TranSystems

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October 31, 2017

Friends of Lake Park
Colleen Reilly, PMP, President
PO Box 71197
Milwaukee, WI 53211

RE: Structural Documentation Review of Lake Park Arch Bridge – Phase 1

Dear Ms. Reilly,

TranSystems was contracted by the Friends of Lake Park to provide an independent review of the structural documentation available for the assessment of Lake Park's Arch Bridge over Ravine Road and examine whether or not there is a viable cost-effective rehabilitation alternative that preserves the original design and extends the service life of the rehabilitated bridge beyond that that were previously estimated. These documents included the available 1905 plan sheets, 2015 In-Depth Inspection Report, which included material testing results, site photos and load capacity rating calculations as well as other documentation on the history of the bridge. On September 27, 2017, TranSystems conducted a site visit to confirm current conditions of the bridge with representatives from the Friends of Lake Park and Milwaukee County also in attendance.

***Photo 1**: Lake Park Arch Bridge*

Based on the review of the available documentation, TranSystems recommends a further refined condition assessment of the structure that would clearly distinguish the areas of the bridge that require structural repairs and those areas that need aesthetic structural patching. A better understanding of these types of repairs, the material properties of the existing concrete, and the associated quantities of the repairs could significantly reduce the cost of the proposed rehabilitation alternative, making the rehabilitation of this bridge a cost-effective alternative over other bridge replacement alternatives.

TranSystems has worked extensively in the preservation and rehabilitation of historic bridges throughout the nation. Previous studies performed by TranSystems typically included other testing and analysis to evaluate rehabilitation alternatives of a historic concrete bridge. The results of the other testing and analysis can significantly influence the comparison of the cost-benefit analysis between the alternatives considered. TranSystems recommends considering the following topics in order to thoroughly evaluate historic concrete bridges, and develop feasible and prudent rehabilitation alternatives:

Material Testing

The 2015 In-Depth inspection report included results and discussion of a material testing program that tested the concrete for strength and asbestos. The compressive strength test results of the concrete ranged from 1,595 psi to 9,882 psi. Note the original design plans specifying a concrete compressive strength of 1600 psi. However, TranSystems routinely performs the additional concrete tests to determine if the concrete is adequate for structural repairs or aesthetic restorations:

1. CHLORIDE ION ANALYSIS - Chloride contents at shallow depths of approximately 1½" (relative to the face of the core) within a core sample are generally indicative of chloride penetration from external sources such as road salt, airborne pollutants, or pollutants present in soil. In comparison, the chloride contents at deeper locations in a core sample may be more indicative of the as-built chloride content of the concrete. Large concentrations of chlorides within the concrete near the steel reinforcement promote corrosion of the reinforcement and will determine the effectiveness of any cathodic protection systems or health monitoring systems installed during rehabilitation. Spalling of the concrete may result from ongoing steel reinforcement corrosion.

2. PETROGRAPHIC ANALYSIS - Petrographic analysis is used to identify if an alkali-silica reaction (ASR) is occurring between the minerals in the aggregates and the soluble alkaline components of the cement paste. ASR was poorly understood at the time this bridge was constructed. ASR weakens the cement bond to the large particles within concrete. The presence of ASR will limit the durability of repairs made to the existing concrete bridge, however better concrete mix formulas for the repair areas will ensure compatibility between the two types of concrete.

Photo 2: *Corner spalls with exposed reinforcing steel on the arch rib*

3. FREEZE-THAW ANALYSIS - Freeze-thaw analysis is used to gauge the durability of concrete under repeated cycles of freeze and thaw conditions. For concrete where the cement bond is already compromised, years of freeze-thaw will lead to failure of the cement bond often initially observed as "pop outs" where the stone aggregate separates from the surface of the concrete element.

The outcome of these additional tests can significantly influence the comparison of the cost-benefit analysis between the bridge alternatives and significantly increase the service life of the bridge and concrete repairs to greater than 30 years. A complete understanding of the chemical characteristics of concrete over 100 years old is important to any historic concrete bridge rehabilitation.

Photo 3: *Underside of deck with delaminations*

Live Load Rating

The 2015 load capacity rating analysis for this structure, in our opinion, was conducted with an overly conservative methodology that does not represent the actual capacity of the structure nor the original design intention of the bridge. This bridge was designed and built in the early 1900's before the Allowable Stress Design (ASD) methodology was developed and used widely. But the load rating method used (AASHTO Load and Resistance Factor Rating, or LRFR) was adopted in 2005 and is geared towards newer structures built within the last few decades. While LRFR can be applied to structures of any age, it is our experience that since LRFR is a probability-based, calibrated methodology compared to the more traditional, uncalibrated strength-based Load Factor Rating (LFR) method – the results from LRFR analysis are typically more conservative, often yielding lower allowable capacities when compared to those methodologies in common use closer to the design date of this bridge.

There is no requirement by State or Federal agencies that LRFR method be applied to an historic structure, and it considered acceptable engineering practice to use a load rating method that is in line with the design method of the bridge. Additionally, the LRFR method is for bridges carrying vehicular loads and there is reason for not applying it to a structure designed for pedestrian use.

The main structural arch rib elements of this bridge exceed the load capacity of their original design loads as well as the vehicular load as investigated and reported in the 2015 report. While there are several bridge elements that cannot carry their original design loads based on the current analysis, a more refined analysis utilizing ASD or LFR could significantly change the allowable capacity of those elements which in turn may reduce the amount of structural repairs/strengthening required, and thusly decrease the overall cost required for the rehabilitation alternative.

Condition Assessment
Based on our cursory site visit, there has not been any significant change in the condition of the bridge since the 2015 inspection. Many of the conditions noted in the field are a combination of failures of previous patch work and poor construction practices. T the deck delaminations appear to be due to a lack of concrete cover over the reinforcing steel. This is evident in that the reinforcing bars are still bonded with the concrete for the majority of the circumference of the bars. While section loss of the steel may be present, the amount of loss is most likely not significant. Therefore, cleaning and rehabilitating several of these areas with the exposed reinforcing is a viable option as long as the material testing of the concrete concludes these types of repairs are compatible.

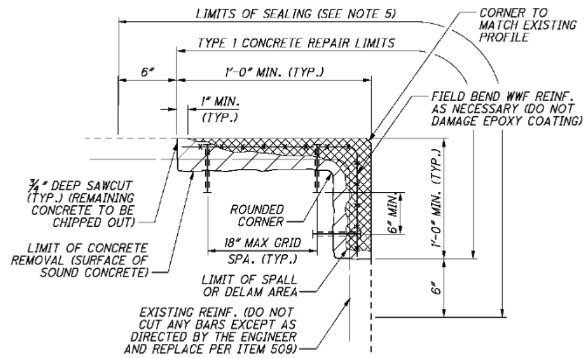
CONCLUSIONS

Based upon our review of the documentation provided by the Friends of Lake Park, TranSystems offers the following engineering opinions for the Lake Park Arch Bridge:

1. Based on the limited material testing data collected on the concrete samples of the original structure, the durability of the existing concrete and the effectiveness of extending the service life of the rehabilitated bridge alternative are inconclusive. Knowing the additional test results and data from the existing 1905 concrete will determine the depth of repairs to remove chloride contamination and ASR while ensuring that a cathodic protection system to limit corrosion in the reinforcing steel will meet the project objectives of extending the service live of the rehabilitated bridge beyond 30 years.

2. A proper analysis of the primary bridge elements should be performed to understand the true load capacity of the bridge. A comparison of engineering methodologies ranging from a basic Capacity/Demand Ratio (Factor of Safety) to ASD, LFR and the completed LRFR analysis will provide a more accurate representation of the capacity of the bridge and any future loading consideration of the exiting bridge.

3. Distinguishing the types of concrete repairs needed for various areas of deterioration would ensure a fair a fair cost-benefit comparison between the proposed alternatives in the 2015 report. Quantifying the types of repair areas into the following five categories could significantly reduce the $1.8M cost estimate:



TYPE 1 REPAIR AT VERTICAL CORNER
(FOR INFORMATION NOT SHOWN SEE TYPE 1 CONCRETE REPAIR)
(NOT TO SCALE)

- Component Replacement In-kind
- Structural Strengthening Retrofit Details
- Structural Repairs (typically greater than 6” in depth)
- Structural Patching (typically 2”-3” depth repair with wire mesh)
- Surface Patching with Sealant

When the distinctive areas of repair types are known, a more refined cost estimate can be developed. While traditional concrete repairs and patching alone will address the spalling and exposed areas of the bridge, coupling them with other methods of rehabilitation will ensure a prolonged service life of a historic structure like this one. For example, using a cathodic protection and health monitoring systems can allow for an “inside view” of the onset of corrosion of the reinforcing, which is especially desirable after chloride contamination and ASR mitigation is completed. A system like this is a powerful tool when used after a finishing treatment, such as fiber reinforced polymer wraps or a penetrating sealants, is installed to rehabilitated areas. By being selective with what repair and rehabilitation options are applied, a viable cost-effective option is attainable that will allow for more than 30 years of service life.

TranSystems’ experience in major rehabilitation of concrete structures from circa 1900 in this region of the country would lead us to believe the Lake Park Arch Bridge can be rehabilitated in a cost-effective manner while extending the service life over 30 years – the design target for rehabilitation. However, to ensure the durability and longevity of the repair work, a complete understanding of the existing concrete is imperative to any feasibility study of preferred alternatives.

Due to the incomplete concrete testing program performed and the overly-conservative load rating analysis of the Lake Park Arch Bridge, TranSystems believes there are multiple combinations of standard industry practices that will provide for a longer service life beyond that previously estimated and could significantly reduce the estimated cost of $1.8M. TranSystems has approached the cost estimating phase on similar rehabilitation projects by providing the owner an “ala carte” selection of restoration alternatives that provide a fair cost-benefit analysis of a new structure versus repairs.

Therefore, TranSystems recommends that additional testing be performed to better identify which type of repair and/or treatment be applied and where. The results would allow for better cost-effective alternatives that can meet the goals of this rehabilitation of the Lake Park Arch Bridge.

Sincerely,
TranSystems Corporation

P.E.

Wesley Weir, PE
Senior Project Manager/Vice President

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June 18, 2018

Colleen K. Reilly, PMP
President, Lake Park Friends
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Re: Lake Park Arch Bridge over Ravine Road – Phase 2: Concrete Testing Results

TranSystems was contracted by Lake Park Friends to conduct the second phase of independent structural engineering services for the Lake Park Concrete Arch Bridge. This phase includes an evaluation of concrete material testing results (performed by others) and a structural analysis of the bridge to better represent the actual capacity of the bridge in comparison to the original design capacity of the bridge. TranSystems previously (as part of Phase 1 services) provided an independent review of the available structural documentation (prepared by others) to help more thoroughly evaluate bridge restoration alternatives. The Phase 1 summary report recommended that additional material tests be performed on the concrete to determine if the Lake Park Ravine Road Bridge's original concrete is suitable for structural repairs and aesthetic restoration (TranSystems, 2017).

This report presents these test results and evaluates their implications for bridge structural repairs or aesthetic restorations. TranSystems is currently performing the structural analysis of the bridge's capacity, however, this will be presented in a separate report.

Lake Park Friends contracted with Giles Engineering Associates, Inc. (Giles) to collected twelve (12) cores and seven (7) sawed beam samples for testing from predefined bridge elements in accordance with ASTM Standards. This work was performed between March 22 and March 23, 2018. The locations for concrete coring were selected in such a manner as to obtain a representative sample of test results for each type of unique structural element on the bridge. Note that eight (8) sawed beam samples were initially collected, but one sample (deck near the north abutment) was omitted due to deterioration of the sample.

The testing program included the following:

- Eight (8) cores were tested for chloride content by the rapid chloride method (equivalent to AASHTO T260) at three depths.
- Four (4) cores underwent petrographic analysis per ASTM C856 with air void analysis per ASTM C457.
- Seven (7) beam samples received free-thaw testing per ASTM C666, Procedure A.

These test results are provided as an Attachment to this report.

In addition, Giles previously performed concrete testing on the structure with results summarized in their "Concrete Coring and Testing" report dated April 24, 2015. Six (6) cores were taken from the thrust blocks on March 17, 2015 and underwent unconfined compression testing per ASTM C39. The results of these tests are also discussed in this report because the compression strength of the concrete determines the load-carrying capacity of the structural elements within the bridge.

CHLORIDE CONTENT

Chloride content testing is performed in order to measure the amount of chloride ions that have penetrated into a concrete sample. These chloride ions can accelerate the initiation of corrosion of steel reinforcement within a structure. This corrosion impacts the structural integrity of reinforced concrete and can reduce the effective life of the structure. Eight core samples were selected for chloride ion content analysis as shown in the figure in Attachment 1: Core 1 (south end of deck), Core 2 (center of deck), Core 4 (south vault wall of abutment), Core 5 (south end of east arch rib), Core 6 (south thrust block), Core 8 (west face of floorbeam), Core 10 (east spandrel column), and Core 11 (east arch rib at center).

Chloride contents located at shallow depths of approximately 1 1/2" (relative to the face of the core) within a core sample are generally indicative of chloride penetration from external sources such as road salt, airborne pollutants, or pollutants present in soil. In comparison, the chloride contents at deeper locations in a core sample may be more indicative of the as-built chloride content of the concrete. Based on TranSystems' experience with rehabilitation of reinforced concrete structures based on guidance from various state agencies, active corrosion of reinforcement is assumed to be taking place when chloride ion content greater than 2.0 lb/yd^3 is present in the concrete. This threshold can serve as a guideline for determining preferred alternatives for the rehabilitation or replacement of these bridge elements.

Results of the chloride content testing are shown in **Table 1**.

The chloride content of the samples ranges from 0.07 lb/yd^3 to 2.19 lb/yd^3 at shallow depths and from 0.22 lb/yd^3 to 0.84 lb/yd^3 at deeper depths. Cores 1 and 2 exhibited higher level of chlorides, which would be expected for a deck surface. Only one sample (Core 2 at shallow depth) had chloride content greater that the 2.0 lb/yd^3 limit. Note that the results at deeper depths (2 inches to 3 inches) are a more significant measure of the chloride content within a concrete sample because this indicates whether chlorides are penetrating deep into the original concrete. Since reinforcing steel is placed with a minimum level of concrete cover (typically 2 inches), chlorides present near the surface of a concrete sample are unlikely to provide a substantial corrosive potential in the reinforcement. The values recorded at deeper depths are minimum and do not show chlorides penetrating deep into the original concrete. Overall, the chloride content testing results are generally within acceptable ranges for a reinforced concrete bridge rehabilitation.

Core Number	Depth	Chloride Content			
		% by Weight	ppm	kg/m^3	lb/yd^3
1 (Deck)	0-1 inch	0.082	820	0.820	1.38
	1-2 inches	0.035	350	0.350	0.59
	2-3 inches	0.047	470	0.470	0.79
2 (Deck)	0-1 inch	0.130	1300	1.300	2.19
	1-2 inches	0.035	350	0.350	0.59
	2-3 inches	0.022	220	0.220	0.37
4 (Abutment)	0-1 inch	0.024	240	0.240	0.40
	1-2 inches	0.022	220	0.220	0.37
	2-3 inches	0.013	130	0.130	0.22
5 (Arch)	0-1 inch	0.004	39	0.039	0.07
	1-2 inches	0.023	230	0.230	0.39
	2-3 inches	0.023	230	0.230	0.39
6 (Thrust Block)	0-1 inch	0.037	370	0.370	0.62
	1-2 inches	0.023	230	0.230	0.39
	2-3 inches	0.018	180	0.180	0.30
8 (Floorbeam)	0-1 inch	0.034	340	0.340	0.57
	1-2 inches	0.053	530	0.530	0.89
	2-3 inches	0.038	380	0.380	0.64
10 (Spandrel Wall)	0-1 inch	0.038	380	0.380	0.64
	1-2 inches	0.094	940	0.940	1.58
	2-3 inches	0.050	500	0.500	0.84
11 (Arch)	0-1 inch	0.048	480	0.480	0.81
	1-2 inches	0.039	390	0.390	0.66
	2-3 inches	0.037	370	0.370	0.62

Table 1 – Chloride ion content test results (green indicates favorable results, yellow indicates levels between 1.0 lb/yd^3 and 2.0 lb/yd^3, and red indicates levels above 2.0 lb/yd^3).

Furthermore, chloride ion content is able to be addressed through both preventative maintenance and active mitigation techniques, and these actions are commonly taken during the rehabilitation of concrete bridges. For this bridge, galvanic (or "sacrificial") anodes would be a good option within new concrete patches. These devices are made from a metal alloy with a more active voltage than the reinforcing steel including in concrete. When these anodes are attached to reinforcement, the difference in voltage potential results in the anodes corroding instead of the reinforcing steel. This is a cost-effective means of extending the life of the reinforcement within a concrete structure. In addition, there are several means of reducing the chloride ion content within an existing structure, including electrochemical chloride extraction (ECE) and impressed current cathodic protection.

PETROGRAPHIC ANALYSIS

Petrographic analysis consists of utilizing microscopes in order to examine the physical and chemical makeup of concrete samples. This analysis can help determine a range of parameters, including: the type, proportions, shape, and condition of aggregates; nature of the cement paste; depth of carbon; bond between paste and aggregate; evidence of alkali-silica reaction; and measures of air entrainment. Among the most important factors evaluated during a petrographic analysis, alkali-silica reaction (ASR) is a swelling reaction that occurs over time in concrete between these materials within concrete. This reaction exerts an expansive pressure within the aggregate, which can result in spalling, serious cracking, or loss of strength in the concrete. Four beam samples were selected for petrographic analysis and air void analysis: Core 3 (north end of deck), Core 7 (top of north thrust block), Core 9 (west arch rib), and Core 12 (west spandrel column).

From the petrographic analysis, no significant evidence of alkali-aggregate reaction was observed in any of the samples. Two of the four cores exhibited small amounts of colorless to white silica gel partially lining the air voids within the concrete; however, this minor presence of siliceous material has not resulted in the development of ASR. The petrographic analysis also indicated that paste/aggregate bond varied from "fair to good" to "good" in the beam samples, and the concrete was originally placed with a high water-to-cement ratio.

In addition, it was determined in the petrographic analysis that the original concrete in the four beam samples were not air-entrained. Air entrainment of concrete did not become a common practice until the 1930's, therefore, concrete without an air void system would not meet the current American Concrete Institute (ACI) recommendations for freeze-thaw resistance.

FREEZE-THAW ANALYSIS

Freeze-thaw analysis is used to gauge the durability of concrete under repeated cycles of freeze and thaw conditions. Seven total beam samples were tested for freeze-thaw resistance: Beam 1 (top of east parapet, north end), Beam 2 (bottom of center floorbeam), Beam 3 (first floorbeam north of midspan), Beam 4 (east arch rib near south thrust block), Beam 5 (west spandrel wall between openings), Beam 7 (south thrust block), and Beam 8 (west arch rib near circular opening). The concrete tested from the south thrust block (Beam 7) consisted of concrete from a more modern rehabilitation project, while the remaining six beam samples tested consisted of concrete original to the structure. Results of the freeze-thaw testing are shown in **Figure 1**.

The results of the freeze-thaw analysis confirmed that without an effective air-entrained void system, the relative dynamic modulus of elasticity reduced quickly when subjected to numerous freeze-thaw cycles. According to ASTM C666, the specimens being tested are subjected to cycles of freezing and thawing, and their frequencies are then measured to determine the change in relative modulus as a function of these cycles. The tests are to be continued until the specimen reaches the desired 300 cycles or until its relative dynamic modulus of elasticity is drops below 60% of the initial modulus, whichever comes first, unless other limits are specified.

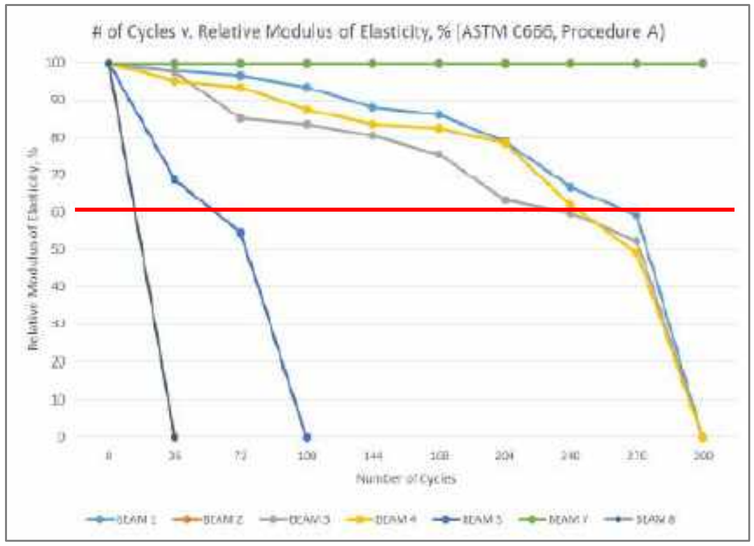

Figure 1 – Freeze-thaw analysis test results (from Giles Engineering Associates, Inc. report dated June 14, 2018).

Based on the test results, the relative dynamic modulus of Beams 2 and 8 (center floorbeam and west arch rib, respectively) reduced to zero after the initial 36 cycles. The relative modulus of Beam 5 (west spandrel wall) dropped below the 60% threshold between 72 and 108 cycles. Beam 1 (top of east parapet), Beam 3 (first floorbeam north of midspan), and Beam 4 (east arch rib near south thrust block) dropped below the 60% threshold between 240 and 276 cycles. Beam 7 (south thrust block) did not experience any notable reduction in its dynamic modulus of elasticity and remained in good condition over the full 300 freeze-thaw cycle; this is because Beam 7 consisted of modern concrete with an effective air-entrained void system which allows for water trapped within the concrete to freeze and thaw freely without causing damage.

Although WisDOT does not have a specification for relative dynamic modulus of elasticity or durability factor, the United Stated Army Corps of Engineers and Federal Aviation Administration typically require projects to have a relative dynamic modulus of elasticity greater than 75% for new concrete. All beam samples except Beam 7 fell below this targeted threshold for modern job mix formulas for concrete. This would be expected with concrete from this era which lacks an effective air void system to accommodate freeze-thaw cycles within the specimens. However, while these test results suggest that the original concrete has a less than desirable resistance to freeze-thaw cycles, the testing procedure clearly states that “neither procedure [from ASTM C666] is intended to provide a quantitative measure of the length of service that may be expected from a specific type of concrete.” As a result, the remaining service life of the concrete cannot be concluded from this freeze-thaw analysis.

While the freeze-thaw analysis shows that the bridge has less than desirable durability against cycles of freezing and thawing, this is not an indicator of the structural integrity of the bridge. The compressive strength of the structure dictates the load-carrying capacity of the bridge. Chloride content and the petrographic analysis provide information regarding the condition of the structure and possible deterioration that has occurred over time. The lack of freeze-thaw resistance in this structure can be partially mitigated by reducing the infiltration of water inside structural members, which can be done by sealing the concrete surfaces or improving the drainage system on the structure.

COMPRESSIVE STRENGTH

Compressive strength is one of the most important engineering properties of concrete. The concrete compressive strength, typically measured in pounds per square inch (psi), indicates the ability of a structural member to resist compressive forces. Because concrete is typically unable to resist large tensile forces, steel reinforcement is included within the concrete to resist tension. Together, the compressive strength of concrete and yield strength of reinforcement dictate the load-carrying capacity of a structural member.

In March 2015, six compressive strength test cores were taken, one from each of the three faces of the north and south abutment thrust blocks. The results of the tests were included in the In-Depth Inspection Report by GRAEF dated July 2015, and are summarized in **Table 2**.

Core Number	Thrust Block	Location	Compressive Strength (psi)
1	North	West Side	2494
2		Center	9882
3		East Side	3166
4	South	Center	7543
5		West Side	1595
6		East Side	1763

Table 2 – Compressive strength test results for concrete testing from In-Depth Inspection Report by GRAEF dated July 2015.

The compressive strength of the tested core samples ranged from 1,595 psi to 9,882 psi. Two of the core samples (Cores 2 and 4) were obtained from concrete that was used to patch the surface of the thrust blocks during a rehabilitation of the structure. As such, the compressive strengths of these elements were much higher than the concrete from original construction, with compressive strengths of 9,882 psi and 7,543 psi for Cores 2 and 4, respectively. The compressive strength test results for the remaining four cores consisting of concrete original to the structure ranged from 1,595 psi to 3,166 psi, with an average compressive strength of 2,255 psi.

Overall these results are consistent with expected values used by AASHTO and several state agencies. According to the *WisDOT Bridge Manual* and AASHTO's *Manual for Bridge Evaluation*, a minimum compressive strength (f'_c) of 2.5 ksi (2500 psi) can be used for structures built prior to 1959 if the strength is unknown and the concrete is in satisfactory condition, while some other agencies recommends a minimum compressive strength of 2.0 ksi (2000 psi) for concrete prior to 1930. Given the test results, these guidelines, and the age of the concrete, these results are consistent with the concrete strength of 2.0 ksi (2000 psi) used in the previous load rating analysis. Note that the original design plans state an allowable stress of 400 psi, which would suggest a compressive strength of 1600 psi based on a factor of safety of 4. However, higher compressive strengths can be considered in an analysis if supported by the results of material testing.

CONCLUSIONS

Based on the concrete testing results, we offer the following summary by bridge element type in **Table 3** below:

Bridge Element	Compressive Strength (psi)	Chloride Content (lb/yd^3)			Petrographic / ASR	Freeze-Thaw (Cycles to 60% dynamic modulus)
		0"-1" Depth	1"-2" Depth	2"-3" Depth		
Deck / Rail	NA	1.38 - 2.19	0.59	0.37 - 0.79	Minor alkali-silica gel, fair to good bond	276
Floorbeam	NA	0.57	0.89	0.64	NA	36 - 276
Spandrel Wall / Column	NA	0.64	1.58	0.84	Fair to good bond	72 - 108
Arch Rib	NA	0.07 - 0.81	0.39 - 0.66	0.39 - 0.62	Fair to good bond	36 - 240
Thrust Block	1595 - 3166 (original), 7543 - 9882 (new)	0.37	0.23	0.18	Minor alkali-silica gel, "plugs", good bond	> 300 (no reduction due to freeze-thaw)

Table 3 – Summary of concrete testing results by bridge element (green indicates favorable results, yellow indicates neutral results, and red indicates unfavorable results). Note that NA indicates a test that was not performed on that type of bridge element.

The concrete testing results indicate that the Lake Park Arch Bridge over Ravine Road could be rehabilitated to meet the project objectives of ensuring a long-term service life pedestrian bridge that provides safe pedestrian passage while maintaining structural integrity and load capacity for at least 50 years. None of the concrete testing results reviewed would eliminate rehabilitation as a feasible alternative for this structure, nor would they prevent remediation of the conditions found in these tests results which are typical for historic concrete bridge rehabilitation projects.

If you have any questions, comments, or require further information, please contact me at wrweir@transystems.com or 216-408-5394.

Very truly yours,

P.E.

Wesley Weir, P.E.
Senior Bridge Engineer / Vice President

IMPORTANT

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May 23, 2018
Revised June 14, 2018

Lake Park Friends
2975 North Lake Park Road
Milwaukee, WI 53211

Attention: Ms. Colleen Reilly
President

Subject: Presentation of test data for existing concrete for
Lake Park Footbridge over Ravine Road
Milwaukee, Wisconsin
Giles Project No. 1M-1803020-REV

Dear Ms. Reilly:

In accordance with our proposal (No. 1MP-1711053, dated November 30, 2017) we performed sampling and testing of the existing concrete for the Lake Park Footbridge. Our client requested that the final version of our report include only a description of the test program and presentation of the resulting test data. No engineering opinion regarding the quality of the concrete or recommendations regarding the anticipated effects of continued aging of the bridge are presented herein. We understand that the structural engineer of record will accept full responsibility for interpreting the test information contained herein and providing such conclusions and recommendations.

SAMPLING PROGRAM

Sampling was performed on March 22^{nd} and 23^{rd}, and included collection of twelve (12) cores and seven (7) sawed beams (one of the beam sample locations was omitted per your direction). The attached site sketch depicts the locations of the samples as well as sample identification numbers that correlate to the laboratory test reports. Prior to coring and cutting operations, concrete radar imaging was performed to avoid cutting or coring through the embedded reinforcing steel. We were able to collect samples in such a manner that no reinforcing steel was severed in the sampling process. Immediately after coring/cutting, the samples were wiped off to remove drill water. Once surfaces appeared dry (generally within 5 to 15 minutes of removal) the samples were sealed in plastic bags. Depth control was employed to prevent coring completely through the members being cored. Accordingly, core samples were broken off at the desired depth. After removal of the concrete core and beam specimens, the holes within the structure were filled with Xypex Patch'n Plug – a fast-setting non-shrink hydraulic cement mixture that seals holes from water penetration (a material properties data sheet can be provided upon request). The samples were then brought to our laboratory for subsequent processing and testing.

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TESTING PROGRAM

The project scope included the following tests and quantities thereof: four (4) cores were to undergo petrographic analysis (ASTM C856) which included air void analysis (ASTM C457); eight (8) cores were tested for chloride content by the rapid chloride method (equivalent to AASHTO T260) at three (3) depths in each core (0 to 1 inch, 1 to 2 inches, and 2 to 4 inches); and eight (8) beam samples (later reduced to seven beams) for freeze-thaw testing (ASTM C666, Procedure A).

Upon arrival of the samples in the laboratory, four core samples were selected for petrographic analysis. These were shipped in protective packaging to American Petrographic Services located in St. Paul, MN.

The remaining eight cores were assigned for water-soluble chloride content testing. The cores were thoroughly cleaned off with a dry towel to remove fine material adhering to the specimens as a result of the coring process. Dust samples were collected with a masonry drill at the required test depths for each core. The dust samples were tested using the RCTW test system.

The seven beam specimens were carefully trimmed to size, wiped clean of fine material created by the sampling and trimming, and then placed in a 73°F lime saturated water bath as required prior to freeze-thaw testing. After the 48-hour minimum duration for soaking in the lime bath was concluded, initial weights and readings of the fundamental transverse frequency were recorded for each beam. The specimens were then loaded into the freeze-thaw chamber and freeze-thaw cycles were initiated. Per the test method, empty slots within the chamber were filled with dummy specimens to maintain even temperatures within the chamber during test cycles. At intervals not exceeding 36 cycles, samples were removed, one at a time, from the chamber during a thawed period and tested for the fundamental transverse frequency before being returned for continued testing.

TEST RESULTS

Chloride Content

Cores 1 (south end of deck), 2 (center of deck), 4 (south vault wall of abutment), 5 (south end of east arch rib), 6 (south thrust block), 8 (west face of floor beam), 10 (east spandrel column), and 11 (each arch rib at center) were tested for this scope item. The attached Laboratory Report for Rapid Chloride Testing provides the test results. It must be noted that other than cores 1 and 2, samples were drilled horizontally, meaning that the depths of testing were depths into the cored member as opposed to vertical depths. Cores 1 and 2 exhibited levels of chloride content ranging from 220 to 1300 parts per million (ppm). Chloride content of core 10 was 380 to 940 ppm. Core 6 was 180 to 370 ppm, core 8 was 340 to 530 ppm), and core 11 was 370 to 480 ppm. Core 4 was 130 to 240 ppm and core 5 was 39 to 230 ppm.

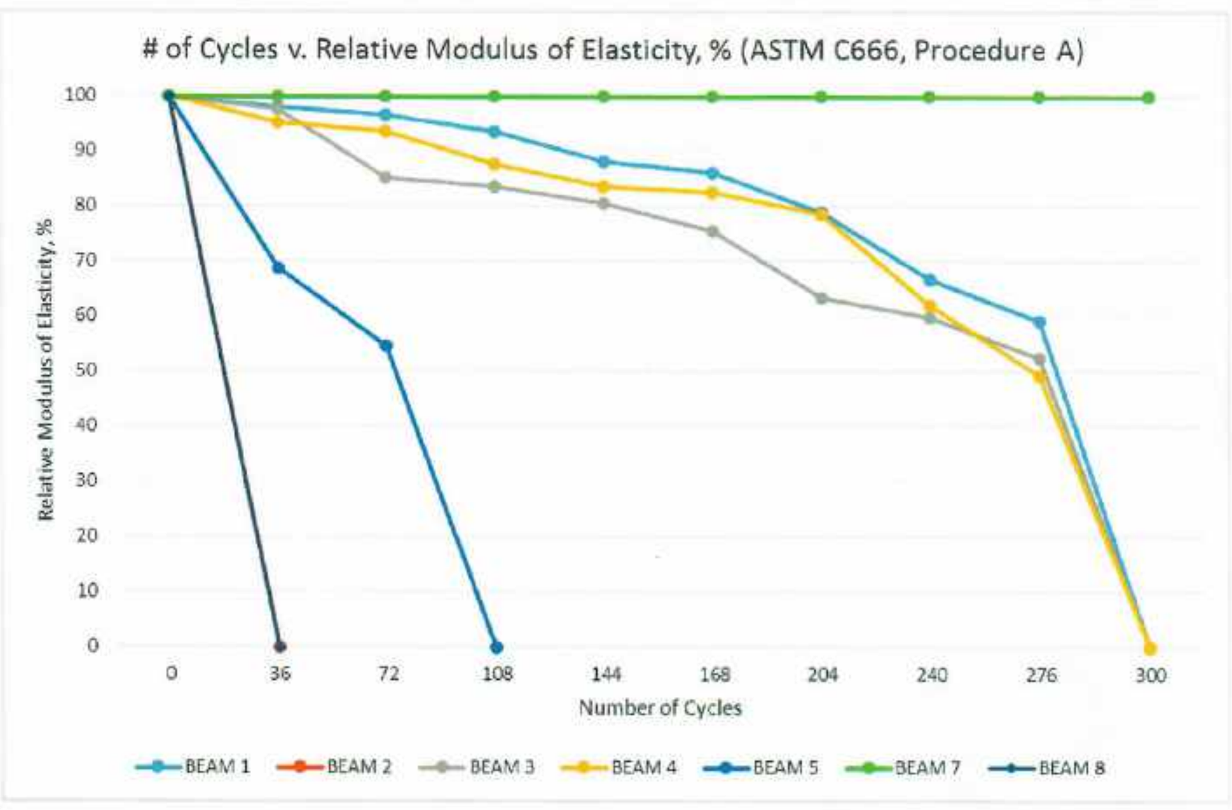
Petrographic Analysis

Cores 3 (north end of deck), 7 (top of north thrust block), 9 (west arch rib), and 12 (west spandrel column) were sent for petrographic analysis. The full report of the testing and analysis performed by the petrographer is attached.

Freeze-Thaw Resistance

Beams 1 (top of east parapet, north end), 2 (bottom of center floor beam), 3 (first floor beam north of mid-span), 4 (east arch rib near south thrust block), 5 (west spandrel wall between openings), 7 (south thrust block), and 8 (west arch rib near circular opening) were tested for freeze-thaw resistance. It should be noted that the south thrust block (represented by beam 7) differed with regard to concrete type; the concrete in this area was apparently part of a more modern rehabilitation project.

For this test, a sample is considered to have failed the parameters of the test if readings fall below 60 percent of the initial modulus. Beams 2 and 8 degraded prior to the conclusion of the initial 36 cycles to the degree that the remaining specimens were untestable. Beam 5 results fell below 60 percent of the initial modulus between 36 and 72 cycles. Beams 1, 3, and 4 fell below 60 percent between 240 and 276 cycles (beams 3 and 4 probably toward the earlier portion and beam 1 in the latter portion of that 36-cycle interval). Beam 7 (which consisted of a modern era concrete patch) did not exhibit a change in readings through the full 300 cycles. The picture below depicts the test results in graphic form. The lines for beams 2 and 8 coincide on the graph, going from 100 to 0 percent after 36 cycles.



CLOSING

As indicated above, no conclusions regarding the evaluation of the test data collected or recommendations regarding the condition of the in-place concrete were requested. We understand that the structural engineer of record will review the data contained herein to provide such services.

Please let us know if you have any questions or if additional information is required.

Thank you,

GILES ENGINEERING ASSOCIATES, INC.

Angela A. Jacobi
Assistant CMT Division Manager
ajacobi@gilesengr.com

Steven P. Homar
Steven P. Homar, P.E.
Materials Testing Division Manager
shomar@gilesengr.com

Enclosures: Site Sketch of Sampling Locations
Laboratory Report for Rapid Chloride Testing, R-180314
Report of Concrete Petrography Testing, 10-09372

Distribution: Lake Park Friends
Attn: Ms. Colleen Reilly (1 via email: colleen.reilly@ch2m.com)

Half Floor Plan
Half Plan showing lateral bracing and abutment.
Half Elevation
Half Longitudinal Section
Cross-Section at "AA" looking toward abutment
Cross-Section near ctr. showing bracing. Similar bracing in all other cross walls.
North
Core 1
Core 2
Core 3
Core 4
Core 5
Core 6
Core 7
Core 8
Core 9
Core 10
Core 11
Core 12
cancelled
East Parapet
Deck
Base of West Parapet
South Abutment Vault South Wall
Floor Beam - bottom
Floor Beam
West Spandrel Column
West Spandrel Wall
East Spandrel Column
West Arch Rib
East Arch Rib
Top of North Thrust Block
Thrust Block
Face of South Thrust Block
Face of Floor Beam
B- Location canceled per client.
SPECIFICATIONS
Live Load. 80 lbs. per sq. ft.
Maximum allowable pressure on earth = 1 Ton to sq. ft.
" stress for steel = 16,000 lbs. per sq. in.
" Compressive " " concrete = 400 " " " "
" shearing " " " = 50 " " " "
No tensile stress figured upon for concrete.
Concrete Proportions;
1 : 2 : 4 Floor slab, coping, & parapet
1 : 2 : 5 All other parts, except that hearting of abutments shall be 1 : 3½ : 7
Finish Mortar { 1 Part Portland Cem. / 1½ " Janesville Sand, or equivalent
Sidewalk Surfaced with rich mortar of Portland Cement & Granite Chips.
Outside exposed surfaces of all walls and of parapet, and of bottom of arches to be smooth-finished. Checking or cracking of walls to be guarded against by using ¼" round steel rods imbedded next to said surfaces every 12".
Centering for arch span to remain in place at least 28 days after completion of arches.
Arch to be given proper camber so that when centering is removed the curve of arch will be true to design.
Concrete to be mixed by machine.
All sections as shown, or of equivalent strength. This strain sheet is the exclusive property of the Newton Engineering Company, until incorporated in contract with said company.
Strain Sheet and Specifications
— for —
Reinforced Concrete Arch Bridge
Lake Park, Milwaukee.
Ferry & Clas, — Architects.
Kahn System Reinforced Concrete
Newton Engineering Co., Milwaukee.
SHEET

GILES ENGINEERING ASSOCIATES, INC.

GEOTECHNICAL, ENVIRONMENTAL AND CONSTRUCTION MATERIALS CONSULTANTS
N8 W22350 JOHNSON ROAD, SUITE A1/WAUKESHA WI 53186 (262) 544-0118 FAX: (262) 549-5868

RAPID CHLORIDE TESTING

CLIENT: Lake Park Friends PROJECT: Lake Park Pedestrian Bridge

DATE: April 24, 2018 PROJECT NO.: 1M-1803020

Samples Taken: 3/22/2018
Test Date: 4/5/2018
Lab Number: R-180314
Comment: Samples taken by Con-Cor under observation of Giles personnel.

Core	Sample	Test Data	Chloride Content		
Number	**Depth**	**mV**	**% by Wt**	**ppm**	**kg/m³**
1	0-1 inch	56.5	0.082	820	0.82
1	1-2 inches	76.3	0.035	350	0.35
1	2-3 inches	69.8	0.047	470	0.47
2	0-1 inch	45	0.13	1300	1.3
2	1-2 inches	76.4	0.035	350	0.35
2	2-3 inches	88	0.022	220	0.22
4	0-1 inch	86.2	0.024	240	0.24
4	1-2 inches	88.4	0.022	220	0.22
4	2-3 inches	95.5	0.013	130	0.13
5	0-1 inch	77.6	0.0039	39	0.039
5	1-2 inches	86.4	0.023	230	0.23
5	2-3 inches	87	0.023	230	0.23
6	0-1 inch	75.6	0.037	370	0.37
6	1-2 inches	86.9	0.023	230	0.23
6	2-3 inches	91	0.018	180	0.18
8	0-1 inch	103.8	0.034	340	0.34
8	1-2 inches	89.8	0.053	530	0.53
8	2-3 inches	99.7	0.038	380	0.38
10	0-1 inch	100	0.038	380	0.38
10	1-2 inches	75	0.094	940	0.94
10	2-3 inches	93.1	0.05	500	0.5
11	0-1 inch	93.7	0.048	480	0.48
11	1-2 inches	99	0.039	390	0.39
11	2-3 inches	101.9	0.037	370	0.37

COMMENT: Samples numbered in accordance with Lake Park Friends sample designations.

Reviewing Engineeer: Steven P. Homar, P.E.

24-LAB-001 Petrographic Examination of Hardened Concrete
ASTM C856

Project No.	10-09372	Date:	April 27, 2018	Date reviewed:	May 8, 2018
Sample ID:	3	Performed by:	W. Reely	Reviewed by:	B. Lemcke

I. General Observations

1. Sample Dimensions: Our analysis was performed on two lapped sides of a 127 mm (5") x 95 mm (3-3/4") x 20 mm (3/4") thick profile section and a 76 mm (3") x 51 mm (2") thin section that were saw-cut and prepared from the original 95 mm (3-3/4") diameter x 127 mm (5") long composite core.

2. Surface Conditions:
   Top: Lightly rough, flat, mortar eroded surface
   Bottom: Rough, irregular, fractured surface

3. Reinforcement: None observed

4. General Physical Conditions: The sample was a composite core composed of an up to 101 mm (4") thick original base concrete overlain by an up to 26 mm (1") thick concrete topping layer. The two concretes were well-bonded.

   Numerous fine aggregate particles had been exposed at the top surface of the topping by mortar erosion, and subsequently smoothed by mortar erosion and traffic wear. A thin, faint layer of a green paint or stain covered approximately 30% of the top surface. A few sub-vertical micro-cracks propagated from the top surface to a maximum depth of 12 mm (1/2"); one micro-crack propagated from the top surface to a maximum depth of 114 mm (4-1/2"). The depth of carbonation was measured from the top surface of the topping, and was negligible. The top surface of the base concrete was slightly rough yet planar. The base concrete was partially carbonated throughout its depth. Several elongate bleed-water voids, up to 12 mm (1/2") in their longest dimension, were observed at various depths within the base concrete, frequently located along coarse aggregate margins and oriented sub-parallel to the top surface. Colorless to white alkali-silica gel was observed lining a few air voids at various depths within the core; the offending aggregates appeared to be chert and partially silicified limestone particles. A few microcracks, up to 57 mm (2-1/4") in length and with random orientation, were observed between 25 mm (1") and 95 mm (3-3/4") depth in the base concrete. A few black, partially-disintegrated bituminous or coal particles, up to 3 mm (1/8") in their longest dimension, were observed at various depths within the base concrete. Some dark brown to black-colored corrosion product was observed on the fractured inner surface of the base concrete and within proximal microcracks. The base concrete appeared air-entrained, but contained an air-void system which was not consistent with current American Concrete Institute (ACI) recommendations for freeze-thaw resistance.

   Sections II and III refer only to qualities of the base concrete, unless otherwise specified.

II. Aggregate

1. Coarse: 19 mm (3/4") nominal sized natural gravel composed of sparitic dolostone, limestone, and some chert/silicified carbonate. The particles were mostly round to sub-round. The coarse aggregate appeared well graded and exhibited good overall distribution.

2. Fine: Natural quartz and carbonate sand with some feldspar and lithic particles. The grains were mostly sub-rounded with many smaller sub-angular particles. The fine aggregate appeared fairly graded and exhibited good overall uniform distribution.

III. Cementitious Properties

1. Air Content:	4.1% total
2. Depth of carbonation:	The base concrete was partially carbonated throughout its depth.
3. Paste/aggregate bond:	Fair to good
4. Paste color:	Similar to but darker than yellowish gray (Munsell® 5Y 8/1)
5. Paste hardness:	Moderately soft (Mohs ≈ 2.5).
6. Microcracking:	A few sub-vertical micro-cracks propagated from the top surface to a maximum depth of 12 mm (1/2"); one micro-crack propagated from the top surface to a maximum depth of 114 mm (4-1/2"). A few micro-cracks, up to 57 mm (2-1/4") in length and with random orientation, were observed between 25 mm (1") and 95 mm (3-3/4") depth in the base concrete.
7. Secondary deposits:	Colorless to white alkali-silica gel was observed lining a few air voids at various depths within the core. Black to dark brown corrosion product was observed on the bottom surface of the core.
8. w/cm:	Estimated at between 0.55 and 0.65 with approximately 7 to 9% residual portland cement clinker particles
9. Cement hydration:	Alites: Fully, few low in coarse-ground clinker particles Belites: Fully, few low in coarse-ground clinker particles

24-LAB-001 Petrographic Examination of Hardened Concrete
ASTM C856

Project No.	10-09372	Date:	April 27, 2018	Date reviewed:	May 8, 2018
Sample ID:	7	Performed by:	W. Reely	Reviewed by:	B. Lemcke

I. General Observations

1. Sample Dimensions: Our analysis was performed on one lapped side of a 229 mm (9") x 95 mm (3-3/4") x 42 mm (1-5/8") thick profile section and a 76 mm (3") x 51 mm (2") thin section that were saw-cut and prepared from the original 95 mm (3-3/4") diameter x 229 mm (9") long core.

2. Surface Conditions:
   Top: Rough, irregular, scaled/fractured surface
   Bottom: Rough, irregular, fractured surface

3. Reinforcement: None observed

4. General Physical Conditions: Several coarse aggregate particles and numerous fine aggregate particles were exposed on the top surface. Many microcracks and macro-cracks, oriented sub-parallel to the top surface and frequently propagating through the full diameter of the core, were observed between the top surface and 45 mm (1-3/4"). Both the macro-cracks and micro-cracks frequently propagated through coarse and fine aggregate particles. The depth of carbonation was measured from the top and bottom surfaces and was negligible at each; however, carbonation was observed within 1 mm (1/32") up to 2 mm (1/16") of a few micro and macro-cracks. A few microcracks were observed oriented sub-parallel to the top surface and which propagated through the diameter of the core, were observed between 45 mm depth and the bottom surface. Numerous irregular consolidation voids, up to 23 mm (7/8") in their longest dimension, were observed between 40 mm (1-9/16") depth and 203 mm (8") depth. Colorless to white alkali-silica gel was observed lining to partially filling several of the consolidation voids. Colorless to white alkali-silica gel was also observed lining several of the micro-cracks proximal to the consolidation voids, and forming gel "plugs" at coarse aggregate margins intersected by the cracks. The reactive aggregates appeared to be chert and partially silicified carbonate aggregate particles. The concrete was not air-entrained and contained an air-void system not consistent with current American Concrete Institute (ACI) recommendations for freeze-thaw resistance.

II. Aggregate

1. Coarse: 19 mm (3/4") nominal sized natural and 'crushed' gravel composed of sparitic dolostone, limestone, and some (hydrous) chert/silicified carbonate. The particles were mostly round to sub-round. The coarse aggregate appeared well graded and exhibited good overall distribution.

2. Fine: Natural quartz and carbonate sand with some feldspar and lithic particles. The grains were mostly sub-rounded with many smaller sub-angular particles. The fine aggregate appeared fairly graded and exhibited good overall uniform distribution.

III. Cementitious Properties

1. Air Content: 4.1% total
2. Depth of carbonation: Negligible from the top and bottom surfaces
3. Paste/aggregate bond: Good
4. Paste color: Similar to but lighter than olive gray (Munsell® 5Y 6/1)
5. Paste hardness: Moderately soft (Mohs ≈ 2.5).
6. Microcracking: Many microcracks and macro-cracks, oriented sub-parallel to the top surface and frequently propagating through the full diameter of the core, were observed between the top surface and 45 mm (1-3/4"). A few microcracks, oriented sub-parallel to the top surface and which propagated through the diameter of the core, were observed between 45 mm depth and the bottom surface.
7. Secondary deposits: Colorless to white alkali-silica gel was observed lining to partially filling many of the consolidation voids. Colorless to white alkali-silica gel was also observed lining several of the micro-cracks proximal to the consolidation voids, and forming gel "plugs" at the top surfaces of coarse aggregate particles intersected by the cracks. White, acicular ettringite and sparitic calcite were observed filling several air voids and microcracks at various depths throughout the core.
8. w/cm: Estimated at between 0.50 and 0.60 with approximately 3 to 5% residual portland cement clinker particles
9. Cement hydration: Alites: Fully, few low in coarse-ground clinker particles
   Belites: Well to fully, few low in coarse-ground clinker particles

24-LAB-001 Petrographic Examination of Hardened Concrete
ASTM C856

Project No.	10-09372	Date:	April 27, 2018	Date reviewed:	May 8, 2018
Sample ID:	9	Performed by:	W. Reely	Reviewed by:	B. Lemcke

I. General Observations

1. Sample Dimensions: Our analysis was performed on one lapped side of a 241 mm (9-1/2") x 95 mm (3-3/4") x 45 mm (1-3/4") thick profile section and a 76 mm (3") x 51 mm (2") thin section that were saw-cut and prepared from the original 95 mm (3-3/4") diameter x 241 mm (9-1/2") long core.

2. Surface Conditions:
   Top: Rough, flat, mortar-eroded surface
   Bottom: Rough, irregular, fractured surface

3. Reinforcement: None observed

4. General Physical Conditions: The top surface had undergone mortar erosion to an unknown depth, exposing numerous fine aggregate particles. An "X" had been marked on the top surface with black marker or paint. The depth of carbonation was measured from the top surface, and ranged from 25 mm (1") to 36 mm (1-7/16"). Many small, irregular zones of poor consolidation were observed throughout the core. Fine bleed water channels were also observed along many aggregate margins. The concrete was not air-entrained and contained an air-void system not consistent with current American Concrete Institute (ACI) recommendations for freeze-thaw resistance.

II. Aggregate

1. Coarse: 19 mm (3/4") nominal sized natural and 'crushed' gravel composed of sparitic dolostone, limestone, and some chert/silicified carbonate. The particles were mostly round to sub-round. The coarse aggregate appeared well graded and exhibited good overall distribution.

2. Fine: Natural quartz and carbonate sand with some feldspar and lithic particles. The grains were mostly sub-rounded with many smaller sub-angular particles. The fine aggregate appeared fairly graded and exhibited good overall uniform distribution.

III. Cementitious Properties

1.	Air Content:	2.5% total
2.	Depth of carbonation:	The depth of carbonation was measured from the top surface, and ranged from 25 mm (1") to 36 mm (1-7/16").
3.	Paste/aggregate bond:	Fair to good
4.	Paste color:	Yellowish gray (Munsell® 5Y 8/1)
5.	Paste hardness:	Moderately soft (Mohs ≈ 2.5).
6.	Microcracking:	None observed
7.	Secondary deposits:	None observed
8.	w/cm:	Estimated at between 0.55 and 0.65 with approximately 3 to 5% residual portland cement clinker particles
9.	Cement hydration:	Alites: Fully Belites: Fully, few low in coarse clinker particles

24-LAB-001 Petrographic Examination of Hardened Concrete
ASTM C856

Project No.	10-09372	Date:	May 7, 2018	Date reviewed:	May 8, 2018
Sample ID:	12	Performed by:	W. Reely	Reviewed by:	B. Lemcke

I. General Observations

1. Sample Dimensions: Our analysis was performed on one lapped side each of two 235 mm (9-1/4") x 95 mm (3-3/4") x 45 mm (3-3/4") thick profile section and a 76 mm (3") x 51 mm (2") thin section that were saw-cut and prepared from the original 70 mm (2-3/4") diameter x 235 mm (9-1/4") long core.

2. Surface Conditions:
   Top: Rough, flat, mortar-eroded surface
   Bottom: Rough, irregular, fractured surface

3. Reinforcement: None observed

4. General Physical Conditions: The top surface had undergone mortar erosion to an unknown depth, exposing numerous fine aggregate particles. An "X" had been marked on the top surface with blue marker or paint. The depth of carbonation was measured from the top surface, and ranged from 3 mm (1/8") to 25 mm (1"). Many small, irregular zones of poor consolidation were observed throughout the core. The concrete was not air-entrained and contained an air-void system not consistent with current American Concrete Institute (ACI) recommendations for freeze-thaw resistance.

II. Aggregate

1. Coarse: 12 mm (1/2") nominal sized natural and 'crushed' gravel composed of sparitic dolostone, limestone, and some chert/silicified carbonate. The particles were mostly round to sub-round. The coarse aggregate appeared well graded and exhibited good overall distribution.

2. Fine: Natural quartz and carbonate sand with some feldspar and lithic particles. The grains were mostly sub-rounded with many smaller sub-angular particles. The fine aggregate appeared fairly graded and exhibited good overall uniform distribution.

III. Cementitious Properties

1. Air Content: 3.7% total
2. Depth of carbonation: The depth of carbonation was measured from the top surface, and ranged from 3 mm (1/8") to 25 mm (1").
3. Paste/aggregate bond: Fair to good
4. Paste color: Similar to, but darker than, yellowish gray (Munsell® 5Y 8/1)
5. Paste hardness: Moderately soft (Mohs ≈ 2.5).
6. Microcracking: None observed
7. Secondary deposits: None observed
8. w/cm: Estimated at between 0.55 and 0.65 with approximately 3 to 5% residual portland cement clinker particles
9. Cement hydration: Alites: Fully
   Belites: Fully, few low in coarse clinker particles

AIR VOID ANALYSIS

PROJECT:

LAKE PARK FOOTBRIDGE
GILES PROJECT NO.: 1M-1803020
MILWAUKEE, WI

APS PROJECT NO: 10-09372

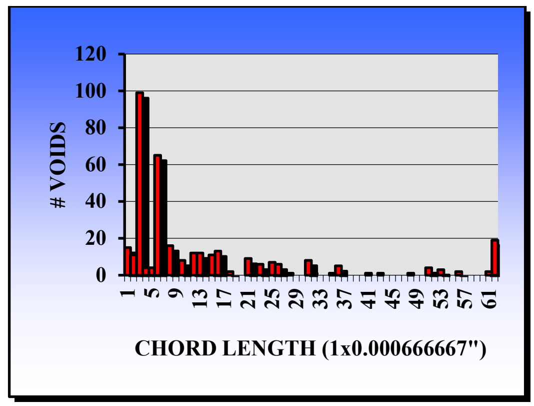
REPORTED TO:

GILES ENGINEERING ASSOCIATES
N8 W22350 JOHNSON DR
WAUKESHA, WI 53186

ATTN: STEVE HOMAR
DATE: APRIL 26, 2018

Sample ID: #3

Conformance: The concrete contains an air void system which is not consistent with current American Concrete Institute (ACI) recommendations for freeze-thaw resistance.



Sample Data

Description:	Hardened Concrete Core
Dimensions:	95 mm (3-3/4") diameter by 127 mm (5") long

Test Data: By ASTM C457, Procedure A

Air Void Content %	4.1
Entrained, % < 0.040"(1mm)	2.7
Entrapped, %> 0.040"(1mm)	1.4
Air Voids/inch	3.8
Specific Surface, in^2/in^3	370
Spacing Factor, inches	0.014
Paste Content, % estimated	25
Magnification	75x
Traverse Length, inches	90
Test Date	4/25/2018
Test Performed By	W. Reely

Magnification: 15x
Description: Hardened air void system.

550 Cleveland Avenue North | Saint Paul, MN 55114
Phone (651) 659-9001 | (800) 972-6364 | Fax (651) 659-1379 | www.amengtest.com | AA/EEO

American Petrographic Services, Inc.

AIR VOID ANALYSIS

PROJECT:

LAKE PARK FOOTBRIDGE
GILES PROJECT NO.: 1M-1803020
MILWAUKEE, WI

APS PROJECT NO: 10-09372

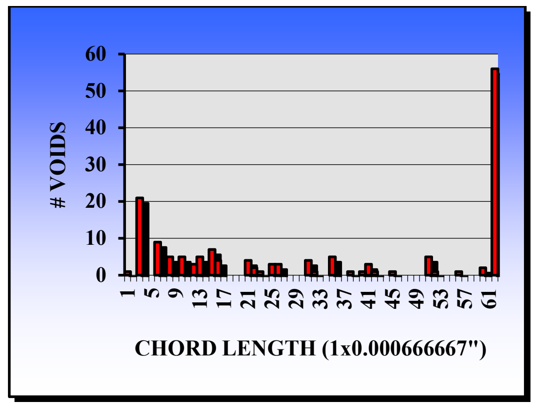
REPORTED TO:

GILES ENGINEERING ASSOCIATES
N8 W22350 JOHNSON DR
WAUKESHA, WI 53186

ATTN: STEVE HOMAR
DATE: APRIL 26, 2018

Sample ID: #7

Conformance: The concrete contains an air void system which is not consistent with current American Concrete Institute (ACI) recommendations for freeze-thaw resistance.

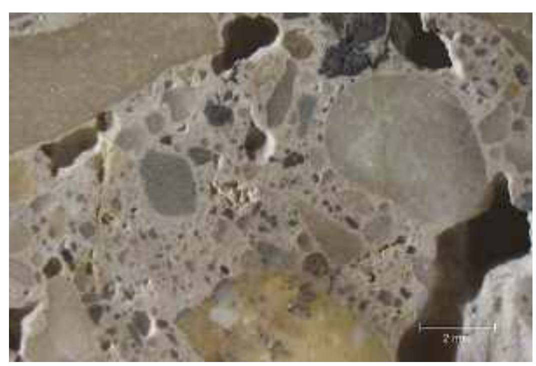


Sample Data

Description:	Hardened Concrete Core
Dimensions:	95 mm (3-3/4") diameter by 229 mm (9") long

Test Data: By ASTM C457, Procedure A

Air Void Content %	8.2
Entrained, % < 0.040"(1mm)	1.4
Entrapped, %> 0.040"(1mm)	6.8
Air Voids/inch	1.7
Specific Surface, in^2/in^3	80
Spacing Factor, inches	0.032
Paste Content, % estimated	22
Magnification	75x
Traverse Length, inches	90
Test Date	4/25/2018
Test Performed By	W. Reely



Magnification: 15x
Description: Hardened air void system.

American Petrographic Services, Inc.

AIR VOID ANALYSIS

PROJECT:

LAKE PARK FOOTBRIDGE
GILES PROJECT NO.: 1M-1803020
MILWAUKEE, WI

APS PROJECT NO: 10-09372

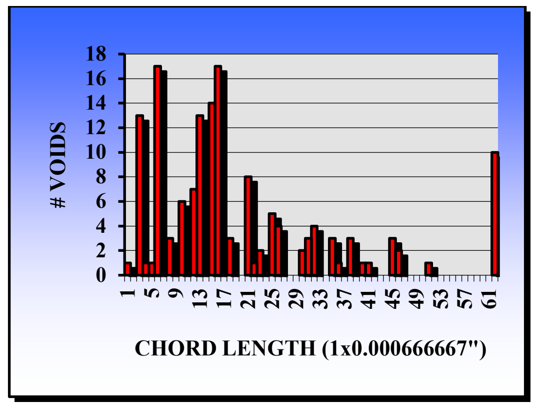
REPORTED TO:

GILES ENGINEERING ASSOCIATES
N8 W22350 JOHNSON DR
WAUKESHA, WI 53186

ATTN: STEVE HOMAR
DATE: APRIL 26, 2018

Sample ID: #9

Conformance: The concrete contains an air void system which is not consistent with current American Concrete Institute (ACI) recommendations for freeze-thaw resistance.



Sample Data

Description:	Hardened Concrete Core
Dimensions:	95 mm (3-3/4") diameter by 241 mm (9-1/2") long

Test Data: By ASTM C457, Procedure A

Air Void Content %	2.5
Entrained, % < 0.040"(1mm)	1.8
Entrapped, %> 0.040"(1mm)	0.7
Air Voids/inch	1.7
Specific Surface, in^2/in^3	270
Spacing Factor, inches	0.024
Paste Content, % estimated	26
Magnification	75x
Traverse Length, inches	90
Test Date	4/26/2018
Test Performed By	W. Reely

Magnification: 15x
Description: Hardened air void system.

550 Cleveland Avenue North | Saint Paul, MN 55114
Phone (651) 659-9001 | (800) 972-6364 | Fax (651) 659-1379 | www.amengtest.com | AA/EEO

AIR VOID ANALYSIS

PROJECT:

LAKE PARK FOOTBRIDGE
GILES PROJECT NO.: 1M-1803020
MILWAUKEE, WI

APS PROJECT NO: 10-09372

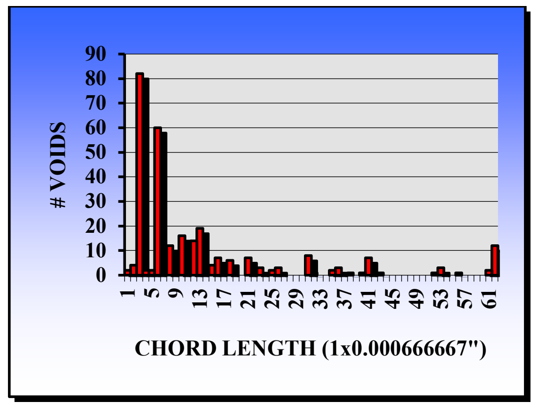
REPORTED TO:

GILES ENGINEERING ASSOCIATES
N8 W22350 JOHNSON DR
WAUKESHA, WI 53186

ATTN: STEVE HOMAR
DATE: APRIL 26, 2018

Sample ID: #12

Conformance: The concrete contains an air void system which is not consistent with current American Concrete Institute (ACI) recommendations for freeze-thaw resistance.



Sample Data

Description: Hardened Concrete Core
Dimensions: 95 mm (3-3/4") diameter by 235 mm (9-1/4") long

Test Data: By ASTM C457, Procedure A

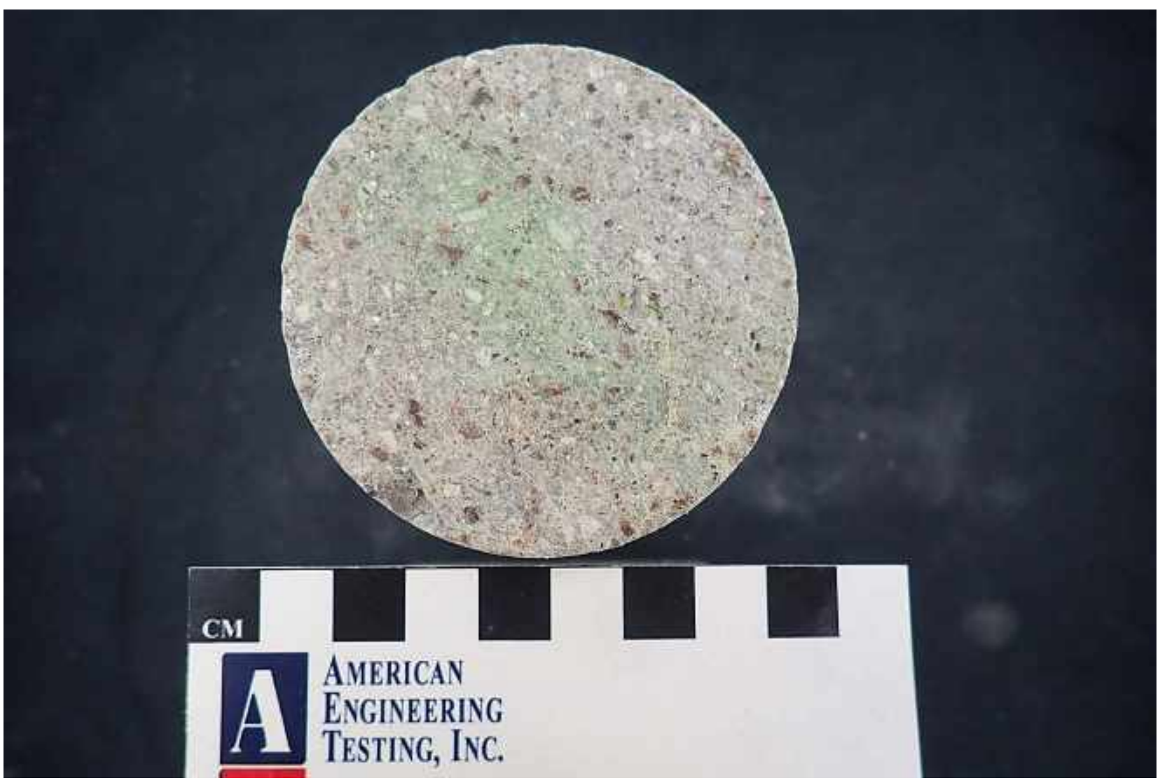
Air Void Content %	3.7
Entrained, % < 0.040"(1mm)	2.4
Entrapped, %> 0.040"(1mm)	1.3
Air Voids/inch	3.2
Specific Surface, in^2/in^3	350
Spacing Factor, inches	0.016
Paste Content, % estimated	28
Magnification	75x
Traverse Length, inches	90
Test Date	4/26/2018
Test Performed By	W. Reely

Magnification: 15x
Description: Hardened air void system.

PHOTO: 1

SAMPLE ID: 3 **DESCRIPTION:** The overall profile of the core as received, with the top surface oriented to the left.

PHOTO: 2



SAMPLE ID: 3 **DESCRIPTION:** The top surface of the core as received.

APS PROJECT NO: 10-09372 **DATE:** MAY 7, 2018

PROJECT: LAKE PARK FOOTBRIDGE
GILES PROJECT NO.: 1M-1803020, MILWAUKEE, WI

PHOTO: 3

SAMPLE ID: 7 **DESCRIPTION:** The overall profile of the core as received, with the outer top oriented to the left.

PHOTO: 4

SAMPLE ID: 7 **DESCRIPTION:** The top surface of the core, as received.

APS PROJECT NO: 10-09372 **DATE:** MAY 7, 2018
PROJECT: LAKE PARK FOOTBRIDGE
GILES PROJECT NO.: 1M-1803020, MILWAUKEE, WI

PHOTO: 5

SAMPLE ID: 9 **DESCRIPTION:** The overall profile of the core as received, with the top surface oriented to the left.

PHOTO: 6

SAMPLE ID: 9 **DESCRIPTION:** The top surface of the core, as received.

APS PROJECT NO: 10-09372 **DATE:** MAY 7, 2018
PROJECT: LAKE PARK FOOTBRIDGE
GILES PROJECT NO.: 1M-1803020, MILWAUKEE, WI

PHOTO: 7

SAMPLE ID: 12 **DESCRIPTION:** The overall profile of the core as received, with the top surface oriented to the left.

PHOTO: 8

SAMPLE ID: 12 **DESCRIPTION:** The top surface of the core, as received.

AET PROJECT NO: 10-09372 **DATE:** MAY 8, 2018

PROJECT: LAKE PARK FOOTBRIDGE
GILES PROJECT NO.: 1M-1803020, MILWAUKEE, WI

PHOTO: 9

SAMPLE ID: 3

DESCRIPTION: Freshly saw-cut and lapped profile of the core with the top surface oriented up, after application of phenolphthalein pH indicator. The pale pink stain of the base concrete indicates partial carbonation throughout its depth. Micro-cracking was mapped with red ink

AET PROJECT NO: 10-09372 **DATE:** MAY 8, 2018

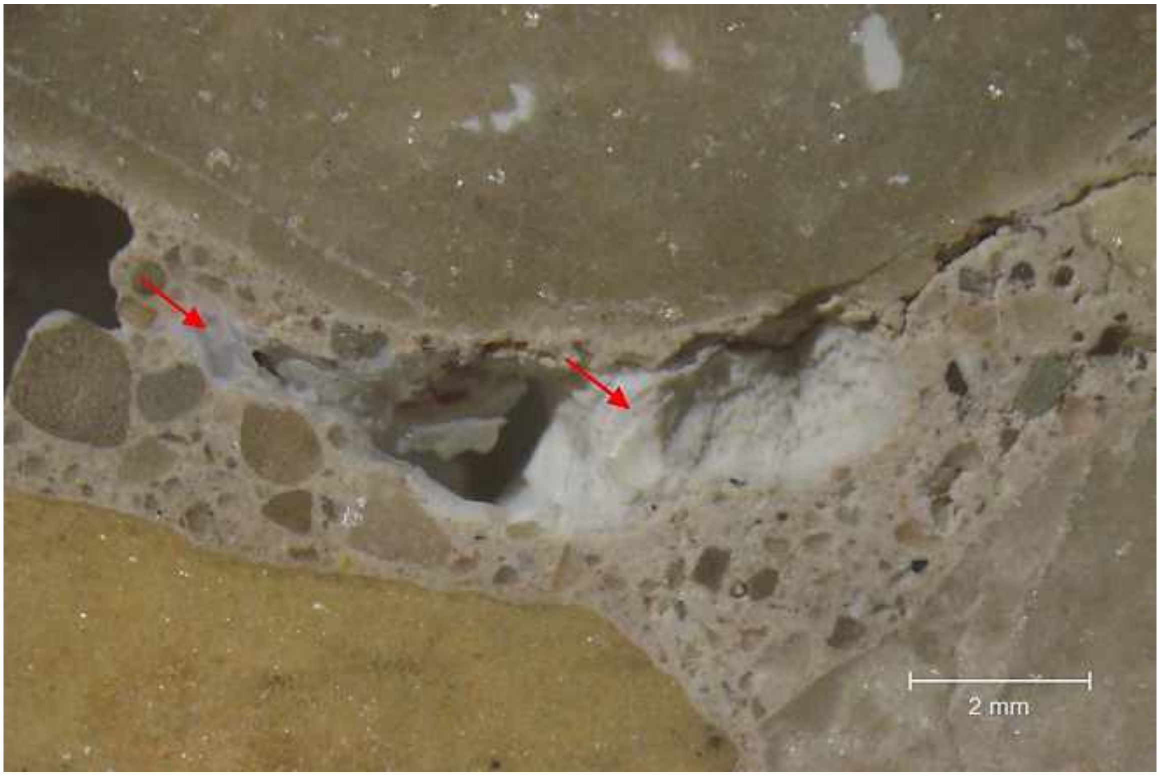
PROJECT: LAKE PARK FOOTBRIDGE
GILES PROJECT NO.: 1M-1803020, MILWAUKEE, WI

PHOTO: 10

SAMPLE ID: 7

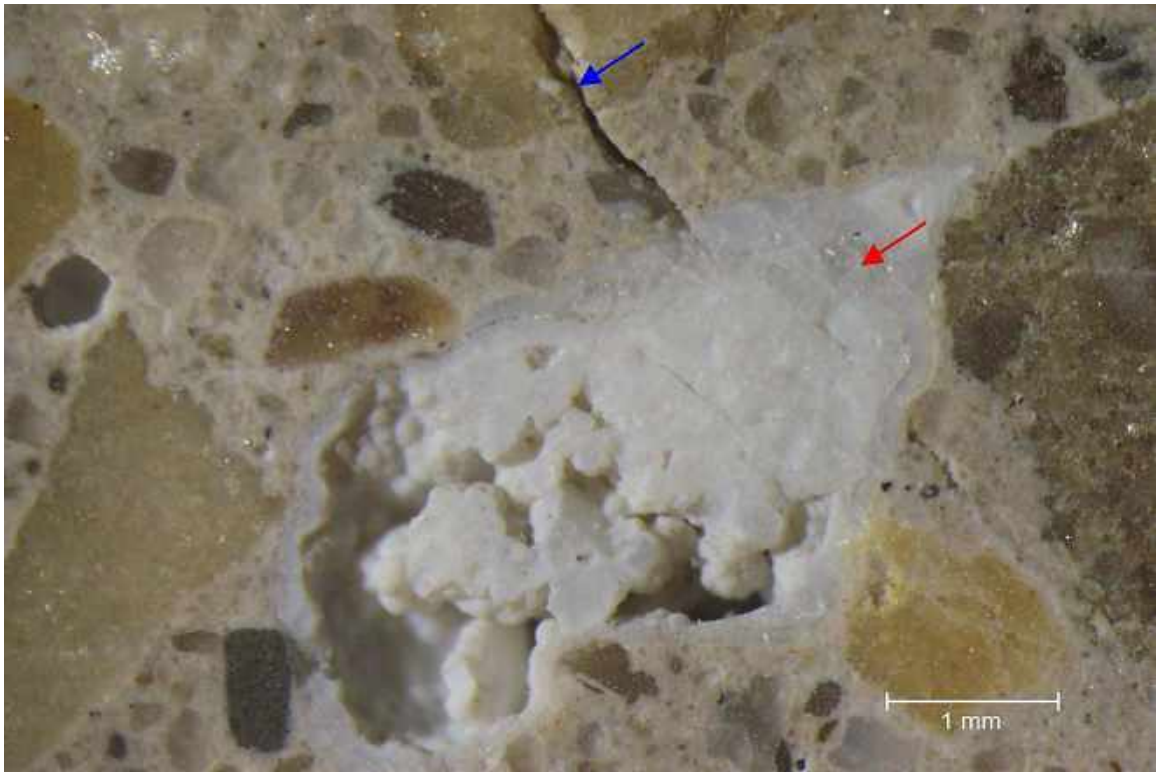
DESCRIPTION: Freshly saw-cut and lapped profile of the core with the top surface oriented up. Phenolphthalein pH indicator (pink stain) was applied to the top and bottom right corners to check for carbonation. Macro-cracking and micro-cracking was mapped with red ink. Note the irregular consolidation voids.

PHOTO: 11



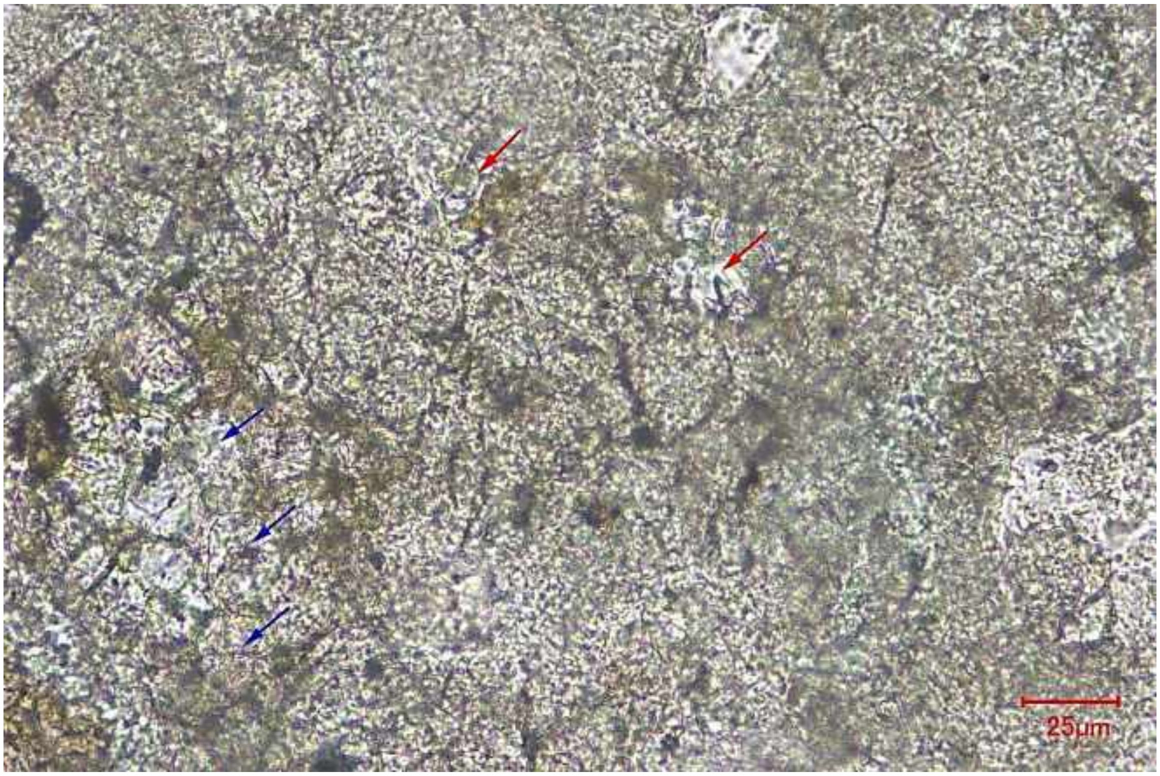
SAMPLE ID: 7 **DESCRIPTION:** White alkali-silica gel (red arrows) partially filling an irregular consolidation void.
MAG: 15x

PHOTO: 12



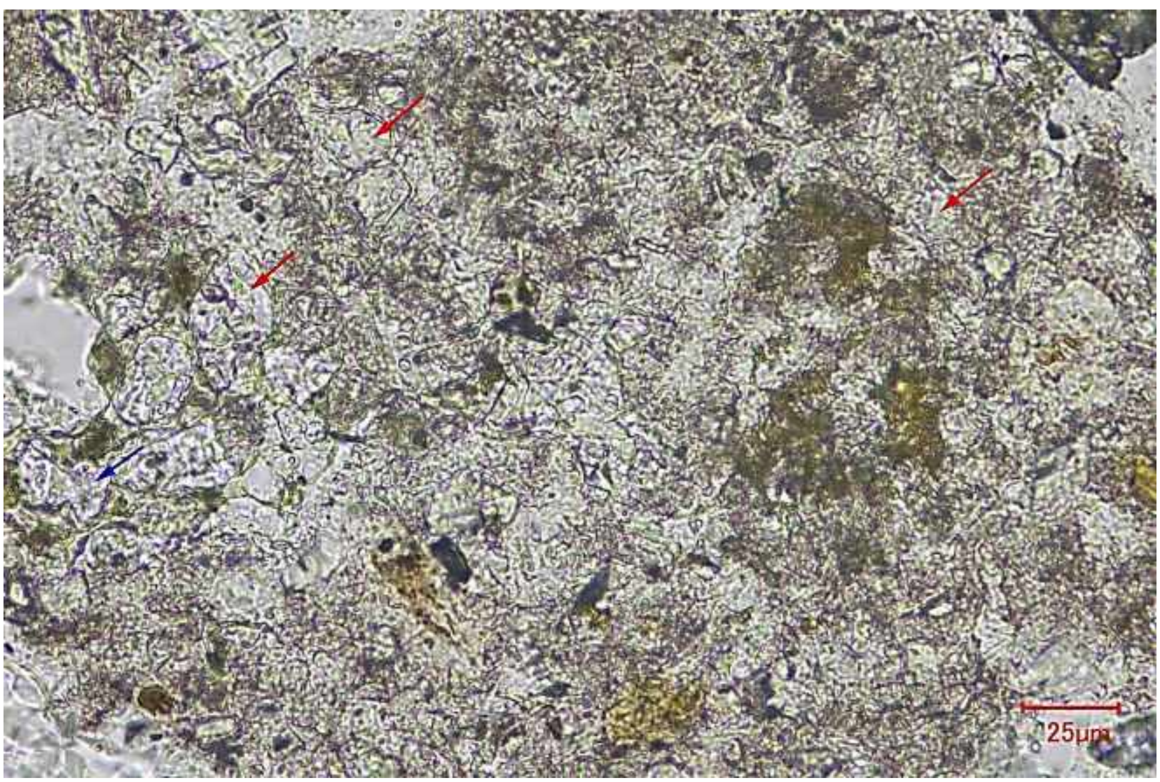
SAMPLE ID: 7 **DESCRIPTION:** White alkali-silica gel partially filling an entrapped-sized air void (red arrow), intersected by a micro-crack which propagated through a proximal limestone aggregate particle with a gel "plug" blue arrow) forming in the crack at the outer surface of the particle.
MAG: 30x

PHOTO: 13



SAMPLE ID: 3 **DESCRIPTION:** Fully-hydrated alite relicts (red arrows) and fully-hydrated belite relicts (blue arrows)
MAG: 400x portland cement clinker particles, in a thin section of concrete, viewed under plane-polarized light.

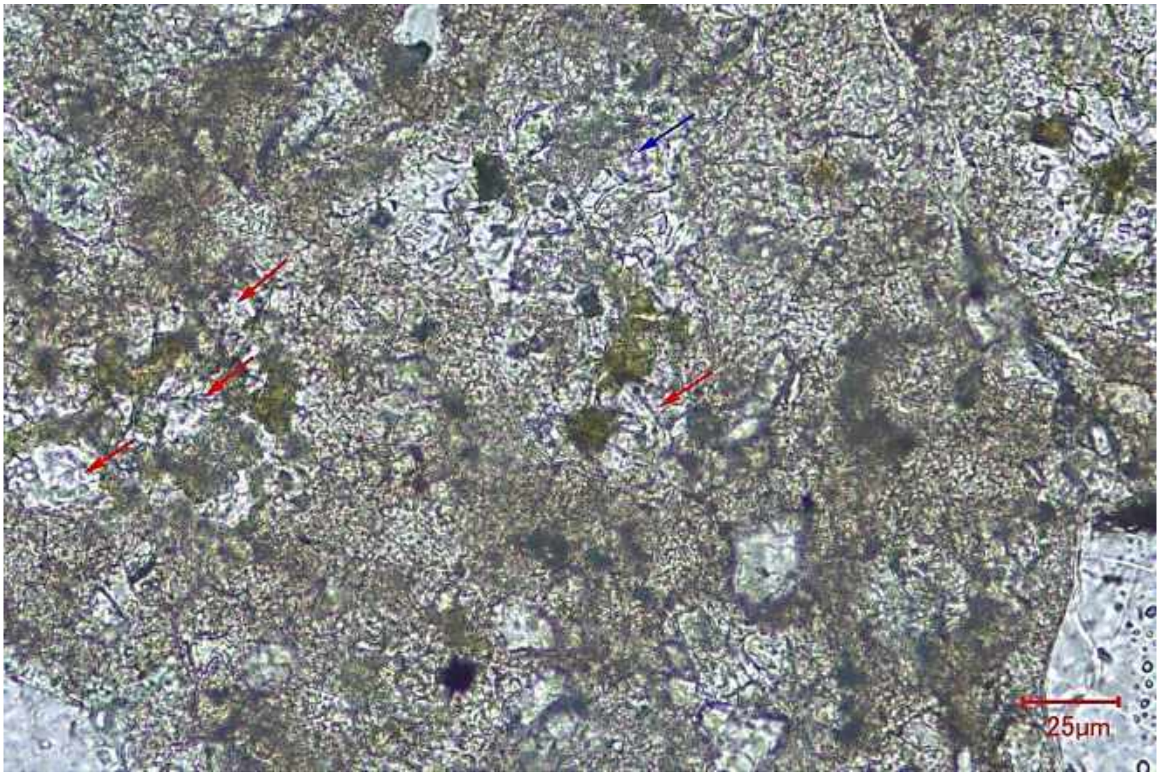
PHOTO: 14



SAMPLE ID: 7 **DESCRIPTION:** Fully-hydrated alite relicts (red arrows) and fully-hydrated belite relicts (blue arrow)
MAG: 400x portland cement clinker particles, in a thin section of concrete, viewed under plane-polarized light.

APS PROJECT NO:	10-09372	**DATE:**	MAY 7, 2018
PROJECT:	LAKE PARK FOOTBRIDGE GILES PROJECT NO.: 1M-1803020, MILWAUKEE, WI		

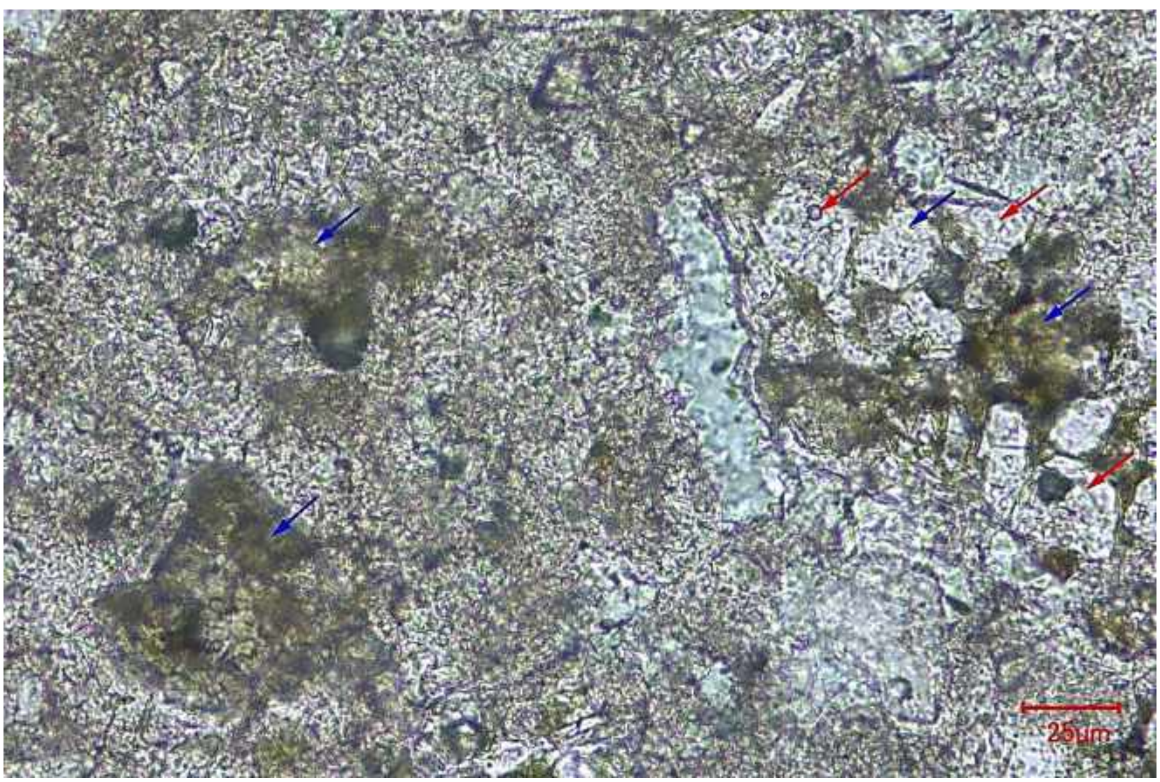
PHOTO: 15



SAMPLE ID: 9
MAG: 400x

DESCRIPTION: Fully-hydrated alite relicts (red arrows) and fully-hydrated belite relict (blue arrows) portland cement clinker particles, in a thin section of concrete, viewed under plane-polarized light.

PHOTO: 16



SAMPLE ID: 12
MAG: 400x

DESCRIPTION: Fully-hydrated alite relicts (red arrows) and moderately to well-hydrated residual belite (blue arrow) portland cement clinker particles, in a thin section of concrete, viewed under plane-polarized light.