Technical report

Benchmark Finite Element Calculations for ASCET Phase III on a Reinforced-Concrete Shear Wall Affected by Alkali-Aggregate Reaction

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Abstract

In this study, finite element (FE) analyses were conducted on a reinforced-concrete (RC) shear wall that is affected by an alkali–aggregate reaction (AAR), which were then applied for a benchmark studies in OECD/NEA/CNSI/ASCET (Organization for Economic Co-operation and Development/Nuclear Energy Agency/Committee on Safety of Nuclear Installations/Assessment of Structures subjected to Concrete Pathologies) Phases II and III assessments. A commercial software has been modified to account for this AAR expansion, which is affected by the stress field and change in physical properties of the concrete. The impacts of boundary conditions, modeling in two and three dimensions, and material properties on the load–displacement curve and crack patterns were carefully evaluated. Finally, although similar load–displacement curves and crack patterns were obtained, the peak load due to brittle failure of an RC shear wall affected by AAR could not be reproduced.

Consequently, it was found that the rotation of the loading stub and anchoring procedure of the base stub were critical conditions for load-displacement relationship of RC shear wall, and meshing capturing the arrangement of reinforcement bars is crucial for FE analysis with two-dimensional (2D) condition, and finally, the occurrence of initial cracks and the loading capacity could not be clearly reproduced. This suggests that consideration of the placement of rebars and covering concrete in the mash setting in three-dimensional (3D) model affected the failure mode of the concrete. It is necessary to consider the possible failure mechanism and to reflect such features in numerical modeling.

1. Introduction

An alkali-aggregate reaction (AAR) is a pathology of degraded concrete. It is a common issue in the concrete structures of nuclear facilities as well as in general civil infrastructure (Hayes et al. 2018; Kawabata et al. 2018a, 2018b; Takahashi et al. 2018; Pourbehi and van Zijl 2019; Li et al. 2020). In Japan, the reinforced-concrete (RC) turbine generator foundation of the Ikata nuclear power plant (NPP) Unit 1 of Shikoku Electric Power exhibits expansion as a result of AAR, as reported by Manabe et al. (2016). In Canada, the Gentilly-2 NPP, Hydro-Quebec, experienced AAR, and its impact on structural performance was investigated (Tcherner and Aziz 2009). In 2009, in the United States, although AAR degradation was observed in many of the concrete walls composing the Seabrook NPP, the US-NRC renewed the facility’s operating license in 2019 (U.S. NRC 2019).

Understanding the mechanism of AAR degradation as well as the structural performance of RC members affected by AAR is necessary for the aging management of NPPs. In response, the ASCET project was formed in the CSNI of the OECD/NEA. The objective of the ASCET activity is to “make general recommendations for aging management of concrete nuclear facilities taking into account the effect of concrete pathologies on structural degradation” (NEA CSNI 2019). The final ASCET Phase I report suggests that a reliable analytical tool considered AAR degradation is necessary to predict the behavior of concrete structures due to AAR degradation. ASCET Phase II was established based on the recommendation of the workshop held at the National Institute of Standards and Technology in Gaithersburg in 2015 as well as the Phase I report. During the ASCET Phase II workshop, it was identified that crack development under cyclic loading and the failure mode of the target RC shear walls showed large discrepancies between the benchmark loading experiment of RC shear walls with a concrete affected by AAR (hereafter, AAR concrete) and the numerical analysis results from many research teams (NEA CSNI 2019). Based on these results, ASCET Phase III was then established to elucidate the mechanism of the variation in crack development and failure modes.

This technical paper is based on a report by S/NRA/R for this ASCET Phase III (Kojima et al. 2018). This report describes in detail the ASCET Phase II and AS-
CET Phase III experimental results and benchmark calculations by the authors. First, a finite element (FE) program is briefly introduced, including an AAR concrete degradation model. Next, the impact of the applied boundary condition, that is, the fixation method of a base stub, which affects the load-displacement relationship of the RC shear walls, is discussed by using two-dimensional (2D) FEM. Then a comparison of 2D and three-dimensional (3D) calculations is presented and the modeling of reinforced concrete elements and resultant load-displacement curves are discussed. Finally, the mechanism of the brittle failure of the AAR concrete RC shear wall is discussed based on a parametric study in material property change due to AAR. Details of and the logic behind the modeling are explained systematically, as such detailed information is necessary to enhance the discussion.

2. Methods

2.1 Outline of the method of analysis

The benchmark simulations were conducted by different research teams in ASCET III, of which the authors of this paper also partook. RC shear concrete walls with regular concrete (sustaining no damage due to AAR) and concrete affected by AAR (AAR concrete, with accelerated AAR introduced by maintaining the relative humidity (RH) above 95% at 50°C) were considered. Cylindrical specimens, cubic specimens, RC blocks, and RC shear walls with regular concrete and AAR concrete were investigated (Gautam et al. 2015; Habibi et al. 2015; Jürrect et al. 2015; Orbovic et al. 2015; Sheikh 2017; NEA CSNI 2019). In the following sections, results obtained from these specimens are used to determine the properties of the materials studied.

For the benchmark calculation performed by the authors, a 2D shell element analysis and a 3D solid element analysis were performed using the commercial structural analysis program FINAS/STAR Version 2015 (ITOCHU 2015) developed by ITOCHU Techno-Solutions Corporation (CTC). The FE program contains a subroutine that is based on the non-orthogonal four-directional smeared crack model, which was originally proposed by Maekawa et al. (2001), for calculating nonlinear RC behavior. This model is beneficial to consider the fixed orthogonal cracks in concrete due to outer load in addition to the cracks produced by AAR reaction. To simulate the structural behavior of concrete affected by AAR, a set of functions that consider the degradation of concrete due to AAR was implemented for the first time. The model is based on findings of several previous studies (Pietruszczak 1996, 2011; Winnicki and Pietruszczak 2008; Goccevski 2016). Coupling the stress-induced anisotropic volume expansion and property changes of concrete with the four-directional smeared crack model permits the evaluation of the structural performance of an RC structure affected by AAR. The AAR model is detailed in the subsequent sections.

2.2 Models of AAR concrete

2.2.1 Model of volumetric expansion in saturation

The basic concept of the AAR expansion model has been well described in the literature (Pietruszczak 1996, 2011; Winnicki and Pietruszczak 2008; Goccevski 2016). The volume expansion strain under the unrestrained condition can be expressed by:

$$\varepsilon_{v}(t) = \varepsilon \left(1 - e^{-\gamma(t-t_0)}\right)$$  \hspace{1cm} (1)

where $\varepsilon_{v}(t)$ is the volumetric expansion strain at time $t$, $\varepsilon$ is the final volumetric expansion strain, $t_0$ is the expansion starting time, and $\gamma$ is a parameter related to the degree of progress of the expansion.

The degree of progress of the expansion strain $\zeta$ is defined as:

$$\zeta = \frac{\varepsilon_{v}(t) - \varepsilon_{v}(t_0)}{\varepsilon_{v}(t_f) - \varepsilon_{v}(t_0)}$$  \hspace{1cm} (2)

where $\zeta$ is a coefficient that considers the reduction in volumetric expansion strain due to the stress field, which affects the AAR gel propagation and resultant accumulated expansive stress of the AAR gels. It is given by:

$$\zeta = \exp \left(-A_1 \sqrt{\frac{-tr(\sigma)}{3f_c}}\right)$$  \hspace{1cm} (3)

where $A_1$ is a material-dependent constant and is used as an input parameter, $f_c$ is the uniaxial compressive strength of the concrete, and $\sigma$ is the stress tensor. For the unrestrained condition (free expansion), $\zeta = 1$.

Therefore, the volumetric expansion strain at time $t$ is expressed as:

$$\varepsilon(t) = \varepsilon \left(1 - e^{-\gamma(t-t_0)}\right)$$  \hspace{1cm} (4)

From Eq. (4), $\zeta$ represents the degree of progress of the expansion strain, as shown by:

$$\zeta(t,\sigma) = \frac{\varepsilon(t) - \varepsilon(t_0)}{\varepsilon(t_f) - \varepsilon(t_0)}$$  \hspace{1cm} (5)

The stress in compression (negative sign) only affects $\zeta$. For a 2D shell element, only $\sigma_x$, $\sigma_y$, and $\tau_{xy}$ are used, and $\sigma_z = 0$, $\tau_{xz} = 0$, and $\tau_{yz} = 0$ are assumed.

2.2.2 Model of the change in material properties

In general, the concrete degradation due to AAR is originated from the cracks in concrete due to expansion of water sorbed silica-gel. Crack openings and limited stress transfer passes reduce the strength and Young’s modulus of concrete. Based on this background, the expansion strain of concrete will give the first order approximation of crack opening and void creation in concrete due to the expansion of silica-gel. In other words, the expansion strain could be the index of the damage in concrete. The model by Winnicki and Pietruszczak is well reflected by this idea and easy to
understand conceptually, this model follows the concept proposed by Winnicki and Pietruszczak (2008). For the RC elements, the restraint due to the reinforcement bars (rebar) must be considered. Originally, \( \zeta \) is assumed to be proportional to the deterioration of the material properties (Winnicki and Pietruszczak 2008). However, in reality, the normalized strain \( \zeta' \), as a result of the force equilibrium and strain consistency in an RC member, represents the degradation. The normalized strain \( \zeta' \) as an index for the material property change is defined as:

\[
\zeta'(t) = \frac{\epsilon_1(t) + \epsilon_2(t) + \epsilon_3(t)}{\delta}
\]

where \( \epsilon_1, \epsilon_2, \) and \( \epsilon_3 \) are the total strains of the element or the Gaussian integral point.

Some may think that the strain itself could be the good indicator of material property change, but in this study, since the concrete properties affected by AAR under the restraint condition is not fully understand, it is assumed that the combination of parameter of \( \zeta'(t) \) and parameters \( G_1, G_2 \) can reflect the material property changes.

For the inputs of the non-orthogonal four-directional smeared crack model after the deterioration, parameters \( G_1, G_2 \) and \( \delta \) are introduced for the deterioration of Young’s modulus, the deterioration in the compressive strength, and the strain at the maximum compressive strength, respectively, as defined by:

\[
E(\zeta') = E_0(1 - G_1 \zeta')
\]

where \( E_0 \) (MPa) is Young’s modulus of concrete without AAR damage and \( E(\zeta') \) (MPa) is the Young’s modulus of concrete whose strain is \( \zeta' \).

\[
f_c(\zeta') = f_{c0}(1 - G_2 \zeta')
\]

where \( f_{c0} \) (MPa) is compressive strength of concrete without AAR damage and \( f_c(\zeta') \) (MPa) is the compressive strength of concrete whose strain is \( \zeta' \).

\[
e(\zeta') = \frac{2f_c(\zeta')}{E(\zeta') + \frac{2f_{c0}(1 - G_2 \zeta')}{E_0(1 - G_2 \zeta')}}
\]

These assumptions are applicable to cases having very high RHs, where the mortar or the cement paste do not show shrinkage, and the AAR gel shows expansion. When the RH is relatively high (e.g., 80% to 90% RH), the mortar demonstrates shrinkage while the AAR gels expand. Moreover, it is likely that \( \zeta' \) may underestimate the damage in concrete in such conditions.

### 2.2.3 Stress—expansion relationship

Once the ratio of the strain under the restrained condition or under a certain stress to the free strain of concrete expansion due to AAR is given, it is possible to calculate the incremental volumetric expansion of concrete. Figure 1 presents the results of the ratio of expansion strain to free strain of concrete due to AAR as a function of applied stress by outer frame, reinforcement bars, and outer loads. (Clayton et al. 1990; Léger et al. 1996; Larive 1997; Berra et al. 2010) with a comparison of the data used in this study. According to this figure, the reduction factor \( \beta \) is defined as the linear interpolation of the data from Clayton et al. (1990), which is used for the expansion strain of concrete due to AAR under stress.

As a first-order approximation, the increment of the expansion strains in the three principle stress directions has a particular relationship with the incremental volume expansion strain \( \Delta \epsilon \), which is described below. This incremental volume expansion strain due to AAR is affected by the stress tensor as well as the factor \( \Delta \epsilon \), and can be expressed as:

\[
\Delta \epsilon^\epsilon = \frac{\beta_\alpha \Delta \epsilon}{\beta_\alpha + \beta_\beta + \beta_\gamma} \Delta \epsilon
\]

From the above equation, the incremental strain due to AAR in each direction can then be defined by:

\[
\Delta \epsilon_\alpha = \frac{\beta_\alpha \Delta \epsilon}{\beta_\alpha + \beta_\beta + \beta_\gamma} \Delta \epsilon
\]

The factor \( \beta_\alpha \) in each direction is assumed to follow the trend in Fig. 1. This equation implies that \( \beta_\alpha \) acts as a weight for distributing the volumetric expansion in each direction of the principal stress according to the magnitude of the principal stress. When \( \beta_\alpha + \beta_\beta + \beta_\gamma = 0 \) (that is, when \( \beta_\alpha = 0, \beta_\beta = 0, \beta_\gamma = 0 \)), \( \Delta \epsilon^\epsilon = 0 \) (\( \alpha = 1, 2, 3 \)) is used.

The incremental tensor of the expansion strain from the AAR is given by:

\[
\Delta \epsilon_\alpha' = \Delta \epsilon_\alpha e_\alpha^{(\alpha)} e_\alpha^{(\alpha)}
\]

![Fig. 1 Ratio of strain to free strain as a function of restrained stress](image-url)
where $A_{ij}$ is the expansion strain incremental tensor from the AAR, $\alpha$ is the principal stress number ($\alpha = 1, 2, 3$), and $\varepsilon^{\alpha i}$ is the vector of the principal stress $\alpha$.

### 2.2.4 AAR reaction affected by temperature and relative humidity.

Heat and moisture transport are also considered in the FE analysis. In particular, the water content and the local equilibrium RH are significant for AAR expansion. Consequently, moisture transport model is necessary to consider the distribution of AAR expansion in the target RC member. In general, a large portion of the water within the concrete is located in its interlayer spaces and gel pores of hardened cement paste (Muller et al. 2013). It is entrapped in the calcium silicate hydrate (C-S-H), which is the main phase of Portland cement hydrates. Furthermore, the dynamic microstructural change of C-S-H plays an important role in the moisture transport and determines the rate of moisture transport for moderate RH (Maruyama et al. 2019). From this point of view, the water vapor in the pore structure could be a potential of moisture transport, and the apparent moisture diffusion coefficient of concrete can be evaluated according to this potential. This approach has been already studied by Maruyama and Igarashi (2015) and moisture diffusion coefficient is modeled as a function of the statistical thickness of water adsorption. In addition, the temperature-dependent sorption isotherm model for hardened cement paste is used, which is based on experimental data (Maruyama and Igarashi 2011; Igarashi and Maruyama 2012; Maruyama and Rymeš 2019). Based on this background, since the model is easy to implement, the model by Maruyama and Igarashi (2015) was used in this study.

Once the temperature and RH histories at a given position are obtained, the concrete expansion can be identified. The parameter $\gamma(T, RH)$, which was originally introduced in Eq. (1), can be determined as a function of the temperature $T$ (K) and relative humidity $RH$ (%) as:

$$\gamma(T, RH) = \gamma_0 \cdot h_1(T) \cdot h_2(RH)$$  \hspace{1cm} (13)

where $\gamma_0$ is a parameter related to the degree of expansion for reference temperature $T_0$ (K) and 100% relative humidity. Meanwhile, $h_1(T)$ is a function that shows temperature dependence, and $h_2(RH)$ is a function that shows humidity dependence, such that

$$h_1(T) = \exp \left[ \frac{E_a}{R} \left( \frac{1}{T_0} - \frac{1}{T} \right) \right]$$  \hspace{1cm} (14)

where $E_a$ is the apparent activation energy (J·mol$^{-1}$) of the rate of AAR expansion of concrete, $R$ is the gas constant (8.314 J·mol$^{-1}$·K$^{-1}$), and $E_a/R = 5500$ K which is derived by the previous research (Larive 1997). For $h_2(RH)$, Fig. 2, which is taken from Kotera et al. (2017), is applied.

In this study, the effects of the AAR-induced crack opening of the concrete on the moisture and heat transfer and heat and water capacity of the concrete were not considered. In addition, the AAR gels should, in theory, contribute to changing the sorption isotherms, which the current model does not consider.

### 3. Input parameters for AAR-degraded concrete

#### 3.1 Volumetric expansion parameters for concrete

The free AAR expansion strain of concrete as a function of time, which is represented by Eqs. (1) and (12), is necessary for this calculation. Based on the experimental data under constant temperature and RH (Orbovic et al. 2015), the volumetric expansion strain of the target concrete without restraint is given by:

$$\varepsilon(t) = 7.0 \times 10^{-3} (1 - e^{-0.007t})$$  \hspace{1cm} (15)

The experimental data were compared with the result of Eq. (15) in Fig. 3. In this figure, experimental data of...
longitudinal and transverse directions of AAR reactive concrete as well as longitudinal deformation of reference (non-reactive) concrete are shown. The parameter $A_l$ for the reduction coefficient $\zeta$ of $\zeta$ was set as $A_l = 5$ according to the authors’ previous parametric studies (Kojima et al. 2017).

### 3.2 Parameters for the material properties

The material properties of the AAR concrete are shown in Table 1 after 260 days and at 995 days. The parameter $G_l$ is necessary to model the change in Young’s modulus due to AAR expansion per Eq. (7) for AAR concrete. From the experimental measurements, Young’s modulus of the undegraded concrete at 28 days was 37700 MPa, and that of the AAR concrete at 995 days was 28100 MPa. Based on Eq. (7), $E_0$ and $E(\zeta(995))$ were set as 37500 MPa and 28100 MPa, respectively. Since $\zeta' = \zeta$ under the non-restraint condition, the free expansion given by Eq. (15) is used and, based on Eq. (16), the parameter $G_l$ was found to be 0.251, as follows:

$$G_l = \left(1 - \frac{E(\zeta(995))}{E_0}\right) \times \frac{1}{\zeta(995)}$$  \tag{16}

A comparison of the experimental and modeled Young’s modulus values for AAR concrete is shown in Fig. 4, from which it can be observed that Eq. (16) overestimated the reduction of Young’s modulus at 240 days. This demonstrates that the compressive strength of the AAR concrete does not deteriorate with aging. Consequently, in the current study, no deterioration has been hypothesized; in other words, the parameter $G_s$ is equal to zero. A constant compressive strength of 63 MPa was applied for the analysis.

For the tensile stiffness behavior, $C$ (Maekawa et al. 2001) was set to 0.4. The strain at the compressive strength can be given by $\varepsilon_c = \frac{2}{\varepsilon_{f_y}}$. For the 2D calculation, Poisson's ratios for both the anchor bolts and the steel plates were set to zero.

### 3.3 Modeling of the RC shear wall specimens

The geometry and dimensions of the RC shear wall specimens are shown in Fig. 5. Regarding the properties of the concrete, the uniaxial tensile strengths of the regular concrete at 975 days and AAR concrete at 995 days were, respectively, 4.39 and 3.18 MPa (Sheikh 2017), and the remaining properties were the same as those given in Section 3.2. The properties of the rebars are summarized in Table 2. For the FE model, the reinforcement ratios were calculated for each component of the RC shear wall specimen (Table 3). The arrangement of the rebars is the same as that shown in Habibi et al. (2015).
3.3.1 2D FE modeling
For the 2D FE analyses, the shear wall, flange pillars, loading stub, base stub, anchor bolts, and steel plates that are shown in Fig. 5 were all modeled using 2D shell elements. The shear wall, flange pillars, loading stub, and base stub were modeled using RC elements, which were originally developed by Okamura and Maekawa (1985). This model is a spatially averaged constitutive law of the element that considers the (1) concrete, (2) rebars, (3) bond behavior between the concrete and rebars as a tensile stiffening curve, and (4) cracks. Therefore, the arrangements of reinforcement bars are easy to implement, while the overall load-displacement of RC members can be reproduced well under the concept of averaged-stress – averaged-strain relationship of RC elements. Elastic elements are used for the anchor bolts and steel plates. The mesh used in the shear walls is depicted in Fig. 6. The mesh size was adjusted so that
the number of divisions of one flange pillar in the horizontal direction was equal to four. Detailed information regarding the mesh can be found in Table 4.

3.3.2 3D FE modeling

Figure 7 shows the model used for the 3D isoperimetric elements. In this analysis, symmetry is assumed, so that only half of the specimen is required to be modeled. The shear wall, flange pillars, loading stub, and base stub were all modeled using 3D solid isoperimetric elements with the RC element model from Okamura and Maekawa (1985).

The mesh size was adjusted so that the number of divisions of the flange pillar in the horizontal direction was equal to four. In addition, the number of divisions in the thickness direction of the shear wall of the half-thickness model was equal to two. Details of the meshing are provided in Table 5.

Based on the material properties (Habibi et al. 2015), the arrangement of the rebars in the shear wall was determined as shown in Fig. 8(a). The RC shear wall was modeled using RC elements, which are aligned in the central layer of the shear wall, and plain concrete elements as shown in Fig. 8(b).

The positions of the rebars in the flange pillar are shown in Fig. 9(a), and based on this, the flange pillar was modeled using RC and plain concrete elements, as shown in Fig. 9(b). For this modeling, different types of RC elements were applied that accounted for varying reinforcement ratios and alignments in different directions. The reinforcement ratios for each direction are given in Table 6 so as that the impact of reinforcement bars on the mechanical behavior of each member was appropriately considered in the analysis. The each ratio was determined by the local reinforcement ratio of the specimen.

4. Parametric studies

4.1 Boundary conditions for loading of the regular concrete shear wall

4.1.1 Cases analyzed

During the benchmark calculation, details of the bound-
The first trial calculation revealed that the stiffness of the calculation result was larger than that obtained in the experimental data. Therefore, to understand the possible experimental conditions, a parametric study was conducted numerically.

For this study, monotonic pushover loading was assumed, while the experimental data was obtained via cyclic loading. The conditions to reproduce the experimental data can be found by comparing the calculation results. In this analysis, the deformation during loading was defined as the relative horizontal deformation between top and base stubs as it is corresponding to the experimental results.

As it is shown later that the stiffness of the experimental data of regular concrete (shown in Fig. 20) shows much smaller than that of calculated one, the reason of this small stiffness can be attribute to the bending deformation of base stub or moving up of the edge of the base stub. For this background, five cases were considered, as illustrated by the schematics shown in Fig. 10. Two cases comprise modeling without the anchor bolts and the other three cases comprise modeling with the anchor bolts. For the cases without the an-

Table 6 Rebar reinforcement ratios of the reinforced concrete elements used in the model.

| Reinforced concrete elements used in the model | Rebar ratios in the horizontal direction (x-axis) (%) | Rebar ratios in the vertical direction (y-axis) (%) | Rebar ratios in the thickness direction (z-axis) (%) |
|-----------------------------------------------|--------------------------------------------------|--------------------------------------------------|--------------------------------------------------|
| Shear wall (A)                                 | 1.600                                            | 1.538                                            | 0.000                                            |
| Flange pillar (B)                              | 1.600                                            | 5.556                                            | 0.880                                            |
| Flange pillar (C)                              | 0.528                                            | 5.556                                            | 0.880                                            |
| Flange pillar (D)                              | 1.600                                            | 0.000                                            | 0.000                                            |
| Loading stub                                   | 2.000                                            | 2.000                                            | 2.000                                            |
| Base stub                                      | 2.000                                            | 2.000                                            | 2.000                                            |

Fig. 8 Arrangement of the rebars and modeling of the rebars in the shear wall (unit: mm).

Fig. 9 Arrangement of the rebars and modeling of the flange pillar.
anchor bolts, in one scenario the base stub is fixed horizontally, while in the other scenario, it is not.

For the cases with the anchor bolts, the parameters are a combination of the initial tensile force of the anchor bolt and the stiffness of the steel plate, as well as the fixing condition of base stub in horizontal direction as provided in Table 7.

4.1.2 Detailed modeling for the cases with anchor bolts

Two different initial tensile forces for the anchor bolts were considered in this study. It was hypothesized that the initial force in the anchor bolt is equivalent to a stress of $\frac{1}{3}f_c$ in the base stub concrete, where $f_c$ is the compressive strength of concrete. This stress is equal to the maximum allowable stress for permanent load, and based on our experience, sometimes, such stress is applied to fix the base stub during the horizontal loading to compensate motion of moving up of base stub. The first tensile force regards the actual compressive strength of 74 MPa, giving an initial tensile force for the two anchor bolts of 9862.5 kN and a stress of 409.6 MPa. In this calculation, force balance was considered between base stub and two anchor bolts. If we assume that the anchor bolts are a high-strength steel (such as 1000 MPa in strength), the obtained stress also satisfy the allowable stress for the permanent load (1/2 of the strength). Therefore, the condition seems reasonable. The second considered force regards the design compressive strength of 30 MPa, giving an initial force and stress of the two anchor bolts of 3747.7 kN and 155.6 MPa, respectively. It is possible that during the experimental campaign, actual compressive strength of concrete cannot be precisely predicted, and the design of the experiment could be based on the design strength. Therefore, for the additional case, the experimental design based on the design strength was used. In these calculations, it is assumed that the width of the base stub is 550.0 mm, the width of the steel plate is 680.9528 mm which are estimated by the picture provided by ASCET office, and the cross-sectional area of the two anchor bolts is 24078.4 mm², which are equivalent

| Case   | Modeling of the base stub | Horizontal direction of the side of the base stub | Initial tensile force of the anchor bolt (kN) | Elastic modulus of the steel plate (MPa) |
|--------|---------------------------|--------------------------------------------------|---------------------------------------------|----------------------------------------|
| B-1    | Without anchor bolts      | Fixed                                            | —                                           | —                                      |
| B-1A   | Without anchor bolts      | Not fixed                                        | —                                           | —                                      |
| B-2    | With anchor bolts         | Fixed                                            | 9862.5                                      | $2 \times 10^6$                        |
| B-3    | With anchor bolts         | Fixed                                            | 3747.7                                      | $2 \times 10^6$                        |
| B-3A   | With anchor bolts         | Fixed                                            | 3747.7                                      | $2 \times 10^5$                        |

Table 7 The cases of analysis using 2D FE modeling.

Fig. 10 The boundary conditions applied for the five cases.
to the two circular section of anchor bolts with a radius of approximately 62 mm guessed from the picture provided by ASCET office.

The stiffness of the plate that is connected to the anchor bolt and transfers the force to the base stub is also studied. One case assumes that the plate is rigid. In this case, Young’s modulus of the steel plate was set to be 2.0 × 10^8 MPa. The second considered case examines an ordinary carbon steel plate with Young’s modulus 2.0 × 10^5 MPa.

(1) Anchor bolts
In the models, the two anchor bolts overlap with the base stub. The locations of these elements are the same as those for some elements of the base stub, as they have different nodes with identical coordinates. As can be gleaned from the drawings and photographs of Habibi et al. (2015) and Sheikh (2017), the diameter of the anchor bolts appears to be approximately 120 mm. According to the condition for the meshing consistency, the width of one anchor bolt was set to 123.8095 mm, and a diameter of 133.4 mm was assumed for the steel sleeve hole for one of the anchor bolts.

The thickness of the two anchor bolts was set to 194.48 mm. This value was determined from the area of the circular section of the anchor bolt with a diameter of 123.8095 mm. The elastic modulus of the two anchor bolts was set as 2.0 × 10^5 MPa. At the bottom of the anchor bolts, a spring with a stiffness of 1605228 N/mm was connected. This number was determined by assuming a bolt length of 1500 mm.

By considering the thickness of the base stub (550 mm) and the thickness of the two anchor bolts (194.48 mm), the thickness of the base stub, which does not overlap with the two anchor bolts’ elements, was set as 355.52 mm (= 550 mm - 194.48 mm). A schematic of the anchor bolts is shown in Fig. 11.

(2) Steel plates
The elements of the two steel plates overlap with some of the elements of the shear wall. Based on the images shown in Habibi et al. (2015) and Sheikh (2017), the length of the plate appears to be approximately 650 mm. By considering the mesh consistency, the length of the plate was set as 680.9524 mm. The height of the plate was assumed to be 72.0 mm according to measurements by hand of the shear wall drawing. The thickness was set as 450 mm as determined by the gap between the thickness of the base stub and the wall. The estimated plate configuration is shown in Fig. 12.

(3) Joint at the lower end of the base stub
To determine the source of the stiffness in the lower portion of the specimen, the bending of the base stub and the resulting moving up from the test bed were considered. For this simulation, joint elements were introduced at the bottom of the base stub. These joint elements are rigid in the compression stress region and have a lower stiffness in the tensile stress region.

(4) Joints
The steel plates are connected to the anchor bolts, with
rigid joints between them. For the contact joints between the steel plates and the base stub, only the \( \gamma \)-direction force is transferred. This is based upon the assumption that the plates hold down the base stub. For the contact joints between the anchor bolts and the base stub, only the \( x \)-direction force is transferred, as the anchor bolts are inserted into the holes of the base stub. Thus, it is assumed that there is no force transfer in the \( y \)-direction. The schematic figure for the steel plate is shown in Fig. 13.

4.1.3 Loading conditions
The loading conditions are shown in Fig. 14. A forced displacement of the monotonic pushover was applied up to 8 mm where the post peak behavior is confirmed. The weight of the specimen itself was considered using concrete and steel densities of 2400 kg/m\(^3\) and 7870 kg/m\(^3\), respectively. For the vertical load, a total force of 800 kN was applied to the nodes on the top side of the loading stub; the loading stub width corresponded to a shear wall panel width of 1300 mm. The forced displacement of the nodes in the middle of the loading stub was controlled.

The analysis procedure was as follows: (1) the initial tensile force of the anchor bolts was applied (only for Case B-2, Case B-3, and Case B-3A), (2) the weight of the specimen itself and the vertical load were applied, and (3) the forced displacement was applied.

The initial tension was applied using the equivalent nodal force of the tension.

4.1.4 Results of analysis
The results of the load–displacement relationship for Cases B-1 and B-1A are shown in Fig. 15 with compared by the regular concrete experimental result and simulation results by Jurcut (2015). As it is shown, the experimental data shows very small stiffness rather than the other calculated results, while there is no significant difference observed between cases B-1 and B-1A and showed the highest stiffness of the members. The elastic behavior of RC members of B-1 and B-1A, which was confirmed within the range less than about 0.5 mm in deformation, is much higher than others, the boundary on the contrary, the distribution of the cracks (which is shown as the red lines in elements representing the crack direction and the principle strain) and their directions are different, as shown in Fig. 16. The bottom left of the flange pillar and shear wall and the right top of the flange pillar display horizontal cracking, which is caused by bending deformation. These trends are common between both B-1 and B-1A. When the base stub is fixed at the side, bottom, and center, almost no damage is found in the base stub, while when the horizontal direction of the side of the base stub is fixed (such as by using L-shape angles on the sides of base stub), damage.
is found on the right side of the base stub. This is due to the deformation of the base stub to the right side, which is constrained by the fixed condition at the center of gravity of the base stub. This is a typical example of artificial and unrealistic damage in numerical modeling. However, due to this, the distribution and pattern of the diagonal cracks in the shear wall differ between the two cases. For B-1A, a slipping behavior on the mid-right region of the shear wall is found. Even though the load–displacement relationship is very similar, the crack pattern, which reflects how the force is transferred through the specimen, is different. The experimental results for the regular concrete specimen (REG A in the ASCET Phase II) did demonstrate significant cracking in the base stub. Therefore, the boundary condition for Case B-1 may be more accurate than that for Case B-1A.

A comparison of B-1 and B-1A suggests that the failure mode of the RC shear wall cannot be identified by the load–displacement relationship, and the calculation results that reproduced the load–displacement relation do not necessarily represent the failure process of the target member.

The load–displacement relationships of Case B-2, B-3, and B-3A are shown and compared with the experimental data in Fig. 17. In the case that the base stub is fixed by the anchor, still the stiffness of the calculation is much higher than that the experimental data as well as calculation by Jurcut (2015). For the maximum loading capacity that is the peak of load in Fig. 17, the capacity is the largest for B-2, followed by B-3 and then B-3A, with no significant difference observed. When these are compared with the maximum loading capacities of B-1 and B-1A, the slightly larger loads of B-2, B-3, and B-3A at 2 mm displacement were confirmed instead of those of B-1 and B-1A. However, in general, all of the load–displacement relationships are quite similar.

For Cases B-2 and B-3, where the rigid steel plate was used, no bending deformation occurs in the steel plates, which is indirectly confirmed by the deformation of the base stub shown in Fig. 18. The stress was uniformly applied from the steel plates to the base stub. On the other hand, in Case B-3A, where the elastic modulus of the steel plate is $2.0 \times 10^5$ MPa, the deformation is curved at the top end of the anchor bolts. Therefore, a concentrated stress can be observed at the center of the top of the base stub (Fig. 18). In general, the difference in the deformation of the base stub cannot be confirmed in Cases B-2 and B-3. The stiffness of the steel plate is significant for the deformation of the base stub in this case.

As shown in Fig. 18, around the maximum load with

Fig. 16 Deformation and crack direction vectors in the RC shear wall specimens for Cases B-1 (left) and B-1A (right), which are without anchor bolts. The magnification of the deformation is 10. The red lines of each element shows the direction of cracks and length of crack has a linear relationship with principle strain in tension.

Fig. 17 Load–displacement relationship Cases B-2, B-3, and B-3A. The experimental data and calculation data for the regular concrete is taken from Jurcut (2015) and Orbovic et al. (2019).
a deformation of 4 mm, the moving up of the base stub from the test bed is found in all cases with anchor bolts. Therefore, the apparent stiffness of the specimen includes the bending deformation of the specimen. For Case B-3A, where the stiffness of the steel plate is that of carbon steel, the bending of the steel plate led to the larger bending deformation of the base stub and the larger moving up deformation.

Figure 19 shows the deformation and crack vector diagram of Cases B-2, B-3, and B-3A. In all cases, slight damage can be observed in the base stub at the central bottom and the mid-left side. This is caused by the anchoring and fixed boundary conditions.

The difference in the initial stress (B-2 and B-3) does not significantly affect the bending cracking and diagonal cracks in the shear wall and flange pillars, while B-2 showed slightly larger bending cracking at the bottom left of the shear wall. Case B-3A, where the steel plate has the stiffness of ordinary carbon steel, displayed a different cracking pattern, especially in the right-side shear wall and the connection between the shear wall and flange pillar. Due to the large deformation of the base stub, the angle of the diagonal crack becomes a bit shallower (approximately 45°), and the wider range of diagonal cracking that occurred in the shear wall did not propagate to the flange pillar. Based on the crack pat-

Fig. 18 Vertical deformation and vertical displacement contour plots of the base stub and steel plate for B-2, B-3, and B-3A at a 4 mm of displacement during the loading. The magnification of the deformation is 100.

Fig. 19 Shown are deformation at a 4 mm displacement and the crack line diagram of Cases B-2, B-3, and B-3A. The result of Case B-1 is shown for reference. The magnification of the deformation is 10. The blue triangles show the neutral axis position at the bottom of the shear walls.
terns, the position of the neutral axis in the shear wall did not significantly change. We propose that this is the main reason why the loading capacity did not significantly change between Cases B-2, B-3, and B-3A.

In general, the base stub is fixed by a sufficient number of prestressed tendons by considering the risk of moving up and the friction between the test bed and the base stub. If the lateral force cannot be borne by the friction, then an additional jig will be introduced. For the AAR concrete, the experimental conditions are complex, as the concrete member will expand as a result of AAR, and thus controlling the geometry of the specimen, including holes for tendons, is difficult. It is possible that the holes for multiple tendons cannot be fitted to the holes of the test bed, and therefore the presented anchoring is a possible solution. The present calculations, however, suggest that such an anchoring method might not be appropriate for lateral loading. At the minimum, the geometry of the specimen and properties of the related parts should have been sufficiently quantified for the benchmark calculation.

Despite the explicit consideration of various boundary conditions for the base stub, there is no significant difference in the load-displacement relationship between the models with and without the anchor bolts. Based on this, the difference between numerical calculations and experimental data of the stiffness of the specimen would provide an explanation for the variant behavior. The most likely cause is the drying of the specimen. In a previous study by one of the authors (Sasano et al. 2018), the stiffness of the RC shear wall was reduced by half as a result of drying. The mechanism for this reduction can be understood in two parts as follows: (1) the Young’s modulus reduction of concrete due to drying, which originates from the mismatch in volumetric expansion and contraction between the coarse aggregate and mortar and the resultant crack opening in the concrete, and (2) shrinkage-induced cracking in the shear wall. To confirm this, the authors are interested in investigating the ambient conditions during the preparation of loading.

4.2 2D and 3D models

In this section, the load–displacement results are compared for the analysis using the 2D shell elements (2D model) and the analysis using the 3D solid elements (3D model). The analyses were conducted under cyclic loading with a regular concrete specimen at an age of 240 days under the same boundary conditions of Case B-1 in Section 4.1.

Figure 20 compares the analysis results for the 2D and 3D models. A clear difference between the two models can be observed. The 3D model has a smaller load at the first breakpoint, which corresponds to bending cracking in the flange pillars. Furthermore, the 2D model has remarkable degradation above the forced displacement of 4 mm, while the 3D model maintains loading capacity until a displacement of 7 mm.

A possible explanation for these two results is the difference in the modeling of the rebars. In the 2D model, the rebars are uniformly distributed, while the 3D model considers the arrangement of the rebars, i.e., it considers the covering concrete. In the 2D model, the stiffness of the covering concrete in the flange pillars is increased over that of the 3D model. Consequently, the initiation of crack propagation in the flange pillars begins at a lower load for the 3D model. For the 2D model, the local reinforcement ratio of each element is small and, therefore, the element located at the edge of the side pillars yields easily, and once yielding occurs, the successive yielding of inner elements is possible. Therefore, even though the total force is similar, the bending deformation led to the resultant yielding of elements occurring from outside to inside, followed by a sudden drop of load-bearing capacity; for the 3D model, the yielding is mitigated by the concentrated local reinforcement ratio.

The similar discrepancy is also found in the post-peak region. 2D model showed rapid decrease in the loading capacity while 3D model kept the loading capacity until the displacement of 7 mm. This rapid decrease of loading capacity in post-peak region of 2D model can be explained by the lack of two-dimensional (XZ plane) confinement of concrete at the bottom of the flange pillars by the reinforcement bars.

It should be mentioned here that a part of mismatch between 2D model and 3D model can be solved by considering the appropriate modeling of the distribution of reinforcements in flange pillars, but if there is a material property change under the 3D or 2D geometrical condition, such as concrete ductility change under confinements, the difference is remained.

Figure 20 additionally compares the calculation results with the experimental data. Although the initial stiffness and load-bearing capacity of experiment were
less than those of the 2D and 3D models, the results of the 3D model are much closer to the experimental results than those of the 2D model. In particular, in the negative displacement region, which is much closer to the experimental result rather than in the positive displacement region, the envelope of the experimental data and 3D model exhibits consistent trends. As the process of damage accumulation and resultant stiffness change in the region from elastic state to the displacement of –4 mm, which is confirmed by the envelope of load-displacement relationship shown in Fig. 20, is similar to that of experimental data, it is considered that the 3D model somehow captured the mechanism that is affected by the bending crack propagation.

4.3 RC shear walls with AAR concrete

4.3.1 Strategy of the analysis

Following the results of Sections 4.1 and 4.2, the structural performance of the AAR shear wall is presented in this section. During ASCET Phase III, the discussion was focused on the mechanism of change in the shear wall failure mode due to AAR, the damping and ductility of the shear wall, and the crack pattern. Therefore, a parametric study of degraded concrete is herein presented in this section.

The analysis of the RC shear wall with the AAR concrete was carried out using the 3D model presented in Section 4.2, with the same calculation procedures and loading conditions as those in Section 4.2.

The analysis procedures composed of four steps, namely 1) self-weight calculation, 2) AAR expansion and damage calculation, 3) calculation of the vertical loading of 800 kN, and 4) cycling horizontal loading calculation. During these calculations, vertical deformation of the nodes on the bottom surface of the bottom stub is fixed. Barycenter nodes (YX-plane) of the bottom stub were fixed horizontally. In addition, during the calculation of step 4, the half-height both sides of the bottom stub were fixed horizontally. During the step 2) for AAR expansion, first free expansion of concrete was decided based on Section 3.1, and then concrete properties of AAR concrete at 28 days were used as input, and based on the confined AAR expansion strain, the material properties were determined according to the equations shown in Sections 2.2.1 and 2.2.2. Therefore, the material properties of elements in the shear wall specimen were different each other, and different from the experimentally obtained properties shown in Table 1. After the step 2), all the material properties, stress, and strain of elements were succeeded to the next steps.

First, the analysis of the RC shear wall with AAR concrete (AAR-1) is compared with the analysis of the RC shear wall with regular concrete (REG; same as the 3D model in Section 4.2), and the comparison is presented in Section 4.3.1. Next, a parametric study with different degraded AAR concretes was performed. The parameters for this parametric study were decided as follows. First, there was little information about the relationship between tensile strength and compressive strength. However, after the expansion of silica-gel and resultant crack-openings inside of concrete may reduce the (splitting) tensile strength rather than compressive strength. Therefore, the possibility that the tensile strength of AAR affected concrete is deteriorated severely. The 50% of tensile strength of AAR-1 was considered as AAR-2. Second, the bond behavior between AAR affected concrete and reinforcement bars were not well understood. Due to the AAR deterioration of concrete, existing crack could reduce the bonding performance, and resultantly, the reduction of shear transfer in concrete through deteriorate bonding is possible. To understand this behavior, bond characteristic parameter that represents the force transfer from concrete to reinforcing bars as a function of tensile strain, is changed so as the rapid decrease in force transfer as tensile strain is increased. This is considered as AAR-3. Third, concrete property change is a function of strain in the present model. However, in the realistic case, drying shrinkage of concrete can be compensated by the AAR expansion of concrete and it produces the large reduction of compressive strength, while the nominal strain is constant. Therefore, 50% of compressive strength and tensile strength of AAR-1 was used for AAR-4. The possibility that only the compressive strength is deteriorated is simulated as AAR-5, which can illustrate the impact of deterioration of tensile strength by comparing AAR-4. In addition to these, the boundary condition impact was also considered again by restraining the rotation of base stub. The cases of AAR-1 and AAR-4 are chosen to change the fixation of base stub. They are noted as AAR-1A and AAR-4A. Finally, AAR-6 was considered. In AAR-6, the possibility that ASR expansion of the shear wall panel was restrained by the side pillars due to the different AAR expansion was analyzed to understand the impact of accumulated compressive strength on the structural behavior. The test cases are summarized in Table 8, with detailed material properties for each case shown in Table 9.

During the calculation, it was found that the sudden change in the reinforcement ratio between the flange pillars and stubs produced unrealistic cracks in some cases. Therefore, the vertical reinforcement ratio of the loading stub and the base stub changed to 5.556%, which is the same value as that of the flange pillars.

4.3.2 Comparison between the regular concrete shear wall and AAR concrete shear wall

A comparison of the load–displacement relationships for REG and AAR-1 is shown in Fig. 21. It can be clearly seen in the close-up figure shown in Fig. 21 that the initial stiffness of AAR-1 is smaller than that of REG. To confirm this trend quantitatively, Young’s modulus of concrete and the initial stiffness of the RC walls are summarized in Table 10. By comparing the ratio of Young’s modulus of the concrete in AAR-1 to that of REG and the ratio of the initial stiffness of AAR-1 to
that of REG, we determine that the reduction of the initial stiffness can mainly be explained by the reduction of Young’s modulus due to AAR, because the expansion by AAR is restrained by the reinforcement bars and neighboring members and crack opening which cause the structural stiffness reduction is not produced.

The first breakpoint of REG and AAR-1 is at approximately 0.4 mm with a load of 680 kN and 0.5 mm with a load of 630 kN, respectively. This 0.1 mm difference in the first breakpoint is a result of the accumulated compressive strength due to the restraint of AAR expansion by the rebars and the reduction of Young’s modulus. The reduction of the corresponding load is equivalent to the reduction of the tensile strength (approximately 67% of REG).

The ultimate load-bearing capacities did not change significantly, as they are 1280 kN and 1231 kN for REG and AAR-1, respectively. To understand this correspondence, crack contour plots and minimum stress contour plots of the concrete at a forced displacement at +6 mm for REG and AAR-1 are shown in Fig. 22. The magnification of the deformation in the figures is \( \times 10 \). Based on the crack contour plots, the bending cracking is more localized in AAR-1 than in REG. When the minimum stress contour plots are compared, it can clearly be confirmed that the compressive stress strut, which transfers the force from the loading stub to the base stub, is different. REG shows a smaller width of the compressive zone on the bottom line of the shear wall as compared to AAR-1, and the width of AAR-1 is double that of REG. Consequently, even though the compressive region of the right-side pillar is almost equal between REG and

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**Table 8** The cases of analysis using 3D FE modeling.

| Case   | Analysis condition                                                                 |
|--------|-------------------------------------------------------------------------------------|
| REG    | Reference calculation for the regular concrete. Equal to the result of the 3D model in Section 4.2. |
| AAR-1  | Reference calculation with AAR concrete.                                             |
| AAR-2  | Tensile strength is 50% of AAR-1.                                                   |
| AAR-3  | Bond characteristic parameter is \( C = 10.0 \) (the reduction of the bond transfer is significant). |
| AAR-4  | Tensile strength and compressive strength are 50% of AAR-1.                         |
| AAR-5  | Compressive strength is 50% of AAR-1.                                               |
| AAR-1A | Rotation of the boundary condition on the loading stub is restrained.               |
| AAR-4A | The same mechanical properties as AAR-4.                                             |
| AAR-6  | Shear wall: tensile strength and compressive strength are 50% of AAR-1.             |
|        | Flange pillars loading stub and base stub: material properties are the same as those of REG. |

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**Table 9** Concrete material properties for each case.

| Case   | Compressive strength \( f'_c \) (MPa) | Uniaxial tensile strength \( f'_t \) (MPa) | Elastic modulus \( E \) (MPa) | Bond characteristic parameter \( C \) (-) | Loading stub |
|--------|--------------------------------------|------------------------------------------|-----------------------------|-----------------------------------|--------------|
| AAR-1  | 63.0                                 | 3.18                                     | 37500                       | 0.4                               | Rotation Free |
| AAR-2  | 63.0                                 | 1.59                                     | 37500                       | 0.4                               | Rotation Free |
| AAR-3  | 63.0                                 | 3.18                                     | 37500                       | 10.0                              | Rotation Free |
| AAR-4  | 31.5                                 | 1.59                                     | 37500                       | 0.4                               | Rotation Free |
| AAR-5  | 31.5                                 | 3.18                                     | 37500                       | 0.4                               | Rotation Free |
| AAR-1A | 63.0                                 | 3.18                                     | 37500                       | 0.4                               | Rotation Fixed |
| AAR-4A | 31.5                                 | 1.59                                     | 37500                       | 0.4                               | Rotation Fixed |
| AAR-6  | Shear wall                           |                                           | 37500                       | 0.4                               | Rotation Free |
|        | Flange                               |                                           | 47150                       | 0.4                               |              |

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**Table 10** Elastic modulus of the concrete and initial stiffness of the RC wall.

| Case          | Elastic modulus (MPa) | Initial stiffness (kN/mm) |
|---------------|-----------------------|---------------------------|
|               | Regular concrete      | 47150                     | 1925                       |
|               | AAR concrete          | 37500 at 28 days          | 28100 at 995 days          | 1350                                     |
| Ratio         | 0.80                  | 0.60                      | 0.70                       |

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Fig. 21 Load–displacement relationships for AAR B2 (experimental), REG, and AAR-1.
AAR-1, the strut width of AAR-1, which is represented by the diagonal line of the white-lined triangle shown at the bottom right of the shear wall, is approximately 36% wider than that of REG. As the compressive strength of REG is 250% of the compressive strength of AAR-1, this change in the width of the compressive zone is balanced by the difference in the compressive strength. Consequently, since the failure mode of the RC shear wall is approximately determined by the compressive strength in the corner of the bottom left of the shear wall, the load-bearing capacities became comparable in this study.

Figure 21 further shows the load–displacement results of the experiment (AAR B2 with an age of 995 days (Orbovic et al. 2019). There was no significant difference between the experimental and analysis results for the maximum capacity. However, the experimental result reached the yield strength at a displacement of around 2 mm and exhibited brittle failure, while the calculation results exhibited more ductile behavior. Figure 23 shows the crack pattern of AAR B2 after the loading experiment. After reaching the peak load, a forced horizontal displacement was applied continuously, and therefore, large spalling is observed in the middle of the shear wall as well as in the top right and bottom left of the side flange pillars. For AAR-1, as shown in Fig. 22(b), from the top left to the bottom right of the shear wall, the compressive stress strut was well confined. Additionally, the force was transferred at the bottom of the right flange pillar, where ductile behavior was observed, while that of the experiment did not occur at the bottom of the left flange pillar. Based on
this phenomenological difference, the observed concrete properties, such as fracture energy in the compression zone or the cohesion of the AAR concrete, would then certainly vary widely from the properties of regular concrete. It is also possible that the deteriorated bond between the reinforcing bars and the concrete due to AAR, which has been experimentally confirmed such as research by Li et al. (2020) could explain this discrepancy.

4.3.3 Parametric study of the AAR concrete shear walls
Based on the results of Section 4.3.1, the mechanism for the AAR B2 RC shear wall having an ultimate capacity at a 2 mm displacement with brittle failure is discussed here. The pushover parametric studies of RC shear walls with different properties for the AAR concrete are discussed. The load–displacement relationships of AAR-1, AAR-2, AAR-3, AAR-4, and AAR-5 are compared along with the AAR B2 results in Fig. 24.

Considering the results of AAR-2 and AAR-3, the bond property and tensile strength change do not significantly affect the results. When the compressive strength is halved, a peak is observed in the load–displacement relationship. AAR-4 and AAR-5 have peaks at 3 mm and 4 mm displacements, respectively. The obtained ultimate loading capacity of AAR-4 and AAR-5 was approximately 1200 kN, which is very close to the experimental value. There is a possibility that a further reduction of the compressive strength or a reduction of the strain at the compressive strength of the concrete could reproduce the peak position with a shorter displacement.

These results suggest the need to study (1) the stress-strain relationship of AAR-deteriorated concrete under compressive loading, (2) the compressive strength of AAR-deteriorated concrete under long-term restraining stresses, and (3) the compressive strength of AAR-deteriorated concrete under 2D confinement.

Additionally, the mechanism of the extremely fragile failure of AAR B2 is still unknown. Because of the confinement of AAR expansion in the shear wall, as well as the water supply process from the outside and resultant enhancement of AAR expansion of the shear wall in the thickness direction, delamination or vertical cracking along with compressive stress bearing direction is possible (Rossi et al. 1996). The authors suspect that this might be one of the reasons for the brittle failure.

Figure 25 shows the load–displacement curves of AAR-1, AAR-1A, AAR-4, and AAR-4A. In this figure, the impact of the restraint for rotation of the loading stub is presented. For AAR-1A and AAR-4A, the rotation of the loading stub is fixed, and only the displacement in the horizontal direction is allowed. In these cases, the ultimate loading capacity is increased dramatically. AAR-1A shows a monotonic increase until a 10 mm displacement, while AAR-4A has a peak around a 4.2 mm displacement, which is similar to that of AAR-4. Interestingly, when the rain of smeared crack of normal direction to the crack are compared, as shown in Fig. 26, the angles of the diagonal cracks vary considerably. Even for the displacement at the ultimate loading capacity, the failure mode of AAR-4A is different from that of AAR-4. It should be noted that the crack pattern of AAR-4A is similar to that of AAR B2, in which the diagonal crack propagated from the top of a side pillar to the bottom of the other with a horizontal flat region at the center of shear wall as illustrated in Fig. 23. Information about the rotation ability of the loading stub is necessary for the benchmark calculation. Based on the calculations shown in Fig. 26, it is then theorized that the rotation of the base stub was restrained in the experiment.

An additional calculation for AAR-6 was performed. In AAR-6, all of the members are regular concrete, except for the shear wall, which is AAR concrete that has

![Fig. 24 Load–displacement relationships for AAR-1, AAR-2, AAR-3, AAR-4, and AAR-5 compared with the AAR B2 experimental result.](image)

![Fig. 25 Load–displacement relationships of AAR-1, AAR-1A, AAR-4, AAR-4A, and AAR B2. In the legend, ft represents the tensile strength of AAR-1 and fc represents compressive strength of AAR-1.](image)
reduced compressive and tensile strengths. Only the shear wall expanded and was strongly confined by the other members. The simulation was performed to determine the impact of the strong expansion restraint on the ductility of the specimen.

The result is shown in Fig. 27, which confirms the peak in the load–displacement and the greater ductility in the behavior compared to that of AAR-4. This is because the flange pillars are regular concrete and, therefore, they are able to bear the force after the peak as compared to AAR-4. Consequently, the ductility of the flange pillars overcame the impact from the significant restraint of AAR expansion in the shear wall.

Interestingly, the force transfer mechanism in AAR-6 is completely different from that in AAR-4. The stresses in the rebars when the displacement is at 4 mm are compared in Fig. 28. According to this figure, the bending behavior is more dominant in AAR-4 than in AAR-6. AAR-6 demonstrates a more homogenous shear transfer mechanism. Consequently, it is evident that the self-induced stress from the restraint of the AAR expansion had a large influence on the force transfer mechanism in the shear wall. Based on this, as discussed previously, the significant restraint of AAR expansion in the shear wall is one of the possible mechanisms of the more brittle failure of AAR B2.

5. Conclusions

From the parameter sensitivity analysis of the experimental data in Assessment of Structures Subjected to Concrete Pathologies (ASCET) Phase II and III, the effects on the failure mode were compared and the following conclusions were obtained:

1. The boundary condition of the RC shear wall has a large influence on the analysis result of structural
performance, such as in the load–displacement curve, ultimate loading capacity, and crack pattern. The rotation of the loading stub and anchoring procedure of the base stub were critical components affecting the calculation results. The restraint of the rotation of the loading stub changed the ultimate loading capacity dramatically, and inappropriate anchoring of the RC shear wall introduced bending behavior during loading.

(2) The results of the two-dimensional (2D) and three-dimensional (3D) analyses were markedly different. Appropriate modeling, in particular, capturing the arrangement of reinforcement bars, is vital. In addition, the occurrence of initial cracks and the loading capacity could not be clearly reproduced. This suggests that consideration of the placement of rebars and covering concrete in the mesh setting in three-dimensional (3D) model affected the failure mode of the concrete. It is necessary to consider the possible failure mechanism and to reflect such features in numerical modeling.

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