Scientific paper

Meso-Scale Modelling of the Mechanical Properties of Concrete Affected by Radiation-Induced Aggregate Expansion

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Abstract

To evaluate the radiation-induced degradation of concrete, a rigid-body spring network model is introduced that takes into account the three phases in concrete: mortar, aggregate, and the interfacial transition zone. The proposed model enables evaluation of the change in the physical properties of concrete affected by aggregate expansion under the free restraint condition. Good agreement with previous experimental data is found for the linear expansion of the concrete specimen and the compressive strength, Young’s modulus, and splitting tensile strength. Based on the numerical results, it is concluded that, to reproduce the physical property changes in concrete, the expansion of mortar due to the radiation-induced expansion of fine aggregate and/or creep behavior must be considered. In addition, it is clarified that an isolated expansion of mortar with a lack of expansion in the coarse aggregate also degrades the concrete and, consequently, analysis of the type of aggregate used is critical for predicting the properties of concrete under neutron irradiation. Furthermore, the impact of inhomogeneous expansion of rock-forming minerals in coarse aggregates on physical property changes is studied, showing that such a partial expansion in the aggregates and the resultant cracks in aggregates greatly influences the reduction of the Young’s modulus, with minimal impact on the reduction of compressive strength. The proposed model can be used to evaluate concrete degradation due to radiation-induced volumetric expansion of aggregate caused by the metamictization of rock-forming minerals.

1. Introduction

As demand for the long-term operation of nuclear power plants (NPP) grows, there is an increasing need to understand the deterioration mechanism of concrete in NPPs. Moreover, there is a global push to extend the operating lifetimes of aging NPPs (e.g., several NPPs are more than forty-years old). For example, in the United States, many NPPs have been approved to extend their operating lifetime to sixty years and efforts are underway to extend their operation lifetimes beyond sixty years. Additionally, in Japan, several NPPs have been approved for operation up to sixty years. To operate NPPs safely for an extended period, it is necessary to understand and predict the effects of long-term operation on the members and equipment in NPPs.

As concrete is a major structural component of NPP buildings and many pieces of equipment and facilities are intricately intertwined with structural concrete members, the concrete members are key components needed for long-term operation and are hardly replaceable. From this viewpoint, the degradation of concrete is a major issue for the long-term operation of NPPs (Sasano et al. 2018; Rymeš et al. 2019).

One of the specific environmental conditions causing a degradation of concrete in a NPP is radiation. According to previous acceleration experiments (Pedersen 1971; Elleuch et al. 1972; Hilsdorf et al. 1978; Field et al. 2015; Maruyama et al. 2017), concrete has been confirmed to deteriorate under neutron irradiation. The mechanism behind this degradation is 1) expansion of aggregates due to metamictization of rock-forming minerals, 2) shrinkage of hardened cement paste (hcp) due to gamma-heating and the resultant drying, and 3) cracking in hcp due to volumetric mismatch between the aggregates and hcp (Maruyama et al. 2017). Neutron bombardment and radiolysis changes the lattice structure of rock-forming minerals (Primak et al. 1955; Bykov et al. 1981; Douillard and Duraud 1996; Hobbs et al. 1998), with silicon-oxide covalent bonds being highly affected. As a result, α-quartz is the primary rock-forming mineral that exhibits a large expansion under neutron irradiation, i.e., increasing by approximately 18% in volume (Bykov et al. 1981).

For concrete engineering, it is important to understand the mechanisms behind the metamictization and volume

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change of rock-forming minerals, the radiation-induced volume expansion (RIVE) of aggregate, and the change in the mechanical properties of concrete. The evaluation of concrete property changes is challenging because of the limited number and size of specimens in accelerated irradiation experiments. As such experiments are always accompanied by gamma irradiation (including secondary gamma effects) and gamma heating, the specimen size is always limited to avoid an undesirable temperature increase. In addition, the specifications of research reactors impose limitations on experiments. Consequently, a method to extrapolate or evaluate concrete property changes based on the obtained data is necessary to connect the results of acceleration experiments and the structural evaluation of reinforced concrete members for the long-term operation of NPPs.

Currently, no study has numerically evaluated the compressive strength of concrete subjected to neutron irradiation, which is the most basic and widely used parameter for the design of reinforced concrete members, and while some numerical studies have evaluated the mechanical properties of concrete subjected to neutron irradiation, most of these have only predicted the volume changes due to gamma heating is minimized (temperature kept under 75°C, which is relatively close to the actual conditions of NPPs). In the experiment, two types of concrete with different coarse aggregates, six different aggregates, and hcp specimens were irradiated by neutron and gamma rays with four averaged fast neutron fluences ($7.76 \times 10^{18}$, $1.14 \times 10^{19}$, $4.52 \times 10^{19}$, and $9.11 \times 10^{19}$ n/cm$^2$, $> 0.1$ MeV).

The mixture proportion and aggregate phase composition as determined by XRD and Rietveld analysis (Maruyama et al. 2017).

### Table 1 The mixture proportions of the simulated concrete Con-A (Maruyama et al. 2017), where S is land sand and GA is altered tuff for the coarse aggregate and $V_{GA}$, $V_S$, and $V_W$ are the volumes of GA, S, and W, respectively.

| w/c | s/a | Unit mass | Air (%) | $V_{GA}$ | $V_S/(V_C+V_W+V_S+Air)$ |
|-----|-----|-----------|---------|----------|------------------------|
| 50  | 45  | 18.3      | 799     | 995      | 2.6                    |

### Table 2 Phase composition of the aggregate as determined by XRD and Rietveld analysis (Maruyama et al. 2017).

| Minerals (mass %) | Chlorite | Illite | Sericite | Biotite | Microcline | Orthoclase | Anorthoclase | Anorthite | Albite | Quartz | SUM |
|-------------------|----------|--------|----------|---------|------------|------------|--------------|-----------|-------|--------|------|
| GA                | 0.5      | ±0.27  | -        | ±0.44   | 3.03       | 0.5        | 0.61         | 2.3       | 0.7   | 91.85  | 100  |
| ±0.63             | ±0.07    | ±0.48  | ±0.58    | ±0.21   | ±1.92  |
| ±0.61             | ±1.07    | ±1.08  | ±0.68    | ±0.7    |
| ±1.45             | ±1.92    | ±0.68  | ±0.7     |
| ±1.19             | ±0.68    | ±100   |

2. Numerical calculation

2.1 Reference experiment

As reference data for numerical calculations, the experimental results of Maruyama et al. (2017) was employed, because the information necessary for mesoscale analysis (e.g., mechanical properties and volumetric changes of aggregate and concrete) are included in these data, the impacts of temperature and gamma-ray and neutron irradiation are separated, and the temperature rise due to gamma heating is minimized (temperature kept under 75°C, which is relatively close to the actual conditions of NPPs). In the experiment, two types of concrete with different coarse aggregates, six different aggregates, and hcp specimens were irradiated by neutron and gamma rays with four averaged fast neutron fluences ($7.76 \times 10^{18}$, $1.14 \times 10^{19}$, $4.52 \times 10^{19}$, and $9.11 \times 10^{19}$ n/cm$^2$, $> 0.1$ MeV).

The mixture proportion and aggregate phase composition as determined by powder X-ray diffraction (XRD) and Rietveld analysis are shown in Tables 1 and 2, respectively. The target concrete (Con-A in Maruyama et al. (2017)) was made from high-early-strength Portland cement (C) (noted as H in Chapter 2 of Maruyama et al. (2017)), land sand (S), and crushed stone from thermally altered tuff with a full of quartz small grains (GA) (GA in Chapter 2 and G3 in Chapter 5 of Maruyama et al. (2017)). The densities of C, S, and GA were 3.14 g/cm$^3$, 2.61 g/cm$^3$, and 2.66 g/cm$^3$, respectively, with the grain size of GA being 5–13 mm. To avoid the strength development due to hydration during irradiation,
high-early-strength Portland cement was used with a water-to-binder ratio of 0.5, cured under the sealed condition for one year at 20°C.

As shown in Table 2, the coarse aggregate used mostly consists of quartz, with the sand contains quartz at approximately 50% by mass. Assuming the quartz is expanded by neutron irradiation, both the coarse aggregate and the concrete mortar were considered to expand.

The specimen size was φ40 × 60 mm³, owing to the specific hole of the research reactor, the reduction of the temperature difference between the surface and central portion of the specimen, and the number of specimens that were to be irradiated. The length change, mass change, Young’s modulus, and compressive strength were measured. It should be noted that the Young’s modulus and compressive strength were separately obtained, and the stress–strain curve could not be obtained because of the limitations of the facility.

During irradiation, specimens were installed in capsules (Maruyama et al. 2017). A valve attached to the capsules released the gases generated by the irradiation, allowing the drying of the samples to proceed. Due to the gamma-ray heating, the temperature of the specimen core was higher than that of the peripheral regions. The temperature inside the samples at the top of the capsule was measured, with a monitoring wire for gamma-ray and neutron fluences providing the temperature distribution for all the samples. The temperatures at the sample centers and surfaces and the maximum temperature difference of the specimens were 63.2–72.6, 57.6–62.6, and 6.2–10.3°C.

The experimental results for Con-A are summarized in Fig. 1. After irradiation, the samples were dried and approximately 3.5% of mass was lost. This is due to that the neutron and the gamma-ray produced heat and radiolysis in the specimens, and pressures built by water vapor, H₂ and O₂ gasses was controlled by releasing the gasses from the capsule. It should be noted that the specimens after the irradiation was dried at 76°C for 3 weeks to ensure the uniform water distribution inside specimen and the same C-S-H alteration due to drying which affects the physical properties of mortar (Maruyama et al. 2014), for all the irradiated concrete specimens. The specimens exhibited a 1% dilation in length and attained a compressive strength ratio (Fc/Fco) and Young’s modulus ratio (Ec/Eco) of 56% and 28%, respectively, at a fluence of 4.52 × 10¹⁹ n/cm². Due to the expansion of the aggregate, concrete was degraded.

For the fast neutron fluence of 9.11 × 10¹⁹ n/cm², it should be noted that the samples were strongly confined by the capsule as the capsule was broken by the expansion of concrete. Therefore, the sample condition at the fast neutron fluence of 9.11 × 10¹⁹ n/cm² was not equal to the other neutron irradiation fluences.

2.2 Simulation strategy
The objective of this study is to understand from an engineering viewpoint the mechanical property changes of concrete caused by the RIVE of aggregate. In this study, the expansion of aggregate elements is first considered to explore the impact of neutron irradiation. The relationship between neutron fluence, neutron energy distribution, the annealing mechanism, the metamictization process, and the resultant RIVE of rock-forming minerals and aggregate will not be considered in this study (Rosseel et al. 2016; Maruyama et al. 2017). As the dimensional change of concrete specimens is the easiest parameter of property change to measure after irradiation,
in this study, the compressive strength ratio is evaluated as a function of the linear expansion of the specimens, as in previous studies (Dubrovskii et al. 1966, 1970; Elleuch et al. 1971; Maruyama et al. 2017). The linear expansion of concrete is a consequence of the damage accumulated from the inhomogeneous volumetric change of concrete components. If one component expands and the expansion strain is restrained by the other components, then, under the self-stress balance condition, the expansion strain is correlated with the created total crack openings in the concrete (Le Pape et al. 2015). In other words, the linear expansion of concrete under the free restraint condition could provide as a first-order approximation an index for the damage in the concrete. From this viewpoint, the data in the available literature are summarized in Fig. 2. These results confirm the consistent trend of the strength ratio ($f_c/f_{c0}$) decreasing as the linear expansion strain is increased. In this figure, the lower limit curve derived by the existing data is shown (Maruyama et al. 2017). Based on the researches by Dubrovskii et al. (1966, and 1970) and the evidence that the high pressure on the fractured hcp causes the cohesion between fragments and strength can be developed in hcp (Feldman and Sereda 1970), when the expansion of concrete is restrained by the capsule, the concrete strength is considered to increase. Therefore, the low limit curve indicates the compressive strength change due to expansion under the free restraint condition.

The calculation procedure follows the experimental procedure. The calculation starts with a concrete cylinder specimen in a matured state. 1) The temperature in-}
the relationship between the properties (related to Section 4.3) were also performed.

Considerations of a)–d) will investigate the impact of the mixture proportion of concrete and type of aggregate. For a) and b), as with the aggregate used in the reference experiment (GA in Table 2), the coarse aggregate was assumed to be thermally altered tuff that contains very small homogenous quartz grains for an aggregate comprising 100% quartz, and the homogeneous expansion was introduced to the coarse aggregate element. The analysis in c) uses concrete with calcite for the coarse aggregate and sandstone for the fine aggregate. As ionic crystals are tolerant to neutron irradiation as compared to crystals containing covalent bonds, a pure calcite aggregate will not demonstrate any expansion within the range of neutron fluence for an ~100 year operational lifetime of commercial plants. Therefore, when calcite is used for the coarse aggregate in concrete and sandstone is used for the fine aggregate, expansion of the mortar will occur while expansion of the coarse aggregate will not occur. Such a scenario is possible in the plants and is taken into consideration in the simulation parameters.

The analysis in d) considers concrete with granite as a coarse aggregate and an inhomogeneous volumetric expansion of the grains (large grain size, inhomogeneous expansion of grains). Due to the large grain size of rock-forming minerals in the granite, cracks can develop inside the aggregate due to uneven volumetric expansion of different rock-forming minerals and anisotropic expansion of rock-forming minerals. The increase in apparent aggregate volume due to crack openings and the reduction of stiffness and strength of aggregates after irradiation might have an impact on the physical property changes of concrete. The influence of creep on the physical property changes of concrete is also evaluated.

Finally, a series of analyses was conducted to obtain mutual relationships between the concrete properties (Young’s modulus, splitting tensile strength, and strain at the compressive strength as a function of compressive strength). For this analysis, a Φ75 × 150 mm³ cylinder specimen with an aspect ratio of 2.0 was used following the general properties of concrete. The same material properties and particle size distribution of aggregate were used to perform the splitting tensile strength analysis (described in Section 2.7) with a specimen size of Φ100 × 25 mm³.

2.3 RBSM constitutive laws

In this study, an RBSM (Kawai 1978) was used for the analysis method, in which each element is assumed to be a rigid body and nonlinear constitutive laws are given to the springs between the elements to represent the fracture behavior of the analysis object (Fig. 3). Discontinuous behavior of concrete, such as cracking, can be reproduced appropriately (Bolander and Saito 1998; Nagai et al. 2005; Ueda et al. 2009; Gong et al. 2017). For example, it is possible to reproduce the post-peak load deformation and fracture behavior of concrete specimens under unconfined compressive stress and for confined concrete (Yamamoto et al. 2008), which is difficult to analyze with finite element methods (FEM) that assume continuous elements.

The concrete specimen was divided into mortar and coarse aggregate elements in order to represent explicitly the difference in volumetric change between the coarse aggregate and mortar. At the aggregate-mortar interface, the “wall effect” of the cement particles reduces the density of the hcp, resulting in a porosity two to three times greater than that of the bulk hcp (Ollivier et al. 1995). Since it has been confirmed from previous studies that the physical properties of the hcp in the ITZ are lower than those in the bulk cement paste, a constitutive law that takes into account the influence of the ITZ was introduced.

2.3.1 Instantaneous constitutive laws for the mortar and aggregate

The constitutive laws for springs connecting mortar elements or aggregate elements to model instantaneous behavior are based on existing laws (Yamamoto et al. 2008, 2014). A schematic of these constitutive laws and the equations are shown in Fig. 4. In this figure, $\sigma$ is the stress of the normal springs; $\varepsilon$ is the strain of the normal springs; $\varepsilon_i$ is the strain at the tensile strength of the normal springs ($f_{\text{n}}$, $f_{\text{th}}$, and $f_{\text{ITZ}}$); $w_{crw}$ is the crack width of the normal springs; $h$ is the length of a spring (mm) (see Fig. 3(b)); $\tau$ is the stress of the shear springs (shear stress); $\gamma$ is the strain of the shear springs; $\tau_i$ is the shear strength of the shear springs; $r_{\tau}$ is the shear strain at $\tau_i$ of the shear springs; $\sigma_{crw}$ is the constant to determine the limit of the shear strength increase; $\phi$ is the internal friction angle of the shear springs; $K^*$ is the softening slope of the shear springs; $\beta_0$, $\chi$, and $\beta_{\text{max}}$ are the constants for the shear softening slope ($K^*$); and
\( \beta_{cr} \) is the shear reduction factor (following the crack width, the shear stress is reduced); \( k^* \) is a constant for \( \beta_{cr} \); \( E_{cm} \) and \( E_{ca} \) are the Young’s moduli of the RBSM normal spring for the mortar and aggregate, respectively; \( f_{cm} \) and \( f_{ca} \) is the tensile strength for the RBSM normal spring of the mortar and aggregate, respectively; \( G_{cm} \) and \( G_{ca} \) are the fracture energies for the RBSM normal spring of the mortar and aggregate, respectively; and \( c_{cm}^* \) and \( c_{ca}^* \) is the cohesion of the RBSM shear spring of the mortar and aggregate, respectively. It should be noted that * denotes parameters related to the springs.

As shown in Fig. 4, \( \tau_f^* \) follows the Mohr–Coulomb yield criterion (Fig. 4(d)) with a limit of \( \sigma_f^* \), the tensile softening behavior follows the 1/4 model using fracture energy (Fig. 4(a)), elastic behavior was assumed in compression (i.e., failure occurs in tension or shear), and tensile cracking of the normal spring reduces the shear stress (Fig. 4(e)).

Figure 5 shows the cyclic behavior of the normal and shear springs, which was originally proposed by Yamamoto et al. (2014). During the cyclic behavior, the unloading stiffness of the normal spring is equal to the elastic modulus and the reloading stiffnesses is the slope connecting the origin of the stress–strain curve and the node at the maximum strain. The unloading and reloading stiffnesses of the shear spring are equal to the shear modulus and the shear stress was cut off by the stress at the shear strength.

**Fig. 4 Constitutive laws for the mortar and aggregate (Yamamoto et al. 2008, 2014).** (a) Tensile model of the normal spring, (b) shear spring model, (c) softening coefficient of the shear spring, (d) Mohr–Coulomb criteria for the shear spring, and (e) shear reduction coefficient. \( E_c^* \) indicates \( E_{cm} \) in the case of mortar and \( E_{ca} \) in the case of aggregate. For \( f_{cm}^* \), \( G_{cm}^* \), \( G_{ca}^* \), and \( c^* \) the same notation is applied.

**Fig. 5 Hysteresis of the stress–strain relationship for (a) the normal spring and (b) the shear spring (assuming that \( \sigma^* \) is constant).**
Table 3 Relationships between spring parameters for the normal springs of the mortar and aggregate elements and the macroscopic material properties.

| Material | Young's modulus | Failure criteria | Softening behavior |
|----------|----------------|------------------|--------------------|
| mortar   | $E^*$          | $c^*$            | $\phi^*$          | $\sigma_s^*$       | $\beta^*_s$ | $\beta_{\mathrm{secs}}^*$ | $\chi^*$ | $\kappa^*$ |
| aggregate| $1.45E_{\mathrm{cm,macro}}$ | 1.0f_{\mathrm{ftm,macro}} | $0.5E_{\mathrm{cm,macro}}$ |  | $-0.05$ | $-0.025$ | $-0.01$ | $-0.3$ |

Table 4 Relationships between spring parameters for the shear springs of the mortar and aggregate elements and the macroscopic material properties.

| Material | Shear modulus | Failure criteria | Softening behavior |
|----------|---------------|------------------|--------------------|
| mortar   | $\eta^* = G^*/E^*$ | $c^*$            | $\phi^*$          | $\sigma_s^*$       | $\beta^*_s$ | $\beta_{\mathrm{secs}}^*$ | $\chi^*$ | $\kappa^*$ |
| aggregate| $1.45E_{\mathrm{cm,macro}}$ | 1.0f_{\mathrm{ftm,macro}} | $0.5E_{\mathrm{cm,macro}}$ |  | $-0.07$ | $-0.025$ | $-0.01$ | $-0.3$ |

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2.3.2 Instantaneous constitutive laws for the ITZ

The constitutive laws of the aggregate–mortar interface are shown in Fig. 6. As stated earlier, the ITZ has properties inferior to those of the bulk mortar, with its lower elastic modulus (Eq. (1)) and strength (Eq. (2)) in tensile and shear loading considered. Furthermore, in the compression regime, a lower compressive stiffness ($E_{\mathrm{ITZ}}^*$) until the ITZ is crushed, was assumed (Fig. 6(a)). Based on the thickness of the ITZ ($t_{\mathrm{ITZ}}^*$), the lower compressive stiffness regime ($E_{\mathrm{ITZ}}^*$) was determined ($E_{\mathrm{ITZ}}^* = E_{\mathrm{ITZ}}^*/h^*$), while the strain below $-\varepsilon_{\mathrm{ITZ}}^*$ was set to be the average of the Young’s moduli of the aggregate and mortar. This constitutive law for the ITZ in the compression regime was introduced to account for the drying shrinkage of the concrete, which is regarded as a composite of the mortar and aggregate in the numerical model (Maruyama and Sugie 2014).

The parameters of the ITZ are given by:

$$E_{\mathrm{ITZ}}^* = \alpha_{\mathrm{ITZ}}E_{\mathrm{cm}}^*, E_{\mathrm{ITZ}}^* = \gamma_{\mathrm{ITZ}}E_{\mathrm{cm}}^*, G_{\mathrm{ITZ}}^* = \alpha_{\mathrm{ITZ}}G_{\mathrm{cm}}^*$$ and

$$f_{\mathrm{ITZ}}^* = \beta_{\mathrm{ITZ}}f_{\mathrm{cm}}^*, G_{\mathrm{ITZ}}^* = \eta_{\mathrm{ITZ}}G_{\mathrm{cm}}^*, c_{\mathrm{ITZ}}^* = \beta_{\mathrm{ITZ}}c_{\mathrm{cm}}^*,$$

where $E_{\mathrm{ITZ}}^*$ and $E_{\mathrm{cm}}^*$ are the Young’s modulus of the ITZ and mortar, respectively; $f_{\mathrm{ITZ}}^*$ and $f_{\mathrm{cm}}^*$ are the tensile strengths of the ITZ and mortar, respectively; $G_{\mathrm{ITZ}}^*$ and $G_{\mathrm{cm}}^*$ are the fracture energy of the ITZ and mortar, respectively; $c_{\mathrm{ITZ}}^*$ and $c_{\mathrm{cm}}^*$ are the shear strength of the ITZ and mortar, respectively; and $\alpha_{\mathrm{ITZ}}$, $\beta_{\mathrm{ITZ}}$, $\gamma_{\mathrm{ITZ}}$, $\eta_{\mathrm{ITZ}}$, $\chi_{\mathrm{ITZ}}$, $\kappa_{\mathrm{ITZ}}$, $\kappa_{\mathrm{cm}}$, $\beta_{\mathrm{cm}}^*$, and $\kappa^*$ are the same as those of the mortar (as shown in Table 4).

Here, it is assumed that the changes in stiffness and strength of the ITZ result from the consolidation of the hcp in the ITZ (as discussed later in Fig. 7). The Young’s modulus and strength of the ITZ were set to be scalar multiples (Eqs. (1) and (2)) of those of mortar.

2.3.3 Moisture content-dependent/time-dependent behavior

Changes in the physical properties of the mortar from drying were introduced, reflecting previous experimental results that have reported that the change in compressive...
strength of mortar with drying has a similar trend to the change in the bending strength of the cement paste (Maruyama et al. 2014). The bending strength of the cement paste increased by 20% from the saturated condition to 80% relative humidity (RH), decreased by 40% from 80% RH to 40% RH, and increased dramatically below 40% RH by 100% at a constant temperature of 20°C. These trends are explained by the change in the pore structure of the hcp due to the agglomeration of C-S-H under drying and the change in the cohesion of C-S-H agglomerations.

Figure 7 shows the input mechanical properties change (MPC) ratio of dried mortar to that of the mortar under the sealed condition. The MPC ratio in Fig. 7 (for $E_{\text{in}}$, $G_{\text{in}}$, $f_{\text{in}}'$, and $G_{\text{in}}'$) was determined by comparing the calibration with the experimental results for the mortar (Maruyama et al. 2014). For the calibration, a cylindrical mesh consisting only of mortar elements was used for a drying analysis under RH and temperature conditions, with changes in mortar properties due to drying included. Then, the compression and tensile loading analyses were performed. Through a comparison of the experimental/predicted mechanical properties, a best-fit function for the MPC ratio was adopted as shown in Fig. 7(a) as a function of the relative water content change $\Delta R$ (introduced later in detail in Section 2.4.1) after drying.

The change in the Young’s modulus of the mortar is explicitly introduced as a function of $\Delta R$ as shown in Fig. 7(b). The mortar Young’s modulus demonstrated a monotonic decrease in the experimental results, thought to be explained by the cracking of hcp around fine aggregates caused by the fine aggregates restraining the shrinkage of hcp under drying.

A time-dependent cracking model was introduced to the proposed RBSM for considering the MPC with cracking (Maruyama et al. 2006; Jang and Maruyama 2017). The schematic of this concept is shown in Fig. 8. This model is prepared for the damaged (showing softening) mortar springs with property change due to drying and heating. This model assumes that the change in material properties does not affect the damage experienced, which is represented by the recorded maximum crack width. Due to this assumption, after cracks occur,
the origin of the stress–strain relationship shifts and the strain at zero-stress changes when unloaded. Specifically, once a crack has occurred, it remains permanently (i.e., \( \varepsilon'_{\text{wcr}} \) in Fig. 8 does not change by MPC), and the combination of the equivalent crack width strain (\( \varepsilon'_{\text{wcr}} \) in Fig. 8) and the updated stress–strain curve (i.e., the curve calculated by Fig. 4(a) using the properties after drying) uniquely determines the stress–strain relationship after the change in material properties. In the case that stress at the maximum strain decreases due to MPC (Fig. 8(b)), the unbalanced force is distributed to the surrounding elements by the convergence calculations.

### 2.4. Moisture transfer and creep

#### 2.4.1. Moisture transfer

The truss network model, originally developed by Bolander and Berton (2004), can be coupled with an RBSM and was used for the heat and moisture transfer analysis. Figure 9 shows a diagram of the truss network model. The model is a one-dimensional model in which the diffusion equations are discretized by a truss element connecting the Voronoi generators. The 1D water/thermal diffusion problem was solved in each truss. The governing equation of the water transfer is:

\[
\frac{1}{\omega} \frac{\partial R}{\partial t} = \text{div}(D(R) \cdot \text{grad } R) + \dot{W}, \tag{3}
\]

and was discretized as:

\[
\frac{A_k \cdot D(R)}{L} \begin{bmatrix} R_1 \\ R_2 \end{bmatrix} + \frac{1}{\omega} \frac{\partial R}{\partial t} = \begin{bmatrix} 1 & -1 \\ -1 & 1 \end{bmatrix} \begin{bmatrix} R_1 \\ R_2 \end{bmatrix} + \frac{1}{6} \left( \begin{bmatrix} 1 & 1 \\ 1 & 1 \end{bmatrix} \right) \begin{bmatrix} \partial R_1 / \partial t \\ \partial R_2 / \partial t \end{bmatrix}
\]

\[
\begin{bmatrix} 0 \\ 0 \end{bmatrix}
\]

\[
\frac{A_k \cdot \omega L}{d_{\text{wv}}} \begin{bmatrix} R_1 - R_{\text{env}} \\ R_2 - R_{\text{env}} \end{bmatrix} = \begin{bmatrix} 0 \\ 0 \end{bmatrix}
\]

where \( R \) is the relative water content (-); \( \omega \) is the volumetric conversion factor (Nakamura et al. 2006); \( W \) is the water consumption from cement hydration (g/mm³), which is not considered explicitly in this study; \( D(R) \) is the moisture diffusion coefficient (mm²/s); \( R_1 \) and \( R_2 \) are the relative water contents in truss nodes 1 and 2, respectively (-); \( R_{\text{env}} \) is the relative water content at the equilibrium state with the environment (-); and \( t \) is time (s). In Eq. (4), the third term on the left-hand side represents the flow on the boundary surface, \( A_k \) is the area of the Voronoi facet between the contiguous nodes \( i \) and \( j \), \( L \) is the length of the truss element connected, and \( d_{\text{wv}} \) is the virtual distance for the boundary condition of water transfer from the matrix to the environment (mm) with \( d_{\text{wv}} = 5 \text{ mm} \), and \( A_k \) is the area of the boundary (mm²). Time discretization was performed using the Crank–Nicholson method (Crank and Nicolson 1947). The diffusion coefficient \( D(R) \) was determined based on previous research (Maruyama et al. 2011) as:

\[
D(R) = D_{\text{so}} \cdot \frac{1}{(5.0 - 9.1R_t + 4.15R_t^2)}, \tag{5}
\]

\[
D_{\text{so}} = \alpha \cdot 1.47 \times 10^{-10} \exp(4.41 \cdot t_w) \frac{\partial \mu}{\partial w_v} \frac{P(T)}{P(T_0)}, \tag{6}
\]

where \( D(R) \) and \( D_{\text{so}} \) are the diffusion coefficients at \( R \) and \( R = 0.6 \), respectively (mm²/s), \( \alpha \) is the constant used to fit the experimental data (\( \alpha = 0.2 \) in this study), \( t_w \) is the statistical water absorption thickness (nm) (Maruyama et al. 2014), \( \mu \) is the chemical potential (J/g), \( w_v \) is the volumetric water content (g/mm³), \( P(T) \) is the vapor pressure at temperature \( T \) (Pa), \( T_0 \) is the reference temperature (293 K), and \( T \) is the temperature (K). The parameters \( \alpha \) and \( t_w \) were taken from a previous study (the data of “N55” was adopted) (Maruyama et al. 2014).

#### 2.4.2. Creep

As a parametric study, the impact of creep on the compressive loading analysis was also investigated. Although a previous study examined the influence of creep on the volumetric change and damage due to the neutron irradiation (Giorla et al. 2015, 2017), which concluded that creep has a significant impact on the occurrence of damage in concrete due to the expansion of aggregate, the impact of creep on the mechanical properties of concrete subjected to irradiation have not yet been evaluated.

To confirm the impact of creep, a basic creep model was used. The drying creep was not introduced because microcracking, which is assumed to cause drying creep, is explicitly introduced in the 3-phase model (Idiat et al. 2011).
The specific basic creep formulation of the JSCE design code formula (JSCE 2010) is given by:

\[ \varepsilon_{bc,c} = \beta \cdot 1.5(C + W) \cdot (W / C)^{1/4} \{ \ln(t') \}^{-0.85} \cdot 10^{-3} \quad \text{and(7)} \]

\[ \varepsilon_{bc}(t,t') = \varepsilon_{bc,c} \cdot [1 - \exp\{-0.09(t - t')^{0.6}\}] \quad \text{(8)} \]

where \( \varepsilon_{bc,c} \) is the ultimate specific basic creep (\( \mu \varepsilon / \text{MPa} \)), \( C \) is the mass of cement per the unit concrete volume (kg/m\(^3\)), \( W \) is the mass of water per the unit concrete volume (kg/m\(^3\)), \( W / C \) is the water-to-cement ratio (-), \( t' \) is the age at loading (in this study, \( t' \) begins from 365 days according to the experiment), \( t \) is the current age, and \( \beta \) is a constant to modify the specific basic creep for the RBSM. Here, \( \beta = 2.5 \) was applied to the RBSM as a result of the calibration; that is, a stress of 1 MPa was applied to the \( 40 \times 60 \text{ mm}^3 \) cylinder, with \( \beta = 2.5 \) found to provide agreement in the time-dependent strain change with the design code formulation for the concrete (Eqs. (7) and (8)).

The basic creep model is applied to the mortar and ITZ, and the ITZ spring was set to have the same creep as the mortar, with the creep utilized for both the normal and shear spring. Hence, the mixture in Table 1 excluding GA (coarse aggregate) was used for the creep calculation (i.e., \( C = 585 \text{ kg/m}^3 \) and \( W = 292 \text{ kg/m}^3 \) for Eq. (7)). The calculated creep coefficient of concrete after infinite time from 365 days was 0.9.

In the numerical model, creep was calculated using a step-by-step procedure (Neville et al. 1983) based on the principle of superposition. To maximize the impact, the basic creep during the unloading due to tensile softening was not considered.

### 2.5 Values used

#### 2.5.1 Basic properties of the mortar and aggregate

Table 5 shows the input material properties that are used to obtain the spring parameters shown in Tables 3 and 4. In this study, two types of compressive loading and tensile splitting test analyses after neutron irradiation were conducted (details are discussed in Section 2.7). One compressive analysis is to reproduce the experimental results of Maruyama et al. (2017) using a \( 40 \times 60 \text{ mm}^3 \) cylinder, and the other is to confirm the effect of shape (a \( 75 \times 150 \text{ mm}^3 \) cylinder was used for this analysis) and to develop the relationship between the compressive and splitting tensile strengths after irradiation. In the splitting tensile strength and compressive loading analysis of the \( 75 \times 150 \text{ mm}^3 \) cylinder, the volume ratio of mortar of Lin et al. (2015) was adopted (see Table 5).

For the mortar, the compressive strength (\( f_{cm,macro} \)) was taken from the target experiment, and the Young’s modulus (\( E_{cm,macro} \)), tensile splitting strength (\( f_{tm,macro} \)), and fracture energy (\( G_{fm,macro} \)) were calculated by:

\[ E_{cm,macro} = 0.22 \cdot f_{cm,macro} + 13.4, \quad (9) \]

\[ f_{tm,macro} = 2.45 \cdot \ln(f_{cm,macro}) - 5.61, \quad \text{and} \]

\[ G_{fm,macro} = 0.01 \cdot f_{cm,macro}^1 \cdot d_{max}^{1/3}, \quad (11) \]

where \( f_{cm,macro} \) is the Young’s modulus of the mortar (GPa), \( f_{tm,macro} \) is the compressive strength of the mortar (MPa), \( G_{fm,macro} \) is the fracture energy of the mortar (N/m), and \( d_{max} \) is the maximum aggregate grain size (mm). Equations (9) and (10) are derived from the existing data for the relationships between the Young’s modulus and compressive strength (Kawakami et al. 1995; Kawakami 2005) and the relationship between the tensile strength and compressive strength (Kosaka and Tanigawa 1975; Kosaka et al. 1975). Equation (11) is the formulation in the JSCE code for...
fracture energy (JSCE 2002). For the parameters for the ITZ, the values presented in Table 6 were used, which were determined to fit the properties of concrete in the sealed condition (discussed below in Table 9).

2.5.2 Shrinkage of the aggregate and mortar
As stated in Section 2.2, the drying shrinkage was introduced into the model before the RIVE of the aggregate. Figure 10 shows the shrinkage of the mortar and aggregate considered in the model. Since the target experiment (Maruyama et al. 2017) did not measure the drying shrinkage of the mortar, the shrinkage data of Maruyama et al. (2014) were employed. It should be noted that the mortar shrinkage strain in the experiment (Maruyama et al. 2017) did not measure the mortar properties (i.e., the compressive strength ratio, taken as a function of $ΔR$. The input mortar shrinkage ($με$) for the analysis is:

$$ε_{sh,mort} = 5162 \cdot ΔR^4 - 8726 \cdot ΔR^3 + 4527 \cdot ΔR^2 - 2733 \cdot ΔR$$

(12)

The shrinkage of the aggregate is based on the experiment of Maruyama et al. (2014), which used the GA aggregate given in Table 1, and is given by:

$$ε_{sh,agg} = -60 \cdot ΔR$$

(13)

where $ε_{sh,agg}$ is the input aggregate shrinkage ($με$).

2.5.3 Change in mortar properties due to irradiation
To investigate the deterioration of mortar properties due to neutron irradiation, the experimental data of Pedersen (1971) were adopted. The specifications of the specimen were as follows. The mixture was 20 g cement, 60 g quartz sand (the maximum grain size was 0.5 mm), and 8 mL water, with w/c = 0.4. The volume fraction of the fine sand was approximately 0.6, given that the cement and quartz sand densities were 3.14 g/cm$^3$ and 2.65 g/cm$^3$, respectively. The specimen shape was $11.3 \times 11.3$ mm, which was cured in a controlled environment for half a year. Moreover, the dose of fast neutron irradiation was $1 - 3 \times 10^{19}$ n/cm$^2$ (the neutron dose for each sample was not stated), and the maximum temperature of the specimen was 80°C.

Figure 11 shows the relationship between the linear expansion and the compressive strength ratio, taken from Pedersen (1971). The quadratic regression curve is given by:

$$f_{cm}/f_{cm0} = -1.835 \times 10^{-9} ε_m^2 - 1.488 \times 10^{-5} ε_m + 1,$$

(14)

where $f_{cm}$ is the compressive strength of mortar, $f_{cm0}$ is the reference compressive strength, and $ε_m$ is the linear expansion of the mortar ($με$).

The strength change ratio of Eq. (11) was introduced to the mortar properties (i.e., $E_{cm}^*, G_{cm}^*$, $f_{cm}^*$, $G_{fcm}^*$, and $c_{sm}^*$). In this study, the impact of the expansion of the mortar is evaluated by a parametric study described below in Section 2.7. For this purpose, the mortar expansion strain is given by the ration of expansion strain of the mortar to that of the quartz (aggregate), or $ε_{sm}/ε_a$.

Therefore, the change in mortar properties after irradiation is introduced to the model using the radiation-induced expansion strain of the mortar and Eq. (11) at each step of the aggregate expansion.

It should be noted that, since little data is available for the mortar properties (i.e., $E_{cm}^*$, $G_{cm}^*$, $f_{cm}^*$, $G_{fcm}^*$).
and $c^*$ when subjected to neutron irradiation, Eq. (11) was directly employed to determine these properties after irradiation.

2.6 General analysis settings

2.6.1 Meshes and boundary conditions

Voronoi meshes were used for the cylindrical models. For compressive loading, two types of mesh were employed, as shown in Fig. 12. In both meshes, the geometry of the coarse aggregates was identical while the mesh sizes in the aggregates varied. The finer aggregate mesh shown in Fig. 12(d) is used to simulate the inhomogeneous expansion of rock-forming minerals in aggregates. The number and dimensional distribution of the aggregates follows the experimental data (see Fig. 13).

A rigid plate element, which is used for boundary conditions and has no volume, is used for the mesh of the compressive loading analysis with the springs attached to the plate set as elastic. The plate elements were attached on the top and bottom surfaces of the mesh depicted in Fig. 12. During the drying and aggregate expansion calculations, the stiffness of the normal and shear springs attached to the plate element was set to 0.001 times that of the mortar stiffness for the free shrinkage condition to prevent them restraining the volumetric change of the concrete. During the loading calculation, the stiffness of the normal and shear springs attached to the plate element was set to 1000 times that of the mortar springs, to realize the friction on the interface between the specimen and the loading plate.

Figure 14 shows the mesh for the splitting tensile strength analysis. As the thickness of the cylinder is not affected by the tensile strength (Lin et al. 2015), the size of the mesh was $\varphi 100 \times 25$ mm$^3$. The aggregate size distribution in the mesh follows the Japanese standard values as shown in Fig. 15, with an aggregate volume fraction of 33% (34.4% in the reference experiment) and an average element size of 2.49 mm. As for the loading process, since it is difficult to use plate elements, the

![Graph](image1.png)

**Fig. 11** Relationship between the compressive strength ratio and linear expansion strain of mortar (after Pedersen (1971)).

![Images](image2.png)

**Fig. 12** The $\varphi 40 \times 60$ mm$^3$ mesh for the compressive loading analysis: (a) appearance, (b) aggregates, and (c) cross-section for coarse mesh, and (d) cross section for fine mesh. The maroon cells are the aggregates. Typical mesh size is 2-3 mm. The mesh size in the aggregate for the fine mesh is ~ 1mm.

![Graph](image3.png)

**Fig. 13** Comparison between the experimental and simulation results for the particle size distribution of the coarse aggregate. In the experiment, the maximum grain size $G_{\text{max}}$ was 13 mm. The Japanese standard (JIS) is also illustrated (the dashed lines show the lower and upper limits).
elements that are located at the sides (colored elements shown in Fig. 16) have an imposed displacement. Note that sensitivity of mesh on the obtained results will be discussed in Section 3.1.

Figure 17 shows the mesh for the compressive loading to evaluate the effect of shape. The dimension of the mesh was $\phi 75 \times 150 \text{ mm}^3$ and the aspect ratio was 2.0 following the parameters of the general concrete property test. The aggregate particle size distribution (following the Japanese standard as shown in Fig. 16), volume fraction (33%), and the mean element size (2.43 mm) were set to be the same as the $\phi 100 \times 25 \text{ mm}^3$ model (Fig. 14) as both models were intended to have an identical mixture and materials. The boundary condition was the same as the cylindrical $\phi 40 \times 60 \text{ mm}^3$ mesh (Fig. 12).

2.6.2 Settings for the volumetric change and loading analysis

The aggregate and mortar expansion (after the water transfer analysis) was introduced at a constant strain at each step in the simulation. The expansion was simulated so that a force equivalent to the expansion strain was applied to the springs. The expansion of the coarse aggregate was introduced in increments of $40 \times 10^{-6}$ to the aggregate spring (located between the aggregate elements). For the mortar, the calculated spring force equivalent to the expansion strain (given by the ratio to the aggregate, as stated below in Section 2.7) was applied to the mortar springs. For the ITZ springs, the average expansion weighted by each spring length ($h_i$ and $h_j$ in Fig. 3) was introduced.

The incremental time for creep during the aggregate expansion was calculated via the following procedure. The neutron fluence is calculated based on Eqs. (16) and (17) (explained in detail in Section 3.1) and the total aggregate expansion. Then, the time at the current step is calculated using the neutron flux and the difference from the previous step gives the incremental time. A neutron flux of $3.6 \times 10^{12} \text{ n/cm}^2/\text{s}$, which is the average neutron flux of the target experiment (Maruyama et al. 2017), was applied.

For the compressive loading analysis, an incremental deformation of $-20 \mu \varepsilon/\text{step}$ was applied, while for the splitting tensile test, a displacement of $0.75 \mu \text{m/step}$ was

---

Fig. 14 The $\phi 100 \times 25 \text{ mm}^3$ mesh for prediction of splitting tensile strength after aggregate expansion: (a) appearance, (b) aggregates, (c) transverse cross-section, and (d) longitudinal cross-section.

Fig. 15 Comparison between the experiment and simulation of the particle size distribution of the aggregate ($\phi 100 \times 25 \text{ mm}^3$ for the tensile splitting analysis and $\phi 75 \times 150 \text{ mm}^3$ for compression analysis). The Japanese standard (JIS) is also illustrated (the dotted line shows the lower and upper limits).

Fig. 16 The boundary condition for the mesh for prediction of the splitting tensile strength.

Displacement was directly given to the colored elements.
applied to the elements selected for the boundary condition (Fig. 16).

2.7 Parameters of the numerical study
This section presents details of the sensitivity analysis described in Section 2.2. As shown in Tables 7 and 8 the parameters for this parametric study are as follows: the amount of linear expansion of the aggregate \( \varepsilon_a \) (composed of quartz), the expansion ratio of the mortar to quartz \( \varepsilon_{m}/\varepsilon_a \), the partial expansion of the mortar, the impact of aggregate mesh size under aggregate fractures, and the mortar property change due to irradiation (Fig. 11). As the GA consists of more than 90 w.t.% of \( \alpha \)-quartz (Table 2), GA is considered to be 100% quartz.

Since no data has been reported on the behavior of mortar in concrete under neutron irradiation, a sensitivity analysis on the mortar expansion was conducted. The fine aggregate accounts for half the volume of the mortar and the fine aggregate is composed of 47.35 w.t.% quartz and 32.4 w.t.% feldspar (i.e., microcline, anorthoclase, anorthite, and albite in the sand), and 20.25 w.t.% of other minerals. Approximately 25 w.t.% of the mortar is \( \alpha \)-quartz (i.e., \( 0.48 \times 0.4735 = 0.23 \)), and it was assumed as a first-order approximation that the mortar expands approximately 0.25 times as much as pure quartz (GA is mostly composed of quartz). Consequently, \( \varepsilon_{m}/\varepsilon_a = 0.25 \) was adopted as a reference value. In addition, \( \varepsilon_{m}/\varepsilon_a = 0.5 \) was also adopted. Based on Pedersen (1971), the swelling of mortar using \( \alpha \)-quartz sand is greater than the isolated expansion of \( \alpha \)-quartz \( (< 3 \times 10^{19} \text{n/cm}^2) \). Here, the expansion of \( \alpha \)-quartz is estimated by a neutron-fluence-\( \alpha \)-quartz expansion relation equation, experimental data, and the standard deviation proposed by Field et al. (2015), in addition to the relation between neutron expansion with temperature and neutron fluence proposed by Maruyama et al. (2017), which was based on Bykov et al. (1981). The larger expansion of the mortar than the \( \alpha \)-quartz in isolation can be explained by cracking. Based on the relationship between the \( \alpha \)-quartz content in mortar and the volumetric expansion, \( \varepsilon_{m}/\varepsilon_a = 0.5 \) is found for the mortar portion of the reference concrete of Maruyama et al. (2017). For a better understanding of the impact of mortar expansion, the results for \( \varepsilon_{m}/\varepsilon_a = 0 \) are also calculated.

In this case (i.e., \( \varepsilon_{m}/\varepsilon_a = 0.25 \) (ND), and 0.5 in Tables 7 and 8), swelling was introduced into the springs on the mortar–aggregate and aggregate–aggregate interfaces by 0.5\( \varepsilon_a \) and \( \varepsilon_a \), respectively, which implies that the total expansion of each aggregate element on an arbitrary axis through a Voronoi generator (linear expansion) is \( \varepsilon_a \) and the volumetric swelling is 3\( \varepsilon_a \).

The target of the investigation into the partial expansion of the coarse aggregates in Table 7 is granite-like aggregates (i.e., large grains of rock-forming minerals and inhomogeneous expansion of minerals are assumed, as mentioned earlier in Section 2.1). To investigate the effect of cracks inside aggregates and the resultant reduction in stiffness and strength of aggregates on the mechanical properties of concrete, expansion was introduced into 40% of the aggregate elements selected at random. The selected aggregate expands by the given \( \varepsilon_a \) while the other aggregate elements do not swell; the swelling is introduced into all the springs attached to the selected aggregate elements by 0.5\( \varepsilon_a \) so that the total expansion of one aggregate element on an arbitrary axis through the Voronoi generator is \( \varepsilon_a \). In addition, the impact of aggregate crack shape is investigated by changing the element size. If the element is finer, then the crack inside the aggregate becomes flat, which may influence the strength and Young’s modulus of the concrete.

Fig. 17 The \( \varphi 75 \times 150 \text{ mm}^3 \) mesh for prediction of compressive strength and Young’s modulus after aggregate expansion: (a) appearance, (b) aggregates, and (c) cross-section. The maroon cells are the aggregates. The typical mesh size was 2-3 mm.
3. Results

3.1 Basic properties and volumetric changes

Table 9 summarizes the experimental and numerical results for the properties of the concrete in the sealed and dried (70°C) conditions. Although the RBSM slightly overestimated the drying shrinkage, the simulation gave reasonable agreement with the experimental results. In particular, the ratios of changes in the compressive strength and Young’s modulus after drying were well reproduced. Furthermore, in the experiment (Lin et al. 2015), the ratio of the splitting tensile strength in the saturated condition to that in the dried condition at 65°C was 0.91–0.99; therefore, the calculated tensile strength ratio after drying (i.e., 0.95) agrees with the experimental results (see f of Table 9).

With regards to tensile splitting strength, one may think that the results are sensitive to the aggregates arrangement. To confirm the sensitivity of the present calculation results, additional calculation was conducted. Six meshes for the same condition as that of the present calculation produced the average strength of 3.50 MPa and the standard deviation of 0.05 MPa and it is concluded that the obtained results are representative. In addition, the three meshes containing the aggregate particles which overlap the cut surfaces, whose coarse aggregate content was 35.0%, did not make a large difference in splitting tensile strength.

The linear concrete expansion was calculated by:

\[ \varepsilon_{\text{lin}} = \varepsilon_{\text{con}} \]

where \( \varepsilon_{\text{lin}} \) is the linear RIVE of concrete and \( \varepsilon_{\text{con}} \) and \( \varepsilon_{\text{dis}} \) are the concrete’s expansions in length and diameter, respectively. The expansion in diameter is the average of the expansion strains in the x- and z-axes. In addition, the length in the sealed condition is used as a reference length.

### Table 7 Parameters for the compressive loading analysis.

| Objective                                      | Notation | Mesh | \( \varepsilon_a \) | Mortar expansion (\( \varepsilon_m/\varepsilon_a \) ratio) | Expansion type | Mortar property change due to irradiation |
|------------------------------------------------|----------|------|---------------------|----------------------------------------------------------|---------------|--------------------------------------------|
| Reference                                      |          |      | 0.1%, 0.2%, 1.5%   | 0.25                                                      | H            | ✓                                         |
| Impact of mortar expansion and deterioration   |          |      | 0.1%, 1.0%, 1.5%   | 0.0 (mortar does not expand)                             | H            | -                                         |
| Isolated mortar expansion                      | Exp-M    | C    | 0.2%, 0.5%, 1.0%, 1.5% | H                                                      | ✓            |
| Isolated mortar expansion                      | Exp-M ND | C    | 0.2%, 0.5%, 1.0%, 1.5% | H                                                      | ✓            |
| Impact of partial expansion of the aggregates  | Exp-P-F  | F    | 0.2%, 1.0%, 1.5%   | P (0.4 volume fraction of the coarse aggregate expands)  | ✓            |
| Impact of creep                                |          |      | 0.1%, 0.2%, 1.5%   | 0.25                                                      | H            | ✓                                         |
| Mutual relationship between properties         | Exp-P-C  | C    | 0.1%, 0.2%, 1.5%   | 0.25                                                      | H            | ✓                                         |

\( \varepsilon_m/\varepsilon_a \): the ratio of the mortar expansion to the aggregate expansion (e.g., for an aggregate expansion (\( \varepsilon_a \)) of 1% and \( \varepsilon_m/\varepsilon_a = 0.5 \), the mortar expansion \( \varepsilon_m \) is 0.5%.

Mesh: C and F denote coarse (Fig. 12c) and fine (Fig. 12d) meshes, respectively. T denotes the \( \phi 75 \times 150 \) mm\(^3\) cylinder (Fig. 17).

“Homogenous expansion” means that all the coarse aggregate elements swell. “Partial expansion” means that only the selected aggregates expand, with 40% of the aggregate elements selected for expansion.

“ND”: No Deterioration of mortar. The deterioration in Fig. 11 is not considered.

“CR”: Considered creep as described in Section 2.4.2. *In the other cases, creep was not introduced.

### Table 8 Parameters for the tensile splitting simulation.

| Objective                                      | Notation | \( \varepsilon_a \) | Mortar expansion (\( \varepsilon_m/\varepsilon_a \) ratio) | Mortar property change due to irradiation |
|------------------------------------------------|----------|---------------------|----------------------------------------------------------|--------------------------------------------|
| Impact of mortar expansion                     |          | 0.1%, 0.2%, 1.5%   | 0.25                                                      | -                                         |
| Impact of mortar expansion                     |          | 0.1%, 0.2%, 1.5%   | 0.25                                                      | ✓                                         |
| Impact of mortar expansion                     |          | 0.1%, 0.2%, 1.5%   | 0.5                                                      | ✓                                         |

\( \varepsilon_m/\varepsilon_a \): the ratio of the mortar expansion to the aggregate expansion.
Figure 18 shows the relationships between the aggregate expansion \( \varepsilon_a \) and the corresponding linear expansion strain of the concrete specimen. The experimental data and their correction data based on Maruyama et al. (2017) are also shown. In the experiment, the aggregate specimens were irradiated at 53°C with fast neutron fluences of 0.70, 1.28, 4.12, and 8.25 \( \times 10^{19} \) n/cm\(^2\). The lower bound is the relationship between aggregate expansion at 53°C and the concrete expansion for concrete irradiated at 57.6–72.9°C. For the upper bound, the aggregate expansion in the experiment is corrected for 70°C (average of the maximum temperature inside the specimens) based on the neutron energy spectrum and the following equations proposed by Maruyama et al. (2017):

\[
\varepsilon_{n,\text{quartz}} = \varepsilon_{n,\text{quartz},\infty} \left[ 1 - \exp \left( -\frac{n}{K(T)} \right) \right]
\]

and

\[
K(T) = 0.3 \times 10^{20} \times \frac{\exp(2261/298)}{\exp(2261/T)},
\]

where \( \varepsilon_{n,\text{quartz}} \) is the expansion volume of quartz for the fast neutron fluence \( n \), \( \varepsilon_{n,\text{quartz},\infty} = 18\% \), which is the maximum expansion volume of the quartz, \( n \) is the fast neutron fluence (> 0.01 MeV) (n/cm\(^2\)), \( K(T) \) is the fast neutron fluence when the expansion reaches half the maximum expansion volume (n/cm\(^2\)), and \( T \) is the absolute temperature (K). In the experiment of Maruyama et al. (2017), the ratio of the fast neutron fluence of > 0.1 MeV to the fast neutron fluence of > 0.01 MeV was 0.632–0.637.

From Fig. 18, the ratio of \( \varepsilon_{m}/\varepsilon_a \) in the RBSM simulation results ranged from 0.71 to 0.76, which is within the possible range deduced from the experimental data. The best-fit trends were the case when the temperature of the specimen is assumed to be 62°C.

Figure 19 shows the typical damage distributions of the specimens after aggregate and/or mortar expansion due to irradiation. It is evident that the damage in the mortar becomes more severe as the \( \varepsilon_{m}/\varepsilon_a \) ratio decreases. In the case of the partial expansion of the aggregate (Exp-P), many cracks are observed inside the aggregates, while no damage in the aggregate is observed for the Exp-M case (only the mortar expanded).

### 3.2 Mechanical properties

Figure 20 shows the calculated relationships between the linear expansion of the concrete \( \varepsilon_{\text{lin},\text{con}} \) and the compressive strength ratio \( f'/f_c \) as compared with the experimental results and the lower boundary curve of \( f'/f_c \) proposed by Maruyama et al. (2017).

The linear expansion is calculated by Eq. (12) and the calculation results for \( \varepsilon_{m}/\varepsilon_a = 0 \), 0.25, 0.5, and 0.5 ND. T.C. denotes the temperature condition.

### Table 9 Comparison of the experimental and simulation results for concrete subjected to drying. The values in parentheses are the ratios to the sealed concrete properties.

|                      | Drying Shrinkage (\( \mu \varepsilon \)) | \( f_c \) (MPa) | \( E_c \) (GPa) | \( f_t \) (MPa) |
|----------------------|------------------------------------------|-----------------|----------------|----------------|
| **Sealed condition** |                                          |                 |                |                |
| Experiment           | -                                        | 68.7            | 35.3           | -              |
| RBSM                 | -                                        | 67.5            | 34.6           | 3.56           |
| **Dried condition (at 70°C)** |                                      |                 |                |                |
| Experiment           | ~452 ± 158 (φ40 × 60) (0.93)             | 63.5            | 27.9           | -              |
| RBSM                 | ~770 (φ100 × 25) (0.98)                  | 66.1            | 27.3           | 3.39           |

\( K(T) = 0.3 \times 10^{20} \times \frac{\exp(2261/298)}{\exp(2261/T)}, \)
ND were minor, which means that mortar deterioration due to expansion does not have a significant influence on the reduction of strength. Consequently, it is concluded that the coarse aggregate expansion rather than mortar deterioration results in the deterioration of concrete properties due to neutron irradiation. In addition, theoretically, when the mortar does not expand, such as when using a limestone fine aggregate, it is possible that the strength ratio is reduced below the proposed lower bound. The proposed model adequately captures the range of experimental results for $f_c/f_{c0}$.

Figure 20(b) shows the results for the cases with mortar expansion without expansion of the coarse aggregate. While Exp-M and Exp-M ND of $\varepsilon_{m,con}$ are $\sim 0.1\%$, showing slight increases over the sealed condition, the $f_{c0}/f_{c0}^{*}$ relationship overlaps with or underestimates that of $f_{c0}/f_{c0}$ as $\varepsilon_{m,con}$ increased. The slight increase in strength at an $\varepsilon_{m,con}$ of 0.1% is explained by the closure of the shrinkage-induced cracks due to the mortar expansion. In the case of $\varepsilon_{m}/\varepsilon_{a}=0.25$, the impact of the RIVE of coarse aggregates exceeded the impact of crack closure.

Figure 20(c) shows the results for the partial expansion of the coarse aggregates. The $f_{c0}/f_{c0}^{*}$ curves of Exp-P-F and Exp-P-C are almost identical and slightly lower than that of $f_{c0}/f_{c0}$ at 1% of $\varepsilon_{m,con}$. This result coincides with the lower limit of the previous experimental results. The impact of crack deterioration due to inhomogeneous expansion of rock-forming minerals is considered to be minor and the decrease in compressive strength is due to cracks resulting from uneven volumetric change between the coarse aggregates and the mortar.

Figure 20(d) shows the impact of creep. The strength reduction due to aggregate expansion was compensated for by the creep. The creep impact is almost comparable to that of the difference between $\varepsilon_{m}/\varepsilon_{a} = 0.25$ and $\varepsilon_{m}/\varepsilon_{a} = 0.50$.

Figure 21 shows the calculated relationships between the linear expansion of the concrete and the Young’s modulus ratio $E_c/E_{c0}$ as compared with the experimental results of Maruyama et al. (2017). The elastic moduli in Fig. 21 are the slopes of a secant to 1/3 $f_c$, following the corresponding experiment (Maruyama et al. 2017). “Review, Field (2015)” denotes the curve obtained from the equation for material properties/RIVE with the neutron dose suggested by Field et al. (2015).

From Fig. 21(a), $\varepsilon_{m}/\varepsilon_{a}=0.5$ shows good agreement with the experimental results of Maruyama et al. (2017), although the simulated results were underestimated by approximately 10% when $\varepsilon_{m,con}$ exceeded 0.25%. As with the compressive strength, a lower mortar expansion led to a lower Young’s modulus as the lower mortar expansion causes more severe damage (Fig. 19). Moreover, the curve from Field et al. (2015) varied greatly from the other data. This can be explained by the large experimental variation due to a difference in the “initial” condition: in some experiments, the concrete was placed in a drying condition (e.g., 60% RH, 100°C) for initial curing, which leads to a reduction in the Young’s modulus before

Fig. 19 Simulated damage in concrete cylinder specimens after aggregate/mortar expansion.
the neutron irradiation (Liu et al. 2014; Maruyama et al. 2014). As the initial stiffness is already lower than in the sealed condition, the reduction of the Young’s modulus can be underestimated, as shown in the curve from Field et al. (2015).

**Figure 21(b)** shows the $E_c/E_{c,0}$ - $\varepsilon_{n,\text{con}}$ relationships for Exp-P-F and Exp-P-C (partial expansion of the aggregate). The Young’s modulus of Exp-P-C was lower than that for $\varepsilon_m/\varepsilon_a = 0.25$. A comparison between Figs. 19(a) and (d) indicate that there were no large differences in the crack distribution in the mortar; therefore, the aggregate cracks reduced the Young’s modulus of Exp-P-F. It should be noted that the constitutive laws for the aggregate elements are a first-order approximation and, in particular, the response after the expanding aggregate elements come into contact again after cracking (hereafter, “re-contact”) is not validated. Further investigation into the constitutive laws of aggregate elements is necessary for a quantitative evaluation of the impact of cracks in aggregate due to inhomogeneous expansion of rock-forming minerals.

**Figure 21(c)** shows the $E_c/E_{c,0}$ - $\varepsilon_{n,\text{con}}$ results for Exp-M and Exp-M ND (isolated mortar expansion). Exp-M has a similar trend to $\varepsilon_m/\varepsilon_a = 0.25$ and Exp-M ND retained a Young’s modulus ratio of ~0.2 even after $\varepsilon_{n,\text{con}}$ exceeded 0.25%. One explanation of this result is the Young’s modulus reduction in the mortar. According to the crack distribution shown in Fig. 22, Exp-P and Exp-P ND had similar crack distributions. When $\varepsilon_{n,\text{con}}$ is larger than 0.36% (Exp-0.5), the Young’s modulus decreases as a result of the separation between the mortar and the aggregate (see Fig. 19(e)), and thus, the Young’s modulus of the concrete is subsequently determined by the mortar, as the aggregate is not in contact with the mortar by $1/3f_c$.

This is also suggested by the stress–strain curves shown in Fig. 23(b) which exhibit a sudden increase in stiffness during the loading process compared to that of $\varepsilon_m/\varepsilon_a = 0.5$ in Fig. 23(a) This indicates a re-contact between aggregate and mortar. Although the figure is not shown here, it was found that the cracks around the aggregate due to the mortar expansion did not close during compressive loading.

**Figure 21(d)** shows the impact of creep on the Young’s modulus ratio, where a large influence in the
earlier stage can be observed. For ~5% linear expansion, the Young’s modulus difference was ~10% of $E_{c0}$ (18% increase of $E_c$ for $\varepsilon_{m}/\varepsilon_a = 0.25$), while at ~1.2% linear expansion, the difference was small and ~4% of $E_{c0}$, but it is an ~52% increase of $E_c$ for $\varepsilon_{m}/\varepsilon_a = 0.25$. Therefore, if tensile creep strain at the same order of magnitude of ordinary concrete is observed, it cannot be disregarded.

Figure 24 shows the tensile splitting ratio $f/f_{0}$ with the linear expansion of the concrete as compared with previous experimental results (Gray 1971; Elleuch et al. 1972) and the regression curves proposed in the review papers of Hilsdorf et al. (1978) and Field et al. (2015). The standard deviation of the regression curve proposed by Field et al. (2015) was ±0.227 MPa. The simulation

![Figure 21](image1.png)

(a) Impact of mortar expansion

![Figure 22](image2.png)

(b) Isolated mortar expansion

(c) Partial expansion of the aggregate

(d) Effect of creep

Fig. 21 Simulated Young’s modulus ratio as a function of the linear expansion of the concrete.

Fig. 22 Comparison of simulated crack distributions between Exp-M and Exp-M ND at $\varepsilon_a = 1.5\%$. 

![Figure 23](image3.png)

![Figure 24](image4.png)
results correspond well with the experimental results. In particular, $\varepsilon_m/\varepsilon_a = 0.5$ best fits the experimental results.

From the numerical results of the compressive and tensile strengths (Figs. 20 and 24) and the Young’s modulus (Fig. 21), it is concluded that the evaluation of mortar expansion is necessary to evaluate the change in concrete properties for concrete affected by neutron irradiation and that the aggregate type is significant for the evaluation.

4. Discussion

4.1 Strength change

4.1.1 Mechanism of strength reduction

Here, the mechanism of strength reduction is investigated through the crack and stress distributions, as shown in Fig. 25. Figures 25(a)–(c) show the crack width distributions after the aggregate expansion and Figs. 25(d)–(f) show it at the peak of compressive stress ($f_c$), while Figs. 25(h)–(g) show the distribution of the stress in the vertical direction (i.e., loading direction) and deformation of elements at $f_c$.

The crack patterns at the peak of compressive stress shown in Figs. 25(d)–(f) are very similar to those obtained in the preliminary experimental loading results (Maruyama et al. 2013). The X-crack patterns and vertical cracks at the both sides were confirmed by the specimens without RIVE. The calculated failure process is well reflected by the load-bearing stress passes and resultant cracks found in the experiment, and consequently, the calculation results seemed reasonable. It should be mentioned here, the aggregate shape may be the important factor unsettled in this paper. Many studies have suggested that the spherical aggregate might overestimate the compressive and tensile strength of sound concrete (Kim and Abu Al-Rub 2011; Suchorzewski et al. 2017; Thirumalaiselvi et al. 2019), although few studies investigated the impact of aggregate shape during deterioration (Havlásek and Jirásek 2017). Since the present calculation is using spherical aggregates, the impact of aggregate shape may be shown in Figs. 25(a)–(c). This result is similar to previous numerical results on the bond strength of a reinforced concrete prism subjected to freeze–thaw cycles (Wang et al. 2019). Since the wider cracks reduce the stress transmission between elements (see Fig. 4(e)), the reduction in $f_c$ can be explained by the significant damage at $f_c$ due to the aggregate expansion.

This hypothesis is supported by the stress distribution at $f_c$, as shown in Figs. 25(g)–(f). The area of decreasing stress with the rise of RIVE corresponds with the area where the crack width increases. For example, the triangular part of both sides shows more significant damage (Figs. 25(d)–(f)) and lower compressive stress as the RIVE of the aggregate proceeds. Furthermore, a path of relatively higher stress (e.g., below $-70$ MPa for $\varepsilon_a = 0.5\%$ and below $-35$ MPa for $\varepsilon_a = 1.5\%$), or the load-bearing path, is created between the aggregates. This is because the aggregate stiffness is higher than that of the mortar. Therefore, the crack in the aggregate might reduce the load transmitted by this path.

The reason the damage at $f_c$ increases as a result of the RIVE can be explained by the following: 1) the wider existing cracks reduce the stress borne by the concrete, and 2) the distribution of the linear expansion of the concrete.

![Graph showing the linear expansion of concrete](image)

**Fig. 24** Predicted splitting tensile strength ratio as a function of the linear expansion of the concrete.

![Graph showing stress–strain curves](image)

**Fig. 23** Simulated stress–strain curves of (a) $\varepsilon_m/\varepsilon_a = 0.5$ and (b) Exp-M.
element and make it easier for these cracks to connect to each other and increase in width during loading through stress re-distribution. 2) Some existing cracks are too wide to close during loading. For example, the cracks near the upper surface around the aggregate do not close for $\varepsilon_a = 1.0\%$ and $1.5\%$ (Figs. 25(a)–(c)). In addition, the longitudinal cracks in the loading direction kept opening as can be seen by comparing Figs. 25(c) and (f). In conclusion, the severe damage due to the aggregate expansion leads to more damage at $f_c$ and a resultant lower $f'_c$.

### 4.1.2 Impact of creep

Figure 26 shows the stress–strain curve for $\varepsilon_m/\varepsilon_a = 0.25$ with/without creep ($\varepsilon_m/\varepsilon_a = 0.25$ and $\varepsilon_m/\varepsilon_a = 0.25$ CR). In

![Simulated crack width distribution after aggregate expansion](image1)

![Simulated crack width expansion at the peak compressive stress $f_c$](image2)

![Calculated stress distribution in the axial direction in the deformed configuration and deformation at $f_c$ for $\varepsilon_m/\varepsilon_a = 0.25$. For (d)–(f), cracks greater than 5 $\mu$m are shown. For (g)–(i), the deformation is magnified five times. The aggregate locations can be confirmed from Fig. 19(f).](image3)
the figure, the strain at zero stress indicates the volumetric change due to drying and aggregate expansion. It is evident that the introduction of creep increased $f_c$ and $E_c$ and, as aggregate expansion increased, the difference in $f_c$ increased. The difference was 5.9 MPa at $\varepsilon_a = 0.2\%$ and 10.2 MPa at $\varepsilon_a = 1.5\%$, and $\varepsilon_a/\varepsilon_c = 0.25$ CR shows a comparable trend with $\varepsilon_a/\varepsilon_c = 0.5$ in the relationship between $f_c/E_c$ and $\varepsilon_a/\varepsilon_c$.

**Figure 27** shows the crack distribution for $\varepsilon_a/\varepsilon_c = 0.25$ and $\varepsilon_a/\varepsilon_c = 0.25$ CR after the aggregate expansion of $1.5\%$ ($\varepsilon_c = 1.5\%$) and **Fig. 28** shows the stress and crack distributions at the compressive strength. From **Fig. 27**, $\varepsilon_a/\varepsilon_c = 0.25$ CR reduced the crack width as compared with $\varepsilon_a/\varepsilon_c = 0.25$ and showed a similar crack distribution with $\varepsilon_a/\varepsilon_c = 0.5$ (see **Fig. 19(e)**). This indicates that the creep mitigated the damage from the aggregate expansion, as confirmed for the case with drying shrinkage by a numerical study of Idiart et al. (2012). This mitigation can be interpreted by the same mechanism of the self-equivalent stress declining as the mortar expansion increases (tensile creep strain is considered as expansion), as confirmed in **Figs. 19(a)–(c)**.

Consequently, the damage mitigation due to the creep led to a higher $f_c$ and $E_c$ in the same manner as discussed in Section 4.1.1. From **Figs. 28(a)–(b)**, it is clear that the ratios of $f_c/E_c$ and $E_c/E_c$ for $\varepsilon_a/\varepsilon_c = 0.25$ CR show higher compressive stress than those for $\varepsilon_a/\varepsilon_c = 0.25$ and are comparable with those of $\varepsilon_a/\varepsilon_c = 0.50$.

In conclusion, it was found that creep reduced the RIVE-induced damage, which led to higher $f_c$ and $E_c$ than the case without creep. The best-reproduced results are for $\varepsilon_a/\varepsilon_c = 0.25$ CR and $\varepsilon_a/\varepsilon_c = 0.50$. The possible mechanism is similar, i.e., apparent expansion of the mortar is important and minor crack creation in the mortar caused by fine aggregate expansion ($\varepsilon_a/\varepsilon_c = 0.50$) and the tensile creep behavior of the mortar ($\varepsilon_a/\varepsilon_c = 0.25$ CR) have an important role. Further research is required to understand the realistic creep behavior as well as the impact of fine aggregate expansion on the mortar under neutron irradiation.

**4.2 Relation to neutron fluence**

The relationship between neutron fluence and $f_c/E_c$ is discussed here. As stated previously in Section 2.2, the metamicitization mechanism and rate of expansion of rock-forming minerals are still unknown. Even the relationship between expansion strain and the resultant aggregate property change has not yet been clarified yet, and discussion is made on this issue for engineering purposes.

First, the equivalent neutron fluences at $\varepsilon_a = 0.1$, 0.2, 0.5, 1.0, and 1.5% are evaluated, as summarized in **Table 10**. Based on Bykov et al. (1981), the rate of expansion of

**Fig. 27** Simulated crack width distributions of (a) $\varepsilon_a/\varepsilon_c = 0.25$ and (b) $\varepsilon_a/\varepsilon_c = 0.25$ CR at $\varepsilon_a = 1.5\%$. Figure (b) is the same as Fig. 19(b).
α-quartz is largely affected by the irradiation temperature, and while the mechanism is not known, even for temperatures less than 100°C, the impact is obvious (Luu et al. 2020). Regarding Fig. 18, the irradiation temperature was taken into consideration; consequently, it was confirmed that the irradiation temperature of 62°C gives the best fit to the RBSM calculation results. Using the inverse function of Eqs. (14) and (15) at 62°C, an equivalent fast neutron fluence for \( \varepsilon_a = 0.1, 0.2, 0.5, 1.0, \) and 1.5% in linear expansion is obtained.

Figure 29 shows the \( f_c/f_{c0} \) as a function of neutron fluence of the calculated results of \( \varepsilon_m/\varepsilon_a = 0, 0.25, \) and 0.5, as compared with the experimental data of Con-A and Con-B (concrete with sandstone coarse aggregate composed of 47% quartz) in Maruyama et al. (2017) and the regression curve of existing data as proposed by Field et al. (2015). It should be noted that the definition of the fluence for the curve of Field et al. (2015) is not specified as some of the existing data does not provide the neutron energy spectrum or appropriate details on the neutron fluence calculation. In addition, some aggregates may contain only slight or trace amounts of rock-forming minerals that are sensitive to the neutron irradiation and such concrete might display minor deterioration of \( f_c/f_{c0} \). Consequently, the data scatter is large.

From Fig. 29, it was found that \( f_c/f_{c0} \) in the calculated results, as well as the experimental results, follow the general decreasing trends. Notably, the results of \( \varepsilon_m/\varepsilon_a = 0.5 \) are very close to the target experiment (Con-A) results. As discussed previously, when the mortar expansion ratio is lower, the damage is increased and the reduction of \( f_c/f_{c0} \) is more significant. Under the condition that the appropriate expansion strain is given based on the neutron irradiation condition, the proposed RBSM calculation is useful to evaluate the changes in physical properties.

### 4.3 Mutual relationships between physical properties

The relationships between physical properties of concrete provide useful insights for field studies and for constitutive laws for numerical simulations, such as the FEM, which enables efficient structural performance calculations. Here, using the result of \( \varepsilon_m/\varepsilon_a = 0.25 \) for \( \varphi 75 \times 150 \) (see Table 7) and the tensile splitting test analysis (\( \varepsilon_m/\varepsilon_a = 0.25 \) in Table 8), the relationships between several properties (\( f_c, E_c, f_t, \) and compressive strain at \( f_c \)) is found. It should be noted that \( \varepsilon_m/\varepsilon_a = 0.25 \) for \( \varphi 75 \times 150 \) and the splitting tensile test analysis are comparable since the same input properties (see Table 5) and aggregate size distribution (see Fig. 15) are used.

Figure 30 shows the relationship between \( f_c/f_{c0} \) and \( E_c/E_{c0} \) from the RBSM with a comparison with the experimental results. It should be noted that the \( f_c \) values of \( \varepsilon_m/\varepsilon_a = 0.25 \) for \( \varphi 75 \times 150 \) were 45.6 MPa and 43.5 MPa in sealed and dried conditions at 70°C, respectively. It can be observed that the calculated results agree with the experimental results and the trend can be represented by a single curve. This curve suggests the possibility of evaluating the degree of deterioration of compressive strength through the change in the Young’s modulus. Once a strong correlation of static modulus and dynamic modulus is confirmed, a non-destructive test can be an effective assessment method.

### Table 10 Relationship between the \( \varepsilon_a \) of GA and the fast neutron fluence (0.1 > MeV) at 62°C.

| \( \varepsilon_a \) (%) | Neutron fluence (n/cm²) |
|----------------------|-----------------------|
| 0.1                  | \( 0.77 \times 10^{19} \) |
| 0.2                  | \( 1.27 \times 10^{19} \) |
| 0.5                  | \( 2.35 \times 10^{19} \) |
| 1.0                  | \( 3.61 \times 10^{19} \) |
| 1.5                  | \( 4.58 \times 10^{19} \) |

![Fig. 29 Calculated compressive strength ratio as a function of neutron fluence. Gray area is the standard deviation of the curve proposed by Field et al. (2015).](image-url)

![Fig. 30 Simulated relationship between the Young’s modulus ratio \( E_c/E_{c0} \) and compressive strength ratio \( f_c/f_{c0} \).](image-url)
ratio $f_{c}/f_{c0}$ and $f_{t}/f_{t0}$ is summarized in Fig. 31, with $f_{c}/f_{c0}$ found to correlate with $f_{t}/f_{t0}$, a trend that can be represented by a single curve. It is also possible that, again, the deterioration of splitting tensile strength can be evaluated by $E_{c}/E_{c0}$.

Figure 32 shows the relationship between strain at $f_{c}$ ($\varepsilon_{c}$) and $f_{c}/f_{c0}$. The strain at the maximum compressive stress is a key parameter for simulating the stress–strain curve of concrete under compression (Desayi and Krishnan 1964; Popovics 1973; Mander et al. 1988). Once this parameter is obtained, it is applicable for the numerical evaluation of the structural response, such as in the fiber model (Aktan et al. 1974) and FEM (Maekawa et al. 2003). The calculated $\varepsilon_{c}$ and $f_{c}/f_{c0}$ are found to have a strong correlation and can be evaluated by a single curve. The proposed equation is shown in Fig. 32. This equation does not correspond with $\varepsilon_{c}$ of sealed concrete (~0.21% of $\varepsilon_{c}$), while the dried cases (0.28% of $\varepsilon_{c}$) without irradiation are well-predicted by the equation. Thus, care should be taken to avoid this discrepancy when applying the proposed relation.

In summary, once the $f_{c}/f_{c0}$ or $E_{c}/E_{c0}$ of concrete under irradiation are obtained, the other parameters, such as $f_{t}/f_{t0}$ and $\varepsilon_{c}$, can be predicted.

4.4 Aging management

For the long-term use of NPPs, the degradation of concrete due to neutron irradiation is a concern. The target reinforced-concrete members are the concrete biological shielding and the reactor pressure-vessel pedestal structure. These members are very difficult to access for an experimental evaluation during reactor operation. In particular, the degraded portion of concrete biological shielding will be on the internal surface where the concrete is likely to be radioactive. In addition, an internal liner or permanent mold is present in some cases. Therefore, the aid of numerical calculation is needed to understand and evaluate the soundness of the degraded concrete. By analyzing the mineral composition of aggregates, the concrete mixture proportions, and the strain of the aggregate expansion, the proposed RBSM can predict the change in the physical properties of concrete due to expansion of the aggregate under the free restraint condition. As confirmed by a previous experiment (Con-A and Con-B in Capsule E of Maruyama et al. (2017)), it was elucidated that the restraint condition increases the strength as well as the Young’s modulus of the expanded concrete. Therefore, the calculated properties from the RBSM under the free restraint condition might be used to conservatively evaluate the strength and soundness of reinforced concrete structures.

Currently, data on aggregate expansion, which is used for the RBSM input, is difficult to obtain. However, as shown in the previous report (Maruyama et al. 2017), the aggregate GA, which is composed of $\alpha$-quartz and showed the largest expansion among the different sandstones investigated under similar temperatures as that of a commercial reactor, could be used as a conservative input. Further study is necessary to obtain the relationship between the expansive strain for different rocks and neutron fluences in the concrete of commercial reactors.

According to Figs. 20 and 30, the linear expansion strain and Young’s modulus ratio would be good factors for evaluating the compressive strength ratio. Previously, different approaches, such as homogenization theory and meso-scale modeling, have been reported for obtaining the Young’s modulus and deformation of concrete due to aggregate expansion (Pomaro et al. 2011; Salomoni et al. 2014; Giorla et al. 2015; Le Pape et al. 2015, 2016). The current results can be linked to those methodologies for the evaluation of concrete properties.

5. Conclusion

By using a rigid body–spring network model (RBSM) that explicitly considers the mortar, aggregate, and their interface, the changes in the mechanical properties of neutron-irradiated concrete were evaluated by introduc-
The volumetric expansion of the aggregate induced cracks in the mortar, aggregate, and their interface, and dramatically changed the properties of concrete. The degree of expansion of the mortar due to fine aggregate expansion and the inhomogeneous expansion of rock-forming minerals and the resultant damage in the aggregate was examined, as well as isolated mortar expansion.

From the results of this study, the following conclusions are made:

1. The ratio of concrete expansion to coarse aggregate expansion, assuming that all aggregate elements expanded, ranged from 0.71 to 0.76, depending on the amount of mortar expansion. Moreover, the calculation results are within the range of the experimental data.

2. Three phase RBSM is developed and calibrated by the loading experiment of concrete without aggregate expansion, mortar after drying, and loading experiment of aggregate. In order to simulate the compressive loading of neutron-irradiated concrete specimens, heating and drying, neutron irradiation and resultant volume expansion of aggregate, creep behavior of mortar, and compressive loading experiment were simulated by the developed RBSM. The simulated results showed a good agreement with the compressive strength and Young’s modulus of neutron irradiated concrete specimens. In particular, the case when the mortar expansion is half of the expansion of quartz in the fine aggregate showed the best agreement with the experimental data. This case is based on the experimental result of the mortar in which the quartz accounted for half of the volume, which showed a larger expansion than pure quartz (Pedersen 1971). The case when the mortar expansion is a quarter of the aggregate expansion of the quartz and with creep considered also shows the best reproduction of the experimental results. This suggests that the expansion of mortar must be considered if the expansion of rock-forming minerals under neutron irradiation occurs in the fine aggregates or creep behavior of the same order of magnitude as ordinary concrete is observed. In addition, if the rock of the fine aggregate is tolerant against neutron irradiation and the rock of the coarse aggregate is expansive under neutron irradiation, then the damage in concrete is maximized and the reduction in the strength and Young’s modulus becomes significant.

3. The change in the constitutive laws of the mortar due to the expansion of fine aggregates by neutron irradiation does not greatly affect the concrete properties after irradiation. This implies that the coarse aggregate expansion-induced cracks are the dominant factor in the strength reduction rather than the mortar deterioration.

4. The isolated expansion of the mortar, which assumed concrete with calcite for the coarse aggregate and sandstone (which exhibits expansion under the neutron irradiation) for the fine aggregate, also introduces a significant reduction of the compressive strength and the Young’s modulus of concretes. Consequently, the type of rock is critical for the behavior of concrete under irradiation.

5. The inhomogeneous expansion of rock-forming minerals in coarse aggregates and the resultant cracks in the aggregates resulted in a slight difference in the relationship between the concrete strength ratio and the liner expansion of concrete with concrete having homogeneous expansion of rock-forming minerals in the aggregates. However, the elastic modulus of concrete with inhomogeneous expansion of the rock-forming minerals in the coarse aggregates was lower than that of concrete with homogeneous aggregate expansion by 5%-10% of the original Young’s modulus at the earlier stages of concrete expansion. Cracks in the coarse aggregates cause a reduction in the Young’s modulus. However, the constitutive laws of the aggregate elements, especially after re-contact, need validation.

6. The degradation of compressive strength due to aggregate expansion is caused by the increase in damage at the maximum compressive stress. This increase in damage can be explained by i) the enlargement of the existing cracks, and ii) existing cracking being too wide to close.

7. The relationships between the compressive strength ratio and the strain at maximum compressive stress, the ratio of splitting tensile strength of concrete after aggregate expansion to the case without aggregate expansion, and the ratio of the Young’s modulus of concrete after aggregate expansion to that without aggregate expansion can be presented by a single curve, regardless of the amount of mortar expansion. It is likely that creep does not affect this curve. For the relationship for the Young’s modulus ratio, the calculated results agreed with the experimental results.

8. In general, the reduction of concrete properties under free restraint can be evaluated with the linear expansion strain of concrete. As the present calculation covers several possible concrete mixture proportions and deterioration processes, the variation of results might be comparable to realistic cases. Using the proposed RBSM, improved estimates can be found once the rock type and aggregate phases are identified.

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Appendix A Experimental procedure for drying and temperature change

The strength development of the concrete is almost stabilized after one year of aging under sealed conditions, because the cement is high-early-strength Portland cement and its rate of hydration is rapid. Therefore, additional strength development due to cement hydration during irradiation experiments was not expected.

The specimens were then shipped from Japan to the Institute for Energy Technology (IFE), Kjeller, Norway. During transportation, the temperature was kept at 5−30°C. Subsequently, they were kept at 20°C until installation into capsules (for around 2−3 months). One month before irradiation, the specimens were installed into a capsule and then inserted into the reactor core. The necessary connections were made and helium gas purging was conducted. During this process, the sample temperatures were less than 30°C.

Before the irradiation, the inside of the capsule was purged with He gas, and during irradiation, a valve was opened when the pressure inside attained 1.4 bar, which was created by water vapor evaporation from the samples and oxygen and hydrogen gases by radiolysis. Therefore, during irradiation, the samples were dried. Due to the gamma-ray heating, the temperature of the specimen cores was higher than their peripheral regions.

Appendix B Spring parameters

1) Mortar spring parameters

The mortar spring parameters in Tables 3 and 4 were determined through the compressive and splitting tensile loading using the mortar mesh. The mesh shapes were φ50 × 100 mm³ and φ100 × 25 mm³ for the compressive and splitting tensile calculations, respectively. The element size was ~2.9 mm. A calibration was performed on three strength levels: $f_{cm,macro} = 20$, 35, and 55 MPa. The other properties were calculated by Eqs. (9)–(11).

Figure B1 shows the results of the calibration, with the calculated results agreeing well with the input mortar properties. Therefore, the mortar spring parameters in Tables 3 and 4 were validated.

2) Aggregate spring parameters

A calibration of the aggregate spring parameters in Tables 3 and 4 was also performed, since no data for the springs are available and the mortar parameters in Tables 3 and 4 are invalid for the aggregate because of the dif-
ference in the shapes of their stress–strain curves. Furthermore, when a portion of the aggregate elements is expanded, as aggregate is composed of grains of different types of rock-forming minerals, some of which are sensitive to neutron irradiation (Denisov et al. 2012), cracking in the aggregate occurs.

Therefore, compressive loading including softening behavior was conducted for the calibration, and the results were compared with the literature (Wawersik and Fairhurst 1970). The cohesion \( c^* \) and the parameter for the shear-softening gradient \( \beta^* \) were calibrated: \( (c^*, \beta^*) = (0.14f_{ca\_macro}, -0.05), (0.22f_{ca\_macro}, -0.05), (0.22f_{ca\_macro}, -0.07), (0.25f_{ca\_macro}, -0.05), \) and \( (0.25f_{ca\_macro}, -0.07) \). For the input aggregate properties, the values in Table 5 and the mesh in Appendix B1) were used.

Figure B2 shows the calculated stress–strain curves for the uniaxial compressive loading. As a result, \( 0.22f_{ca\_macro} \) and \( -0.07 \) gave reasonable agreement with the target strength and steep compression softening shown in the experiment (Wawersik and Fairhurst 1970). Consequently, the aggregate parameters shown in Table 4 were employed.

Appendix C Relationship between \( f_{c}/f_{c0} \) and \( E_{c}/E_{c0} \) of the \( \varphi 40 \times 60 \text{ mm}^3 \) specimen

The relationship between \( f_{c}/f_{c0} \) and \( E_{c}/E_{c0} \) for the \( \varphi 40 \times 60 \text{ mm}^3 \) specimens are shown in Fig. C1 and compared with the experimental data. The obtained data are nearly consistent with the experimental data.

The experimental data for \( \varepsilon_{m}/\varepsilon_{a} = 0, 0.25, 0.50, 0.5 \text{ ND}, \) \( \) and \( 0.25 \text{ CR} \) are on the same trend line. This implies that the damage evolution in the concrete that is a consequence of the inhomogeneous volume change between the mortar and coarse aggregates determines the Young’s modulus and compressive strength based on the same mechanisms. The tensile creep strain and expansion of the fine aggregates and resultant volumetric change of the mortar are important. At the same time, the relationship between \( f_{c}/f_{c0} \) and \( E_{c}/E_{c0} \) is not affected by the consideration of creep and mortar expansion. Consequently, the obtained data of Fig. 31 are reliable from this perspective.

Appendix D Calculation information

The computation times of the drying analysis (720 steps), the aggregate expansion analysis (375 steps), and the loading analysis (700 steps) for \( \varepsilon_{m}/\varepsilon_{a} = 0.25 \_1.5\% \) were 33 hours, 11 hours, and 20 hours, respectively, by the supercomputer system ITO at Kyushu University with 4 cores (8 threads). In brief, total calculation time was 64 hours. The RBSM was parallelized by OpenMP. One node of ITO consists of 36 cores with a clock speed of 3.0 GHz and DDR4 memory with 192 GB.