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Effect of alkali–silica reaction on the shear strength of reinforced concrete structural members. A numerical and statistical study

Victor E. Saouma [a], Mohammad Amin Hariri-Ardebili [a], Yann Le Pape [b,*], Rajagopalan Balaji [a]

[a] Department of Civil Engineering, University of Colorado, Boulder, CO 80305, USA
[b] Oak Ridge National Laboratory, One Bethel Valley Road, Oak Ridge, TN 37831, USA

HIGHLIGHTS

- Alkali–silica reaction (ASR) affects reinforced structures shear strength.
- Statistical analysis indicates large scattering of post-ASR strength losses/gains.
- Competitive structural and materials mechanisms affect the residual shear strength.

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ABSTRACT

The residual structural shear resistance of concrete members without shear reinforcement and subject to alkali–aggregate reaction (ASR) is investigated by finite element analysis. A parametric numerical study of 648 analyses considering various structural members' geometries, boundary conditions, ASR-induced losses of materials properties, ASR expansions and reinforcement ratios is conducted. As a result of competitive mechanisms (e.g., ASR-induced prestressing caused by the longitudinal reinforcement) and loss of concrete materials properties, important scatter in terms of gain or loss of shear strength is observed: about 50% of the studied configurations lead to a degradation of structural performance. The range of variation in terms of post-ASR shear resistance is extremely scattered, in particular, when ASR results in out-of-plane expansion only. Influencing factors are derived by two methods: (i) visual inspection of boxplots and probability distributions, and (ii) information criteria within multiple-linear regression analysis.

1. Introduction

Whereas alkali–silica reaction (ASR) has been reported in numerous hydro-electric dams, only recently has there been evidence of such occurrences in nuclear power plants (NPPs): in Japan, Ikata No. 1, Shikoku Electric Power (Takatura et al., 2005; Shimizu et al., 2005); in Canada, Gentilly 2 (Tcherner and Aziz, 2009; Sanborn, 2015); and in the United States, Seabrook (ML121160422, 2012; Haberman, 2013), for which the US Nuclear Regulatory Commission (NRC) issued Information Notice (IN) 2011-20, "Concrete Degradation by Alkali Silica Reaction," on November 18, 2011. Considering that US commercial reactors in operation have reached the age when ASR degradation could be visually detected and that numerous nonnuclear infrastructures (transportation, energy production) have already experienced ASR in a large majority of the States (e.g., a US Department of Transportation (DOT) survey reported by Touma (2000)), the susceptibility and significance of ASR for nuclear concrete structures must be addressed in the perspective of license renewal and long-term operation beyond 60 years. Yet, ASR has seldom, if ever, been reported in the open literature in connection with its significance for Safety Class I[1] nuclear concrete structural components, e.g., the concrete biological shield, the concrete containment building (CCB) and the fuel handling building.

The interaction of ASR with concrete shear strength is of primary interest. At the material level, the concrete degrades and

Abbreviations: AAR, alkali–aggregate reaction; ASR, alkali–silica reaction; AIC, Akaike Information Criterion; BIC, Bayesian Information Criterion; BC, boundary condition; B, beam; CCB, concrete containment building; DOT, Department of Transportation; FR, fully restrained; IN, Information Notice; IQR, interquartile range; LWR, light water reactor; MLR, multiple linear regression; NPP, nuclear power plant; NRC, Nuclear Regulatory Commission; P, panel; PDF, probability distribution function; PWR, pressurized water reactor; Q1, first quartile; Q2, second quartile; Q3, third quartile; R, restrained; RC, reinforced concrete; SLR, simple linear regression; SSE, sum of squared errors; TB, truncated beam; U, unrestrained.

* Corresponding author.
E-mail address: lepapeym@ornl.gov (Y. Le Pape).

[1] Class I referring to leak tightness, this study does not address the impact of ASR-induced cracking on possible gas diffusion.

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undoubtedly its shear resistance is decreased. On the other hand, at the structural level, ASR induces additional compressive stresses, similar to a prestressing effect, which increase the shear resistance of the structural component. An illustration of this effect is given by the analysis of the simple Mohr–Coulomb equation: $\tau = c + \sigma \tan\phi$. For a cohesion of $c = 3$ MPa and an internal friction angle of $\phi = 40^\circ$, then for a compressive stress σ of 0, 3 and 5 MPa, the resulting shear strength would be 3.0, 5.5 and 7.2 MPa respectively. Furthermore, the ASR expansion is likely to reduce the crack opening due to shear, hence additional aggregate interlock is present Blight and Alexander (2008).

The effects of ASR on structural members' resistance are apparently contradictory. When shear reinforcement is present, the expansion confinement results in a prestressing of concrete that contributes to an increasing shear capacity, e.g., Giannini et al. (2013), or an absence of significant change Fan and Hanson (1998). However, it was observed that shear failure may shift from a truss mechanism to a arch mechanism Wang and Morikawa (2012). Also, excessive ASR-induced self-prestressing may cause reinforcement yielding and failure (Nakamura et al., 2008; Miyagawa, 2013). Of particular concern is the case of beams, or slabs, in absence of shear reinforcement (Bach et al., 1993; den Uijl and Kaptijn, 2003; Schmidt et al., 2014). den and Kaptijn (2002, 2003) tested the shear capacity of six beams sawn from two flat-slab bridges suffering from ASR in the Netherlands. The failure mode was of the shear-tension type, whereas flexural-shear failure would have been expected if no ASR was present. The shear failure capacity was 25% lower than the expected theoretical resisting capacity in absence of ASR. In situ shear tests on a cantilever bridge deck conducted by Schmidt et al. (2014) in Denmark showed significant loss of bearing capacity (i.e., smaller shear failure force) in the severely ASR-affected area. Finally, effect of shear span ratios on the failure modes of ASR-affected beams (truss or arch like) has not been investigated to the best of the authors' knowledge.

The absence of shear reinforcement (i.e., in thickness) permitted by ACI 349 (Code Requirements for Nuclear Safety-Related Concrete Structures) is common in nuclear concrete structures, resulting in a lack of confinement that primarily imposes out-of-plane ASR expansion. Hence, the residual (i.e., post-ASR) shear-bearing capacity relies in large part on the concrete bulk shear resistance. The residual shear capacity (i.e., reduced shear-carrying capacity following an accidental design scenario, such as a seismic excitation) of ASR-affected Safety Class I structural components is expected to depend on two competitive mechanisms: (1) the extent of material damage, i.e., microcracking, potentially facilitating their coalescence into a macrocrack and(2) the relative in-plane compressive prestressing induced by some level of structural confinement and the orientations of the reinforcement, potentially increasing the shear capacity.

This question remains unresolved, and further investigation is needed to determine the potential impact of ASR on the structural resistance of nuclear structures. This article presents an extensive parametric numerical study providing novel insight on the effect of ASR on the shear capacity of reinforced concrete (RC) members without shear reinforcement.

2. Model and methodology

2.1. ASR constitutive law

The theoretical underpinning of the model used in this paper has been presented by the authors separately, Saouma and Perotti (2006). It will be briefly reviewed. The ASR expansion is considered to be a volumetric one, whose rate is given by the function

$$\dot{\varepsilon}_V^{ASR}(t) = \Gamma_t(f'_t|w_c, \sigma_I|COD_{max})\Gamma_c(\overline{\sigma}, f'_c)g(h)\dot{\xi}(t,\theta)\varepsilon^\infty|_{\theta=\theta_0} \quad (1)$$

where ε^∞ is the final volumetric expansion as determined from laboratory tests at temperature θ_0. $0 \leqslant \Gamma_t \leqslant 1$ is a parameter that reduces the expansion in the presence of large tensile stresses (macrocracks absorbing the gel), $f\prime_t$ the tensile strength, and σ_I the major (tensile) principal stress. Similarly, $0 \leqslant \Gamma_c \leqslant 1$ is a parameter that accounts for the absorption of the gel due to compression-induced stresses, and $\overline{\sigma}$ and $f\prime_c$ are the hydrostatic stress and the compressive strength of the concrete. $0 \leqslant f(h) \leqslant 1$ is a function of the humidity (set to zero if the humidity is below 80%), and $\dot{\xi}t, \theta$ is the kinetics law given by

$$\xi(t,\theta) = \frac{1 - e^{-\frac{t}{\tau_C(\theta)}}}{1 + e^{-\frac{(t-\tau_l(\theta))}{\tau_C(\theta)}}} \quad (2)$$

where τ_l and τ_c are the latency and characteristic times respectively. The first corresponds to the inflexion point, and the second is defined in terms of the intersection of the tangent at τ_L with the asymptotic unit value of ξ, Fig. 1.

The latency time τ_l and characteristic times τ_c are given by

$$\begin{aligned} \tau_l(\theta) &= \tau_l(\theta_0)\exp\left[U_l\left(\tfrac{1}{\theta} - \tfrac{1}{\theta_0}\right)\right] \\ \tau_c(\theta) &= \tau_c(\theta_0)\exp\left[U_c\left(\tfrac{1}{\theta} - \tfrac{1}{\theta_0}\right)\right] \end{aligned} \quad (3)$$

expressed in terms of the absolute temperature ($\theta^o K = 273 + T^\circ C$) and the corresponding activation energies. U_l and U_c are the activation energies' minimum energy required to trigger the reaction for the latency and characteristic times respectively.

Once the volumetric ASR strain is determined, it is decomposed into a tensorial strain in accordance with the three weight factors associated with the principal stresses (Saouma and Perotti, 2006). Note that the material follows a time-dependent deterioration as

$$\frac{E(t,\theta)}{E_0} = 1 - (1-\beta_E)\xi(t,\theta) \quad (4)$$

$$\frac{f'_t(t,\theta)}{f'_{t0}} = 1 - \left(1-\beta_f\right)\xi(t,\theta) \quad (5)$$

where E_0 and f'_{t0} are the original elastic modulus and tensile strength, and β_E and β_f are the corresponding residual fractional values when ε_{ASR} tends to ε_{ASR}^∞.

2.2. Model selection

A representation of the analyzed structural components in regard to actual CCB geometry is illustrated in Fig. 2. On the left side are sketched the *beam*, the *truncated beam* and the *panel* under investigation. Because of the large curvature radius of the CCB (about 20–25 m), reinforcements are considered straight for the sake of simplicity.

2.3. Boundary conditions

As stated earlier, two sets of boundary conditions are considered: (1) those applicable during ASR expansion (simulation increments 1–73) and (2) those applicable during the application of external shear force (simulation increments 74–174). The former is to capture potential ASR strain realignments caused by external constraints (Multon, 2004) as implemented in the author's model (Saouma and Perotti, 2006).

The choice of geometries (beam, truncated beam and panel) and boundary conditions (unrestrained, restrained and fully restrained) reflects an attempt to model gradually increasing structural restraints, as existing in actual Safety Class I concrete structures

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