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Diagnosis and prognosis of ASR in an airfield pavement

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Abstract

A concrete runway at an international airport began to exhibit multiple modes of distress several years after construction, prompting a thorough root cause failure investigation. Surface distresses observed included popouts, paste pop-offs, spalls, and cracking. The original coarse aggregate source was exhausted during construction, necessitating the use of a second coarse aggregate and three primary concrete mixture designs as construction progressed. Construction records indicated that both coarse aggregates were considered potentially reactive per the accelerated mortar bar test, and one of the mixture designs did not use any mitigation in the form of supplementary cementing materials (SCMs).

Petrographic examination of core samples identified the presence of ASR in all three concrete mixtures, although it was not linked to the observed surface distress. The cause of the surface distress was ultimately attributed to other factors; however, the presence of cracking and regions of high porosity at the pavement surface could still facilitate access to moisture to any ASR gel that formed and reactive silica was still present in the aggregate. Based on the petrographic findings, a residual expansion testing program was initiated to address concerns about future expansion potential from ASR.

The residual expansion testing program included eighteen 50-mm diameter cores immersed in 1N NaOH at 38°C and nine 150-mm diameter cores stored over water at 38°C. The cores have been monitored for up to fifteen months, and the data thus far indicate minimal risk of deleterious future expansion from ASR in the pavement. This information will be valuable in designing an effective rehabilitation and repair strategy to extend the service life of the pavement.

Keywords*: ASR; failure analysis; pavements; petrography; residual expansion*

1. INTRODUCTION

A concrete runway constructed at a major international airport in the United States began to exhibit multiple modes of surface distress several years after construction. The types of distress observed included popouts, paste pop-offs, spalls, and cracking, and these increased in severity with time. RJ Lee Group was engaged with by owner in a root cause failure investigation starting in 2016 when the pavement was approximately 10 years old.

Surface distress of concrete airfield pavements is potentially a much more serious concern than for highways and local streets, because of the risk of foreign object debris (FOD). Concrete that is spalled from joints, cracks, and popouts in an airfield pavement is considered FOD, and could cause damage if ingested into aircraft engines. FOD can also be blown through the air by high-velocity jet blast, causing harm to nearby vehicles and personnel. The US Federal Aviation Administration (FAA), notes that pavement materials can be the most common source of FOD at airports [1].

Smith and Van Dam [2] provide several case studies of ASR-affected airfield pavements in the United States, including Phoenix Sky Harbor International Airport, Hartsfield Jackson Atlanta International Airport, Bangor (Maine) International Airport, and Colorado Springs Municipal Airport. The affected pavements at these airports included runways, taxiways, and aprons, and were constructed between the 1960s and 1990s using specifications with less-rigorous aggregate reactivity testing and without mitigating measures such as limits on cement alkali loading or the use of supplementary cementing materials (SCMs). Repair and remediation efforts including partial-depth and full-depth repairs, and overlays have been able to extend the service life of ASR-affected pavements, while crack sealing and surface treatments were found to be largely ineffective. Even low-severity cases that ultimately did not limit service life (35+ years before reconstruction) still produced FOD issues, while more severe cases were noted to significantly reduce service life, with reconstruction efforts at Colorado Springs beginning roughly 10 years after the pavements were constructed.

A review of the construction records found that the coarse aggregates were potentially reactive and that one of the three primary mixture designs used on the project did not contain any SCMs to mitigate against the risk of ASR. This raised the possibility that ASR could reduce the expected service life, and a laboratory investigation was initiated to first conduct petrographic examination of core samples extracted from the pavement, followed by residual expansion testing of additional core samples.

This paper focuses on the authors' application of petrography and residual expansion testing for diagnosis and prognosis purposes. The residual expansion testing program was influenced by the US Federal Highway Administration (FHWA) recommendations issued in 2010 [3] and the RILEM AAR-6.1 [4]. The petrographic investigation was used to evaluate whether ASR was present, whether it could be identified as a cause of the observed surface distress, and to provide a qualitative description of the severity of any reaction that had occurred to date. The residual expansion testing program would then be used to provide the owner with an estimate of future expansion potential of each concrete mixture, information that is critical to designing an effective rehabilitation and/or repair strategy for the runway.

2. MATERIALS AND METHODS

2.1 Materials and Mixture Designs

The initial investigation included a review of construction documents and submittals. The original coarse aggregate source was unable to supply the project for its full duration. As a result, coarse aggregate from a second source was procured for the remainder of the project. Because the coarse aggregate consistent of both a traditional coarse fraction (25 x 4.75 mm) and an intermediate (9.5 x 2.36 mm) size common to paving mixtures, this resulted in three primary mixture designs used for the construction of the runway. Table 2.1 provides details on the aggregates and cementitious materials and Table 2.2 summarizes the proportions for the three mixture designs. Coarse Aggregate 1 and Intermediate Aggregate 1 were from the same source, and Coarse Aggregate 2 and Intermediate Aggregate 2 were from the same source.

The US Federal Aviation Administration (FAA) specifications in force at the time of the project [5] relied upon the accelerated mortar bar test, ASTM C1260 [6], to determine aggregate reactivity, and a modified version of ASTM C1567 [7] with materials combined in job mixture proportions to evaluate mitigation measures. All coarse and intermediate aggregates had 14-day ASTM C1260 expansions greater than 0.10%, however petrographic descriptions of Coarse Aggregate 2 and Intermediate Aggregate 2 provided in construction documents indicated that material from that source was likely innocuous. The fine aggregate was from a third quarry and had a 14-day expansion below 0.10%.

Table 2.1: Aggregates and cementitious materials

Material	Description	ASTM C1260 (14-day)
Coarse Aggregate 1	Mixed natural gravel (predominantly granitic and metasedimentary)	0.13%
Coarse Aggregate 2	Crushed porphyritic volcanic rock	0.16%
Intermediate Aggregate 1	Crushed stone; same mineralogy as Coarse Aggregate 1	0.14%
Intermediate Aggregate 2	Crushed stone; same mineralogy as Coarse Aggregate 2	0.19%
Fine Aggregate	Manufactured siliceous sand	0.05%
Cement	ASTM C150 Type II/V, Low-alkali, 0.59% Na_2O_{eq}	N/A
Fly Ash	ASTM C618 Class F, 3.8% CaO	N/A

The ASTM C1260 results should have necessitated the use of mitigation measures for all concrete mixtures, one of the primary mixtures (Mix 1) used on the project contained no mitigation measures. Mix 2 and Mix 3 each contained 20% fly ash (by mass of total cementitious materials). Documentation from the producer of Coarse Aggregate 1 and Intermediate Aggregate 1 did indicate that 14-day expansions in ASTM C1567 would be reduced to 0.02% with 15% fly ash, although it did not indicate what fly ash was used to achieve these results. The authors did not find records of modified ASTM C1567 tests for the full Mix 1 job mixture aggregate combination. Mix 2 and Mix 3 both had 14-day modified ASTM C1567 expansions of 0.02%, indicating that these mixtures should acceptably mitigate ASR risk per specification requirements. Concrete produced with Mix 1 was therefore considered to contain the highest risk of developing deleterious ASR in the field.

Table 2.2: SSD Mixture proportions (all values in kg/m^3 unless noted) and ASTM C1567 results

Material	Mix 1	Mix 2	Mix 3
Coarse Aggregate 1	991	0	0
Coarse Aggregate 2	0	832	825
Intermediate Aggregate 1	186	322	0
Intermediate Aggregate 2	0	0	338
Fine Aggregate	695	685	679
Cement	356	297	297
Class F Fly Ash	0	74	74
Water	148	160	166
w/cm	0.42	0.43	0.45
14-day expansion, Modified ASTM C1567	N/A	0.02%	0.02%

2.2 Field Investigation

It is beyond the scope of this paper to describe all aspects of the investigation, which included numerous site visits by the authors and others in 2016 and 2017 to document the types and extent of distress on the runway, and to obtain core samples for laboratory testing. This section provides details of a targeted portion of the larger investigation, focused on determination of the potential effect of ASR on the remaining service life of the pavement.

The site visits were made at night due to limited opportunities for runway closures. Figure 2.1 and Figure 2.2 show examples of the different modes surface distresses, photographed during the initial documentation of the pavement condition. Construction records were also examined to map the locations of each mixture design along the runway. Subsequent site visits were primarily conducted for sample extraction purposes.

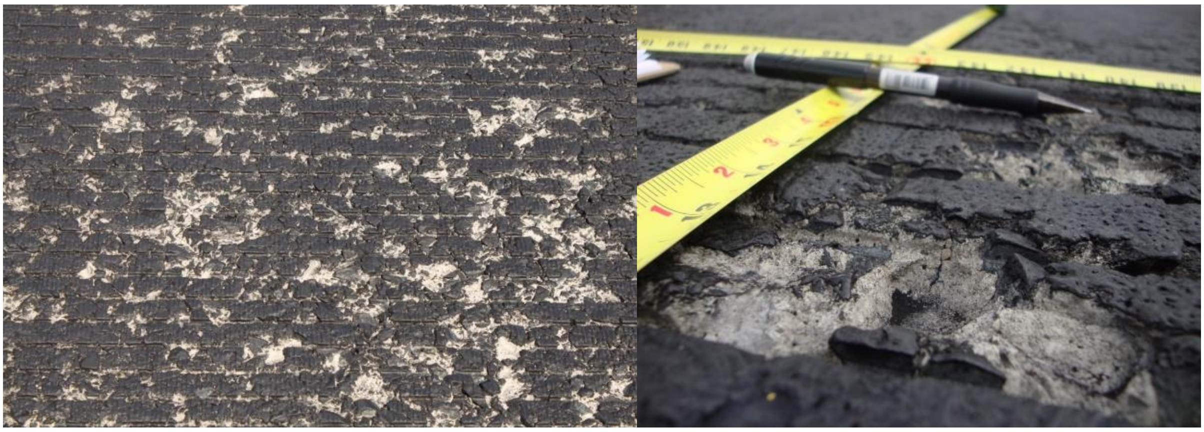

Figure 2.1: Examples surface distress (popouts and loss of paste).

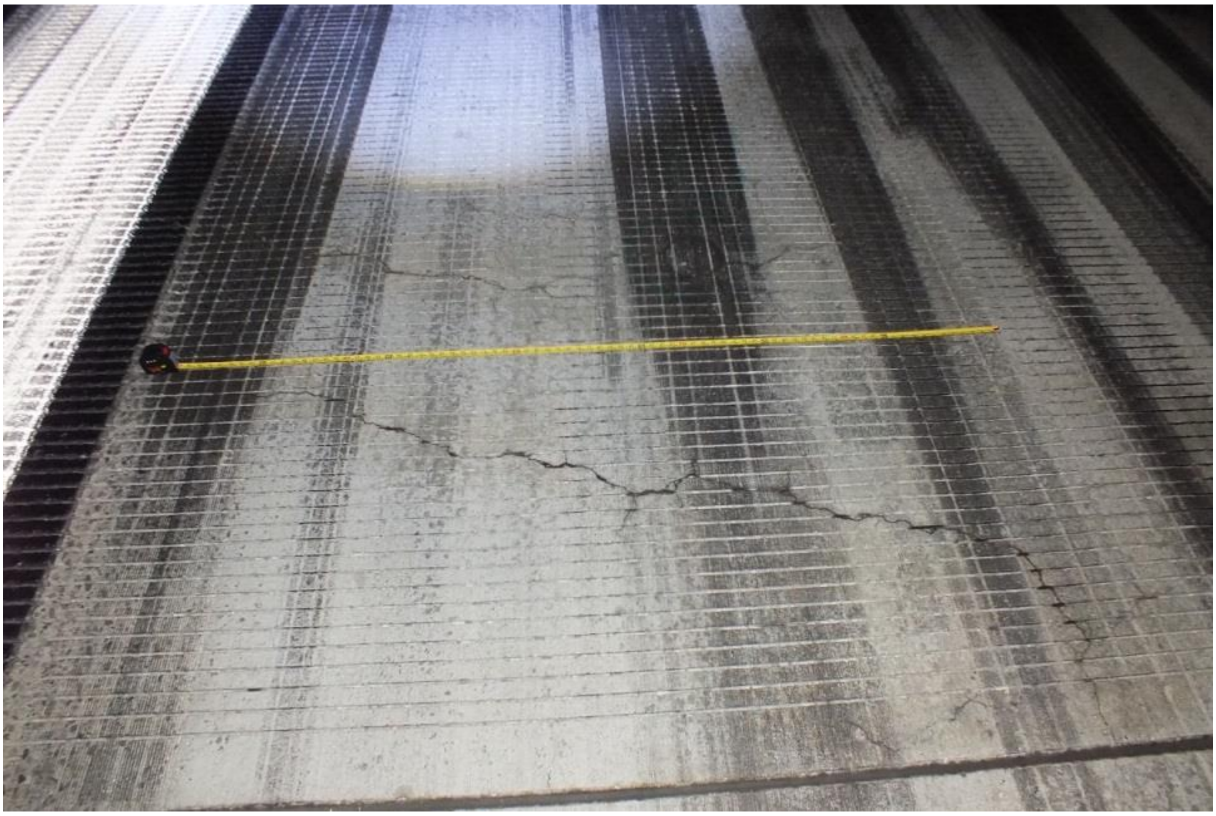

Figure 2.2: Example of surface cracking.

2.3 Laboratory Investigation

2.3.1 Petrography

Petrographic examinations following ASTM C856, *Standard Practice for Petrographic Examination of Hardened Concrete* [8] and ASTM C1723 *Standard Guide for Examination of Hardened Concrete Using Scanning Electron Microscopy (SEM)* [9] were conducted on concrete representing all three mixture designs over the course of the root cause investigation. Cores were visually examined in the as-received condition and using a binocular microscope at magnifications up to 50x. The core was then cut at a depth of 100 mm from the top of the core using a table-top masonry saw with a 355-mm diamond blade. Based on the visual observations, the top and bottom portions of the core were then cut at select locations to create two 12.7-mm-thick cross-sectioned slabs which were polished using successively finer silica carbide abrasives to 1000 grit size creating a finish satisfactory for microscopical examination showing excellent reflection of a distant light source when viewed at a low incident angel, and no noticeable relief between paste and aggregate surfaces. These polished cross sections were examined using stereo-optical microscope at magnifications up to 50x.

Three to four thin sections were prepared from select locations on the opposing slab to examine the concrete microstructure at various depths as indicated in the photographs in Figure 2.3. The thin sections were prepared from a 44 mm x 29 mm x 10 mm block that was vacuum-impregnated with fluorescent-dyed epoxy and mounted onto a glass slide. The thin sections were created to a thickness of 20 to 25 µm using a Pelcon automatic thin section machine. The final polish was completed manually with a 0.25 µm diamond oil to result in a highly polished surface for analysis. Thin sections were examined using polarized light microscopy (PLM) in plane-polarized and cross-polarized light modes at magnifications from 10x to 200x, and using reflectance fluorescent light at magnifications of 10x to 40x. The thin sections were finally examined using scanning electron microscopy with energy dispersive X-ray spectroscopy (SEM/EDS) on an ASPEX Personal SEM at magnifications of 20x to 2000x.

2.3.2 Residual Expansion

Specimen preparation and testing for residual expansion primarily followed the procedures given in Appendix F of Fournier et al. [3], with slight variations. Cores were extracted from three locations of interest for each of the three mixture designs, with one 150-mm diameter core and two 50-mm diameter cores taken adjacent to each other from each location. All cores were drilled to the full depth of the pavement (approximately 480 mm), wiped of excess water, labelled, and wrapped in plastic bags for shipment to RJ Lee Group's laboratory. The top 50 mm of each core was sawn off to exclude near-surface effects and was retained for additional petrographic examination at RJ Lee Group. The 150-mm diameter cores were cut to a length of 300 mm for testing and the 50-mm diameter cores were cut to a length of 200 mm. The cut cores were then shipped to The University of Texas at Austin for instrumentation and residual expansion testing.

Two test procedures were conducted in parallel to evaluate the residual expansion potential of all three mixture designs. The first test procedure involved storing the 150-mm diameter cores over water (>95% relative humidity) at 38 °C. The second test procedure involved storing the 50-mm diameter cores in 1N NaOH solution at 38 °C.

Figure 2.3: Photograph of cross-sectioned core showing locations of thin section preparation outlined in black.

The DEMEC Mechanical Strain Gauge system [10], widely used for in-situ monitoring of ASR-affected structures as described in [3], was adapted to measure residual expansion of the cores. DEMEC target discs (a.k.a. DEMEC points) were affixed to the sides of the cores in pairs so as to provide three 150-mm gauge lengths for axial expansion measurements. The gauge lengths were set at increments of approximately 120° rotation. This instrumentation method was selected to avoid the risk of damage caused by drilling into the ends to install gauge studs. The 150-mm gauge length was selected for commonality and to ensure that the DEMEC target discs were located at least 25 mm from each end of the core. Expansion measurements were made to the nearest 0.001 mm and specimens were also weighed at the time of each expansion measurement (nearest 0.1 g for the 50-mm diameter cores and nearest 0.02 kg for the 150-mm cores). The expansions of all three gauge lengths on each core were averaged to account for any warping that might occur.

An initial conditioning period was required to allow the cores to reach initial moisture saturation (hygrometric equilibrium, according to [3]). Following RILEM AAR-6.1 recommendations, a reduced temperature of 23 °C was used for the initial conditioning period [4]; this is intended to avoid promoting further development of ASR until after the cores reached saturation. During this period, frequent measurements were taken to monitor the swelling from moisture uptake and note when this ceased; at this point the specimens would be placed in 38 °C storage. The initial conditioning period lasted six days for the 50-mm cores stored in 1N NaOH, and between 50 and 54 days for the 150-mm cores stored over water. Specimens were pre-cooled to 23 °C for 20 ± 4 hours prior to subsequent measurements, which were made initially at weekly intervals, and then less frequently as the testing progressed. Testing continued to between 315 and 351 days for the 50-mm cores in 1N NaOH, and to between 420 and 456 days for the 150-mm cores over water.

3. RESULTS

3.1 Petrography

Petrographic evaluation of the cores representing the three different mixture designs verified that the concrete placed were generally consistent with the mixture designs described in project submittal documents. Visual examination of the cores received at the laboratory after air drying revealed that the cores extracted from Mix 1, which contained the natural gravel coarse aggregate (Coarse Aggregate 1 and Intermediate Aggregate 1) and no SCMs, exhibited damp patches or wet rims around coarse aggregate particles (Figure 3.1), a characteristic that is indicative of ASR [3]. Thin section evaluation using SEM/EDS confirmed the presence of slight-to-minor ASR; examples are shown in Figure 3.2. This primarily occurred within metagranite grains containing strained quartz (Figure 3.3), although a trace amount of rhyolite with amorphous or microcrystalline quartz was present in the aggregate population and also exhibited ASR.

Figure 3.1: Photograph of air dried core from Mix 1, showing wet rims around coarse aggregate indicative of ASR.

Damage due to ASR in Mix 1 concrete was minor with ASR gel formation mostly confined within cracks in the aggregate particles, in adjacent air voids, or extending out into the paste to a limited extent and sometimes bridging aggregates. The natural gravel aggregate was from an alluvial deposit containing a variety of rock types. In addition to those currently identified as alkali-silica reactive, metamorphosed sedimentary rocks and volcanics were present in the aggregate. The abundance of potentially reactive silica in the form of strained quartz and microcrystalline quartz within the aggregate population warranted further investigation into the potential for future expansion.

Microscopic examination of concretes from Mix 2 and Mix 3 showed that the crushed volcanic coarse aggregate (Coarse Aggregate 2 and Intermediate Aggregate 2) also showed slight or innocuous ASR of amorphous and microcrystalline quartz. The reaction was less common in this aggregate type than observed in the natural gravel (Coarse/Intermediate 1). The ASR gel formation was generally confined within cracks in the aggregate particles and adjacent air voids, with little-to-no cracking into the paste. Concrete in core samples from Mix 2 which contained both the crushed volcanic coarse aggregate and crushed natural gravel showed more significant ASR than concrete in core samples from Mix 3, which only contained the crushed volcanic rock type (Coarse/Intermediate 2). The majority of ASR in cores from Mix 2 was detected in the Intermediate Aggregate 1. Only a trace amount of ASR gel within pockets inside of aggregate particles was found in Mix 3 cores; examples are presented in Figure 3.4.

The surface distresses in the pavement which had prompted the investigation were not linked to ASR in any of the cores. The majority of ASR was found in the middle and bottom of the cores, resulting in microcracks within the aggregate, and was only detected using SEM/EDS.

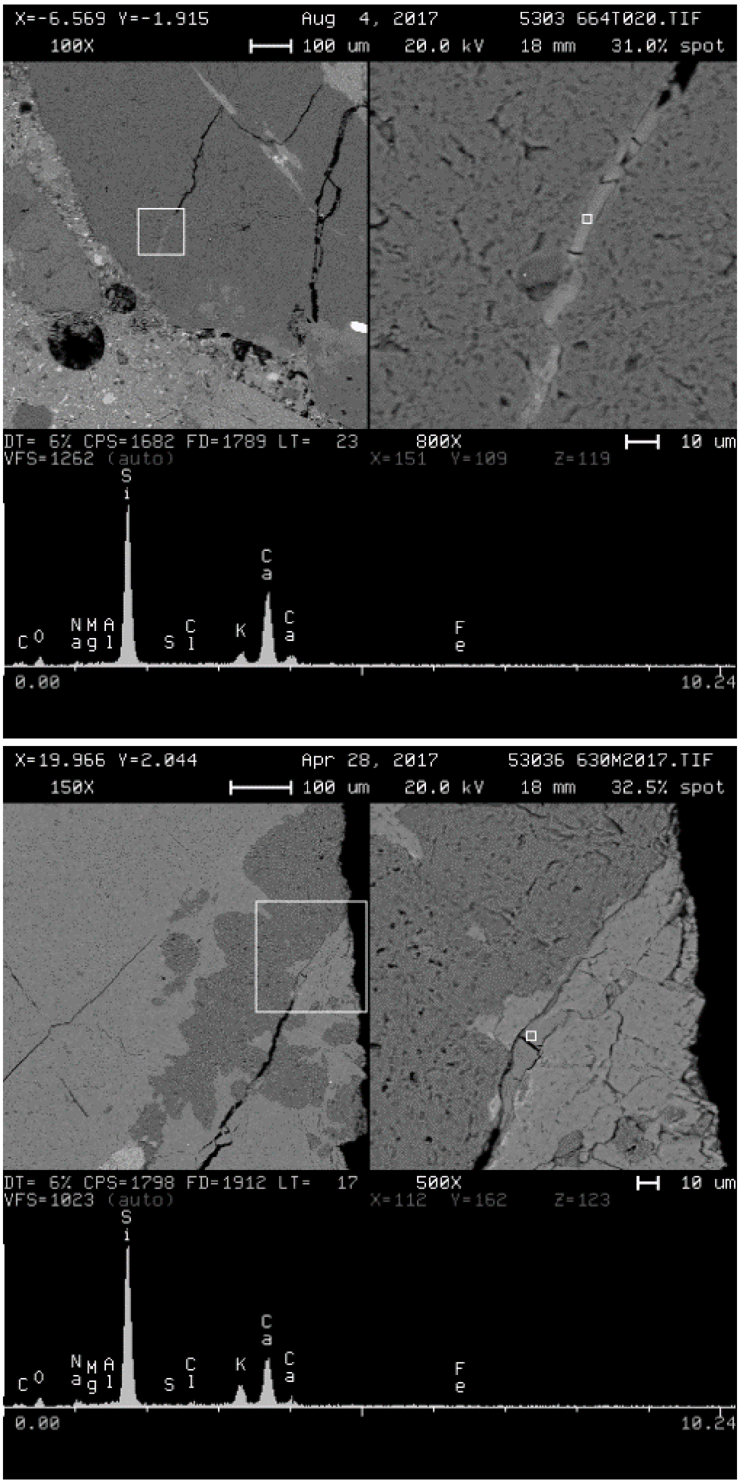

Figure 3.2: Backscattered electron (BSE) images with EDS spectra of ASR gel within cracks in Coarse Aggregate 1.

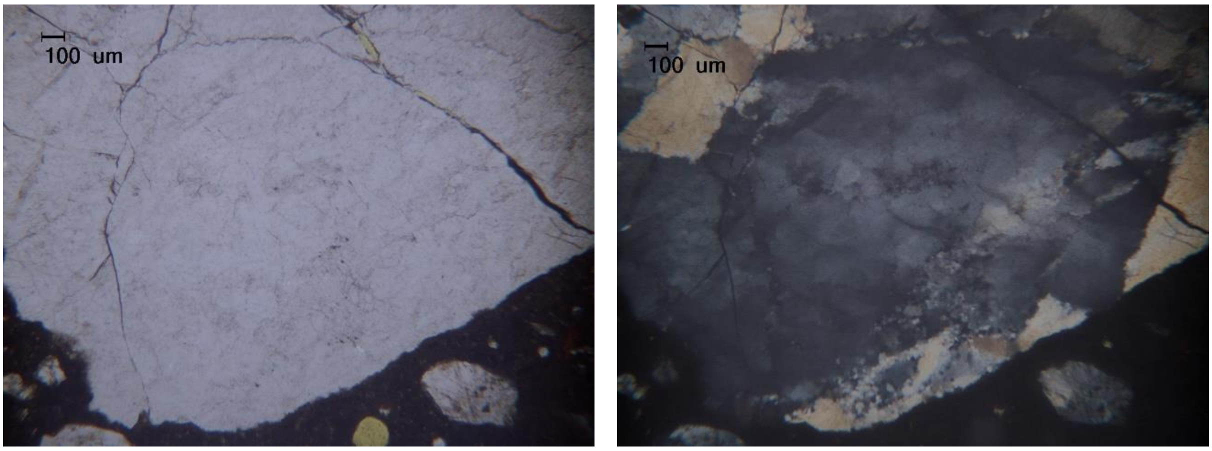

Figure 3.3: Polarized light micrographs in different light modes of reactive metagranite containing strained quartz found in Coarse Aggregate 1 and Intermediate Aggregate 1.

Figure 3.4: Backscattered electron (BSE) images with EDS spectra of ASR gel within pockets in Coarse Aggregate 2.

3.2 Residual Expansion

Figure 3.5 and Table 3.1 present the results of the residual expansion testing of 50-mm diameter cores in 1N NaOH at 38 °C; expansions are presented as average values for the three gauge lengths on each core. Figure 3.5 contains plots of the raw expansion, mass change, and adjusted expansion versus time. The adjusted expansions were calculated by subtracting the raw expansions at 6 days, to account any swelling during the initial conditioning period. Table 3.1 presents both the final raw and adjusted expansions at 351 days (315 days for 3-J-1 and 3-J-2) and the expansion measured over the final 99 days of monitoring.

Adjusted final expansions ranged from 0.023% to 0.051%, while expansions over the final three months ranged from -0.008% to 0.009%. While some slight expansion and mass gain were continuing for most specimens when residual expansion testing concluded, there was no indication that the rate of increase was accelerating for either property.

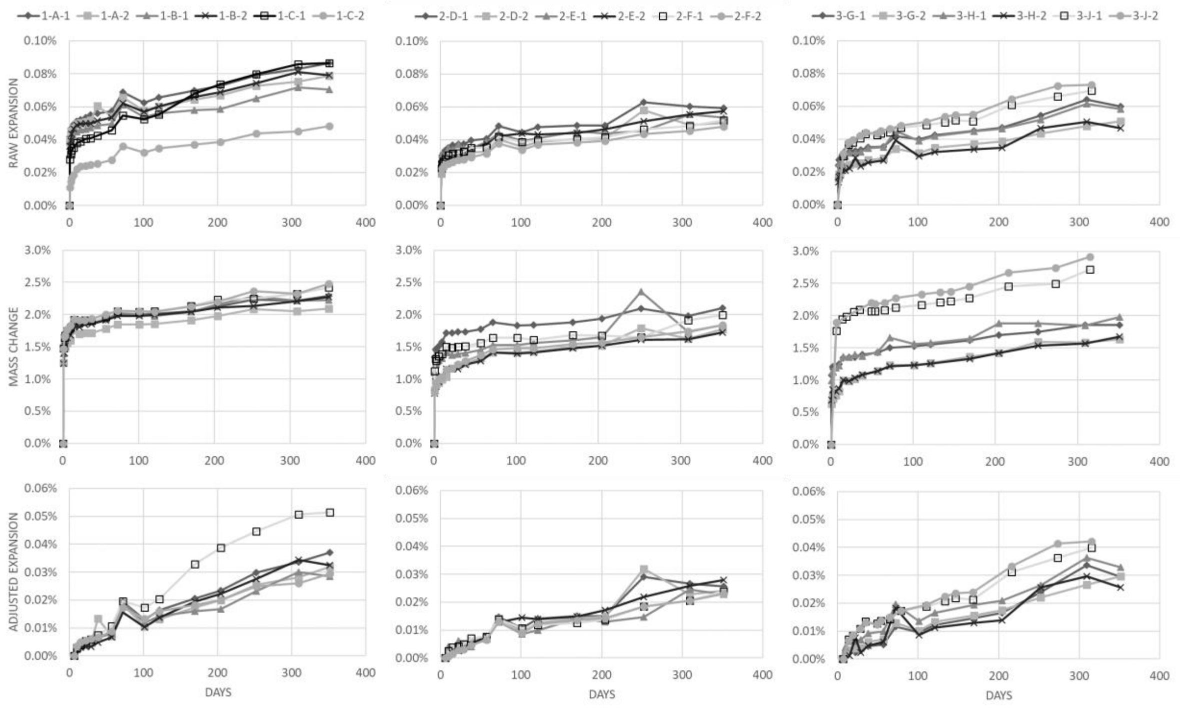

Figure 3.5: Raw expansion, mass change, and adjusted expansion for 50-mm diameter cores stored in 1N NaOH at 38 °C.

Table 3.1: Expansion Details of 50-mm Cores (1N NaOH at 38 °C)

Mix	Location	Core	Days in NaOH	Average Raw Final Expansion	Adjusted Final Expansion	Expansion Over Final 99 Days
1	A	1	351	0.086%	0.037%	0.007%
		2	351	0.079%	0.032%	0.004%
	B	1	351	0.070%	0.028%	0.005%
		2	351	0.079%	0.032%	0.005%
	C	1	351	0.086%	0.051%	0.006%
		2	351	0.048%	0.029%	0.004%
2	D	1	351	0.059%	0.025%	-0.004%
		2	351	0.050%	0.024%	-0.008%
	E	1	351	0.054%	0.023%	0.009%
		2	351	0.057%	0.028%	0.006%
	F	1	351	0.052%	0.024%	0.006%
		2	351	0.048%	0.023%	0.005%
3	G	1	351	0.060%	0.030%	0.006%
		2	351	0.051%	0.030%	0.008%
	H	1	351	0.051%	0.026%	0.006%
		2	351	0.047%	0.026%	0.000%
	J	1	315	0.070%	0.027%	0.009%
		2	315	0.073%	0.028%	0.009%

Figure 3.6 and Table 3.2 present the results of the residual expansion testing of 150-mm diameter cores over water at 38 °C; expansions are presented as average values for the three gauge lengths on each core. Figure 3.6 contains plots of the raw expansion, mass change, and adjusted expansion versus time. The adjusted expansions were calculated by subtracting the raw expansions at 50 days (54 days

for core 3-J), to account any swelling during the initial conditioning period. The end of the initial conditioning period is marked with a vertical line in the plots of raw expansion and mass change. Table 3.2 presents both the final raw and adjusted expansions at 456 days (420 days for core 3-J) and the expansion measured over the final 105 days of monitoring.

Adjusted final expansions ranged from 0.005% to 0.009%, while expansions over the final three months ranged from 0.000% to 0.002%. The expansion and mass change trends over the final three months of monitoring indicate that both properties had essentially stabilized at the end of the testing.

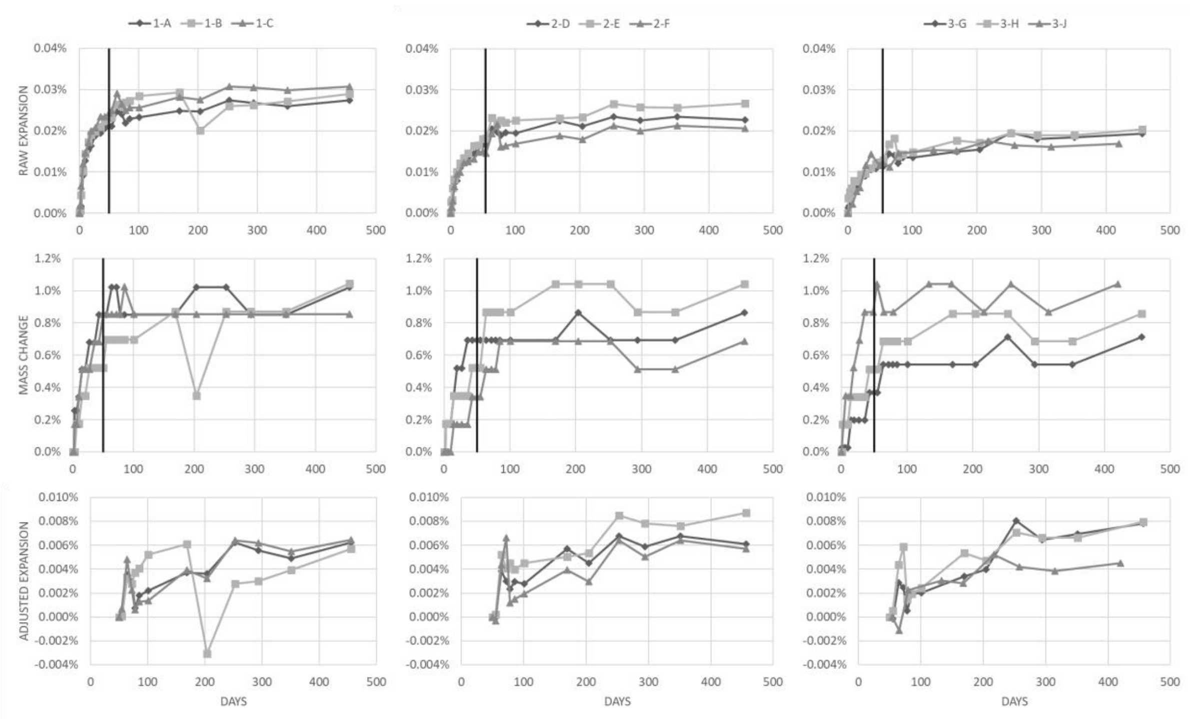

Figure 3.6: Raw expansion, mass change, and adjusted expansion for 150-mm diameter cores stored over water at 38 °C.

Table 3.2: Expansion Details of 150-mm Cores (Over Water at 38 °C)

Mix	Location	Days over Water	Average Raw Final Expansion	Adjusted Final Expansion	Expansion Over Final 105 Days
1	A	456	0.027%	0.006%	0.001%
	B	456	0.029%	0.006%	0.002%
	C	456	0.031%	0.007%	0.001%
2	D	456	0.023%	0.006%	0.000%
	E	456	0.027%	0.009%	0.001%
	F	456	0.021%	0.006%	0.000%
3	G	456	0.019%	0.007%	0.001%
	H	456	0.020%	0.008%	0.001%
	J	420	0.017%	0.005%	0.001%

Table 3.3 presents a summary of the average results by concrete mixture. In the 1N NaOH test, Mix 1 cores expanded the most with an average adjusted expansion of 0.035% per core, while Mix 2 cores expanded the least, with an average adjusted expansion of 0.025% per core. The average adjusted expansions over water similar for the three mixtures, with Mix 1 cores expanding an average of 0.006% and Mix 2 and Mix 3 cores both expanding an average of 0.007%.

Table 3.3: Adjusted Expansion Summary

Mixture Design	50-mm Cores in NaOH	150-mm Cores over Water
Mix 1	0.035%	0.006%
Mix 2	0.025%	0.007%
Mix 3	0.028%	0.007%

The averaged results for each mixture were then used as inputs to assess the overall potential for future expansion following the methodology described by Bérubé et al. [11] and recommended by the US FHWA [3]. The 1N NaOH test results were used to determine residual aggregate reactivity (RAR), while the over-water test results were used to determine residual concrete reactivity (RCE). Because all three mixtures had average expansions in NaOH below 0.04%, the RAR coefficient was 0 (neglible). The over-water expansions, however, indicated a low, but not negligible potential, and were assigned an RCE coefficient of 2.

This methodology also uses the effects of ambient conditions (humidity and temperature), confinement (reinforcement or external restraint), and available alkali loading. A humidity coefficient (HUM) of 1.0 was selected based on the pavement thickness, exposure to rain, and contact with the ground. An average annual air temperature of 17 °C resulted in a temperature coefficient (TEM) of 0.7. A structural confinement coefficient (STR) of 1.0 was conservatively chosen; the pavement is unreinforced, but pavements are still typically subject to some lateral confinement, which would yield a lower STR value. The investigation intended to include pore solution analysis to measure available alkalis, but it was not possible to extract sufficient pore solution from crushed samples. The alkali loading coefficient (ALK) was thus estimated from the mixture proportions and an assumption that 60% of the cement alkalis would be available. Some mill certificates were available, but given expected variability in materials, a slightly higher assumed Na_2O_{eq} of 0.60% was used. This resulted in available alkali loadings of 1.28 kg/m^3 for Mix 1 and 1.07 kg/m^3 for Mixes 2 and 3, and an ALK coefficient of 1 (low) for all three mixtures.

The future expansion potential was calculated as a current rate of expansion (CRE) using equations (1) and (2) as follows:

$$\mathrm{CRE} = [\mathrm{RAR}\ \times \mathrm{ALK}] \times \mathrm{HUM} \times \mathrm{TEM} \times \mathrm{STR} \tag{1}$$

$$\mathrm{CRE} = \mathrm{RCE} \times \mathrm{HUM} \times \mathrm{TEM} \times \mathrm{STR} \tag{2}$$

Equation 1 yields a CRE value of 0 for all three mixtures as a result of the negligible RAR coefficient and Equation 2 yields a CRE value of 1.4 for all three mixtures. The CRE values obtained from this methodology are qualitative in nature and correspond to a negligible-to-low continuing rate of expansion from ASR in the pavement for all three mixtures, including Mix 1 that did not contain fly ash for mitigation.

Long-term monitoring of the pavement could more conclusively establish the progression of ASR, but the combined assessment of the petrographic and residual expansion investigations is that the limited amount of ASR present was not the cause of the observed surface distresses and there is at most, a low potential for continuing expansion. Given that testing with 1N NaOH exposure is more severe, yet it yielded a lower CRE rating than testing over water, the risk of deleterious expansion and resulting damage to the pavement is deemed to be minimal at this time.

4. CONCLUSIONS

An airfield pavement exhibiting several forms of surface distress and containing potentially reactive aggregates was the subject of an extensive root-cause failure analysis investigation. One focus of this investigation was to determine whether ASR was present and if so, its severity and potential to cause future distress in the pavement. The following conclusions were drawn from the petrographic and residual expansion investigations, which included petrographic examination of more than 100 core samples and residual expansion testing of 27 core samples:

- Petrographic examination of core samples found the presence of ASR in concrete from all three mixture designs used in the construction of the pavement, but not linked to the surface distresses observed in the field investigation.

- The severity of ASR was qualitatively determined to be negligible-to-slight, and more prevalent in the gravel aggregate (Coarse/Intermediate 1) than the crushed volcanic stone (Coarse/Intermediate 2).
- Residual expansion testing of cores in 1N NaOH at 38 °C indicated a negligible potential for future expansion, while testing over water at 38 °C indicated a low potential for future expansion.
- The combined assessment is that there is only a minimal risk of deleterious expansion from ASR in the pavement, although long-term monitoring could provide more conclusive data. Repair and/or rehabilitation decisions are likely to be primarily driven by the severity and progression of the unrelated surface distresses.
- Two key differences from the FHWA-recommended procedures in [3] for residual expansion testing of cores were employed in this study and are recommended for future use: (1) An initial conditioning temperature of 23 °C while the core samples reached hygrometric equilibrium, as recommended in RILEM guidance [4], may reduce the chances of concurrent swelling from re-saturation and ASR; (2) Pre-cooling of cores to 23 °C for expansion measurements also reduced the potential measurement errors caused by thermal expansion and contraction and avoids the need for temperature corrections to measurements made in the initial conditioning period.

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