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60 Northrup Dr
Brentwood, NH 03833
Phone: (603) 772-1417
Fax: (603) 772-1418

668 Main Street, Suite 250
Wilmington, MA 01887
Phone: (978) 988-2115
Fax: (978) 988-2117

ASFE PEER REVIEWED 2007

March 31, 2010

Department of Public Works
Town of Durham
100 Stone Quarry Drive
Durham, New Hampshire 03824
Attention: Mr. Dave Cedarholm

Re: Concrete Evaluation Report
Oyster River (aka Mill Pond) Dam #071.03
Durham, New Hampshire
SA Project No. 075-07-003

Ladies and gentlemen:

The attached Report presents the results of Oyster River Dam concrete evaluation provided by Stephens Associates Consulting Engineers, LLC (SA) for the Subject Project. This Report has been prepared in general conformance with our Agreement for these services, and is subject to the provisions of that Agreement and the limitations presented throughout the Report, including Text, Figures, Tables, and Appendices.

We trust that this Report meets your current needs, and appreciate the opportunity to assist you on this Project. Please contact us if you have any questions.

Sincerely,
Stephens Associates Consulting Engineers, LLC

Bethel A H Stephens

Bethel A.H. Stephens
Principal Engineer

BAS:tgbg

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CONCRETE EVALUATION REPORT
OYSTER RIVER DAM #071.03
aka Mill Pond Dam, aka Durham Falls Dam
DURHAM, NEW HAMPSHIRE
SA PROJECT NO. 075-07-003
MARCH 31, 2010

Prepared for:

TOWN OF DURHAM, DEPARTMENT OF PUBLIC WORKS
100 Stone Quarry Drive
Durham, New Hampshire 03824

Prepared by:

Stephens Associates Consulting Engineers, LLC

Bethel A.H. Stephens, PE
Principal Engineer

CONCRETE EVALUATION R EPORT
OYSTER RIVER DAM
DURHAM, NEW HAMPSHIRE

TABLE OF CONTENTS

CONCRETE EVALUATION REPORT
OYSTER RIVER DAM
DURHAM, NEW HAMPSHIRE

1. INTRODUCTION

This Report ("Report") provides the results of the concrete evaluation performed by Stephens Associates Consulting Engineers, LLC ("SA," "we," "our," or "us") for the Town of Durham ("Town," "Client," "you," "your," etc.) of the Oyster River Dam (aka Mill Pond Dam) in Durham, New Hampshire. This Report is subject to the limitations presented herein, including Figures, Tables and Appendices. SA performed these services for the Town in general accordance with Amendment 2, dated November 13, 2009 and authorized November 16, 2009, to our Agreement dated April 1, 2008 and authorized May 25, 2009. Professor David Gress, PhD, PE participated in several aspects of the evaluation under separate agreement with the Town. Professor Gress's participation is generally described herein.

This Report first describes our Project understanding, background, and scope of services. We then describe existing conditions, coring observations and results of concrete laboratory testing, followed by SA's evaluation and conclusions.

1.1 Project Understanding and Background

Much of our understanding of the Project is presented in our existing Agreement for the Project, supplemented by our Dam Evaluation Report dated March 17, 2009 prepared under that Agreement, and our subsequent correspondence/discussion with Mr. David Cedarholm, PE, Town Engineer for the Town of Durham.

We understand from our email correspondence and discussions with Mr. Cedarholm that the Town wanted an engineering evaluation of the condition of the existing structural concrete of the Dam in consideration of the Town's near-future approach to addressing the deficiencies noted by SA in our March 17, 2009 Report.

We understand that concrete repairs were needed to address the New Hampshire Department of Environmental Services (NHDES) Dams Bureau December 10, 2002 Letter of Deficiency (LOD) at the locations identified in the LOD. Since then NHDES has lowered the hazard classification of the Dam to Class A. We are uncertain as to the current applicability of the deficiencies cited in the LOD. The Town, however, is considering the preservation of the Dam by repairs as a Town resource.

In our opinion, the Dam will continue to function safely as a dam for 5, and perhaps as long as 10, years without concrete repairs. If the Town's goal is to repair the Dam for 5 to 10 year lifespan, the Town may consider repairing the gates to be operable, providing overtopping protection for the right abutment, and (perhaps) repairing cosmetic concrete. In our opinion, structural repair of the concrete is not necessary for repairs intended for a 5 to 10 year lifespan.

Inasmuch as SA's recommended repairs to the Dam, especially to structural concrete, for 20- to 25-year lifespan would be more than those lasting 5 to 10 years, and given the Town's goals as stated in SA's email correspondence with Mr. Cedarholm, we recommended evaluating the *type (the nature)*, *extent (severity and amount) and cause* of concrete deterioration, to be used in determining the amount of concrete removal for

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design of the longer-term (20- to 25-year) repair. Our scope of services, described below, is structured accordingly.

The Town retained Professor David Gress, PE of the University of New Hampshire to consult with the Town and SA on the concrete evaluation. Professor Gress helped with concrete soundings and coring observations, performed laboratory testing of concrete samples, and consulted with the Town and SA on laboratory test results and conclusions.

SA evaluated the concrete at representative areas of the upstream face, crest, downstream face, ribs and gate structure at the right abutment of the Dam. The left abutment consists of a fish ladder owned by NH Fish and Game and at the request of the Town, was not included in our evaluation. Town and public records and photographs indicate significant repairs were made to the crest and downstream spillway lip/face of the Dam in 1974 (apparently dedicated in 1975). Areas of the supporting ribs and the gate structure at the right abutment were also previously repaired. For the purpose of this report, we refer to the concrete from the 1913 construction as "original" and the newer concrete from the 1974 repairs as "1974." Since SA does not know of any repairs made to the dam other than the 1974 rehabilitation, we assumed that any observed overlays or repair patches were constructed in 1974. Comments that do not reference either "original" or "1974" apply to both concretes.

Detailed descriptions of the Dam location, dimensions and history may be found in our Dam Evaluation Report dated March 17, 2009.

1.2 Purpose and Scope of Services

The purpose of our services was to evaluate the type, extent and cause of concrete deterioration to assist us in future design of repairs to meet the Town's desired lifespan of 20 to 25 years, and in estimating costs of those repairs, under later amendment to our Agreement.

Our scope of services may be summarized as follows:

1. SA planned and coordinated our field services to evaluate the concrete.
2. SA retained a subcontractor to perform ground penetrating radar, or GPR, non-destructive testing (NDT) to identify reinforcing steel location at selected areas of the Dam prior to obtaining the concrete cores described below. SA observed the NDT and selected potential locations for concrete cores while on-Site and in consultation with Professor Gress. SA engaged a concrete coring subcontractor ("Procut") to drill 4- and 6-inch-diameter concrete cores of the Dam at selected locations. The purpose of the cores was to obtain samples for field examination and for use in laboratory testing. One location on the spillway upstream face was intentionally chosen for coring through the reinforcing steel to evaluate the condition of the steel.
3. SA evaluated the concrete surface for extent (i.e. area) of concrete deterioration that may need repair. SA used visual observations and sounding with a common hammer for this evaluation. Professor Gress assisted SA with sounding the spillway upstream face and crest.
4. SA prepared this Report of Results and Conclusions based on the data obtained in our evaluation and the laboratory test results.

2. EXISTING CONDITIONS

2.1 Existing Condition Observations

SA visited the Site on November 19, 23 and 25, 2009 to observe the existing condition of the Dam concrete. The Dam impoundment had been dewatered by the Town by opening of the gates (low-level outlets), exposing and providing access to the upstream face and inside of the cells. SA visually observed and photographed the concrete, noting areas of visible concrete damage and deterioration on the top surface and underside of the slab and on the supporting piers, or "ribs," of the Dam. Appendix A contains photographs of typical conditions. Detailed descriptions of our previous visual observations, crack locations, etc. can be found in our Dam Evaluation Report.

The upstream face of the Dam spillway consists of an approximately 12-inch-thick, sloping slab that spans across vertical, 12-inch-thick piers, or "ribs." The spillway crest profile is curved, with a walking surface about two feet wide, and the downstream face consists of a sloped/rounded lip above the vertical ribs. The combination of sloping slabs and ribs create interior "cells," which we were able to observe and sound while the impoundment was dewatered. Cells are numbered 1 through 9 from right to left looking downstream. See Figure 1.

With assistance from Professor Gress, SA sounded the surface of the upstream spillway face and crest, interior of each cell and the downstream face of the Dam at the gate structure. Surface sounding is used to detect delamination of concrete, which is a cracking or separation of a layer of concrete roughly parallel to the surface. We used a common hammer to strike the surface of the concrete, and listened to the characteristics of the resulting sound. Professor Gress used a cross pein hammer. We used his cross pein hammer attached to the end of a pole to sound hard-to-reach areas. Hollow or dull sounds indicate areas where concrete has likely delaminated or significantly deteriorated while higher-pitched ringing indicates sound concrete. The upstream face of the Dam gate structure (between the right abutment and the spillway) was visually observed but not sounded, as it was not readily accessible even after dewatering.

Upstream Spillway Face:

Less than half of the height of the upstream face was visible at the left end of the Dam due to sedimentation against the Dam. The sediment build-up gradually decreased in height across the length of the spillway from left to right. In general the exposed surface of the upstream face of the Dam appears to be in good condition. In many areas SA observed evidence of a thin layer of repair mortar (about 1/8-inch thick), much of which had spalled off or was easily knocked off with a standard hammer. We observed small aggregate (about ½" to ¾" in diameter) at the surface of the concrete below the coating of repair mortar, and noted a small spall at the joint between the slab panels of Cells 3 and 4.

We observed visible "sags" of about ½ to ¾ of an inch between the ribs of the upstream spillway face. We checked for, but did not observe, similar, analogous "sags" or flexural cracking on the interior face, indicating that this phenomenon is likely due to the way the slab was formed and poured during construction, not due to creep or other structural distress.

Through surface sounding, SA detected a few areas of possible delamination at the upstream face. An area about one foot above the water line and about one foot in diameter on the upstream face of Cell 1 sounded hollow. We were able to dislodge about a ¼-inch thick piece of the concrete and the concrete below sounded

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good. Horizontal surface lines at cold joints on the upstream face of both Cells 1 and 2 sounded like there could be delamination or cracking beneath the surface along the joint.

We observed the joint between the original concrete and the 1974 concrete from the spillway upstream face and noted a significant crack between the two concretes near the joint between Cells 6 and 7.

Spillway Crest and Downstream Lip/Face:

The Dam spillway crest and downstream lip/face of the Dam were repaired in 1974 with a layer of repair concrete along the length of the Dam spillway, which extended over the lip/face. SA observed some light surface scaling along the spillway crest and lip/face of the Dam. Moss and algae are growing on the downstream face of the Dam. When struck with a hammer using slightly more force than a typical sounding, a thin layer of the surface concrete would typically crumble, which is uncharacteristic of sound concrete.

Sounding indicated a possible delamination along the joint between Cells 5 and 6 along the crest. At the joint above the rib located between Cells 6 and 7, the corner of the repair material over Cell 6 sounded delaminated. We also noted a hollow sounding area extended from the joint up to 4 feet over Cell 7.

Ribs and Cell Interiors:

The supporting ribs and the cell interiors exhibit a significant amount of cracking, spalling and efflorescence staining as described in our Dam Evaluation Report. In general, we observed erosion near the waterline of the downstream ends of the ribs. 1974 repair concrete appeared to have spalled from previously repaired ribs. Three of the ribs sounded delaminated above a crack that ran parallel to and about 2 inches below the upstream ceiling. Two ribs also appeared to have small delaminations near the intersection of the rib and the front face of the Dam, or lip.

Sounding revealed that Cell 1 contained delaminated areas of patch material, and of the original concrete adjacent to the patch material, over a significant area of the upstream ceiling/slab. Five of the cells had significant delaminated areas in the downstream ceiling or along the lip. Some cells had delaminations of what appears to be the repair concrete, while other delaminated areas were directly adjacent to the patches in the original concrete.

Dam/Gate Structure at Right Abutment:

The downstream wall of Dam at the gate structures (between the right abutment and the spillway) contains several areas of cracking, spalling and efflorescence as are described in our Dam Evaluation Report. We observed an apparent joint between two types of concrete about 5 feet up from the base of the downstream face of the Dam at the gate structure. A large area directly below this line sounded hollow when tapped indicating a large delamination. The top of the abutment appeared sound except for a small area near an existing spall. As stated above, the upstream face of the Dam near the gates/right abutment was visually observed but not sounded, as it was not accessible even after dewatering.

2.2 Concrete Scanning and Core Drilling Observations

On November 19, 2009, SA engaged the concrete scanning services of ProScanning, Inc. of Boston, MA ("Proscanning"). ProScanning used ground-penetrating radar to attempt to locate, and to mark locations of, existing reinforcing in the concrete structure at six separate 10- to 60-square-foot areas. The purpose of the scanning was to reduce the risk of unwanted coring through existing reinforcing and to verify the approximate location and spacing of the existing reinforcing steel for possible later evaluation.

SA engaged Procut of New Hampshire ("Procut") to drill cores at the Dam on November 23 and 25, 2009. ProCut drilled 13 four- or six-inch-diameter cores in areas marked by ProScanning and/or selected by Professor Gress/SA. The cores ranged from 4 inches to 11 inches long. SA observed the coring and photographed the cores. Professor Gress wrapped the cores and transported them for laboratory testing. Procut patched the core holes with fast-setting cementicious material. Because of logistical limitations, Procut cored the locations on the Dam face near the gates during refilling of the impoundment. The added water elevation combined with the cracking at Core #13 led to water discharge from the open core hole, creating a temporary difficulty in cementing the hole. Professor Gress obtained a fast-setting paste from his laboratory, but the paste was unsuccessful so Professor Gress expeditiously procured a rubber plug at a nearby hardware store. Procut used the plug to seal the water and cemented the hole. Approximate core locations are shown on the Core Location Drawings in Figures 1 through 3. See Table 1 for detailed descriptions of the concrete core field observations. Our generalized description of concrete cores is as follows:

Top Spillway Crest:

Four 4-inch diameter (Cores #1, 2, 3 and 5) and one 6-inch diameter (Core #6) cores were taken through the crest of the Dam spillway above Cell 6. Lengths varied from 4 inches to 12 inches. We switched to the 6-inch-diameter cores since large aggregate (greater than 2-inch diameter) was observed in the 4-inch cores and the ratio of core diameter to aggregate size should be at least 2 to 1 for accurate concrete physical property testing. The bottom edge of one 12-inch-long core penetrated through the ceiling of the cell below, indicating that the spillway crest is about 12 inches thick. The top 3 to 4 inches appear to be the repair concrete from the 1974 Dam rehabilitation. SA observed very large, mostly rounded aggregate (up to about 3-inch diameter) in the original concrete at the bottom of the cores. We observed few visible cracks along the edges of the cores. Procut encountered what was assumed to be a steel reinforcing bar at about 10 inches below the surface at Core #6. Drilling was stopped before the bar was cut. Proscanning had indicated that steel was probably located along the bottom of the spillway crest, but that it was difficult to get clear readings for the location of the steel because the concrete was not dry enough.

Upstream Spillway Face:

Proscanning scanned the slab over Cell 7 and above the rib between Cells 6 and 7, from the upstream/top side of the slab, as representative of other portions of the structure and because it was accessible to scanning and coring. Proscanning detected horizontal reinforcing bars near the underside of the slab spaced at about 6 inches on center and two vertical bars about 4 inches apart over the support/rib. The spacing of the horizontal bars detected by Proscanning differed from that of photographs taken during construction of the Dam, which indicated bars spaced at about 12 inches on center. We chose a location over a rib to intentionally drill through the reinforcing to observe its condition (Core #4). We selected that location because positive bending moment is lowest there. At about 11 inches below the surface the driller cored through a steel bar and the core broke off at what appeared to be a horizontal cold joint between the slab and the supporting rib. SA observed that the steel reinforcing bar was about ¾" square and did not show significant corrosion or deterioration. An approximately 11-inch-long core was taken through the same hole into the rib directly below the upstream face core location. Another square bar was encountered about 1 to 2 inches into the rib. Each core was removed in one solid piece with no significant visible cracks.

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Cell Ribs:

Proscanning scanned the majority of the rib between Cells 7 and 8 as representative of the structure and accessible to scanning and coring equipment. The ground penetrating radar only detected reinforcing steel at the downstream vertical edges of the rib. Procut took three 6-inch-diameter cores (Cores #7, #8 and #9) horizontally through the rib between Cells 7 and 8. The cores were intact (no visible cracking and no steel) and were about 12 inches long. The aggregate appeared to be well distributed throughout the cores, appearing mostly rounded with maximum size close to 3 inches in diameter.

Dam/Gate Structure at Right Abutment:

Procut took four 6-inch-diameter cores in the downstream face of the Dam near the gates for the low level outlets next to the right abutment. Procut took the cores two to six feet above the concrete pad in the center of the base of the wall (at the right gate). Of the four cores, they obtained two intact, about 12-inch-long cores of original concrete (Cores #10 and #12), one core through original concrete that came out of the hole in several pieces (Core #11), and one 7-inch-long core of repair concrete (Core #13). The original concrete contained large, rounded aggregate similar to that observed at the Spillway crest and ribs. The repair concrete was cementicious material with small aggregate, up to about ¼" in diameter.

Cores #10 and #12 had significant cracking parallel to the face of the Dam, which SA observed both in the cores and in the core holes. Core #11 was in fragments and contained a piece of square reinforcing steel about 3 inches from the concrete surface. SA observed brown staining of the concrete in the plane of the steel, but the bar appeared relatively free of corrosion. We observed several significant cracks, up to 1/8-inch wide and parallel to the face of the Dam, inside the core hole, as well as a couple of small cracks perpendicular to the face extending about 3 inches into the Dam from the surface.

Core #13 came from just below a visible horizontal line between the original concrete (above) and a concrete patch (below) through an area of the wall that sounded delaminated. The outer 7 inches of 1974 concrete came out of the core hole intact with no visible cracking. The concrete scanner had indicated that the concrete in this area appeared to be delaminated. SA observed space between the original and 1974 concrete, about ¼-inch-wide, at the back of the core hole. We observed a horizontal square steel bar in the core about 3 ½ inches from the surface. The gap we observed between the 1974 and original concretes at the back of the Core #13 hole indicates that the patch is likely being held in place by the existing reinforcing steel.

3. CONCRETE LABORATORY TESTING

Professor Gress took possession of the cores on site and transported them to the laboratory for physical property testing and petrographic analysis. Professor Gress performed physical tests including splitting tensile strength (3 cores, 6 tests total), compression strength (4 core locations, 5 tests total), elastic modulus (2 core locations, 3 tests total) and unit weight and absorption (1 core location, 2 tests total). The cores from the downstream face of the Dam near the gates at the right abutment were not tested for physical properties. Based on our discussions with Professor Gress, we understand that he judged the concrete in the core samples taken from that area as too deteriorated for such testing. Results of the physical properties testing are shown in Tables 2 through 6.

Tested tensile strength of the original concrete ranged from 240 psi to 580 psi, with an average of 380 psi. Tested compressive strength for the original concrete ranged from 4730 psi to 6960 psi with an average of 5780 psi. The 1974 concrete was not tested for compressive strength since the sample sizes were not long enough to meet the length to diameter ratio needed according to ASTM standards. One sample of 1974 concrete was tested for tensile strength with a resulting value of 330 psi. The elastic modulus testing resulted in a range of values for the original concrete of 3650 ksi to 4130 ksi, with an average of 3770 ksi. Again, no samples of the 1974 concrete were tested for elastic modulus due to inadequate sample size.

Professor Gress tested Core #4 for unit weight and absorption, testing both the upstream face slab and rib parts of the core. Unit weight values ranged from 139 to 142 pounds per cubic foot. The absorption values were 2.4 and 3.6 percent for the upstream face and rib samples, respectively. Absorption measures the concrete's ability to absorb moisture, and is typically expected to be about 5 percent. Values higher than expected would indicate poor quality concrete. Possible reasons for lower than expected values could include the large aggregate size and the presence of ASR gel in the voids.

Seven cores selected by Professor Gress were cut and polished for microscopic examination (i.e., petrographic study). According to Professor Gress's report (Appendix B), the examination of the original concrete revealed significant amounts of microcracking and macrocracking of the concrete, both in the cementicious materials and in the aggregates. Many of the aggregate particles exhibited reaction rims indicative of Alkali Silica Reaction (ASR). Pieces of mica, most likely from the aggregate, were observed throughout the concrete matrix, indicating that many of the large aggregate pieces are weak and deteriorated. Examination of the interface between the original and 1974 repair concretes in Cores #5 and #6 show that the mortar at the interface was poorly consolidated and that the 1974 concrete is delaminating from the original. Based on our discussions with Professor Gress, it is his opinion that this is not a significant factor in the concrete deterioration.

Thin sections were made of Core #10, from the downstream face of the gate structure at the right abutment, for viewing with a petrographic microscope. Petrographic analysis revealed evidence of ASR and extensive cracking in both the aggregate and the cement paste. Multiple layers of ASR products indicate that the reaction has likely been active for a long time period.

Three cores were tested for the presence of ASR gel by applying a solution of Uranyl Acetate Dihydrate to the split surface of the concrete after the tensile strength testing. When viewed under ultraviolet light, ASR gel appears as a greenish glow. The tested samples of original concrete show significant signs of ASR, while the samples of 1974 concrete showed very small amounts. According to Professor Gress, some ASR is typical of most concretes in New England because of the types of natural aggregates found there.

More detailed discussion of concrete testing procedures and results can be found in Professor Gress's Report titled "Evaluation of the Concrete of the Oyster River Durham Falls Dam Concrete," attached as Appendix B. Since Dr. Gress did not suspect ASR damage initially, based on visual examination of the Dam, the test to determine if ASR is ongoing was not performed. This test should be performed prior to finalizing the design of any repairs to aid in final repair method and material selection.

4. EVALUATION AND CONCLUSIONS

In evaluating options, the Owner's objectives for the continued use of the structure need to be considered in the context of the prospective useful life of the repairs and repaired structure, along with cost considerations. Although it may continue to function safely for another 5 to 10 years without repair, it is probable that the Dam is approaching the end of its economical and useful service life.

Based on visual observations of the dam, the concrete appeared to be in relatively good condition for its age. The results of the laboratory testing, however, combined with observations of the Dam and the cores, indicate that the original concrete, and to a lesser extent the 1974 concrete, is deteriorating from materials related distress (MRD) exhibited in the form of cracking, delaminating and spalling. Our evaluation indicated that the primary causes of the MRD are most likely Alkali Silica Reaction (ASR) and freeze-thaw damage. The scaling and pitting of the concrete surface observed on the Spillway crest and the downstream face of the ribs may also be due to erosion from the water flow and possibly biological growth of surface algae and moss.

ASR is caused by a chemical reaction between the alkali in the cement paste and silica in the concrete aggregate. The reaction causes the concrete to expand when exposed to moisture, leading to cracking. The service life and/or cost of repair will depend on whether or not ASR continues to make the concrete dimensionally unstable, as continued ASR will cause continued expansion leading to deterioration of traditional repairs of bonded overlay-type replacement concrete. Further laboratory testing is therefore needed to determine if the ASR is ongoing (i.e. the concrete is continuing to expand and deteriorate) or if the alkali has been depleted. When alkali in the concrete is depleted, ASR is complete and expansion of the concrete from ASR typically ceases. Repairs that bond to concrete where ASR is ongoing are susceptible to failing due to the expanding concrete behind it causing delamination of the repair material. Laboratory testing indicated that a significant amount of ASR has occurred in the original concrete, while the 1974 concrete showed evidence of only a small amount of ASR.

The laboratory results indicate the presence of extensive microcracking. Freeze-thaw damage occurs when water within the concrete cracks and pores freezes and expands causing additional microcracking, macrocracking and surface scaling/spalling. Under freeze-thaw conditions cracks can propagate as water infiltrates the new cracks and freezes and expands. Cycles of freezing and thawing may result in progressively deeper cracks. Petrographic analysis indicated that freeze-thaw damage has likely occurred in both the original and 1974 concrete. ASR and freeze-thaw damage typically exhibit similar crack patterns, and cracking caused by ASR allows frost action to occur more readily, often leading to accelerated concrete deterioration.

The results of concrete physical properties testing show that the concrete is adequately strong in compression. The relationship between concrete tensile and compressive strengths is not directly proportional, but the expected approximate range of tension to compression strength ratio in sound concrete is 7% to 10%. The ratio of average tensile to average compressive strength of the original concrete is 6.6%. Original concrete samples from the spillway crest and the rib between Cells 6 and 7 had tension to compression ratios of 8.2% and 6.0%, respectively.

Elastic modulus of sound concrete can be estimated as a function of compressive strength and unit weight using a numeric relationship from the American Concrete Institute (ACI). SA and Professor Gress used this relationship to estimate expected elastic moduli based on the compression tests. The measured elastic modulus of the cores ranged from 91% to 98% of the expected modulus calculated with the ACI equation using the measured unit weights and compressive strength of the cores. The expected elastic modulus of a concrete with an average compressive strength of 5780 psi would be about 4200 ksi, which is greater than the average tested modulus of 3770 ksi.

The fact that some of the cores have tensile strengths and elastic moduli slightly lower than expected could be due to microcracking resulting from MRD. It is SA's opinion that other factors, such as the strength, shape and mineralogy of the aggregate, may also be contributing to lower than expected values.

The concrete at downstream face of the gate structure near the right abutment exhibited the most severe damage and deterioration. The extensive cracking in the cores from this area made the cores unusable for strength testing. The abutment is exposed to water from the pond and river surface, rain and splashing, but unlike the spillway, which is protected from freezing by the water flowing over it, the downstream face concrete is continually exposed and thus much more susceptible to freeze-thaw damage. SA observed water infiltrating the cracks in the core hole from Core #13. This indicates that water from behind the Dam may be infiltrating the extensive cracks in the gate structure, which may freeze leading to more severe deterioration.

With the exception of the downstream face of the supporting ribs and a few spall locations in cell ceilings, corrosion of reinforcing steel does not appear to be a significant problem. Steel corrosion may be occurring in areas that sound delaminated; however, we observed no significant rust staining in these areas. Existing corrosion could propagate into adjacent steel at spalled areas where steel is exposed causing further delamination and spalling, as well as weakening of the structure, if the existing corrosion is not removed and the existing spalls patched.

Delamination of some areas of the concrete repairs from the 1974 rehabilitation, located by sounding and confirmed with coring and petrographic analysis, is likely due to a combination of factors, most notably base concrete expanding from ASR and causing the bond to fail. Delaminated concrete that is no longer bonded to the original concrete, and therefore not integral with the original concrete, cannot transfer structural loads between the original base concrete and the delaminated repair concrete. Areas of delamination are likely to spread as moisture gains access to the cracks in the plane of delamination and freezes, or if ASR is ongoing. Ultimately, areas of delamination are at risk of spalling, or detaching completely from the base concrete, exposing the original base concrete and any reinforcing steel below, and increasing the likelihood for additional deterioration.

As the Town evaluates its options for the continued use of the dam they should take into consideration several factors including the desired remaining service life of the Dam, the cost of present and future repairs and maintenance, the appearance of the structure and the consequences of structural failure if nothing is done.

5. LIMITATIONS

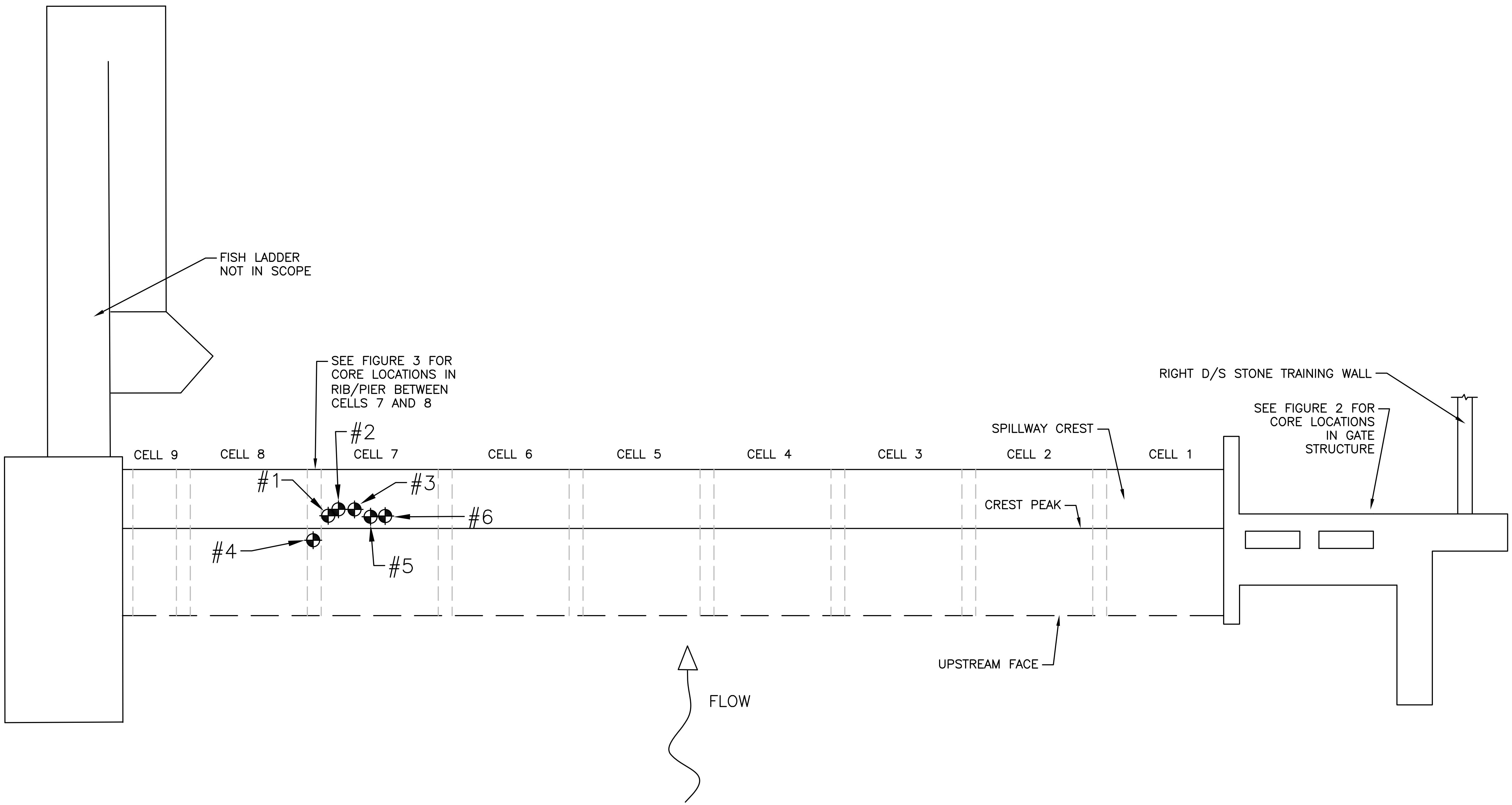
Stephens Associates Consulting Engineers, LLC (SA) has prepared this Report based on the information available to us at this time, including but not limited to, information furnished through the Client, the Owner and their representatives for the proposed Project. If any of the information noted herein is incorrect or has changed, SA should be notified and retained to review the corrections and changes and amend this Report. If SA is not retained for these purposes, we cannot be responsible for the impact of those conditions on the performance of the Project. Upon completion of plans and specifications, SA should be retained to review the final design documents before issuance for construction bid. This review will allow us to check that our engineering recommendations have been interpreted and implemented properly in the design. At that time, it may be necessary to submit supplementary recommendations, which SA will do on a time and expense basis according to our Agreement for the Project.

SA's scope of services did not include an environmental assessment of any kind, including but not limited to assessments for the presence or absence of wetlands or hazardous or toxic materials or organisms (e.g., fungi, flora, fauna, bacteria, viruses, etc.) in the soil, surface water, groundwater, or air, on or below or around this site. Any observations of odors, colors, or unusual or suspicious items or conditions noted by SA were incidental to our services, and any statements regarding such observations are strictly for the information of the Client.

We recommend that SA be retained to provide services during construction including assistance with shop drawing/submittal review and engineering observation of construction. These services will assist the Owner with quality assurance through observation of compliance with design concepts, specifications and recommendations and will allow for the implementation of design changes where necessary due to conditions that differ from those anticipated.

This Report has been prepared by SA for the exclusive use of the Client and for the specific application to the subject Project, as conceived at this time. The Report is for study and schematic design only, and by itself is not sufficient to prepare an accurate cost estimate or construction "bid." Subject to the limitations inherent in the agreed scope of work as to the degree of care, amount of time and expenses to be incurred, and subject to any other limitations contained in the Agreement for SA's services, SA has performed its services with the degree of care and skill ordinarily exercised by other professional engineers under similar circumstances at the time the services were performed. No warranties are implied or expressed.

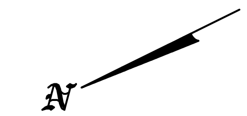
FIGURES



CORE LOCATION PLAN

NOT TO SCALE

INDICATES APPROXIMATE LOCATION AND NUMBER OF CONCRETE CORE



Not to Scale

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TOWN OF DURHAM
Department of Public Works
100 Stone Quarry Drive
Durham, NH 03824

CLIENT
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Department of Public Works
100 Stone Quarry Drive
Durham, NH 03824

Original Work:
By: J. Turner Date: 3-17-09
Checked By: ______ Date: ______

Final Figure:
By: NAO Date: 3-17-10
Checked By: BAS Date: 3-17-10

Revisions:
1. By: ______ Date: ______
2. By: ______ Date: ______
3. By: ______ Date: ______

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Project: Sheet 1 of 1
Number: 075-07-003
Name: **OYSTER RIVER DAM DURHAM, NH**

Subject: **FIGURE 1 - CONCRETE EVALUATION PLAN CORE #1 TO 6 LOCATIONS**

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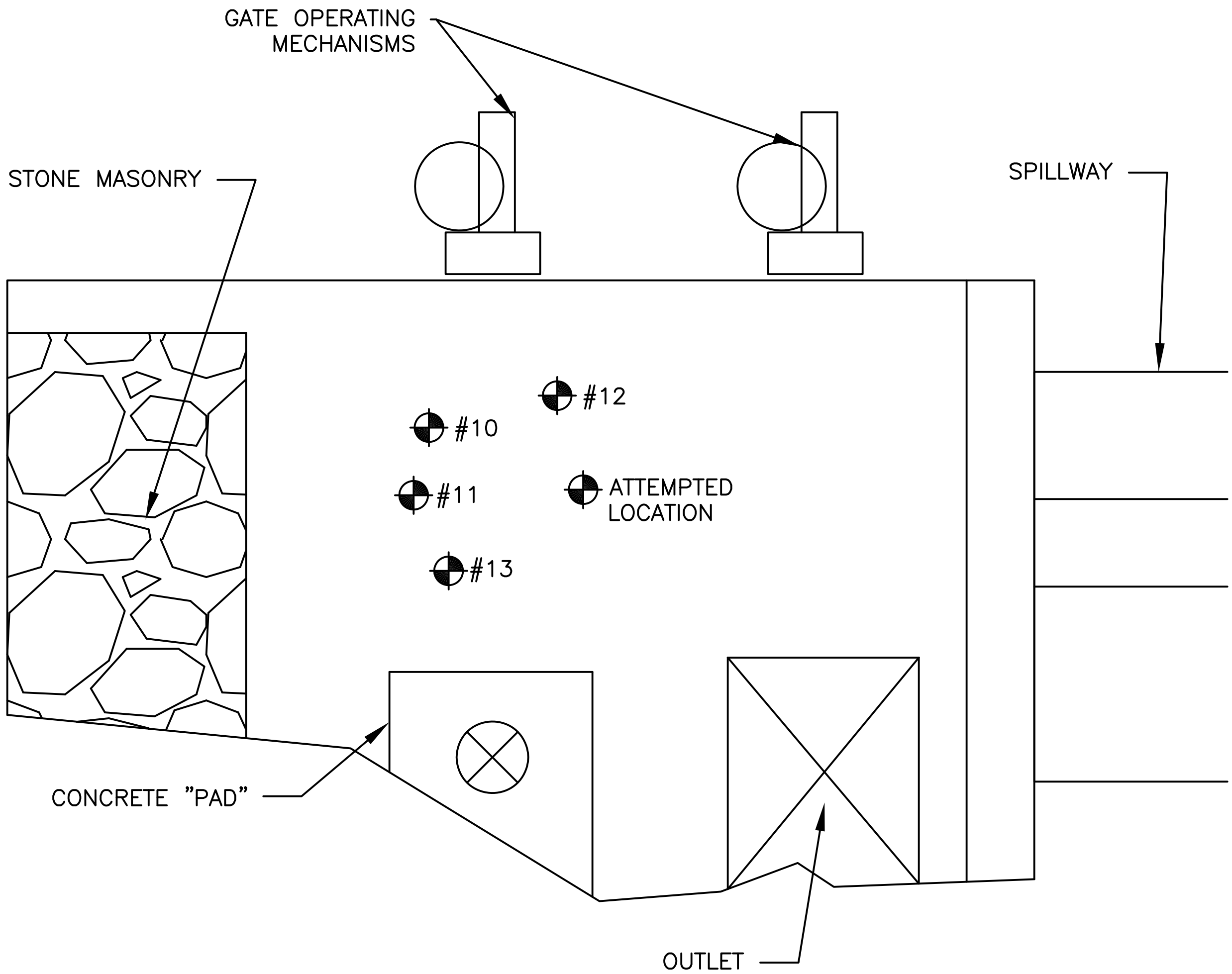
Project: Number: 075-07-003 Sheet 1 of 1
Name: Oyster River Dam Concrete Eval.
Durham, New Hampshire

Original Work:
By: NAO Date: 3-17-2010
Checked By: BAS Date: 3-17-2010

Subject: Figure 2 - Downstream Elevation
Approximate Locations - Cores #10 - #13



DOWNSTREAM ELEVATION AT RIGHT GATE STRUCTURE
NOT TO SCALE

⊕ INDICATES APPROXIMATE LOCATION AND NUMBER OF CONCRETE CORE

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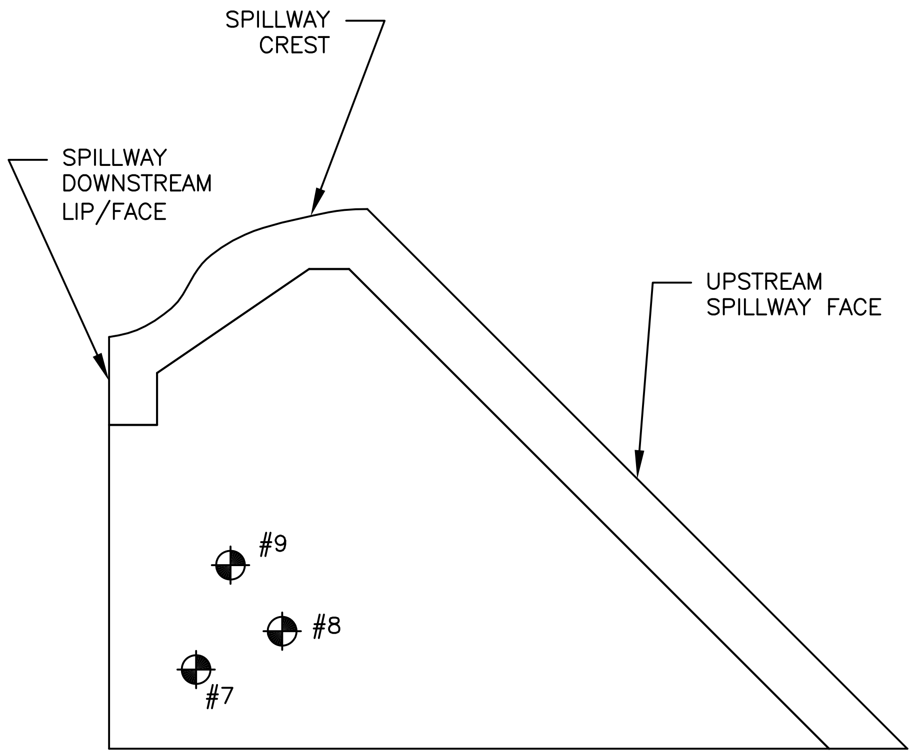
Project: Number: 075-07-003 Sheet 1 of 1
Name: Oyster River Dam Concrete Eval.
Durham, New Hampshire

Original Work:
By: NAO Date: 3-17-2010
Checked By: BAS Date: 3-17-2010

Subject: Figure 3 - Section through Spillway
Approximate Locations - Cores #7 - #9



SPILLWAY SECTION
PIER/RIB BETWEEN CELLS 7 AND 8
NOT TO SCALE

INDICATES APPROXIMATE LOCATION AND NUMBER OF CONCRETE CORE

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TABLES

Project: Number: 075-07-003 Sheet 1 of 2
Name: Concrete Evaluation, Oyster River Dam
Durham, New Hampshire

Original Work:
By: B. Stephens Date: February 18, 2010 Subject: **Table 1 - Coring Observation Notes**
Checked By: Date:

Core Number	Location	Observations
1	Top of dam above Cell 6 near Rib 6	Diameter: 4" Length: ~12" - one solid piece - top few inches appear to be different type concrete with smaller aggregate - one corner of core through thickness of crest - cracks visible on side of core
2	Top of dam above Cell 6 about 1 foot right of joint over Rib 6	Diameter: 4" Length: ~4" - top layer (patch material) came out of hole, bottom layer (original concrete) stuck in hole - small (#3?) bar visible at D/S edge of core hole - driller observed that the "original" concrete felt stronger than "repair" concrete
3	Top of dam above Cell 6 about 2 feet right of joint over Rib 6 and 2 feet D/S of ridge	Diameter: 4" Length: ~11" - top 4-5" broke off below repair concrete. - bottom 5-7" solid; no cracking or spalling
4	Upstream face at Rib 6	Diameter: 4" Length: ~22" - top 11" came out as one solid piece with square reinforcing bar at bottom. Bar appeared to have been resting on "form" at cold joint. - 10-11" solid piece below (into rib). Steel about 1-2" down.
5	Top of dam above Cell 6 near center of span, about 1' D/S of ridge	Diameter: 4" Length: ~12" - 2 layers visible, one solid piece - very large aggregate (~3+" dia.) at bottom of core

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	Project: Number:	075-07-003	Sheet 2 of 2
	Name:	Concrete Evaluation, Oyster River Dam	
Original Work:		Durham, New Hampshire	
By: B. Stephens	Date: February 18, 2010 Subject:	**Table 1 - Coring Observation Notes**	
Checked By:	Date:		

Core Number	Location	Observations
6	Top of dam above Cell 6 near center of span, about 1' D/S of ridge	Diameter: 6" Length: ~10" - core broke at large aggregate about 8.5" down - top ~3" appear to be repair concrete with small aggregate - repair well bonded to original - hit steel bar about 10" down and stopped coring
7, 8, 9	Rib #7 from left side near bottom outside corner	Diameter: 6" Length: ~12" - no visible cracking - no steel encountered - well distributed aggregate (large sized)
10	downstream face of right gate structure at abutment	Diameter: 6" Length: ~12" - came out of hole in one solid piece - cracking in plane parallel to face of wall
11	downstream face of right gate structure at abutment	Diameter: 6" Length: ~12" (hole depth) - core broke into many pieces before removal - encountered steel about 3" from face - very large aggregate - horizontal cracks through core and in core hole - brown staining at level of steel - small pieces of concrete crumbled easily away from aggregate
12	downstream face of right gate structure at abutment	Diameter: 6" Length: ~12" - one solid piece - several visible horizontal cracks - some large aggregate
12	downstream face of right gate structure at abutment	Diameter: 6" Length: ~7" - hit square steel bar ~3 1/2" from surface - solid piece repair concrete, no visible cracks - small aggregate (<1/2") - water coming through cracks behind core

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Table 2

Splitting Tension Strength Data

Core/ sample	Location on Dam	Splitting Tension Strength (psi)
#2 New	Crest	330
#2 Old	Crest	580
#3 Old	Crest	370
#8 Old A	Rib	380
#8 Old B	Rib	240
#8 Old C	Rib	350

Average of Old:	380	psi
Average of Crest (Old):	480	psi
Average of Rib:	320	psi

Table 3

Compressive Strength Data

Core/ sample	Location on Dam	Compression Strength (psi)
#1 Old	Crest	4740
#3 Old	Crest	6960
#4 (Top)	U/S Face	6560
#4 (Bott.)	Rib	4730
#7	Rib	5890

Average of Old:	5780	psi
Average of Crest (Old):	5850	psi
Average of Rib:	5310	psi

Table 4

Tension and Compression Comparison
(Original Concrete)

Sample Location	Average Splitting Tension (psi)	Average Compression (psi)	Tension/Compression Ratio
Crest	480	5850	0.082
Rib	320	5310	0.060
Overall Average	380	5780	0.066

Notes:
- Laboratory test result tables created from data provided by UNH testing laboratories.
- "Old" indicates sample from dam's original concrete (not repair concrete.)
- "New" indicates sample from 1970's repair concrete.

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Table 5

Unit Weight Data

Core/ sample	Location on Dam	G_s	Unit Weight (lb/ft^3)	Absorption (percent, %)
#4 Top	U/S Face	2.28	142.2	2.4
#4 Bott.	Rib	2.23	139.4	3.6
Average		2.26	140.8	3.0

Table 6

Elastic Modulus Data

Core/sample	Elastic Modulus, psi	ACI: Estimate of E^a, psi	Ratio
#4 Top	4075684 4193791 Average 4.13×10^6	4.53×10^6	0.913
#4 Bottom (Rib)	3625171 3671902 Average 3.65×10^6	3.74×10^6	0.976
#7 Rib	3857110 3932754 Average 3.89×10^6	4.17×10^6	0.934
Average Rib	3.77×10^6	3.96×10^6	0.955
Overall Average	3.89E+06	4.19E+06	0.929

ACI Eqn: $E = 33 * \text{unit weight}^{1.5} * \text{SQRT}(f'_c)$

Notes:
- Laboratory test result tables created from data provided by UNH testing laboratories.
- "Old" indicates sample from dam's original concrete (not repair concrete.)
- "New" indicates sample from 1970's repair concrete.

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APPENDIX A

PHOTOGRAPHS

Photo Number: 1
Description: Dewatered Dam looking upstream.

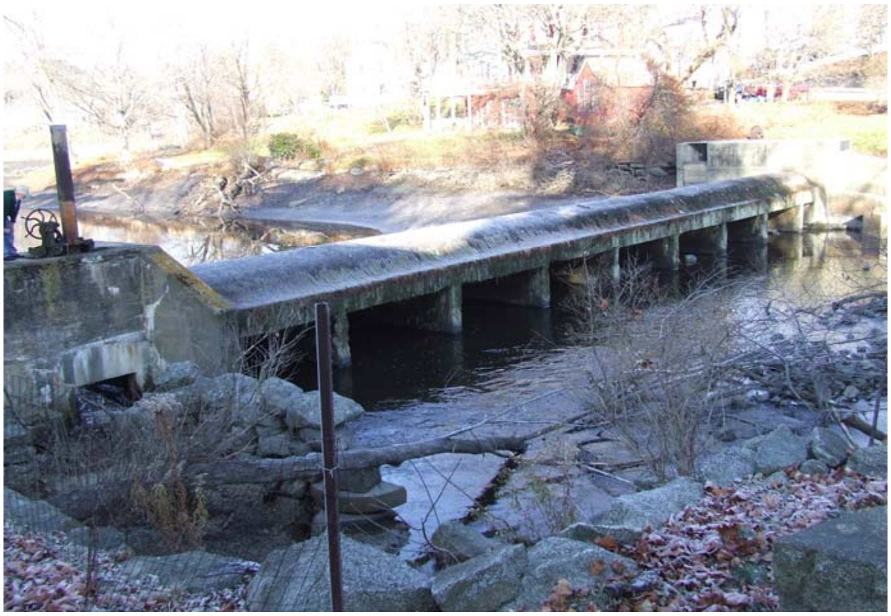

Photo Number: 2
Description: Downstream face of spillway.

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Photo Number: 3
Description: Upstream face of spillway.

Photo Number: 4
Description: Upstream face of spillway.

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Photo Number: 5

Description: Upstream spillway face and crest above Cells 7 and 6. Note visible separation of repair concrete at crest.

Photo Number: 6

Description: Typical surface deterioration of upstream face of spillway.

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Photo Number: 7

Description: Spillway crest.

Photo Number: 8

Description: Moss and algae growth on downstream face of spillway.

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Photo Number: 9	
Description:	Cracking and efflorescence on Rib in Cell 2.

Photo Number: 10	
Description:	Rib between Cells 2 and 3.

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Photo Number: 11

Description: Rib between Cells 4 and 5.

Photo Number: 12

Description: Crack and delamination at joint between rib and ceiling slab in Cell 7.

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Photo Number: 13	
Description:	Interior of Cell 1.

Photo Number: 14	
Description:	Interior of Cell 2.

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Photo Number: 13

Description: Cracking spalling and efflorescence at downstream ceiling of Cell 3.

Photo Number: 14

Description: Downstream ceiling of Cell 4.

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Photo Number: 17

Description: Interior of Cell 5.

Photo Number: 18

Description: Downstream ceiling of Cell 8.

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Photo Number: 19

Description: Downstream face of gate structure near right abutment.

Photo Number: 20

Description: Top of gate structure near right abutment.

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Photo Number: 21

Description: Core #1.

Photo Number: 22

Description: Core #4.

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Photo Number: 23

Description: Core #6.

Photo Number: 24

Description: Core #7.

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Original Work:
By: B. Stephens Date: November 2009 Subject: **Appendix A - Photographs**
Checked By: Date:

Photo Number: 25	
Description:	Core #10.

Photo Number: 26	
Description:	Cracking in core hole #10.

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Project: Number: 075-07-003 Sheet 14 of 15
Name: Concrete Evaluation, Oyster River Dam
Durham, New Hampshire

Original Work:
By: B. Stephens Date: November 2009 Subject: **Appendix A - Photographs**
Checked By: Date:

Photo Number: 27	
Description:	Core #11 pieces.

Photo Number: 28	
Description:	Cracking in core hole #11.

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Photo Number: 29	
Description:	Core #13.

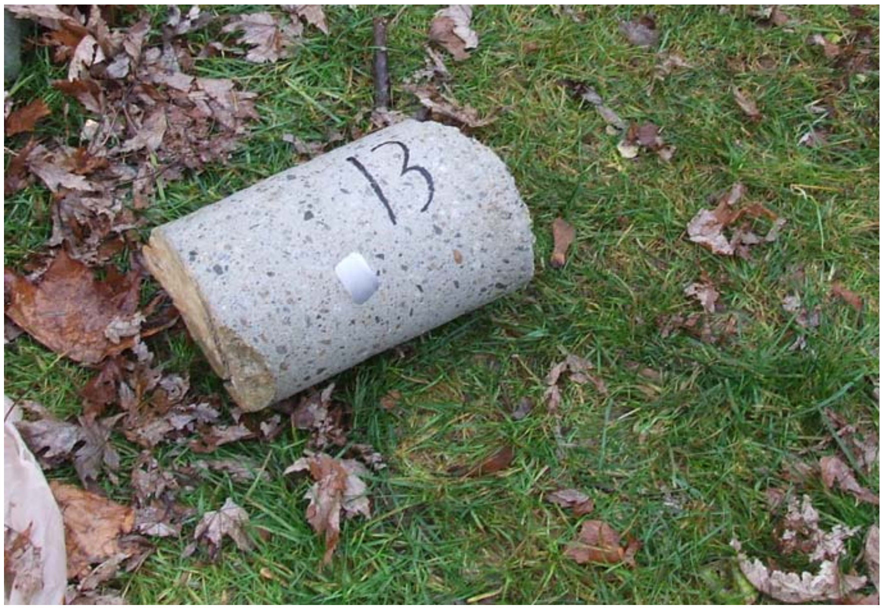

Photo Number: 30	
Description:	Core hole locations in gate structure at right abutment.

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APPENDIX B

CONCRETE TESTING RESULTS

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Evaluation of the Concrete of the Oyster River Durham Falls Dam Concrete

Final Report

to

David Cedarholm, PE

Durham Town Engineer

and

Bethel Stephens, P.E.

and

Robert S. Stephens, PE, PG

Stephens Associates Consulting Engineers, LLC

by

Dr. David Gress Ph.D., P.E.

Consulting Engineer

Portsmouth, NH

March 25, 2010

TESTING PROTOCOL

The Oyster River Durham Falls Dam testing protocol consisted of visual observation of the Dam, taking audio soundings, coring of selected areas, and laboratory evaluation of the concrete obtained during coring. The laboratory evaluation of the cores consisted of performing conventional hardened properties testing (compressive and splitting tension strength, unit weight, and elastic modulus), cutting and polishing for microscopic evaluation, making thin sections for detailed petrographic analysis and coating new fractured surfaces for testing for specific materials related distress.

TESTING RESULTS

Visual Analysis

The visual analysis of the concrete was consistent with the findings of the recent report, Dam Evaluation Report Oyster River Dam, SA Project No. 075-07-003, March 17, 2009. In general the concrete was found to be overall in good condition for its age. There are areas where erosion of the concrete was noted such as the top crest of the spillway, the downstream face of the ribs and portions of the right abutment. The right side abutment, especially on the downstream face showed signs of cracking and efflouresce of what appeared to be calcium hydroxide.

Audio Soundings

Soundings were taken over the surface of the dam that would have been expected to have gone through freezing and thawing cycles. This was accomplished by using a geological hammer to tap the surface being evaluated while listing to the ring of the sound produced. Sharp high pitch rings are consistent with high quality concrete whereas thuds and lower pitch rings suggest areas that have lower quality concrete and/or areas that have delaminated. In general the concrete was found to be in relatively good condition. There were some minor areas on the upstream portion of the crest where the 1974 bonded overlay had delaminated. There were also some areas in the interior downstream components of cells 1 and 2 (including their ribs) which definitely were of lower quality concrete. The right abutment also had areas that were of lower quality concrete.

Cores

Cores were taken at the approximate locations as shown in Table 1. Cores #1 through #4 were four inches in diameter and cores #5 through #13 were six inches in diameter. It was decided to increase the size to six inches due to the relatively large maximum aggregate size in excess of 2” so as not to bias the laboratory testing results.

Laboratory Testing

Physical Tests

Cores were selected for compressive strength testing (ASTM C 39/C 39M – 01, Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens), elastic modulus (ASTM C 469 – 02, Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression), splitting tension (ASTM C 496 – 96, Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens) and unit weight and absorption **(**ASTM C 127 – 01 Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Coarse Aggregate).

Physical tests performed on the cores are presented in Table 2.

Splitting Tension

The results of the splitting tension testing are presented in Table 3. The tensile strengths except for the #2 original concrete were much lower than expected. The average for the original concrete, identified by "Old" was 380 psi. The strength of the 1974 overlay was 330 psi however this is based on only one sample. Overall these data show the splitting tensile strength to be much less than expected based on the visual analysis of the concrete suggesting the possible existence of micro cracks in the concrete.

Compression Strength

The compression strength testing data are presented in Table 4. As was expected the strengths were relatively high with an average for the original concrete of 5780 psi.

Compression and Tension

A comparison between compression strength and tensile strength gives an excellent indication of micro cracking. A Materials Related Distress (MRD) typically affects the relationship between the two strength properties by reducing the tensile strength. The ratio of tensile strength to compression strength is shown in Table 5. These data show the tensile strength to be about 10 percent of the compressive strength for core #2 and about 5 to 6 percent for cores #3 and #8. The concrete in an undisturbed state would be expected to have a ratio of between 7 and 8 percent. The 10 percent value is higher and the 5/6 is exceptionally lower than what would be expected. Such variation gives a strong indication of the presence of a MRD.

Unit Weight

Table 6 shows the unit weight of the original concrete to be 142.2 lbs/ft^3 for the newer concrete and 139.4 lbs/ft^3 for the older original concrete. The absorptions were 2.4 percent for the newer concrete and 3.6 percent for the older original concrete. The absorption data are lower than what would be expected.

- ***Elastic Modulus***

The elastic moduli data of the concrete are presented in Table 7. The average elastic modulus for the concrete obtained from the upstream face (#4 Top) was 4.13 x10^6 psi and the average rib (#4 Bottom & #7) was 3.77 x10^6 psi. Using the compressive strength and the unit weight it is possible to estimate the elastic modulus of a concrete based on American Concrete Institute criteria. Using the ACI equation the estimated elastic moduli were 4.53 x10^6 psi for the upstream face and an average of 3.96 x10^6 psi for the rib sections. Ratios of the actual to the predicted elastic moduli ranged from 0.913 for the upstream face to an average of 0.955 for the rib sections. These data show as with the tensile strength data that there is significant evidence of a Materials Related Distress.

Petrographic Testing

Selected cores were cut and polished for visual observation under a standard binocular microscope. A technique to enhance the visual identification of cracks was performed on several polished sections by coating them with low viscosity epoxy impregnated with an ultra violent fluorescing pigment. Other tests conducted included, applying the new fractured surface of the splitting tension and compression specimens with a solution of Uranyl Acetate dihydrate then viewing under ultra violet light to evaluate for the presence of Alkali Silica Reaction (ASR) and making thin sections to view the concrete on a petrographic microscope. The tests conducted on the cores are as presented in Table 2.

Polished Sections

Selected cores were cut using a diamond blade then polished on a lapidary wheel using various grit sizes. Figure 1 shows the upper portion of the concrete from the 1974 rehabilitation. Cracking of the concrete is shown in a close up of section A in Figure 2. Micro cracks were apparent in Figure 3, the same sample as in Figure 1, after epoxy was applied with the Ultra Violent (UV) pigment. The surface was repolished to enhance the presence of cracks when viewed under UV light. These cracks are most likely due to Freeze Thaw damage.

Figure 4 shows the concrete of the spillway crest obtained in core # 5. Sections B, C, D, and F show four different concretes. Section B is the 1974 repair concrete, C is a layer of mortar that was applied to achieve a bond between the old original concrete shown in section D and the new concrete in section B. The lower portion shown by section F contains very large aggregate and is well compacted compared to the concrete of section D which has little coarse aggregate and shows poor compaction. This concrete was apparently mixed on site and hand placed which would be the expected procedure in 1913. The original concrete as seen on the lower portion consists of a natural glacial gravel material most likely obtained in the near vicinity of the Dam. It contains various minerals mostly of igneous origin with a maximum aggregate size of approximately 2.” Several of the large aggregate pieces consisted of a deteriorated granite that was so weak it came apart during the polishing process as noted by the small pieces of mica scattered throughout the concrete. Figure 5 shows a close up of section A which contains a crack, most likely from freeze thaw damage. It also contains several pores filled with a white powder deposit. Figure 6 shows a close up of the mortar applied to enhance the bond at the old and new interface. This layer was poorly compacted as shown by the voids at the old interface as well as in the matrix of the layer. Several other voids containing the white powder are shown on the close up view of section E in Figure 7. These voids appear to be interconnected by a crack that initiated in the aggregate particle on the right side of figure 7.

The polished section of Core #5 was coated with a low viscosity epoxy that was impregnated with an UV fluorescing pigment and repolished so as to visually enhance the micro cracks. Figure 8 shows the polished section coated with the epoxy under UV light. A view of the mid section of the epoxy coated Core #5 under UV light is shown in Figure 9. Micro cracks are visible within the aggregate as well as the paste matrix. Reaction rims are also emphasized by the dark areas on the on the edge of most of the aggregates.

An overall view of the polished section of Core #6 is shown on Figure 10. This shows four different concretes the 1974 repair concrete, the interface mortar, a high entrapped air concrete with little aggregate, and the typical original described above. The surface concrete shows signs of freeze thaw damage as noted by the parallel cracks as shown in the close up view of section a in Figure 11. The mortar interface layer is poorly consolidated and is delaminating at the old concrete interface as shown by Figure 12. Aggregate internal cracking and matrix cracking are shown on the close up view of section C on Figure 13.

Core # 9 obtained horizontally from the rib is shown on Figure 14. This concrete appears more uniform because the core was taken horizontally. If the core was obtained vertically as the previous cores then it would be expected that several placements would be viewed showing variable mixes. Overall this concrete shows cracking and reaction rims as the previous original concrete. Figure 5 shows a close up of section A with cracking within the aggregate particle as well as around it. Figure 14 shows a crack radiating up towards the surface of the concrete originating from an aggregate particle that has expanded as shown in the close up view in Figure 16. Another aggregate particle that has expanded is shown in Figure 17, a close up view of section C. This aggregate is severely cracked due to internal expansion. Reaction rims are also visible on the outside surface of the aggregate. The aggregate particle of section D, shown in Figure 18 has micro cracks. The interface of the aggregate and the paste also shows cracking. The flat like dark line near the right top side of the aggregate particle is a sliver of mica. These small slivers are randomly dispersed throughout the concrete and most likely came from the very weak granite particles that degraded during the mixing and placement of the concrete. A severely cracked aggregate particle is shown in the close up view of section E in Figure 19. This aggregate particle also has a very dark reaction rim. An advanced crack matrix is shown in the close up view of section F of Figure 20.

The polished sample of Core # 12 taken from the downstream face of the right abutment is shown on Figure 21. Many of the aggregate particles have reaction rims as noted in the previous polished sections. Section A shows a highly carbonated area with significant cracking. The polished sample shows massive cracking as noted by the arrows radiating from box B. Extreme swelling of the aggregate is noted by the large gap shown in the aggregate particle in the close up of section C in Figure 23. The area of paste to the left of the particle shows a very large crack, the same width as the aggregate crack, which has filled in with a reaction product. Cracking through and around aggregate particles is shown in the close up view of section D in

Figure 24. Several aggregate particles in this view also show reaction rims. This concrete has undergone significant expansion.

Uranyl Acetate Testing

After the splitting tensile testing was completed the two approximately equal size pieces from each test were evaluated for Alkali Silica Reaction (ASR) by coating one of the two pieces with a solution of Uranyl Acetate Dihydrate and viewing both under an ultraviolet light source (UV). Similarly this procedure was followed for Core # 1, a compression strength sample. Areas that contain ASR gel show a greenish glow. Figures 25 through 28 show the cores that were evaluated for ASR under UV light and normal light. The photograph on the top of the figures is of the samples under normal light.

Core # 1 shown in Figure 25 definitely has signs of ASR as noted by the greenish glow under the UV light source. The concrete placed during the 1974 rehabilitation is shown in Figure 26 and 27. Although not significant, several of the aggregate particles definitely show signs of ASR in the repair concrete. The original concrete definitely shows significant signs of ASR as noted by Core # 8 of Figure 28.

Thin Sections

Thin sections were made so a petrographic analysis of the original concrete could be determined. Core # 10 from the right abutment downstream face was selected for the analysis. The concrete was visually identified as being typical of the concrete observed in the other cores. Two sections were cut for preparing the thin sections. The first was parallel to the wall surface and referred to as the vertical section. The second was taken perpendicular to the surface and referred to as the horizontal section. These thin sections were evaluated using a petrographic microscope.

Figure 29 through Figure 36 are from the vertical section and Figure 37 through Figure 45 are from the horizontal section. Extensive aggregate cracking and deposits of ASR gel are shown on Figure 29. The blue color is from a dye used to emphasize voids and cracks that are empty. When the light source is changed from normal to polarized it is referred to as crossed nicols, this allow for the identification of various crystalline and amorphous materials. For example, Figure 30 shows a dark substance that is amorphous ASR gel by using polarized light. Layered ASR gel is shown in Figure 31. The same view under polarized light shows various layers of crystalline and amorphous ASR gel. The ASR reaction has been active for a long time period as shown by the multiple layers. Figure 33 shows the same area as figures 31 and 32 but under lower magnification. Extreme cracking is noted in the aggregate in Figure 34. The silica from the aggregate combines with alkali and calcium to make the ASR gel as shown deposited in the air void. Ettringite, a mineral with composition $(CaO)_3(Al_2O_3)(CaSO_4)_3 \cdot 32\ H_2O$ is produced naturally within portland cement during initial hydration. When a portland cement system becomes compromised by excessive cracking the Ettringite moves around and is redeposited.

This is referred to as secondary Ettringite. Such is shown by Figures 35 through 38 where the secondary Ettringite has deposited within a crack at the aggregate paste interface. Figure 39 shows cracking through a granite particle due to ASR. Deposits of Ettringite are also indicated on Figure 39, 40 and 41. The crack shown in Figure 41 with the ASR gel gives a good indication of the destructive impact that the reaction has had on the dimensional stability of the concrete. This concrete has definitely expanded and in the process it has undergone excessive cracking as indicated by Figure 42 and 43. Some of these cracks could be associated with freezing and thawing damage. Once a concrete opens up from cracking it becomes vulnerable to other distress mechanisms.

The portland cement used in the construction of the Oyster River Durham Falls Dam was a natural cement made of limestone with a high clay content (argillaceous limestone) from Rosendale, New York. The burn temperature of this cement was much lower than a normal portland cement resulting in a cement different from a normal portland cement. The Rosendale cement would have been expected to have a fast initial set, followed by a very slow and gradual rise in strength. There are always considerable amounts of unhydrated cement resulting in a common identifying feature when the concrete is viewed under a microscope as shown by Figure 44. Another characteristic of the Rosendale cement is the presence of a higher than normal, compared to normal portland cement, percentage of calcium hydroxide as shown in Figure 46.

DISCUSSION

The physical test properties of the Durham Falls Dam show the compressive strengths to be very high with an average strength of approximately 5,780 psi. Compression strength, although universally utilized as a major indicator of concrete quality world wide, is not capable of detecting the presence of ASR.

The tensile strength data, average of 380 psi, show the concrete to be less than what would be expected based on the compressive strengths. The ratio of tensile strength to compressive strength was approximately 5 to 6 percent which is lower than what would be expected. This strongly suggests the concrete is severely cracked internally which was observed in the petrographic evaluation. One core showed a high ratio of 10 percent which is typical of a high quality concrete. Highly variable data such as these are to be expected for a concrete structure that is subject to ASR.

The elastic modulus is an excellent indicator of the presence of micro and macro cracks. The elastic moduli data show a difference between the elastic modulus expected and what was determined on the cores. The ratio of experimental to expected elastic moduli ranged from 91 to 98 percent. This also shows the effect of microcracking. These ratios being less than expected suggests the structural capability of the concrete has been reduced due to MRD.

The Uranyl Acetate testing showed the presence of ASR gel on the cracked faces of the splitting tension samples. Although this test is not by itself a sole indicator of the destructive presence of ASR it does show a significant amount of gel has been produced within the concrete.

The petrographic analysis showed beyond doubt that the concrete has ASR and extensive microcracking and macrocracking has occurred throughout the concrete matrix as well as within the reactive aggregate. Expansion of the old concrete relative to the newer 1974 repair concrete would be expected to cause the bond to fail and/or tensile cracks to occur. Delaminating and tensile cracking were both observed in the repair concrete during the acoustic hammer testing and while viewing the polished sections.

Nothing is known about the remaining potential for continued expansion due to ASR. This was not part of the testing protocol because visually there was no indication that ASR was a major player in the quality of the concrete. If the alkali of the concrete has been consumed ASR can not continue and the present state (dimensional stability) of the concrete would be expected to remain about the same in the future except for the effect of freezing and thawing. On the other hand, if the alkali content of the concrete is high enough then the ASR will continue resulting in increased expansion of the concrete. Further expansion of the concrete will increase microcracking and macrocracking, lower concrete strength, accelerate delaminating and increase surface erosion. The rate of deterioration will increase due to the compounding effect of opening the system to moisture which is required for both the freeze thaw and ASR mechanism to occur. Such a condition would be problematic to the useful service life of the Dam.

SUMMARY

The physical and petrographic testing of the Durham Falls Dam shows the concrete has deteriorated from ASR and freeze thaw. These distresses have decreased the concrete's tensile and elastic properties and will have an impact on the useful serviceability of the Dam. Nothing is known about the remaining ASR expansion potential so the predicted remaining service life is difficult to address. However, it is known that once ASR opens a concrete up by micro and macrocracking then other distresses like freeze thaw can have a major impact. Deterioration increases at an exponential rate because it is easier for water, a key component, to enter the interior of the concrete thus accelerating the distress. This accelerated effect will be first noted on the areas presently showing erosion and cracking.

It is very probable the effective service life of the Oyster River Durham Falls Dam is approaching its economical and useful service life.

RECOMMENDATIONS

It is difficult to make a viable recommendation for the future use of the Oyster River Durham Falls Dam due to lack of knowledge of the remaining ASR potential expansion in the existing concrete. It is recommended that this be determined prior to deciding the future fate of the Dam. If the reaction has not stopped then a conventional rehabilitation, bonding new concrete to the existing, will not be possible. This technique was used in the 1975 rehabilitation strategy and is the primary cause of the existing delamination and debonding noted during the evaluation. On the other hand, if the reaction has come to completion, it is possible to bond new concrete to the existing concrete making the rehabilitation much more cost effective. The alternative of not being able to bond to the existing concrete would increase the rehabilitation cost to the extent that the cost of removing the dam would be more financially palatable.

Table 1 Core approximate locations

Core	Approximate location
#1	Top of spillway crest in cell 6
#2	Top of spillway crest in cell 6
#3	Top of spillway crest in cell 6
#4	Upstream face through reinforcing steel
#5	Top of spillway crest in cell 5
#6	Top of spillway crest in cell 5
#7	Rib between cell 8 and 7 taken horizontally
#8	Rib between cell 8 and 7 taken horizontally
#9	Rib between cell 5 and 6 taken horizontally
#10	Downstream face of right abutment taken horizontally
#11	Downstream face of right abutment taken horizontally
#12	Downstream face of right abutment taken horizontally
#13	Downstream face of right abutment taken horizontally

Table 2 Core and Sample testing identification

Core	Splitting Tension	Compression strength	Elastic Modulus	Unit weight	Uranyl Acetate	Petrographic		
						Cut and polish	Epoxy impregnate	Thin section
#1		X			X	X	X	
#2	X				X			
#3	X	X			X			
#4 Top		X	x	X		X	X	
#4 Bottom		X	X	X				
#5						X		
#6						X		
#7		X	x					
#8	X				X			
#9						X		
#10								X
#11								
#12						X		
#13								

Table 3 Splitting tension strength data

Core/sample	Splitting tension strength, psi
#2 New	330
#2 Old	580
#3 Old	370
#8 Old A	380
#8 Old B	240
#8 Old C	350
Average Old	380
Average #8	320

Table 4 Compressive strength data

Core/sample	Compression strength, psi
#1 Old	4740
#3 Old	6960
#4 Top	6560
#4 Bottom (Rib/Web)	4730
#7 Rib/Web	5890
Average Old	5780

Table 5 Tension and compression comparison data

Core/Sample	Splitting tension	Compression	Ratio Tension/Compression
#2 Old	580	5780[a]	0.100
# 3 Old	370	5780[a]/6960	0.064/0.053
# 8 Old Average	323	5780[a]	0.055

Note: [a] average all Old

Table 6 Unit weight data

Core/sample	Gs	Unit Weight, lb/ft^3	Absorption, percent (%)
#4 Top	2.28	142.2	2.4
#4 Bottom (Rib)	2.23	139.4	3.6

Table 7 Elastic modulus data

Core/sample	Elastic Modulus, psi	ACI: Estimate of E[a] , psi	Ratio
#4 Top	4075684		
	4193791		
	Average 4.13 x10^6	4.53 x10^6	0.913
#4 Bottom (Rib)	3625171		
	3671902		
	Average 3.65 x10^6	3.74 x10^6	0.976
#7 Rib	3857110		
	3932754		
	Average 3.89 x10^6	4.17 x10^6	0.934
	Average Rib 3.77 x10^6	3.96 x10^6	0.955

Note: [a] E = unit weight$^{1.5}$x $f_c'^{1/2}$ (see Tables 4 and 6)

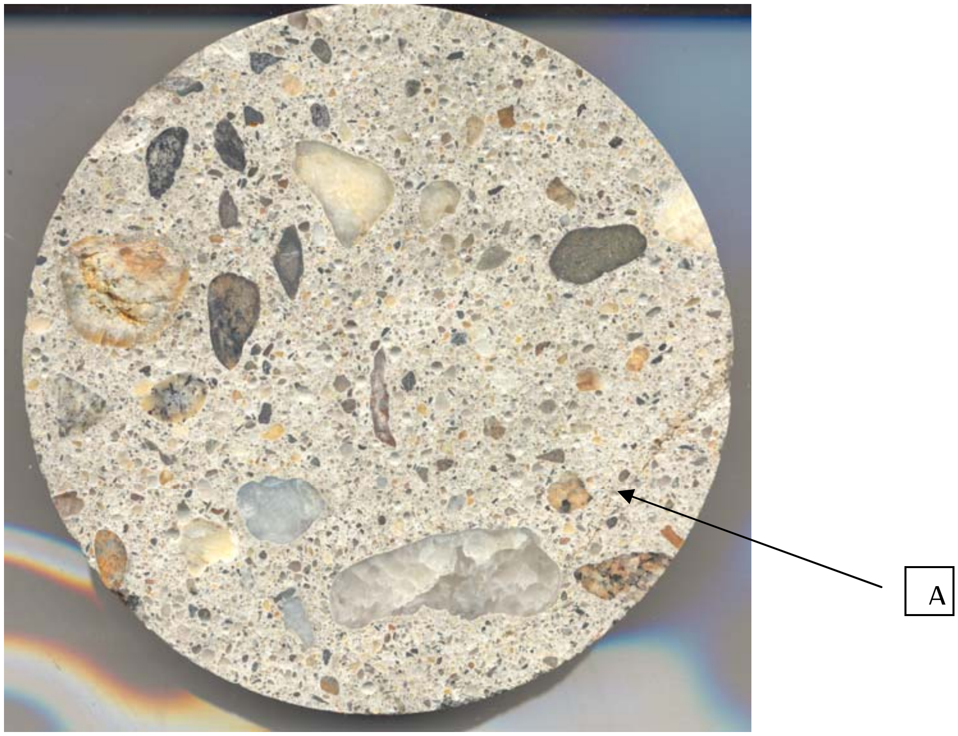

Figure 1 Core # 1 polished surface of top

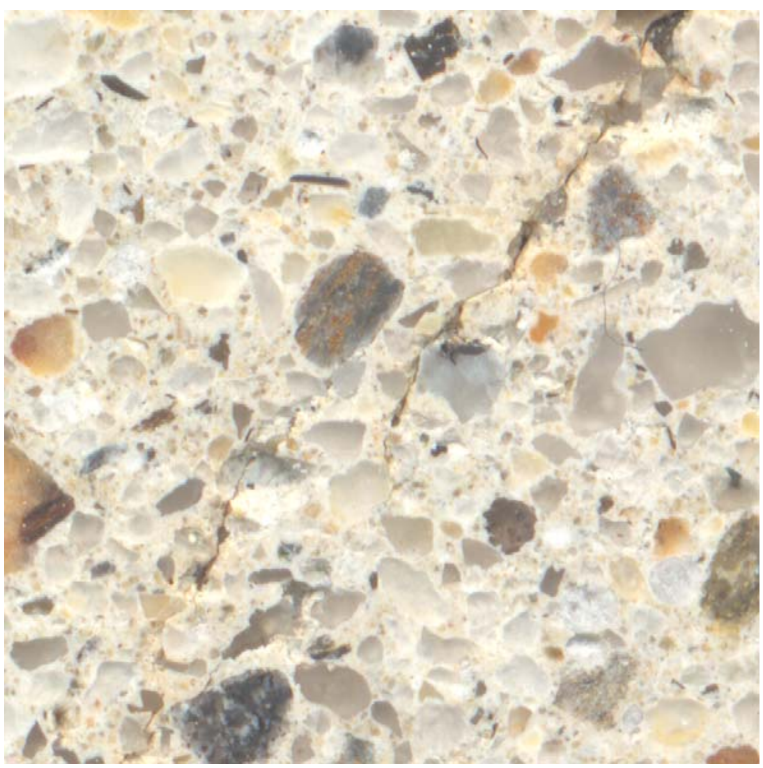

Figure 2 Section A close up of Figure 1 core #1

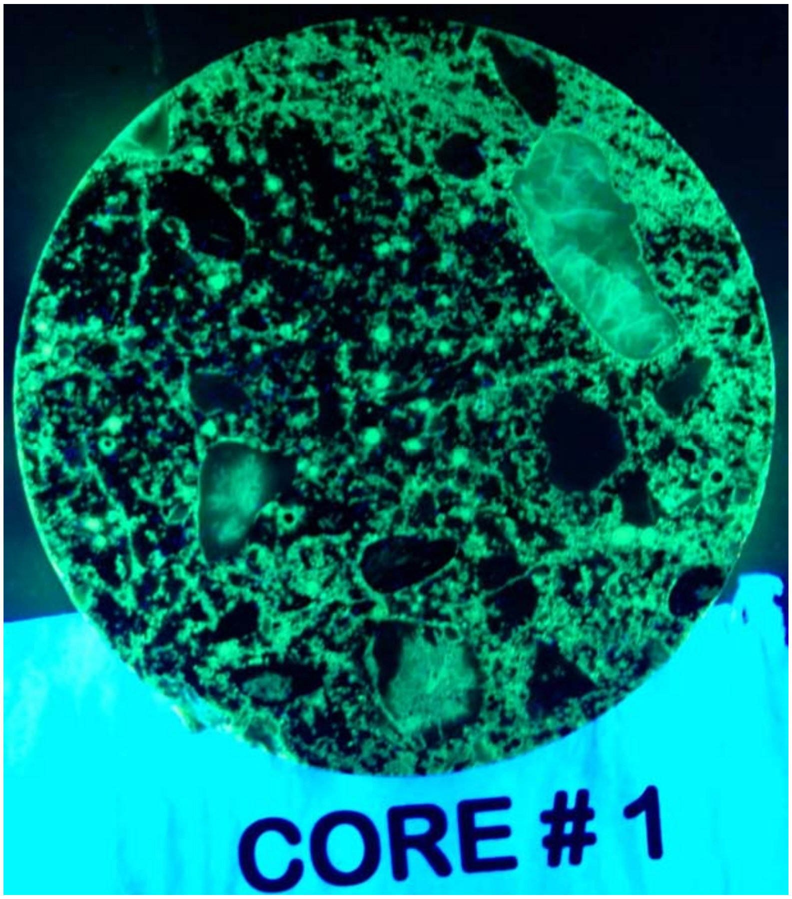

Figure 3 Core #1 polished and coated with impregnated epoxy under UV to enhance microcracks

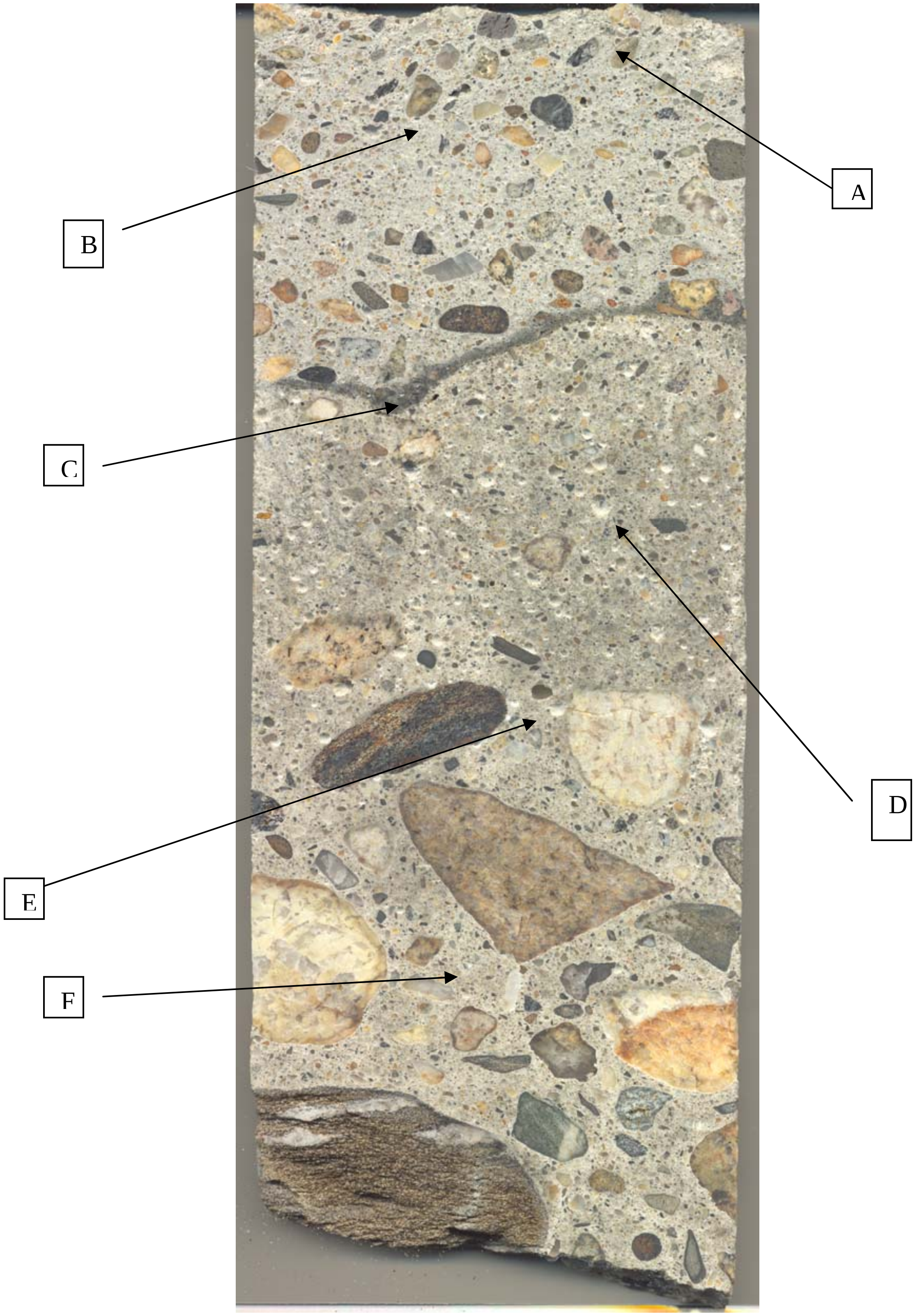

Figure 4 Core # 5 polished surface

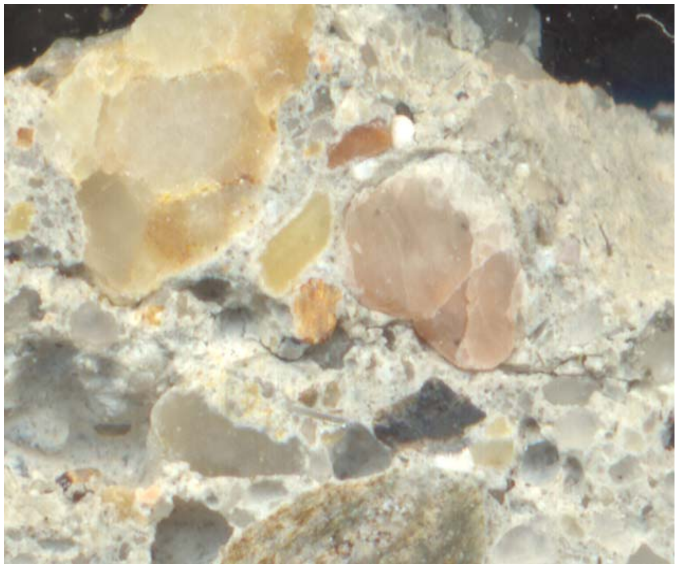

Figure 5 Section A close up of Figure 4 core #5

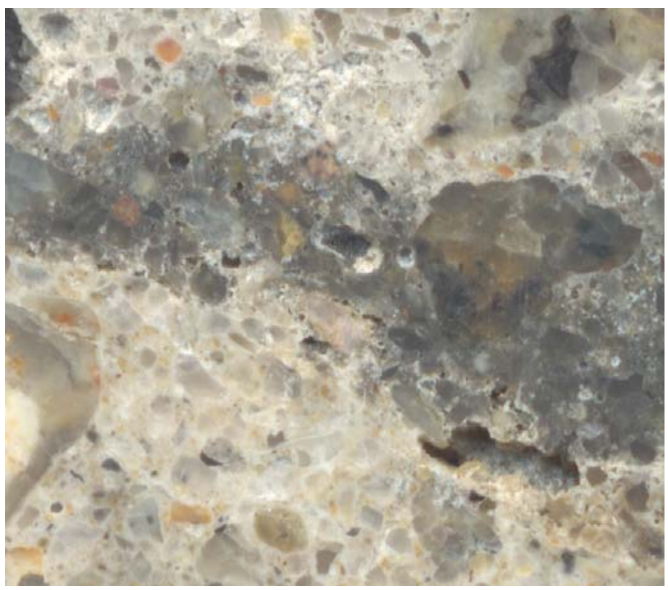

Figure 6 Section C close up of Figure 4 core #5

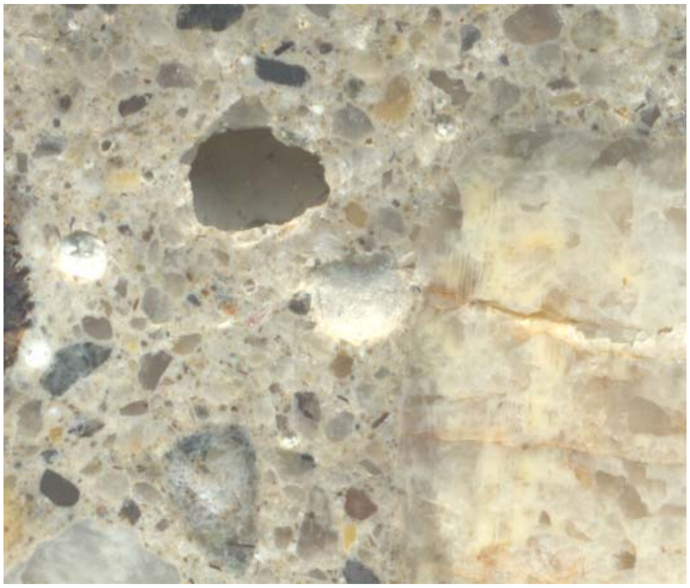

Figure 7 Section E close up of Figure 4 core #5

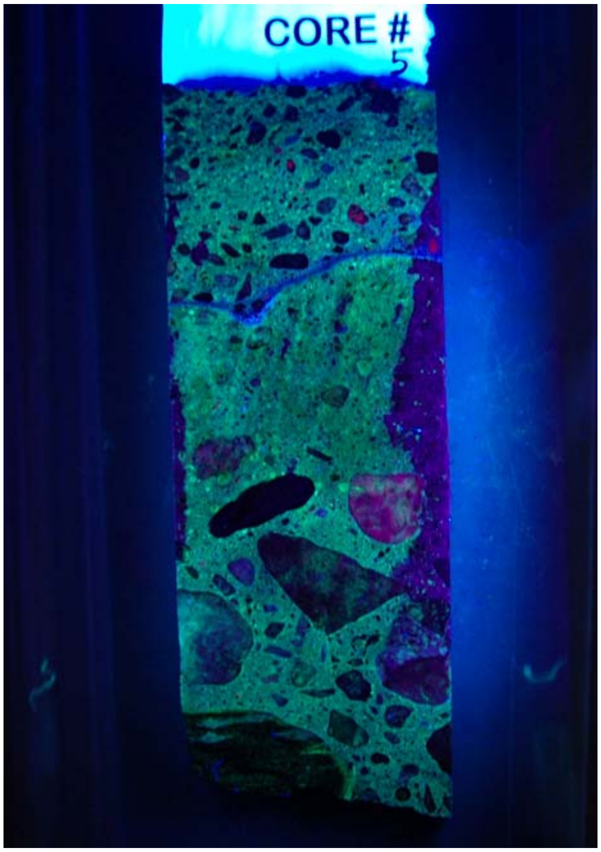

Figure 8 Core 5 polished section coated with impregnated epoxy and viewed under UV

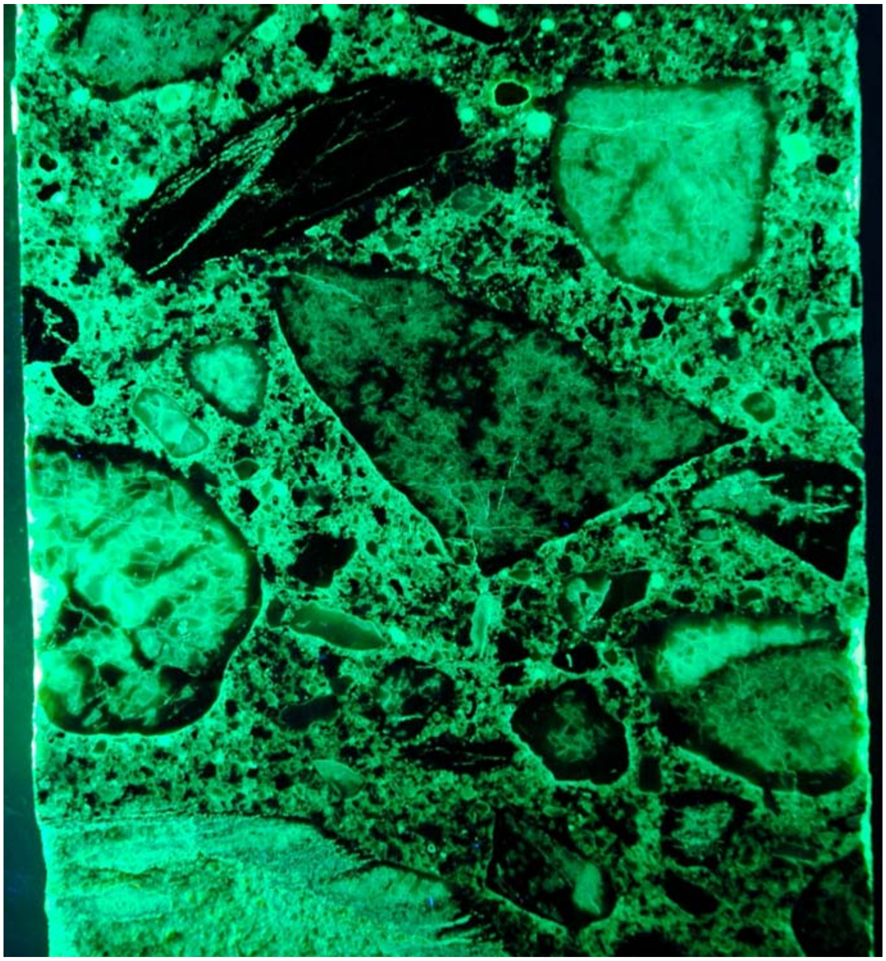

Figure 9 Core 5 polished mid section coated with impregnated epoxy under UV to emphasize micro cracks (see Figures 4, 7 and 8)

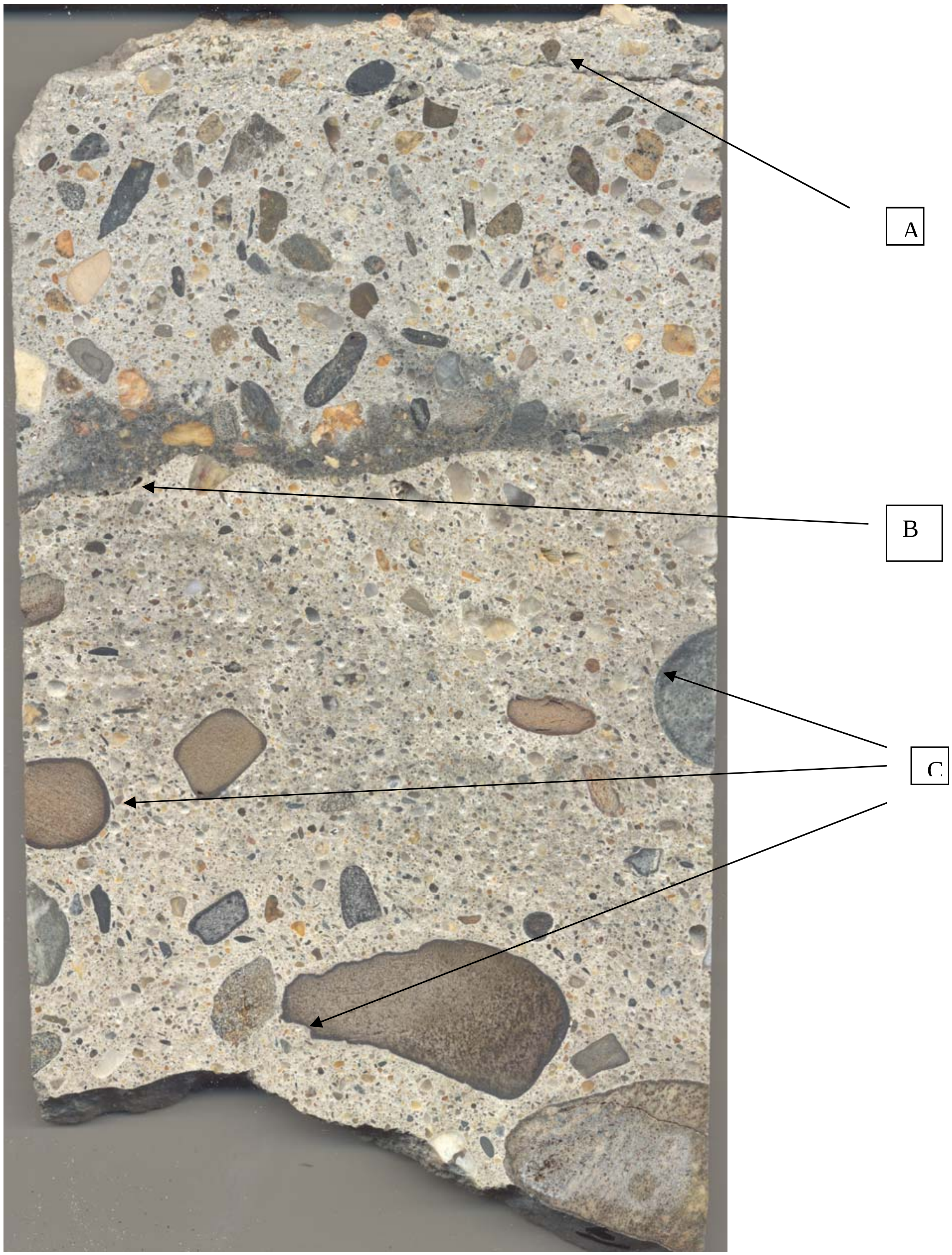

Figure 10 Core # 6 showing polished surface

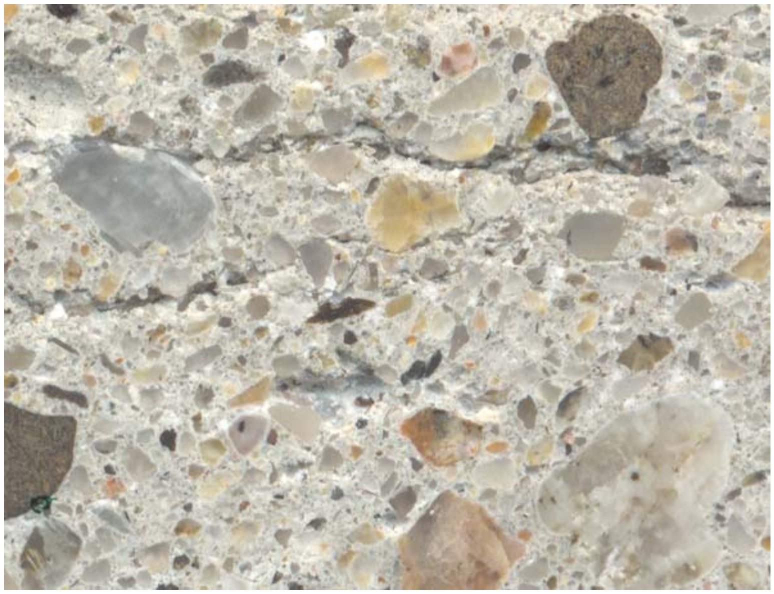

Figure 11 Section A of Figure 10 of Core # 6

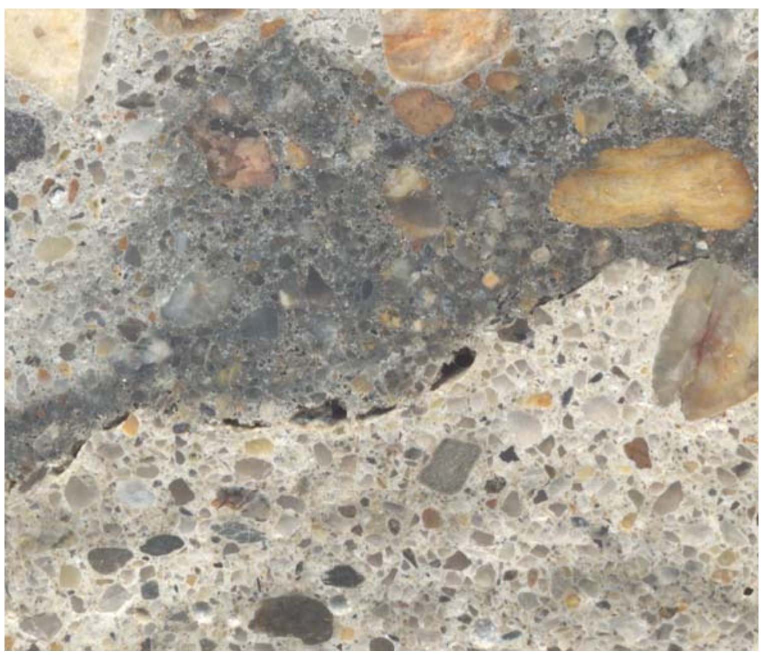

Figure 12 Section B of Figure 10 Core # 6

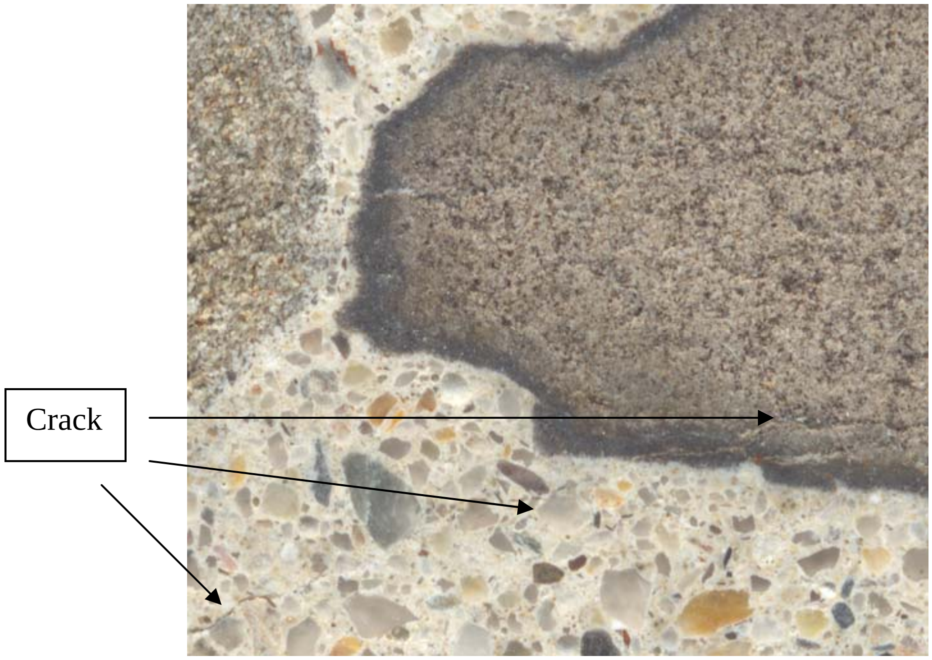

Figure 13 Section C of Figure 10 Core # 6

Figure 14 Polished section of Core # 9

Figure 15 Section A close up of Figure 14 Core #9

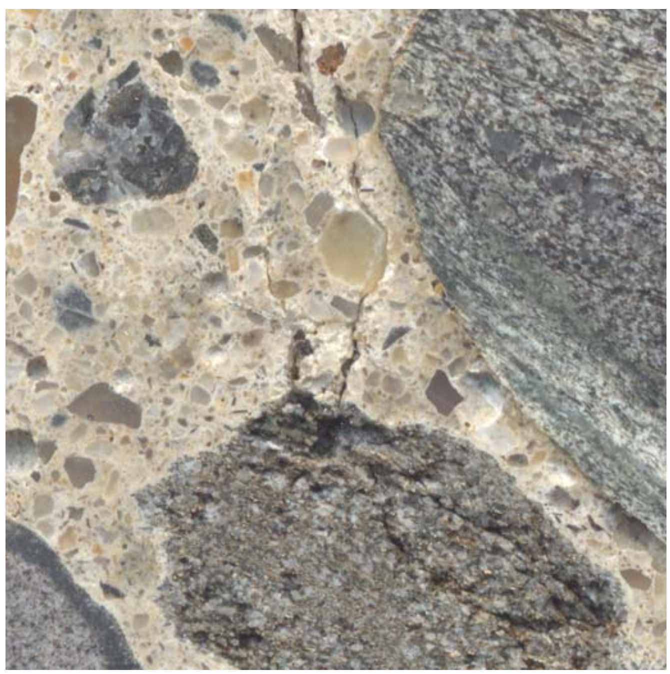

Figure 16 Section B close up of Figure 14 core #9

Figure 17 Section C close up of Figure 14 core #9

Figure 18 Section D close up of Figure 14 core #9

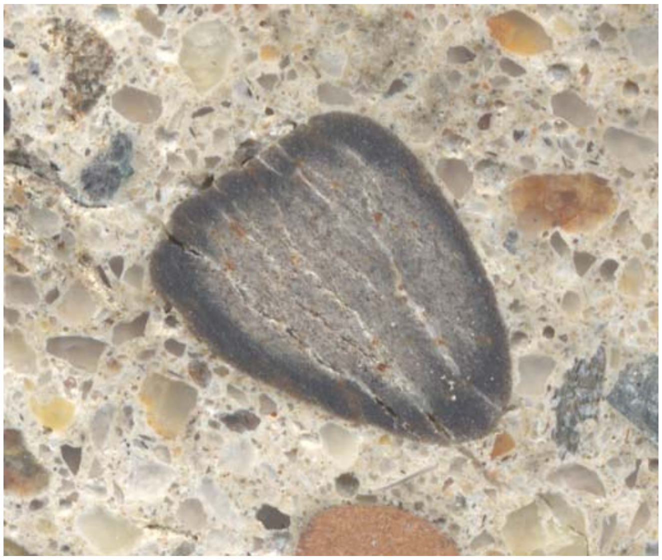

Figure 19 Section E close up of Figure 14 core #9

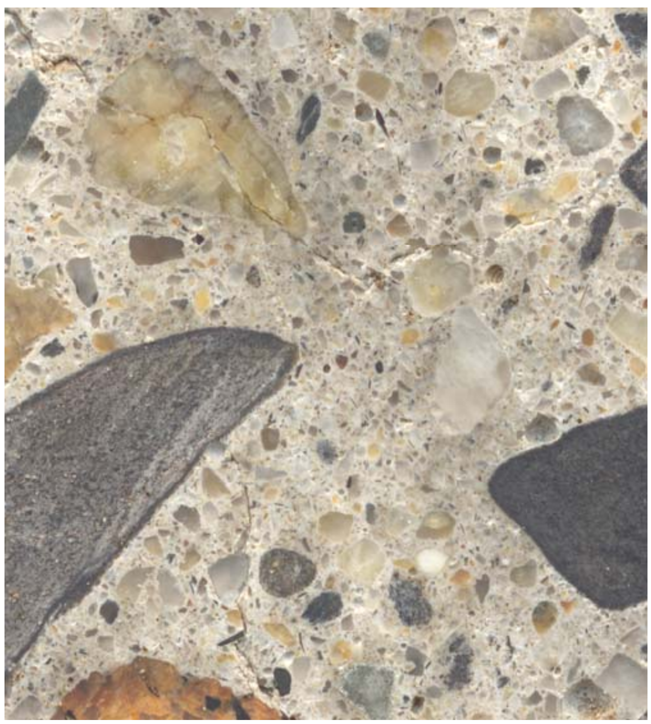

Figure 20 Section F close up of Figure 14 core #9

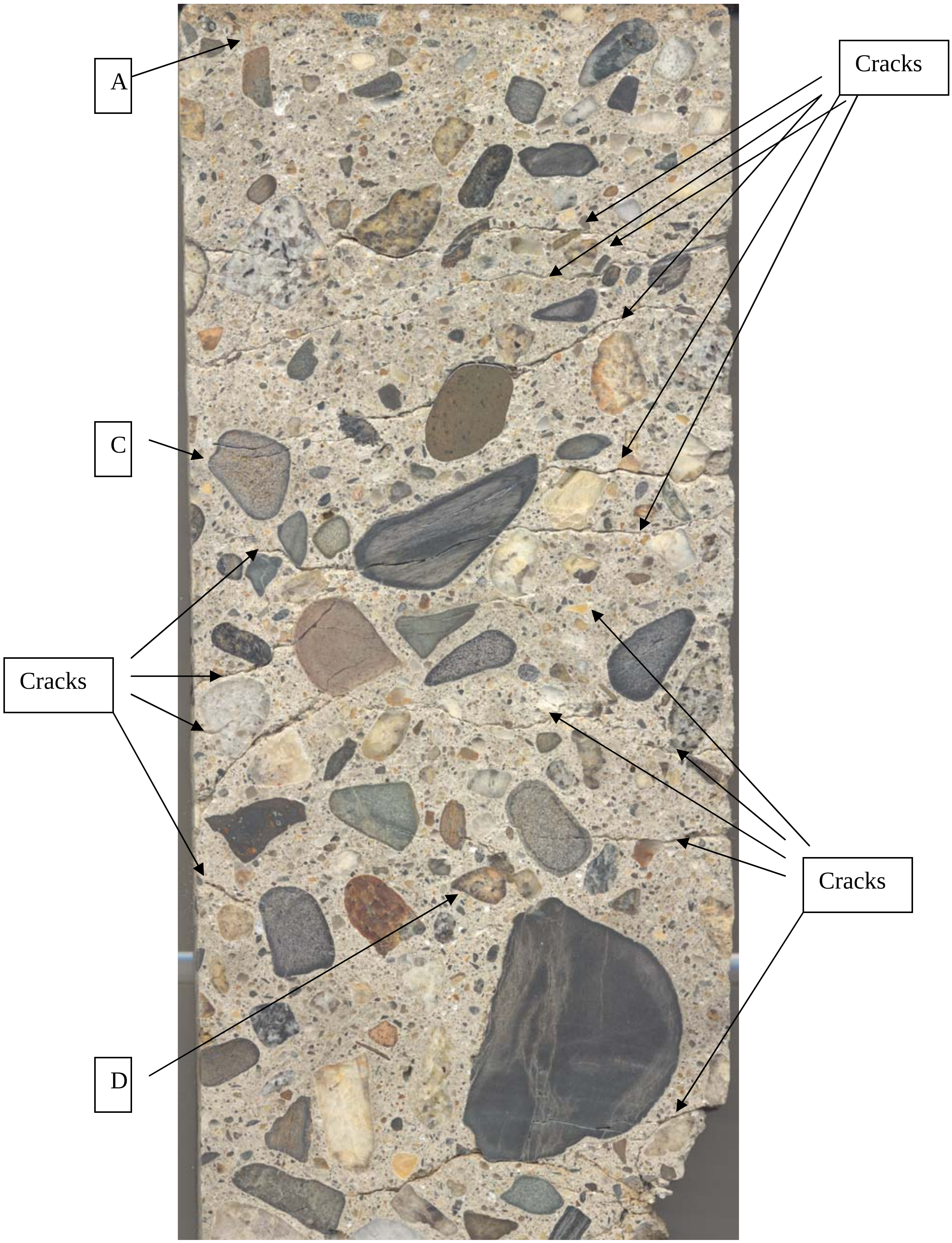

Figure 21 Polished surface of core #12

Figure 22 Section A of Figure 21 core #12

Figure 23 Section C of Figure 21 core #12

Figure 24 Section D of Figure 21 core #12

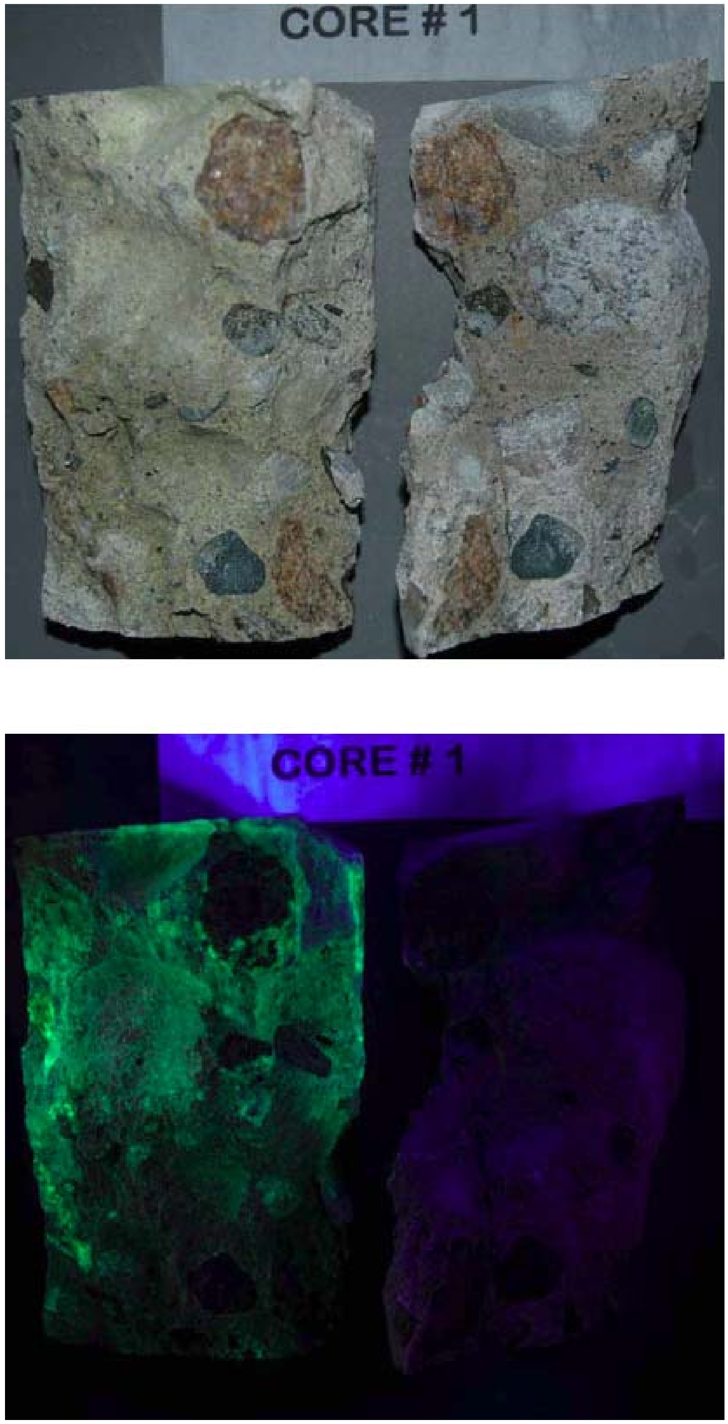

Figure 25 Core #1 compression sample treated with Uranyl acetate dihydrate (left side) under normal light (top) and ultra violet light (bottom)

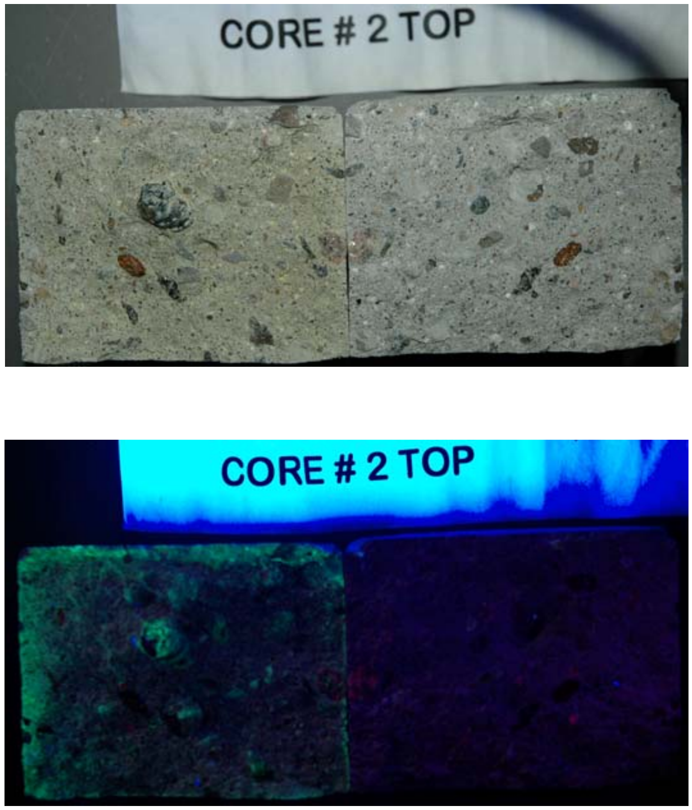

Figure 26 Core #2 tensile sample treated with Uranyl acetate dihydrate (left side) under normal light (top) and ultra violet light (bottom)

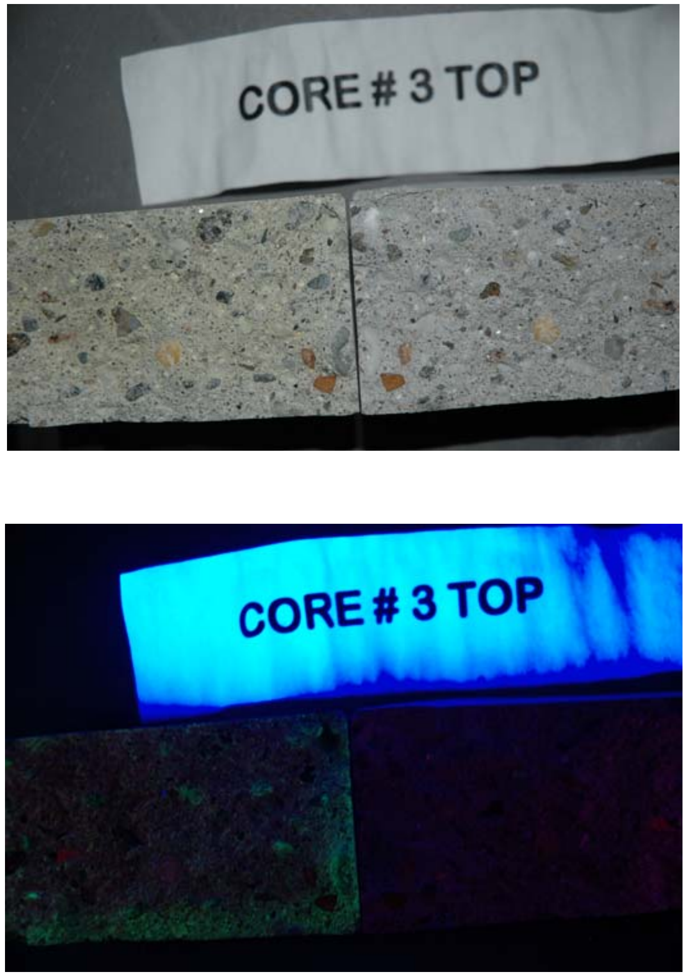

Figure 27 Core #3 Top tensile sample treated with Uranyl acetate dihydrate (left side) under normal light (top) and ultra violet light (bottom)

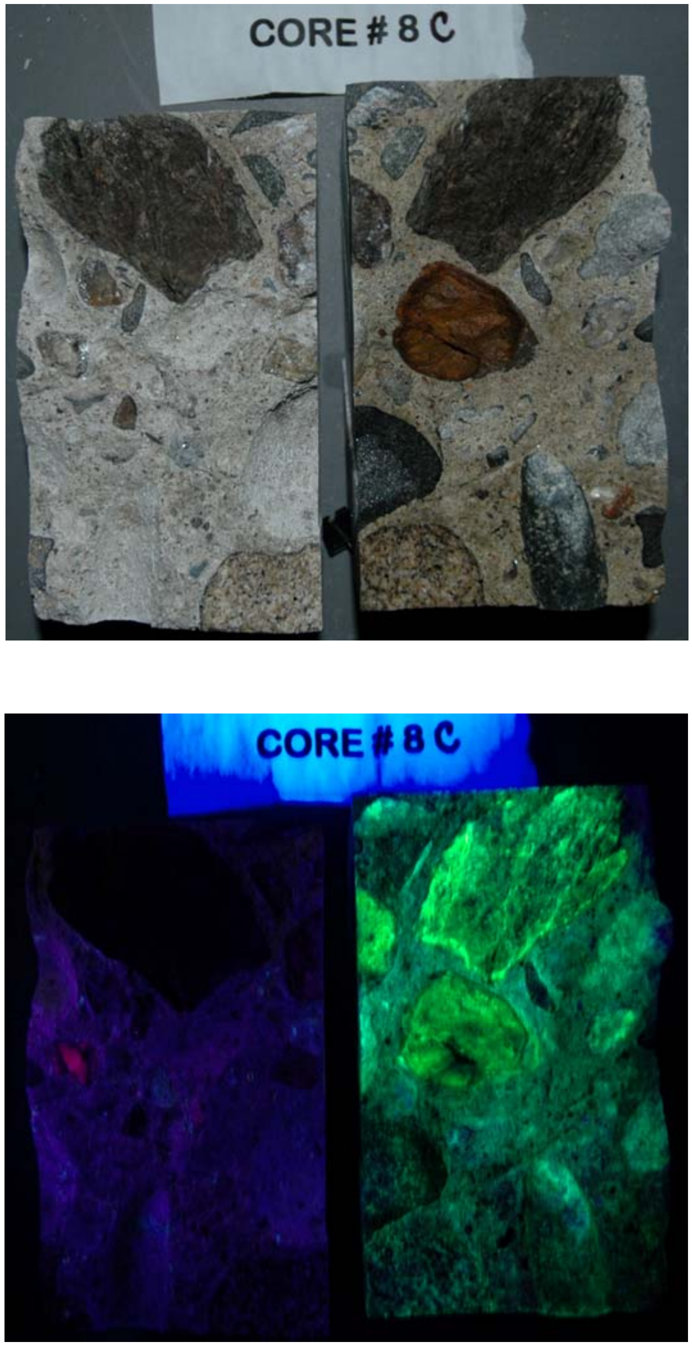

Figure 28 Core # 8C tensile sample treated with Uranyl acetate dihydrate (right side) under normal light (top) and ultra violet light (bottom)

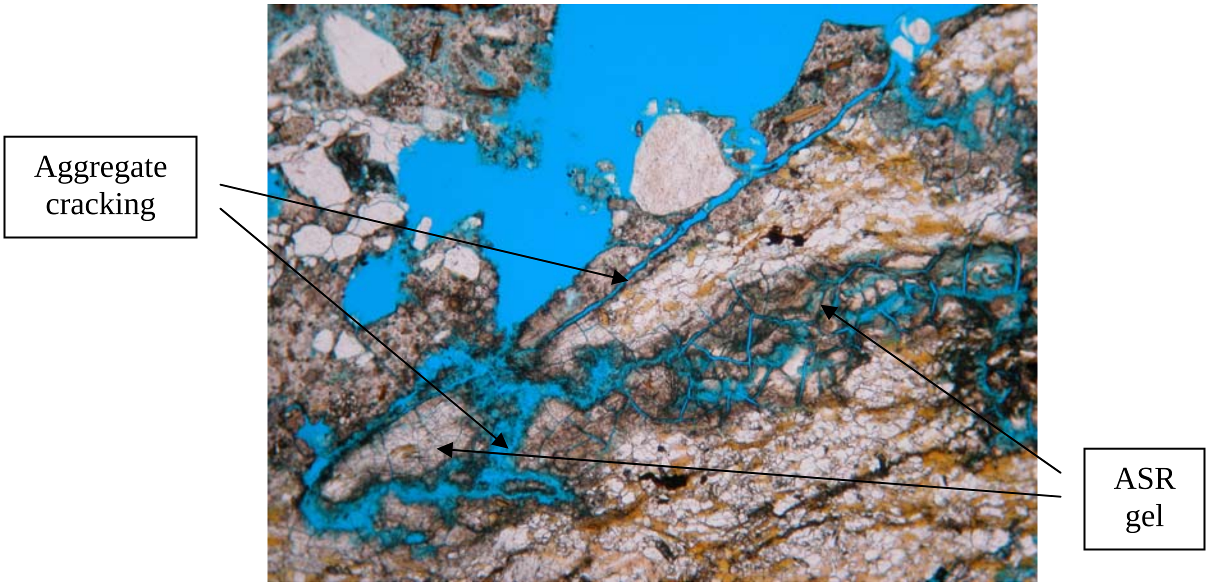

Figure 29 Core 13 vertical section showing ASR gel inside cracked granite coarse aggregate

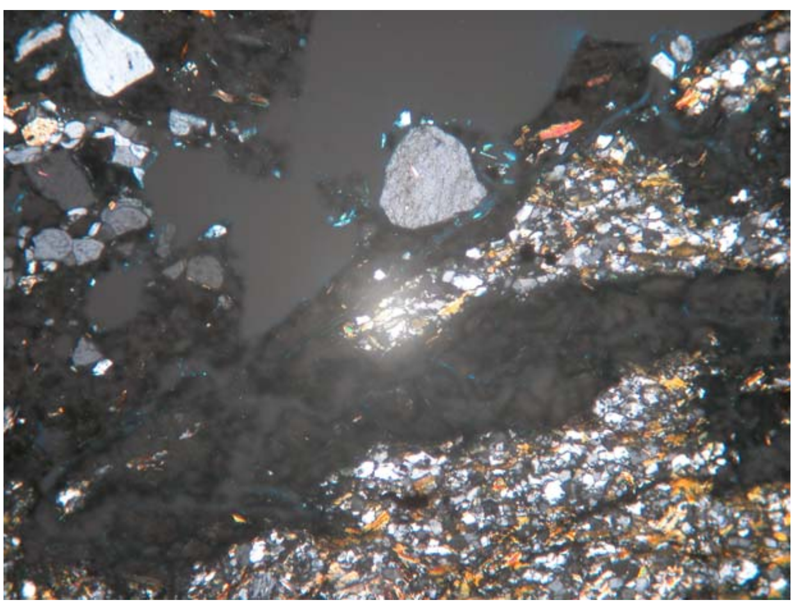

Figure 30 Core 13 vertical section showing amorphous gel (dark portion)

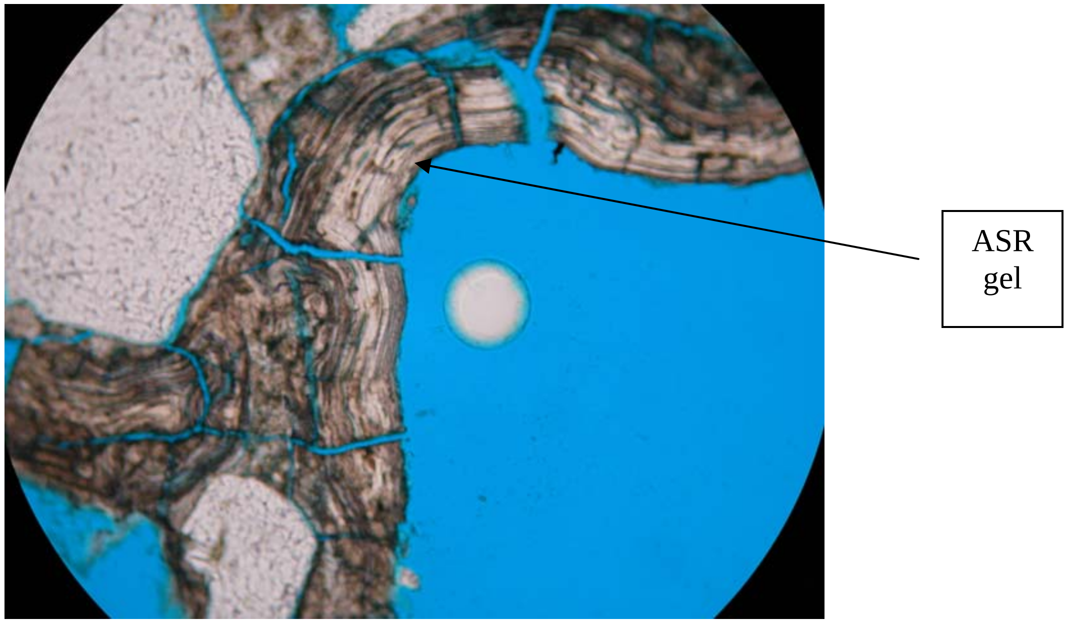

Figure 31 Core 13 vertical section showing layered ASR gel at the edges of a crack

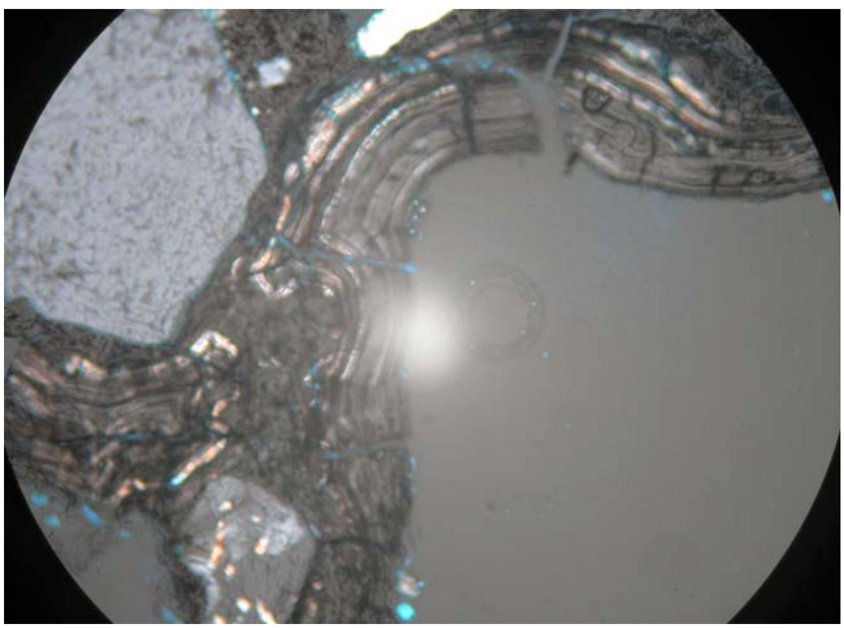

Figure 32 Core 13 vertical section showing the alternate layers of crystalline (colored portion) and non-crystalline gel (dark colored) under crossed polarized light (see Figure 31)

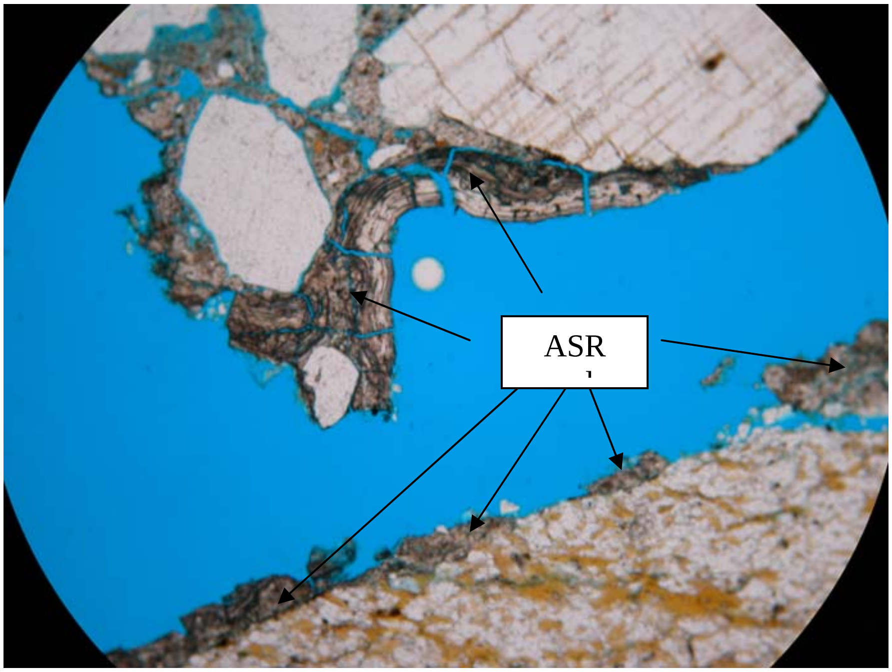

Figure 33 Core 13 vertical section showing a lower magnification view of Figure 31 and 32

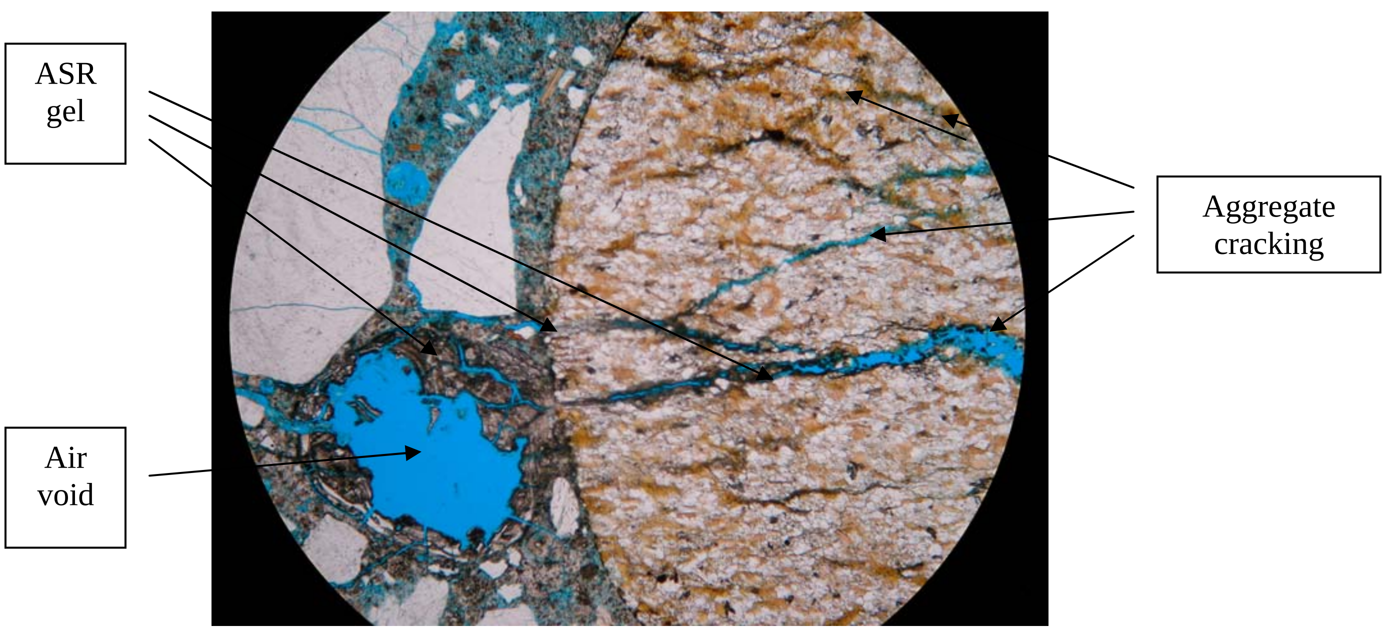

Figure 34 Core 13 vertical section showing aggregate cracking, ASR gel

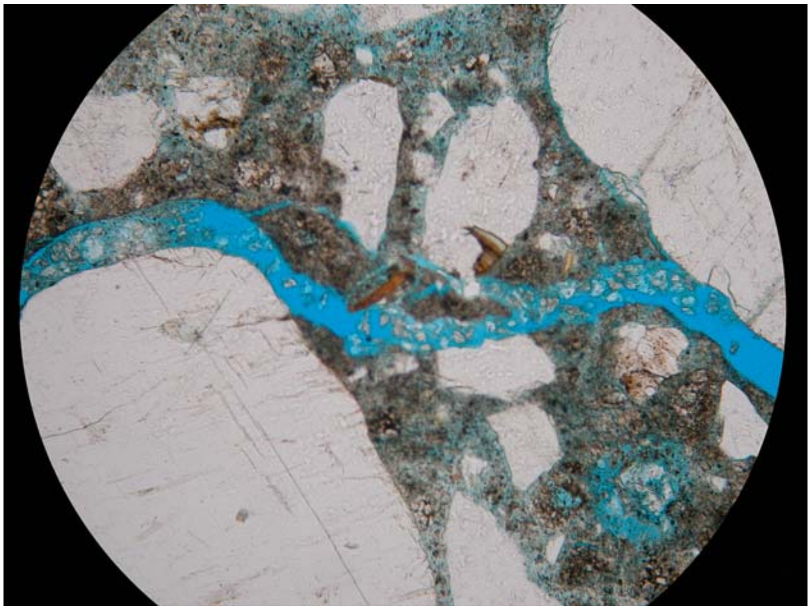

Figure 35 Core 13 vertical section showing Ettringite along the crack at the aggregate-paste interfaces

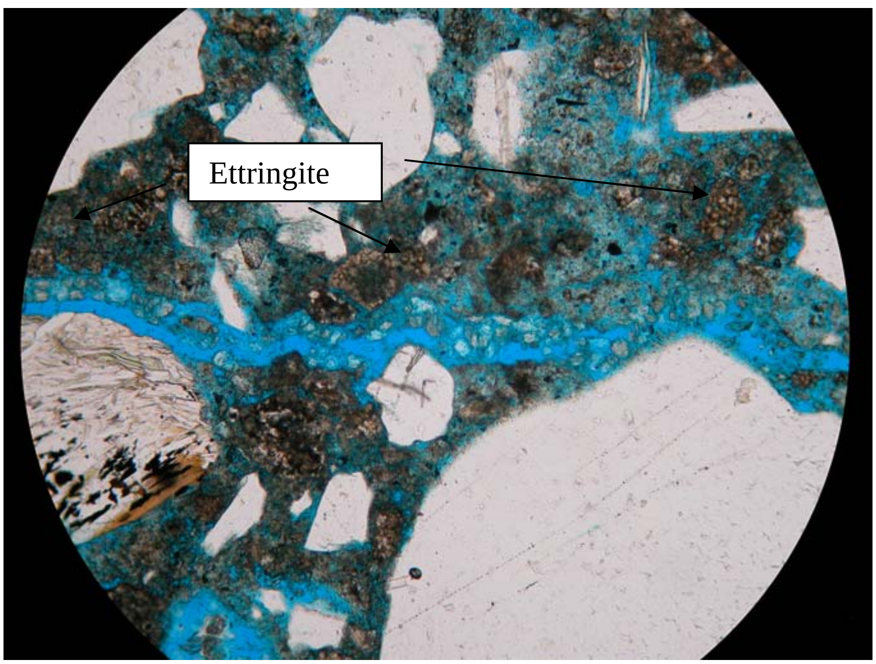

Figure 36 Core 13 horizontal section showing secondary ettringite

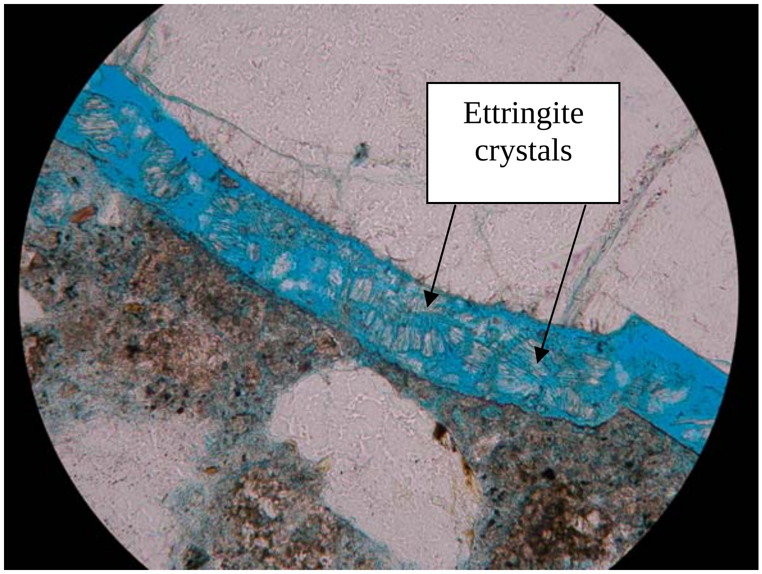

Figure 37 Core 13 horizontal section showing enlarged view of Figure 36

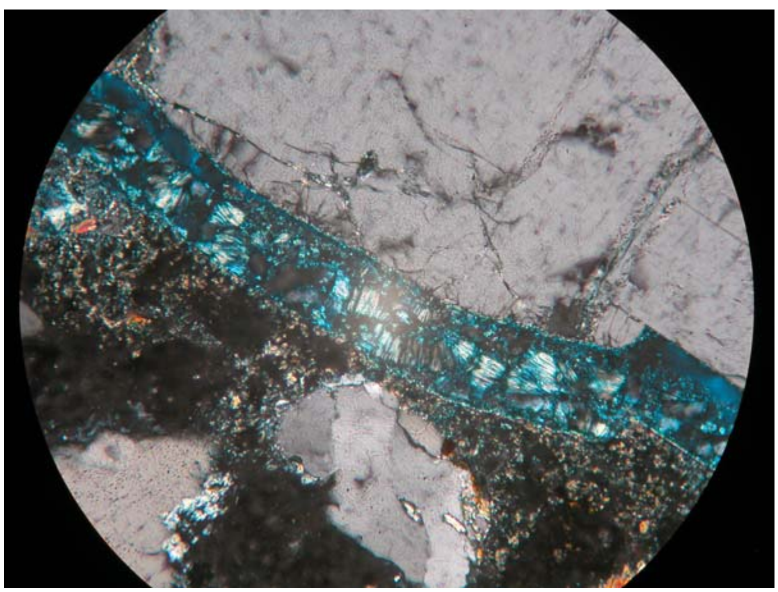

Figure 38 Core 13 horizontal section showing cross polarized light (see Figure 37)

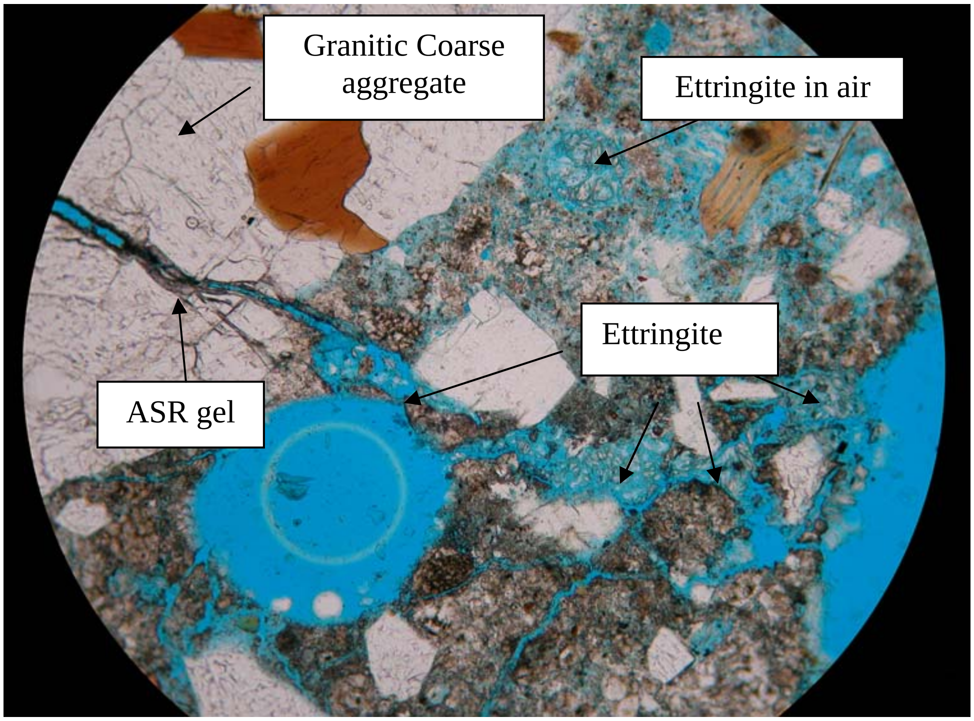

Figure 39 Core 13 horizontal section showing ASR and secondary Ettringite

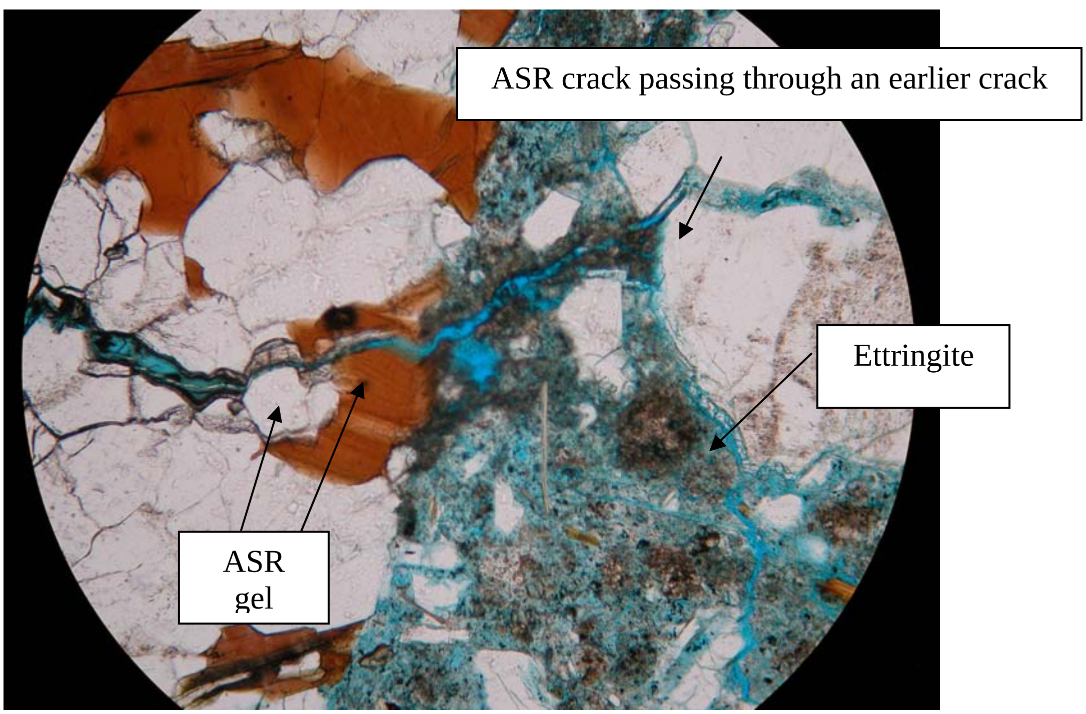

Figure 40 Core 13 horizontal section showing ASR and DEF in the same place

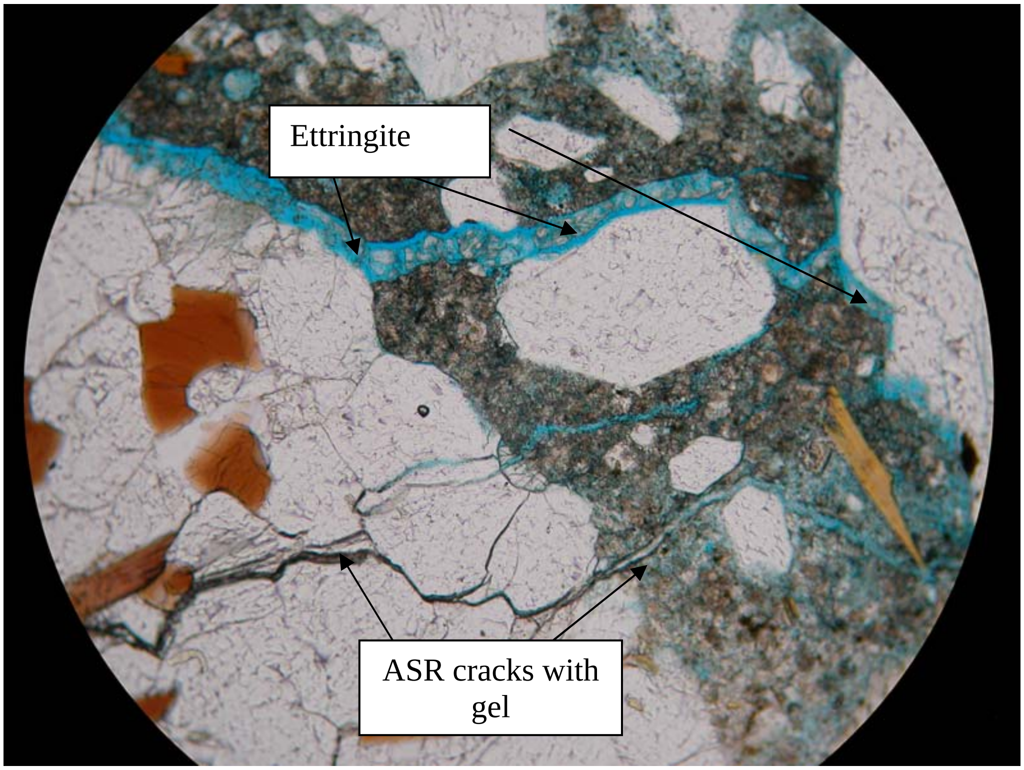

Figure 41 Core 13 horizontal section showing ASR and Ettringite

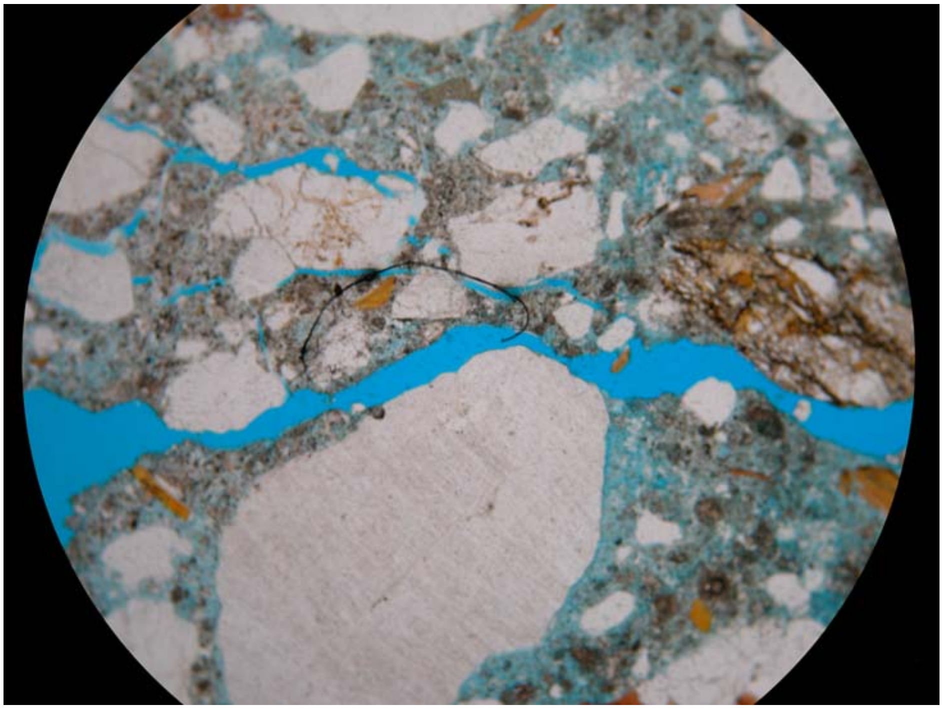

Figure 42 Core 13 horizontal section showing Typical crack pattern

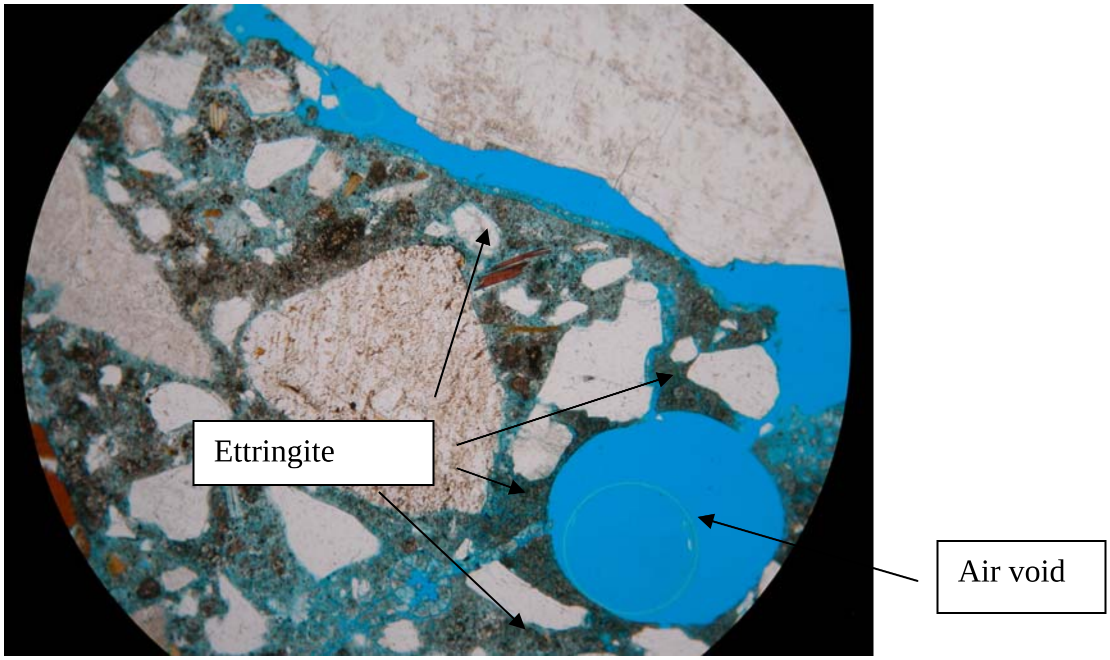

Figure 43 Core 13 horizontal section showing Another example of crack pattern

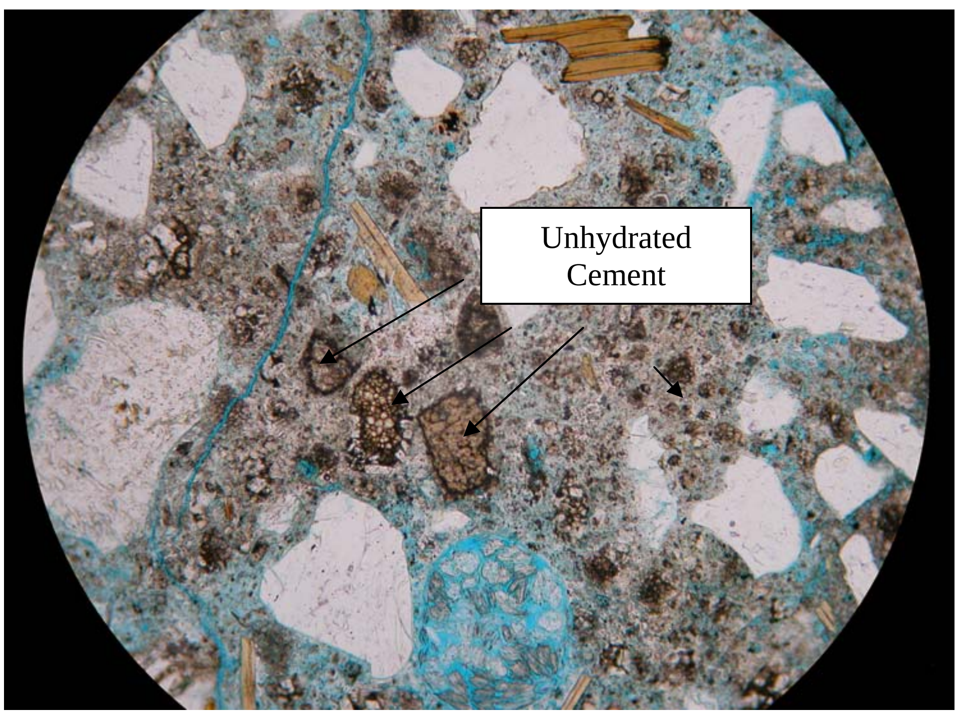

Figure 44 Core 13 horizontal section showing large number of unhydrated, coarser portland cement particles typical of Rosendale cement.

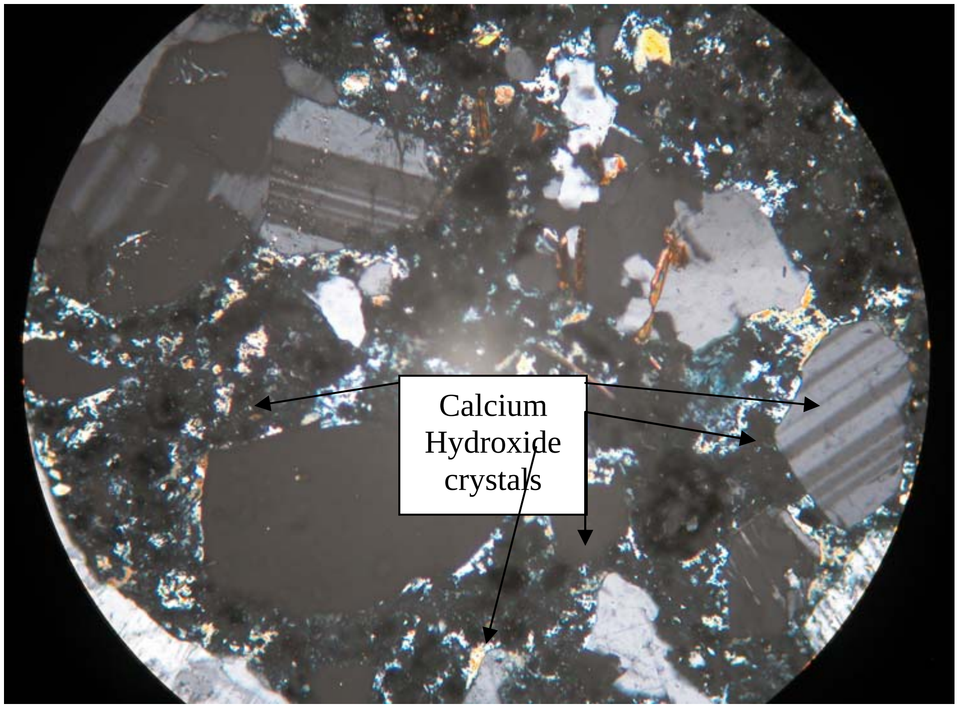

Figure 45 Core 13 horizontal section showing well crystallized, coarser calcium hydroxide crystals at the aggregate-paste interfaces as well as in the matrix.