Structural Performance Evaluation and Monitoring of Reinforced Concrete Shear Walls Affected by Alkali-Silica Reactions

Shohei Sawada1*, Yoshikazu Takaine2, Takashi Okayasu3, Arinori Nimura3 and Ryu Shimamoto4

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1. Introduction

Formerly, in plant life management for nuclear power facilities in Japan, the alkali-silica reaction (ASR) was not regarded as a main deterioration factor affecting the structural performance of nuclear concrete structures since aggregate was carefully selected at the time of construction and signs of ASR were not observed in the past operational experiences except in the case of the turbine generator pedestal of Ikata Unit 1 (Takakura et al. 2005; Manabe et al. 2016). On the other hand, ASR on nuclear concrete structures was identified in Canada and in the USA (Chenier et al. 2012; US NRC 2011; NextEra Energy 2012). Studies focusing on ASR have been actively conducted mainly in Europe (OECD NEA 2017, 2019; IRSN 2016) and North America (Hayes et al. 2018a; Ferche et al. 2019) in recent years. Attention on ASR in the field of civil engineering is also increasing in Japan (Torii et al. 2016; Kawabata et al. 2018; Takahashi et al. 2018), as evidenced by a string of discussions on ASR by academic societies such as the Japan Concrete Institute (Torii et al. 2008; Yamada et al. 2014, 2017). Consequently, the Nuclear Regulatory Authority of Japan also considers ASR as a potential risk to nuclear facilities and has been conducting a research project related to ASR in recent years. In light of the above trends, more detailed evaluations of ASR may be required for plant life management when ASR is detected in nuclear concrete structures. Therefore, it is necessary to accumulate knowledge on soundness evaluation methods of nuclear concrete structures affected by ASR.

Nuclear power facilities in Japan are rigid structures consisting of extremely thick concrete wall members. The structural performance of these facilities against severe loads such as earthquakes relies mainly on reinforced concrete (RC) shear walls. It is therefore important to understand the influence of ASR on the structural performance of shear walls. However, only a few studies focusing on the structural performance of shear walls affected by ASR (Murazumi et al. 2005; Habibi et al. 2015; Panesar et al. 2017, 2019) exist in the literature. Murazumi et al. (2005) conducted lateral loading tests using wall specimens with or without ASR. From the comparison of the load-displacement relationships, the authors concluded that there is no significant indication that ASR lowers the stiffness and strength of walls, even though inadequate slippage between the wall part and the base slab occurred in the tests. Other experiments using wall type specimens were performed by Habibi et al. (2015), Panesar et al. (2017) and by Ferche et al. (2019). These experiments also confirmed the shear performance of ASR affected walls and indicated that ASR walls increased in initial stiffness and ultimate strength but lateral displacement at ultimate strength became smaller at same ASR expansion with the longer age. The control specimens without ASR, however, showed asymmetric load-displacement curves in the positive and negative directions, which may imply inadequate experimental set-up. Both tests showed positive results for the structural performance of the shear walls affected by ASR, but
their experimental correctness is disputable.

In addition to understanding the behavior of ASR affected concrete members subjected to external loading, it is also important to develop how to evaluate the structural performance of these members using the information obtained from the in-situ real conditions and how to monitor the degree of ASR. This paper describes the results of monitoring and lateral loading testing of reduced RC walls affected by ASR and subsequent simulation analyses, and proposes practically appropriate structural performance evaluation and monitoring methods for ASR affected RC members, based on the results of experiments and analyses.

2. Experiments

When expansion due to ASR occurs in RC members, internal stress is generated depending on the boundary conditions. This stress acts as pre-stress and influences the structural performance of RC members. This is one of the key factors to evaluate the structural performance of ASR affected structures in addition to material property changes of concrete due to ASR. Therefore, reduced RC shear walls were prepared with variable constraint conditions and their lateral loading tests were conducted after application of a certain degree of ASR expansion. During ASR expansion, the state of the RC walls was also measured by several techniques in order to assess applicable monitoring methods. The flow of the experiment from fabrication to the loading test is shown in Table 1.

### 2.1 Specimens

The test cases are listed in Table 1. Two types of specimens were fabricated, one under moderate constraint condition and the other under severe constraint condition during exposure. Each specimen consists of a shear web wall, two flange walls, and upper/lower slabs. The dimensions of the web wall are 1500 mm length, 1000 mm height and 200 mm thickness with vertical/horizontal steel reinforcement ratio of 1.0%. The details of the specimen are shown in Fig. 1. This specimen is exactly the same shape as that used in a previous study conducted under ASR free condition (Oyamoto et al. 2017), so the results of the case CSW-0 in the above study are used in this study as a reference case for the ASR free condition, even though the materials used are not exactly the same.

A set of web walls (numbered as No. 1, No. 2, No. 3 and No. 4) were cast at the same time, demolded after two days and cured indoors for 39 days in sealed condition covered with concrete curing sheets. Expansive concrete with reactive aggregate was used only in the shear web wall, and ASR free concrete was used for the other parts. During sealed curing, for the No. 2 and No. 4 specimens, two flange walls and upper/lower slabs were fabricated around the web wall. Afterwards, a wet pad in
which water was periodically poured was placed on both surfaces of all web walls to maintain high humidity during exposure (see Fig. 2), and the specimens, covered with rain sheets, were moved outdoors. Throughout the entire period, all specimens were exposed to outside ambient temperatures. The condition of the specimens during exposure is shown in Fig. 3, in which wet pad and rain sheets were removed. The ambient temperature history inside the rain sheets after casting of concrete of specimen Nos. 3 and 4 is shown in Fig. 4, together with internal temperature of specimens recorded by thermo-couples embedded in the web walls as mentioned in Section 2.3. The temperature seasonally varied from a maximum of 45°C to minimum of minus 5°C.

The No. 1 and No. 2 specimens were mainly used for loading tests, whereas the No. 3 and No. 4 specimens were used only for monitoring. After 12 months of exposure, at which point the wet pad was removed, the No. 1 and No. 2 specimens were transported to a testing facility and kept under air-drying condition for one month until loading test start. During air-drying exposure, two flange walls and upper/lower slabs were attached to
the web wall of No. 1 in preparation for the loading test. Lower slab, two flange walls and upper slab were cast approximate one week, two week and one month after air-drying started, respectively.

2.2 Materials
The materials used and the mix proportions of ASR concrete are listed in Tables 2 and 3, respectively. Crushed andesite that included cristobalite, tridymite and glass as ASR reactive minerals was used as reactive aggregate only for coarse aggregate. Crushed limestone and crushed sand of limestone were used as non-reactive aggregate. In consideration of the pessimum effect, the mixing ratio of the coarse aggregate was set to 3:7 for reactive aggregate (andesite)/non-reactive aggregate (limestone). Ordinary Portland cement with 0.57% alkali content as Na₂O equivalent (=NaO+0.658×K₂O) was used. Sodium hydroxide was added to the mixture to achieve total alkali content of 5.5 kg/m³ in concrete. In order to understand the potential expansion of the reactive concrete, a preliminary experiment was conducted using concrete prism specimens (100×100×400 mm) wrapped in wet cloth and under different temperature conditions, one at 40°C and the other at a temperature simulating that to which the wall specimens were exposed. The expansion after 12 months of exposure was approx. 2600×10⁻⁶ and 1600×10⁻⁶ at 40°C and at the simulated temperature, respectively.

Table 4 lists the mechanical properties of rebar used for the web wall and flange wall. Tensile tests using type 2 test pieces specified by JIS Z 2201 were performed according to JIS Z 2241.

2.3 Monitoring
For the purpose of capturing the behavior of the web wall
caused by ASR, surface cracks, surface length changes, rebar strains, concrete strains, through-thickness elastic wave velocities, and surface hardness were periodically measured during exposure. The patterns of visually observed surface cracks were recorded and crack widths were measured with crack gauges. Change in 250 mm gauge length (X, Y) of contact points embedded on the surface of the web wall were measured as surface length changes. Rebar strains were measured by strain gauges attached on horizontal (X) and vertical (Y) rebars. Concrete strains in three directions (in-plane directions X, Y and through-thickness direction Z) were obtained only for the No. 3 and No. 4 specimens by concrete strain gauges (KM-100BT, Tokyo Measuring Instruments Laboratory Co., Ltd., with thermocouple built-in) that embedded in the web wall. Surface length change, rebar strain and concrete strain were temperature corrected by the ambient temperature, the average value of 4 thermocouples embedded in the web wall and the value of thermocouples built-in the concrete strain gauges, respectively. Elastic wave velocities in the through-thickness direction and surface hardness were measured by the ultrasonic method (NDIS 2009) and ultrasonic contact impedance method (Inaba et al. 2015), respectively. Core specimens with 75 mm diameter were collected based on JIS A1107 from the web walls of specimen Nos. 3 and 4. After cutting a length of approx. 20 mm from both ends, compressive strength and static elastic modulus of core concrete expanding under the restrained condition were obtained according to JIS A1107 and JIS A1149, respectively. The arrangement of the monitoring instruments mentioned above is shown in Fig. 5.

| Type of steel | Position | $f_y$ (MPa) | $f_u$ (MPa) | $E_s$ (GPa) | $\varepsilon_y$ (μ) | $\varepsilon_u$ (%) |
|--------------|----------|-------------|-------------|-------------|-----------------|-----------------|
| D13 | SD295 | Web wall | 342.8 | 488.1 | 193.4 | 1832 | 27.4 |
| D13 | SD295 | Flange wall (horizontal rebar) | 350.0 | 513.4 | 186.8 | 2045 | 28.8 |
| D16 | SD390 | Flange wall (vertical rebar) | 472.6 | 647.7 | 201.6 | 2438 | 22.8 |

*: The values are average results of 3 specimens.

| No. 3 specimen | No. 4 specimen |
|----------------|----------------|
| Web wall: 9 | Web wall: 9 |
| Flange wall: 4 | Flange wall: 4 |
| Upper/lower slab: 4 | Upper/lower slab: 4 |
| Thermocouple: 4 | Thermocouple: 4 |
| Contact gauge plug (XY): 6 | Concrete strain gauges (XY): 4 |
| Concrete strain gauges (Z): 6 | Concrete strain gauges (Z): 6 |
| Elastic wave velocity: 8 | Elastic wave velocity: 8 |
| Cored concrete (φ75×150): 8 | Cored concrete (φ75×150): 8 |

*: No. 1 and No. 2 specimens have same arrangement except cored concrete, which is obtained from only No. 3 and No. 4 specimens.

Fig. 5 Arrangement of monitoring instruments during exposure.
In addition, cylindrical specimens of ASR concrete with 100 mm diameter and 200 mm height that were prepared at the same time as web concrete casting were exposed wrapped in wet cloth under the same condition as the web wall (outside ambient temperature). Compressive strength, static elastic modulus and tensile strength were periodically obtained according to JIS A1107, JIS A1149 and JIS A113, respectively. Poisson's ratio was also obtained as the ratio of the transverse strain to the longitudinal strain at 1/3 of the maximum strength in the stress-strain curve. Strains for the static modulus and Poisson's ratio were measured using strain gauges (PL-60 and PL-90, Tokyo Measuring Instruments Laboratory Co., Ltd.). Surface length change was measured as change in 100 mm gauge length of contact points on rings wrapped around the surface of cylindrical specimen. Elastic wave velocity was also measured in the similar manner to the web wall.

2.4 Lateral loading tests

After one-year exposure, the No. 1 and No. 2 specimens were subjected to lateral loading tests. The lower slab of the specimen was tightly fixed to the reaction floor using 20 PC steel rods. The lateral force was cyclically applied to the upper slab of specimen by four 1000 kN hydraulic jacks while applying a constant vertical force. The vertical force was applied through the steel frame by pulling the four PC steel rods using 500 kN center hole type hydraulic jacks in order to apply constant stress of 2 MPa evenly over the entire cross-sectional area of the web wall. The set-up of the loading apparatus is shown in Fig. 6. The loading history was controlled by the shear deformation angle \( \gamma \) (see Appendix for calculation method) of the web wall according to Fig. 7.

During the lateral loading tests, applied force, horizontal and vertical displacement, and rebar strain were measured. The surface cracks were also recorded at the peak and at the time of unloading of each loading cycle. The arrangements of strain gauges and contact type displacement meters are shown in Figs. 5 and 8, respectively.

3. Experimental results

3.1 Monitoring

The property changes of cylindrical specimens using same ASR concrete of the No. 3 and No. 4 specimens are shown in Figs. 9 to 12. As shown in Fig. 9, expansion reached approx. \( 2.78 \times 10^{-5} \) until 18 months of exposure. The compressive strength of the cylindrical specimens...
remained unchanged from the 91 days value, whereas the static elastic modulus and elastic wave velocity decreased with exposure time, as shown in Fig. 10. Poisson's ratio decreased in a similar manner to the static elastic modulus and elastic wave velocity (see Fig. 11). Strain at compressive strength increased with exposure time (see Fig. 12). These values represent the properties of ASR concrete under free constraint.

The surface crack patterns of specimen Nos. 3 and 4 at 18-month exposure are shown in Fig. 13. Cracks in the web wall were observed after 6-month exposure for the No. 3 specimen (rebar constraint only) and 12-month exposure for the No. 4 specimen (constraint from rebar and surrounding members). Maximum crack width, approx. 0.1 mm, was similar in both specimens, but the number of cracks was more pronounced for the No. 3 specimen than the No. 4 specimen. More cracks observed in upper part may be caused by the fact that water was poured from only upper end of wet pad. In the No. 4 specimen, clearly visible cracks with maximum width of approx. 0.4 mm were observed in the flange walls that restrained web wall expansion. There were no cracks on the upper/lower slabs of the No. 4 specimen. In the previous study conducted by Hayes et al. (2018b) in larger scale, no cracking was observed on the surface of the confined specimen after one-year exposure. Visual inspection might not be a worthy method to detect signs of ASR development for actual concrete members of substantial thickness, but in reality, ASR expansion through thickness may not be uniform since the exposed surface is normally somewhat dry. This might produce expansion gradient resulting in surface cracking.

The average values in each direction of surface length, rebar strains, and concrete strains of the web wall against exposure time are shown in Figs. 14 to 16. The figures gradually increased with exposure time similar to the constraint-free cylindrical specimens, but the tendency in the in-plane direction is different depending on the principle direction and constraint conditions. Greater constraint results in a smaller value, namely expansion constrained by rebar only was larger than expansion con-

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Fig. 7 Loading history.

For overall deformation

For curvature calculation

Fig. 8 Layout of contact type displacement meters.

Fig. 9 Expansion of cylindrical specimen of ASR concrete for No. 3 and No. 4.

Fig. 10 Change in mechanical properties and elastic wave velocity of cylindrical specimen of ASR concrete for No. 3 and No. 4.
strained by rebar and surrounding members, and horizontal direction expansion constrained by massive upper/lower slabs was smaller than vertical direction expansion constrained by relatively small flange walls. In-plane expansion in the center part is smaller than surface, which might imply that distribution of ASR expansion through thickness was not uniform even though the surface was maintained high humidity. On the other hand, the expansion in the thickness direction, in which confinement was almost free, was more pronounced than in the other directions. The difference was greater under severe constraint condition, i.e., in the No. 4 specimen. This is the well-known anisotropic ASR expansion stemming from confinement conditions, such as boundary conditions and rebar arrangement. As shown in Fig. 17, the volumetric expansion of the two specimens was nearly identical, showing the ASR induced volumetric expansion to be independent of confinement conditions when one direction is free from confinement,
as mentioned elsewhere (Multon and Toutlemonde 2006; Gautum et al. 2017; Hayes et al. 2018). The strains observed in the flange walls and upper/lower slabs were small, $555 \times 10^{-6}$ to $1129 \times 10^{-6}$ and $93 \text{ to } 166 \times 10^{-6}$, respectively.

Figures 18 and 19 show changes in the average elastic wave velocity through thickness of the web wall and in the hardness of the web wall surface, respectively. The elastic wave velocities remained constant until 6-month exposure, rapidly decreased at 12 months, and then remained constant until 18 months. This trend is similar to that of the static elastic modulus and elastic wave velocity of cylindrical specimens shown in Fig. 10. This implies that changes in the static elastic modulus of web wall concrete can be monitored by non-destructive techniques such as the ultrasonic method. The hardness of the web wall surface gradually increased until 6 months and remained constant after that. This is a similar trend to that of the compressive strength of cylindrical specimens (see Fig. 10). However, this hardness is the property of mortar and not concrete, so direct comparison of the results is not meaningful.

The results of compressive strength tests of core con-

![Graph](image1)

![Graph](image2)

![Graph](image3)

![Graph](image4)
Concrete extracted from the web wall are shown in Fig. 20 together with the results of the constraint free cylindrical specimens. The elastic wave velocity results are also plotted. All the data of the core concrete constrained during exposure show smaller values than the constraint free concrete. As shown in Fig. 21, cracks of widths ranging between 0.08 and 1.2 mm in parallel to the in-plane direction of the wall were observed in the cover concrete, but the inside concrete between rebars was intact.

All trends mentioned above were also observed in the No. 1 and No. 2 specimens.

3.2 Lateral loading tests

Figure 22 shows the expansion of constraint free cylindrical specimens using the same ASR concrete of the No. 1 and No. 2 specimens subjected to loading tests. The expansion reached approx. $2240 \times 10^{-6}$ at 12 months and declined to approx. $2100 \times 10^{-6}$ at the time of the loading tests due to dry shrinkage under air-drying condition. The development of compressive strength and static elastic modulus (average of three cylindrical specimens) of ASR concrete used in the No. 1 and No. 2 specimens is shown in Fig. 23, and the typical compressive stress-strain re-

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**Fig. 17 Volumetric expansion of the web wall concrete.**

**Fig. 18 Elastic wave velocities through thickness of web wall.**

**Fig. 19 Surface hardness of web wall.**

**Fig. 20 Properties of core concrete.**

**Fig. 21 Side surface of cored concrete (12 months).**
The relationships of ASR concrete after 6 and 12 months of exposure are shown in Fig. 24 together with the results of ASR free concrete. The difference between ASR concrete and ASR free concrete stems from different aggregate and mix proportions in addition to the ASR effect.

The relationship between shear force and deformation angles (total angle and shear angle) is shown in Fig. 25. For all specimens, bending cracks in the flange walls occurred first, and then shear cracks and rebar yielding occurred in sequence, and the strength of the wall reached the maximum value. Finally, shear cracks reached the flange walls, spalling of the cover concrete along the shear cracks occurred, and final failure was reached.
Crack patterns at the shear deformation angle $\gamma=4/1000$ rad are depicted in Fig. 26, together with the final failure state. Cracks generated by applying lateral force (blue and red lines in Fig. 26) show similar patterns irrespective of ASR and boundary conditions. Figures 27 and 28 show the development of the maximum width and the number of cracks across the middle height of the web wall. Before the application of lateral force, the ASR specimens (Nos. 1 and 2) had initial cracks (black lines in Fig. 26), which increased under air-drying condition. The maximum width of initial cracks with rebar constraint only (No. 1) was about twice that with constraint from rebar and surrounding members (No. 2). The number of initial cracks of ASR specimens was almost the same, approximately 15, regardless of boundary conditions. The angle of initial cracks was also less difference. After the cycle of $\gamma=2/1000$ rad, the crack width at the time of unloading, which corresponds to the residual crack width after an earthquake, was almost the same regardless of ASR and boundary conditions, although there was a difference in the crack width at the peak of loading. The increase in the number of cracks with the increase in the shear deformation angle was almost the same in all specimens.

The development of average rebar strain of the web wall with the increase of loading cycle is shown in Fig. 29. Before lateral loading, tensile strain was already generated in the ASR specimens due to ASR expansion. The values in the specimen constrained by rebar only was nearly double that in the specimen constrained by

Fig. 26 Crack patterns at the shear deformation angle 4/1000 rad (Left) and final failure (Right).
rebar and surrounding members. After lateral loading start, the average rebar strains gradually converged and became almost the same regardless of the ASR and boundary conditions, which implies that internal stress generated by ASR expansion decreased with the increase of damage of the web wall.

The comparison of the equivalent linear shear modulus between experimental values Ge and calculated values Gc is shown in Fig. 30. Ge was determined by the method of least squares using the data to the point when shear cracks occurred on the web wall. The point of shear cracks occurrence was visually confirmed, so it is possible that judgement was a little late for the ASR specimens since it was difficult to determine the occurrence of shear cracks due to the presence of the initial cracks. Gc was calculated using the static elastic modulus and Poisson's ratio obtained by the compressive strength test of the cylindrical concrete specimen. As shown in Fig. 30, Ge and Gc are almost identical, which shows that the equivalent linear shear modulus of walls can be estimated using the static elastic modulus and Poisson's ratio of the concrete even for ASR affected walls.

The envelope curves of the shear force-shear deformation angle relationship are shown in Fig. 31. Although
initial stiffness of ASR concrete specimens (Nos. 1 and 2) was slightly smaller than the ASR free concrete specimen (No. 0), as shown in Fig. 28. The ASR specimens show greater strength until they reach ultimate strength after the occurrence of shear cracks, even though the compressive strength of ASR free concrete was larger than that of ASR concrete, as shown in Fig. 22. This tendency is somewhat pronounced under the severe constraint condition (No. 2), but ultimate strength was almost the same regardless of ASR and boundary conditions. The shear deformation angle at maximum strength tended to be slightly smaller the stronger the restraint.

Figure 33 shows the development of equivalent damping factor, $h_{eq}$, which was calculated according to Eq. (1) (see Fig. 32).

$$h_{eq} = \frac{1}{4\pi} \cdot \Delta W / W_e$$  \hspace{1cm} (1)

where, $\Delta W$ is the area of one cycle of the history loop and $W_e$ is the equivalent potential energy.

As can be seen in Fig. 33, $h_{eq}$ of ASR specimens (Nos. 1 and 2) is larger than ASR free specimen (No. 0), which implies energy absorption capacity of ASR specimen

Fig. 30 Comparisons of initial shear modulus.

Fig. 31 Comparisons of envelope curves of shear force-shear deformation angle relationships.
Increases. The increase of energy absorption capacity maybe be caused by fracture area of the web wall of ASR specimens became wider than ASR free specimen, which can be seen in Fig. 26. This wider fracture area of ASR specimen can be seen in the results of simulation analyses shown in a later section (see Fig. 44).

4. Analyses

When ASR occurs in real concrete structures, it may be necessary to perform the structural performance evaluation of ASR affected concrete members by numerical analysis, according to the extent of ASR expansion. The analytical evaluation procedure considered in this paper is shown in Fig. 34. This procedure postulates that the influences of ASR can be simulated by considering two factors, i.e., initial expansion strain applied to concrete caused by ASR, and mechanical properties of concrete affected by ASR. First, analyses considering ASR induced expansion only are conducted, and subsequently analyses with external loading are conducted. Then the structural performance of concrete members is evaluated to compare the results on strength and deformation with safety criteria.

4.1 Analytical models

The non-linear FE analysis code “CARC-ASe” developed by Kajima Corporation was used in this analytical study. This code incorporates the multi directional crack model developed by Hauke and Maekawa (1999). The stress-strain relationship of concrete is shown in Fig. 35. The Fafitis-Shah model (Fafitis and Shah 1985) shown in

Fig. 32 Calculation of the equivalent damping factor.

Fig. 33 Comparisons of equivalent damping factors.

Fig. 34 Analytical evaluation procedures.
Eq. (2) was used on the compression side. For the softening characteristics after compressive strength, the fracture energy model by Nakamura and Higai (2001) shown in Eq. (3) was applied, and compressive softening coefficient \( k \) was set to be close to the fracture energy \( F_c G_c \) per element dimension \( L \). The Collins model (Vecchio and Collins 1982) shown in Eq. (4) was used for the compression strength reduction coefficient \( \beta \) due to cracks in the orthogonal direction. The tension stiffening model (Maekawa and Okamura 1991) shown in Eq. (5) was used for the tension side, and the softening coefficient \( C \) after the tensile strength was set to 0.4 because the wall was sufficiently reinforced with rebars. For the shear characteristics after cracking, the Habasaki model (Habasaki et al. 2000) was used. The mechanical properties of cylindrical specimens of both ASR concrete (at 12 months exposure) and ASR free concrete are shown in Table 5. Only these values included the ASR effect in this model, but other values indicated in Eqs. (2) to (5) were used the same irrespective of ASR since information on the ASR effect is very limited. In addition, the anisotropic effect of ASR expansion was not considered in this model.

\[
\begin{align*}
\sigma = \beta \cdot f_c \cdot \left\{ 1 - \left( \varepsilon_c / \varepsilon_u \right)^{\alpha} \right\} & : \text{until compressive strength} \\
\sigma = \beta \cdot f_c \cdot \exp \left( -k \cdot \left( \varepsilon_c / \varepsilon_u \right)^{1.5} \right) & : \text{after compressive strength}
\end{align*}
\]

\[ G_{fc} = 8.8 \sqrt{F_c} \]

where, \( \alpha = E \left( f_c / \varepsilon_u \right) \), \( \sigma_c \) is compressive stress (MPa), \( \varepsilon_c \) is compressive strain, \( f_c \) is compressive strength (MPa), \( \varepsilon_u \) is compressive strain at compressive strength, \( E \) is static elastic modulus (MPa), \( k \) is compressive softening coefficient, \( \beta \) is compression strength reduction coefficient due to cracks, \( \varepsilon_c \) is maximum tensile strain in the orthogonal direction, \( \sigma_t \) is tensile stress, \( \varepsilon_t \) is tensile strain, \( f_t \) is tensile strength (MPa), \( \varepsilon_{t0} \) is tensile strain at tensile strength, and \( C \) is softening coefficient.

As shown in Fig. 36, the Zulfiqar-Filippou model (Zulfiqar and Filippou 1990) considering the Bauschinger effect was used for the stress-strain relationship of the rebar. The yield strength and elastic modulus of the reinforcement bar were determined from the results of the tensile tests, and the second stiffness was set to 1/1000 of the initial stiffness. The upper and lower slabs were modeled as elastic bodies.

The analytical model is shown in Fig. 37. Considering a symmetric condition, the half model in the thickness direction of the web wall was used for No. 1, No. 2 and No. 4 specimens, 1/8 model was used for No. 3 specimen. Concrete was modeled with an eight-node solid element and divided into 8 in the thickness direction (4 in half model). A rebar was modeled with a rod element and rigidly connected to the concrete element.
Initial expansion strain caused by ASR applied to the concrete elements was considered from two different approaches. One is the strain of the cylindrical specimen exposed under the constraint free condition, and the other is the strain equivalent to the rebar strain of the wall specimen exposed under constraint conditions. In the latter case, the strain applied to concrete elements was increased step by step and the analysis was stopped when the strain of rebar reached the experimentally measured value. The boundary conditions and loading conditions are shown in Figs. 38 and 39, respectively. For No. 1 and No. 3 specimens, in which web concrete expansion was restricted by rebar only, the different boundary conditions during exposure are applied according to the subsequent loading condition. In the case modelling only a web wall without loading (No. 3), expansion is simply applied to the concrete elements of the web wall fixed with one
center point. In the case with loading after expansion (No. 1), however, all members are modelled, and the concrete elements of all members, i.e., web wall, flange walls, and upper/lower slab, are expanded with lower slab bottom fixed in the vertical direction (see Fig. 38, No. 1) in order to eliminate influence of surrounding members to web wall expansion. For No. 2 and No. 4 specimens, in which web concrete expansion was restricted by rebar and surrounding members, all members are modelled, and expansion is applied to the concrete elements of the web wall only under the condition of lower slab bottom fixed (see Fig. 38, Nos. 2 and 4). In the case simulating the loading test, after the application of axial force, static loading by displacement control at both ends of the upper slab was applied with the lower slab bottom fixed (see Fig. 39).

### 4.2 Analyses considering ASR induced expansion only

Table 6 shows deformation and cracks induced by ASR expansion at 12 months exposure and compares the differences between initial expansion strain considerations. In the case of rebar constraint only, almost no cracks occurred on the surface of the web wall but cracks around rebar were observed. No cracks on the surface is different from the experimental results and may be caused by uniform strain applied to the concrete elements in the analyses, even though strain induced by ASR is not uniform in the experiments as mentioned in Section 3.1. In the case of rebar plus surrounding members constraint, cracks on the web wall occurred around the boundary only. The extent of damage in the flange walls differs depending on the method of application of initial strain, with the case applied free expansion showing more significant damage than the case applied strain equivalent to the rebar strain. The latter more closely approximates the experimental results.

Concrete strains between numerical and experimental results are compared in Fig. 40. For both the No. 3 and No. 4 specimens, in-plane concrete strains in the case applied free expansion are larger than the experimental

|          | Free expansion | Strain equivalent to the rebar strain |
|----------|----------------|--------------------------------------|
| No. 3    |                |                                      |
| No. 4    |                |                                      |

Fig. 39 Boundary conditions during loading.
results. In the case applied estimated expansion, the analytical results come close to the experimental values. Concrete strains in the thickness direction are smaller than the experiments since these analyses did not consider anisotropic ASR expansion.

Figure 41 shows the relationship between the ratio of observed concrete strain to free expansion strain obtained from cylindrical specimens and internal compressive stress induced by ASR expansion. This in-plane internal stress was obtained as average values of the numerical results at the positions corresponding to the concrete gauges. The strain ratio, in which the creep effect is presumably included, decreases proportionally with the increase of internal compressive stress and shows good agreement with previous studies (Clayton et al. 1990; Larive 1997; Berra et al. 2010).

4.3 Analyses considering ASR induced expansion and loading

The shear force-deformation angle relationships of the experimental and numerical results in the reference case are compared in Fig. 42. For the reference case, both cyclic and monotonic analyses were conducted. As can be seen, both numerical results show good agreement with the experimental results. The failure mode in analysis also shows a similar tendency of the experiment, in which the ultimate state was reached by shear failure of the web wall, as shown in Fig. 43. Since both cyclic and monotonic results show a good co-relation to the experiment, the remaining analyses were conducted using only monotonic loading.

Figure 44 compares the shear force-deformation angle relationships of the experimental and numerical results of ASR specimens No. 1 and No. 2. The failure states around maximum strength in the analyses are also depicted in same figure. The numerical results of No. 1 show a somewhat lower trend than the experimental results, but it can be said that both results show almost good agreement with the experimental results until maximum strength. The numerical results show larger deformation at maximum strength and gentler descending curve than the experimental results in both cases. The failure mode is similar to the experiments in which the ultimate state was reached with progress of the shear failure of the web wall. Although the analysis did not consider anisotropic ASR expansion, it can reasonably simulate the experiments. This may also indicate that the anisotropic effect may not influence in-plane shear performance of the wall as long as the range of expansion is under approximately 3000×10^{-6}.

5. Discussion

5.1 Influence of ASR on in-plane shear performance

Since the materials used in the concrete of reference specimen No. 0 was different from the ASR concrete of the No. 1 and No. 2 specimens, direct comparison of the load-deformation relationships mentioned in Section 3.2.
does not purely reflect the solo influence of ASR. The load-deformation relationship of the ASR free specimens can be evaluated by numerical analysis using the mechanical properties of ASR concrete of the No. 1 and No. 2 specimens without expansion. As shown in Figs. 22 and 23, at 6 months of exposure, expansion was not significant and mechanical properties were not degraded yet. The analysis using the mechanical properties of ASR concrete at 6 months of exposure was conducted, and the results of load-deformation (referred as No. 0') were compared with ASR specimens No. 1 and No. 2 in Fig. 45. The initial stiffness in the ASR case is lower but the strength after the occurrence of shear cracks remains higher in the case of sever constraint until ultimate strength is reached. The ultimate strength of all cases is nearly identical, irrespective of ASR.

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Fig. 43 Numerical results of failure mode of reference case No. 0 (at shear deformation angle of 6/1000 rad).

Fig. 44 Numerical results of shear force-deformation angle relationship and failure modes (at shear deformation angle of 6/1000 rad) of ASR concrete specimens.
5.2 Structural performance evaluation

Once degradation due to ASR is observed, performance evaluation of the structure should be conducted. One of the methods is to perform numerical analysis of members affected by ASR. Judging from the results of the analytical study mentioned in Section 4, in-plane structural performance of ASR affected walls can be generally evaluated by considering mechanical properties changes of concrete and expansion strain due to ASR, if the degree of expansion is no greater than approximately $3000 \times 10^{-6}$.

A challenge in conducting analysis for in-situ real ASR affected walls is how to obtain two key factors, namely the mechanical properties of ASR affected concrete and the value of ASR expansion to be applied to the model. A traditional method to obtain the mechanical properties of concrete consists in extracting a core of concrete from the wall and conducting compressive strength tests. This method, however, cannot fully acquire the real properties of wall concrete since internal stress is released during coring. As can be seen in Fig. 46, the elastic wave velocities of the core concrete are smaller than those of the wall at the same position but before coring. The static elastic modulus has a co-relationship with the elastic wave velocity as shown in Fig. 47, so the static elastic modulus after coring of concrete affected by ASR might be smaller. Using the properties of cores is not appropriate for precisely predicting the structural performance of ASR affected walls, but it is assumed that performance may be estimated with a reasonable margin with respect to safety criteria. The other factor, ASR expansion, is more difficult to estimate from the state of in-situ real walls. Surface length change is one of the relevant information items, but it does not represent the concrete strain as shown in the left-hand side of Fig. 48. In addition, the surface length change from the beginning of ASR expansion cannot be obtained since the values can be only measured after a certain degree expansion has occurred. On the other hand, rebar strain has a co-relationship with concrete strain (see the right-hand side of Fig. 48), and the values affected by ASR expansion can be obtained by cutting of rebars on which strain gauges are installed (Manabe et al. 2016). The rebar strains include also the creep effect according to the constraint conditions, as mentioned in Section 4.2.

In summary, numerical analysis using core properties of ASR affected concrete and rebar strains as ASR expansion with the effect of constraint can produce practically appropriate results for structural performance evaluation.

5.3 Monitoring

After safety evaluation of ASR affected structural members, monitoring of the state of ASR should be conducted. As mentioned in the previous section, the key factors for safety evaluation are mechanical property changes of concrete and expansion strain due to ASR. Mechanical properties can be obtained from cored concrete, but this method is destructive and periodical use is not ideal from the viewpoint of minimization of structural damage (AIJ 2021). As mentioned in Section 2.3, elastic wave velocity of wall concrete changes with the progress of ASR expansion. Figure 47 shows the relationship between the elastic wave velocity and the static elastic modulus of cylindrical specimens and cored concrete. These two properties have a good relationship, so monitoring of elastic wave velocity may capture the
trend of mechanical property changes of ASR affected concrete. The other key factor, ASR expansion including constraint effect, can be estimated by measuring rebar strain. The strain gauges installed for obtaining initial rebar strain can be continuously used for monitoring, allowing acquisition of the trend of strain progression as demonstrated by Manabe et al. (2016). It is also beneficial to directly measure internal ASR induced concrete stress, which is alternative parameter for safety evaluation similar to rebar strain. This can be monitored by stress meter as suggested by Saouma (Saouma and Hariri-Ardebili 2014). Other methods such as surface crack observation and surface length change can be also applied to monitoring. Even though these methods do not represent internal ASR expansion, they may be useful for yielding supplemental data to rebar strain and/or internal stress.

6. Conclusions

The main conclusions of this investigation are summarized below.

(1) Due to the expansion caused by ASR, many small cracks were generated on the surface of the wall with rebar restraint only. A small number of cracks were also observed on the walls with greater restraint.

(2) Surface length change, rebar strain and concrete strain (in-plane direction) showed smaller expansion than free expansion. The expansion was suppressed more than the restraining effect of the rebar and the surrounding members, so the cause might be creep by the action of the internal compressive stress.

(3) The thickness direction strain of concrete showed remarkable expansion exceeding the free expansion strain, and anisotropic expansion in which expansion was predominant in the unconstrained direction was confirmed.

(4) The transition of the elastic wave velocity of the wall by the ultrasonic method has a good co-relationship with the transition of the static elastic modulus of the cylindrical specimens, indicating that non-destructive testing could capture the degradation trend of concrete due to ASR expansion.

(5) The compressive strength, static elastic modulus, and elastic wave velocity of the extracted core specimens were smaller than those of the cylindrical specimens, suggesting that the mechanical properties of the extracted core may decrease due to stress release.

(6) Although the ASR specimens had initial cracks, the progression of residual crack width and the increase in the number of residual cracks with the increase in the shear deformation angle during loading test were about the same as in the case without ASR.

(7) In the case of the ASR specimens, initial tensile strain of the rebar was generated to resist concrete expansion, which depends on the degree of restraint. The difference from the case without ASR gradually narrowed as the shear deformation angle increased and almost disappeared at the maximum strength regardless of the presence or absence of ASR and the degree of restraint.

(8) When the ASR expansion was within the range concerned (approximate 3000×10^-6 in free expansion) in this experiments and numerical analyses, the initial stiffness of the ASR specimens became slightly smaller than that of the ASR free specimens, but the maximum strength remained at the same level with or without ASR.

(9) The structural performance of ASR affected concrete members may be somewhat reasonably evaluated in a practical manner by numerical analysis using the core properties of ASR affected concrete and using rebar strains as ASR expansion with the effect of constraint.

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References

AIJ, (2021). “AIJ guidelines for maintenance and management of structures in nuclear facilities.” Tokyo: Architectural Institute of Japan.

Berra, M. Faggiani, G., Mangialardi, T. and Paolini, A. E., (2010). “Influence of stress restraint on the expansive behaviour of concrete affected by alkali-silica reaction.” Cement and Concrete Research, 40, 1403-1409.

Clayton, N., Cumie, R. and Moss, R. N., (1990). “The effects of alkali-silica reaction on the strength of prestressed concrete beams.” The Structural Engineer, 68(15), 287-292.

Chenier, J.-O., Komjendovic, D., Gocevski, V., Picard, S. and Chretien, G., (2012). “An approach regarding aging management program for concrete containment structure at the Gentilly-2 Nuclear Power Plant.” In: Proc. 33rd Annual Conference of the Canadian Nuclear Society and 36th Annual CNS-CNA Student Conference, Saskatoon, Saskatchewan, Canada 10-13 June 2012. New York: Curran Associates, Inc., Vol. 1, 260-272.

Fafitis, A. and Shah, S. P., (1985). “Lateral reinforcement for high-strength concrete columns.” In: ACI Special Publication SP-87: High-Strength Concrete. Farmington Hills, Michigan, USA: American Concrete Institute, 213-232.

Ferche, A. C., Gautam, B., Habibi, F., Panesar, D. K., Sheikh, S. A., Vecchio, F. J. and Orbovic, N., (2019). “Material, structural and modelling aspects of alkali aggregate reaction in concrete.” Nuclear Engineering and Design, 351, 87-93.

Gautam, B., Panesar, D., Sheikh, D. and Vecchio, F. (2017). “Effect of multi-axial stresses on alkali-silica reaction damage of concrete.” ACI Materials Journal, 114-M52, 595-604.

Habasaki, A., Kitada, Y., Yamada, M. and Nishikawa, T., (2000). “Constitutive model of shear transfer for pre-cracked RC plate subjected to combined axial and shear stress.” Journal of Structural and Construction Engineering (Transactions of AIJ), 65(538), 139-145. (in Japanese)

Habibi, F., Sheikh, S. A., Orbovic, N., Panesar, D. K. and Vecchio, F. J., (2015). “Alkali aggregate reaction in nuclear concrete structures: Part 3: Structural shear wall elements.” In: Proc. 25th International Conference on Structural Mechanics in Reactor Technology (SMiRT25), Manchester, UK 10-14 August 2015. New York: Curran Associates, Inc., Vol. 1, 182-191.

Hauke, B. and Maekawa, K., (1999). “Three-dimensional modelling of reinforced concrete with multi-directional cracking.” Doboku Gakkai Ronbunshu (Journal of Materials, Concrete Structures and Pavements, JSCE), 634(45), 349-368.

Hayes, N., Le Pape, S., Ma, Z. J. and Le Pape, Y., (2018a). “Identification of mechanisms to study alkali-silica reaction effects on stress-confined concrete nuclear thick structures: Interpretation of the complete monitoring data and nondestructive evaluation of the alkali-silica reaction test assembly (Report No. ORNL/SPR-2018/965).” Washington DC: Office of Nuclear Energy, US Department of Energy.

Hayes, N. W., Gui, Q., Abd-Ellsamed, A., Le Pape, Y., Giorla, A. B., Le Pape, S., Giannini, E. R. and Ma, Z. J., (2018b). “Monitoring alkali-silica reaction significance in nuclear concrete structural members.” Journal of Advanced Concrete Technology, 16(4), 179-190.

Inaba, Y., Ichinose, T. and Kanda, T., (2015). “Basic study on estimating strength of concrete by UCI method.” Journal of Structural and Construction Engineering (Transactions of AIJ), 80(710), 519-526. (in Japanese)

IRSN, (2016). “Annual report of the Institute for Radiological Protection and Nuclear Safety.” Fontenay-aux-Roses, France: Institut de Radioprotection et de Sûreté Nucléaire (IRSN).

Kawabata, Y., Yamada, K., Sagawa, Y. and Ogawa, S., (2018). “Alkali-wrapped concrete prism test (AW-CPT) - New testing protocol toward a performance test against alkali-silica reaction.” Journal of Advanced Concrete Technology, 16(9), 441-460.

Larive, C., (1997). “Combined contributions of the alkali-silica reaction and its mechanical effects.” Thesis (PhD). École Nationale des Ponts et Chaussées. (in French)

Maekawa, K. and Okamura, H., (1991). “Non-linear analysis and constitutive models of reinforced concrete.” Tokyo: Gihodo-Shuppan Co.

Manabe, R., Kawae, H., Ogawa, K. and Matsuura, M., (2016). “Maintenance management of turbine generator foundation affected by alkali silica reaction.” Journal of Advanced Concrete Technology, 14(9), 590-606.

Multon, S. and Toutlemonde, F., (2006). “Effect of applied stresses on alkali-silica reaction-induced expansions.” Cement and Concrete Research, 36, 912-920.

Murazumi, Y., Watanabe, Y., Matsumoto, M., Mitsugi, S., Takiguchi, K. and Masuda, Y., (2005). “Study on the influence of alkali-silica reaction on structural behavior of reinforced concrete members.” In: Proc. 18th International Conference on Structural Mechanics in Reactor Technology (SMiRT18), Beijing 7-12 August 2005. New York: Curran Associates, Inc., Vol. 3, 2036-2042.

Nakamura, H. and Higai, T., (2001). “Compressive fracture energy and fracture zone length of concrete.” In: Modeling of Inelastic Behavior of RC Structures under Seismic Loads. Reston, Virginia, USA: American Society of Civil Engineers, 471-487.

NDIS, (2009). “Non-destructive testing of concrete Part 1: Ultrasonic method (NDIS Standard 2429-1).” Tokyo: The Japanese Society for Non-destructive
The shear deformation angle $\gamma$ of the wall is calculated by dividing the shear deformation $\delta_\gamma$ by the internal normal height of the web wall $h_0$. $\delta_\gamma$ is the measured horizontal deformation $\delta_h$ minus the bending deformation $\delta_f$ determined by the curvature.

$$\gamma = \frac{\delta_\gamma}{h_0}$$

$$\delta_\gamma = \delta_h - \delta_f$$

$$\delta_f = \sum_{i=1}^{n} h_i (\phi_i \cdot \Delta X_i)$$

where, $\gamma$: Shear deformation angle, $h_0$: Height of web wall, $\delta_\gamma$: Horizontal deformation, $\delta_f$: Shear deformation, $\delta_f$: Bending deformation, $h_i$: Distance from the center of the measurement section to the bottom of the force stub, $\phi_i$: Average section curvature, $\Delta X_i$: Section length, $X_L$: Elongation of the left side of the section, $X_R$: Elongation of the right side of the section and $L_0$: Distance between left and right measuring points.