inspection of the pier cap during the initial study phase. It was agreed that the condition of the cap would affect the overall viability of all deck replacement alternatives. The existing cap exhibits extensive areas of wide pattern cracking and, if the condition was not correctable or was indicative of more significant defects, it is possible that the replacement of the cap, deck and superstructure would not be cost effective and the structure should be programmed for replacement instead. It was determined at the conclusion of the presentation that the initial step during Final Design would be to perform a hands-on inspection of the cap and to obtain concrete cores for chloride testing in order to evaluate whether the cap should be replaced.

Superstructure Replacement Using Conventional Steel Girders

This alternative would include the removal and replacement of the entire deck and superstructure using new built up steel girders and conventional cast-in-place concrete techniques under staged construction.

Pedestrian access would be maintained pursuant to the Phase 1 Report. This alternative would fully address the deficiencies in the deck and existing superstructure as well as improving the vertical underclearances to the extent provided by the Inverset alternatives. It would not correct the existing substandard horizontal clearances and the longitudinal deck joint would be retained for the reasons stated in the Phase 1 Report.

Typically, the existing superstructure elements would be removed and the new girders set using cranes. However, it would not be possible to place the northerly two girders of the northeast and northwest spans using this method due to the proximity of the overhead utilities. It is sometimes possible to initially remove existing girders beneath energized power lines using a pair of excavators because of their limited reach. The girders can then be set outside the clearance requirements of the High Voltage Proximity Act and cranes used to lift the girders onto trucks for removal. The process is reversed when setting the new girders.

Cranes would initially pick the new members from the trucks and place them on the substructure outside the required clearances with the excavators then used to set the girders in their final positions under the energized lines. The multi-span configuration of the existing bridge will result in extended lane closures on Gordons Corner Road when using this method which will adversely affect the viability of this alternative.

The only potential staging areas for cranes are on Gordons Corner Road in either approach to the bridge or on Route 9. Staging on the Gordons Corner Road approaches is not feasible because of the existing utilities which run north and south over both approaches. Similarly, cranes could only be staged on Route 9 south of the bridge due to the overhead utility lines running along the north fascia. As there are no overhead utilities south of the bridge, the southeast and southwest spans can be constructed concurrently. However, because of the two-span configuration, either the northeast or northwest span must remain in place until construction of the other is completed in order to provide a platform for one of the excavators. The span initially used to support the equipment could not be fully removed until the concrete deck of the first span had cured sufficiently to support the excavator required to remove and set the northerly girders of the second span. As staging areas for the cranes would be located south of the bridge, picks for both removing the existing girders and placing the new members would cross the southerly spans. With 11 picks required for each span, Gordons Corner Road would most probably have to be closed and traffic detoured while performing this work. Closing the overpass entirely to traffic for the entire period of construction would still require two-stage construction as the two northerly spans would have to be constructed sequentially to provide support for the excavators.

Based on the foregoing, it would require either three construction stages with limited closures of Gordons Corner Road or two stages using a full detour to perform the work of this contract if the utilities were to remain in place. The other alternative is to relocate the utilities to allow staging of cranes on the north side of the bridge which, while still requiring two construction stages, would allow traffic to be maintained on Gordons Corner Road at all times. Because it is not considered feasible to close Gordons Corner Road for the full construction duration, two alternatives were evaluated to install the conventional girders. The first is to construct the improvements in three stages without relocating the overhead utilities and the second is to relocate the utilities and perform the work in two stages.

No Utility Relocation

The anticipated sequence of operations is as follows:

1. Partially remove the concrete island on Gordons Corner Road in the west approach and construct temporary pavement as necessary to accommodate staged construction, restrict Gordons Corner Road traffic to one lane in each direction, implement a center work zone and remove the center concrete median.

2. Reset Gordons Corner Road MPT to shift traffic to the south side of the bridge while maintaining one lane in each direction.

3. Remove all concrete encasement and diaphragms on the northeast (assumed first stage construction) span. Removal of the encasement and diaphragms on the other spans cannot be performed at the same time as the diaphragms and tie rods provide lateral stability for the stringers. This work will require nighttime lane closures on Route 9.

4. Install shielding on the bottom flanges of the stringers in the northeast span for removal of the concrete deck. This work will require nighttime lane closures on Route 9.

5. Construct cuts in Route 9 median and install temporary pavement and closure measures. This work will require nighttime lane closures on Route 9. It is assumed that one lane in each direction could be closed concurrently, each night to expedite construction.

6. Prepare bridge bearings and seats for new girders. This work will require nighttime lane closures on Route 9. It is assumed that one lane in each direction could be closed concurrently, each night to expedite construction.

7. Commencing at 9 PM:

    a. Close Gordons Corner Road to traffic and implement detour.
    b. Implement MPT measures on Route 9 to restrict traffic to one lane in each direction and shift traffic to the southbound lanes using the median cuts.
    c. Set crane adjacent to the southeast span.
    d. Remove the existing girders in the northeast span.
    e. Set proposed girders for the northeast span.
    f. Remove crane, and once girders are secure, reestablish two lanes of northbound and southbound traffic on Route 9.

8. Install shielding for construction of deck. While it might be possible to perform this work while the girders are being placed, it is assumed that the work would have to be performed after Route 9 is reopened to traffic. Therefore, this work will require nighttime lane closures on Route 9.

9. Install shear studs and diaphragms on the northeast span.

10. Construct deck and approach slab for the northeast span.

11. After appropriate curing period, perform items 3, 4, 7, 8, 9 and 10 for the northwest span.

12. Construct sidewalks and parapets on the northeast and northwest spans.

13. Reset MPT to shift traffic to the north side of the bridge maintaining one lane in each direction on Gordons Corner Road.

14. Perform items 3, 4 and 6 for both the southeast and southwest spans. This work will require nighttime lane closures on Route 9. It is assumed that one lane in each direction could be closed concurrently, each night to expedite construction.

15. Perform item 7 for the southeast span.

16. Perform item 7 for the southwest span on a separate night.

17. Perform item 8 for the southerly spans and concurrently, remove temporary median cuts on Route 9 and restore areas. This work will require nighttime lane closures on Route 9. It is assumed that one lane in each direction could be closed concurrently, each night to expedite construction.

18. Perform item 9 for both southerly spans.

19. Perform item 10 for both southerly spans.

20. Remove all shielding. This work will require nighttime lane closures on Route 9. It is assumed that one lane in each direction could be closed concurrently, each night to expedite construction.

21. Reopen Gordons Corner Road to full width, remove temporary pavement and reconstruct island in west approach.

Utility Relocation

The anticipated sequence of operations is as follows:

1. Temporarily relocate all aerial utilities in the vicinity of the bridge which would conflict with the installation of the girders.

2. Partially remove the concrete island on Gordons Corner Road in the west approach and construct temporary pavement as necessary to accommodate staged construction, restrict Gordons Corner Road traffic to one lane in each direction, implement a center work zone and remove the center concrete median.

3. Reset Gordons Corner Road MPT to shift traffic to the south side of the bridge while maintaining one lane in each direction.

4. Remove all concrete encasement and diaphragms on the northeast and northwest spans. Removal of the encasement and diaphragms on the south side of the bridge being used to maintain Gordons Corner Road traffic cannot be performed at the same time as the diaphragms and tie rods provide lateral stability for the stringers. This work will require nighttime lane closures on Route 9. It is assumed that one lane in each direction could be closed concurrently each night to expedite construction.

5. Install shielding on the bottom flanges of the stringers in the northeast and northwest spans for removal of the concrete deck. This work will require nighttime lane closures on Route 9. It is assumed that one lane in each direction could be closed concurrently each night to expedite construction.

6. Construct cuts in Route 9 median and install temporary pavement and closure measures. This work will require nighttime lane closures on Route 9. It is assumed that one lane in each direction could be closed concurrently each night to expedite construction.

7. Prepare bridge bearings and seats for new girders. This work will require nighttime lane closures on Route 9. It is assumed that one lane in each direction could be closed concurrently each night to expedite construction.

8. Commencing at 9 PM:

    a. Close Gordons Corner Road to traffic and implement detour.
    b. Implement MPT measures on Route 9 to restrict traffic to one lane in each direction and shift traffic to the southbound lanes using the median cuts.
    c. Set crane adjacent to the northeast span.
    d. Remove the existing girders in the northeast span.
    e. Set proposed girders for the northeast span.
    f. Remove crane, and once girders are secure, reestablish two lanes of northbound and southbound traffic on Route 9.

9. On a separate night, commencing at 9 PM:

    a. Close Gordons Corner Road to traffic and implement detour.
    b. Implement MPT measures on Route 9 to restrict traffic to one lane in each direction and shift traffic to the northbound lanes using the median cuts.
    c. Set crane adjacent to the northwest span.
    d. Remove the existing girders in the northwest span.
    e. Set proposed girders for the northwest span.
    f. Remove crane, and once girders are secure, reestablish two lanes of northbound and southbound traffic on Route 9.

10. Install shielding for construction of deck. While it might be possible to perform this work while the girders are being placed, it is assumed that the work would have to be performed after Route 9 is reopened to traffic. Therefore, this work will require nighttime lane closures on Route 9.

11. Install shear studs and diaphragms on the northeast and northwest spans.

12. Construct deck and approach slabs for the northeast and northwest spans.

13. Construct sidewalks and parapets on the northeast and northwest spans.

14. Reset MPT to shift traffic to the north side of the bridge maintaining one lane in each direction on Gordons Corner Road.

15. Perform items 4, 5 and 7 for both the southeast and southwest spans. This work will require nighttime lane closures on Route 9. It is assumed that one lane in each direction could be closed concurrently, each night to expedite construction.

16. Perform item 8 for the southeast span.

17. Perform item 9 for the southwest span on a separate night.

18. Perform items 10, 11, 12 and 13 for the southeast/southwest spans.

19. Remove all shielding. This work will require nighttime lane closures on Route 9. It is assumed that one lane in each direction could be closed concurrently, each night to expedite construction.

20. Reopen Gordons Corner Road to full width, remove temporary pavement and reconstruct island in west approach.

These alternatives have approximately the same estimated construction costs as the crane installed Inverset panels and are only slightly less expensive than the SPMT installed Inverset panels. However, they significantly increase the duration of traffic impacts on both Route 9 and Gordons Corner Road. The temporary median cuts on Route 9 would be required for a longer period of time which could adversely affect public safety and as demolition and construction would be occurring over Route 9 after the highway is reopened to traffic, there is a potential for debris to fall into live traffic lanes. They are more forgiving than the Inverset panels in accommodating deviations in the existing construction. They also eliminate the need to restrict Route 9 traffic to one lane in each direction for a full weekend period. The use of built up girders provides more flexibility in the construction schedule as they would not be tied to specific rolling dates for the girders required with the Inverset system and, therefore, the advertisement date of the project would not be on the critical path.

The No Utility Relocation alternative eliminates eight months of disruption to traffic on both Gordons Corner Road and Route 9, but with three construction stages, still requires 365 calendar days of fulltime lane closures on Gordons Corner Road. The estimated 55 calendar days of nighttime lane closures required on Route 9 is the greatest of any of the superstructure replacement alternatives. The total estimated cost of this alternative is $3,400,000.00

Pursuant to the discussion of the Inverset alternatives in the original report, relocating the utilities is estimated to take eight months which was determined to be the equivalent of 105 calendar days of fulltime lane closures on Gordons Corner Road for comparative purposes. While the actual 325 calendar day duration of fulltime lane closures on Gordons Corner Road for bridge construction is slightly less than that of completing the work with the existing utilities in place, the additional period of equivalent fulltime lane closures caused by the utility relocations results in this alternative having the greatest impacts to traffic on Gordons Corner Road at 430 calendar days. The estimated 43 calendar days of nighttime lane closures on Route 9 is also slightly less than the 55 calendar days required for working around the utilities. The estimated construction cost for this alternative is $3,500,000.00, which is roughly the same as for the crane installed Inverset alternative and only $300,000.00 less than the SPMT installed Inverset alternative

Ratings were not performed for either of these alternatives as it is assumed that the proposed design would be developed to exceed the capacities of the rating vehicles. Additionally, these alternatives were not evaluated based on future maintenance value as this value would be the same as for the Inverset alternatives. Therefore, the conventional girder replacements were evaluated against the Inverset panels by comparing cost, impacts to Gordons Corner Road and impacts to Route 9. The following are the tables summarizing these results.

ALTERNATIVE	CONSTRUCTION COST	VALUE
Conventional, No Utility Relocation	$3,400,000	8
Conventional, Utility Relocation	$3,500,000	8
Inverset Crane Install	$3,500,000	8
Inverset SPMT Install	$3,800,000	7

ALTERNATIVE	GORDONS CORNER ROAD LANE RESTRICTIONS	VALUE
Conventional, No Utility Relocation	365 CD	0
Conventional, Utility Relocation	430 CD	0
Inverset Crane Install	180 CD	7
Inverset SPMT Install	75 CD	10

ALTERNATIVE	ROUTE 9 LANE RESTRICTIONS	VALUE
Conventional, No Utility Relocation	55 WD	0
Conventional, Utility Relocation	43 WD	0
Inverset Crane Install	14 WD	10
Inverset SPMT Install	14 WD	10

DESIGN MATRIX				
	Conventional, No Utility Relocation	Conventional, Utility Relocation	Inverset Crane Install	Inverset SPMT Install
Construction Cost	8	8	8	7
Gordons Corner Road Lane Restrictions	0	0	7	10
Route 9 Lane Restrictions	0	0	10	10
Final Score	8	8	25	27

CD = Calendar Days WD = Working Days

Based on the foregoing, the SPMT installed Inverset panels represent the best possible alternative for this project.

Cap Condition

CMX performed a hands-on inspection of both the pier and abutments on December 11th and December 12th, 2008. All areas of unsound concrete were measured and documented on a plan identifying limits of potential repairs under this contract. A copy is attached to this memorandum. In general, despite the extensive area of wide map cracking, the substructure is in fairly good condition for a bridge of this age. Most of the defect areas have not manifested into spalls, but are areas of unsound concrete. In those areas where spalls do exist, only a few have exposed reinforcing steel. The reinforcing steel at these spalls, while exhibiting surface rust, does not appear to be severely corroded where the expansion of the steel would have been the cause of the spall. The wide map cracking does not appear to have any correlation to the defective areas as the vast majority of the concrete exhibiting this cracking was sound.

As a follow-up to the physical inspection, CMX contracted with Certified Testing Labs to obtain and test ten cores at various locations in the pier. These cores were obtained on December 16th and December 17th, 2008. It was originally anticipated that eight cores would be taken in the columns of the pier and two would be taken in the cap. However, based on the identified areas of unsound concrete and the extent of the pattern cracking in the cap, CMX elected to reverse the priority of the core locations and obtained eight cores in the cap and two in the columns. It was also originally expected that all cores would be tested for chloride content alone. However, the pattern of the pier cracks and the fact that the concrete was not displaced on either side of the cracks, did not seem consistent with corrosion of the underlying reinforcing steel as the mechanism for causing these defects. Because of the visible quartz coarse aggregate used in the existing concrete, CMX elected to perform a petrographic analysis on one core (#10) to test for alkali-silica reactivity in addition to determining chloride content. The results of all tests are attached to this memorandum. Nine of the cores were tested for chlorides a distance of 1-1/8" below the surface of the concrete. The core which was used for the petrographic analysis was also tested for chlorides throughout the entire sample length. The results of the tests indicate that the existing concrete does have sufficient chloride content to induce corrosion in the underlying reinforcing steel. The sample for the petrographic analysis indicated a chloride content of 0.3% by mass of concrete which exceeds the 0.2% threshold level to initiate corrosion. The petrographic analysis concluded that the cracking in the concrete is probably primarily due to cyclical freeze/thaw action of the non-air entrained concrete. Secondarily, the cracking may be caused by alkali-silica reactivity. The report did indicate that the cracking may also be caused by corrosion of the reinforcing steel and states that a #5 deformed bar which was recovered did show evidence of surface corrosion.

However, based on our inspection of the reinforcing steel recovered at this core and at several other cores, we do not believe that the cracking is being caused by corrosion of the underlying reinforcing steel. Except for the few locations where the concrete has spalled revealing shallow cover on the underlying reinforcing steel, it appears that most of the reinforcing steel has in excess of 3" of cover. The sample recovered for the petrographic analysis had a cover of 3-7/8". At two other locations, we observed concrete cover of 4" and 3" at a third location. While the reinforcing steel did show some indication of surface rusting, there was no section loss in the steel and no visible increase in the diameter of the bars usually associated with corrosion leading to the formation of cracks and spalls. It may very well be likely that the surface corrosion existed at the time the concrete was originally poured.

After reviewing the results of the coring program and conducting the field inspection, it is our opinion that the pier cap can be rehabilitated for use in this project without any significant reconstruction. It is recommended for consideration by NJDOT that chloride extraction measures be incorporated into the proposed design following the procedures of the Norcure system or other alternative systems.

Conclusion

Based on the results of the supplemental evaluation and testing program, it is our recommendation that the existing pier cap be retained and that spot repairs be performed in conjunction with the superstructure replacement to remove existing chlorides, patch the spalls and unsound concrete and seal all the existing cracks. We also recommend that the superstructure and deck be replaced using Inverset panels erected using SPMT's.

Assuming that NJDOT concurs with this assessment, the next immediate step would be for CMX to meet with representatives of SPMT companies on the site to review the recommended staging areas in order to verify the viability of this approach. Once the design proceeds assuming the use of SPMT's, any change to modify the erection method will significantly delay the project and result in additional design costs. Therefore, the decision must be made at this time whether or not to proceed with the use of SPMT's to erect the Inverset panels.

GDB:nac

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CMX

JOB NO. 070804507	SHEET 1 OF 5
PROJECT: GORDONS CORNER RD/RT9	
SUBJECT: GC RD TRAFFIC IMPACTS	
COMPUTED BY: GB	DATE 1/27/09
CHECKED BY: JMM	DATE 1/29/09

WORK DAY ANALYSIS

1. NO UTILITY RELOCATION

STAGE 1B

1. CENTER WORK ZONE, REMOVE ISLAND, MODIFY SIGNAL

TOTAL = 7CD

STAGE 2 (NE SPAN)

A. RESET MPT, MODIFY SIGNAL
DEMO DECK, WINGWALL PARAPETS & REMOVE APPROACH SLABS

14 CD

ASSUMED 9CD FOR ENCASEMENT REMOVAL & SHIELDING WILL BE CONCURRENT W/ STAGE 1B & START OF (A)
ASSUMED 6CD FOR RT9 PREP CONCURRENT W/ A

B. REMOVE GIRDERS, SET GIRDERS, INSTALL SHIELDING
INSTALL SHEAR STUDS, DIAPHRAGMS

20 CD

C. FORM & POUR DECK

14 CD

D. CURE DECK 28CD

TOTAL = 76 CD

STAGE 3 (NW SPAN)

A SAME WORK AS STAGE 2 (A) BUT ASSUME CONCURRENT W/ STAGE 2 (D). ALSO 7CD FOR ENCASEMENT REMOVAL & SHIELD CONCURRENT W/ STAGE 2 (D)

JOB NO.	070804502	SHEET 2	OF 5
PROJECT:	GORDONS CORNER (GC) RD/RT9		
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B. SAME AS STAGE 2 (B) = 20 CD
C SAME AS STAGE 2 (C) = 14 CD
D SAME AS STAGE 2 (D) = 28 CD

E FORM/POUR/CURE BRIDGE SIDEWALK = 0 — ALL N
ASSUMED CONCURRENT W/ D

F FORM/POUR BRIDGE & WINGWALL PARAPET — ALL N
= 14 CD

G FORM/POUR/CURE APPROACH CURBS — ALL N
W/ SUBGRADE PREP = 9 CD

H FORM/POUR SIDEWALKS & APPROACH SLABS — ALL N
= 10 CD

I CURE/SAWCUT APPROACH SLABS — ALL N
= 17 CD

J INSTALL FENCE & GUIDERAIL 7 CD — ALL N

K RESET MPT, MODIFY SIGNAL 4 CD

TOTAL = 123 CD

CMX

JOB NO.	070804502	SHEET 3	OF 5
PROJECT:	GC / RT 9		
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STAGE 4 (SOUTH SPANS)

A. DEMO DECK ETC = 21 CD

B REMOVE / SET GIRDERS, SHEAR STUDS, SHIELDING, DIAPHRAGMS
28 CD

C FORM & POUR DECK 21 CD

D CURE DECK 28 CD

E-K SAME AS STAGE 3 = 61 CD

TOTAL = 159 CD

TOTAL GC LANE CLOSURE DURATION = 365 CD

2. UTILITY RELOCATION

STAGE 1B SAME AS NO UTILITY RELOCATION = 7 CD

STAGES 2 & 3 EACH SAME AS STAGE 4 NO UTILITY RELOCATION = 159x2 = 318 CD

UTILITY RELOCATION IMPACTS, EQUIV TO 105 CD PER INVERSET CRANE ANALYSIS

TOTAL 430 CD

RT 9 WORK DAY ANALYSIS

NO UTILITY RELOCATION

A. NE/NW SPANS (SINGLE CREW/1 MPT SET)
ENCASEMENT REMOVAL = 6WD
SHIELDING FOR DECK REMOVAL = 3WD
SHIELDING FOR DECK CONST = 3WD
12WD x 2 = 24WD

B SE & SW (2 CREW / 2 MPT SET)
ENCASEMENT REMOVAL = 6WD
SHIELDING DECK REMOVAL = 3WD
SHIELDING DECK CONST = 3WD
12WD

C RT 9 PREP WORK
MEDIAN CUTS, PREP PIER 4WD (2CREW/2MPT SET)
PREP ABUT BEARINGS = 2WD (" ")
RECONST MEDIAN = 3WD (" ")

D SUPER REPLACEMENT, NE, NW, SE, SW = 4 WD (SINGLE CREW/1 MPT SET)

E REMOVE ALL SHIELD 2 CREW/2MPT SET = 6WD

TOTAL WD = 55
28 SINGLE CREW SETS
27 2CREW SETS
MPT SET UPS 28 + 2(27) = 82

CMX

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PROJECT:	GC RD/RT9		
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UTILITY RELOCATION

A NORTH & SOUTH SPANS (2 CREW/2MPT SETS)
SAME AS (B) NO RELOC = 12WD

12x2 = 24WD

B RT 9 PREP (2CREW/2MPT SETS)
SAME AS (C) NO RELOC = 9WD

C SUPER REPLACEMENT (SINGLE CREW/1MPT SET)
= 4WD

D REMOVE SHIELD (2 CREWS/2MPT SETS)
SAME AS (E) NO RELOC = 6WD

TOTAL WD = 43

4 SINGLE CREW SETS
39 2 CREW SETS
MPT = 2(39) + 4 = 82

USE UNIT COSTS FROM PHASE I REPORT
CREW COSTS INCL EQUIPMENT:
CONC. ENCASEMENT REMOVAL
10300 $/WD 1CREW
14600 $/WD 2CREW

DECK DEMO & SEAT PREP
11125 $/WD

SHIELDING (IN & OUT)
7302 $/WD 1CREW
10552 $/WD 2CREW

COMMON WORK

1. GC MPT = 160000
2. SIGNAL MODS = 100000
3. PHASE IA = 5000

$265,000

1. NO UTILITY RELOCATION

A. COMMON WORK = $265,000

B ENCASEMENT REMOVAL
NE = 6WD x 10300 = 61800
NW = 6WD x 10300 = 61800
SE/SW = 6WD x 14600 = 87600

$211200

C SHIELDING DECK REMOVAL

NE = 3WD x 7302 = $21906
NW = 3WD x 7302 = 21906
SE/SW = 3WD x 10552 = 31656

$75468

D. RT 9 PREP

ROADWAY CUT/RESTORE = 70000
PREP BRGS 2 x 11125 = 22250

$92250

E DEMO DECK

NE = 14 CD
NW = 14 CD
SE/SW = 21 CD

49 CD x 11125 = $545125

DISPOSAL = 576 TONS x 100$/TON = $57600

$602725

F. NEW SUPER/DECK

1. DISPOSAL 552 TONS x 100$/TON = $55200
2. GIRDERS/DIAPH 176261 LB x 3.25 = 572848
3. BEARINGS 40 x 1000 = 40000
4. SHIELDING NEW DECK = 75468
5. DECK CONC 225 CY x 1200$/CY = 270,000
6. REBAR 35000 LB x 2$/LB = 70000
7. SAWCUT 6100 SF x 2$/SF = 12200

CMX

JOB NO.	070804502	SHEET 3	OF 5
PROJECT:	GCRD/RT9		
SUBJECT:	COST EST CONVENTIONAL		
COMPUTED BY:	GB	DATE	1/27/09
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8. PARAPET 387LF x 250 $/LF = 96750
9. BRIDGE FENCE 387 x 110 $/LF = 42570

$ 1,235,036

G APPROACH WORK
SAME AS CIP ALT = $ 270,800

H SHIELD REMOVAL
6 WD x 10552 = 63312

I MPT

24	SINGLE	x 1250 =	30000
4	RT9 SHIFT	x 4000 =	16000
27	2 LANE	x 2500 =	67500
			113500

TOTAL = $ 2,929,291

x 15% CONTINGENCIES = $ 3,368,685

SAY $ 3,370,000

CMX

JOB NO.	070804502	SHEET 4	OF 5
PROJECT:	GC RD / RT 9		
SUBJECT:	COST EST CONVENTIONAL		
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2. RELOCATE UTILITIES

A COMMON WORK — $265,000

B. UTILITY RELOCATION — $275,000

C. ENCASEMENT REMOVAL
N & S SPANS = 72 WD x 14600 = $175200

D. SHIELDING DECK REMOVAL
6 WD x 10552 = $63312

E RT 9 PREP (SAME) = $92250

F DEMO DECK
21 CD x 2 = 42 CD x 11125 = 467250
DISPOSAL 57600
$524850

G NEW SUPER/DECK
SAME AS NO RELOCATE EXCEPT
REDUCED SHIELDING
1,235,036 − 75468 + 63312 = $1,222,880

H APPROACH WORK (SAME) $270800

I SHIELD REMOVAL (SAME) $63312

CMX

JOB NO.	070804502	SHEET 5	OF 5
PROJECT:	GC R10/RT9		
SUBJECT:	COST EST CONVENTIONAL		
COMPUTED BY:	GB	DATE	1/27/09
CHECKED BY:	JMM	DATE	1/29/09

J. MPT

39	2LANG	x 2500	97500
4	RT 9 SHIFT	x 4000 =	16000
			$113500

TOTAL = $3066,104

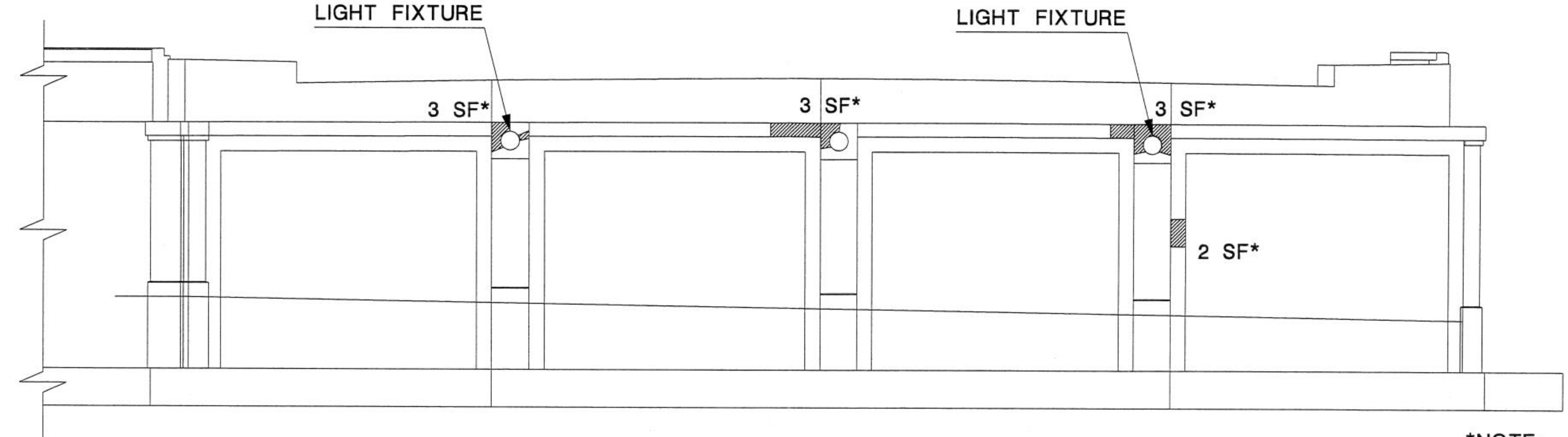
x 15% CONTINGENCIES = $3,526,020

SAY $3,530,000

STATE	FEDERAL PROJECT NO.	SHEET	TOTAL SHEETS
NJ			

STRUCTURE NO. 1303-156

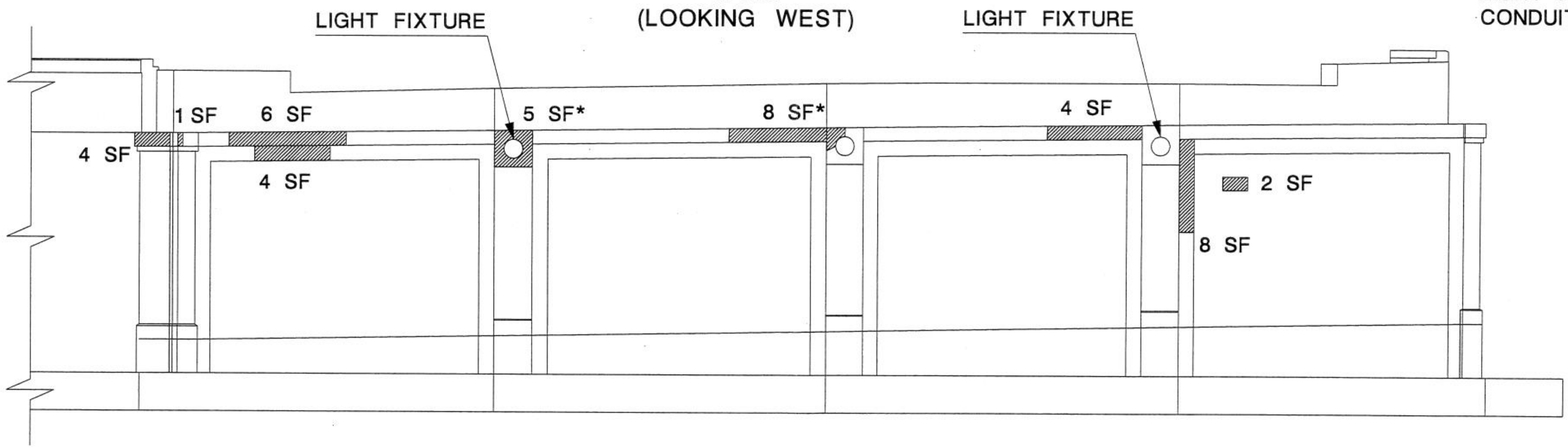
STRUCTURE NAME
GORDONS CORNER OVER ROUTE 9



WEST ABUTMENT ELEVATION

N.T.S.
(LOOKING WEST)

*NOTE:
SPALLED AND UNSOUND CONCRETE CAUSED BY LIGHT FIXTURES AND STEEL CONDUIT IN ABUTMENT.



EAST ABUTMENT ELEVATION

N.T.S.
(LOOKING EAST)

LEGEND:

- AREA OF SPALLED/ UNSOUND CONCRETE

CONTROL SECTION	1303	JOB NO.0708045	
DES BY	J. MILDENBERG	CHK. BY	G. BITSKO
DWN. BY	K. WINITS		
EST. BY		CHK. BY	
SPECS BY			
IN CHARGE OF	-		

ALL DIMENSIONS SHOWN ON THIS SHEET ARE IN ENGLISH UNITS

NEW JERSEY DEPARTMENT OF TRANSPORTATION
BUREAU OF STRUCTURAL ENGINEERING

ABUTMENT CONDITION PLAN
GORDONS CORNER ROAD OVER ROUTE 9
CONTRACT NO. 000983530

MANALAPAN MUNICIPALITY — MONMOUTH COUNTY

CMX

REVISION	BY	C'K'D	DATE

GREGORY D. BITSKO
N.J. P.E. LICENSE #GE 035238

BRIDGE SHEET NO. ____ OF ____

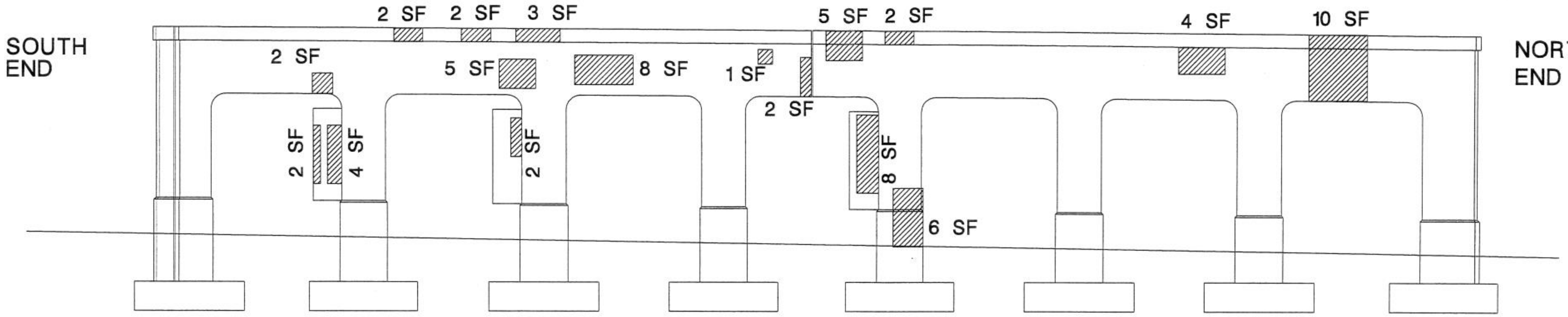
XX

FILE NAME : **SOFIL601*M*P*0708015*BRIDGE*B-WABUT-DEF.DGN
DATE/TIME : 1/29/2009 9:10:36 AM

STATE	FEDERAL PROJECT NO.	SHEET	TOTAL SHEETS
NJ			

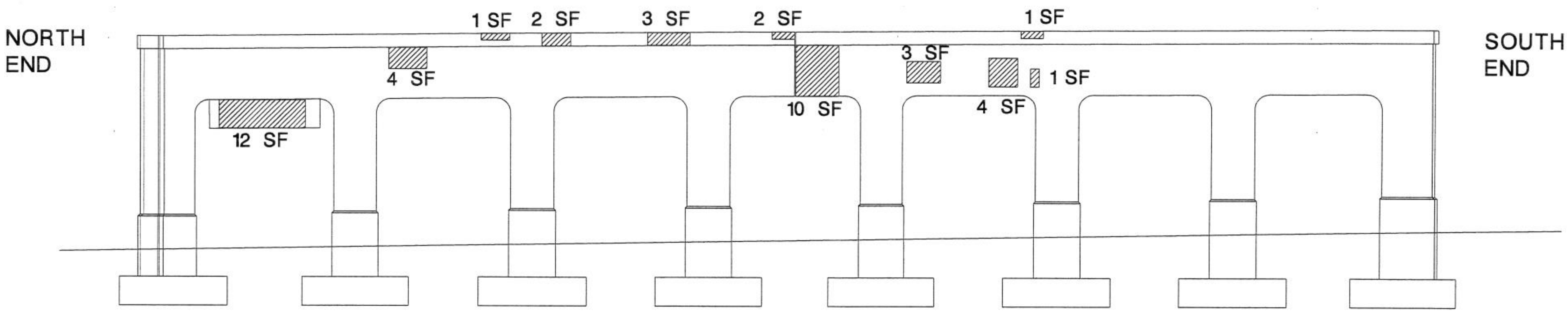
STRUCTURE NO. 1303-156

STRUCTURE NAME
GORDONS CORNER OVER ROUTE 9



EAST PIER ELEVATION

N.T.S.
(LOOKING WEST)



WEST PIER ELEVATION

N.T.S.
(LOOKING EAST)

LEGEND:

- AREA OF SPALLED/ UNSOUND CONCRETE

CONTROL SECTION	1303	JOB NO.0708045	
DES BY	J. MILDENBERG	CHK. BY	G. BITSKO
DWN. BY	K. WINITS		
EST. BY		CHK. BY	
SPECS BY			
IN CHARGE OF	-		

ALL DIMENSIONS SHOWN ON THIS SHEET ARE IN ENGLISH UNITS

NEW JERSEY DEPARTMENT OF TRANSPORTATION
BUREAU OF STRUCTURAL ENGINEERING

PIER CONDITION PLAN

GORDONS CORNER ROAD OVER ROUTE 9
CONTRACT NO. 000983530

MANALAPAN MUNICIPALITY

MONMOUTH COUNTY

CMX

REVISION	BY	C'K'D	DATE

GREGORY D. BITSKO
N.J. P.E. LICENSE #GE 035238

BRIDGE SHEET NO. ____ OF ____

XX

FILE NAME - **$OFILE01*M*P*0708045*BRIDGE*B-PIER-DEF.DGN
DATE/TIME - 1/29/2009 9:11:24 AM

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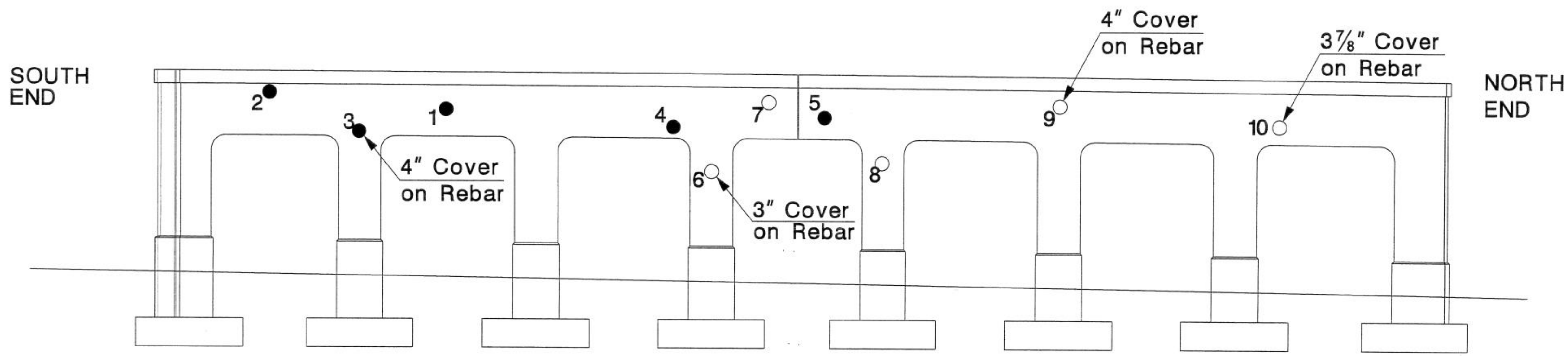
NJ			

STRUCTURE NO. 1303-156

STRUCTURE NAME
GORDONS CORNER OVER ROUTE 9



CORING LOCATION PLAN

N.T.S.

LEGEND:

● - CORE TAKEN AT WEST FACE
○ - CORE TAKEN AT EAST FACE

CONTROL SECTION	1303	JOB NO.0708045	
DES BY	J. MILDENBERG	CHK. BY	G. BITSKO
DWN. BY	K. WINITS		
EST. BY		CHK. BY	
SPECS BY			
IN CHARGE OF ______ -			

ALL DIMENSIONS SHOWN ON THIS SHEET ARE IN ENGLISH UNITS

NEW JERSEY DEPARTMENT OF TRANSPORTATION
BUREAU OF STRUCTURAL ENGINEERING

CORING LOCATION PLAN
GORDONS CORNER ROAD OVER ROUTE 9
CONTRACT NO. 000983530

MANALAPAN MUNICIPALITY
MONMOUTH COUNTY

CMX

REVISION	BY	C'K'D	DATE

GREGORY D. BITSKO
N.J. P.E. LICENSE #GE 035238

BRIDGE SHEET NO. ______ OF ______

XX

FILE NAME • **SDFIL601*M*P*0708045*BR1056*B-PIER-CORE.DGN
DATE/TIME • 1/29/2009 9:17:40 AM

CTL
CERTIFIED TESTING LABORATORIES, INC.

CLIENT: CMX Engineering
P.O. Box 900
Manalapan, NJ 07726

DATE TAKEN: 12-16-08

PROJECT: Overpass Bridge for Gordons Corner Road
Over Route 9, Manalapan, NJ

LAB NO.: B-319-08

MATERIAL: Concrete Cores

REPORT NO.: CC-1, page 1 of 2

TEST REQUIRED: Chloride Content (opt. Petrographic-ASR)...see additional reports.

IDENTIFICATION:	B-1	B-2	B-3	B-4	B-5
Length of core submitted:	5.5"	7.5"	7.0"	5.0"	8.5"
Length of core prepared:	-	-	-	-	-
Diameter (inches):	4.0	4.0	4.0	4.0	4.0
Area (sq. inch):	-	-	-	-	-
Crushing Load:	-	-	-	-	-
Ratio H to D (L/D)	-	-	-	-	-
Correction factor Ratio:	-	-	-	-	-
Corrected crushing load:	-	-	-	-	-
Compressive Strength PSI	-	-	-	-	-
Date Tested:	N/A	N/A	N/A	N/A	N/A
Date Cast:	+ 20 years	+ 20 years	+ 20 years	+ 20 years	+ 20 years

LOCATION: Bridge cores (south side), see attached drawings.

REMARKS: CTL patched all core holes.

Respectfully submitted,
Certified Testing Laboratories, Inc.

Terry Kifer

Terry Kifer, General Manager

TK/ah

RECEIVED
JAN 09 2009
CMX INC.

Main Office
155 U.S. Route 130 • Bordentown, NJ 08505 • (609) 298-3255 • Fax (609) 298-7288

CTL

CLIENT: CMX Engineering
P.O. Box 900
Manalapan, NJ 07726

DATE TAKEN: 12-16-08

PROJECT: Overpass Bridge for Gordons Corner Road
Over Route 9, Manalapan, NJ

LAB NO.: B-319-08

MATERIAL: Concrete Cores

REPORT NO.: CC-1, page 2 of 2

TEST REQUIRED: Chloride Content (opt. Petrographic-ASR)...see additional reports.

IDENTIFICATION:	B-6	B-7	B-8	B-9	B-10
Length of core submitted:	8.3"	3.0"	4.7"	4.0"	7.8"
Length of core prepared:	-	-	-	-	-
Diameter (inches):	4.0	4.0	4.0	4.0	3.0
Area (sq. inch):	-	-	-	-	-
Crushing Load:	-	-	-	-	-
Ratio H to D (L/D)	-	-	-	-	-
Correction factor Ratio:	-	-	-	-	-
Corrected crushing load:	-	-	-	-	-
Compressive Strength PSI	-	-	-	-	-
Date Tested:	N/A	N/A	N/A	N/A	N/A
Date Cast:	+ 20 years	+ 20 years	+ 20 years	+ 20 years	+ 20 years

LOCATION: Bridge cores (nortth side), see attached drawings.

REMARKS: CTL patched all core holes.

Respectfully submitted,
Certified Testing Laboratories, Inc.

Terry Kifer, General Manager

TK/ah

CTL

CERTIFIED TESTING LABORATORIES, INC.

January 20th, 2009

LAB NO: B-319-08
REPORT NO: CH-1

CMX
P.O. Box 900
Manalapan, NJ 07726

Attention: Gregory Bitsko

RE: Overpass Bridge for
Gordon's Corner Road over Route 9
Manalapan, NJ

Mr. Bitsko:

We are pleased to submit the final analytical package for samples submitted on 12/16/08.

All tests were performed by an independent laboratory, EMSL Analytical.

Should you have any questions pertaining to this report, please feel free to contact the undersigned.

Thank you for considering Certified Testing Laboratories, Inc. for your testing and inspection requirements. We look forward to working with you in the future.

Respectfully submitted,
CERTIFIED TESTING LABORATORIES, INC.

Terry Kifer

Terry Kifer, General Manager

TK/ah

Main Office
155 U.S. Route 130 • Bordentown, NJ 08505 • (609) 298-3255 • Fax (609) 298-7288

Asbestos • Lead • Environmental • Materials & Indoor Air Analysis

EMSL Analytical

http://www.emsl.com

3 Cooper St.
Westmont, NJ 08108
Phone: (856) 858-4800
Fax: 8568584571

Attn: **Terry Kifer**
Certified Testing Laboratories Inc
155 U.S. Route 130
Bordentown, NJ 08505

Phone (609) 298-3255
Fax: (609) 298-7288

1/19/2009

The following report covers the analysis performed on samples submitted to EMSL Analytical on 1/12/2009. The results are tabulated on the attached data pages for the following client designated project:

Project ID: overpass bridge for Gardens Corner Rd, over Rt.9 M

The reference number for these samples is EMSL Order #010900144. Please use this reference when calling about these samples.

If you have any questions, please do not hesitate to contact me at (856) 858-4800.

Reviewed and Approved By:

Julie Smith - Laboratory Director or
other approved signatory
NJ-NELAP Accredited:04653

ACCREDITED IN ACCORDANCE WITH nelac

The test results contained within this report meet the requirements of NELAC and/or the specific certification program that is applicable, unless otherwise noted.

EMSL Analytical
3 Cooper St., Westmont, NJ 08108
Phone: (856) 858-4800 Fax: (856) 858-4571 Email: jsmith@emsl.com

Attn: **Terry Kifer**
Certified Testing Laboratories Inc
155 U.S. Route 130
Bordentown, NJ 08505
Fax: (609) 298-7288 Phone: (609) 298-3255

Customer ID: CERT50
Customer PO:
Received: 01/12/09 1:36 PM
EMSL Order: 010900144

EMSL Proj: overpass bridge for Gardens Corner Rd, over R

Report Date: 1/19/2009

Client Sample Description	Collected	Lab ID	Method	Parameter	Concentration	Reporting Limit	Units	Analysis Date	Analyst
B-1	1/12/2009 10:30:00 AM	0001	9056	Chloride	730	25	mg/Kg	1/19/2009	mmazur
B-2	1/12/2009 10:30:00 AM	0002	9056	Chloride	500	25	mg/Kg	1/19/2009	mmazur
B-3	1/12/2009 10:30:00 AM	0003	9056	Chloride	1400	250	mg/Kg	1/19/2009	mmazur
B-4	1/12/2009 10:30:00 AM	0004	9056	Chloride	1400	250	mg/Kg	1/19/2009	mmazur
B-5	1/12/2009 10:30:00 AM	0005	9056	Chloride	1600	250	mg/Kg	1/19/2009	mmazur
B-6	1/12/2009 10:30:00 AM	0006	9056	Chloride	1900	250	mg/Kg	1/19/2009	mmazur

EMSL Analytical
3 Cooper St. Westmont, NJ 08108
Phone: (856) 858-4800 Fax: (856) 858-4571 Email: jsmith@emsl.com

Attn: **Terry Kifer**
Certified Testing Laboratories Inc
155 U.S. Route 130
Bordentown, NJ 08505

Fax: (609) 298-7288 Phone: (609) 298-3255

Customer ID: CERT50
Customer PO:
Received: 01/12/09 1:36 PM
EMSL Order: 010900144

EMSL Proj: overpass bridge for Gardens Corner Rd, over R

Report Date: 1/19/2009

Client Sample Description B-7 *Collected:* 1/12/2009 10:30:00 AM *Lab ID:* 0007

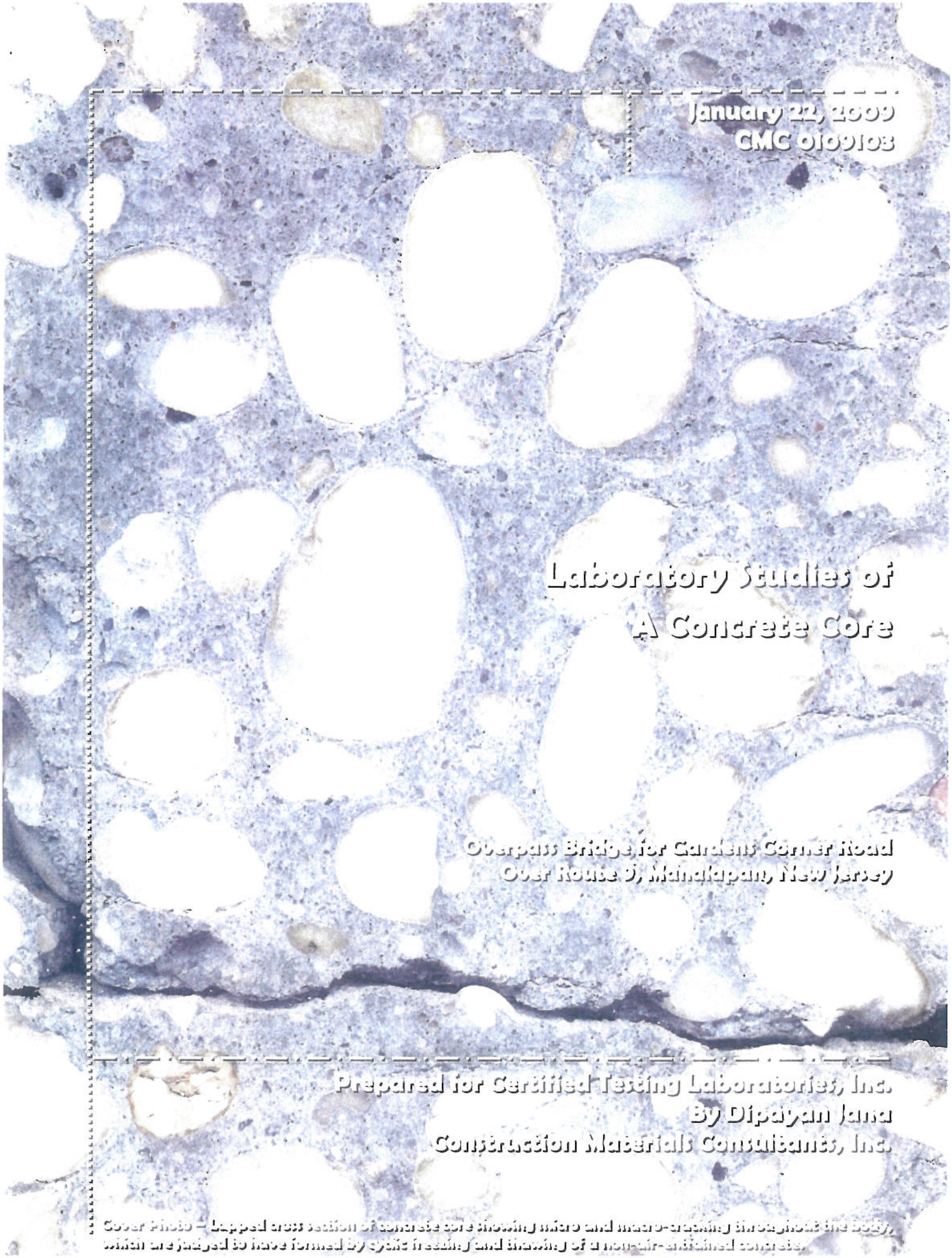
Method	*Parameter*	*Concentration*	*Reporting Limit*	*Units*	*Analysis Date*	*Analyst*
9056	Chloride	900	25	mg/Kg	1/19/2009	mmazur

Client Sample Description B-8 *Collected:* 1/12/2009 10:30:00 AM *Lab ID:* 0008

Method	*Parameter*	*Concentration*	*Reporting Limit*	*Units*	*Analysis Date*	*Analyst*
9056	Chloride	1700	250	mg/Kg	1/19/2009	mmazur

Client Sample Description B-9 *Collected:* 1/12/2009 10:30:00 AM *Lab ID:* 0009

Method	*Parameter*	*Concentration*	*Reporting Limit*	*Units*	*Analysis Date*	*Analyst*
9056	Chloride	1500	250	mg/Kg	1/19/2009	mmazur



January 22, 2009
CMC 0109108

Laboratory Studies of A Concrete Core

Overpass Bridge for Gardens Corner Road
Over Route 9, Manalapan, New Jersey

Prepared for Certified Testing Laboratories, Inc.
By Dipayan Jana
Construction Materials Consultants, Inc.

Cover Photo – Lapped cross section of concrete core showing micro and macro-cracking throughout the body, which are judged to have formed by cyclic freezing and thawing of a non-air-entrained concrete.

SUMMARY & DISCUSSION

Purpose & Background – Reported herein are the results of detailed laboratory studies of a concrete core received from Terry Kifer of Certified Testing Laboratories, Inc. The core was, reportedly, collected from an Overpass Bridge for Gardens Corner Road over Route 9 in Manalapan, New Jersey, which shows various degrees of concrete deterioration, such as visible cracking.

The purposes of the investigation are to determine: (a) the composition, quality, and condition of concrete in the core; (b) evidence, depth, and degree of any deleterious chemical deterioration of concrete within the core, such as alkali-silica reaction; (c) acid-soluble chloride contents of concrete at the exposed, mid-depth, and bottom ends of the core to investigate the potential of chloride-induced corrosion of reinforcing steel in concrete; and (d) evidence of any materials deficiencies and physical and/or chemical deterioration in the concrete after many years of service in a moist outdoor environment of cyclic freezing and thawing.

Determined Concrete Composition – Based on detailed petrographic examinations, the concrete in the core is determined to be *non-air-entrained*, and made using the following ingredients, in the following estimated proportions:

(a) Natural siliceous gravel coarse aggregate (strained quartzite) having a nominal maximum size of $^{3}/_{4}$ in., particles are well-graded, well-distributed, and show evidence of chemical unsoundness as alkali-silica reaction of a few gravel particles where the reactive particles have been cracked and the cracks are associated with alkali-silica reaction gels;

(b) Natural siliceous sand fine aggregate having a nominal maximum size of $^{3}/_{8}$ in. and containing major amounts of strained and unstrained quartz and quartzite, and minor to trace amounts of feldspar, schist, siltstone, chert, shale, ferruginous rocks, and mafic minerals; particles are well-graded and well-distributed; despite the presence of some potentially alkali-silica reactive particles in fine aggregate (strained quartz), there is no clear evidence of such a reaction of fine aggregate in the concrete;

(c) A moderately dense, gray, hardened portland cement paste having an estimated portland cement content of $5^{1}/_{2}$ to 6 bags per cubic yard; and an estimated water-cement ratio of 0.40 to 0.45; and,

(d) An air content estimated to be less than one percent.

Chloride Contents of Concrete – Acid-soluble chloride contents of concrete, from approximately $^{1}/_{2}$ in. thick sections saw-cut from the top exposed end, the mid-depth location, and the bottom end of the cores are 0.209, 0.083, and 0.046 percent, by mass of concrete (corresponding to 1.39, 0.55, and 0.30 percent by mass of cement). Therefore, the concrete has a higher chloride content at the top exposed end than that in the body and shows a progressively decreasing chloride contents from surface downwards – which are indicative of introduction of chloride into the concrete from the external environment (not uncommon in a bridge environment where deicing salts have been applied over the years). The chloride contents at all depths (i.e. from surface down to a depth of 8 in. have exceeded the industry documented minimum threshold chloride content of 0.20 percent by mass of cement to initiate corrosion of reinforcing steel in concrete in the presence of oxygen and moisture.

A No. 5 deformed reinforcing steel is present at a depth of $3^{7}/_{8}$ in., which shows surface corrosion – probably due to the detection of the high chloride level at that depth.

Therefore, similar to the evidence detected in the core, the possibility of chloride-induced corrosion of reinforcing steel in concrete exists in other areas if reinforcing steels in concrete in other areas have been exposed to such high chloride levels in the presence of oxygen and moisture.

Carbonation of Concrete – The core shows a minimal 5 mm depth of carbonation from the exposed surface end, which is not unusually high and is indeed indicative of a dense, well-consolidated reasonable water-cement ratio concrete which has prevented signification migration of atmospheric carbon dioxide into the concrete to further aggravate the corrosion of reinforcing steel issue by atmospheric carbonation – above the corrosion potential from the determined high chloride levels. Therefore, corrosion of steel, if evident, in any area of the concrete neighboring to the present core location is judged more likely to be chloride-induced than carbonation-induced.

Macro and Micro-Cracking – The most important clear and visible deterioration of concrete detected in the core are macro and micro-cracking, which occur as (a) closed polygonal-shaped map cracking on the exposed surface, many of which have extended into the interior as vertical cracks (i.e. oriented perpendicular to the exposed surface) to depths of $^{1}/_{2}$ in., and (b) severe parallel to subparallel macro and micro-cracks within the concrete body, oriented parallel to the exposed surface, transected and circumscribed the aggregate particles, and extended at least to a depth of $4^{1}/_{2}$ in. from the exposed surface (i.e., the top $4^{1}/_{2}$ in. concrete shows severe cracking compared to the bottom half). The core also has a major crack at a depth of $3^{3}/_{4}$ in., which broke the core into two halves. This major crack occurs at the depth of the reinforcing steel in the concrete – where the major crack may have formed by corrosion of reinforcing steel in concrete and/or during the core removal process.

<u>**Cracking Due to Cyclic Freezing and Thawing**</u> – Due to the non-air-entrained nature of the concrete, and its reported exposure to an outdoor environment of cyclic freezing and thawing, and the configuration of the map cracks on the surface and parallel severe cracking within the body from near the exposed surface region to a depth of $4^{1}/_{2}$ in – are all indicative of concrete deterioration (cracking) by cyclic freezing and thawing at critically water-saturated conditions.

As a result, cracking is judged to be initiated and aggravated mostly by the cyclic freezing and thawing of a non-air-entrained concrete.

<u>**Cracking Due to Deleterious Alkali-Silica Aggregate Reaction**</u> – A secondary mechanism of cracking, at local scale is detected as potentially expansive alkali-silica aggregate reaction, as mentioned before, which is associated with a few strained quartzite gravel coarse aggregate particles that are cracked and generated alkali-silica reaction gels which were deposited within those cracks either within the reactive particles or in cracks extended from the particles into the neighboring paste. The extent, abundance and severity of cracking due to alkali-silica reaction is determined to be significantly lower than cracking formed by cyclic freezing and thawing of a non-air-entrained concrete at critically water-saturated conditions.

<u>**In Summary**</u> – The concrete core shows evidence of map cracking on the exposed surface and severe micro and macro cracking at the top $4^{1}/_{2}$ in. of the exposed surface region – which are determined: (a) primarily due to cyclic freezing and thawing of a non-air-entrained concrete at critically saturated conditions, and, (b) secondarily due to alkali-silica reaction of a few reactive strained quartzite gravel coarse aggregate particles. The depth and degree of cracking due to freezing and thawing deterioration is judged to be more severe than that detected on local scale due to alkali-silica reaction of a few coarse aggregate particles. Due to the detection of high chloride levels in the concrete, down to the depth of at least 8 inches, the potential for corrosion of reinforcing steel in the concrete exists.

* * * * * *

1. INTRODUCTION

Received for detailed laboratory studies was a concrete core from Terry Kifer of Certified Testing Laboratories, Inc. The core was, reportedly, collected from an Overpass Bridge for Gardens Corner Road over Route 9 in Manalapan, New Jersey.

The purposes of the investigation are to determine: (a) the composition, quality, and condition of concrete in the core; (b) evidence, depth, and degree of any deleterious chemical deterioration of concrete within the core, such as alkali-silica reaction; (c) acid-soluble chloride contents of concrete at the exposed, mid-depth, and bottom ends of the core to investigate the potential of chloride-induced corrosion of reinforcing steel in concrete; and (d) evidence of any materials deficiencies and physical and/or chemical deterioration in the concrete after many years of service in a moist outdoor environment of cyclic freezing and thawing.

2. METHODOLOGY

The core was examined using the methods and procedures of ASTM C 856 "Practice for Petrographic Examination of Hardened Concrete." Petrographic examinations include:

(1) Detailed visual examinations of the core, as received;

(2) Low-power, stereomicroscopical examinations of freshly fractured and lapped cross sections of core at magnifications of up to 100X; and

(3) Examinations of oil immersion mounts and blue dye-mixed epoxy-impregnated thin section of core in a petrographic microscope at magnifications of up to 1000X.

Additionally, acid-soluble chloride contents of saw-cut sections of the core at various depths were determined by using the methods of ASTM C 1152 "Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete."

3. LABORATORY STUDIES

3.1 SAMPLE

Photographs and Dimensions – Figure 1 shows field photographs of the overpass, from where the core was, reportedly, obtained. Figure 2 shows the concrete core, as received. Figures 3, 4, and 5 show lapped cross sections of the core depicting cracking and aggregate grading/distribution. Figure 6 shows photomicrographs of lapped cross section of the core.

The core has a diameter of $3^7/_8$ in. and a nominal length of $8^1/_4$ in.

Surfaces – Figure 2 shows the exposed end surface of the core containing closed polygonal shaped map cracking (also notice in Figure 5) and fresh fractured opposite end.

Cracks, Joints, and Large Voids – The core contains: (a) closed polygonal-shaped map cracking on the exposed surface, many of which have extended into the interior as vertical cracks (i.e. oriented perpendicular to the exposed surface) to depths of $^1/_2$ in., and (b) severe parallel to subparallel macro and micro-cracks within the concrete body, oriented parallel to the exposed surface, transecting and circumscribing the aggregate particles, and extending at least to a depth of $4^1/_2$ in. from the exposed surface. The core has a major crack at a depth of $3^3/_4$ in., which broke the core into two halves. There are no joints or large voids present in the core.

Embedded Items – A No. 5 deformed reinforcing steel is present at a depth of $3^7/_8$ in., which shows surface corrosion – probably due to the detection of the high chloride level at that depth. There are no fibers or other reinforcement detected in the core.

Resonance – The core has a ringing resonance, when hammered.

3.2 PETROGRAPHIC EXAMINATIONS

3.2.1 Coarse Aggregate

Type and Nominal Maximum Size – Coarse aggregate is natural siliceous (strained quartzite) gravel having a nominal maximum size of $^3/_4$ in. (Figures 2 through 9).

Properties (Color, Angularity, Density, Hardness, Shape, Alteration, Coating, Cracking) – Particles are well rounded, dense, hard, clear to light gray, massive-textured, equidimensional, and uncoated. A few particles are cracked due to alkali-silica reactivity. A majority of the particles are transected by parallel cracks extended through the concrete to a depth of $4^1/_2$ in. from the exposed surface.

Gradation and Distribution – Coarse aggregate particles are well-graded and well-distributed (Figures 3, 4, and 5).

Potential Alkali-Aggregate Reactivity – Alkali-silica reaction gel is present within the voids and microcracks associated with a few reactive gravel particles (Figure 9).

3.2.2 Fine Aggregate

Type and Nominal Maximum Size – Fine aggregate is natural siliceous sand having a nominal maximum size of $^3/_8$ in., and containing quartz, quartzite, ferruginous rocks, and mafic minerals (Figures 2 through 9).

Properties (Color, Angularity, Density, Hardness, Shape, Alteration, Coating, Cracking) – Particles are subrounded, subangular, variably dense, moderately hard, variably colored, light to dark gray, reddish brown, brown, clear, white, massive-textured, equidimensional, unaltered, and uncoated.

Gradation and Distribution – Fine aggregate particles are well-graded and well-distributed (Figures 3, 4, and 5).

Alkali-Aggregate Reactivity – There is no evidence of such a reaction of fine aggregate in the concrete.

3.2.3 Paste

Properties (Color, Hardness, Porosity, Luster) – Paste is medium gray, dense and hard. Freshly fractured surfaces have subvitreous lusters and subconchoidal textures.

Residual and Relict Portland Cement Particles – Residual and relict portland cement particles are present and estimated to constitute 4 to 6 of the paste volume (Figure 9).

Calcium Hydroxide – The calcium hydroxide component of cement hydration occurs as small, platy, patchy units and is estimated to constitute 4 to 5 percent of the paste volume.

Degree of Cement Hydration – Hydration of portland cement is normal.

Pozzolanic and Cementitious Materials – Besides portland cement, there is no evidence of any pozzolanic or cementitious admixtures found in the concrete.

Estimated *w/c* and Portland Cement Contents – The textural and compositional features of the paste are indicative of a portland cement content estimated to be $5^1/_2$ to 6 bags per cubic yard and a water-cement ratio estimated to be 0.40 to 0.45.

Secondary Deposits – Alkali silica reaction gel is detected within some voids and microcracks associated with a few reactive gravel particles (Figure 9).

Depth of Carbonation – Carbonation has extended to a maximum depth of 5 mm from the top exposed surface.

Aggregate-Paste Bond – Bond between the aggregate particles and paste is moderately weak to weak at the locations of cracks.

Microcracking – There is evidence of microcracking due to cyclic freezing and thawing and alkali silica reaction (Figures 4 through 10).

3.2.4 Air

Air occurs as a few coarse, near-spherical and irregularly-shaped voids having sizes up to 3 mm, which are characteristic of entrapped air.

Concrete in the core is non-air-entrained having an air content estimated to be less than one percent (Figure 6).

3.3 CHLORIDE ANALYSES

Approximately $^{1}/_{2}$ in. thick sections saw-cut from the top exposed end, the mid-depth location, and the bottom end of the core were analyzed for acid-soluble chloride contents.

Results are given in the attached Table.

CONSTRUCTION MATERIALS CONSULTANTS, INC.

Dipayan Jana, PG
President, Petrographer

DJ:jlh

Table: Data of Chloride Contents of Concrete Core
at the top (exposed surface region), mid-depth, and bottom locations

Sample ID	Location	Approximate Depth from the Exposed Surface (in.)	Percent Chloride by Mass of Sample	Percent Chloride by Mass of Cement
B-10 Northbound	Top Exposed Surface Region	0 to $^{1}/_{2}$	0.209	1.39
	Mid-depth location	$3^{3}/_{4}$ to $4^{1}/_{4}$	0.083	0.55
	Bottom End	$7^{3}/_{4}$ to $8^{1}/_{4}$	0.046	0.30

[1]Assuming a cement content of 15 percent by mass of concrete

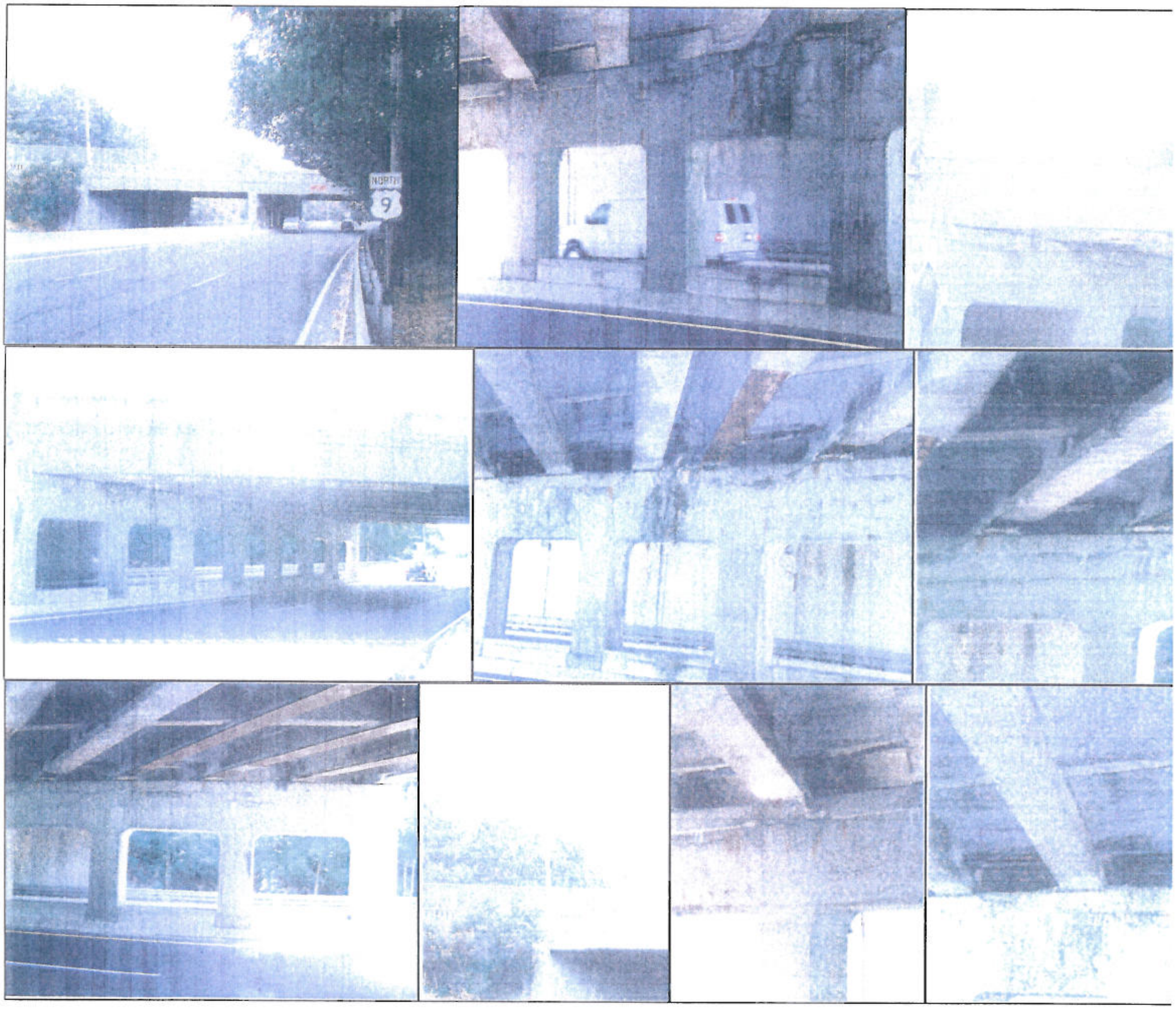


Figure 1: Field photographs showing the condition of the concrete overpass along with cracking at various locations of concrete.

Figure 2: Shown are: (a) the top, exposed surface of concrete with map cracking (top left); (b) the bottom fresh fractured surface of concrete, (top right); and (c) the side view of Core B-10 showing a crack at mid-depth location that broke the core into two halves (bottom photo).

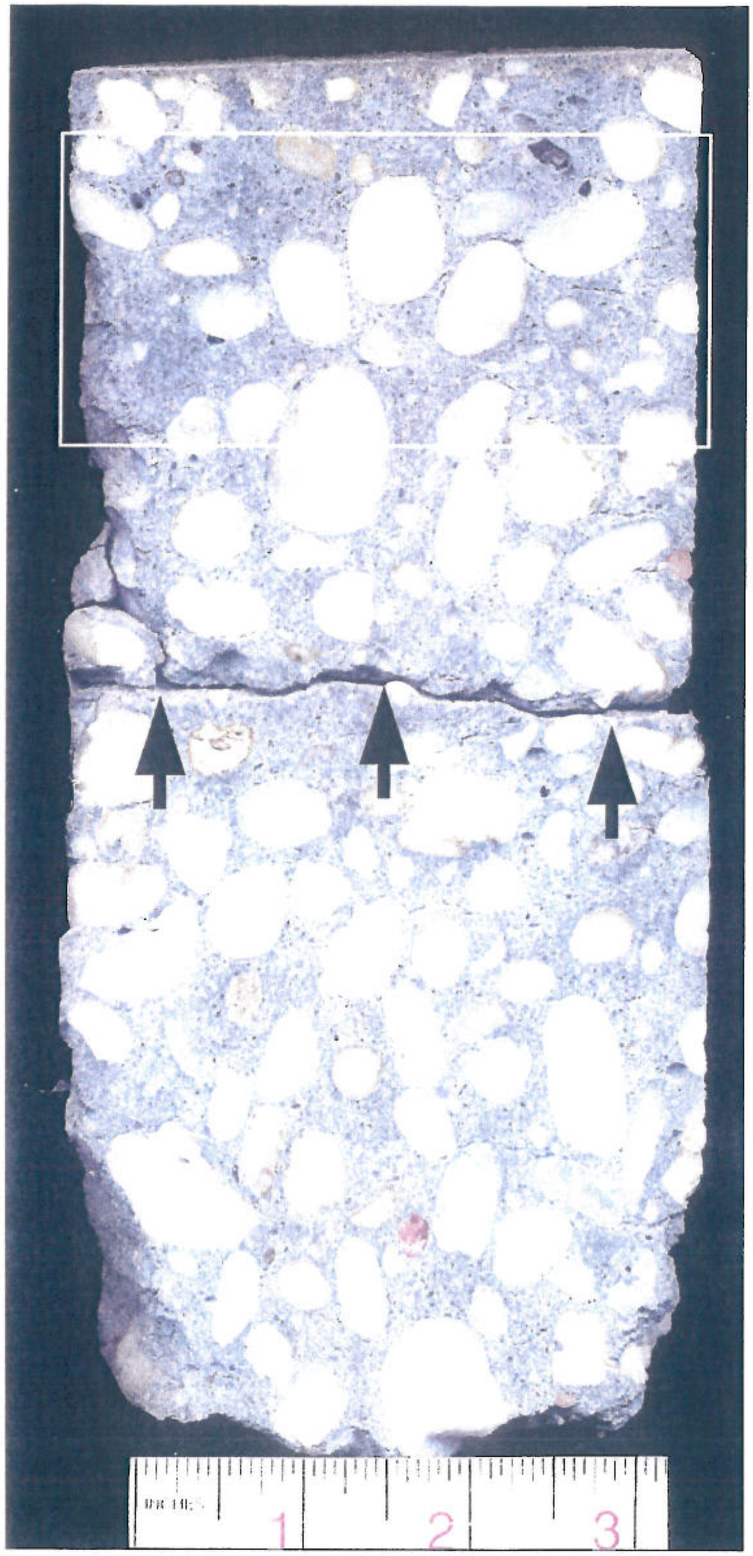


Figure 3: Lapped cross section of Core B-10 showing: (a) the well-distribution and overall good grading of the aggregate particles; (b) a crack at a depth of $3^3/4$ in. (arrows), which broke the core into two pieces. The boxed area is enlarged in the next figure.

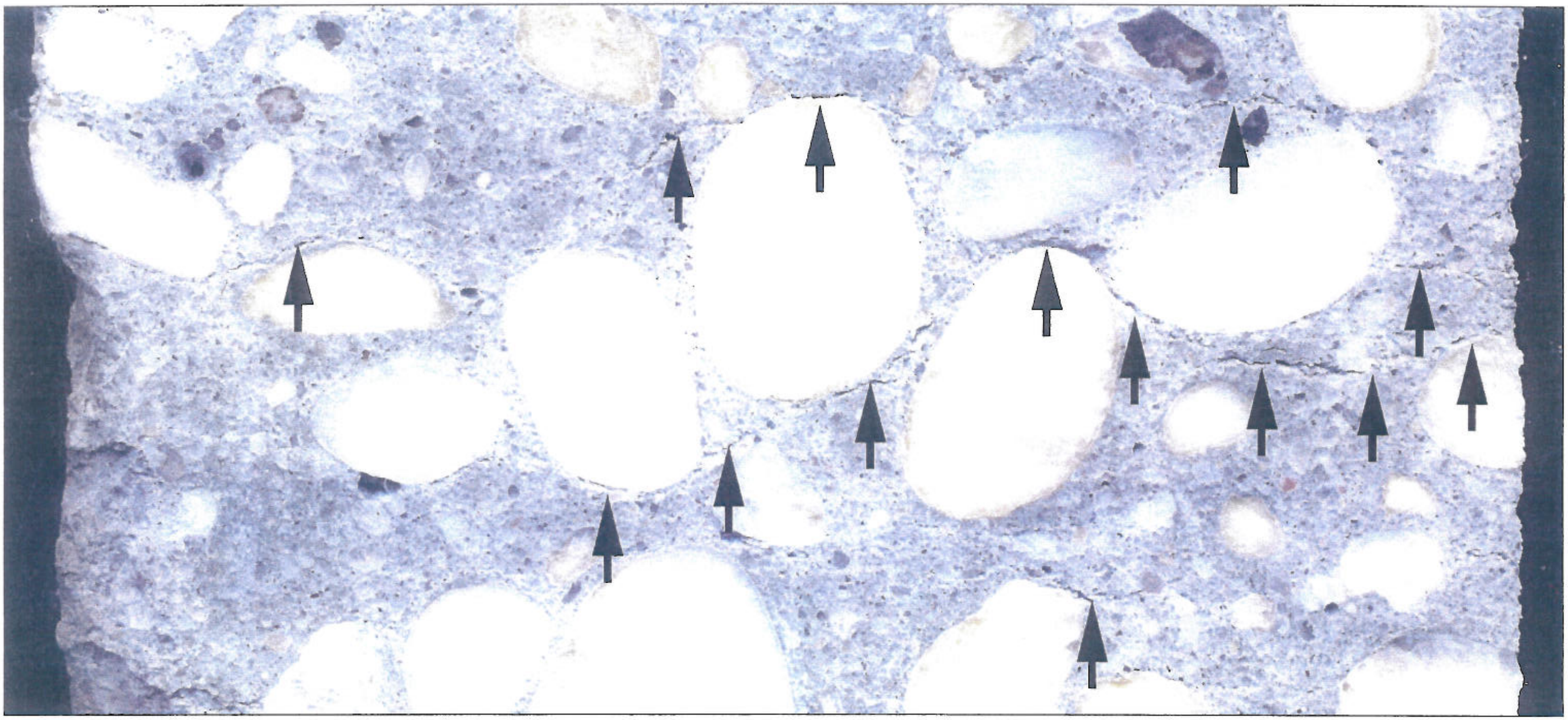


Figure 4: Enlarged view of the boxed area in Figure 3 showing macro and microcracking (arrows) through out the top half to the mid-depth location of the concrete core, and the resulting weak aggregate-paste bonds.

Figure 5: Plan view (top photo) and corresponding cross sectional view (bottom photo) of the core showing the pattern of map cracking at the surface and subparallel cracks in the body of the concrete transecting and circumscribing the aggregate particles.

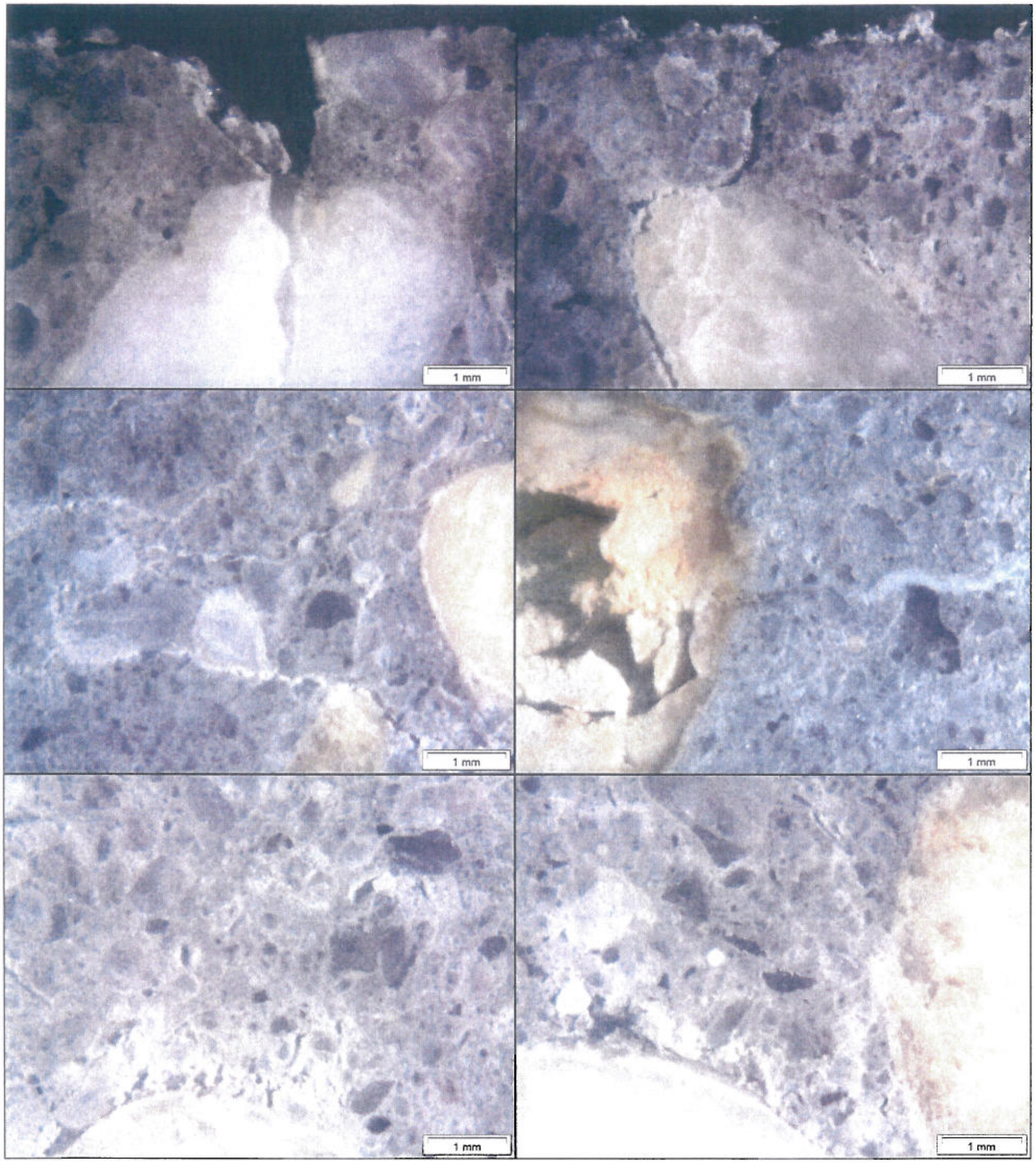


Figure 6: Photomicrographs of lapped cross section of Core B-10 showing: (a) micro and macrocracks at the top surface region, which have transected and circumscribed the aggregate particles (top photos); (b) micro and macrocracking throughout the core (middle photos); (c) the weak aggregate-paste bonds; and (d) the non-air-entrained nature of the concrete (all photos).

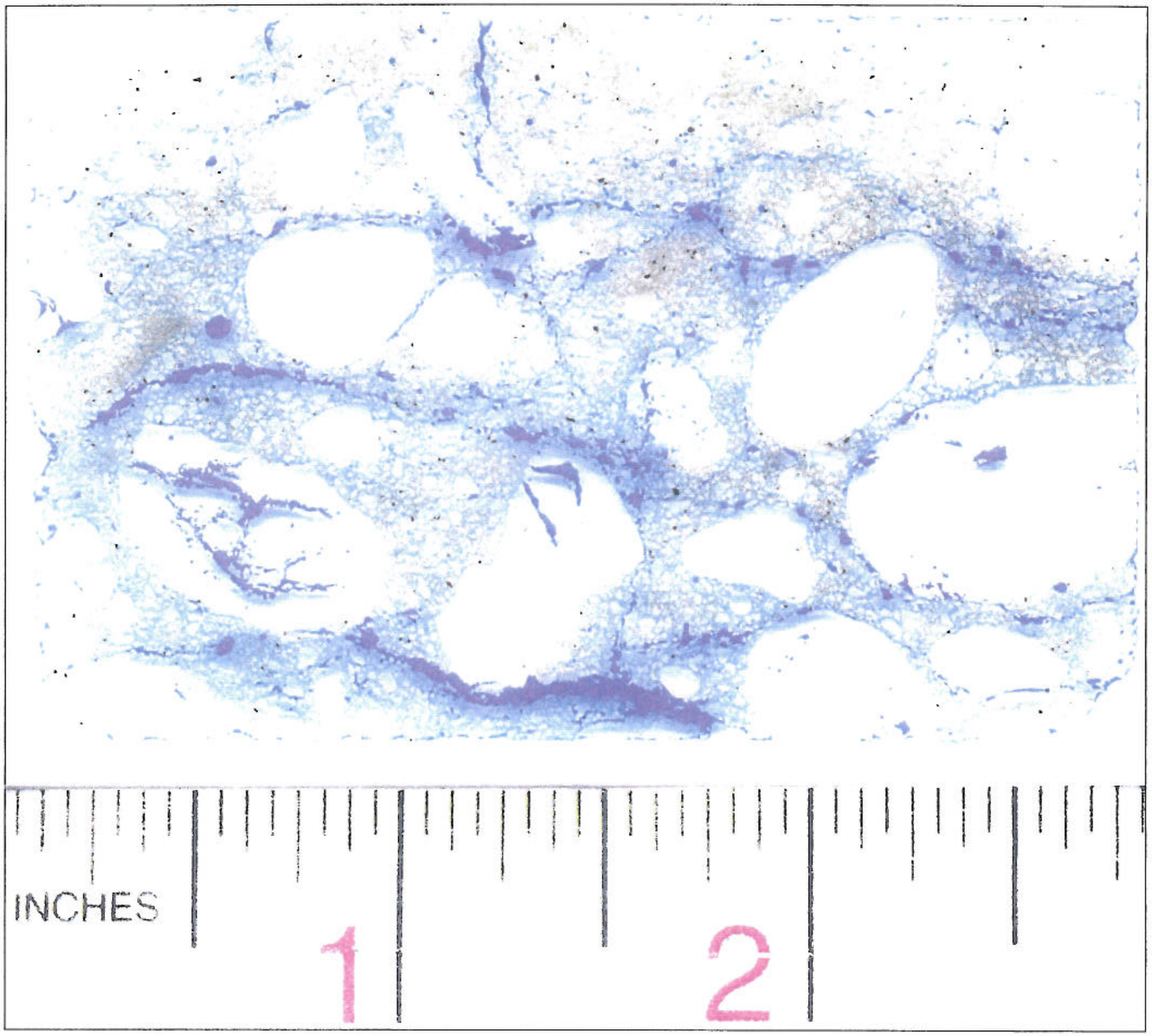


Figure 7: Blue dye-mixed epoxy-impregnated thin section of Core B-10 showing severe microcracking at the top surface region and in the body, as highlighted by the blue dye-mixed epoxy used to impregnate the sample.

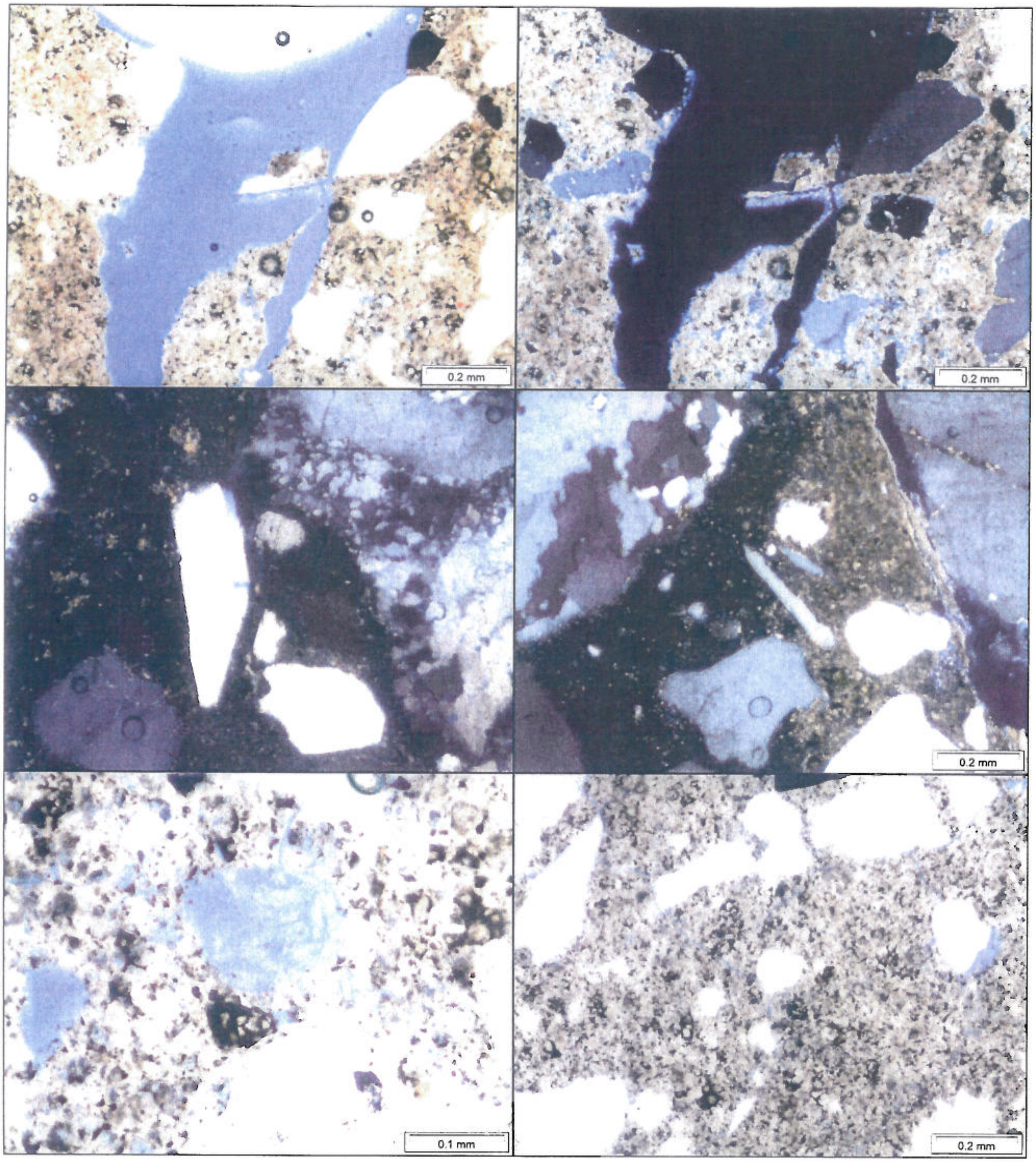


Figure 8: Photomicrographs of blue dye-mixed epoxy-impregnated thin section of Core B-10 showing: (a) microcracking at the top carbonated surface region (top photos); (b) the gravel coarse aggregate and natural silicious sand fine aggregate (middle photos); (c) the non-air-entrained nature of the concrete (bottom left); and (d) residual portland cement particles present in the paste (bottom right).

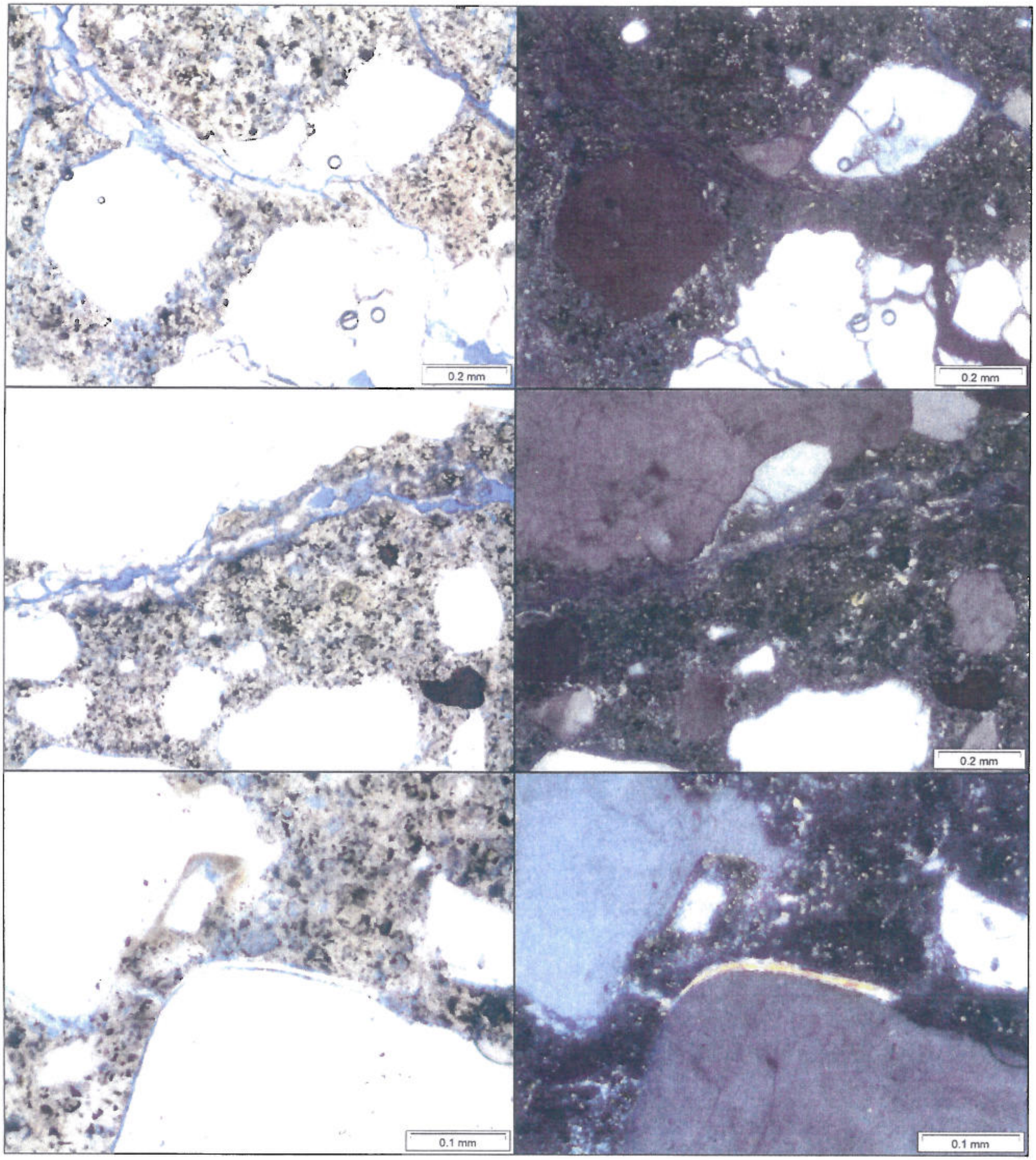


Figure 9: Photomicrographs of blue dye-mixed epoxy-impregnated thin section of Core B-10 showing microcracking throughout the core associated with alkali-silica reaction gel within the cracks.

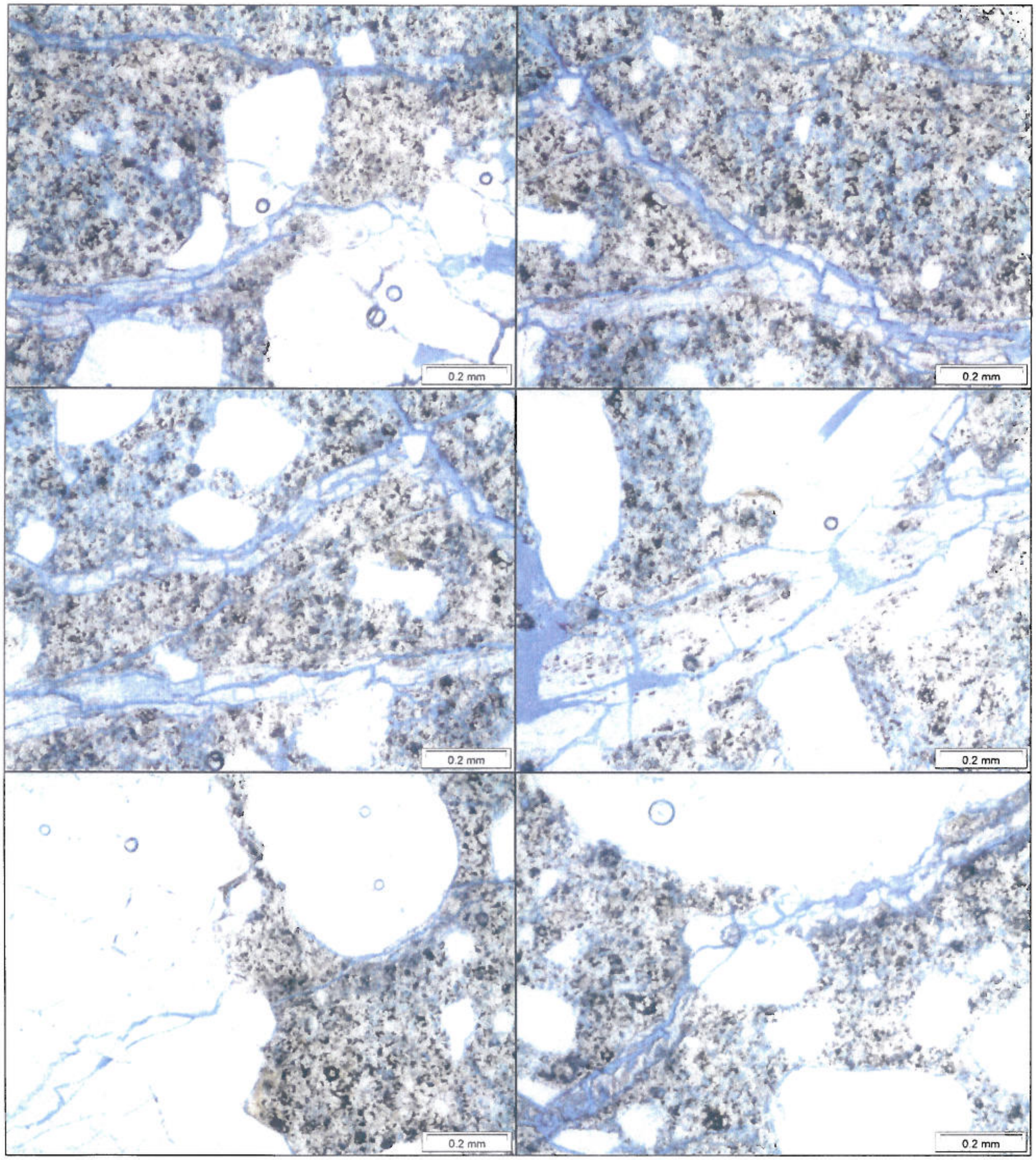


Figure 9 (Cont'd): Photomicrographs of blue dye-mixed epoxy-impregnated thin section of Core B-10 showing microcracking throughout the core associated with ASR within the cracks.