

Article

Analytical Study on the Load-Bearing Performance of RC Beams Subjected to ASR Expansion

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Abstract: The alkali–silica reaction (ASR), a major cause of cracks in concrete, is a critical issue in the maintenance of social infrastructure. In this study, a concrete mesoscale model was meticulously developed, and a coupled stress–moisture model was also developed to reproduce ASR degradation. The aim was to investigate the effect of ASR degradation on the bending load-carrying capacity of RC beams. The concrete mesoscale model, specifically designed to reproduce ASR degradation, was modeled from three phases: coarse aggregate, mortar, and ITZ (interfacial transition zone). ASR was considered as the expansion of the coarse aggregate, and the objective was to reproduce expansion cracks with numerical analysis using the mesoscale model. Uniaxial compression tests were carried out on cylindrical specimens with ASR-accelerated deterioration to clarify the relationship between ASR deterioration and compressive properties, and the experimental results were used to identify material parameters in the mesoscale analysis model. The results showed that the model proposed in this study can reproduce the change in compressive properties due to expansion cracking. Finally, RC beams were constructed using the mesoscale model, and the effect of ASR degradation on the bending load-carrying capacity of the RC beams was investigated. The results showed that the presence of expansion cracks caused the initial stiffness of the load-displacement curves to decrease, but the bearing capacity tended to increase. This suggests that factors other than cracks, such as chemical prestress and boundary conditions in this model, have a strong influence on the load-bearing capacity of deteriorated RC beams.

Keywords: ASR; concrete; mesoscale modeling; FEM; moisture transfer analysis; ITZ; cracking



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1. Introduction

The alkali–silica reaction (hereinafter referred to as ASR), one of the deterioration phenomena of concrete, causes cracks in concrete and is a significant problem in the maintenance of concrete structures. The alkali–silica gel has high water absorption and expansion properties, so it expands when water is supplied from outside the concrete, causing internal stresses in the concrete, breaking down the surrounding cement paste, and causing microcracks and cracks over time as shown in Figure 1 [1–3]. Figure 2a shows the results of the observation of cracks around aggregate due to aggregate expansion under black light irradiation with fluorescent resin impregnation (meso-observation), and Figure 2b shows the results of the observation of cracks, typically cracks in the shape of a tortoiseshell, on RC members and structures due to ASR (macro-observation). Although the observation scales are different, it is important to clarify the effects of these cracks on durability and load-carrying capacity to realize rational maintenance management and to select appropriate repair and reinforcement measures.

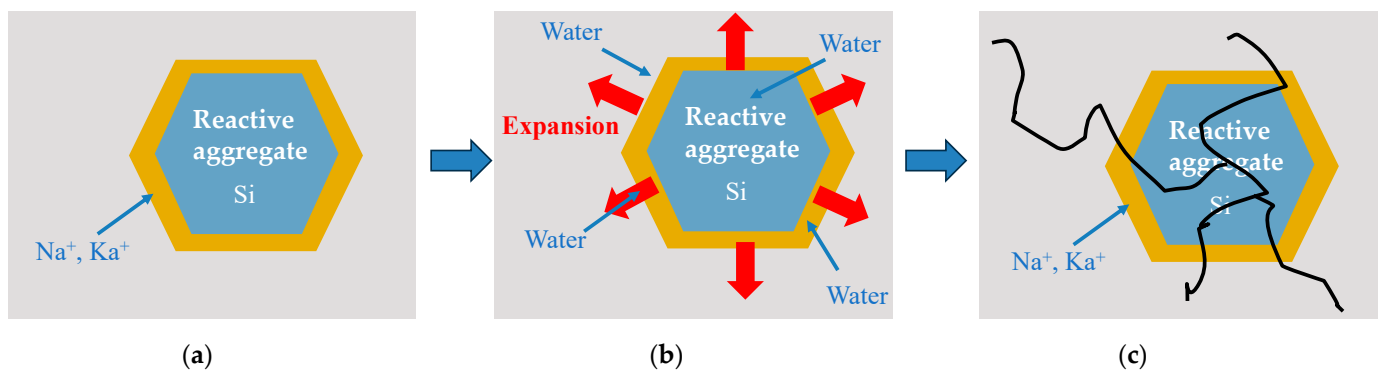


Figure 1. General expansion mechanism of ASR. (a) Generation of the alkali-silica gel; (b) water absorption expansion of gel; (c) occurrence of cracking.

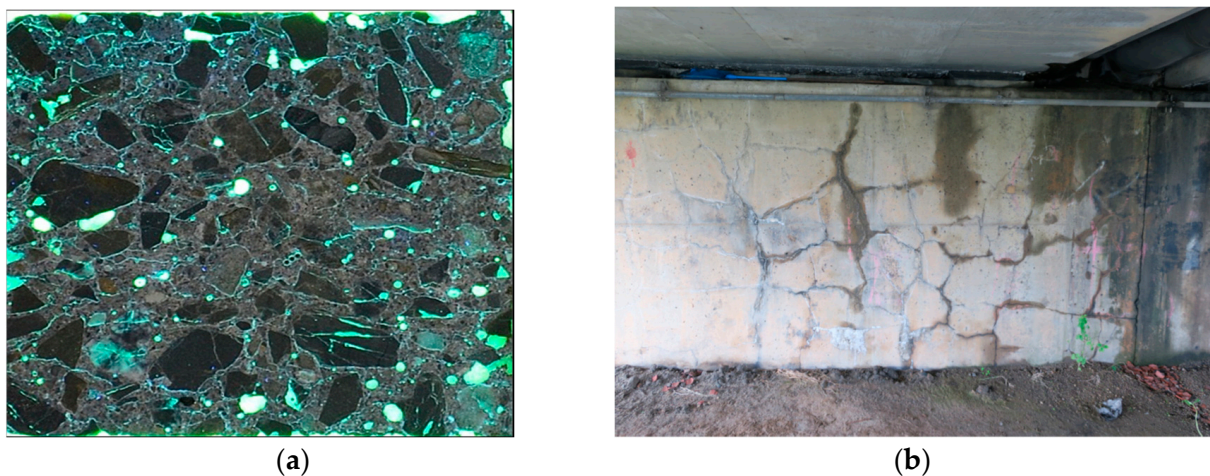


Figure 2. Cracks due to ASR: (a) Cracks around aggregate (mesoscopic observation); (b) cracks on bridge abutment surface (macroscopic observation).

In summary, the evaluation of the internal expansion phenomena caused by ASR is still under research, and the inability to evaluate and predict the performance of deteriorated structures caused by internal expansion phenomena such as ASR is also an issue.

As an overview of the existing research, many studies on the durability of concrete due to deterioration have been conducted to date, focusing on deterioration due to salt damage and other factors [4–6]. However, the risks associated with internal swelling reactions (ISRs), including ASR, have attracted attention both domestically and internationally, and research on the evaluation and prediction of these expansion phenomena is still in its infancy. The Japan Concrete Institute (JCI) has established the research committee (JCI-TC-211A). The main topics of the research on the internal expansion reaction are the modeling of expansion prediction and the modeling of mechanical performance degradation due to expansion. Regarding the modeling of expansion prediction, a model that takes into account the effect of temperature and humidity and the diffusion of alkali ions in the reaction process has been proposed. However, few expansion models consider the effect of reinforcing steel bars, and there are only a few cases in which structural performance evaluation is possible. In addition, structural models that describe macroscopic behavior (cracking, loss of bearing capacity, etc.) corresponding to microscopic expansion phenomena are still under study, making it difficult to evaluate ASR deterioration in real structures. Tatsuki et al. [7] cut RC slab members from a railroad viaduct damaged by ASR approximately 24 years after completion, observed the appearance, and conducted material strength tests. The results showed the compressive strength and static modulus of elasticity had dropped to 50–70% and 53–60%, respectively, due to cracking. Fan et al. [8] fabricated cylindrical specimens

and RC beam specimens, accelerated ASR, and observed the structural and mechanical behavior of the specimens. At 6 months of age, the compressive strength, tensile strength, and dynamic modulus of the cylindrical specimens were 24, 38, and 31% lower, respectively, than the corresponding values at 28 days of age. ASR caused visible cracking in the RC beams, but the flexural load-carrying capacity was similar to that of the non-reactive beams. In addition, there have been various previous studies on the effect of ASR on the mechanical properties of concrete, such as its compressive strength [9–12], elastic modulus [9,12], and fracture energy [9], and the relationship with the decline in mechanical properties has been clarified.

Thus, ASR expansion is known to cause a reduction in the mechanical properties of concrete materials. However, bending loading tests on RC beams showed that ASR expansion caused visible cracks in the RC beams but had little effect on structural performance. The literature differs on whether or not ASR expansion reduces the shear capacity of RC beams, and a systematic relationship between the amount of ASR expansion and load-carrying capacity is needed to clarify the effect of ASR on structural performance. The ultimate goal of this research is to quantitatively evaluate the remaining load-bearing capacity of reinforced concrete structural members or structures due to ASR deterioration and to develop rational maintenance methods based on that. To achieve this, it is necessary to propose realistic analysis methods and analysis models, and this research focuses on that point. At the same time, it will contribute to the clarification of the relationship between ASR expansion and the load-carrying capacity and fracture behavior of RC beam members, which has not been firmly established by previous studies. To clarify the relationship between ASR expansion and load-carrying capacity through achieving the above goal, this study proposes an ASR expansion model that takes into account the effects of external restraints and reinforcing bars. Furthermore, by quantifying the amount of cracking (width, direction, density, etc.), the relationships between the amount of expansion, the amount of cracking, and load-bearing capacity will be systematized, with the ultimate goal of achieving efficient load-bearing capacity evaluation.

Specifically, in this study, uniaxial compression tests of concrete deteriorated by accelerated expansion were first conducted (Section 2), and observations of crack width and crack density were made to examine the state of internal crack damage in concrete with the same degree of ASR. Using these experimental results, the effects of ASR internal crack damage on mechanical properties are experimentally discussed. In Section 3, a concrete mesoscale model was developed to reproduce ASR degradation, and the material model was identified and validated based on the experimental results in Section 2. In Section 4, based on the mesoscale model developed in Section 3, a mesoscale model of RC beams was developed to reproduce ASR expansion mechanism based on a moisture-crack coupling theory. The ASR expansion theory and the coupled moisture-cracking theory were developed and applied based on the RC beam model. Section 5 summarizes and describes the findings of this study.

2. Uniaxial Compression Test of Concrete Deteriorated by Accelerated Expansion

Very little is known about the mechanical properties of concrete subjected to ASR damage. Here, uniaxial compression tests in accordance with JIS A 1108:2018, “Method of test for compressive strength of concrete”, are performed on concrete deteriorated by accelerated expansion to investigate the effects of ASR deterioration on mechanical properties.

2.1. Specimen Fabrication and Testing Methods

2.1.1. Specimen Fabrication Method

Table 1 shows the mixing conditions for the specimens used in this experiment. The cement used was ordinary Portland cement (density 3.16 g/cm^3 , $\text{Na}_2\text{Oeq} = 0.51\%$), and a non-reactive aggregate, crushed limestone sand (surface dry density 2.65 g/cm^3 , water absorption 1.50%), was used as fine aggregate. Reactive aggregate (surface dry density 2.54 g/cm^3 , water absorption 1.50%) and a non-reactive aggregate, crushed limestone

(surface dry density 2.70 g/cm^3 , water absorption 1.43%), were used as coarse aggregate at a pesimum mixture ratio of 3:7. The reactive aggregate is andesite, showing a mottled texture with opalescent veins, and as a result of the alkali–silica reactivity test (chemical method) of JIS A 1145, “Alkali-silica reactivity test of aggregates”, the alkali concentration reduction Rc was 168 mmol/L and the dissolved silica content Sc was 710 mmol/L, which were judged “not harmless”. The maximum size of the coarse aggregate was set at 25 mm to mitigate the non-uniformity in the concrete cross-section due to the mixed coarse aggregate. In accordance with JIS A 5308, “Ready-mixed concrete”, the target slump and air content were set to $10 \pm 2.5 \text{ cm}$ and $4.5 \pm 1.5\%$, respectively. In this experiment, W/C was set as high as 60% to increase the amount of pore water to allow ASR expansion to develop with internally held water only in the fully sealed condition. The mixing ratio was determined by trial mixing in the laboratory to obtain the target slump and air content based on N. In addition, AE water reducer was used to ensure the dispersion of cement in the concrete and to obtain appropriate workability. The specimens were cylindrical in shape, $\varnothing 100 \times 200 \text{ mm}$. To adjust the total amount of alkali, NaCl reagent was added to the mixing water to obtain $\text{Na}_2\text{O}_{\text{eq}}$ amounts of 5.5 kg/m^3 (specimen name: R5.5) and 4.0 kg/m^3 (specimen name: R4.0). As a test specimen without expansion, a 1.5 kg/m^3 of NaCl reagent was also prepared (test specimen name: N), which was cement-derived alkali only, and no NaCl reagent was added.

Table 1. Specifications of beam elements.

Specimen	W/C (%)	Unit Weight (kg/m^3)					Admixture		
		W	C	S	G		AE Water Reducer (g/m^3)	AE Agent (mL/m^3)	NaCl (g/m^3)
					Reactive Aggregate	Limestone			
N									0
R4.0	60	180	300	811	285	707	750	9	4656.3
R5.5									7487.1

After demolding, the specimens were sealed and cured using aluminum tape. At 7 days of age, the seals were temporarily unsealed to replenish the internal water content, and the specimens were vacuum-absorbed for 4 h before being sealed and cured again. To replenish the water lost through the hydration reaction and secure sufficient water for the ASR, vacuum absorption was performed again at 50 days of age, and the lumber was resealed and cured with sufficient water inside. ASR expansion was performed at a room temperature of 20°C . The amount of ASR expansion was measured over time, and the material was tested after the expansion had been stabilized.

2.1.2. Test Procedures

Figure 3 shows the installation of a cylindrical specimen during a compressive strength test. In this test, after measuring the ASR expansion and observing the appearance of the specimens of N, R4.0, and R5.5, the specimens were subjected to static compressive loading. A universal loading test device with a capacity of 1000 kN was used for loading, and the loading rate was controlled so that the average strain rate was approximately $10^{-7}/\text{s}$. At the same time, data on longitudinal and transverse strain and load were collected. Polyester strain gauges with a test length of 60 mm were affixed in the axial and circumferential directions at the center of the specimen’s side face to measure longitudinal and transverse strain. Static modulus of elasticity was calculated from the stress–strain curve by calculating the slope connecting the stress points at 50μ and at $1/3$ of the maximum stress.

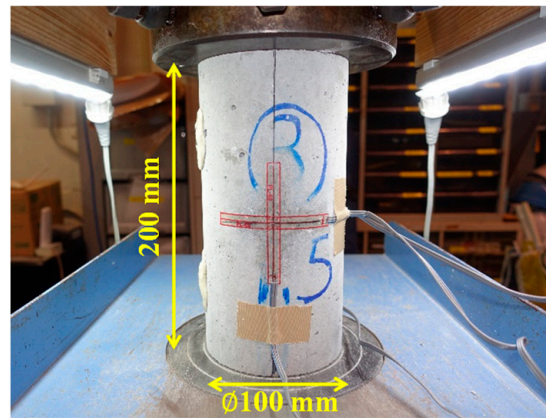


Figure 3. Installation of cylindrical specimen.

2.2. Results and Discussion

2.2.1. ASR Damage Status

The ASR expansion of N, R4.0, and R5.5 before the compressive strength test was about 100 μ , 1800 μ , and 3000 μ , respectively. Photographs of the surface cracks taken at 60° intervals before the loading test are shown in (a) N, (b) R4.0, and (c) R5.5 in Figure 4, and enlarged photographs of the red frames are shown in Figure 5. In addition, the internal cracking situation due to fluorescent resin impregnation is shown in Figure 6. In these figures, the surface of N is sound, while cracks are observed on the surfaces of R4.0 and R5.5, and white precipitates are observed on the surfaces of R4.0 and R5.5, which appear to be traces of ASR gel exudation. The crack width was larger on R5.5 than on R4.0, as shown in Figure 5.

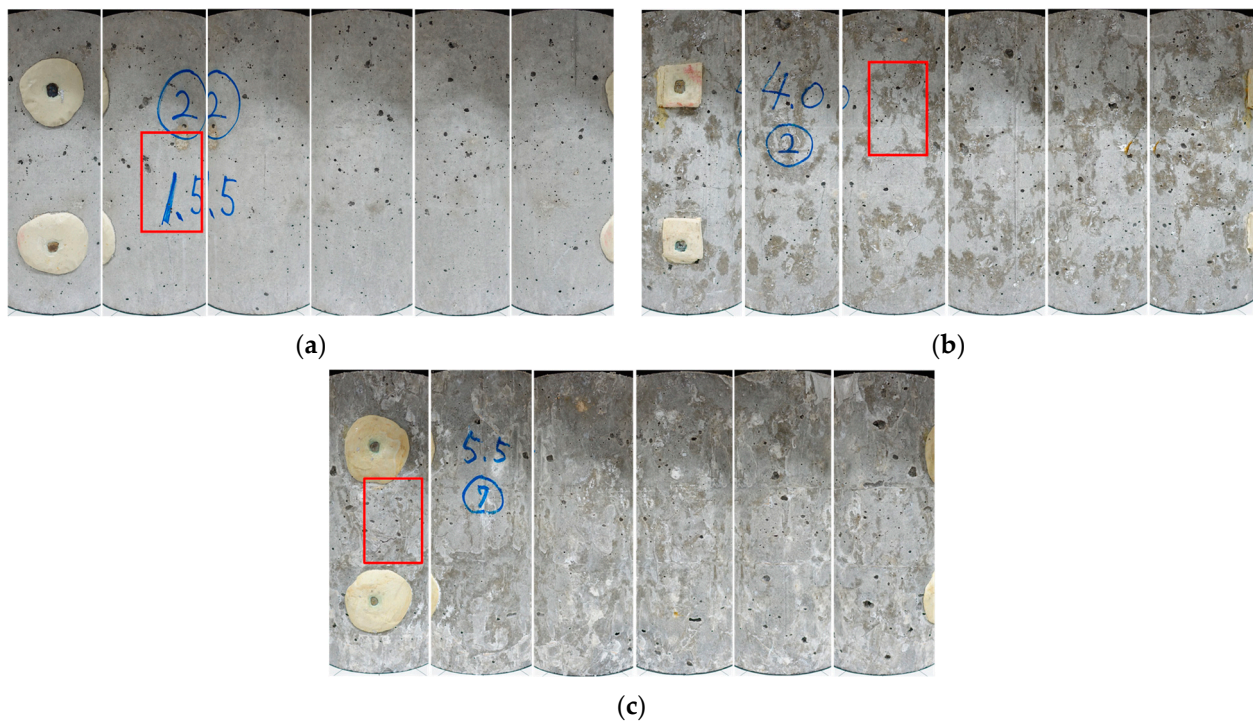


Figure 4. Surface cracks before loading test of cylindrical specimen: (a) Specimen N; (b) Specimen R4.0; (c) Specimen R5.5.

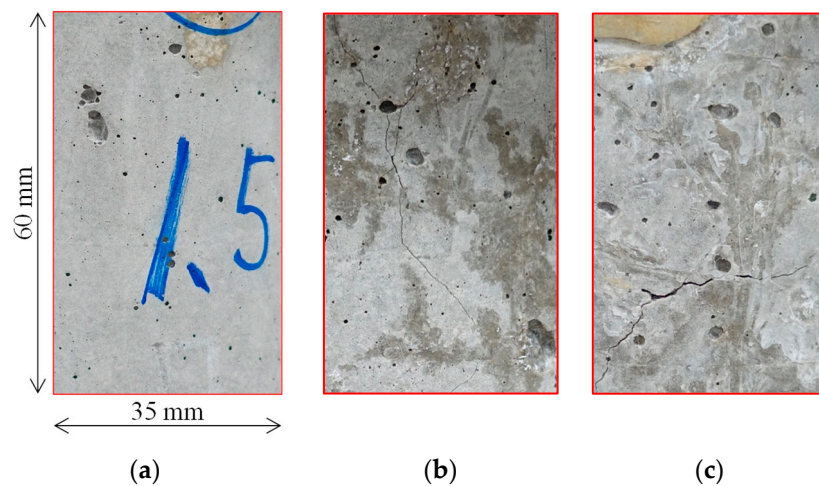


Figure 5. Enlarged view of the test piece surface: (a) Specimen N; (b) Specimen R4.0; (c) Specimen R5.5.

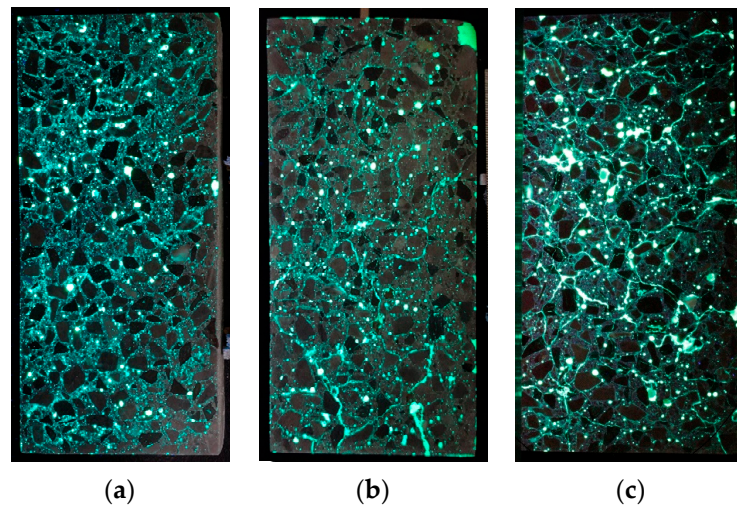


Figure 6. Fluorescent photograph of internal cracks: (a) Specimen N; (b) Specimen R4.0; (c) Specimen R5.5.

2.2.2. Static Compressive Strength

The nominal stress–strain curves obtained from the loading tests are shown in Figure 7. In the sound case N, the compressive strength was 40 to 50 MPa, and the modulus of static elasticity was 30 to 40 GPa. In contrast, the modulus of static elasticity and maximum load decreased in cases R4.0 and R5.5, where ASR expansion degradation occurred. The amount of surface expansion and the fracture condition of the specimen surface indicate that the maximum load and the modulus of static elasticity decreased in case R5.5, where the expansion degradation was more severe. The stress–strain curve in Figure 7 shows that the specimen in case N tends to soften and fail after reaching its compressive strength, as is generally observed when cylindrical specimens are subjected to compressive loading. On the other hand, the static modulus of elasticity of cases R4.0 and R5.5 was smaller than that of case N, and the specimens failed without showing a clear compressive strength. Case R5.5 had a smaller modulus of static elasticity than case R4.0, and the modulus of static elasticity increased between approximately 1000 and 1500 μ . The stress–strain curve was convex downward, which is characteristic of this case. In particular, as can be seen in Figure 7c, the stress–strain curve becomes unstable and diverges as the strain increases. This indicates that the expansion cracks have had some effect on the strain gauge. This characteristic was seen only in the ASR-degraded specimens compared with the sound specimens.

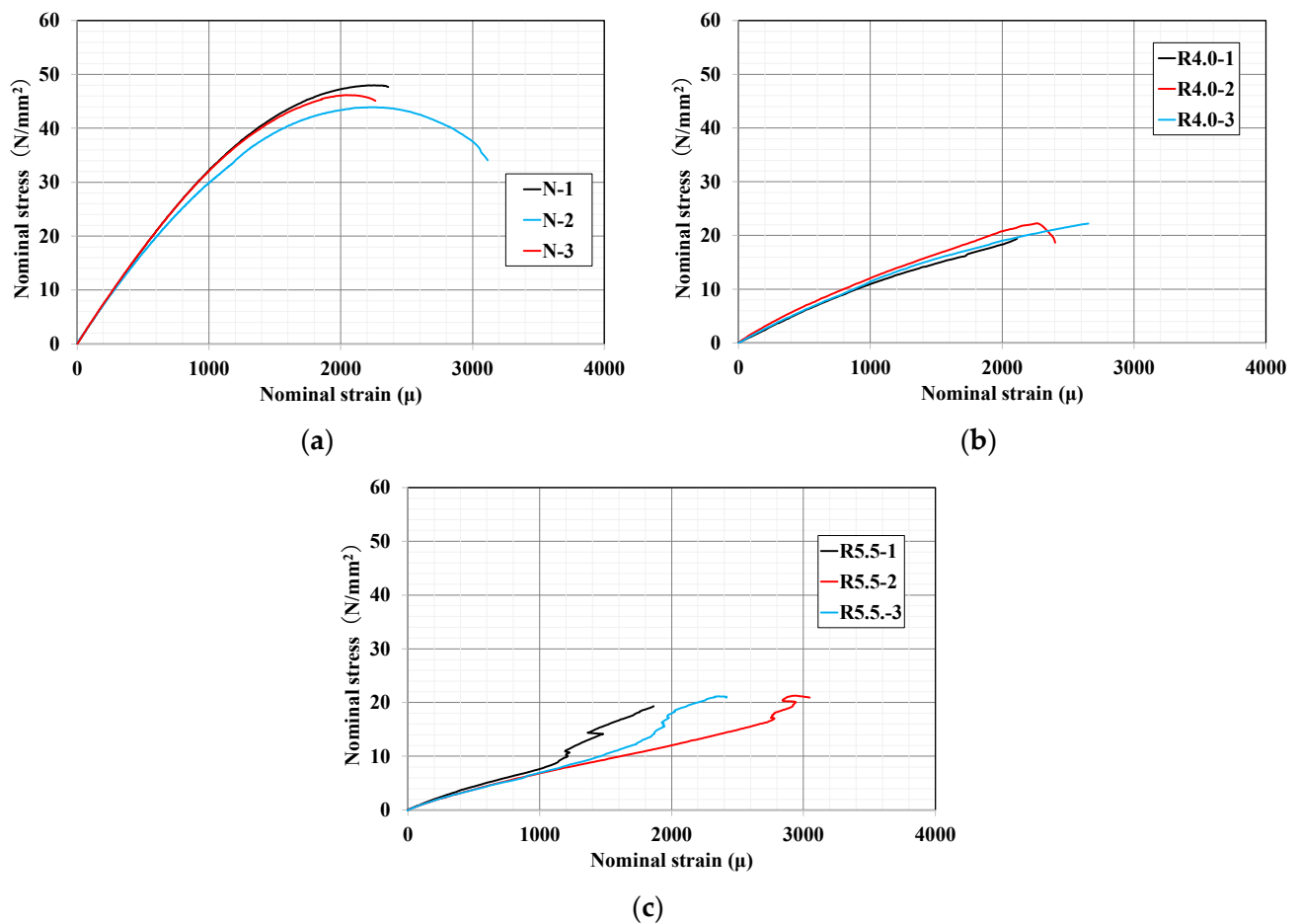


Figure 7. Nominal stress–strain curve: (a) Case N; (b) Case R4.0; (c) Case R5.5.

3. Mesoscale Modeling of Concrete by FEM

Based on the background that quantitative and systematic evaluation of ASR deterioration has not been performed, this study aims to propose a model to evaluate the effect of ASR deterioration on load-bearing performance. This section describes the specific modeling process.

3.1. Mesoscale Modeling Policy

3.1.1. Overview

The analytical dimension was three-dimensional, and the concrete was a three-phase model consisting of coarse aggregate, mortar, and interfacial transition zone (hereinafter referred to as ITZ). The coarse aggregate was assumed to be spherical, with diameters of 5, 7.5, 10, 12.5, 15, 20, and 25 mm, randomly placed according to a Fuller curve [13] with a coarse aggregate fraction of 40%. After determining the coarse aggregate arrangement, the entire model, including the mortar section, was divided. A four-node tetrahedral element was used for both the coarse aggregate and the mortar because of the need to create a spherical mesh, and a six-node pentahedral element (triangular prism) was used for the ITZ because of its mechanical and geometrical properties. Figure 8 shows the coarse aggregate distribution, and Figure 9 shows the cylinder specimen based on mesoscale modeling.

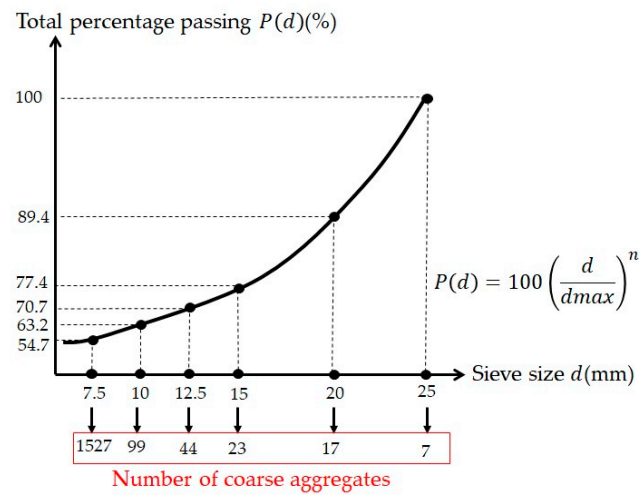


Figure 8. Coarse aggregate distribution.

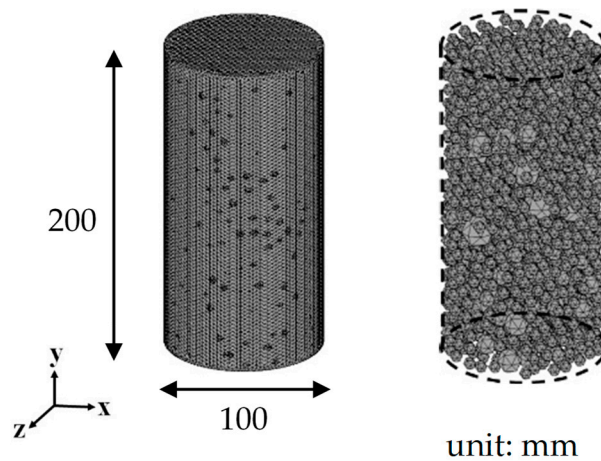


Figure 9. Cylinder specimen based on mesoscale modeling.

3.1.2. Material Properties and Cracking Model

Various researchers have proposed mesoscale models for concrete, with a static modulus of elasticity of coarse aggregate ranging from 40 to 75 GPa and that of mortar from 11 to 35 GPa [14]. The present model does not directly assume the material property parameters, but rather, identifies them by comparing and matching analytical and experimental results based on the results of previous uniaxial compression tests of sound and ASR-impaired cylindrical specimens. Details are given in Section 3. Based on previous papers [15–17], the Poisson's ratio was set to 0.23 for the coarse aggregate and 0.2 for the mortar. Therefore, the model assumes that the coarse aggregate and mortar are elastoplastic, subject to the linear Drucker–Prager yield condition. The concrete was assumed to follow a linear softening law with a slope depending on the element size, based on Equations (1)–(3), proposed by Bazant [18] after the tensile strength.

$$E_c = \left(\frac{1}{E} - \frac{2G_f}{f_t^2 h} \right)^{-1} \quad (1)$$

$$G_f = \frac{1}{100} (d_{max})^{\frac{1}{3}} f_t^{\frac{1}{3}} \quad (2)$$

$$h = \sqrt[3]{V} \quad (3)$$

where E is the static modulus of elasticity, E_c is the slope after softening, f_t is the tensile strength, G_f is the fracture energy, d_{max} is the maximum diameter of coarse aggregate, f'_{ck} is the compressive strength of concrete, h is the average element length, and V is the element volume. In this analysis, the maximum diameter of the coarse aggregate is set to 25 mm. The model does not consider crushing on the compressive side.

3.1.3. ITZ Modeling

(1) Overview

ITZ is a materially discontinuous region between the aggregate and paste, as shown in Figure 10. ITZ is said to be a thin and fragile region that always exists in concrete because it has more voids than other aggregates and pastes, its void area is large, and its connectivity is high [19–21]. The representation of ITZ is an important factor in the mesoscale modeling of concrete, and researchers have attempted to model ITZ using various methods [22]. In this study, to reflect the thinness and fragility of ITZs in the analysis, interface elements, as shown in Figure 11, were used and reproduced using tests in previous papers and previous calculation equations.

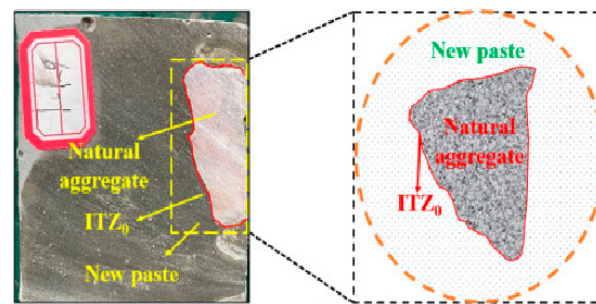


Figure 10. ITZ inside concrete (cited in Liu et al. (2022) [23]).

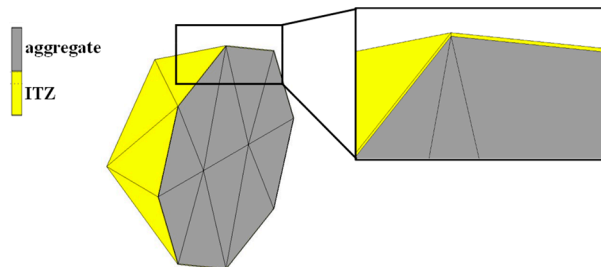


Figure 11. Diagram of the ITZ model.

The interface element is represented by stresses on the relative displacements of the element surface (corresponding to Figure 12: numbers 4, 5, and 6) and the bottom surface (corresponding to Figure 12: numbers 1, 2, and 3). This 3D interface element is given a relative displacement component from one normal component and two shear components and is represented with respect to the local coordinate system.

$$v_n = u_1^{top} - u_1^{bottom} \quad (4)$$

$$v_s = u_2^{top} - u_2^{bottom} \quad (5)$$

$$v_t = u_3^{top} - u_3^{bottom} \quad (6)$$

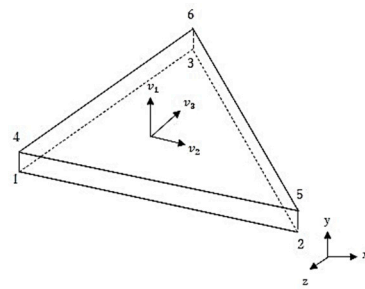


Figure 12. The 3D interface element conceptual diagram.

Based on the relative displacement component, the effective displacement is defined as follows:

$$v = \sqrt{v_n^2 + v_s^2 + v_t^2} \quad (7)$$

The corresponding deformation is the relative deformation of the top and bottom surfaces of the element, and the element can also be used as a very thin element such that the top and bottom surfaces coincide. The mass of the element does not exist because the volume of the element is considered to be zero.

The adhesion strength is introduced as a function of displacement, characterized by an initial reversible reaction (Figure 13: displacement 0 to v_c) followed by an irreversible reaction upon reaching a critical displacement (v_c). The irreversible portion is characterized by increasing damage ranging from the beginning of delamination to complete delamination. In the bilinear type adhesion constitutive law used in this study (Figure 13), the adhesion stress is calculated from Equation (8), as can be seen from the geometric features of the figure:

$$\left. \begin{aligned} \tau &= \frac{2G_c}{v_m} \frac{v}{v_c} & \text{if } 0 \leq v \leq v_c \\ \tau &= \frac{2G_c}{v_m} \left(\frac{v_m - v}{v_m - v_c} \right) & \text{if } v_c < v \leq v_m \\ \tau &= 0 & \text{if } v_m < v \end{aligned} \right\} \quad (8)$$

where τ : adhesion stress, G_c : fracture energy, v_c : critical displacement, and v_m : maximum displacement. Because the adhesion properties between the materials reproduced in this study are assumed to have different behaviors in the shear and vertical directions, differences in properties due to direction are taken into account by defining the ratio of the maximum shear stress to the maximum tensile stress and the ratio of the fracture energy.

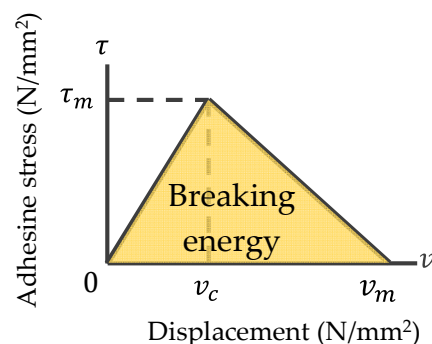


Figure 13. Adhesion composition law (bilinear type).

(2) Modeling

As mentioned above, this study models an ITZ using interface elements. The parameters used to determine the mechanical properties of ITZs are the fracture energy G_c , bond strength τ_m , critical displacement v_c , and maximum displacement v_m . The fracture energy ranges from 0.01 N/mm to 0.08 N/mm and the adhesion strength from 1.5 to 2.5 MPa [24–35], although each researcher has set different values and modeled them differently. In this

study, the fracture energy and adhesion strength were set to $G_f = 0.08$ N/mm and $\sigma_t = 2.5$ MPa, respectively, in order to observe the phasing of ITZ fracture. The maximum displacement is obtained by dividing twice the fracture energy (G_c) by the adhesion strength (τ_m), based on the characteristics of the bilinear constitutive law. The critical displacement (V_c) was set to 0.9 times the maximum displacement (V_m), based on the idea that it is the same as the value in the shear direction. The properties of the interface elements applied to the ITZ in this study are shown in Table 2.

Table 2. Concept of adhesion parameters.

	Directions	Adhesion Strength τ_m (MPa)	Critical Displacement V_c (mm)	Maximum Displacement V_m (mm)	Breaking Energy G_f (N/mm)
ITZ	Normal (Tensile)	σ_t	$0.7 V_m$	$\frac{2G_f}{\tau_m}$	0.08
	Shear	$2\sigma_t$	$0.7V_m$	$\frac{2G_f}{\tau_m}$	0.08

3.2. Parameter Identification

In Section 2, uniaxial compression tests were conducted on concrete deteriorated by accelerated expansion to study the effect of ASR deterioration on the mechanical properties of concrete. In this section, similar tests are conducted on the analysis using the mesoscale model described in Section 3.1, and the material parameters of the mesoscale model are identified and validated by comparison and fitting with the experimental results.

3.2.1. Analysis Model

A FEM model of a cylindrical specimen (Figure 14a,b) was created using the concrete mesoscale model described above. The total number of nodes was 23,704, and the total number of elements was 1,124,174. The boundary conditions for the analysis are shown in Figure 14c. The coefficient of friction between the loading surface and the cylindrical specimen was set to 0.3.

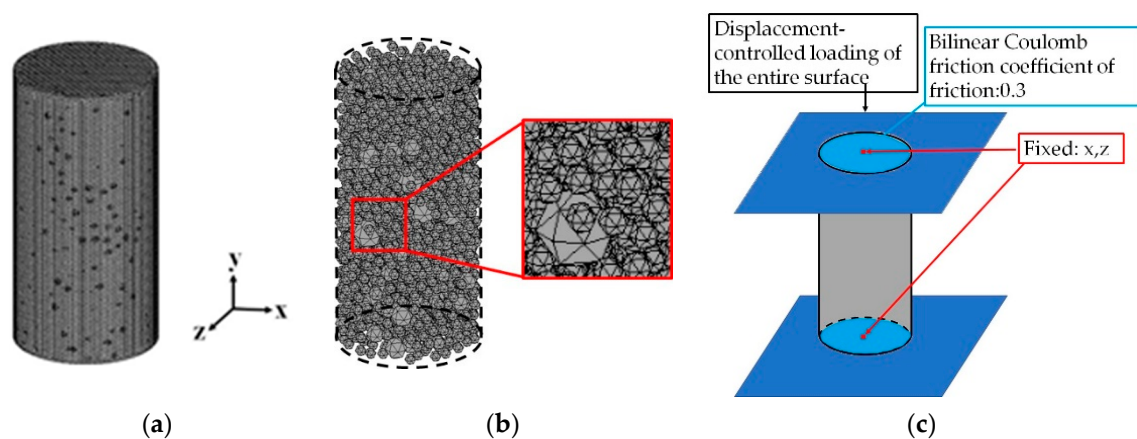


Figure 14. Analysis model diagram: (a) Cylindrical specimen diagram; (b) internal coarse aggregate arrangement diagram; (c) boundary condition.

3.2.2. Analysis Method

First, uniaxial compression tests were performed on the above model without expansion degradation, and the parameters were adjusted to match the nominal stress–strain curve obtained experimentally. The resulting curve is shown in Figure 15. The mean error was 6.8%, and the static modulus of elasticity up to 200 μ was 33,115.9 (MPa) in the analysis and 31,281.7 (MPa) in the experiment. The maximum load was 46.6 MPa in the analysis

and 46.3 MPa in the experiment. The material parameters identified by the adjustment are shown in Table 3.

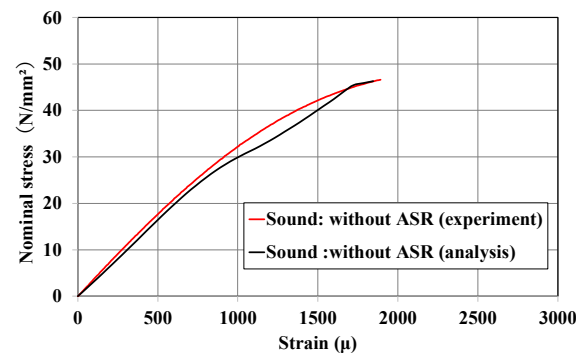


Figure 15. Nominal stress–strain curve (sound: without ASR).

Table 3. Material parameters.

	Elastic Modulus (MPa)	Poisson's Ratio	Compressive Strength (MPa)	Tensile Strength (MPa)
Coarse aggregate	75,000	0.23	150	7.5
Mortar	40,000	0.2	35	3.5

3.3. Validation by Expansion–Compression Analysis

Next, a uniaxial compression analysis was performed on the analytical model described in Section 3.2.1 with ASR degradation, i.e., aggregate expansion, to examine the validity of the model in reproducing changes in the mechanical properties associated with ASR degradation. In the experiments described in Section 2, two series were considered in the case of expansion and degradation, with alkali contents of 4.0 kg/m³ and 5.5 kg/m³, respectively, and approximately 2000 μ and 3000 μ of surface expansion were measured as a result of ASR expansion, respectively. Therefore, in this analysis, two cases of alkali content of 4.0 kg/m³ and 5.5 kg/m³ were considered, and the aggregate expansion was given such that the surface expansion was measured in these cases. The results of the uniaxial compression tests on the ASR-degraded cylindrical specimens are shown in Figure 16. In addition, Figure 17 shows surface cracks before loading.

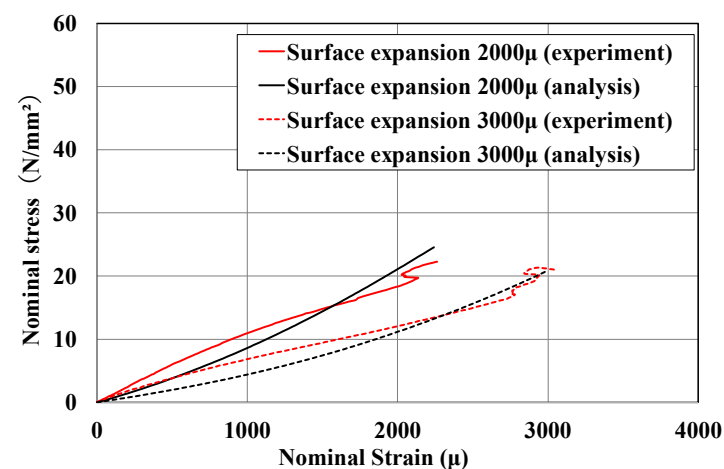


Figure 16. Nominal stress–strain curve (ASR degraded specimen).

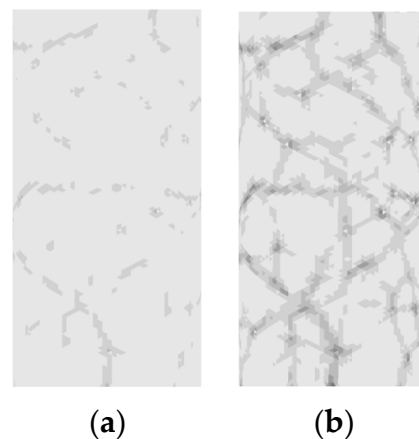


Figure 17. Surface cracks before loading: (a) Surface expansion 2000 μ ; (b) surface expansion 3000 μ .

Figure 16 shows that the trend of the change in compressive properties is generally consistent between the experimental and analytical results. The stress–strain data under ASR expansion and deterioration showed decreased initial stiffness and maximum load due to expansion cracks. The stress–strain data for the ASR expansion-deteriorated condition showed decreased initial stiffness and maximum load due to expansion cracking. The results of the reduction in the static modulus of elasticity of concrete subjected to alkali–silica reaction are consistent with previous studies [36]. The stress–strain curves of the concrete were found to be more convex than those of the healthy case. This is considered to be due to the fact that a cavity is generated inside the specimen as a result of cracking. When the cavity closes due to loading, a reaction force is less likely to be generated. The analytical results were more convex downward than the experimental results, but this may be due to the condition of the loading surface, such as the effect of the unevenness of the contact surface and the specimen due to expansion and deterioration.

4. Load-Carrying Capacity of RC Beams with ASR Deterioration

In this section, the concrete mesoscale model is used to create an RC beam model, and ASR degradation analysis and bending loading analysis are conducted to discuss the cracking behavior and mechanical performance associated with ASR degradation (expansion–cracking–loading coupling). The validity and applicability of the analytical model are fundamentally examined by comparing the fracture and response properties obtained from the analysis with previous experimental data.

4.1. Analysis Procedure

In this analysis, (1) ASR degradation analysis based on aggregate expansion and (2) bending load analysis for RC beams with ASR degradation are conducted in two stages to evaluate fracture properties and load-carrying capacity. Figure 18 shows the flowchart of the ASR degradation analysis. In the ASR degradation analysis, aggregate expansion is first generated based on the relative humidity determined by the moisture transfer analysis. Then, based on the FEM, the stresses (cracks) associated with the aggregate expansion are solved. As described in Section 4.2.1, the permeability is corrected based on the cracks to couple the stresses and moisture in this study. The increment is then updated by substituting the corrected hydraulic conductivity with the current one. This is repeated for a set number of steps to complete the ASR degradation analysis. In the bending loading analysis, static loads are applied to deteriorated RC beams solved based on the ASR deterioration analysis to investigate the effects of ASR deterioration on the fracture properties and load-carrying capacity.

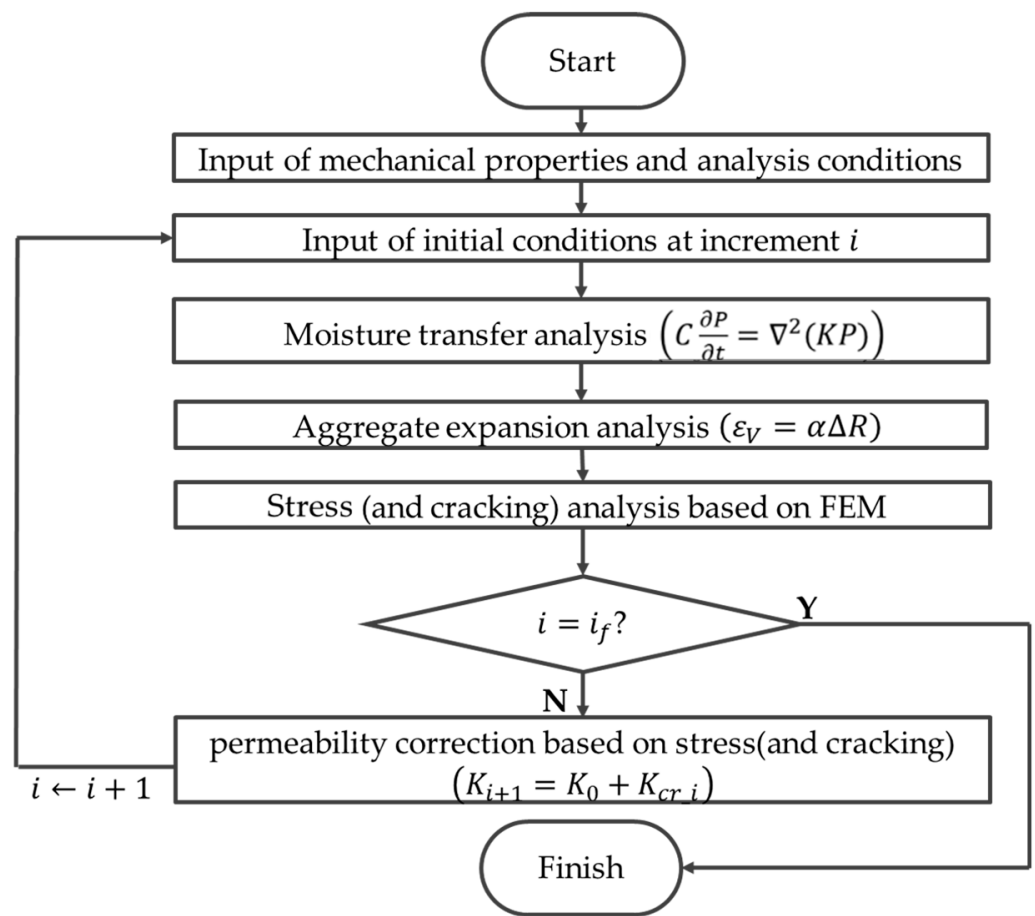


Figure 18. ASR aggregate expansion procedure.

4.2. Modeling of RC Beams

4.2.1. Analysis Target and Model

The rectangular RC beam (Figure 19) and its analytical model (Figure 20) are shown in the figure. To reduce the computational load, a quarter of the beam was modeled and analyzed by halving the beam span and depth directions, respectively. This study uses a three-point bending loading of a flexure-fracture RC beam as an example for analytical investigation. Because the mesoscale model described in Section 2 was used, the concrete portion was modeled from three phases: mortar, coarse aggregate, and ITZ. The rebar was modeled as an elastoplastic body, according to von Mises, instead of a truss element, which is commonly used for the analysis of combined deterioration (rebar corrosion). The elements used were four-node hexahedral elements for the mortar, coarse aggregate, and rebar, and six-node interface elements for the ITZ. D10 bars were used for the tensile main bars and $\Phi 6$ bars for the compressive main bars. Additional $\Phi 6$ bars were placed as shear reinforcement bars with a span of 100 mm. The boundary conditions defined for the analytical model were fixed in the y-direction at a position 100 mm from the edge of the model, in addition to the symmetry plane constraint. For the rebars, solid elements were used to create a mesh that faithfully reproduced the ribs and nodes of the deformed steel bars. The total number of nodes was 599,987, and the total number of elements was 2,620,082. In this analysis, complete adhesion was assumed at the interface between the rebars and concrete.

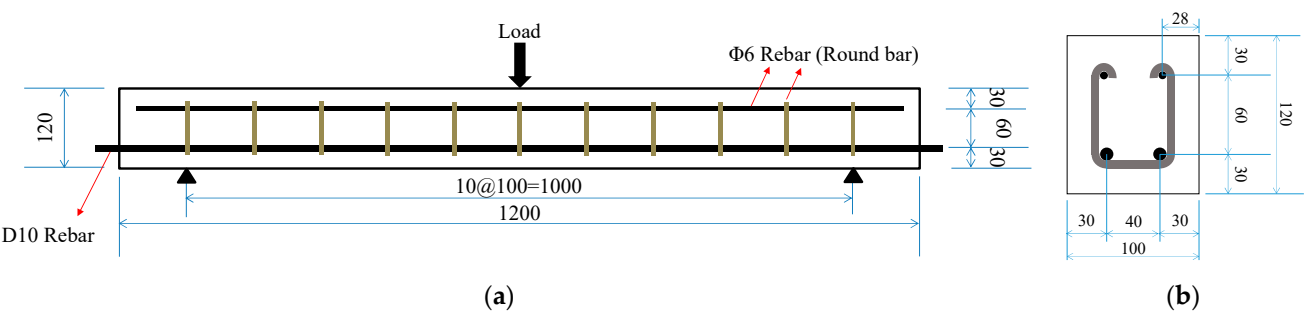


Figure 19. Shape dimensions and reinforcement status of analysis target (unit: mm): (a) Side view; (b) cross-section view.

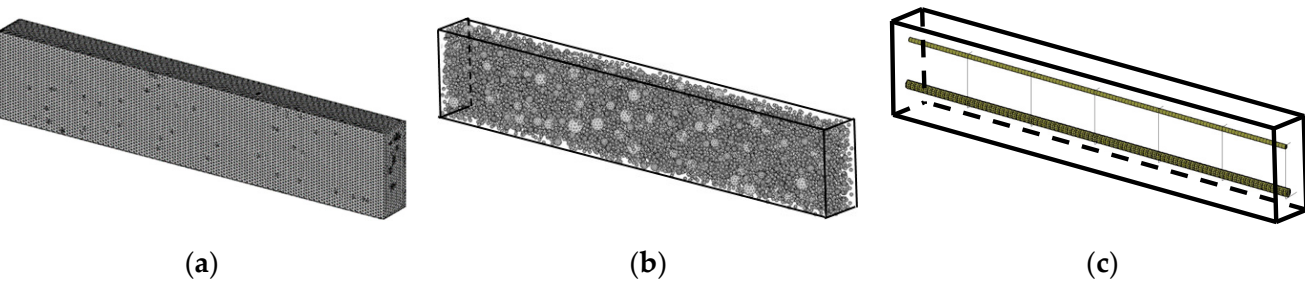


Figure 20. Analysis model diagram: (a) Overall view; (b) coarse aggregate placement status; (c) reinforcement arrangement.

4.2.2. Rebar Modeling and Material Properties

Table 4 shows the rebar’s material properties. For the constitutive law of the rebar, the bilinear stress–strain relationship shown in Figure 21 was assumed, with a hardening factor of 1/100. Yield conditions are assumed to be in accordance with von Mises.

Table 4. Material properties of rebar.

	Elastic Modulus (GPa)	Yield Strength (MPa)
Rebar	188	358

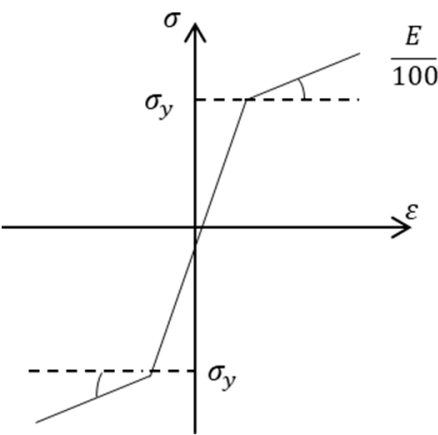


Figure 21. Stress–strain relationship in uniaxial conversion of rebar.

4.3. Modeling of ASR Expansion

This section describes a method for depicting ASR expansion in the analytical model and a coupled moisture-transfer-cracking method for precisely resolving the effect of moisture as a reaction factor in ASR degradation.

4.3.1. Moisture Movement–Crack Coupling

This study aims to develop a more complex degradation mechanism by considering that cracks in concrete affect not only the mechanical properties but also the mobility of materials in concrete. In other words, the cracked areas are considered more susceptible to moisture penetration, which is reflected in the mesoscale model. By changing the water penetration rate according to the cracks, we attempted to couple the moisture movement analysis with the stress analysis. Specifically, the cracks were considered in terms of two parameters, width and direction, and the effect of the cracks was reflected in the moisture inflow by changing the moisture diffusion coefficient based on these parameters.

First, for the moisture movement performance of concrete without cracks, the moisture permeability $K = 50 \text{ g/mm}^3 \cdot \text{hr} \cdot \text{mmHg}$ and the moisture capacity $C = 0.1 \text{ g/mm}^3 \cdot \text{mmHg}$, which were determined inverse analytically from the experiments in a previous study [37], were used to discretize Equation (9), which is portrayed as a one-dimensional diffusion equation and is solved by discretizing.

$$C \frac{\partial P}{\partial t} = K \frac{\partial^2 P}{\partial x^2} \quad (9)$$

Next, the moisture migration performance at the crack initiation zone, i.e., the coupled moisture migration–cracking method, is described. This model evaluates the crack width as the strain after the tensile strength. In other words, the point at which cracking occurs is set to zero at the tensile strength, and the crack width is considered to increase as the strain increases from that point. As shown in Figure 22, the model defines this strain as the crack strain, which is calculated in each direction. Equations for crack strain are shown in Equations (10)–(12),

$$\varepsilon_{cr_x} = \varepsilon_{p_x} - \varepsilon_0 \quad (10)$$

$$\varepsilon_{cr_y} = \varepsilon_{p_y} - \varepsilon_0 \quad (11)$$

$$\varepsilon_{cr_z} = \varepsilon_{p_z} - \varepsilon_0 \quad (12)$$

where ε_{cr_x} , ε_{cr_y} , and ε_{cr_z} are the crack strain in the x, y, and z directions, respectively; ε_{p_x} , ε_{p_y} , and ε_{p_z} are the principal strains in the x, y, and z directions, respectively; and ε_0 is the crack strain. In this model, moisture movement at the cracks is assumed to be based on Hagen–Poiseuille flow. Therefore, the x-directional moisture movement velocity u_{xy} is associated with y-directional tensile failure and the x-directional moisture movement velocity u_{xz} is associated with z-directional tensile failure, as shown in Figure 23. The following Equations (13) and (14) can be expressed,

$$u_{xy} = \frac{1}{2\rho\nu} \frac{\partial P}{\partial x} \left(y^2 - \frac{B_y^2}{4} \right) \quad (13)$$

$$u_{xz} = \frac{1}{2\rho\nu} \frac{\partial P}{\partial x} \left(z^2 - \frac{B_z^2}{4} \right) \quad (14)$$

where P is the water vapor pressure (MPa), ρ is the density of water vapor, ν is the kinematic viscosity, B_y is the crack width in the y-direction, and B_z is the crack width in the z-direction.

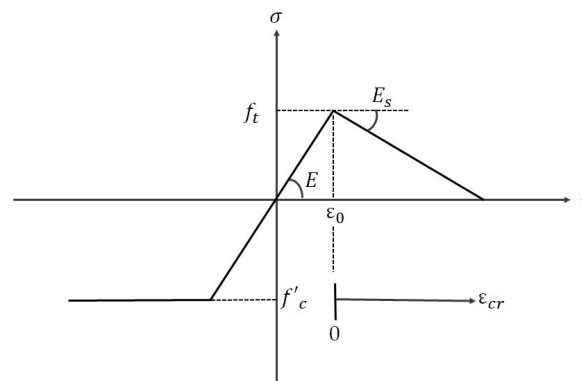


Figure 22. Stress–strain relationship in the uniaxial conversion of rebar.

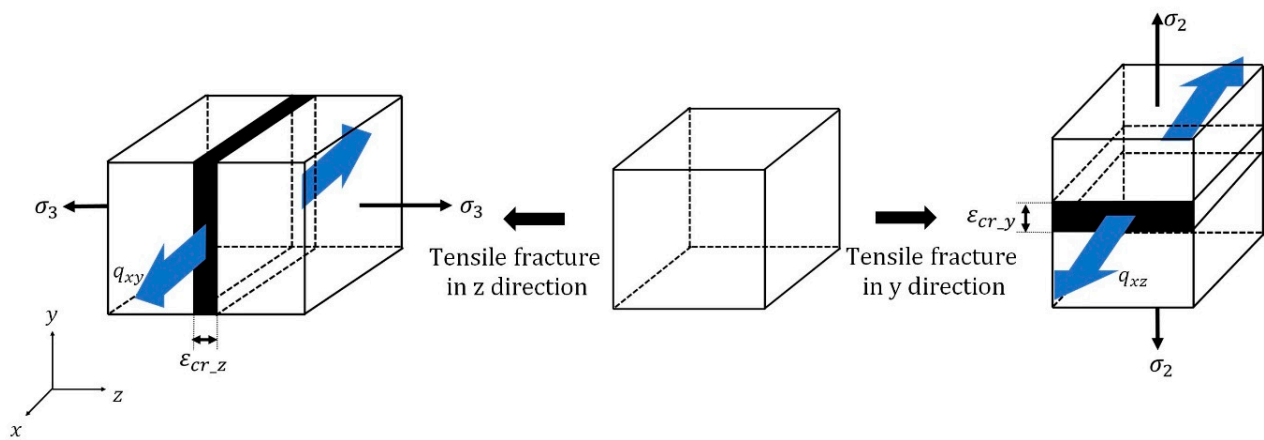


Figure 23. Cracking and permeability concept.

The area divisions of Equations (13) and (14), respectively, give the flow rate of moisture moving through the cracks. Equations (15) and (16) are shown below.

$$Q_{xy} = \int_{-\frac{B_y}{2}}^{\frac{B_y}{2}} u_{xy} dA = -\frac{L}{12\rho\nu} B_y^3 \frac{\partial P}{\partial x} \quad (15)$$

$$Q_{xz} = \int_{-\frac{B_z}{2}}^{\frac{B_z}{2}} u_{xz} dA = -\frac{L}{12\rho\nu} B_z^3 \frac{\partial P}{\partial x} \quad (16)$$

Also, considering the averaged moisture flux q at the cracked section,

$$q_{xy} = \frac{Q_{xy}}{B_y L} = -\frac{B_y^2}{12\rho\nu} \frac{\partial P}{\partial x} \quad (17)$$

$$q_{xz} = \frac{Q_{xz}}{B_z L} = -\frac{B_z^2}{12\rho\nu} \frac{\partial P}{\partial x} \quad (18)$$

Thus, the average moisture flux in the x direction is

$$q_x = q_{xy} + q_{xz} = -\frac{B_y^2 + B_z^2}{12\rho\nu} \frac{\partial P}{\partial x} \quad (19)$$

Similarly, the average fluxes in the y- and z-directions are

$$q_y = q_{yx} + q_{yz} = -\frac{B_x^2 + B_z^2}{12\rho\nu} \frac{\partial P}{\partial y} \quad (20)$$

$$q_z = q_{zx} + q_{zy} = -\frac{B_x^2 + B_y^2}{12\rho\nu} \frac{\partial P}{\partial z} \quad (21)$$

The matrix notation of Equations (19)–(21), based on the crack strain described above, can be expressed as follows:

$$\begin{aligned} \lambda_x &= \frac{B_y^2 + B_z^2}{12\rho\nu} = \frac{l_0^2}{12\rho\nu} (\varepsilon_{cr_y}^2 + \varepsilon_{cr_z}^2) \\ \lambda_y &= \frac{B_x^2 + B_z^2}{12\rho\nu} = \frac{l_0^2}{12\rho\nu} (\varepsilon_{cr_x}^2 + \varepsilon_{cr_z}^2) \\ \lambda_z &= \frac{B_x^2 + B_y^2}{12\rho\nu} = \frac{l_0^2}{12\rho\nu} (\varepsilon_{cr_x}^2 + \varepsilon_{cr_y}^2) \\ (B_x &= l_0 \varepsilon_{cr_x} \quad B_y = l_0 \varepsilon_{cr_y} \quad B_z = l_0 \varepsilon_{cr_z}) \end{aligned} \quad (22)$$

$$\begin{bmatrix} q_x \\ q_y \\ q_z \end{bmatrix} = \begin{bmatrix} -\lambda_x & 0 & 0 \\ 0 & -\lambda_y & 0 \\ 0 & 0 & -\lambda_z \end{bmatrix} \begin{bmatrix} \frac{\partial P}{\partial x} \\ \frac{\partial P}{\partial y} \\ \frac{\partial P}{\partial z} \end{bmatrix} \quad (23)$$

Here, Equations (22) and (23) are equivalent to Fick's law, which describes diffusion in proportion to the energy (concentration) gradient, thus defining the moisture diffusion coefficient λ and allowing moisture movement to be described as a diffusion equation.

Next, the direction of the crack is calculated from the direction of the principal stress and is calculated as the angle of rotation from the overall coordinate. In the analysis, the effect of the crack direction is reflected in the moisture movement by multiplying the crack direction coordinate transformation matrix r , shown in Equation (24), by the moisture transfer coefficient vector λ :

$$\begin{aligned} r &= \begin{bmatrix} \cos\psi & -\sin\psi & 0 \\ \sin\psi & \cos\psi & 0 \\ 0 & 0 & 1 \end{bmatrix} \begin{bmatrix} \cos\theta & 0 & \sin\theta \\ 0 & 1 & 0 \\ -\sin\theta & 0 & \cos\theta \end{bmatrix} \begin{bmatrix} 1 & 0 & 0 \\ 0 & \cos\varphi & -\sin\varphi \\ 0 & \sin\varphi & \cos\varphi \end{bmatrix} \\ &= \begin{bmatrix} \cos\psi\cos\theta & \cos\psi\sin\theta\sin\varphi - \cos\varphi\sin\psi & \sin\psi\sin\theta + \cos\psi\cos\varphi\sin\theta \\ \cos\theta\sin\psi & \cos\psi\cos\varphi + \sin\psi\sin\theta\sin\varphi & \cos\varphi\sin\psi\sin\theta - \cos\psi\sin\varphi \\ -\sin\theta & \cos\theta\sin\varphi & \cos\theta\cos\varphi \end{bmatrix} \end{aligned} \quad (24)$$

Therefore, considering the moisture diffusion coefficient K_{cr} , which takes into account the effect of crack width and direction, and the moisture movement through the concrete pores before crack initiation as mentioned above, the governing equation for moisture movement in concrete in this study can be expressed as Equation (25):

$$\begin{aligned} C \frac{\partial P}{\partial t} &= K_x \frac{\partial^2 P}{\partial x^2} + K_y \frac{\partial^2 P}{\partial y^2} + K_z \frac{\partial^2 P}{\partial z^2} \\ K &= \begin{bmatrix} K_x \\ K_y \\ K_z \end{bmatrix} = K_0 + K_{cr} = \begin{bmatrix} K_{x0} \\ K_{y0} \\ K_{z0} \end{bmatrix} + \begin{bmatrix} K_{x_cr} \\ K_{y_cr} \\ K_{z_cr} \end{bmatrix} \end{aligned} \quad (25)$$

4.3.2. Modeling of ASR Aggregate Expansion

Because this study aims to understand analytically the change in mechanical behavior of concrete due to ASR, ASR is considered as aggregate expansion and is represented as a mechanical model based on a three-dimensional mesoscale. In the following, we

describe how aggregate expansion is represented in the actual analysis. In this study, ASR is considered as aggregate expansion, and the expansion is given as an increase in the volumetric strain of the aggregate. In general, it is more realistic to treat ASR gel in the analysis as an actual porous material that can hold stress, and such a model is also considered in the existing research [38,39]. It is said that there are two types of expansion models for ASR gel in the analysis: a reaction rim model and a gel pocket model [24]. In this study, in order to model the expansion state of the reaction rim model, the above-mentioned expansion-giving method is adopted. The model was developed considering water and alkali content as the strain parameters. The model was developed by referring to papers by Ueda et al. [40] and Muranaka et al. [41]. Figure 24 shows a schematic diagram of ASR expansion.

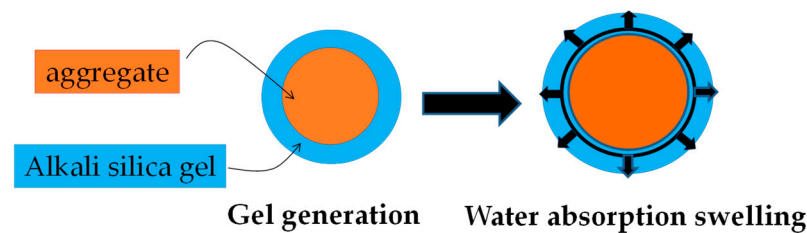


Figure 24. Cracking and permeability concept.

In this study, ASR is considered as a two-step reaction. In the first step, the reactive silica of the aggregate reacts with alkali to form an alkali–silica gel. Gel formation is assumed to be isotropic, as shown in the figure, and the amount of gel formed is given by Equation (26),

$$T_{pr} = \int_0^t k_{gel} \times C \times A \times dt \quad (26)$$

where T_{pr} is the amount of ASR gel produced (-), A is the unit reaction coarse aggregate volume (g/L), dt is the time increment (T), k_{gel} is the coefficient of the expansibility of the gel (-), and C is the alkaline ion concentration (mol/L). At this point, the gel is assumed not to affect the deformation of the concrete, and no aggregate expansion is assumed to have occurred. In the second stage, the gel generated in the first stage absorbs water and causes volumetric expansion strain in the aggregate. The volumetric expansion strain is given by Equation (27),

$$\varepsilon_V = f(C, \Delta R) = k_{gel} \times \left(\frac{E_{gel}}{E_{agg}} \right) \times T_{pr}(C) \times \Delta R \quad (27)$$

where ΔR is the change in relative moisture content of the reactive aggregate (-), k_{gel} is the coefficient on the expansibility of the gel (-), E_{gel} is the Young's modulus of the gel, and E_{agg} is the Young's modulus of the reactive coarse aggregate. As shown above, the aggregate volume expansion strain could be expressed as a function of the alkali and moisture content. In the present paper, the objective is to evaluate the effect of moisture on ASR at the three-dimensional mesoscale and the relationship between aggregate expansion, cracking properties, and load-bearing capacity:

$$\alpha = k_{gel} \times \left(\frac{E_{gel}}{E_{agg}} \right) \times T_{pr}(C) \quad (28)$$

And then, the volume expansion strain ε_V is

$$\varepsilon_V = f(C, \Delta R) = \alpha \Delta R \quad (29)$$

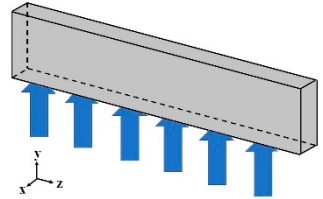
In other words, the ASR phenomenon assumes that aggregate expansion occurs as a result of the action of moisture only on concrete that has already produced a certain amount of alkali–silica gel. In this case, α is a function of the alkali content and is a constant

that determines the expansion based on the alkali content because it is a coefficient for the change in relative humidity.

4.4. Analysis Cases

This study aimed to clarify the relationships between aggregate expansion due to ASR, cracking, and load-carrying capacity. The study focuses on the amount of aggregate expansion, which is one of the indicators of ASR deterioration. It examines how the amount of expansion affects the load-bearing capacity of concrete. Because ASR degradation is a phenomenon in which alkali–silica gel absorbs water and expands as water enters the concrete, it can be said to depend on the way water moves. Therefore, this analysis examined how the cracking behavior associated with ASR deterioration changes depending on the method of water inflow and how it affects the load-bearing capacity of the concrete. In other words, this analysis is a parametric study in which the expansion method, the amount of expansion given to one aggregate, and the method of water inflow are used as parameters. The analysis cases are shown in Table 5.

Table 5. Analysis case.

Case	Expansion Type	α	Moisture Inflow Type
Sound	---	---	---
Constant 1	Constant expansion	0.006	---
Constant 2		0.008	
From the bottom	Moisture dependent	0.006	
From the top		0.006	

Two cases were considered for the expansion method: the constant expansion case, in which the expansion is not dependent on moisture content and is caused by a direct change in relative humidity as a boundary condition, and the moisture-dependent case, in which the expansion is caused based on the change in relative humidity obtained from the moisture transfer analysis assuming moisture inflow from the RC beam surface. The amount of expansion per aggregate was adjusted using α , the expansion performance described in Section 4.3.2, as a parameter. This study assumed an alkali content of 4.0 kg/m³ and 5.5 kg/m³ for the ASR-deteriorated RC beams. Therefore, as described in Section 3, α was calculated backward from the surface expansion of the cylindrical specimens. As a result, $\alpha = 0.006$ for an alkali content of 4.0 kg/m³ and $\alpha = 0.008$ for an alkali content of 5.5 kg/m³ were used as analytical parameters. As for the water inflow conditions, as shown in Table 5, bottom and top inflow in the y-direction, inflow in the x-direction, and inflow in the z-direction were considered.

4.5. Results and Discussion

4.5.1. Deterioration Due to Aggregate Expansion in Each Case

Figures 25 and 26 show the cracking behavior and relative humidity distribution. Figure 25 shows that cracks appeared simultaneously around the entire aggregate at

constant expansion, and that these cracks tended to propagate and connect. On the other hand, in the case of water absorption expansion, which assumes moisture transfer from the bottom and top surfaces, cracks in the y-direction were more pronounced, showing a different tendency from that of constant expansion. The relative humidity distribution map shows that the moisture transfer performance is affected by the cracks, confirming the coupling of stress (cracks) and moisture transfer. Comparing the crack distribution map and the relative humidity distribution map, the relative humidity tends to be lower in the cracked area due to the increase in moisture transfer performance.

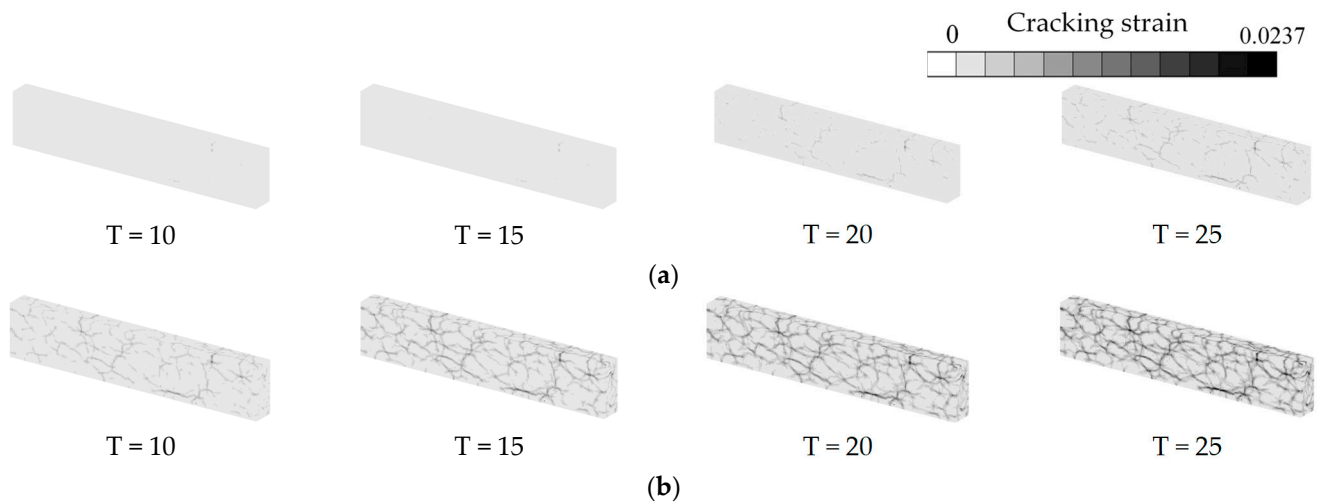


Figure 25. Crack distribution: (a) Constant 1; (b) Constant 2.

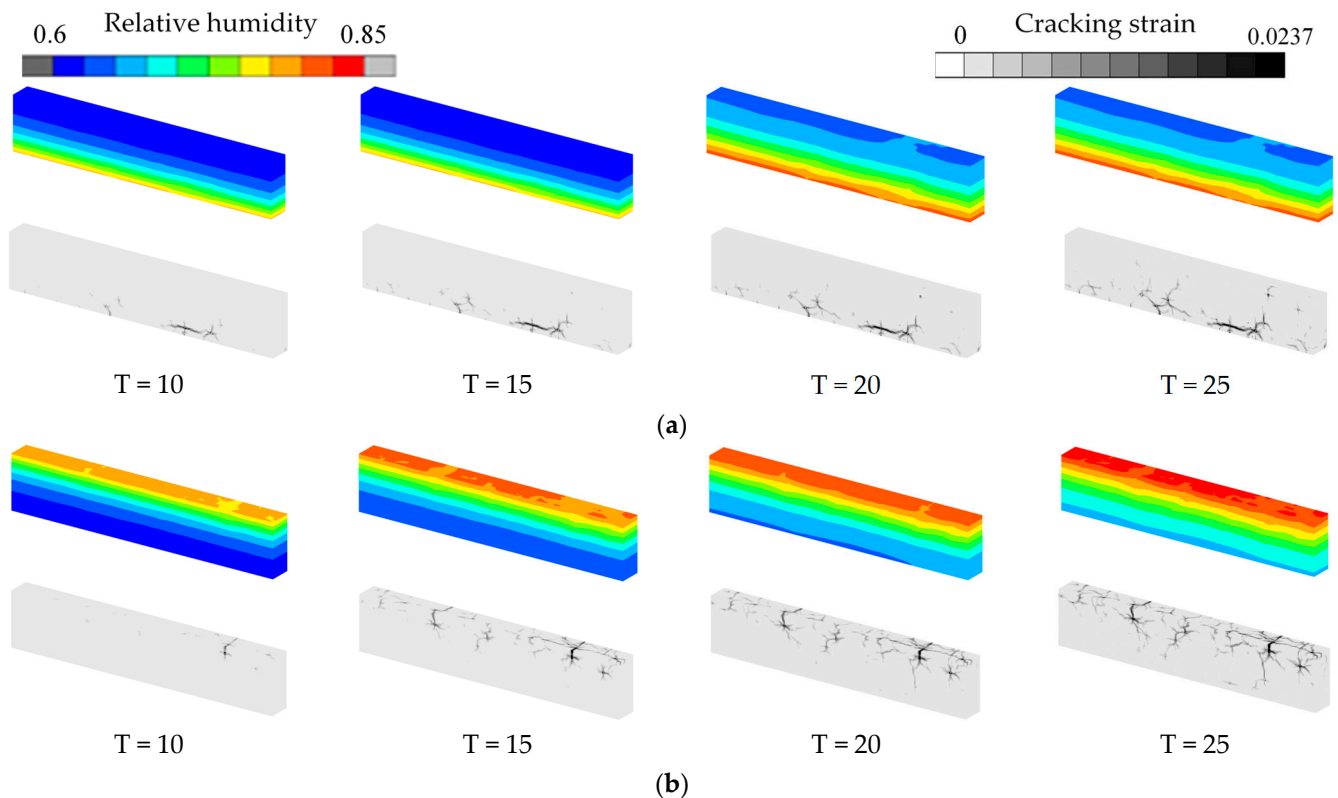


Figure 26. Humidity and crack distribution: (a) From the bottom; (b) from the top.

4.5.2. ASR-Deteriorated RC Beams with Respect to Flexural Load-Carrying Capacity

Cracking behavior and load-displacement curves at the ultimate state of a sound and an ASR-deteriorated RC beam subjected to bending loading analysis are shown.

(1) Constant expansion

To investigate the relationship between the amount of aggregate expansion and load-carrying capacity, results for the cases of Constant 1 and 2 and soundness are shown in Figures 27 and 28. In case of Constant 2, cracks in the shape of a cracked shell due to ASR are conspicuous, and the expansion cracks show fracture propagation with bending load. The load-displacement curve shows that the cracks were caused by expansion cracks. The load-displacement curves for Constant 1 and 2 show that the initial stiffness of the expanded and deteriorated cases decreased compared with the healthy cases, but the maximum load increased. This is thought to be due to the internal stress generated by the expansion of the aggregate, which exerted a restraining force on the rebar, a kind of chemical prestress.

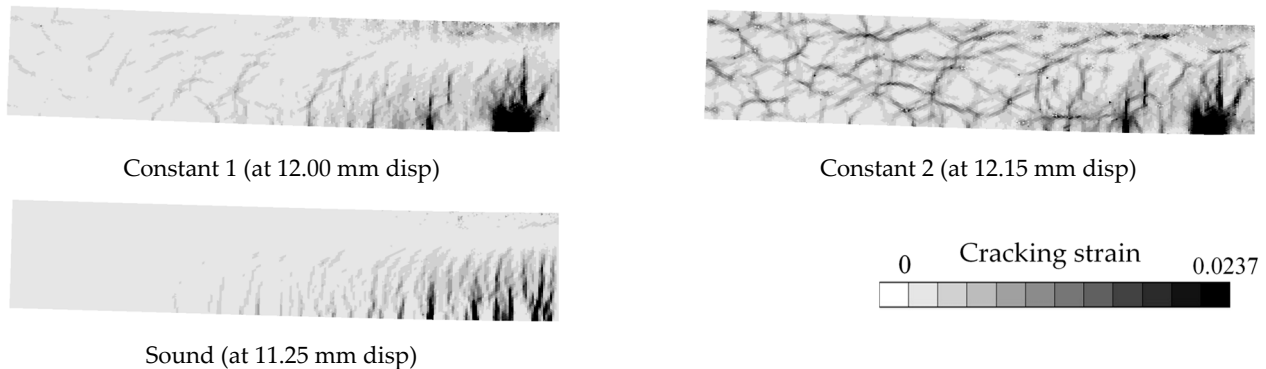


Figure 27. Bending fracture situation.

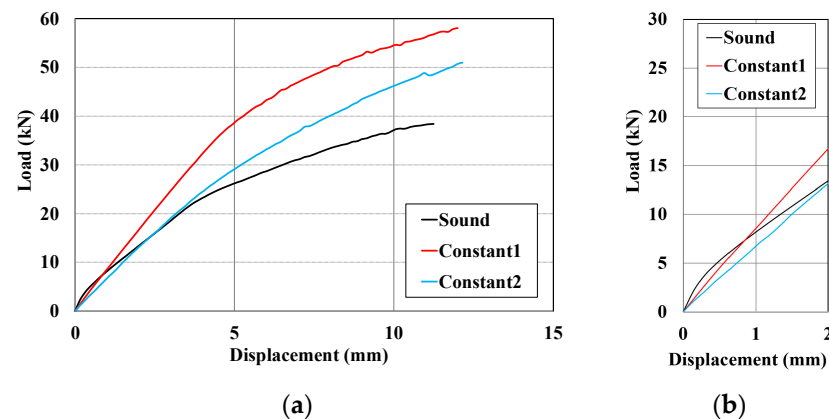


Figure 28. Load-displacement curve: (a) Whole trend; (b) initial stiffness.

(2) Water-Dependent

To examine the relationship between water inflow conditions and load-carrying capacity, results for cases from the bottom and from the top are shown in Figures 29 and 30. In most cases, an increase in maximum load was observed, but in the case from the top, a decrease in maximum load was observed due to a decrease in toughness compared with the healthy case. This is due to cracks generated by ASR expansion that propagated with bending loading and became macroscopic cracks.

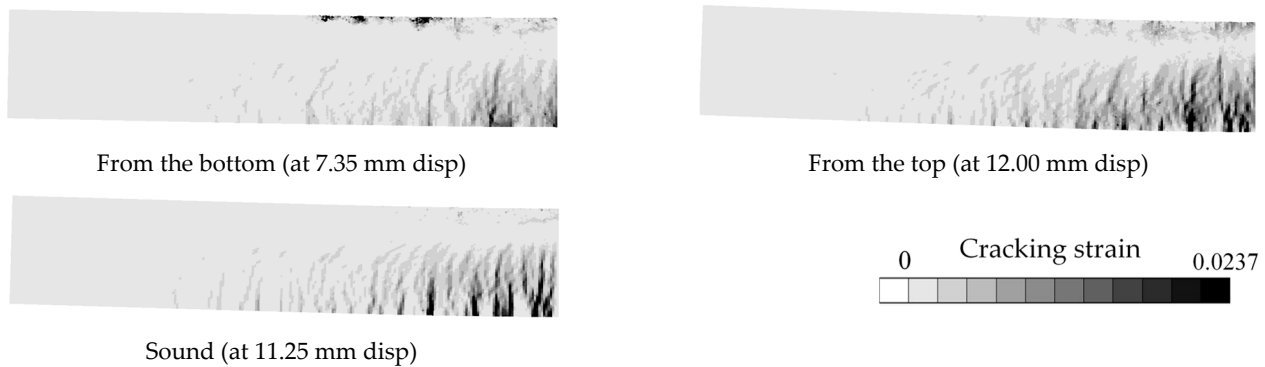


Figure 29. Bending fracture situation.

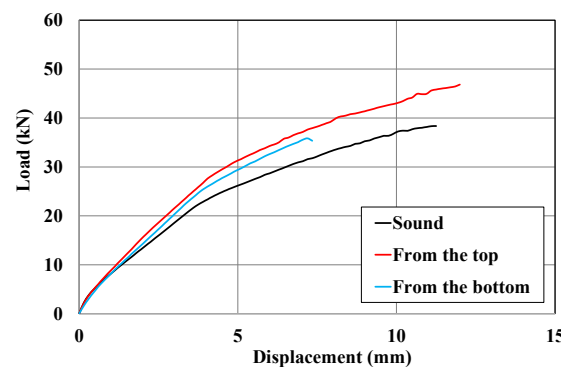


Figure 30. Load-displacement curve.

4.5.3. Relationship between Aggregate Expansion Cracks Associated with ASR Deterioration and the Load-Carrying Capacity of RC Beams

In this analysis, an RC beam model was created to reproduce ASR degradation, and the relationship between ASR degradation and load-carrying capacity was investigated. As a result, the failure behavior was obtained depending on the method of expansion, and the relationship between the method of ASR deterioration and the load-bearing capacity was also obtained.

In the constant-expansion case, the maximum load capacity increased while the initial stiffness decreased in the expansion-degraded case. This may be due to the predominant effect of the chemical prestress caused by the expansion rather than the effect of cracking due to ASR degradation [42]. However, the microcracks propagated with bending loading and became macroscopic cracks, decreasing the maximum load. Leaving ASR-deteriorated cracks unattended is not advisable for structural performance reasons.

The figure focusing on the water inflow condition shows that the top surface inflow increased the load-carrying capacity compared with the sound case. This can be attributed to the fact that the expansion of the top surface of the RC beams causes a convex upward warping deformation, which results in a resistance force against bending. The bottom inflow had little effect on the load-bearing capacity between 0 and 4 mm, and failure occurred at a displacement of about 7.5 mm. This is considered to be because fine cracks were generated at the bottom of the RC beams. Although these fine cracks had little effect on the load-bearing capacity, failure from the fine cracks progressed with loading, resulting in a decrease in the maximum load and failure.

In the moisture-dependent case, the amount of cracking was small in the case of inflow from the z-direction, and cracking occurred only at the ends of the beams. This result suggests that the amount of cracking is not the only factor that affects the load-carrying capacity; factors other than cracking, such as chemical prestress and boundary conditions, have a strong influence.

In addition, the model assumed perfect adhesion between the rebar and mortar, which did not accurately reflect the decrease in rebar adhesion due to ASR deterioration. Thus, the load-bearing capacity did not decrease.

4.6. Challenges and Future Prospects

The model proposed in this study provided some insight into the cracking behavior associated with ASR expansion and the relationship between ASR expansion and the flexural fracture behavior of RC beams. It is expected that the results obtained will contribute to the quantitative evaluation of the remaining load-bearing capacity of actual structural members and structures that have been degraded by ASR. However, in order to obtain a more accurate assessment, several issues must be resolved in the future. For example, problems with the constitutive law and the modeling of the steel bars meant that the effect of expansion cracking was not properly reflected in the mechanical properties, increasing the maximum load in cases where expansion deterioration occurred.

One of the possible methods to reconsider the constitutive law is to change the currently applied linear Drucker–Prager yielding condition and apply a nonlinear constitutive law that can reproduce plastic behavior in more detail. However, the RC beam model proposed in this analysis has many elements, and it is difficult to directly apply a nonlinear constitutive law from the viewpoint of convergence. Therefore, generating a mesh partition with an improved element aspect ratio and reducing the number of elements is necessary to create a model that can more straightforwardly depict fracture.

The future prospect of this study is to propose a model to reproduce the effects of combined deterioration (steel bar corrosion) associated with ASR deterioration, that is, to apply a model of steel corrosion to the model proposed in this study in order to understand the effects of combined deterioration on structural performance quantitatively. Specifically, the simplest method is to apply the expansion with moisture as a parameter based on the solid-type rebar model proposed in this study. In addition, we believe that a more precise ASR degradation can be reproduced by applying an expansion model with alkali as a parameter instead of the proposed single-dependent formula for aggregate expansion that is currently applied.

5. Conclusions

This paper aims to evaluate the quantitative impact of ASR on load-bearing capacity because ASR has become an issue in the maintenance and management of social infrastructure. In this study, we first proposed a concrete mesoscale model to reproduce ASR degradation, identified the material model, and reproduced its mechanical behavior based on uniaxial compression tests of cylindrical specimens. Furthermore, an RC beam model using the mesoscale model was developed as a fundamental study, and the effects of various ASR degradation factors on load-bearing performance were examined. The following is a summary of the conclusions obtained from this study.

1. A mesoscale model consisting of three phases, namely, coarse aggregate, mortar paste, and ITZ, was proposed, which can take into account the coupling between water movement and cracks specific to ASR.
2. The material parameters were identified based on uniaxial compression tests of cylindrical specimens subjected to expansion and deterioration, and it was shown that the proposed model can reproduce the changes in compressive properties due to expansion and cracking.
3. A mesoscale model of an RC beam was constructed using a rebar mesh that reproduced the shape of the nodes and ribs in detail. A coupled analysis of moisture movement and expansion using this model clarified the effects of moisture movement and expansion cracks.
4. Although the initial stiffness of the flexural load-bearing capacity of ASR-deteriorated RC beams decreased, no decrease in load-bearing capacity was observed, and it was shown that there is a possibility that it may actually improve. This is thought to be due

to the effects of the analysis model, such as the bond between the rebar and concrete, as well as the effects of chemical prestress due to expansion.

5. Even in cases where the amount of cracking was small, a decrease in maximum load and toughness was observed, and it is considered inappropriate for maintenance to evaluate the degree of decrease in load-carrying capacity simply based on the amount of cracking.

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