Impact of Aggregate Properties on the Development of Shrinkage-Induced Cracking in Concrete under Restraint Conditions

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ABSTRACT

Concrete specimens were prepared with the same mixture proportion except for their constituent coarse aggregates, namely limestone and sandstone that possess different inherent drying shrinkage values. The strain at a cross section perpendicular to the drying direction under restricted and unrestrained conditions was observed using a digital image correlation method. It was confirmed that initiation and propagation of cracks were greatly affected by the types of aggregate. Analysis by the rigid-body spring network method, which explicitly incorporates aggregate shrinkage and properties of the interfacial transition zone (ITZ), revealed that aggregate shrinkage and fracture energy of the ITZ greatly influence fine-crack distribution and localization of cracks. It was elucidated that pure limestone with a small drying shrinkage value can reduce the number of visible cracks in concrete under restraint conditions since it allows fine cracks to form around coarse aggregate particles that absorb the localization of cracks, thus limiting wider cracks.

Keywords: Aggregate (D); Interfacial Transition Zone (B); Drying (A); Microcracking (B); Shrinkage (C).

1. Introduction

Cracking in concrete produced by drying shrinkage is an undesirable phenomenon since it causes disfigurement of building surfaces and jeopardizes the durability of concrete by facilitating the ingress of water, carbon dioxide, and other aggressive substances. Hence, the shrinkage mechanism of cement paste [1-10], aggregate properties and their restraint performances in concrete [11-20], the effect of aggregate particle size on shrinkage [11, 21-23], water movement behavior [17, 24-32], and formulae to predict drying shrinkage of concrete based on mixture proportions and environmental conditions [33, 34] have been thoroughly studied. Due to the high volume ratio of aggregates in concrete, the impact of aggregate properties on concrete shrinkage has gained fresh attention in recent research, consequently leading to more intense investigations.

Meanwhile, the behavior of concrete under restraint conditions has not been characterized as well as its behavior under free conditions. While numerous studies on cracking behavior and criteria for the initiation of through-cracking criteria have been conducted [35-40], in general, restrained concrete shows a complex behavior during the drying process. Concrete members contain a gradation of water content, which results in intrinsic shrinkage strains that are mutually restrained by each other within each concrete member and by an outer restraining body. Considering this inhomogeneity of stress in concrete members, the majority of the research has focused on engineering or design purposes but not on better understanding of material characteristics. Thus, in addition to through-cracking, the initiation of cracking and the propagation process at the meso-scale (scale from micrometer to millimeter) should be studied.

It has been reported that a series of concretes that had the same volumetric mixture proportions except for coarse aggregate type showed different numbers of through-cracks in reinforced concrete prism specimens under restraint conditions while differing in minor surface cracking [41, 42]. Fig. 1 presents a comparison of surface crack patterns in concrete with differing coarse aggregates. The crack patterns of two types of concrete recorded at 20°C and a relative humidity (RH) of 60% are shown on the left of Fig. 1. These concrete types contained a coarse aggregate of pure limestone and exhibited a concrete shrinkage of about 600 microns, while the concrete on the right side of Fig. 1 contained a coarse aggregate of sandstone with a large amount of clay minerals and displayed a concrete shrinkage of 1000 microns. Concrete with limestone shows a smaller number of through-cracks than concrete with sandstone as the aggregate.

The main driver of concrete shrinkage is the cement paste. A similar amount of cement paste is used to make concrete with limestone and sandstone aggregates. Therefore, ignoring the role of the cement paste allows an isolated comparison of the shrinkage caused purely by the aggregates. Under restraint conditions, concrete members produce internal stresses, and while the expected stresses in concretes with different aggregates are similar, their cracking behaviors are quite different. While a concrete mix with a limestone aggregate had a small number of through-cracks and a large number of minor surface cracks, concrete with sandstone aggregates exhibited the opposite behavior [43]. In other words, the shrunk volume of concrete is differently apportioned to through-cracks and minor/micro cracks depending on the aggregate used.

The cracking behavior of concrete is generally considered to be highly complex. Studying aggregate-dependent crack initiation can lead to a better understanding of this phenomenon and aid in formulating mechanisms to control cracking.

Although crack propagation due to drying is difficult to observe experimentally, we recently developed a measurement method for crack propagation using a digital image correlation method (DICM) to overcome this issue [44]. In the present study, we first applied a waterproof coating on the surface of concrete specimens to determine crack proportions in fabricated concrete walls, as well as to provide the background color for the DICM analysis. Second, as comparing numerical simulations with experimental data is quite informative [18, 19, 45, 46], we applied rigid-body-spring networks (RBSN) [47-49] to reproduce trends of crack initiation and propagation behavior in order to understand the impact of aggregate properties on these processes.

2. Experimental Techniques

2.1 Materials

Two concrete specimens with different shrinkage properties were prepared with a water-to-cement ratio of 0.55 using ordinary Portland cement whose properties are summarized in Table 1. The differences in shrinkage properties were realized by using two different coarse aggregates, namely limestone (GL) and sandstone (GS). GL is very pure limestone and shows almost no shrinkage, whereas GS has a large amount of chlorite and shows large shrinkage. Short-term length change isotherms of GL and GS in three orthogonal directions [50] are reproduced in Fig. 2. We denote concrete containing GL and GS as LS and SS in this study, respectively. Concrete mixture proportions of LS and SS were designed to keep the unit volume of coarse aggregate constant in order to isolate the effect of aggregate properties on the total shrinkage. In addition, in order to eliminate size effects due to differences in particle size distribution, each aggregate was first screened with sieves of 5 mm to 10 mm, 10 mm to 15 mm, and 15 mm to 20 mm, and then the three grades of aggregates were mixed uniformly. It should be noted that all the aggregates are prepared in saturated surface dry conditions before mixing. Viscosity improver was used to avoid bleeding and evade any changes in concrete quality; hence, fluidity was not controlled in this mix design. Details of the materials used are listed in Table 2. The mix proportions of concrete and their respective fresh properties are listed in Table 3 and the properties of the aggregates are listed in Table 4. The details of the experiments carried out to study aggregate properties are introduced in Section 2.2. Note that concrete specimens were demolded at the age of one day and subjected to underwater curing using a saturated calcium hydroxide solution at a temperature of 20°C ± 2°C for one year to avoid additional progression of hydration during the subsequent tests.

2.2 Aggregate properties

Short-term length change isotherms of the aggregates were determined with a humidity-controlled thermo-mechanical analyzer (BrukerAXS TMA4000SA with HC9700) to study volume changes of the aggregates. Three samples with dimensions of 3 mm × 3 mm × 6 mm were cut from each aggregate specimen with a diamond saw in three orthogonal directions taking into account their anisotropy. The original aggregates were the largest among the aggregate batches. Length changes were measured with specimens placed under controlled RH levels of 80%, 60%, 40%, and 20% at 20°C for four hours. A linear variable differential transformer with a precision of 0.5 μm, a resolution of 0.0025 μm, and a contact load of 0.098 N was used to measure changes in the lengths of the samples.

One sample was analyzed for each direction and each aggregate type; therefore, our experimental results cannot be considered as being representative values. Despite this, our results confirm a difference in shrinkage properties of LS and SS.

The Young’s modulus and Poisson’s ratio of the aggregates are also considered an important factor for preventing large shrinkage of the cement paste [11]. Therefore, these values were calculated by measuring the ultrasonic velocity of the aggregates. Ultrasonic pulse velocities of the P-wave (longitudinal elastic wave) and the S-wave (transverse elastic wave) of water-saturated aggregate samples were measured using an ultrasonic probe (V103-RM and V153-RM, Panametrics-NDT), and a pulsar-receiver (5077PR, Parametric-NDT). The voltage of the pulse oscillator was -400 V, the frequency was 1.0 MHz, and the pulse repetition frequency was 100 Hz for the transmission method. The width of the samples was measured as being 10 mm with a digital micrometer caliper with an accuracy of 0.020 mm. Reference curves were obtained by direct contact, and the period of the pulse peak in the reference curve was subtracted from the period of the pulse peak in the sample record to determine the propagation time. The pulse velocities of the P-wave (V_p) and S-wave (V_s) were calculated from the sample width and propagation time. Using the saturated aggregate density (ρ), Poisson's ratio and Young's modulus were determined by using V_p and V_s according to the following equations:

$$v = \frac{1 - 2\left(V_s / V_p\right)^2}{2 - 2\left(V_s / V_p\right)^2} \qquad (1)$$

$$E = V_p^2 \rho \frac{(1 + v)(1 - 2v)}{1 - v} \qquad (2)$$

Results of the three samples and three measurement times for each sample were averaged. One aspect of the background of this experiment is addressed here. The dynamic measurement result for the modulus of elasticity does not always correspond to the static loading result, and the value obtained by the dynamic method is generally larger than that obtained by the static loading test [51]. This is generally explained by the presence of fine cracks in the rock and this tendency is likely to be found in cases where the specimen size is large. The authors agree that the static loading test is more suitable than the dynamic method, however due to size limitations the authors selected the dynamic method to consider the properties of the aggregate.

2.3 Unrestrained shrinkage experiment

Strain distribution in a section of a concrete specimen during the drying process was evaluated by DICM, accompanied by linear deformation and mass change measurements. In the present study, we focused on the section perpendicular to the drying surface, and thus a water vapor impermeable coating to the sides of the specimens was applied.

Concrete samples were placed in a steel mold with a diameter of 100 mm and a height of 200 mm. After a one-year curing period, disks with a diameter of 100 mm and a height of 9 mm were cut out with a diamond saw. Two circular surfaces were coated with a two-part epoxy resin (Kikusui primer EPW, Kikusui Co.) to form a white vapor impermeable film as shown in Fig. 3. Preliminary testing, which constituted of periodic mass change measurements of a concrete sample fully coated by this resin, confirmed that the coating was sufficiently impermeable to water vapor and elastic enough to not affect the volume change of concrete. The details are introduced in Appendix. Black acrylic ink was sprayed on the test surfaces to create a random pattern consisting of dots with a diameter ranging from 10 μm to 100 μm for the digital image correlation analysis. Specimens were dried under conditions of 20°C ± 2°C and 60% ± 5% RH. Changes in the length of the specimens were measured with the micrometer MHN3-25MB (Mitsutoyo Co.) with a resolution of 0.001 mm and a precision of ±0.003 mm, and the corrected length of specimens was obtained by calculating the difference in length with respect to a reference stainless steel bar. Specimens were subjected to the first length measurement before drying followed by subsequent measurements every few days. Measured values of three diameters were averaged and recorded as the drying shrinkage strain. All measurements were executed in a room at a temperature of 20°C ± 2°C.

Changes in mass were measured with a precision balance with an accuracy of 0.04 g at the same time as the length measurement, and the rate of change was determined with respect to the initial mass. Each condition had three samples and averaged values are used for the discussion except for the DICM image results shown in Fig.9 and 10. In Fig.9 and 10, the sample most representative of typical results from the three samples is shown.

2.4 Details of DICM

Digital image correlation (DIC) measurements were performed with a CCD camera Atik 383L+ (Artemis CCD Ltd., 3326x2504 pixel) and an Ai AF Nikor 35mm f/2D lens (Nikon Co.) as shown in Fig. 4. A reference image was captured before drying. In this setup, each pixel had a length of 0.043 mm [44, 50].

A commercial program (Vic-2D, Correlated Solutions, Inc.) was used for the DIC analysis. An algorithm for maximizing a normalized cross-correlation criterion (NCC) between the deformed subset and the reference subset was implemented in Vic-2D. A subset of 25 pixels x 25 pixels, a step of 5 (5-pixel-spacing between centers of subset), and a decay filter (90% center-weighted Gaussian filter) with a size of 15 were applied for conducting DIC to determine local displacement and strain distributions. A cross-section containing entrained and entrapped air bubbles on the specimen surface was omitted in the image analysis due to the occurrence of defocusing and shadow dropping in air bubbles causing inaccuBackgracy in DIC results. This can sometimes cause the abortion of the DIC calculation itself. Even small cracks on the concrete surface can produce a large expansive strain in the DIC algorithm. Parameters were set based on data from our preliminary study [44]. We have also demonstrated that a positive maximum principle strain distribution is well reflected by a micro-crack distribution as confirmed by a fluorescent epoxy impregnation method [44].

Fine cracks have been detected and observed with a scanning electron microscope (SEM) and fluorescent epoxy impregnation techniques; however, these methods are not suited for observing changes in crack development. In this study, the use of DICM should allow the measurement of the development of strain distribution through discrete data and permit an informed discussion of the behavior of fine cracks.

2.5 Restricted shrinkage experiment

Specimens with dimensions of 100 mm × 100 mm × 400 mm were subjected to underwater curing with a saturated calcium hydroxide solution for a year and then cut out to have dimensions of 185 mm × 100 mm × 9 mm. Specimens were then fixed in a steel frame with two sections of acrylic adhesive as shown in Fig. 5 and subjected to a restricted shrinkage measurement. Similar to the unrestrained specimens, two cutout surfaces were coated with a vapor impermeable film, which permitted water to escape only through the sides of the specimens. Specimens were then subjected to drying at a temperature of 20°C ± 2°C and a RH of 60% ± 5%.

The target surfaces of DIC measurements were prepared by spraying black acrylic ink on the white, impermeable epoxy coating to produce a random pattern. DIC measurements were recorded with the camera system by following the protocol in the previous section 2.3 and 2.4. In the measurement setup shown in Fig. 6, each pixel had a length of 0.079 mm. For each concrete mixture, one specimen was examined. Obtained data were evaluated by comparing against values in the literature and by numerical analysis.

3. Experimental results and discussion

3.1 Unrestrained shrinkage experiments

Results of the length change measurement experiments on the disc specimens are shown in Fig. 7. The concrete total strain of SS was approximately 120 μ larger than that of LS after 91-day drying. This shrinkage strain difference was more than the theoretical calculated strain difference of 80 μ, which was estimated by multiplication of 200 μ, the difference in the aggregate shrinkage strain (taken from Fig. 2), by 0.4, the volume fraction of the coarse aggregate. This result suggests that the difference in shrinkage behavior of concrete can be mainly attributed to the drying shrinkage strain of the aggregate and that the average aggregate shrinkage difference might be more than 200 μ, while there remains a possibility that the properties of the interfacial transition zone (ITZ) influence the drying shrinkage of concrete [52, 53].

Changes in mass shown in Fig. 8 were larger for SS than for LS. This can be attributed to excess water released from the sandstone aggregate, as its water absorption is twice as large as that of the limestone. The results of DIC measurement for LS and SS are shown in Figs. 9 and 10, respectively. Based on the literature showing that expansive strains in the maximum principal strain distribution correspond to fine cracks, subsets showing strains larger than 500 μ were taken as the areas containing fine cracks [44]. Minimum principal strains obtained with DIC are shown in Fig. 9, where the LS aggregate did not show shrinkage prior to drying. With drying age, changes in color from yellow to purple were observed at mortar parts, suggesting that the shrinking zone developed from the perimeter towards the inside of the specimen. This tendency was also confirmed for SS as shown in Fig. 10. Similar to those observed in the minimum principle strain distribution, some areas showed maximum principal strain distributions changing from yellow to red (i.e., expansive strains developed from the perimeter towards the inside of the specimen, particularly around aggregates). These phenomena likely reflect crack initiation and propagation due to drying and resultant shrinkage strain distribution.

Development of numerous fine cracks (i.e., expansive strains) could be confirmed at the center of the LS specimens even at an age of 14 days, while expansive strains in SS were not significant even at an age of 44 days. The differential cracking tendency is a fertile area for further exploration. To discuss this difference in cracking progression, the minimum strain distributions over the test surface as a function of material age are summarized and shown in Fig. 11. The minimum principal strains at 2.5 mm, 11.5 mm, 24.5 mm, 37.5 mm, and 47.5 mm from the original point, averaging over the ±6.5 mm of each point on 10 radial lines with arbitrarily selected angles, were averaged. The detailed calculation procedure is represented in Fig. 11(c). A gradient of shrinkage strain can be found in a region 35 mm from the drying surface of the LS specimen even at the age of one day. The behavior of the region remained unchanged after 14 days. Taking into account the minimum principal strain distribution shown in Fig. 11(a) and (b), it can be hypothesized that the excessive shrinkage takes place due to accelerated drying caused by fine crack propagation from the surface and around the coarse aggregates, and the aforementioned development of fine cracks originates due to an uneven shrinkage behavior between limestone and mortar. The synergistic impact of crack development and acceleration of drying inside of specimen is represented in Fig. 11.

On the other hand, the gradient of shrinkage strain remained within 20 mm from the drying surface in the SS specimens. Taking into account the maximum principal strain distributions shown in Fig. 10, the shrinkage strain gradient can be attributed to an inhibition of drying due to the large shrinkage of aggregate resulting in less uneven changes in the volumes of aggregate and mortar. In addition to the intrinsic shrinkage properties of GS and mortar, the water supply from the aggregate to mortar might help maintain the moisture content of the mortar, resulting in the inhibition of uneven shrinkage between the aggregate and mortar. This mechanism seems likely since the mass change of SS was larger than that of LS as shown in Fig. 8. It has also been proven that water-saturated, porous aggregates can compensate for the drying of surrounding mortar [54, 55].

The present study has not experimentally confirmed that the aggregate-mortar ITZ influences the fine crack generation around the aggregate. However, the literature suggests that the hydrophilicity of the aggregate surface may affect crack propagation [56]. Differences in hydrophilicity cause different characteristics of the ITZ, such as the thickness of voids on the surface of aggregates, bond strength, and stiffness of the nominal mortar-aggregate interface, to arise through the influence of the cement particle packing process on the surface aggregate in fresh concrete. However, the result of a numerical analysis of unrestrained shrinkage under ideal conditions showed that the bond strength between aggregate and mortar exerts no significant effect [43]. On the other hand, it has been observed that the surface of the limestone surface is denser than that of sandstone [57], and a larger bond strength with the limestone aggregate surface would be expected [56]. Therefore, the shear stress on the limestone surface is stronger than that on the sandstone surface. In addition to this, cross cracks between aggregates are more probable in the case of limestone aggregates due to stronger bond strengths, hence facilitating easy crack propagation.

3.2 Restricted shrinkage experiment

The minimum and maximum principal strain distributions in the LS and SS specimens are shown in Figs. 12 and 13, respectively. Regions with the maximum principal strain larger than 500 μ were assumed to be origins for possible generation of fine cracks and are shown in orange or yellow in the maximum strain distribution images. Sample recordings during the progression of drying through permeable surfaces are shown from the top to the bottom of the figures. The migration of shrinkage from the periphery towards the inside can be confirmed from the minimum principal strain distributions. The maximum strain distribution images of

LS confirm that numerous fine cracks were generated at the beginning of the drying process and that they gradually progressed towards the inside of the specimen. The fine cracks did not show any interlinking with each other, at least at the observed surfaces, and were evenly distributed when the limestone aggregate was used. This can be attributed to the large difference in shrinkage and (likely) large bond strength between the mortar and limestone aggregate.

On the other hand, a few fine cracks were present at the surface of the SS specimens that proceeded to combine into a single large through-crack during drying. Differences in shrinkage between the sandstone aggregate and mortar could potentially be controlled by two factors: the intrinsic shrinking properties of the aggregate and the inhibition of the shrinkage of mortar with water released from the aggregate. Less uneven shrinkage behavior between the aggregate and mortar decreases the possibility of crack generation around aggregates. A smaller bond strength between the aggregate surface and mortar also decreases the possibility of cross cracking between aggregates. Assuming these effects, cracks generated at the surface of concrete would accelerate the drying process along the surface of cracks, and consequently promote shrinkage near the cracks. Therefore, cracks are not distributed, and instead a single crack grows towards the inside of the specimen. As a result, shrinkage cracks are integrated and localized, allowing water release for further shrinkage and localization of the crack, while progression of the crack opening may release stress around it and contribute towards closing other cracks and suppress the acceleration of drying around them.

This observed behavior of cracking in concrete under restraint conditions is consistent with the results from our previous experimental studies [41, 42]. Therefore, even though the number of specimens is limited in the current study, these present experimental results accurately reflect the typical cracking behavior in concrete as affected by aggregate properties.

4. Numerical study

4.1 RBSN

The Rigid-Body-Spring Networks (RBSN) model developed by Kawai [47] has been applied extensively for structural analysis. RBSN deals with crack propagation of concrete directly [48] since it represents a continuum material as an assembly of rigid particle elements interconnected by zero-size springs along their boundaries [47]. Being nonlinear, these zero-size springs can simulate the cracking behavior of a continuum material. In the present modeling, each interface between two rigid particles was divided into several triangles sharing the barycenter of the interfacial plane, with each triangle having three individual springs, one for a normal force and two for orthogonal tangential forces. In existing studies (for e.g., [49]), the interfacial plane has a rotation spring for bearing momentum, while in the present study, several divided triangles with springs for normal forces bearing the momentum acting on the interfacial plane as shown in Fig. 14 were used instead. At the same time, the nonlinearity of normal and tangential springs can take into account the nonlinearity of the rotation behavior on the interfacial plane.

The nonlinearity and discrete behavior of the continuum material is emulated by cracks developing at the interfaces of the rigid particles. For this reason, crack patterns and the resultant nonlinear behavior of the target model are significantly affected when a mesh design is employed. To solve this problem, random geometry using Voronoi diagrams was applied [48].

Concrete sections under restraint conditions, similar to the results shown in Section 3.2, were subjected to the numerical calculation. To evaluate the impact of aggregate properties in isolation, three different phases, namely the mortar matrix, aggregates, and mortar-aggregate interfaces, were modeled.

For the mortar matrix, the tensile behavior of mortar was modeled using linear elasticity to tensile strength, followed by a bilinear softening branch of a 1/4 model [58], as shown in Fig. 15(a). The parameters for mortar behavior in the tension field are the tensile strength f_t, the tensile fracture energy G_{ft}, and the distance between the Voronoi generators (centroid of rigid particle) h. The mortar behavior in the compression field is shown in Fig. 15(b), whose S-type curve is derived from the relationship between stress and volume under hydrostatic pressure conditions [59]. Tangential springs represent the shear transfer mechanisms of cracked and uncracked mortar matrices (Fig. 16(a)). The softening process was modeled by the following equations [59]:

$$\tau = \begin{cases} G\gamma & (\gamma < \gamma_f) \\ \max(\tau_f + K(\gamma - \gamma_f), 0.1\tau_f) & (\gamma_f < \gamma) \end{cases} \tag{3}$$

where G: shear stiffness (N/mm^2), τ_f: shear strength (N/mm^2), γ_f: strain at the maximum stress in shear strain and shear-stress relationship, and K: shear softening coefficient. A linear relationship between shear strain and shear stress was first assumed until the stress reached the peak. Following the peak, the softening process was determined by the strain and stress normal to the plane on which the shear force was acting, while the minimum value was assumed to be $0.1\tau_f$.

The shear strength was defined by the Mohr-Coulomb type criterion (Fig. 16(b)) and is represented by the following equations [59]:

$$\tau_f = \begin{cases} c - \sigma \tan\phi & (\sigma > -\sigma_b) \\ c + \sigma_b \tan\phi & (\sigma < -\sigma_b) \end{cases} \tag{4}$$

where c: cohesion parameter (N/mm^2), φ: angle of internal friction (degree), and σ_b: maximum shear strength of a normal spring (N/mm^2).

The softening process of shear springs is a function of normal stress as shown in Fig. 16 (c):

$$K = \beta G \tag{5}$$

$$\beta = \min(\beta_0 + \chi(\sigma / \sigma_b), \beta_{\max}) \tag{6}$$

where β_0, $\beta_{\max}$, χ : parameters for shear softening [59].

Shear transfer is reduced when the strain in the normal direction to the plane that shear stress acts upon is in the post-peak region. This process is represented by a coefficient β_{cr} [60] as shown in Eqs. (7) - (9):

$$\tau = \begin{cases} \beta_{cr} G\gamma & (\gamma < \gamma_{ft}) \\ \beta_{cr} \max(\tau_f + K(\gamma - \gamma_{ft}), 0.1\tau_f) & (\gamma_{ft} < \gamma) \end{cases} \quad (7)$$

$$\beta_{cr} = \frac{\varepsilon_t}{\varepsilon} \exp\left\{ \frac{\kappa}{\varepsilon_{tu}} (\varepsilon - \varepsilon_t) \right\} \quad (8)$$

$$\tau_{ft} = c - f_t \tan\phi \quad (9)$$

where $\gamma_{ft}=\tau_{ft}/G$, ε_t: strain at the peak of normal stress, ε_{tu}: ultimate normal strain when stress attains zero, ε: normal strain, and κ: reduction factor for shear transfer due to cracking.

Linear elasticity was assumed for the aggregate. In general, the strength of the aggregate is larger than that of mortar, and therefore the strength of the aggregate was not taken into account. In the present study, the amount of drying shrinkage of the aggregate was considered as a parameter. We assumed two different magnitudes of shrinkage, specifically 0 μ and −400 μ corresponding to the limestone and sandstone coarse aggregates, respectively. These shrinkage values were designed based on former research of sand stone shrinkage that found that 400μ was almost the maximum shrinkage at 60% RH in the available sandstones [61]. The Young’s modulus of aggregates may also affect the cracking behavior through a restraining role for mortar shrinkage. The average Young’s modulus of sandstone available in Japan was found to be approximately 65GPa by ultra-sonic pulse velocity measurement [62] and double of this value was used for comparison.

The interface between the aggregate and mortar (i.e., the ITZ [63]) was modeled explicitly, although quantitative data relating to the ITZ has been scarcely reported. The ITZ is generally considered to be produced by the “wall effect” of the cement particle packing process on the surface of the aggregate [64]. Our ITZ is more porous than normal mortar and different physical properties were expected as a result.

In a previous RBSN study dealing with concrete as a two-phase material, the compressive failure of cylindrical concrete specimens was accurately reproduced by taking the average of the physical properties of mortar and the aggregate as the value for the ITZ [60]. However, the tension field properties have not been studied comprehensively, although the tension in the porous ITZ zone must have a large impact on the cracking behavior of concrete.

Calculation parameters for tensile strength, Young’s modulus, and fracture energy were adopted based on a literature survey of porosity distribution findings [64-67], nano-indentation results, an SEM analysis [64, 68], and physical properties testing [56, 63, 69-71]. As the porosity in the ITZ is more than three times that of normal mortar located far from the surface of the aggregate [64, 68], the Young’s modulus and the tensile strength of the ITZ must be half of the normal mortar under the assumption of a linear or exponential relationship between porosity and physical properties. Therefore, in the present simulation, 0.50 and 0.75 times the Young’s modulus and tensile strength of normal mortar were considered for the calculations. With regard to fracture properties, Alexander et al. [72] have reported that the fracture energy of the ITZ is possibly less than 10% of that of bulk cement paste or mortar of dolomite aggregates with ordinary Portland cement and silica fume. Therefore, 0.1, 0.2, and 0.4 times the fracture energy of normal mortar were considered as the ITZ parameters.

Adopting these parameters, the different stress-strain relations in the compression and tension fields were applied to the modeling of the ITZ. The schematic is presented in Fig. 17.

Former research has found that when a limestone aggregate is used, the texture of cement hydrates in the ITZ is densified [57] and the fracture energy and bond strength of the ITZ are strengthened [56, 57]. Note that aggregate particle size and surface roughness remain important factors for ITZ properties [63, 71, 73, 74]. Therefore, these trends of ITZ properties in the case of concrete containing limestone aggregates should be considered during the interpretation of a numerical analysis.

4.2 Truss networks model for mass transfer

Water diffusion in the mortar and aggregate was modeled using a random lattice, whose mesh was defined by a Voronoi diagram, originally developed by Bolander and Berton [75]. Lineal conduit elements connect the Voronoi generators and special nodes set on boundary surfaces, which are named “Surface truss nodes”, are the centroid of surfaces of Voronoi mesh facing the boundary. The schematic of the lattice model is shown in Fig. 18 and both the Voronoi generators and surface truss nodes are shown.

The governing equation of potential flow of Eq. (10) was modeled assuming potential flow in the linear conduit as described in Eq. (11):

$$\frac{\partial w}{\partial \mu}\frac{\partial \mu}{\partial t} = \mathrm{div}\left(K(w)\,\mathrm{grad}\,\mu\right) - \frac{\partial w_{hyd}}{\partial t} \tag{10}$$

$$\frac{A_e K(w)}{L}\begin{bmatrix} 1 & -1 \\ -1 & 1 \end{bmatrix}\begin{Bmatrix} \mu_1 \\ \mu_2 \end{Bmatrix} + \frac{\partial w}{\partial \mu}\frac{1}{\omega}\frac{A_e L}{6}\begin{bmatrix} 2 & 1 \\ 1 & 2 \end{bmatrix}\begin{Bmatrix} \partial\mu_1/\partial t \\ \partial\mu_2/\partial t \end{Bmatrix} + \frac{A_b K(w)}{d_{env}}\begin{Bmatrix} \mu_1 - \mu_{env} \\ \mu_2 - \mu_{env} \end{Bmatrix} = \begin{Bmatrix} 0 \\ 0 \end{Bmatrix} \tag{11}$$

where w: volumetric water content (g/mm^3); w_{hyd}: water consumption by cement hydration (g/mm^3); $K(w)$: water transfer coefficient (mm^2/s·g/mm^3·g/J); μ_1, μ_2: chemical potential of water in conduit nodes 1 and 2, respectively (J/g); μ_{env}: chemical potential of water in the environment (J/g); d_{env}: nominal distance for the boundary condition of water transfer from the matrix to the environment (mm); t: time (s); and ω: volumetric conversion factor, (2.0 in case of 2-dimensional flow), A_e: the area of the Voronoi facet between the contiguous nodes i

and j, L: length of conduit from node 1 to 2. The third term in left hand formula represents the flow on the boundary surface. It is assumed that a hypothetical element with conduit length of d_{env} was set on the boundary surface and this will be shown in eq.(15) . In the present method, the total volume of the conduits was set as the total volume of the target Voronoi elements [49]. For the time development, the entire matrix was assembled based on eq. (11) and the Crank–Nicolson scheme was applied with equidistant time steps.

The water transfer model was based on the research of Maruyama et al. [76]. The global chemical potential of water was considered for the flow potential, and the water transfer coefficient of hardened cement paste K (w) was derived from experimental data. The results were reproduced by the following equations:

$$K(w) = \frac{1}{\left(5.0 - 9.1R + 4.15R^2\right)} K_{60} \qquad (12)$$

$$K_{60} = 1.47 \times 10^{-13} \times \exp\left(4.41 t_w\right) \qquad (13)$$

$$t_w = w_g / \rho_w / S \qquad (14)$$

where K_{60}: reference water transfer coefficient where the relative water content was 0.6 ($mm^2/s \cdot g/mm^3 \cdot g/J$); R: relative water content (-); t_w: statistical thickness of adsorption (nm); w_g: mass water content where the reference state is oven-dry conditions at 105 °C (g/mm^3); ρ_w: density of liquid water in the mortar or aggregate (0.001 g/mm^3); and S: water vapor BET surface area (mm^2/mm^3). In the present study, the water transfer coefficient of hardened cement paste was assumed to be the same as that of mortar.

The following equation was used for this boundary condition:

$$J_{w,bnd} = A_b K\left(w\right) \frac{\mu - \mu_{env}}{d_{bnd}} \qquad (15)$$

where $J_{w,bnd}$: flux at the boundary (g/s); A_b: area of the finite area on the boundary (mm^2); μ_{env}: global chemical potential of water vapor of the environment; and d_{bnd}: imaginary distance from the boundary to the environment (3 mm).

For the properties of the aggregate and the ITZ, the water capacity *dw*/*du* was based on preliminary experimental data [62], and the water transfer coefficient of the aggregate was assumed to be 10 times that of mortar since the aggregate reaches equilibrium faster than hardened cement paste according to previously measured sorption isotherms [50]. Cracked ITZ, which is expected to have a larger water transfer coefficient, was not considered in the present calculation. This assumption may produce conservative results of cracking behavior in concrete in terms of how it is affected by aggregate properties.

In the calculation, the moisture related properties of mortar are estimated from the water to cement ratio of the mixture proportion. Uniformity of the mortar matrix is assumed because a viscosity improver is used in the reference concrete. There is an additional possibility that the water in the aggregate can move during the young age of the sample due to osmotic pressure caused by the ion concentration of pore solution [55] but this phenomenon is considered negligible.

4.3 Analysis outline

In the present study, cracks that might be affected by a presence of aggregate are discussed by RBSM analysis results. Due to the limitations of meshing geometry and the calculation process of RBSM, the target cracks are yielded according to the representative mesh size, which is about 5 mm in the present study as shown in Fig. 19. In other words, the cracks within 5 mm intervals can not be shown directly in the present calculation, and the physical role of these cracks are numerically represented by a reduction of spring stiffness, which is a function of its strain. For this reason, complete reproduction of concrete behavior, which shows true multi-scale cracking and resultant change in macroscopic physical properties, and quantitative evaluation of the reproducibility of the present calculation are impossible because applicable quantitative indices can not be obtained. However, qualitative evaluation gives insight into understanding the role of aggregate in concrete with regard to the crack propagation process. Therefore, the authors attempt to obtain the key parameters of the aggregate in cracking behavior through parametric studies. Although these parametric studies are discussed by relative comparisons, the parameters used in the calculations are set to be as realistic as possible.

Specimens with dimensions of 200 mm × 100 mm × 9 mm under restraint conditions discussed in Section 3.2 were the target of the present numerical study. For the calculation, the quasi-two dimensional mesh shown in Fig. 19(a) was used. Voronoi meshing with a representative diameter of 5 mm was applied in the X-Y plane, while the same section was held in each element in the Z-direction to better understand the crack propagation process in the specimen.

The upper and bottom edges of the model were considered as the boundaries of moisture transport as shown in Fig. 19(b). The environmental conditions were considered to be 20°C and 60% RH. The boundary conditions for force equilibrium and the restraint body of stainless steel bars were modeled by a large spring whose ends were connected to a rigid plate adjoining the edge of the concrete specimen. The stiffness of the spring was calculated from the Young's modulus (205 GPa) and sections (32 mm^2 × 16 mm^2) of the members.

Calculations were performed until 91 days after drying. The shrinkage of the mortar or aggregate was modeled as a function of the relative water content ($R = w/w_0$) as shown in the following equation:

$$\Delta\varepsilon_{sh} = \alpha_{sh} \cdot \Delta R$$

where α_{sh}: coefficient of conversion from relative water content to shrinkage strain and R: relative water content (i.e., the ratio of water content w to the maximum water content w_0). The shrinkage of the mortar or aggregate was considered as an isotopic equivalent nodal force in the calculations.

In the present study, the creep of hardened cement paste was not considered since our preliminary experiments showed that the tensile creep coefficient of hardened cement paste was only 0.1 [77]. Further, given the small tensile stress in the present study, the creep strain in the hardened cement paste or mortar should not have a large impact on the calculation results. In our analytical hypothesis, tensile creep of concrete can be explained by the fine cracks dues to stress and drying shrinkage and the resultant reduction of Young's modulus of concrete [50].

Material properties and parameters used in the calculations are summarized in Tables 5 - 7. In addition, a summary of parameters for numerical analysis and a companion group showing the objectives and notations of the parameter sets are presented in Table 8, where Sh XX is shrinkage of XX microns; EaXX is Young's modulus of the aggregate of XX GPa, XXE is Young's modulus of the ITZ of XX times the Young's modulus of mortar, XXft is tensile strength of the ITZ of XX times the tensile strength of mortar, and XXGft is fracture energy of the ITZ of XX times the fracture energy of mortar.

4.4 Numerical analysis results and discussion

4.4.1 Moisture transfer

Fig. 20 shows the results of the drying process. As the analysis takes into account differences in the water capacity and the water transfer coefficient between the aggregate and mortar, oscillation in drying depth was observed. After 91 days, almost exclusively within 1 or 2 mm from the surface, the mortar attained equilibrium with the surrounding environment. On the other hand, the center of the specimen still indicated more than 74% of RH.

4.4.2 Cracking behavior

(1) Reproduction of the experimental trends

Fig. 21 represents the time-dependent cracking behavior under drying of Sh0_E135_0.4Gft and Sh400_E65_0.1Gft. No cracking was observed until the first day of drying. At three days after drying, several fine cracks were observed on the top and bottom edges of the specimens. In the case of Sh400_E65_0.1Gft, a surface crack propagated along the aggregate surfaces, and a relatively large crack was observed on the upper-left part while many cracks stayed near aggregates in the case of Sh0_E135_0.4Gft. These observations can be explained by the large shrinkage of the aggregate and the small fracture energy of the ITZ. At seven days after drying, Sh0_E135_0.4Gft exhibited a crack distribution with one growing from the top left, and the other growing from the bottom right. On the other hand, Sh400_E65_0.1Gft showed a through-crack. This can also be explained by the large shrinkage of the aggregate and the small fracture energy of the ITZ. A large aggregate shrinkage enhances crack propagation along the aggregates since they shrink during the drying process and a larger stress becomes localized on the surface of the aggregates. This tensile stress promotes crack propagation along the crack surface. Furthermore, the smaller fracture energy of the ITZ means that crack propagation reduces the fraction of the concrete specimen that can bear the total shrinkage-induced stress produced by a restraining body. Therefore, cracks can easily grow under restraint conditions at their front. Thus, the smaller fracture energy of the ITZ has an impact on the localization of cracking.

The cracking pattern in concrete is determined by both fine cracks bridging aggregates and wide cracks propagating and connecting the ITZ zones of aggregates. This is similar to the phenomena observed in Section 3.2. In the crack pattern present after 91 days of drying, crack localization is very intense in the case of Sh400_E65_0.1Gft. The close-up figures (Fig. 22) confirm wider small cracks around the aggregate in the case of Sh0_E135_0.4Gft. These results imply that Sh0_E135_0.4Gft transformed the elastic energy accumulated by restraining of shrinkage into multiple fine cracks around the aggregate, while Sh400_E65_0.1Gft did so by localizing one large through-crack. These modeled tendencies reproduced the experimental results in Section 3.2. In the next section, the contribution of each parameter to crack localization is discussed.

(2) Impact of individual parameters

Fig. 23 shows cracking behaviors after 91 days of drying, as affected by differences in Young's modulus and aggregate shrinkage. In this figure, the Young's modulus of the aggregate had little effect on the cracking pattern in concrete under restraint conditions. This can be explained by the fact that almost all the mortar was under the tension field in the X-direction, and the restraining role of the aggregate for mortar shrinkage did not have a large impact in contrast to the case of free shrinkage [78].

On the contrary, the shrinkage of the aggregate had a large impact on cracking behavior in concrete. In the case where the aggregate showed low shrinkage, small cracks were distributed around the aggregate while the concrete containing an aggregate with large shrinkage exhibited one large crack. Thus, the more similar the aggregate and mortar were in terms of their shrinkage properties, the greater was the localization of cracking in concrete under restraint conditions.

Fig. 24 shows the impact of the tensile strength and Young's modulus of the ITZ on the cracking pattern. Large cracks appeared in a different position for the Sh0_0.75E_0.5f_t condition. The results indicated that if we increase the tensile strength of ITZ with constant fracture energy, localization of cracking is confirmed.

In Fig. 25, cracking patterns in concrete as affected by the fracture energy of the ITZ are shown. During the crack development of cracking, initial crack patterns among SH0_0.1Gft, Sh0_0.2Gft, and Sh0_0.4Gft were similar almost the same until they were dried for 1 day, because the mesh geometry isare common. However after 1 day drying, But after that, grown cracks were differentiated. The smaller the fracture energy was, the more localized and wider the cracks were. The difference in cracking patterns was largest between Sh0_0.1Gft and Sh0_0.2Gft.

Crack distribution evaluated quantitatively is shown in Fig. 26. It shows the frequency of springs assorted by crack widths in a logarithmic scale. Fig. 26(a) shows the crack distribution for aggregates with different shrinkages, Fig. 26(b) shows that for different strength of ITZ, and Fig. 26(c) shows the crack distribution for different fracture energies of the ITZ. In Fig. 24(a), the concrete with a smaller aggregate shrinkage value shows a high frequency of cracks with widths ranging from 0.001 mm to 0.01 mm while large cracks of the order of 0.1 mm show a low frequency. Thus, smaller aggregate shrinkage apparently distributes energy into small cracks of 0.01 mm ~ 0.001 mm in width by way of compensation for cracks of the order of 0.1 mm in width. The same trend was observed in the cases of Sh0-0.1Gft and Sh0-0.2Gft.

These analytical studies confirmed that the localization of cracking becomes increasingly apparent when aggregate shrinkage is larger, strength of ITZ is larger, or the fracture energy of the ITZ is smaller.

As discussed in the earlier sections, aggregate type has a large impact on the properties of the ITZ and aggregate shrinkage that governs shrinkage-induced cracking in concrete under restraint conditions. Consequently, it can be concluded that the coarse aggregate of pure limestone, which shows smaller drying shrinkage and may densify the ITZ, can reduce the number of visible cracks in concrete under restraint conditions. This is since it allows fine cracks around the coarse aggregate that absorb the localization of cracking.

5. Conclusion

Concrete specimens with aggregates having different properties including drying shrinkage were prepared and a water-impermeable coating was applied to control the drying direction. Changes in the shrinkage strain distribution of the cross-section of the specimen perpendicular to the drying direction were observed with a digital image correlation method (DICM) under restricted and unrestrained conditions.

The drying shrinkage of the aggregate was found to play a dominant role in determining the drying shrinkage of concrete. The DICM confirmed that when limestone aggregates with a small drying shrinkage were used under unrestrained conditions, the difference in the drying shrinkage between the aggregate and the mortar caused cracks around the aggregate, forming shrinkage strain gradients from the drying surface to the inside. This tendency was smaller when sandstone aggregates with a larger drying shrinkage were used. This was partly due to the porosity of the sandstone aggregate, which allowed it to release excessive water to suppress drying shrinkage during the early stages of drying.

Under restricted conditions, specimens with the limestone aggregate showed discontinuous fine cracks developing both at the surface and into the interiors. Meanwhile, for specimens with the sandstone aggregate, fine cracks were distributed over the surface while a single large crack extended inside with time.

With the aid of numerical analysis, parameters that could contribute to cracking behavior in concrete, such as the Young's modulus and aggregate shrinkage, strength, stiffness, and fracture energy of the ITZ, were studied. Aggregate shrinkage and the fracture energy of the ITZ were found to govern shrinkage-induced cracking of concrete under restraint conditions. Based on both the experimental and numerical analysis results, it can be concluded that when the difference in drying shrinkage between the aggregate and mortar is considerable, or the fracture energy of the interfacial transition zone (ITZ) is very large, the distribution of fine cracks contributes to the suppression of macroscopic cracks. However, when the difference is small or the fracture energy is small, the development of a single large crack, promoted by the associated drying progression, becomes significant. This consequently leads to the formation of a localized macroscopic crack. Pure limestone, which shows smaller drying shrinkage and may densify the ITZ in concrete under restraint conditions, allows fine cracks to form around coarse aggregate particles that absorb stress and limit crack localization, and thus control macroscopic cracks.

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Appendix

A.1 Coating material performance

The impervious coating used in this study is compared with the normal concrete surface by evaporation mass per surface area. Environmental conditions during the experiment were 20 ºC and 60±5% RH. The coating material itself also showed some mass change under these conditions and the material coated on the metal surface was also measured. The results are summarized in Fig. A-1. Based on this figure, approximately 94% of vapor evaporation from the concrete surface was prevented until 150-day-drying by using the coating material. The slower evaporation rate introduced a smaller water content gradient in the specimen and can mitigate cracking on the concrete surface due to large shrinkage differences derived from a steep water content gradient.

For this reason, the impact of water vapor evaporation from the coating is considered as negligible for surface cracks caused by the internal restraint in the present study.

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