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Monitoring Alkali-Silica Reaction Significance in Nuclear Concrete Structural Members

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Abstract
A large-scale testing program on alkali silica reaction (ASR)-affected concrete structural members without shear reinforcement representative of structural members found in nuclear power plants is presented. Three concrete specimens, designed to experience a free expansion rate of approximately 0.15% per year were fabricated and placed within a controlled environmental chamber (38 ± 1°C (100 ± 2°F) and 95 ± 5% relative humidity (RH)). Sixty-four (64) embedded transducers and twelve (12) long-gauge fiber-optic sensors provide evidence of strong anisotropic expansion and oriented ASR-induced cracking resulting from the confinement effect caused by the reinforcement layout and additional structural boundary conditions. Surface cracking is not indicative of internal ASR-induced damage/expansion.

1. Introduction
Alkali-silica reaction (ASR) is a widely recognized degradation mechanism affecting numerous transportation and hydroelectric concrete infrastructures. However, recent disclosures in the nuclear industry in Canada, in Japan, and in the U.S. (Takatura et al. 2005; Shimizu et al. 2005; Tcherner and Aziz 2009; U.S. Nuclear Regulatory Commission 2011; NextEra Energy Seabrook 2013) raised concern about the structural significance of ASR on concrete nuclear structures in light water reactors (LWRs), such as the containment building, the biological shield or the spent fuel handling building. Demand for a better understanding of the mechanisms and effects of ASR in nuclear power plant structures has increased. While lessons-learned from the available operating experience and aging management programs can be valuably shared between the different industries confronted with this distress mechanism, typical reinforced concrete structural members in LWRs present specific challenges resulting from their geometry (thickness ranging from approximately 0.60 m to 1.5 m) and their reinforcement ratios and layout. In particular, shear reinforcement, i.e., transverse through thickness, is not required by ACI 318 (2014) and ACI 349 (2013) codes for structural strength due to the significant depth of concrete contributing to the shear resistance of the structural element. The absence of shear reinforcement in structural members subjected to ASR does not result unambiguously in either a gain or a reduction of shear capacity (Bach et al. 1993; den Uijl and Kaptijn 2003; Nakamura et al. 2008; Saouma et al. 2016). Several additional factors seem to contribute to the modification of the shear capacity: in particular, the reinforcement ratio, the ASR-induced expansion, and the structural boundary conditions (Saouma et al. 2016). ASR-induced expansion occurs primarily in the unloaded direction (Larive 1997), i.e., applied mechanical compression stresses result in the volumetric expansion transfer or redistribution in the direction of lower loading (Multon and Toutlemonde 2006). Applied mechanical stresses are caused not only by in-service external loading, but can also be induced by retrained ASR-expansion due to the presence of reinforcement (Multon et al. 2005), and unfavorable rigid structural boundary conditions (Saouma et al. 2016). Hence, it can be hypothesized that two competing mechanisms notably influence the residual shear capacity: (1) The ASR-induced self-prestressing in the direction of the reinforcement, and (2) the formation of anisotropic ASR-induced damage in the bulk of the reinforced concrete members.

Acknowledging the lack of experimental data on ASR-affected reinforced concrete structural members without shear reinforcement and subjected to different structural boundary conditions, and their relevance for the nuclear industry, a novel set of highly-instrumented large-scale experiments were designed and fabricated. This work includes two phases: (1) The monitoring of expansions and damage development in all directions inside the massive concrete and on the surface, and, (2) The destructive testing of the specimens to assess the post-ASR residual shear capacity. This article specially reports on the first phase of this program.

2. Large-scale testing program
2.1 Test specimens
The structural specimen detail was designed to closely
resemble a typical nuclear power plant containment structure, that is, a thick-walled concrete structure with no shear reinforcement. Specimens were designed with a thickness of 1.0 meter (3.28 ft.) and reinforced only in the plane of the wall with two elevations of intersecting large steel reinforcement leaving the thickness of the wall entirely unreinforced.

Three concrete specimens were conceptualized and constructed. An overview of the layout of the three specimens is shown in Fig. 1. The first specimen, referred to as the confined ASR specimen (CASR), was confined in a relatively rigid steel frame to simulate the additional confinement by surrounding concrete that would be present in a nuclear power plant (NPP) containment structure. This steel frame confines against expansion in the plane of the wall, forcing a preferred direction for expansion as the thickness direction of the specimen.

The second specimen, referred to as the unconfined ASR specimen (UASR), with identical mix design and steel reinforcement detail was designed and constructed with no surrounding steel frame. Thus, this specimen is unrestrained by exterior boundary conditions but still partially restrained against expansion in the plane of the wall by the steel reinforcing bars.

The third specimen, referred to as the control specimen (CTRL), with identical steel reinforcement was designed and constructed with two changes to the mixture design to minimize the potential for expansion from ASR. Sodium hydroxide (NaOH), used to promote the development of ASR in the CASR and UASR specimens, was not used in the mix design for the CTRL specimen. Instead, lithium nitrate was added to mitigate against the alkalis contributed by the cement. Thus far, no ASR expansion has been observed in this specimen.

For the sake of practicality, the specimens were cast horizontally. Hence, the actual through-wall thickness corresponds to the vertical direction (Z).

(1) Dimensions and Reinforcing Details
Specimen dimensions were selected to represent the scale of a typical NPP containment structure. The through-thickness dimension (Z-direction) is 1.0 meter (3.3 ft.). The dimensions within the plane of the wall were selected accordingly at 3.5 meter (11.5 ft.) and 3.0 meter (9.8 ft.) for the X-direction and Y-direction respectively. These dimensions are shown in Fig. 2.

The reinforcement layout for the specimens was also selected to most closely resemble that of a NPP structure. The reinforcement layout consists of US #11 Gr. 60 reinforcing steel bars with a nominal diameter of 35.81 mm (1.41 in.) spaced at 25.4 cm (10 in.) on-center resulting in two elevations of reinforcing bars embedded in the concrete specimens with 7.62 cm (3 in.) of concrete cover. The reinforcement layout results in reinforcement ratios for each direction as noted in Table 1. Additionally, the reinforcing steel bars were installed with square heads (10.16 cm × 10.16 cm × 2.54 cm) (4 in. × 4 in. × 1 in.) made of steel plate to achieve full development length within a relatively short distance inside the specimen.

To allow access to the bottom surface, the concrete specimens are elevated 76 cm (30 in.) above the floor and vertically supported at the corners by four steel columns capped with 45.7 cm × 45.7 cm (18 in. × 18 in.) steel plates.

(2) Concrete Formulation
As part of this study a trial testing was conducted to develop two mixtures; a reactive mixture that can exhibit rapid free expansion of approximately 0.15% per year; and a mitigated control mixture containing lithium nitrate that is designed not to expand from ASR. The mixture components used in both the reactive and control specimens included a highly-reactive greenschist coarse aggregate from North Carolina; a non-reactive manufactured sand from the Knoxville, Tennessee area; and a low-alkali Type II Portland cement with an equivalent alkali content of 0.41% Na₂O_eq. A 50% w/w sodium hydroxide solution (NaOH) was added to the

| Direction | Reinf. ratio |
|-----------|--------------|
| X-direction | 0.67% |
| Y-direction | 0.68% |
| Z-direction | 0.00% |

Note: Reinforcement ratio reported is total longitudinal reinforcement area divided by gross cross-sectional concrete area.

Fig. 1 Layout of the three large-scale specimens.

Fig. 2 Dimensions and coordinate axes of concrete specimens.
reactive specimens to increase the alkali content to 5.25 kg m\(^{-3}\) (1.50% Na\(_2\)O\(_{eq}\) by mass of cement). A 30% w/w lithium nitrate (LiNO\(_3\)) solution admixture was added to the control specimen at 150% of the manufacturer's recommended dosage (sometimes referred to in the literature as the "standard dose" of a molar ratio of [Li]/[Na+K] = 0.74 in the mixture) to mitigate the potential for ASR (McCoy and Caldwell 1951; Folliard et al. 2006; Thomas et al. 2007; Kim and Olek 2012). In addition, a high-range water-reducing admixture (meets requirements for ASTM C494 Type F) and hydration stabilizer admixture (meets requirements for an ASTM C494 Type D retarder) were added to maintain a slump value between 15 to 20 cm (6 to 8 in.) and offset the effects of warm ambient temperatures that would otherwise accelerate setting of the concrete during placement. The design water-to-cement ratio, w/c, was 0.50 for both mixtures. The mixture proportions used for the reactive and control specimens are shown in Table 2.

(3) Casting and Curing Conditions
Casting took place on July 23rd, 2016. In an attempt to mitigate potential damage sources other than ASR (e.g. thermal cracking), the formworks were insulated prior to the concrete placement by placing rigid foam sheathing insulation with an R-value of 3 around the sides. Additional sheathing was installed on top of the specimens, shortly after initial set and final finishing of the top surface. The insulation was placed with edges overlapping and secured in place with tape and plastic wrap.

To avoid any additional detrimental delayed ettringite formation (DEF) induced expansion, the temperature within the concrete specimens during early-age curing was kept below 70°C by substituting 70% of the mixing water with ice. The use of ice also permitted placement temperatures of 20°C or less, which complemented the hydration stabilizing admixture in terms of extending the time to set, ensuring sufficient time to place and finish the concrete in the laboratory.

All formworks were removed on August 4, 2016. After final finishing and setting of the concrete, each exposed specimen surface was sprayed with curing compound and then covered with a layer of wet burlap and plastic sheeting to minimize any moisture loss and mitigate early-drying-induced cracking. The burlap was periodically moistened until full operation of the environmental chamber.

(4) Steel Confinement Frame
In order to simulate the range of structural boundary conditions present in a large structure such as a NPP containment building, two cases are considered: (1) a reinforced concrete specimen unconstrained laterally, and, (2) a similar specimen encased in a rigid steel frame, restraining lateral deformation in the plane of the reinforcement and allowing unrestrained expansion through the specimen thickness. In order to reduce frictional effects between the steel frame and the concrete specimen, a single 1.5 mm-thick layer of high-density polyethylene (HDPE) was placed at the concrete-steel interface.

To provide sufficient rigidity, the confinement frame was designed for maximum stiffness. A steel plate girder cross-section was designed consisting of two 76 mm (3 in.) flanges and three 51 mm (2 in.) webs as shown in Fig. 3. Because of limited lifting capabilities, the frame was designed as four sections joined by slip-critical bolted connections, each consisting of twelve splice plates and 144 bolts. An illustration of the connected elements of the steel confinement frame is shown in Fig. 4.

The cross-section and connections were designed for a maximum pressure of 8 MPa at the steel frame-concrete interface and a corresponding maximum deflection of 2.5 mm (3/32 in.).

2.2 Material testing
Three groups of companion cylinders (150 cylinders total) made of the same concrete batches of the CASR, UASR, and CTRL specimens were prepared to measure

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**Table 2 Design proportions of concrete mixtures.**

| Materials          | Quantity, kg m\(^{-3}\) (lb yd\(^{-3}\)) | Reactive | Control |
|--------------------|------------------------------------------|----------|---------|
| Coarse Aggregate   | 1180 (1989)                              | 1180 (1989) |
| Fine Aggregate     | 728 (1227)                               | 728 (1227) |
| Cement             | 350 (590)                                | 350 (590) |
| Water*             | 175 (295)                                | 175 (295) |
| 50% NaOH solution  | -                                        | 9.8 (17) |
| 30% LiNO\(_3\) solution | 1.9 (20.0)                      | -        |
| Water reducing admixture | 2.0 oz/cwt                  | 2.0 oz/cwt |
| Stabilizer admixture | 2.0 oz/cwt                  | 2.0 oz/cwt |

Note: Aggregate quantities are given for oven-dry materials. Water quantities assume aggregates in saturated-surface dry (SSD) condition. (*) indicates that 70% of the mass of mixing water was replaced by ice, and the actual w/c ranged between 0.46 to 0.52.

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Fig. 3 Cross-section of steel confinement frame.
the evolution of the elastic modulus, compressive strength and splitting tensile strength with the ASR.

While the UASR and CTRL cylinder specimens were removed from molds 48 hours after casting, each CASR cylinder specimen was kept in a relatively rigid cylindrical steel mold with a wall thickness of 6.4 mm (1/4 in.) until the time of testing. The cylindrical steel molds were used to promote ASR-induced expansion primarily in the vertical direction, i.e. parallel to the casting direction, while the unrestrained cylinders were allowed to expand in all directions.

All cylinders were stored in the environmental chamber containing the large specimens. Mechanical testing was performed at 7, 28 days, and then at 3, 5, 6, 9, and 12 months. The elastic modulus, compressive strength and splitting tensile strength were measured using ASTM C469, ASTM C39 and ASTM C496, respectively.

2.3 Facilities

The specimens are stored in an environmental chamber designed for temperature and humidity control of $38 \pm 1^\circ \text{C} (100 \pm 2^\circ \text{F})$ and $95\% \pm 5\%$ RH. In order to maintain the temperature and RH during operation, a heating system consisting of both heating evaporators and heating units accommodated by air circulators was designed.

The environmental chamber was delivered as panels consisting of embossed steel filled with foam insulation. Each panel has a set of locks to secure adjacent panels to each other. The floor connection is sealed by a vinyl sealer placed underneath the wall panels to the concrete floor of the high bay laboratory. To allow the construction of the three concrete specimens, the environmental chamber was built around the specimens a few weeks after casting.

A full power up of the heating system was completed August 17, 2016 to test the operation of the heating and misting system as well as the lighting system. After confirmation of the systems working order, the system was powered down to finalize all connections and prepare the chamber for full time operation. All concrete specimens were uncovered at this time. The completed chamber measures approximately 16.2 m (53 ft.) long, 7.3 m (24 ft.) wide, and 3.7 m (12 ft.) high. This area allows for all three reinforced concrete specimens and all concrete cylinders for material testing to be contained within the same environment.

The chamber was initialized for full operation on August 19, 2016 (concrete age of 26 days). The chamber has been operated uninterrupted, except for periodic inspections, at the specified environmental conditions. The relatively high temperature and moisture content constitute a working safety hazard. Hence, the chamber is periodically shutdown for inspection on an average frequency of two days per month. During shutdowns, the average temperature and RH are about $25^\circ \text{C}$ ($\approx 77^\circ \text{F}$) and 60% (transient of about 4 hours). After the shutdown period, the chamber is restarted; the temperature and humidity return to the original set points within 6 hours. The time period of chamber shutdowns is small (less than 5% of overall monitoring time). The change in temperature during chamber shutdowns is accompanied by thermal strains of the concrete; however, because continuity is observed in the measured strains before and after temperature changes, the chamber shutdowns have no effect on concrete confinement provided by the steel frame.

2.4 Monitoring and sensing techniques

The concrete specimens were heavily instrumented to monitor local strain and temperature within the bulk of the specimens as well as structural deformations as the ASR progressed. In total, three different types of sensors are being utilized, as illustrated in Table 3.

| Sensor Type                  | Quantity |
|------------------------------|----------|
| Temperature: Thermocouple    | 12       |
| Deformation/Strain:          |          |
| Strain transducer            | 64       |
| Long gauge FO extensometer   | 12       |

Fig. 4 Plan view of assembled steel confinement frame.

(1) Embedded Temperature Sensors

Temperature monitoring is required to: (1) assess the (absence of) risk for DEF, (2) verify the uniformity of temperature within the specimens, and (3) provide potential thermal correction factors for additional measurements taken during temperature transients such as at early-age and during the environmental chamber shutdowns. Monitoring of temperature was initiated shortly before concrete placement using thermocouples that were built into the strain transducers.

A total of 4 thermocouples were placed in each specimen in such a way that two temperature gradients could be obtained. A vertical line of three thermocou-
amples were installed near the center mass of the specimen in order to measure the vertical (Z-direction) temperature gradient with two sensors installed 25 cm (10 in.) above and below the bottom and top surfaces of the specimens and one thermocouple installed mid-depth within the specimen. The fourth thermocouple within each specimen was placed at mid-depth near a corner to measure the X and Y plane temperature gradient.

(2) Embedded Strain Sensors
A total of 64 100 mm-gauge strain transducers (KM-100B from Tokyo Sokki Kenkyujo) were embedded in the concrete specimens. The transducers were installed using nylon cable ties to a support structure of 3 mm (1/8 in.)-diameter smooth steel bars installed between the two layers of steel reinforcement prior to concrete placement. These sensors have shown remarkable durability in previous research including ASR studies (Herrmann et al. 2008; Bracci et al. 2012).

These strain transducers sense the change in distance between two circular disks mounted on the ends of a tube which is wrapped with a protective coating and tape.

The strain transducers were arranged within the specimens as shown in Fig. 5. The placement of the strain transducers was designed to (1) measure strains in all directions, (2) evaluate strains special variability by increasing the density of sensors in a single quadrant, and (3) limit possible interactions with nondestructive evaluation (NDE) based on acoustic wave propagation techniques by reducing the density of sensors in the opposite quadrant.

(3) Long-Gauge Fiber-Optic Deformation Sensors
In addition to the local strain measurement, the overall structural expansion was also monitored for each principal direction. High precision (≤ 2 µm) and accuracy fiber-optic (FO) extensometers (SOFO standard deformation sensor from SMARTEC/Roctest) (Inaudi 1997; Glišić et al. 2013) were placed at the bottom surface of the specimens and inside the concrete, for the horizontal and vertical deformations measurements, respectively. The vertical FO extensometers, of 0.8 m-gauge length, measure the deformation between the top and bottom reinforcement layers and were attached to a 3 mm (1/8 in.)-diameter steel smooth bar with nylon cable ties before concrete placement. The 1.5 m-gauge length horizontal FO extensometers were placed at the bottom surface of the specimens to allow access to the top surface for NDE. The sensors ends are supported by angle-plates, anchored 7.6 cm (3 in.) deep in concrete, i.e., reaching the plane of reinforcement.

The layout of extensometers is similar for both the restrained and unrestrained specimens as shown in Fig. 6.

3. Results

3.1 Early-age
The internal temperature history of each specimen is shown in Fig. 7. The reported temperature for each specimen is the recorded maximum of the four thermocouples embedded in that specimen. The CASR and UASR specimens were cast first in the early morning, and the CTRL specimen was cast after midday when the temperature within the lab was higher. For all specimens, the internal temperature remained below 70°C, which

![Fig. 5 Layout for embedded strain transducers with specimen coordinate axes and specimen dimensions as described in Fig. 2.](image-url)
prevents the occurrence of DEF in the specimens. The temperature of each specimen slowly decreased from the peak of hydration heat stabilizing to room temperature of the laboratory over a period of nearly 20 days, thus minimizing the risk of thermal cracking.

3.2. ASR-Induced expansion

(1) Monitoring Data Correlation

Despite the differences in the two sensor systems (gauge length in particular), the expansion results between the two types of sensors are very agreeable. Figures 8 and 9 show the correlation between the local strain transducers and structural deformation sensors oriented to measure strain and deformation in the through-thickness (Z) direction for the CASR and UASR specimens. The mean of the transducer strain data is plotted against the structural deformation. The standard deviation (SD) lines indicate one standard deviation above and below the mean strain collected from the local strain sensors. The identity line designates perfect correlation. The linearity of the correlation plots indicates a good correlation between the short and long gauge lengths sensors in collecting expansion data. Hence, in the following sections, only the local strains collected by the embedded strain sensors will be reported and analyzed.

(2) Expansion

The expansion collected by the embedded strain sensors for each specimen in each direction is shown in Fig. 10. For the sake of readability, only the average expansions are plotted. The CTRL specimen exhibits relatively low shrinkage and no trend toward expansion even at a late stage of the experiment. The CASR specimen shows more vertical expansion and less lateral expansion than the UASR specimen. Because of the longer span of the steel confinement frame resisting Y-direction expansion, more deflection would be expected in this direction when compared to the X-direction if the pressure exerted by the ASR-induced expansion is relatively uniform; this reasoning would explain the differences in the Y-direction and X-direction expansions of the CASR specimen.

The volumetric expansion of the two reactive specimens is shown in Fig. 11. Other studies have concluded that ASR-induced volumetric expansion is independent
of stress state or boundary conditions when at least one direction is unloaded or unconfined (Multon and Tout-lemonde 2006; Gautam et al., 2017). The volumetric expansion of the two reactive specimens is nearly equivalent after one year of accelerated testing confirming these conclusions even for large-scale specimens.

3.3 Material properties

Figures 12 and 13 present the respective evolution of the compressive strength and elastic modulus with time, collected on cylinders made of the non-reactive concrete (CTRL) and reactive concrete allowed free expansion (UASR) or confined in metallic mold (CASR) While the elastic moduli show similar evolution for all specimens until 28 days, the CTRL present a much higher gain in strength at a lower age. The difference in compressive strengths between the reactive (CASR and UASR) and unreactive (CTRL) concrete specimens can be attributed to the addition of sodium hydroxide to the reactive concrete mix. Research has found that the addition of sodium hydroxide to concrete causes a significant reduction in compressive strength due to the formation of porous cement paste; however, the modulus of elasticity is not affected by the addition of sodium hydroxide (Smaoui et al., 2005). Following a hardening until about 200 days, the compressive strength measured on the CASR and UASR specimens exhibit a decrease apparently independent of the confinement. The elastic moduli collected on the reactive sample specimens show a rapid decrease at a relatively young age. The UASR specimens exhibit higher loss of elastic moduli (> 50% loss at > 250 days) than the confined specimens (= 25% at > 250 days).

4. Discussions

4.1. Significance for in-situ conditions assessment

(1) Visual Inspections

The CASR and UASR specimens were periodically inspected for surface cracking. After the first surface cracking was observed, inspection was conducted monthly for new surface cracks. Visible surface cracks were marked with different colors at each inspection to capture the periodic time history of surface cracking. The side surfaces (X-Z and Y-Z planes) and top surface (X-Y plane) of the UASR specimen and the top surface
Visible cracking first occurred on the lateral sides of the UASR specimen with an early (≈ 150 days) primary crack orientation indicating a preferred direction of expansion through the thickness, in agreement with the measured strains. At a later stage, branching and pattern-cracks developed on the sides as shown in Fig. 14. Limited and hardly discernible cracking could be observed on the top surface of the UASR specimen after ≈ 300 days. Figure 15 shows timings of first observations of visible cracking on the UASR specimen and corresponding levels of expansion. However, no cracking was observed on the top surface of the CASR specimen after one year of accelerated testing.

In a NPP containment structure, the cracking that would occur generating through-thickness expansion would not be observable as containment structures are typically continuous walls, and hence, the thickness plane is not accessible similar to the CASR specimen in this research.

As a result, assuming that the development of ASR-induced cracking follows a similar chronology in the field, visual inspection may not be a reliable approach to diagnose the formation of ASR in concrete structures without transverse reinforcement until an advanced stage of development.

(2) Sensors Resilience
The high moisture content, fairly high-temperature, and high-alkalinity of the operating environment poses a significant challenge to the durability of the sensors. Of the 64 strain transducers and 12 long-gauge FO extensometers embedded in the specimen or attached to the surface of concrete, only one long-gauge FO sensor was lost as a result of unrelated construction works of the environmental chamber.

The long-gauge FO deformation sensors have proven to be a rugged and robust sensor. Three conclusions for these sensors can be made from the monitoring campaign: (1) the sensors are resilient to the harsh environment presented by the environmental chamber; (2) the sensors are easily affixed to concrete surfaces; (3) the sensor measuring device and software are reliable and simple to use. For these reasons, the long-gauge FO deformation sensors have strong potential for field implementation as an automated method to track residual expansion of ASR-affected structures.

(3) Expansion Anisotropy
The macroscopic expansion measured on both reactive specimens is strongly anisotropic. While this can be attributed, to some degree, to the casting direction of the concrete specimens (Smaoui et al. 2006), most of the anisotropy of the macroscopic expansion measured in the present research stems from the boundary condition (steel confinement frame) and reinforcement layout. Figure 16 presents both CASR and UASR vertical (Z) expansions as a function of one-third volumetric expansion. Figure 17 shows the CASR and UASR expansion in the X direction as a function of one-third volumetric expansion. On both figures, the identity line represents a situation of perfectly isotropic expansion.

Both specimens exhibit anisotropic expansion with a preferred direction along the thickness of the specimens (Z). For the UASR specimen, this anisotropic expansion...
is caused by the difference in reinforcement ratio between the in-plane directions (X, Y) and through-thickness direction (Z) in addition to the preferred expansion attributed to casting direction. For the CASR specimen, the anisotropic expansion is caused by both the reinforcement and the passive restraint from the steel confinement frame, the latter accounting for a 33% increase in vertical (Z) expansion compared to the UASR specimen after one year of accelerated testing.

The effect of the boundary conditions is particularly noticeable in the in-plane directions. The expansion of the CASR specimen in the X-direction is notably lower after one year of accelerated testing when compared to the expansion of the UASR specimen in the same direction. The additional restraint of the boundary condition (steel confinement frame) accounts for a 50% reduction in X-direction expansion. The boundary condition produces a similar effect on the Y-direction expansion; however, the additional reduction of expansion due to the confinement is less than that of the X-direction reduction due to the longer span of the steel confinement frame resisting Y-direction expansion.

This anisotropy in the macroscopic expansion is likely to be related to a preferred orientation of the ASR-induced cracks under local stress. Due to the specific geometry of the reinforcements, the stress distribution is non-uniform across each specimen, making it difficult to analyze the cracking distribution and therefore the anisotropy in macroscopic strain without relying on nonlinear numerical analysis. This experimental campaign represents a good opportunity to test and validate structural models for ASR, notably to check whether such a model can capture the anisotropy of the macroscopic strain.

(4) Effect of Confinement Materials Properties

Figures 18 and 19 show the respective evolution of the relative modulus of elasticity, and compressive strength (normalized by the values of these properties at 28 days after casting) with the averaged expansion measured on the corresponding large specimens. It must be noted that the expansion of the large specimens is not necessarily representative of the expansion of cylinders. However, the measured expansion on the large specimens is used as a reference to compare the behavior of the CASR and UASR cylinders.

Figure 18 compares the evolution of elastic modulus in the confined and unconfined cylinders. It can be seen that the stiffness degradation in the UASR cylinders was significantly higher than that in the CASR cylinders. While the relative modulus of elasticity of the UASR cylinders decreased to 43% at a linear expansion of 0.12%, the CASR relative modulus decreased to 63% of the relative modulus of elasticity at an estimated linear expansion of 0.13% and slight recovery was noticed afterward. This behavior may be attributed to the contribution of the lateral confinement stress in reducing the loss of the elastic modulus (Gautam et al. 2017).

As shown in Fig. 19, both the UASR and CASR cylinders showed a remarkable gain in compressive strength until an estimated expansion of about 0.10% and followed by noticeable reduction at an expansion of ≈ 0.12% before a partial gain was observed. As two competing mechanisms, the ongoing hydration of cement and ASR damage, were progressing during early stage of expansion, the increase in compressive strength due to cement hydration seemed to overcome the loss of strength due to ASR (Multon et al. 2005; Na et al. 2016). At later stage (i.e., for expansion > 0.1%), the ASR damage effects seemed to be dominant. It must also be noted that the compressive strength was affected by the direction of ASR-induced damage, the CASR cylinders (where cracking is primarily oriented parallel to the loading direction) showed higher strength values than that of the UASR, except for the last measurements the UASR was higher.

The slight recovery of the mechanical properties at a late stage of ASR expansion was reported in the literature (Swamy and Al-Asali 1988; Ahmed et al. 2003; Gautam 2016). This recovery can be attributed to the
continuation of cement hydration process (Swamy and Al-Asali 1988), or to the transformation of ASR gel in cracks into a more calcium-rich gel (eventually resembling C-S-H gel) which can contribute to the concrete regaining strength and stiffness (Gautam 2016).

5. Conclusions and prospective work

(1) Three large-scale specimens, representative of concrete structural members thickness and reinforcement ratio without transverse reinforcement found in LWRs NPPs, were fabricated, heavily instrumented and monitored under controlled accelerated ASR conditions at 38°C and = 95% RH.

(2) The different types of deformation instrumentation, i.e., long-gauge fiber optics or embedded transducers, have yielded comparable and dependable expansion measurements despite the severity of the operating conditions, i.e., moderate temperature, high humidity and high alkalinity.

(3) Both reinforcement layout and boundary conditions cause highly anisotropic expansion pointing to the need for advanced structural models capable of capturing expansion anisotropy for analysis of in-the-field behavior.

(4) Surface cracking is not indicative of internal ASR-induced damage or expansion for concrete structures where reinforcement layout or confinement drives expansion primarily in an unobservable plane direction. This potentially allows ASR to cause significant distress to the structure without any visible evidence of its presence.

(5) Visible surface cracking was not evident on the CASR specimen even with Z-direction expansion exceeding 0.3%. Given that acoustic NDE methods are most influenced by the onset of ASR damage (Giannini and Folliard 2012; Giannini et al. 2016; Kim et al. 2017), the observations of the CASR specimen highlight the need to monitor critical NPP structures before visible evidence of damage appears at the surface, and support the concept that online monitoring of these structures using acoustic methods can provide information not available from visual inspection.

(6) Data collection and surface monitoring will continue until the end of the monitoring phase of the research program. Several research collaborators were invited to perform NDE on the large-scale specimens to evaluate the degree of ASR within and near the surface of the specimens.

(7) Following the conclusion of the monitoring phase of the program, the specimens will be subjected to destructive testing to assess the post-ASR residual structural capacity.

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