

Effects of pore structure of dolomite coarse aggregates on frost resistance of concrete

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Abstract: Freeze-thaw damage is a major durability threat to concrete in cold regions. Moving beyond the conventional emphasis on the mortar matrix and interfacial transition zone (ITZ), this study investigates the influence mechanism of the pore structure of dolomite coarse aggregates on the frost resistance of concrete. Three types of aggregates with distinct pore characteristics (clay microcrystalline dolomite aggregate with a pore volume of 1.34%, microcrystalline dolomite aggregate with a pore volume of 9.75%, and Xenotopic dolomite aggregate with a pore volume of 1.48%) were examined using a multi-scale experimental framework integrating mercury intrusion porosimetry (MIP), nano-scratch testing, and three-dimensional X-ray computed tomography (X-CT), in conjunction with 400 rapid freeze-thaw cycles. The pore structure of the aggregate exerts coupled effects on degradation of the ITZ and evolution of the internal cracks, thus serving as a strong determinant of the freeze-thaw deterioration of concrete. After cycling, compared with the concrete made with 1.48% dense and low-porosity aggregates, concrete containing a high-porosity, capillary-dominated aggregate with 9.75% porosity exhibits pronounced deterioration, where the peak stress and elastic modulus are reduced by 22.0% and 32.9%, respectively, and the ITZ fracture toughness decreases by 42.2%, accompanied by accelerated development of interconnected crack networks. In contrast, concrete prepared with a dense, low-porosity aggregate with 1.48% porosity shows improved resistance, with only an 18.6% reduction in the peak stress. A damage-dependent, nonlinear constitutive model was further developed and effectively reproduced the stress-strain response of freeze-thaw-damaged concrete. These findings provide mechanistic insight and practical support for designing frost-resistant concrete in regions where high-quality aggregates are limited and highlight the need for aggregate selection criteria that consider porosity as well as pore-structure descriptors.

Key words: freeze-thaw cycles; dolomite coarse aggregates; pore structure; concrete; constitutive model; influence mechanism

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Globally, concrete is the most widely used material in civil engineering owing to its excellent mechanical properties, cost efficiency, and durability. In China, it serves as the primary material for key railway infrastructure components such as sleepers, bridges, and tunnel linings^[1]. With the ongoing expansion of railway networks into extreme climatic zones, the demand for high-durability concrete for use in cold and high-altitude environments has become increasingly stringent, where the freeze-thaw action is one of the dominant drivers of deterioration in such service conditions^[2].

Conventional research on frost resistance has often been built on the implicit premise that coarse aggregates are high-quality and sufficiently dense, with a porosity lower than that of the mortar matrix; thus, frost damage is governed mainly by the mortar and the interfacial transition zone (ITZ)^[3]. Accordingly, the mortar and ITZ have traditionally been regarded as the most critical contributors to frost resistance. With continuous advances in high-performance concrete, the microstructure and transport resistance of mortar and the ITZ have been significantly improved^[4]. However, recent comparative studies indicate that the lithology (i. e., parent rock type) can markedly affect the freeze-thaw performance of the coarse aggregate, demonstrating that aggregates should not be treated merely as inert constituents in durability design. This shift in understanding makes it necessary to revisit frost resistance from an aggregate-centered perspective, especially when aggregates possess non-negligible pore networks^[5].

This need is further amplified by the practical constraint of a global shortage of high-quality aggregate sources^[6]. In many cold-region projects, engineers must increasingly rely on locally available aggregates with

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higher porosity or more complex pore structures, which motivates systematic investigation into how high-porosity coarse aggregates affect frost resistance and how such aggregates can be utilized in producing high-performance, frost-resistant concrete^[7]. Among the commonly used rock types, carbonate aggregates are widely adopted because of their accessibility and economic advantages^[8]. In China, magnesium-containing dolomite resources are abundant, with deposits distributed across multiple provinces and regions and a total reserve of approximately 0.3136 km³. Importantly, many of these deposits are located in frost-prone areas such as Ningxia, Jilin, and Inner Mongolia^[9]. Despite this resource endowment, a considerable portion of dolomite cannot be effectively utilized in civil engineering, making the comprehensive utilization and harnessing of dolomite increasingly promising^[10].

Existing studies provide important but still fragmented insights into aggregate-related factors. Huang et al.^[11] demonstrated the superiority of continuously graded aggregates over single-sized congeners. The size of the aggregate can indirectly influence the workability and permeability^[12], while the volume fraction of the aggregate affects the compactness and frost resistance of the resulting concrete^[13]. Beyond these macroscopic effects, aggregates with high porosity and coarse pore structures increase the water-absorption capacity of concrete^[14-15], where high absorption may increase the local moisture content in the ITZ region^[16]. This effect is critical because internal moisture is a primary precondition for freeze-thaw damage^[17].

Transport-based frameworks further indicate that the permeability of concrete depends not only on the total porosity but also on the critical size of the pore throat, connectivity of the pore network, and tortuosity^[18]. These parameters can be quantified using descriptors such as the critical pore diameter^[19]. In cement-based composites, the ITZ typically contains coarser pores. When the volume fraction of the aggregate is sufficient, these pores may connect with pore networks in the aggregate to form continuous transport pathways, accelerating the ingress of water into the interior of the concrete and potentially impairing its frost resistance^[20]. Consistently, the pore connectivity has been emphasized as a key indicator of diffusivity and permeability^[21]. These attributes of the pore network strongly influence the rate and degree of saturation of concrete, thereby governing its susceptibility to freeze-thaw damage^[22-23]. Shankaramurthy et al.^[24] confirmed that when the aggregate has a larger critical pore diameter compared to the mortar, it can significantly “coarsen” the overall pore structure of concrete, thereby accelerating the initial water absorption rate and

increasing the risk of freeze-thaw damage owing to rapid water ingress.

Field investigations at a project site further highlighted the criticality of this topic. Because transporting limestone or granite to the site was inconvenient and expensive, locally available dolomite was selected as the coarse aggregate for concrete track slabs. After service in a cold-region environment, some slabs exhibited significant freeze-thaw damage. This field evidence indicates that under realistic supply constraints where dolomite is adopted out of necessity, its mineral composition and pore-structure characteristics may govern moisture uptake and transport, the evolution of saturation, and the initiation and accumulation of freeze-thaw deterioration. However, the quantitative mechanism by which the pore structure of dolomite aggregates translates into concrete-level frost resistance remains insufficiently understood, along with the associated aggregate-matrix interfacial degradation and damage evolution. Accordingly, in this study, a systematic experimental framework is designed for investigating dolomite aggregates with varying pore-structure characteristics and to correlate the evolution of the pore-structure with the frost resistance and development of mechanical damage in concrete.

In this study, three types of dolomite aggregates quarried from the investigated project site were employed to systematically explore the mechanism by which the pore structure of coarse aggregates affects the frost resistance of concrete. The mineral composition and micromorphology of the aggregates were characterized by petrographic analysis and scanning electron microscopy (SEM). An innovative method of evaluating the frost resistance of aggregates was developed. Mercury intrusion porosimetry (MIP) was used to analyze the evolution of the pore structure of the aggregate before and after freeze-thaw cycles. Rapid freeze-thaw tests were conducted to evaluate the mass loss ratio and changes in the relative dynamic elastic modulus of concrete. Uniaxial compressive stress-strain curves obtained after different freeze-thaw cycles were used to establish a uniaxial compressive constitutive model that accounts for the degree of freeze-thaw damage. Nano-scratch tests were used to quantitatively assess the mechanical degradation of the aggregate-matrix interface under freeze-thaw conditions. X-ray computed tomography (X-CT) was used to achieve three-dimensional reconstruction and visualization of the evolution of the internal pore structure and crack propagation in concrete. This study elucidates the mechanism by which the pore structure of dolomite aggregates governs the frost resistance of concrete. The data provide practical support for the structural design of concrete employing high-porosity aggregates for applica-

tion in cold regions.

1 Experiment

1.1 Materials

The binder materials comprised P·O 42.5 ordinary Portland cement and Class F Grade I fly ash. The fine aggregate was natural river sand with a fineness modulus of 2.7 and a Zone II grading. A polycarboxylate-based, high-performance, water-reducing admixture was used, exhibiting a 27% water reduction rate and 40% solid content. Three types of dolomite coarse aggregates (Types A-C) with distinct mineral compositions were used. The aggregates comprised continuously graded crushed stone in two size fractions, namely, 5-10 and 10-20 mm. The basic properties, including the mineral composition of the parent rocks, were tested according to the technical

requirements specified in standard Q/CR 865—2022. The results of the petrographic analysis in Fig. 1 indicate that Types A, B and C aggregate exhibit a clay-bearing micritic texture, a micritic texture, and a xenotopic texture, respectively, all characterized by a massive structure. The mineral compositions of the different coarse aggregates are presented in Table 1, confirming that calcite and dolomite are the primary constituents of all three aggregate types.

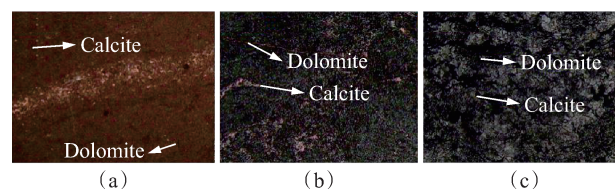


Fig. 1 Cross-polarized light images of coarse aggregates with different lithologies. (a) Type A; (b) Type B; (c) Type C

Table 1 Mineral compositions of coarse aggregates

Designation	Apparent density/ ($\text{kg} \cdot \text{m}^{-3}$)	Lithology	Texture	Mineral composition
Type A	2 650	Clay-bearing micritic dolomite	Clay-bearing micritic	Calcite 97%-99%, Dolomite 1%-3%
Type B	2 570	Micritic dolomite	Micritic	Dolomite 85%-90%, Calcite 10%-15%
Type C	2 675	Xenotopic dolomite	Xenotopic	Dolomite 96%-98%, Calcite 2%-4%

1.2 Mix proportion and specimen preparation

The concrete mix proportions used in this study are listed in Table 2. In the specimen nomenclature, the prefix “D” denotes dolomite aggregate, while the letters “A”, “B”, and “C” correspond to the three aggregate types defined in Table 1. Therefore, DA-concrete, DB-concrete, and DC-concrete refer to concretes prepared with Type A, Type B, and Type C dolomite coarse aggregates, respectively. The specimens were prepared as

follows: 27 cube specimens ($100 \text{ mm} \times 100 \text{ mm} \times 100 \text{ mm}$) per aggregate type for uniaxial compressive stress-strain curve testing at different freeze-thaw stages; 5 cube specimens ($100 \text{ mm} \times 100 \text{ mm} \times 100 \text{ mm}$) per aggregate type for preparing samples for nano-scratch testing and X-CT scanning; and 3 prism specimens ($100 \text{ mm} \times 100 \text{ mm} \times 400 \text{ mm}$) per aggregate type for analysis of the dynamic modulus of elasticity and mass loss rate after freeze-thaw cycling.

Table 2 Mix proportions of railway slab concrete

Coarse aggregate	Cement	Fly ash	River sand	Coarse aggregate		Water	Superplasticizer
				5-10 mm	10-20 mm		
DA-concrete	405	45	673	359	838	126	5.4
DB-concrete	405	45	673	351	821	126	5.4
DC-concrete	405	45	673	323	739	126	5.4

1.3 Test methods

1.3.1 Freeze-thaw cycling tests

Before testing, the coarse aggregates and concrete specimens were immersed in water tanks for 4 d. The coarse aggregates and concrete specimens were subjected to rapid freeze-thaw cycling tests for 400 cycles according to the method specified in Standard GB/T 50082—2024. After every 50 freeze-thaw cycles, three $100 \text{ mm} \times 100 \text{ mm} \times 400 \text{ mm}$ prism specimens prepared from each aggregate type were used for evaluating the dynamic modulus of elasticity and mass loss rate.

1.3.2 Construction of stress-strain curve

Uniaxial compressive stress-strain tests were performed on $100 \text{ mm} \times 100 \text{ mm} \times 100 \text{ mm}$ cube specimens after the specified number of freeze-thaw cycles. The loading protocol was as follows: displacement-controlled loading at 0.5 mm/min was applied when the load was below 10 kN. This was followed by force-controlled loading at 2 kN/s from 10 to 550 kN. Finally, displacement-controlled loading at 0.02 mm/min was applied until failure. Three specimens were tested for each group, and the average curve was taken as the final stress-

strain curve.

1.3.3 Evaluation of performance of coarse aggregate

Each coarse aggregate type (10 kg) was weighed according to the mix proportion and sieved through a 4.75 mm square-mesh sieve to remove particles smaller than 4.75 mm. Subsequently, 5 kg portions of each aggregate were weighed as the initial mass (m_0), placed in specimen containers for rapid freeze-thaw testing, and subjected to freeze-thaw cycling alongside the other specimens. After every 50 freeze-thaw cycles, the aggregates were resieved through the 4.75 mm sieve and the residual mass (m_i) was recorded. The mass loss rate was calculated as

$$L_{m,agg} = \frac{m_0 - m_i}{m_0} \times 100\% \quad (1)$$

After testing, the aggregates were returned to the containers for subsequent cycling.

SEM analysis employing a high-resolution Zeiss Merlin Compact field-emission scanning electron microscope was used to observe the internal crystal structures. After 400 freeze-thaw cycles, 3–5 mm bulk particles (total mass of ~2 g) were prepared from the pre- and post-cycling aggregates. Pore structure analysis was conducted using a MicroActive AutoPore V 9600 mercury porosimeter.

1.3.4 Nano-scratch test

Specimens measuring 15 mm × 15 mm × 5 mm were sectioned from three concrete cube samples subjected to 0, 100, 200, 300, and 400 freeze-thaw cycles, then polished following a meticulous preparation procedure to ensure surface flatness and minimize measurement errors. The sectioned specimens were sequentially wet-ground using diamond grinding discs with grit sizes of 400, 1 200, and 2 000 mesh, followed by final polishing with a cloth impregnated with a 0.25 μm colloidal silica suspension. After polishing, the samples were ultrasonically cleaned in anhydrous ethanol and dried to remove surface contaminants and loose particles. The tests were conducted using a Keysight G200 nanoindenter equipped with a Berkovich indenter (with a tip radius of ~20 nm and angle between the edge and centerline of 65.3°, providing a displacement resolution of 0.01 nm).

A three-stage scanning protocol was implemented. This method comprised a preliminary surface roughness assessment via a 20 μN pre-scan, followed by main scratch scanning under 50 mN load, and a final 20 μN post-scan to determine the residual scratch depth. The indenter traversed the sample at 4 μm/s with three parallel scratches per specimen spaced 60 μm apart, producing a total scratch length of 200 μm. The average depth of the three scratches was used to determine the microfracture toughness.

$$R_c = \frac{F_T}{\sqrt{2l(d)A(d)}} \quad (2)$$

where R_c is the microfracture toughness; F_T is the tangential load; d is the depth of the scratch; $l(d)$ is the edge length of the measured volume; $A(d)$ is the projection of the measured volume on the plane perpendicular to the direction of the scratch; and $l(d)A(d) = 2.2808d^{2.5}$ [25].

1.3.5 X-CT scanning test

Cylindrical specimens (Φ30 mm × 30 mm) were core-drilled from concrete cubes subjected to 0, 100, 200, 300, and 400 freeze-thaw cycles. X-CT scanning was performed using a Carl Zeiss Xradia 620 Versa system operated at 140 kV with a 1 s exposure time and 16 μm voxel resolution. Three-dimensional reconstruction was used to analyze the evolution of the pore structures and 3D cracks in the concrete subjected to different freeze-thaw cycles.

2 Results and Discussion

2.1 Characterization of coarse aggregates

2.1.1 Mass loss rate of coarse aggregates

The mass loss rates of the three types of dolomite coarse aggregates subjected to freeze-thaw cycling are shown in Fig. 2. Before 150 cycles, the mass loss rate was comparable and gradual for Types A and C aggregates. A notable divergence occurred at 200 cycles, where the mass loss rate of Type A sample increased sharply. In contrast, the mass loss rate of Type B aggregate increased in an approximately linear fashion throughout the test, where this sample consistently exhibited the highest mass loss rate across all cycles. The mass loss rate was consistently lowest for Type C aggregates, indicating superior resistance to freeze-thaw damage. The distinct mass loss patterns of the three aggregate types underscore the critical role of the pore structure characteristics in determining the durability of the aggregates under freeze-thaw conditions.

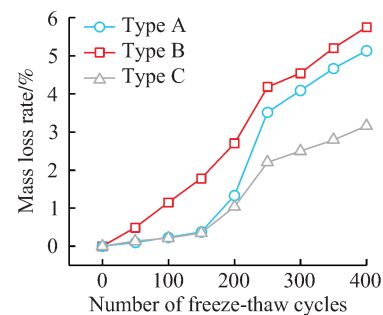


Fig. 2 Mass loss rate of different type of coarse aggregates after freeze-thaw cycles

2.1.2 Micromorphology of coarse aggregates

To reveal the mechanism underlying the mass loss variations, SEM was used to observe the micromorphol-

ogy of the different coarse aggregates (Fig. 3). Type A aggregates comprised generally fine particles with localized regions of loosely packed particles; these structural inhomogeneities likely served as initiation sites for microcracking, leading to the abrupt mass loss observed after 200 cycles. Type B aggregates exhibited pronounced microcracking, attributable to the compositional disparity between dolomite ($\text{CaMg}(\text{CO}_3)_2$) and calcite (CaCO_3). This disparity weakened the interparticle bonding, and the pervasive microcracks and weak interfacial zones significantly compromised the frost resistance, yielding the highest mass loss. Type C aggregates were characterized by a densely packed microstructure without significant cracking or structural discontinuities; this homogeneous and coherent microstructure effectively inhibited crack initiation and propagation, correlating with the optimal frost resistance of this aggregate type.

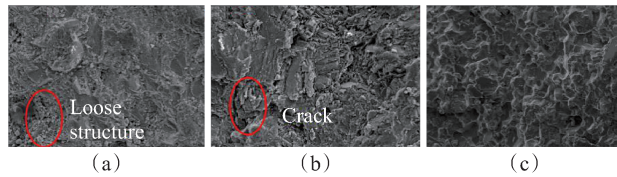


Fig. 3 Micro-morphologies of three types of coarse aggregate. (a) Type A; (b) Type B; (c) Type C

2.1.3 Evolution of pore structure of coarse aggregates

Assessing the pore structure characteristics of the aggregates, classified according to Butt's classification^[26], is essential for understanding moisture movement in the aggregates and the corresponding freeze-thaw behavior^[27]. The measurements in Fig. 4 revealed initial porosity values of 1.34%, 9.75%, and 1.48% for Types A, B, and C aggregates, respectively. After 400 freeze-thaw cycles, the total porosity increased to 3.70%, 13.12%, and 2.81%, representing increases of 2.36%, 3.37%, and 1.33%, respectively.

Before freeze-thaw cycling, the proportion of capillary pores in Types A, B, and C coarse aggregates was 7.18%, 82.72%, and 2.62%, respectively, and the corresponding absolute capillary porosity was 0.10%, 8.07%, and 0.04%. This indicates that Type B coarse aggregate has the highest proportion of internal capillary pores. Compared with the other aggregates, connectivity between the capillary pores of the aggregate and the pore network within the cement mortar matrix during concrete preparation is more likely. This phenomenon promotes water absorption and consequently makes the concrete more susceptible to freeze-thaw damage. After freeze-thaw cycling, the capillary porosity of Types A, B, and C coarse aggregates increased to 0.35%, 9.68%, and 0.10%, respectively. Overall, the total porosity and capillary porosity of Type B coarse aggregate before and af-

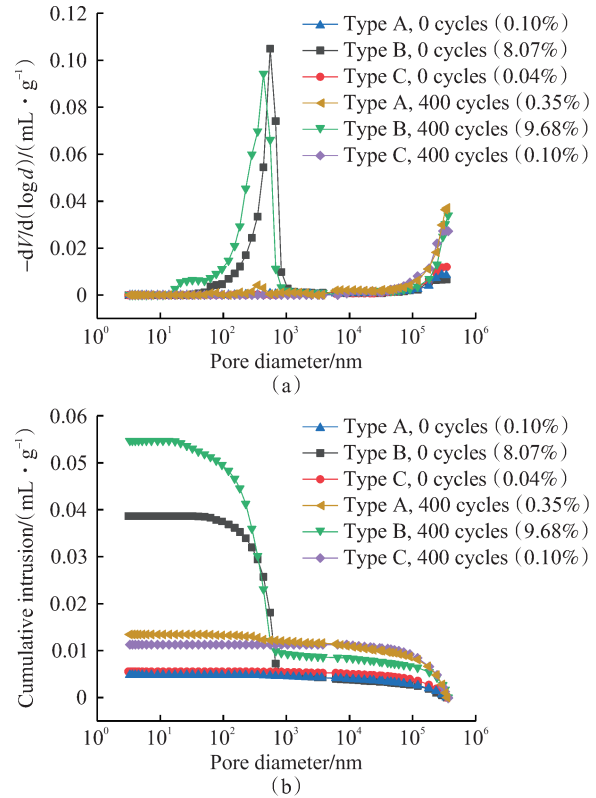


Fig. 4 Variation of pore structure of coarse aggregates before and after freeze-thaw cycles. (a) $-dV/d(\log d)$; (b) Cumulative intrusion

ter freeze-thaw cycling were higher than those of Types A and C aggregates. Type B aggregates also exhibited the largest increase in porosity. To more intuitively characterize the pore structure of the coarse aggregates, the water absorption rates of Types A, B, and C coarse aggregates were measured before freeze-thaw cycling, yielding values of 0.12%, 0.74%, and 0.11%, respectively.

The similar initial porosity of the Types A and C aggregates accounts for their comparable performance in the early freeze-thaw stages. However, the superior chemical stability of dolomite compared to that of the more soluble calcite in Type A aggregates conferred enhanced durability to Type C, minimizing damage progression in the latter. Type B aggregates, with a high initial capillary porosity and heterogeneous mineral composition, facilitated water intrusion and promoted crack propagation through synergistic freeze-thaw and dissolution mechanisms. Consequently, Type B aggregates exhibited the most significant porosity increase and the poorest frost resistance, underscoring the critical role of the pore structure and mineral composition in determining the durability of the aggregates.

2.1.4 Analysis of fractal dimension of coarse aggregates

To better characterize the connectivity of the internal pore system within the coarse aggregates, the fractal di-

mension D of the pore structure was quantitatively analyzed. D is commonly employed as an indicator of the complexity of pore networks^[28]. Here, D for the pore network of the coarse aggregates was quantified using a thermodynamically derived fractal model, for which the predictive fidelity surpasses that of conventional geometric analogs such as the Menger sponge^[29]. The core equation for calculating D is given as

$$\ln\left(\frac{W_n}{r_n^2}\right) = D \ln(Q'_n) + C \quad (3)$$

where r_n denotes the pore radius at the n -th level, nm; W_n represents the cumulative mercury intrusion volume corresponding to the pore radius r_n , mL/g; Q'_n is the thermodynamic work required for the intrusion of mercury into the pore network under the applied pressure; and C is a fitting constant.

The relevant parameters from the MIP tests are substituted into this equation^[30]. The variation in D for the different types of coarse aggregates before and after freeze-thaw cycling is illustrated in Fig. 5.

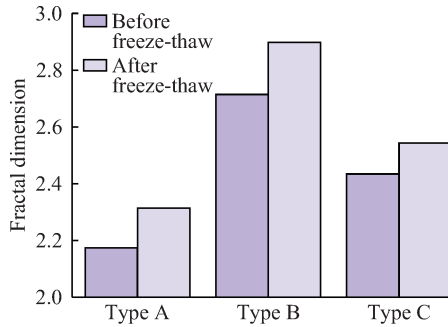


Fig. 5 Fractal dimension of coarse aggregates before and after freeze-thaw cycles

As shown in Fig. 5, the fractal dimension of the coarse aggregates increased after freeze-thaw exposure, indicating a consistent increase in the complexity of the internal pore systems of the aggregates. Specifically, D for Type A aggregates increased from 2.174 to 2.314, representing a 6.42% increase. A more pronounced change from 2.714 to 2.898 was observed for Type B aggregates, representing a 6.78% increase. In comparison, Type C aggregates exhibited a relatively modest increase from 2.434 to 2.543, representing a 4.48% increase. Overall, these results suggest that freeze-thaw action drives the pore structure of the aggregates toward a more complex and heterogeneous configuration. This trend can be attributed to freeze-induced expansion and progressive micro-damage, which promote pore enlargement and the formation of interconnected pore networks.

An increase in D generally implies a more irregular pore geometry and a denser, more intricate pore network. Such a pore system tends to exhibit higher connectivity, providing additional pathways for moisture trans-

port, thereby accelerating water ingress and redistribution within the aggregate. Under freeze-thaw conditions, this enhanced connectivity increases the extent of internal saturation and amplifies ice-induced pressures during freezing^[31]. This promotes microcrack initiation, propagation, and coalescence, leading to accelerated frost deterioration. Consequently, aggregates exhibiting a larger increase in D are expected to exhibit lower frost resistance. This interpretation is consistent with the comparatively higher mass loss of Type B aggregates during freeze-thaw cycling and further suggests that concrete produced with Type B aggregates is more likely to develop more severe internal damage under repeated freeze-thaw action than the concretes incorporating Type A or Type C aggregates.

2.2 Frost resistance of concrete

2.2.1 Evolution of frost damage and underlying mechanisms

The macroscopic appearance, mass loss rate, and relative dynamic elastic modulus jointly serve as indicators of the deterioration of concrete under freeze-thaw cycling. As shown in Fig. 6, the surface damage evolved differently among the three concrete types. Before 100 cycles, only DB-concrete exhibited cement-mortar spalling. After 200 cycles, DA-concrete and DC-concrete developed pitted surfaces with scattered spalling, whereas DB-concrete was characterized by deeper voids and exposed aggregates. After 300 cycles, severe mortar spalling became widespread in the DA-concrete and DC-concrete; localized cracking and aggregate fracture were observed in the DB-concrete. After 400 cycles, the damage in the DA-concrete and DC-concrete progressed mainly through pit deepening, while DB-concrete experienced extensive spalling and macrocracking.

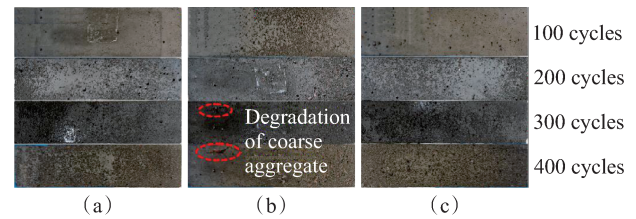


Fig. 6 Apparent morphology of specimens after freeze-thaw cycles. (a) DA-concrete; (b) DB-concrete; (c) DC-concrete

Consistent with the morphological observations, the changes in the mass of the samples (Fig. 7(a)) were used to further quantify the freeze-thaw resistance. During the early stage of cycling, roughly the first 80 cycles, all concrete specimens exhibited an apparent mass increase, primarily attributed to progressive moisture uptake. Freeze-thaw action can increase the pore connectivity and promote microcrack development in the cement paste and ITZ, thereby enhancing capillary suc-

tion and water ingress during the thawing phase. Consequently, the gain in the absorbed water can temporarily exceed the negligible surface scaling at this stage, leading to a net mass increase^[32].

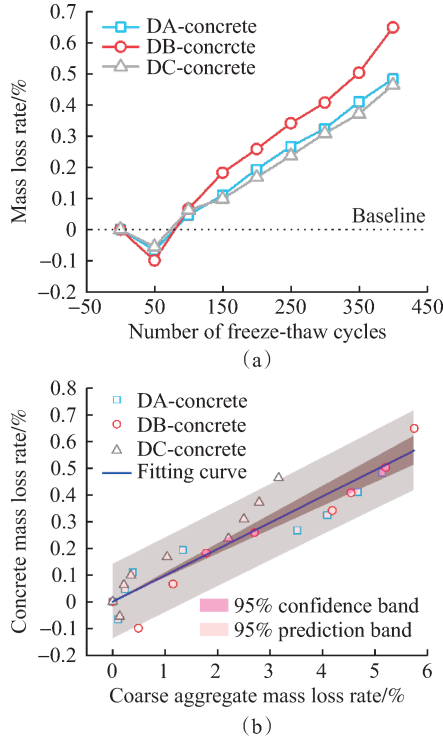


Fig. 7 Analysis of mass loss rate. (a) Mass loss rate of different type of concrete after freeze-thaw cycles; (b) Variation of concrete mass loss rate with coarse aggregate mass loss rate

This phenomenon was not observed for the aggregates themselves because the aggregates are rapidly driven to near-saturation after pre-soaking, leaving limited additional capacity for further water uptake. Thus, the assessment of the mass change of the aggregate mainly reflects particle loss after sieving rather than incremental internal moisture retention. Among the three mixes, DB-concrete absorbed the most water, consistent with its higher porosity. After sufficient cycling, the mass loss became dominant, where the mass loss rate was highest for DB-concrete (0.65%), followed by DA-concrete (0.48%) and DC-concrete (0.46%).

To assess the robustness and generalization capability of the regression model constrained by small sample sizes, leave-one-out cross validation (LOOCV) was used to assess the correlations among the key parameters. For a dataset containing n samples, the i -th sample is sequentially taken as the test sample, and the remaining $n-1$ samples are used for model fitting. The prediction error for the held-out sample is then computed. After repeating this procedure for all n samples, the overall cross-validation error is obtained by summing the prediction errors. The LOOCV errors were quantified using the root mean square error and mean absolute error, denoted

by u and v , respectively:

$$u = \sqrt{\frac{1}{n} \sum_{i=1}^n (y_i - \hat{y}_{-i})^2} \quad (4)$$

$$v = \frac{1}{n} \sum_{i=1}^n |y_i - \hat{y}_{-i}| \quad (5)$$

where y_i is the observed value of the i -th sample; $\hat{y}_{-i}$ is the predicted value obtained from the model trained using all samples except the i -th one. RMSE and MAE calculated on the training set quantify the model's fitting accuracy, with RMSE being sensitive to larger deviations and MAE reflecting the average absolute error.

When LOOCV-RMSE (u) and LOOCV-MAE (v) are close to the corresponding training RMSE and MAE, the model demonstrates satisfactory robustness and no evident overfitting. This validation procedure was applied consistently throughout the subsequent regression analyses, and the cross-validation errors are reported alongside the training errors.

To ensure the reliability of the regression analyses, a standardized statistical evaluation framework was established. The significance level (p -value) was used to test whether the regression coefficients (e. g., the slope in the linear models and the quadratic-term coefficient in the nonlinear models) are significantly different from zero. Pearson's correlation coefficient (r) was employed for the initial assessment of the linear association between variables. The coefficient of determination (R^2) was used to evaluate the overall goodness-of-fit of the selected regression model (including both linear and nonlinear forms). The RMSE and MAE were also determined to quantify the absolute prediction error. Furthermore, LOOCV was performed, and the resulting LOOCV-RMSE and LOOCV-MAE were used to assess the generalization ability and robustness of the model constrained by small sample sizes; this validation procedure is not repeated in subsequent sections.

Fig. 7(b) presents the regression analysis of the correlation between the mass loss ratio of the concrete $L_{m,con}$ and that of the coarse aggregate $L_{m,agg}$. Based on 27 datasets ($n = 27$), a significant and positive proportional relationship between the two variables was observed. The linear regression expression is given in Eq. (6). The significance test for the regression slope yielded $p = 4.66 \times 10^{-18}$. The model showed a high goodness of fit, with $R^2 = 0.947$ and $r = 0.940$ ($p = 3.34 \times 10^{-13}$), indicating that the linear correlation is extremely significant in a statistical sense. The prediction errors were low (RMSE is 0.066 8 and MAE is 0.053 7), and LOOCV-RMSE (0.069 5) and LOOCV-MAE (0.055 9) were close to the training errors, suggesting good robustness of the fitted model. These results quantitatively support the abovementioned conclusion in the main text that the dura-

bility of coarse aggregates is a key factor governing the overall frost resistance of concrete and further validate the rationality.

$$L_{m, \text{con}} = 0.0985 L_{m, \text{agg}} \quad (6)$$

The relative dynamic elastic modulus (Fig. 8(a)) provides a measure of the deterioration of the internal microstructure of the samples. After 400 cycles, DA-concrete and DC-concrete retained 94.6% of their initial modulus, whereas that of DB-concrete declined to 90.6%. The pronounced reduction in the initial modulus of DB-concrete is attributed to its higher porosity and pre-existing microcracks, which facilitate water penetration and exacerbate ice-induced expansion during freezing. The modulus of DA-concrete and DC-concrete followed similar decay trends, reflecting the comparable pore characteristics of these samples and confirming that the porosity and pore structure are the dominant factors governing the freeze-thaw performance of concrete.

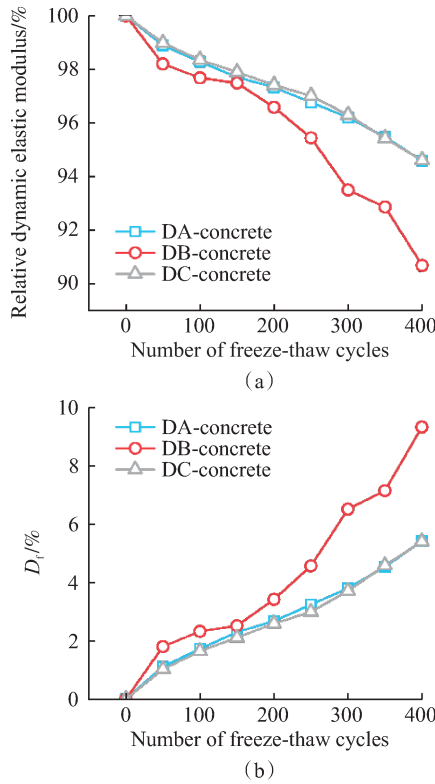


Fig. 8 Analysis of relative dynamic elastic modulus. (a) Relative dynamic elastic modulus of concrete; (b) Change in damage variable with different freeze-thaw cycles

The damage variable D_f derived from the relative dynamic elastic modulus based on the following equation enables quantitative analysis of the progression of internal damage in concrete.

$$D_f = 1 - E_{dN}/E_{d0} \quad (7)$$

where E_{dN} is the relative dynamic elastic modulus after N freeze-thaw cycles, and E_{d0} is the initial relative dynamic elastic modulus (0 cycles).

As illustrated in Fig. 8(b), D_f increased most rapidly in DB-concrete, consistent with its inferior pore characteristics.

2.2.2 Stress-strain curves

To characterize the strength degradation in the three concrete types under freeze-thaw cycling, uniaxial compressive stress-strain curves were constructed after different cycles. The stress-strain curves of the concrete with various freeze-thaw cycles are shown in Fig. 9.

The peak stress of all concrete mixes decreased monotonically as the number of freeze-thaw cycles increased (Fig. 10(a)). Before freeze-thaw cycling, the peak stress was comparable for the three mixes, with values of 78.5, 79.0, and 81.7 MPa for DA-concrete, DB-concrete, and DC-concrete, respectively. Throughout the test, DC-concrete consistently exhibited the highest peak stress and the slowest deterioration rate, indicating stronger resistance to freeze-thaw damage.

Before approximately 200 cycles, DA-concrete and DB-concrete followed nearly identical degradation trends, with only minor differences in the peak stress. Beyond this stage, the strength loss accelerated for DB-concrete and its peak stress became markedly lower than that of DA-concrete, whereas that of DC-concrete remained the highest. Accordingly, before 200 cycles, the residual peak-stress of DC-concrete was greater than that of DA-concrete and DB-concrete, and the residual peak-stress of DA-concrete was approximately equal to that of DB-concrete; after 200 cycles, the residual peak-stress decreased sequentially in the order of DC-concrete, DA-concrete and DB-concrete. The accelerated late-stage deterioration of DB-concrete may be associated with the higher porosity of the aggregate and the capillary-dominated pore structure of the concrete.

Fig. 10(b) presents a statistical examination of the relationship between the peak stress reduction percentage R_{ps} and the damage variable D_f . Based on regression analysis of $n = 27$ samples, a linear relationship was obtained, as expressed in Eq. (8), with a significant slope ($p = 1.19 \times 10^{-20}$). This linear correlation is extremely significant, with $r = 0.955$ ($p = 1.12 \times 10^{-14}$) and $R^2 = 0.966$. The model also shows good predictive accuracy (RMSE is 1.975 and MAE is 1.532). The LOOCV-RMSE (2.123) and LOOCV-MAE (1.627) further confirm the robustness of the model. Overall, this analysis provides solid statistical significance support for the following equation.

$$R_{ps} = 2.7192 D_f \quad (8)$$

Fig. 11(a) shows the evolution of the peak strain with the freeze-thaw cycles. Before cycling, the peak strains were 1.436×10^{-3} , 1.469×10^{-3} , and 1.412×10^{-3} for DA-concrete, DB-concrete, and DC-concrete, respec-

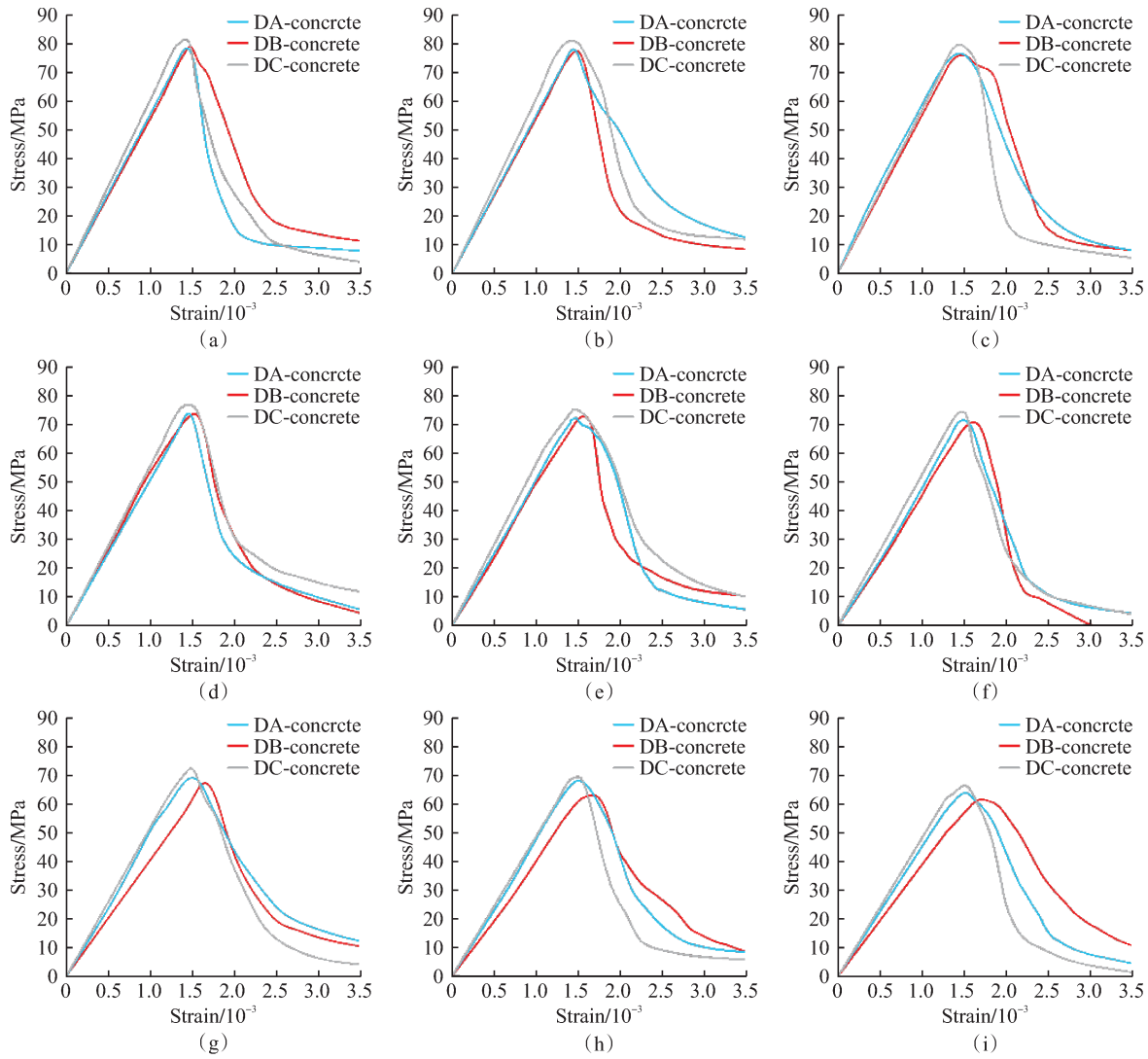


Fig. 9 Stress-strain curves of concrete with different freeze-thaw cycles. (a) 0 freeze-thaw cycles; (b) 50 freeze-thaw cycles; (c) 100 freeze-thaw cycles; (d) 150 freeze-thaw cycles; (e) 200 freeze-thaw cycles; (f) 250 freeze-thaw cycles; (g) 300 freeze-thaw cycles; (h) 350 freeze-thaw cycles; (i) 400 freeze-thaw cycles

tively. The increase in the peak strain per cycle followed the order DC-concrete, DA-concrete, and then DB-concrete. DB-concrete exhibited the highest peak strain and the fastest increase, reflecting more severe development of internal damage in this specimen.

Fig. 11 (b) investigates the relationship between the percentage increase in the peak strain I_{ps} and the damage variable D_f using regression analysis. Based on $n = 27$ datasets, a statistically significant and positive linear correlation was observed, consistent with Eq. (9); the regression slope was significant ($p = 1.80 \times 10^{-16}$). The model provides a good fit, with $R^2 = 0.929$ and a high Pearson correlation coefficient ($r = 0.945$; $p = 1.24 \times 10^{-13}$). The prediction errors are relatively low (RMSE is 1.617 and MAE is 1.362). Moreover, LOOCV-RMSE (1.744) and LOOCV-MAE (1.447) did not increase significantly compared with the training errors, indicating good stability and robustness of the model. There-

fore, the linear relationship is supported by solid statistical evidence.

$$I_{ps} = 1.5089D_f \quad (9)$$

The elastic modulus was obtained by linearly fitting the ascending segment of the stress-strain curves of the concrete samples. Fig. 12 (a) indicates that before cycling, the elastic modulus was highest for DC-concrete at 57.8 GPa, followed by DA-concrete at 54.7 GPa and DB-concrete at 53.8 GPa. After 400 cycles, the elastic modulus of DB-concrete exhibited the largest reduction of 32.9%, and that of DA-concrete and DC-concrete showed comparable decreases of 22.9% and 23.7%, respectively. The residual elastic modulus was highest for DC-concrete at 44.1 GPa, intermediate for DA-concrete at 42.2 GPa, and lowest for DB-concrete at 36.1 GPa. The similar decay in the modulus of DA-concrete and DC-concrete originates from the comparable porosity of the aggregates in both samples, leading to analogous

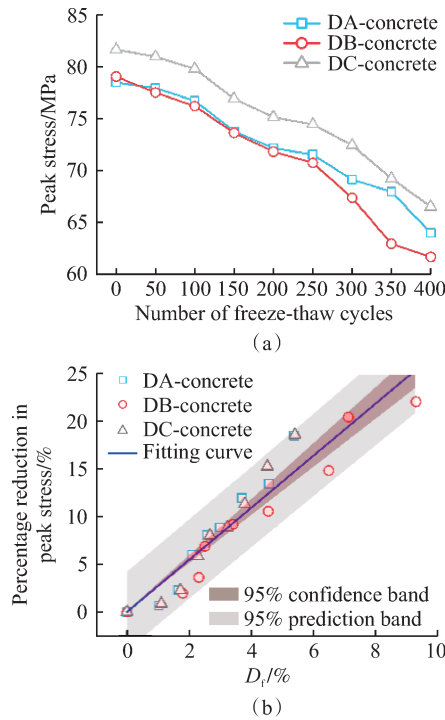


Fig. 10 Analysis of peak stress. (a) Relationship between peak stress and number of freeze-thaw cycles; (b) Relationship between reduction percentage in peak stress and freeze-thaw damage variable

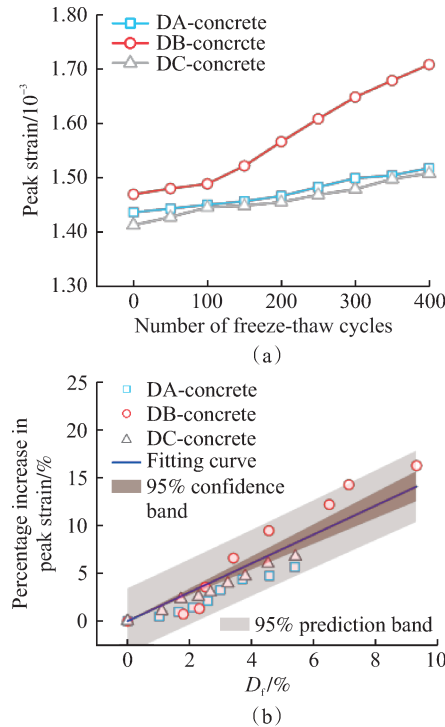


Fig. 11 Analysis of peak strain. (a) Relationship between peak strain and number of freeze-thaw cycles; (b) Relationship between percentage increase in peak strain and freeze-thaw damage variable

freeze-thaw damage levels, given that the mineral composition exerted a negligible influence on the elastic modulus. The aggregates in DB-concrete had the maxi-

mum porosity and caused the most severe degradation in the elastic modulus of the concrete during freeze-thaw cycling.

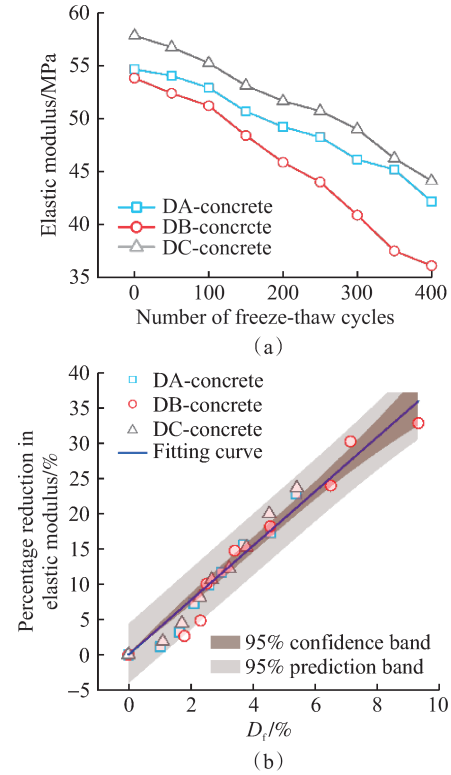


Fig. 12 Analysis of elastic modulus. (a) Relationship between elastic modulus and number of freeze-thaw cycles; (b) Relationship between percentage reduction in elastic modulus and freeze-thaw damage variable

To clarify the quantitative relationship between the reduction ratio of the elastic modulus R_{pe} and the damage variable D_f , a second-order polynomial regression was performed using $n = 27$ experimental datasets (Fig. 12 (b)). The results reveal a significant linear relationship between the two variables, with a notably significant regression slope ($p = 1.87 \times 10^{-24}$). The model exhibited excellent goodness-of-fit ($R^2 = 0.983$) and r as high as 0.979 ($p = 6.86 \times 10^{-19}$). The prediction errors were relatively small, with RMSE of 1.988 and MAE of 1.494. Moreover, LOOCV-RMSE (2.103) and LOOCV-MAE (1.577) were close to the training errors, indicating robust model performance. Therefore, the relationship demonstrates very strong statistical significance and reliability.

$$R_{pe} = -6.7891 \times 10^{-4} D_f^2 + 3.8698 D_f \quad (10)$$

Note that the rapid freeze-thaw regime used in this study follows GB/T 50082—2024 and is an accelerated testing protocol. In this test, the minimum temperature is -20°C , each cycle lasts 4 h, and the specimens are typically maintained in a highly saturated state during cycling. Compared with natural exposure in cold regions, freeze-thaw actions in the field generally involve variable

temperature amplitudes and rates that are often slower and span longer durations; the minimum temperature may be lower, and the moisture boundary conditions are not constant.

Within this context, however, the qualitative conclusion that the pore structure of the aggregate governs the internal saturation degree and transport behavior, ITZ degradation, and subsequent crack evolution should remain applicable and instructive. The deterioration rate and key uniaxial compressive stress-strain parameters obtained under rapid cycling in this study should be interpreted as conservative estimates for severe, high-saturation conditions^[33]. Future work may employ regional climate data and outdoor exposure validation for localized calibration, thereby establishing an equivalent-cycle coefficient for linking laboratory tests to field conditions in different regions. This would provide more accurate guidance for tailoring the durability of concrete in cold regions^[34].

2.2.3 Uniaxial compressive constitutive model

Based on the physical meanings of the parameters in the uniaxial compressive stress-strain curves obtained in this study, the percentage reduction in the peak stress R_{ps} , percentage increase in the peak strain I_{ps} , and percentage reduction in the elastic modulus R_{pe} are defined as follows:

$$R_{ps} = 1 - \sigma_{c,r}/\sigma_{c,r0} \quad (11)$$

$$I_{ps} = \varepsilon_{c,r}/\varepsilon_{c,r0} - 1 \quad (12)$$

$$R_{pe} = 1 - E_c/E_{c0} \quad (13)$$

where $\sigma_{c,r}$, $\varepsilon_{c,r}$, and E_c denote the peak stress, peak strain, and elastic modulus after N freeze-thaw cycles, respectively; $\sigma_{c,r0}$, $\varepsilon_{c,r0}$, and E_{c0} denote the peak stress, peak strain, and elastic modulus in the unfrozen state. These variation rates were plotted versus D_f to determine the parameters from the stress-strain curve.

To describe the uniaxial compressive response of concrete subjected to freeze-thaw cycling, a damage-mechanics-based uniaxial compressive constitutive model recommended in the Chinese standard GB/T 50010—2010 was adopted. The nominal stress σ is given by

$$\sigma = (1 - D_c)E_c\varepsilon \quad (14)$$

where ε is the axial strain, and D_c is the compressive damage variable.

D_c is defined in a piecewise form in terms of the normalized strain x , leading to the following equations.

$$x = \frac{\varepsilon}{\varepsilon_{c,r}} \quad (15)$$

$$D_c = \begin{cases} 1 - \frac{\rho_c n}{n - 1 + x^n} & x \leq 1 \\ 1 - \frac{\rho_c}{\alpha_c (x - 1)^2 + x} & x > 1 \end{cases} \quad (16)$$

$$\rho_c = \frac{\sigma_{c,r}}{E_c \varepsilon_{c,r}} \quad (17)$$

$$n = \frac{E_c \varepsilon_{c,r}}{E_c \varepsilon_{c,r} - \sigma_{c,r}} \quad (18)$$

In Eq. (16), α_c governs the post-peak branch. To ensure continuity of the ascending branch at $x = 1$, the following equation is adopted, such that the stress-strain curve transitions smoothly and continuously at the peak into the post-peak branch.

$$\rho_n = n\rho_c \quad (19)$$

Substituting Eqs. (16)–(19) into Eq. (14) yields the stress-strain relationship given as follows:

$$\sigma = \begin{cases} \sigma_{c,r} \frac{nx}{n - 1 + x^n} & x \leq 1 \\ \sigma_{c,r} \frac{x}{\alpha_c (x - 1)^2 + x} & x > 1 \end{cases} \quad (20)$$

The evolution of parameters $\sigma_{c,r}$, $\varepsilon_{c,r}$, and E_c with freeze-thaw damage can be expressed by the following equations. Accordingly, once D_f is known, the complete stress-strain curve can be obtained using Eq. (20).

$$\sigma_{c,r} = (1 - R_{ps})\sigma_{c,r0} \quad (21)$$

$$\varepsilon_{c,r} = (1 + I_{ps})\varepsilon_{c,r0} \quad (22)$$

$$E_c = (1 - R_{pe})E_{c0} \quad (23)$$

2.2.4 Cross validation

For each concrete type, compressive stress-strain curves were obtained at nine freeze-thaw cycle levels. The stress-strain curves were split at the cycle level, as summarized in Table 3.

Table 3 Definition of training and validation groups for cross validation

Group	N	Purpose
Training set	0, 100, 200, 300, 400	Parameter calibration
Validation set	50, 150, 250, 350	Independent cross validation

In the calibration stage, only the training-set data were used to fit the functional relationships between the damage variable D_f and the key evolution functions $R_{ps}(D_f)$, $I_{ps}(D_f)$, $R_{pe}(D_f)$, and $\alpha_c(D_f)$. The analytical equations for the key parameters are shown as follows:

$$R_{ps}(D_f) = a_s D_f \quad (24)$$

$$I_{ps}(D_f) = a_i D_f \quad (25)$$

$$R_{pe}(D_f) = b_1 D_f + b_2 D_f^2 \quad (26)$$

$$\alpha_c(D_f) = \exp(d_0 + d_1 D_f + d_2 D_f^2) \quad (27)$$

The validation-set curves were not used in any stage of the fitting. The key parameters obtained from the analysis of the training set group are summarized in Table 4.

Table 4 Coefficients of the D_f -dependent evolution functions fitted using the training set

Type	a_s	a_i	b_1	b_2	d_0	d_1	d_2
DA-concrete	3.170 3	1.014 2	-2.398 2	118.917 9	3.289 3	-71.618 1	975.295 7
DB-concrete	2.301 3	1.751 9	2.652 5	5.622 0	2.082 3	26.744 4	-313.761 6
DC-concrete	3.187 4	1.251 1	2.839 8	17.393 8	2.999 6	-42.572 8	764.068 1

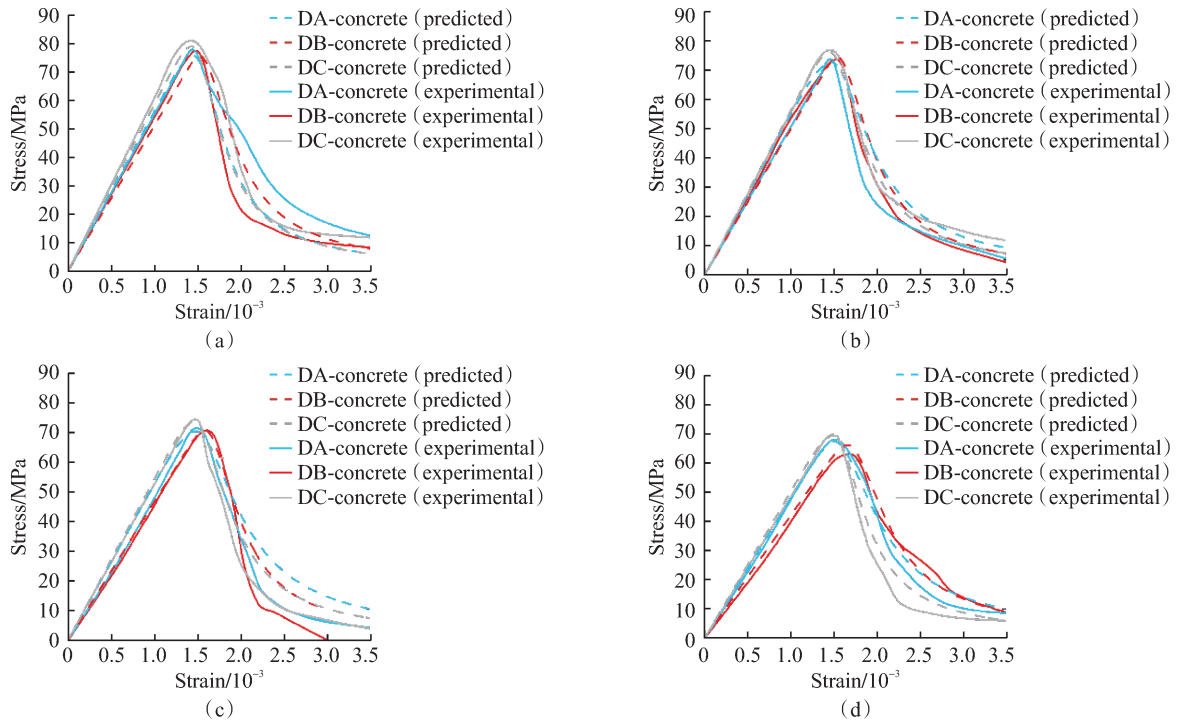
The stress-strain curves of the validation set were predicted, and the cross-validation accuracy was assessed using the point-wise Pearson correlation coefficient (r), R^2 , RMSE, and MAE. The cross-validation results for different cycles are shown in Table 5. Fig. 13 presents the experimental stress-strain curves and the predicted curves obtained from the model. As shown in Table 5

Table 5 Cross-validation metrics of D_f -dependent evolution functions fitted using the training set

Type	N	r	R^2	RMSE/ MPa	MAE/ MPa
DA-concrete	50	0.976	0.916	6.251	4.373
	150	0.973	0.873	5.635	4.404
	250	0.988	0.904	6.914	5.979
	350	0.996	0.990	1.967	1.358
DB-concrete	50	0.969	0.930	5.649	3.259
	150	0.988	0.968	4.112	3.331
	250	0.981	0.916	6.785	4.588
	350	0.996	0.988	2.106	1.788
DC-concrete	50	0.992	0.956	4.846	3.882
	150	0.996	0.966	3.758	3.247
	250	0.994	0.962	4.503	3.671
	350	0.992	0.976	3.182	2.289

and Fig. 13, the predicted curves align well with the experimental data in both the pre-peak and post-peak stages. For the validation set, the minimum correlation coefficients for DA-, DB-, and DC-concrete were 0.973, 0.969, and 0.992, respectively. These results indicate that the constitutive model maintains strong predictive ability even under previously untested freeze-thaw conditions.

The above parameter calibration and cross validation were carried out based on a single mix proportion and a specific material system. In this study, the constitutive model was trained using the stress-strain data obtained from tests on slab concrete from a railway track produced with the binder and admixture system described in this paper, together with three dolomite coarse aggregates exhibiting distinct pore characteristics. Therefore, the validation of the proposed constitutive model is currently constrained to within the strength range, material composition, and freeze-thaw testing conditions considered in this study. To extend the model to concretes of other strength grades, coarse aggregates of different lithologies such as granite and limestone, or alternative admixture and supplementary cementitious material systems, additional experimental data are required to examine the gen-

**Fig. 13** Comparison of experimental and predicted stress-strain curves at different freeze-thaw cycle levels. (a) $N=50$; (b) $N=150$; (c) $N=250$; (d) $N=350$

erality of the model parameters and, where necessary, to revise the parameter evolution relationships that depend on the damage factor D_f . In future research, systematic validation across multiple material systems will be conducted to further delineate the applicability boundary and enhance the generality of the proposed model.

2.3 Analysis of freeze-thaw damage mechanisms

2.3.1 Evolution of ITZ scratch depth and fracture toughness

The characteristics of the coarse aggregate primarily influence the stress-strain behavior of concrete by affecting the properties of the ITZ, as reported in the Ref. [16]. In this study, nano-scratch tests were performed to evaluate the performance of the ITZ in the three concrete mixes. Because the coarse aggregates are more homogeneous than the mortar matrix, the aggregate regions typically exhibit higher fracture toughness during nano-scratch tests. Fig. 14 presents the average scratch depth and fracture toughness of the ITZ with different freeze-thaw cycles. The scratch depth increased markedly, whereas the fracture toughness decreased continuously as the number of freeze-thaw cycles increased.

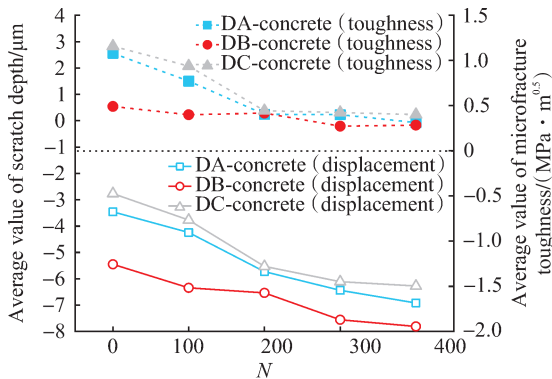


Fig. 14 Scratch depth and fracture toughness of ITZ with different freeze-thaw cycles

Before freeze-thaw cycling, the fracture toughness of the ITZ was 1.079, 0.491, and 1.157 $\text{MPa} \cdot \text{m}^{0.5}$ for DA-, DB-, and DC-concrete, respectively, with corresponding scratch depths of 3 459.9, 5 449.6, and 2 765.6 nm. After 400 freeze-thaw cycles, the fracture toughness decreased to 0.313, 0.284, and 0.401 $\text{MPa} \cdot \text{m}^{0.5}$, whereas the scratch depth increased to 6 918.7, 7 807.7, and 6 272.0 nm, respectively. DB-concrete consistently exhibited the lowest fracture toughness and highest scratch depth, both before and after cycling. Before freeze-thaw cycling, the fracture toughness of the ITZ in DB-concrete was only 45.5% of that of DA-concrete and 42.4% of that of DC-concrete, while its scratch depth reached 157.5% of that of DA-concrete and 197.1% of that of DC-concrete.

This phenomenon stems from the high porosity of Type B coarse aggregates, which increases their water absorption capacity. Consequently, a water-enriched film is formed at the aggregate-matrix ITZ. Ultimately, Type B aggregates exhibit the lowest ITZ fracture toughness and the highest scratch depth values. In contrast, both Type A and Type C aggregates exhibit lower porosity, resulting in a denser ITZ and enhanced fracture toughness. Despite the similar porosity of Types A and C aggregates, the dolomitic composition of Type C aggregates further improves the performance of the ITZ via crack deflection and bridging, thus contributing to enhanced frost resistance of the concrete prepared with Type C aggregates.

In summary, to enhance the frost resistance of concrete in cold regions, priority should be given to the use of low-porosity dolomite coarse aggregates with strictly controlled water absorption. At comparable porosity levels, fine-grained dolomite further improves the frost resistance of concrete.

2.3.2 X-CT analysis of pore structure and crack evolution

To elucidate the influence of the type of coarse aggregate on the microstructural evolution of concrete under freeze-thaw action, X-CT scanning and three-dimensional reconstruction were performed on cored specimens subjected to different freeze-thaw cycles. This enabled quantitative characterization of the pore-structure development and three-dimensional crack propagation.

As shown in Fig. 15(a), the porosity of DA- and DB-concrete increased as the number of freeze-thaw cycles increased and was well fitted by an exponential trend ($R^2 = 0.99$). In contrast, the porosity of DC-concrete exhibited an approximately linear increase ($R^2 = 0.97$). The initial porosity was 0.55%, 0.57%, and 0.43% for DA-, DB-, and DC-concrete, respectively. After 400 cycles, the porosity reached 2.04%, 2.26%, and 1.05%, respectively. Relative to the initial values, these values correspond to porosity increases of 2.74-, 2.97-, and 1.47-fold, respectively.

To quantify the relationship between the porosity growth rate P and the freeze-thaw damage variable D_f , regression analysis was performed using $n = 15$ datasets (Fig. 15(b)). The results indicate a positive linear relationship between P and D_f , as expressed in Eq. (28), with a statistically significant slope ($p = 1.40 \times 10^{-7}$). A significant positive correlation was also confirmed by the Pearson analysis, yielding $r = 0.873$ ($p = 2.12 \times 10^{-5}$). The model showed a satisfactory goodness-of-fit, with $R^2 = 0.870$. The prediction errors were relatively small (RMSE is 0.244 and MAE is 0.160), and LOOCV-

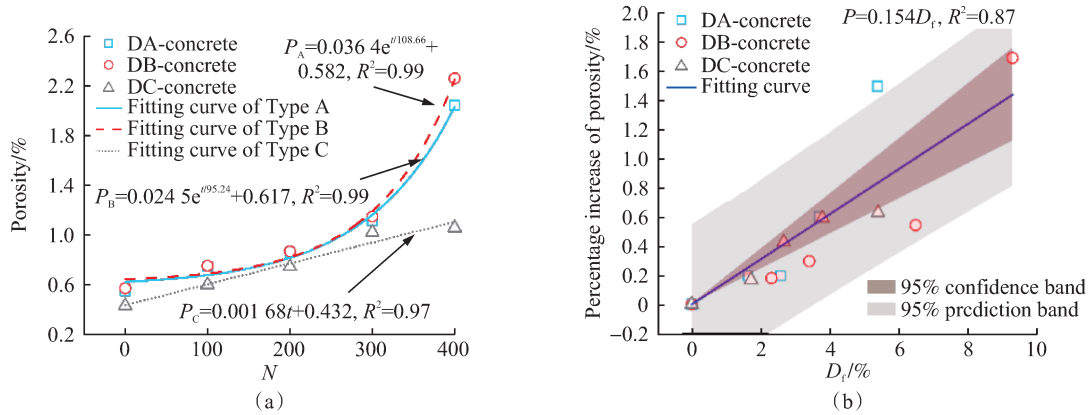


Fig. 15 Analysis of concrete porosity with different freeze-thaw cycles. (a) Porosity variation curves; (b) Porosity growth rate versus freeze-thaw damage variable

RMSE (0.285) and LOOCV-MAE (0.184) remained within a reasonable range and were consistent with the training errors. Although the sample size was relatively limited ($n = 15$), the strong statistical significance ($p < 0.001$), in conjunction with the consistent cross-validation results, supports the reliability of the above conclusion that porosity growth is linearly correlated with the damage level.

$$P = 0.154D_f \quad (28)$$

Based on the pore volume distribution, the pores were classified into four categories^[35], 0-0.001 mm³, 0.001-

0.01 mm³, 0.01-0.1 mm³, and >0.1 mm³, named Categories I-IV, respectively. Fig. 16 reveals that after 400 freeze-thaw cycles, only the Category IV porosity increased significantly in the concrete. Initially, the Category IV porosity was 0.23%, 0.18%, and 0.37% for DA-, DB-, and DC-concrete, respectively. After 400 cycles, these values increased to 1.92%, 1.77%, and 0.74%, corresponding to increases of 7.6-, 10.9-, and 2.0-fold, respectively. The larger increase in the Category IV porosity in DA- and DB-concrete compared to that in DC-concrete largely explains the faster loss of the peak stress under freeze-thaw cycling for the former two.

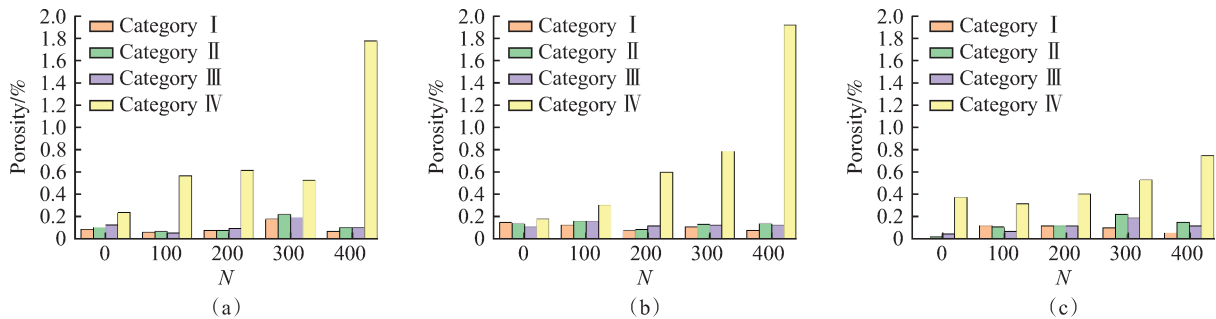


Fig. 16 Variation in porosity corresponding to different pore volumes. (a) DA-concrete; (b) DB-concrete; (c) DC-concrete

Fig. 17 presents the evolution of the crack volume with the increase of freeze-thaw cycles. The crack volume increased approximately linearly in DA- and DC-concrete, whereas that of DB-concrete exhibited an abrupt increase after 200 cycles, consistent with the sharp rise in the peak strain. This transition marks a shift from continuous to discontinuous behavior of the medium, reducing the stiffness and necessitating greater deformation to compensate for the loss of the load-bearing area. Energy dissipation through crack propagation delays stress concentration, and interaction among the crack-tip plastic zones enhances nonlinear deformation, macroscopically manifesting as strain softening, where the peak strain shifts toward higher strain levels.

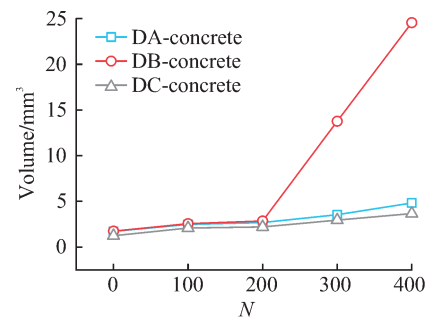


Fig. 17 Volume of cracks in concrete

The crack volume generally increased with cycling, with local fluctuations that can be attributed to the coalescence and interconnection of smaller cracks to form larger ones. The initial crack volumes were 1.72, 1.74,

and 1.24 mm^3 for DA-, DB-, and DC-concrete, respectively. After 200 cycles, the crack volume increased to 2.68, 2.86, and 2.20 mm^3 , respectively. After 400 cycles, the values reached 4.82, 24.56, and 3.66 mm^3 , representing increases of 1.80-, 13.14-, and 1.96-fold, respectively.

Fig. 18 presents three-dimensional reconstructions of the internal cracks after 200 and 400 cycles, demonstrating that the connectivity of the cracks increased with the increase of freeze-thaw cycles. DB-concrete developed connected macrocracks as early as 200 cycles, whereas the cracks in DA- and DC-concrete remained stratified and less connected. After 400 cycles, DB-concrete exhibited a densely interconnected crack network, whereas more dispersed microcracking was retained in DA- and DC-concrete. This provides three-dimensional evidence that the abrupt increase in the peak strain is associated with rapid expansion of the crack volume and crack interconnection.

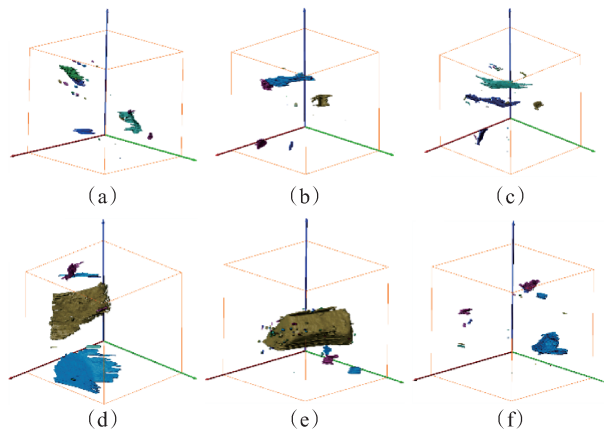


Fig. 18 Illustration of 3D reconstruction of fissure with different freeze-thaw cycles. (a) DA-concrete, $N=200$; (b) DB-concrete, $N=200$; (c) DC-concrete, $N=200$; (d) DA-concrete, $N=400$; (e) DB-concrete, $N=400$; (f) DC-concrete, $N=400$

3 Conclusions

This study elucidates the mechanism by which the pore structure of dolomite coarse aggregates affects the performance of concrete during freeze-thaw cycles. The main conclusions are as follows:

(1) The pore-structure characteristics of dolomite coarse aggregates fundamentally determine the frost resistance of concrete. Type B aggregates, with a porosity of 9.75% and a capillary-pore-dominated system, induced the most severe deterioration in concrete; after 400 freeze-thaw cycles, the peak stress decreased by 22.0% and the elastic modulus declined by 32.9%. In contrast, concrete prepared with Type C aggregates, characterized by a porosity of 1.48% and a dense microstructure, ex-

hibited excellent frost resistance, with only an 18.6% loss in the peak stress. Although Type A aggregates had a relatively low initial porosity of 1.34%, localized microstructural inhomogeneity led to a moderate degree of freeze-thaw damage in the corresponding concrete.

(2) A damage-dependent constitutive model was established, which accurately captures the nonlinear stress-strain behavior of concrete with progressive freeze-thaw damage. This model provides a valuable theoretical tool for predicting the mechanical performance of frost-affected concrete and offers practical criteria for selecting frost-resistant aggregates. Under the conditions of this study, dolomite aggregates with a porosity below 1.5% and a dense microstructure performed better. Therefore, a comprehensive evaluation combining porosity with pore-structure parameters such as the pore-size distribution and connectivity is recommended for aggregate selection.

(3) DB-concrete exhibited the most pronounced ITZ weakening, with a 42.2% reduction in the fracture toughness and a 43.3% increase in the scratch depth; correspondingly, an extensively interconnected crack network developed after 400 cycles, where the crack volume increased 13.14-fold. In comparison, the ITZ in DC-concrete maintained significantly better mechanical properties, and the concrete showed only limited and isolated microcracking, where the crack volume increased 1.96-fold. DA-concrete presented an intermediate level of ITZ damage and moderate crack propagation, characterized by layered microcracking without complete interconnectivity. These results provide consistent micro- and meso-scale evidence supporting the conclusion that the pore structure of the aggregate governs the propagation of internal damage in concrete under freeze-thaw action.

(4) From an engineering perspective, when the availability of high-quality aggregates is limited and cost constraints necessitate the use of higher-porosity aggregates, the adverse effects of these aggregates can be mitigated through auxiliary measures, such as sealing capillary pores using a polymer (or similar material); impregnation to reduce water absorption and strengthen the surface of the aggregate; optimizing air-entraining strategies to obtain a favorable air-void system, thereby improving the freeze-thaw durability; and adjusting the mix-design parameters to densify the matrix and enhance the frost resistance. Note that the conclusions of this study are primarily derived from dolomite aggregates within the porosity range investigated in this work. Future studies should extend the analysis to higher porosity ranges and comparative investigations across multiple aggregate lithologies to systematically evaluate the generality and ap-

plicability boundaries of the proposed mechanism and model.

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白云岩粗骨料孔隙结构对混凝土抗冻性的作用机制研究

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摘要: 冻融损伤是寒区混凝土耐久性的主要威胁。本研究突破传统上对砂浆基体与界面过渡区的关注, 重点探究白云岩粗骨料的孔隙结构对混凝土抗冻性能的作用机制。通过综合应用压汞、纳米划痕、X-CT等多尺度试验方法, 结合400次快速冻融循环试验, 对3种具有不同孔隙特征骨料(孔隙率1.34%的泥微晶白云岩骨料、孔隙率9.75%的微晶白云岩骨料和孔隙率1.48%的不等晶白云岩骨料)进行了研究。结果表明, 骨料孔隙结构通过对界面过渡区性能退化与内部裂纹扩展的耦合作用, 显著调控混凝土的冻融劣化进程。经历冻融循环后, 与采用1.48%致密低孔隙率骨料相比, 采用9.75%高孔隙率、以毛细孔为主的骨料制备的混凝土表现出明显的性能衰退, 其峰值应力和弹性模量分别下降了22.0%和32.9%, 界面过渡区断裂韧性降低了42.2%, 并伴随相互连通裂纹网络的加速发展。而采用1.48%致密低孔隙率骨料制备的混凝土表现出更优的抗冻性, 峰值应力仅损失18.6%。进一步建立了一个考虑损伤的非线性本构模型, 能够再现冻融损伤混凝土的应力-应变响应。本研究旨在为缺乏优质骨料地区的抗冻混凝土设计提供理论依据, 并指出在骨料选用标准中需综合考量孔隙率与孔隙结构特征参数。

关键词: 冻融循环; 白云岩粗骨料; 孔隙结构; 混凝土; 本构模型; 影响机制

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