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**MULTI-SCALE COLLABORATIVE
PERFORMANCE RESEARCH OF ULTRA
HIGH PERFORMANCE CONCRETE:
FROM MATERIALS TO STRUCTURES**

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PhD

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**Multi-scale Collaborative Performance
Research of Ultra High Performance Concrete:
From Materials to Structures**

YU Jiangjiang

**A thesis submitted in partial fulfilment of the
requirements for the degree of Doctor of
Philosophy**

May 2025

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YU Jiangjiang

ABSTRACT

Ultra-high performance concrete (UHPC) is an advanced composite material with multi-scale characteristics and complex microstructure, which has shown great potential for application in fields such as civil engineering, water conservancy engineering, and construction engineering. With the development of structural design and construction technology towards refinement, multi-scale modeling of concrete materials and structures, and integrated material structure analysis have become hot research areas. The multi-scale analysis method developed based on homogenization theory establishes the relationship from microscopic materials to macroscopic components by introducing representative volume elements (RVEs) containing microstructural information. This method can not only accurately predict the equivalent thermodynamic properties of materials, but also reveal their performance evolution laws, providing theoretical guidance for material design and proportioning optimization. This study focuses on the multi-scale mechanical properties of UHPC materials and structures, and systematically conducts computational analysis and research in the following four aspects: (1) multi-scale analysis of UHPC material properties; (2) Multi scale study on fiber orientation and toughening mechanism; (3) Multi scale characterization of interfacial bonding performance between ordinary concrete and UHPC; (4) Multi scale evaluation of bending performance of reinforced concrete UHPC composite beams. The specific research content and main conclusions are as follows:

(1) A micro-scale numerical finite model including the coarse aggregates, cement paste, interface zone and steel fibers was constructed using the random modeling method. The concrete damage plasticity (CDP) model and cohesion model were integrated to reveal the damage evolution characteristics of each phase component during the compression process. The numerical simulation results show that the damage initiation in the interface transition zone (ITZ) is earlier than that in the mortar and coarse aggregate phases, and its damage development pattern has significant differences; The stress state of fibers exhibits a compressive tensile transition characteristic, and their toughening efficiency is strongly correlated with the spatial distribution angle.

(2) Based on the Mori Tanaka homogenization theory, the influence of fiber volume fraction and coarse aggregate content on the elastic mechanical properties of UHPC was systematically studied

through the experiments and multi-scale numerical simulations. Further combined with the continuous progressive damage model, the damage evolution parameters of UHPC under uniaxial tensile, compressive, and four point bending loads were calibrated, and a quantitative correlation model between microscopic component parameters and macroscopic mechanical response was established.

(3) The orientation distribution characteristics of fibers near the fracture surface were quantitatively characterized based on image processing technology. The orientation distribution function (ODF) was reconstructed using the mixed closure approximation method, and the reliability of the method was verified through experimental data. A multi-scale strength prediction model considering fiber orientation tensor was established by combining the two-step mean field homogenization method and Puck failure criterion, the collaborative effect mechanism of fiber content and spatial distribution on the tensile properties of UHPC was revealed.

(4) A multi-scale interface analysis method based on the equal roughness transformation is proposed. By constructing RVE models with different roughness characteristics, the equivalent traction-separation constitutive relationship of the multi-scale interface is obtained. This research reveals the failure mechanism of interfaces under tensile and shear loads, quantitatively evaluated the sensitivity of roughness parameters to interface strength, and provided theoretical support for the design of heterogeneous concrete interfaces.

(5) A multi-scale interface modeling technique has been developed, which maps microscopic geometric features to equivalent mechanical parameters of macroscopic interfaces through the homogenization method. The three-point bending test indicates that the calculation model can accurately predict the structure failure mode, load-deflection curve, and interface slip behavior of composite beams with different interface roughness. The research results show that increasing interface roughness can significantly enhance the overall collaborative performance of composite beams and effectively suppress the development of interface cracking.

Key Words: Ultra high performance concrete, multi-scale analysis, homogenization, material and structure, performance prediction

Publications arising from the thesis

- [1] **Yu Jiangjiang**, Zhang Boshan, Chen Weizhen, Liu Hang. Experimental and multi-scale numerical investigation of ultra-high performance fiber reinforced concrete (UHPFRC) with different coarse aggregate content and fiber volume fraction[J]. Construction and Building Materials, 2020, 260: 120444.
- [2] **Yu Jiangjiang**, Zhang Boshan, Chen Weizhen, Liu Hang. Multi-scale analysis on the tensile properties of UHPC considering fiber orientation[J]. Composite Structures, 2022, 280: 114835.
- [3] **Yu Jiangjiang**, Zhang Boshan, Chen Weizhen, Liu Hang, Li Heng. Multi-scale study on interfacial bond failure between normal concrete (NC) and ultra-high performance concrete (UHPC)[J]. Journal of Building Engineering, 2022, 57: 104808.
- [4] Zhang Boshan, **Yu Jiangjiang***, Chen Weizhen, Liu Hang. Failure evolution and fiber toughening mechanism of ultra-high performance concrete under uniaxial compression[J]. Journal of Sustainable Cement-Based Materials, 2022: 1-19.
- [5] Zhang Boshan, **Yu Jiangjiang**, Chen Weizhen, Liu Hang, Li Heng. Multi-scale investigation on the interfacial behavior of between NC and UHPC in flexural members considering influence of the interface morphology[J]. Composite Structures, 2024, 345: 118388.

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Chapter 1 Introduction

1.1 Background and Significance

In the 1990s, the reactive powder concrete (RPC) developed by Richard ^[1] was recognized as the most significant milestone in the evolution of Ultra-high performance concrete (UHPC). Through high-pressure compaction and curing under elevated temperature-pressure conditions, RPC components achieved compressive strengths exceeding 800 MPa ^[1], establishing the highest recorded strength for cementitious materials at that time. In 1994, French researchers De Larrard and Sedran formally designated this advanced cement-based composite as ‘Ultra-high performance concrete’ (UHPC) in their seminal work, a nomenclature subsequently adopted by the international scientific community. The French Civil Engineering Association later formulated interim design guidelines for UHPC materials, laying the technical foundation for its engineering applications. As an innovative addition to the concrete family, UHPC has revolutionized structural design and rehabilitation strategies in civil engineering (shown as Figure 1.1). Current statistics indicate global deployment of UHPC in over 400 bridges as primary or partial structural material, with more than 150 bridges incorporating UHPC as their main structural system.



(a) Bridge deck pavement.



(b) Bridge pier components.



(c) Bridge pier reinforcement.



(d) Main beam structure.

Figure 1.1 Application of UHPC in Bridge Structures.

UHPC is an innovative cementitious composite material renowned for its exceptional mechanical properties, fracture toughness, and superior durability. It is regarded as one of the most promising construction materials for future sustainable infrastructure ^[2]. Distinguished from conventional concrete and high-performance concrete (HPC), UHPC features an ultra-low water-to-binder ratio

(w/b = 0.15–0.25) and is typically composed of optimally graded ultrafine powder particles, steel fibers, and tailored chemical admixtures. These components collectively enable outstanding fresh-state properties (e.g., high flowability with minimum slump flow ≥ 160 mm) and hardened-state characteristics, including remarkable mechanical performance (28-day compressive strength ≥ 120 MPa^[3], tensile strength ≥ 5 MPa^[4], and elastic modulus ranging from 40 GPa to 60 GPa). Furthermore, the dense matrix of UHPC effectively inhibits ingress of corrosive agents such as carbon dioxide, sulfates, and chlorides, demonstrating exceptional durability^[4, 5]. The unique combination of ultra-high strength, ductility, and longevity enables the application of UHPC in diverse structural systems, including prefabricated elements, cantilever structures, bridge overlays, joint systems, and structural rehabilitation/strengthening projects. However, its extremely low water-to-cement ratio induces elevated shrinkage deformation during setting and hardening period^[6], which increase cracking susceptibility. While steel fiber reinforcement significantly enhances tensile performance, it concurrently escalates material costs^[7], posing challenges for large-scale adoption. To mitigate cost barriers and facilitate broader implementation, researchers have explored innovative solutions. Notably, the strategic incorporation of coarse aggregates into UHPC mortar has ushered in a new developmental stage. Extensive studies have investigated coarse aggregate utilization, alternative binders, and optimized fine aggregate compositions to enhance the elastic modulus and develop sustainable mix designs^[8-10].

The concrete, including UHPC as a kind of cement-based composites, is a typical composite material exhibiting multi-scale characteristics and complex microstructures. For the material level analysis, researchers typically consider the performance differences and interaction mechanisms between microscopic phase components, while for the macroscopic concrete structural design and analysis, the material is generally homogenized by ignoring its internal microstructural information. Although this analytical approach enhances computational efficiency in practical engineering design, the intrinsic multi-scale correlations between microstructural damage and failure mechanisms and the performance degradation of macro-components or structures cannot be clearly revealed. The evolution of micro-damage in UHPC materials or structures is not only influenced by macroscopic loading but also significantly affects by the macroscopic performance. However, the specific manner of microstructure and phase properties determine its macroscopic behavior remains ambiguous, and no unified multi-scale analysis and computational framework has been established

for UHPC performance characterization. Considering the above issues, this study investigates and reveals the intrinsic relationships of UHPC materials across different scales—from micro to macro, local to global, and material to structural levels. Specifically, a series of multi-scale computational and analytical methods is developed focusing on the multi-scale performance of UHPC materials, multi-scale fiber orientation and toughening mechanism, multi-scale interface bonding, and multi-scale bending performance of composite structures. These analytical approaches also provide valuable references and inspiration for research on other cement-based composites.

1.2 Key issues to be addressed

From the analysis of the research status, it can be seen that the degradation of the macroscopic performance of UHPC materials or structures will be affected by their microstructural characteristics, and the macroscopic failure under environmental and load is also the result of the accumulation of damage at the microscopic scale. Therefore, in order to reveal the strength degradation of UHPC repair materials or structures and the cracking and failure mechanism during long-term service, it is necessary to carry out research from a smaller scale and establish a multi-scale relationship between the performance of microscopic multiphase composite structures and the macroscopic homogenization performance of concrete. Therefore, this paper will study the multi-scale calculation and analysis methods of UHPC material properties, multi-scale fiber orientation and toughening mechanism, multi-scale interfacial bonding properties, and multi-scale flexural properties of RC-UHPC composite structures, and the key problems to be solved in this paper are as follows:

(1) A multi-scale analysis model of UHPC materials was established by using a multi-scale homogenization method coupled with a progressive damage model, and the macroscopic response of mesoscopic components to stress states such as compression, tensile and bending of UHPC was calculated and predicted.

(2) The mesoscopic stochastic method was used to establish a fine numerical model of UHPC to reveal the fiber toughening mechanism of UHPC under compression and the damage evolution process of each mesoscopic phase.

(3) The orientation tensor description method was used to analyze the orientation characteristics of fibers in UHPC, establish the multi-scale relationship between fiber orientation characteristics and tensile properties, and predict the tensile strength of UHPC under different fiber orientation

parameters.

(4) For NC-UHPC composite components with different roughness characteristics and substrate strength, a calculation method for RVE homogenization in the interface region with random roughness characteristics is proposed to reveal the degradation mechanism of NC-UHPC interface under tensile and shear at the material scale.

(5) The multi-scale analysis and calculation method is raised to the structural level, and a multi-scale analysis method for the flexural performance of reinforced concrete-ultra-high-performance concrete composite structures is proposed, revealing the influence of interface mesoscopic characteristics on the macroscopic bending response of composite structures.

1.3 Main research contents

The main research content is divided into 8 chapters, and the specific research content of each chapter is as follows:

Chapter 1 is the introduction, which mainly expounds the research background, significance, key issues, main research contents and research technical routes of multi-scale research on ultra-high performance concrete.

In Chapter 2, the state-of-the-art literature review on the mechanical properties of UHPC is separately presented. The first part of this chapter discusses the static mechanical properties of UHPC, including the performance of the elastic properties, the compressive performance, the tensile behavior, and the flexural behavior. The second part presents a review of the fiber toughening mechanisms in UHPC, and the interfacial adhesion performance, fiber orientation, and toughening performance were investigated. The third part is a review of the interfacial bonding performance between the normal concrete and the UHPC, while the influencing factors and numerical modeling methods of the bonding interface have been discussed. The last part is a review for the multi scale calculation method for the mechanical behavior of the fiber reinforced concrete and composites, and the modeling calculation and analysis methods are discussed.

Chapter 3 is an experimental study, firstly, the raw materials of ultra-high performance concrete are collected, four mix ratios are designed based on the stacking model based on the ideal gradation curve theory, and then the properties of the mixture, mechanical properties and microstructure of hardened ultra-high performance concrete are experimentally studied.

In Chapter 4, the mesophase damage evolution process of ultra-high performance concrete

compression process based on the mesoscopic stochastic method is developed, and the difference of compression performance under different fiber and coarse aggregate contents is analyzed, while the toughening and crack resistance mechanism of fiber on ultra-high performance concrete during compression is revealed.

Chapter 5 is a review of the mesoscale analysis methods and failure criteria of composite materials, and a multi-scale calculation and analysis method for the mechanical properties of ultra-high performance concrete based on the progressive damage model and multi-scale homogenization theory is proposed. The compressive, tensile and flexural properties of hardened ultra-high performance concrete were verified and predicted.

Based on the fiber orientation distribution characteristics of the tensile specimen section, the relationship between different fiber orientation tensors and multi-scale tensile properties is established by using the method of describing the orientation tensor and the two-step homogenization method in chapter 6. The elastic properties under different fiber orientation parameters and fiber content were calculated, and the corresponding tensile strength was predicted by coupling the progressive failure model.

In Chapter 7, combined with the NC-UHPC core extraction and double shear tests, the rough characteristics of the interface are first analyzed, and a multi-scale analysis method of the debonding behavior of the NC-UHPC interface is proposed for the case with different rough interfaces. The failure of the ordinary concrete matrix is described by the elastoplastic damage model coupled with gradient enhanced damage evolution, the interface part is represented by the cohesive zone model (CZM), and the UHPC is characterized by the second-order mean field homogenization method. The RVE with different roughness is established, and the effective traction-separation law is obtained, and on this basis, the tensile and shear separation mechanism of the NC-UHPC interface is revealed.

In Chapter 8, the multi-scale calculation method is elevated to the structural level, and the complex interface geometry on the mesoscale is described by a homogeneous framework, and the macroscopic flat interface structure is given after an effective mechanical response is obtained. Through the experimental and comparative studies, the three-point bending failure mode, load-deflection and load-interface slip curves of RC-UHPC composite beams under different interface roughness are verified and predicted. The influence of interfacial mesoscopic characteristics on the

overall response of RC-UHPC composite beams was evaluated.

Finally, the summarizes of the full thesis is listed in chapter 9, and looks forward to the multi-scale computational analysis methods and further research work of UHPC materials-structures.

Chapter 2 Literature Review

2.1 Mechanical Performance of UHPC

2.1.1 Elastic Performance

UHPC has emerged as a research hotspot in civil engineering materials due to its superior mechanical properties and durability (shown as Figure 2.1). Generally, UHPC demonstrates compressive strengths exceeding 120 MPa with elastic modulus values ranging between 40–60 GPa [11]. As a fundamental mechanical parameter of concrete, elastic modulus directly reflects the stiffness characteristics. Enhancing the elastic modulus of UHPC can effectively increase the structural component/system stiffness, thereby reducing the deformation under external loads. Meanwhile, high-modulus UHPC facilitates large deformation recovery, positively influencing structural seismic performance. For UHPC with lower elastic modulus, matrix cracking may occur due to excessive deformation. When crack propagation exceeds critical thresholds, it not only compromises durability but severely impairs structural safety and stability [12]. Current data indicate that the compressive strength of UHPC surpasses conventional concrete by approximately 200%–300%, while its elastic modulus improvement about 50% [3]. The increase in elastic modulus of UHPC is much lower than the increase in compressive strength, which may lead to insufficient stiffness of UHPC [13]. Therefore, increasing the elastic modulus of UHPC materials is of great significance for enhancing the overall bearing capacity of UHPC structures and reducing the risk of structural damage and cracking.

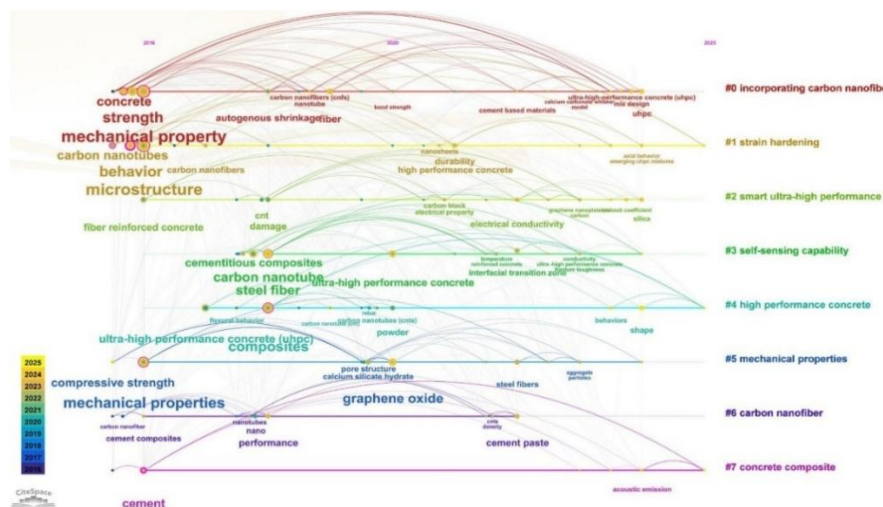


Figure 2.1 Research papers on the related topic of concrete [14].

The elastic modulus of cementitious materials including concrete and UHPC is influenced by

multiple factors, such as the composition or strength of cement paste, aggregate characteristics, interface transition zone (ITZ) properties, and material compactness [15, 16]. Compared to conventional concrete, UHPC incorporates modified formulations through the addition of supplementary cementitious materials (e.g., quartz powder or ultrafine silica fume), fibers, and specialized admixtures. Current research efforts have extensively investigated the effects of various parameters on the elastic modulus, including supplementary cementitious materials, nanomaterials, fiber reinforcement, water-to-binder ratios, and coarse aggregate incorporation.

As for the influence of cementitious materials, Yazici [17] investigated the mechanical properties of reactive powder concrete (RPC) incorporating class C fly ash and ground granulated blast furnace slag. The results demonstrated that steam curing and autoclave treatment significantly enhanced the compressive strength and elastic modulus of RPC. The addition of fly ash increased the modulus of RPC, whereas the incorporation of blast furnace slag reduced its modulus. Amin [18] explored the effectiveness of partially replacing cement with silica fume and metakaolin in UHPC containing ceramic waste as coarse aggregates. The study revealed that UHPC with 20% silica fume and 10% metakaolin achieved an elastic modulus of 51.7 GPa, representing a 14.2% improvement compared to the control group. Zhao [19] used nanoindentation technology to analyze the nanomechanical behavior of UHPC cement paste. The results indicated that hydration products (occupying ~50% of the paste volume) primarily consisted of high-stiffness hydration phases. Notably, unreacted cement and fly ash particles exhibited superior mechanical properties to hydration products, functioning as micro-aggregates to enhance UHPC performance. Soliman [20] developed and evaluated eco-friendly ultra-high-performance glass powder concrete (UHGC) with a compressive strength of 220 MPa. The optimal glass powder replacement ratio for cement was identified as 20%, which accelerated hydration reactions and achieved an elastic modulus of 55 GPa while maintaining optimal compressive strength development.

As the elastic modulus of steel fibers (approximately 200 GPa) significantly exceeds that of hardened UHPC cementitious matrices, numerous studies have investigated the enhancement of UHPC's elastic modulus through steel fiber incorporation. Yang [21] experimentally analyzed the compressive behavior of UHPC with varying steel fiber volume fractions and aspect ratios. The results demonstrated that both compressive strength and elastic modulus increased with higher fiber volume fractions or larger aspect ratios. Ahmad [22] revealed that while the elastic modulus improves

with the increase of the fiber content, this enhancement plateaus when the volume fraction exceeds 3.1%. Hannawi ^[23] comparatively evaluated the effects of different fiber types on the elastic modulus, the results indicates that steel fibers provide more substantial modulus improvement compared to the alternative fiber materials.

The incorporation of coarse aggregates is recognized as an effective strategy to enhance the mechanical performance and reduce the cost of UHPC ^[24]. Wang ^[25] demonstrated that adding coarse aggregates into UHPC not only preserves its superior properties—high strength, ductility, and durability, but further improves elastic modulus while reducing shrinkage and creep deformation. Kim ^[26] involved the maximum packing density theory to develop coarse aggregate-incorporated UHPC, It was found that the compressive strength and elastic modulus of UHPC containing coarse aggregate were improved compared to UHPC without coarse aggregate. The compressive strength and elastic modulus of UHPC containing 620 kg/m³ of coarse aggregate with a maximum particle size of 8 mm reached 185.6 MPa and 57 GPa, respectively. The research results of Sun ^[27] indicate that as the aggregate/cement ratio increases (0.6-1.2), the compressive strength and flexural tensile strength of UHPC show a trend of first increasing and then decreasing, while the elastic modulus shows a linear increase trend.

2.1.2 Compressive performance

Compressive strength is a critical indicator for evaluating the mechanical performance of cementitious materials and a key parameter in structural design of cement-based composites ^[28].

Figure 2.2 shows the publication number of the compressive performance of UHPC.

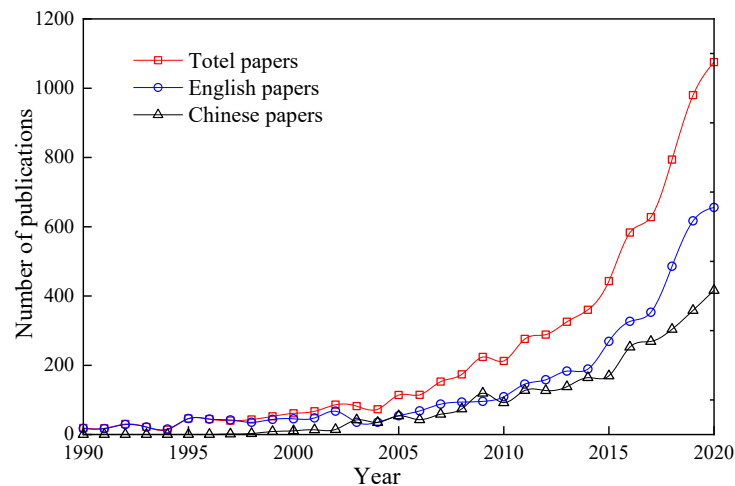


Figure 2.2 Number of publications about compressive performance of UHPC.

The compressive strength of UHPC is governed by multiple factors, including water-to-cement

ratio (Figure 2.3), hydration degree, fiber content/geometry, curing conditions, as well as aggregate characteristics and content. The study of Regalla ^[29] investigated the effects of granite powder substitution for cement on the microstructure and macroscopic mechanical properties of UHPC. Under autoclave curing conditions, the compressive and flexural strengths initially increased then decreased with higher granite powder replacement ratios (0–25%), while decreasing with elevated water-to-binder ratios (0.16–0.20). Yu ^[30] employed a modified Andreasen & Andersen particle packing model to design the densely compacted UHPC. The fly ash, finely ground blast furnace slag, and limestone powder were adopted to replace the cement, but it was found that these substitutes did not have a significant impact on UHPC during the initial pouring stage. However, for the 28 and 91 days curing age, UHPC incorporating GGBFS demonstrated superior compressive performance. Mohammed ^[31] utilized silica fume substitution to refine the microporous structure in UHPC, thereby enhancing microstructural integrity and improving compressive strength. Li ^[32] used ceramic tile powder to replace part of cement, and a new type of green low-carbon UHPC was developed. The results showed that the addition of ceramic tile powder had an impact on the compressive strength of UHPC at different ages within $\pm 10\%$. At a dosage of 25%, the compressive strength growth rates reached 104.6% and 51.8%, respectively.

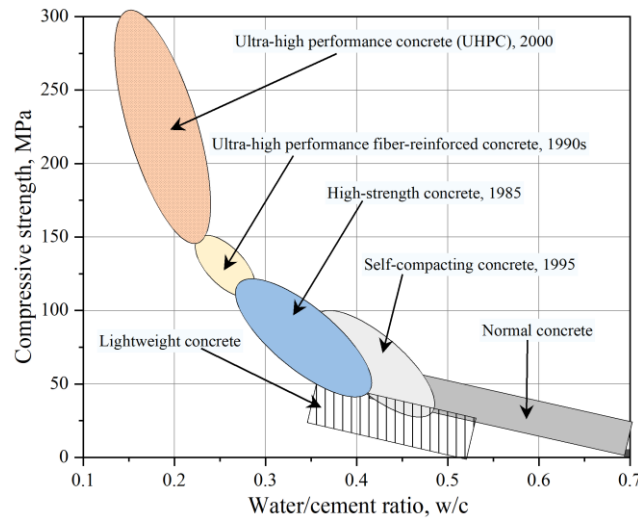


Figure 2.3 The variation trend of compressive strength for different types of concrete.

The influence of fibers on the compressive behavior of UHPC is equally significant. Steel fibers markedly enhance the toughness of UHPC, inducing a distinct strain-hardening phase in the stress-strain curve and substantially improving post-cracking ductile behavior ^[33]. Rezakhani ^[34] conducted a series of compression tests on UHPC with 0%, 2%, and 4% fiber content. The

experimental stress-strain curves showed that steel fiber incorporation increased the strength of UHPC by 12% compared to fiber-free counterparts, while specimens with 2% and 4% fiber volume fractions exhibited comparable compressive strengths. Deng ^[35] investigated the effects of steel-polypropylene hybrid fibers (SF-PF) on the uniaxial compressive stress-strain behavior of UHPC. The results demonstrated that single-component SF significantly enhanced toughness, while higher aspect ratios of PF improved the fiber utilization efficiency, further augmenting material ductility. The synergistic incorporation of SF-PF not only facilitated stronger interfacial bonding within UHPC but also changed the failure modes.

Wille ^[36] investigated the influence of coarse aggregates with particle sizes of 7–16 mm on the compressive strength of UHPC. The results indicated that incorporating an optimal amount of coarse aggregates could enhance concrete strength. Wu ^[37] investigated the effects of coarse aggregate types and contents (expressed as mortar replacement ratios) on the compressive and flexural performance of UHPC. The results revealed that appropriate coarse aggregate addition can significantly improved compressive strength and elastic modulus, while slight reductions in flexural strength and post-peak ductility. Aggregates with varying stiffness levels effectively enhanced either compressive strength or post-peak ductility. Notably, as a more economical material compared to other raw components, Wille ^[38] demonstrated that coarse aggregates could replace expensive quartz sand without compromising UHPC strength. Cetin ^[39] established that maximum compressive strength was achieved when coarse aggregate content ranged between 36% and 40%. In addition, as the content of coarse aggregate increases, the amount of cementitious material in UHPC decreases accordingly, which inhibits the shrinkage of the UHPC matrix hydration process and also reduces the generation of hydration heat ^[40].

2.1.3 Tensile Performance

The ultra-high strength, outstanding toughness performance, and excellent durability of UHPC are all based on the synergistic bearing mechanism between its dense matrix and high-strength fibers, while tensile performance is the most direct and effective indicator reflecting the true synergistic effect between the matrix and fibers in fiber-reinforced concrete ^[41]. The number of publications on the tensile properties of UHPC shows an increasing trend as Figure 2.4. Unlike ordinary fiber reinforced concrete, the dense microstructure of UHPC results in extremely strong bonding and adhesion between fibers and matrix. Studies have shown that the bond strength between UHPC

fibers and matrix is more than 20 times that of ordinary concrete ^[42]. The factors that affect the tensile mechanical behavior of UHPC by matrix properties include the composition and strength of the cementitious material system, particle size and distribution, while the factors that affect the tensile properties of fibers include fiber type, geometric shape, dosage, distribution characteristics, etc. The figure 2.5 and 2.6 is the keywords analysis of tensile performance of UHPC ^[43], which indicates that the research hotspots of tensile properties is the hybrid fiber, the volume friction and the space orientation.

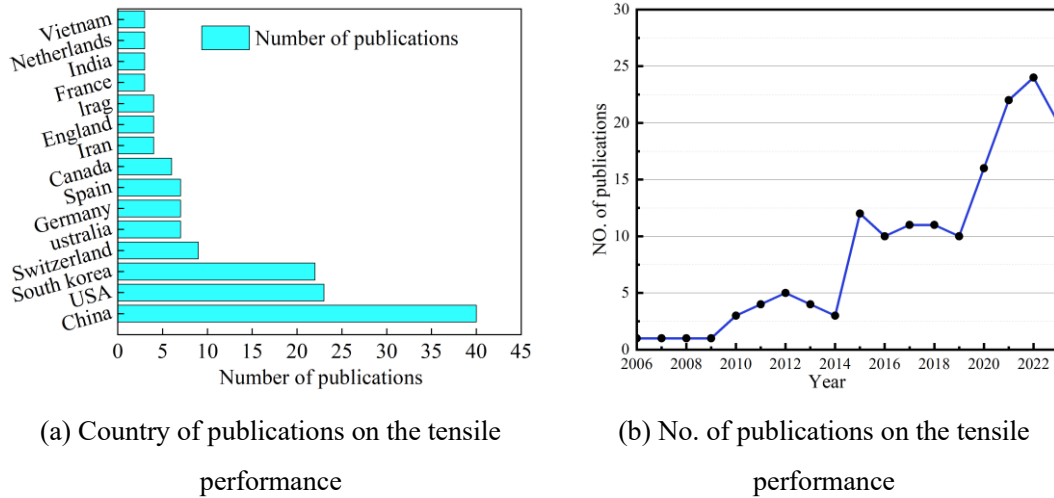


Figure 2.4 Research trends in tensile properties of UHPC.

In terms of the influence of the characteristics of cementitious materials on tensile properties, Wang ^[44] used nanoindentation and scanning electron microscopy (SEM) to investigate the elastic modulus and hardness distribution characteristics of the steel-fiber matrix and fiber-matrix-aggregate interface regions in FRC, taking into account two factors: water cement ratio and silica fume content. It was found that adding silica fume to FRP would enhance ITZ and improve the adhesion of the matrix to the fibers. You ^[45] explored the feasibility of using liquid crystal display glass powder instead of quartz powder to prepare UHPC through the study of volcanic ash activity, bulk density, and direct tensile testing. As a result, it was found that UHPC with a 50% glass powder substitution rate has outstanding tensile strength and tensile properties. Lee ^[46] analyzed the effects of adding carbon powder, nickel powder, graphite powder, and steel slag to UHPC on its tensile behavior through experiments. The results showed that with the increase of carbon powder, nickel powder, and graphite powder content, the tensile strength of UHPC decreased, while with the increase of steel slag content from 5% to 10%, the tensile strength of UHPC increased. Yoo ^[47] studied the pull-out resistance of steel fibers in UHPC matrix containing slag and fly ash, and found

that as the matrix strength increased, the fiber pull-out resistance increased, thereby improving the tensile properties of UHPC.

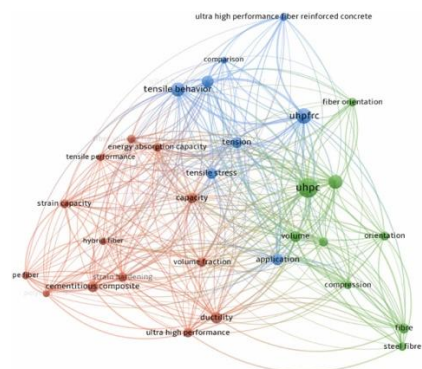


Figure 2.5 Keyword analysis.

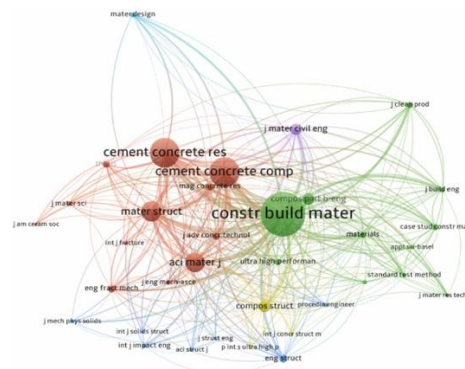


Figure 2.6 Published journal analysis.

In terms of the fiber properties, Shin ^[48] studied the tensile properties of steel fiber UHPC in three types: round, triple twist, and six twist. The experimental results show that due to the larger circumference of the three twisted steel fibers, the anchoring effect is significant. Therefore, twisted steel fibers have better pull-out resistance than round steel fibers, and the tensile performance increases with the degree of twisting. Wille ^[49] found that a fiber volume fraction of 1% is sufficient to trigger strain hardening behavior in UHPC, while a twisted steel fiber UHPC with a content of 1.5% can achieve a tensile strength of 13 MPa and an ultimate tensile strain of 0.6%. Yoo ^[50] extensively compared the tensile behavior of UHPC reinforced with straight steel fibers and deformed steel fibers, and found that the effective order of fiber types for improving tensile performance is: straight steel fibers, twisted steel fibers, half hook steel fibers, and hook end steel fibers. Zhang ^[51] compared the effects of different fiber morphologies on the tensile stress-strain curve of UHPC and found that when the steel fiber content and aspect ratio were the same, the end hooked steel fiber improved the toughness of UHPC better than the straight steel fiber; When the amount and shape of steel fibers are the same, the larger the aspect ratio of steel fibers within a certain range, the higher the tensile deformation capacity and energy absorption capacity per unit volume of UHPC, and the more obvious the strain hardening characteristics of UHPC. Jang ^[52] studied the effect of corroded steel fibers on the pull-out and tensile properties of UHPC, and found that the increase in roughness caused by rust on the surface of smooth steel fibers has a positive impact on improving the pull-out and tensile properties of UHPC. UHPC prepared from steel fibers treated with corrosive electrolyte solution for 3 hours exhibits the best tensile properties. Alani ^[53] studied the tensile behavior of UHPC prepared from polyethylene terephthalate (PET) fibers. The

results indicate that the addition of PET fibers regulates the rigidity and brittleness behavior of UHPC. In the case of 20% silica fume and 1% PET fiber content, the ductility index of UHPC significantly increases, and the splitting tensile strength reaches 9.31 MPa. Zhang ^[54] found that fiber orientation significantly affects the tensile stress-strain relationship of UHPC, fracture mode in mortar, stress evolution in fibers, and interfacial bond slip behavior. The fibers in UHPC will only work when the tensile direction of the fiber and the component are the same, and optimizing the fiber orientation can also enable UHPC to achieve strain hardening transition from brittle failure to ductile failure.

The research on the influence of fiber orientation or spatial arrangement characteristics on the tensile properties of UHPC is also receiving increasing attention. After the occurrence of tensile and bending cracks, the bridging effect is directly affected by the fiber orientation characteristics. For a single fiber embedded in UHPC, the pull-out force is determined by its direction and matrix density ^[55], and many predictive models have been developed to reveal the pull-out load displacement relationship ^[56]. Zhang ^[57] analyzed the fracture load of FRC embedded with uniformly distributed fibers, and the fracture load results were significantly higher than those of FRC with inclined fibers. Lee^[58] found that straight steel fibers embedded in UHPC reach the maximum pull-out load at an inclination angle of 30 ° to 45 °. Cao ^[55] studied the pull-out characteristics of hook fibers in UHPC matrix at different inclination angles, and a model for analyzing the pull-out failure evolution of hook fibers was developed. The macroscopic statistical effects can reflect the influence of fiber orientation on the reinforcement performance of a large number of fibers dispersed in the matrix. Arunothayan ^[59] quantitatively studied the influence of fiber distribution characteristics on the bending behavior by comparing the fracture modulus of 3D printed UHPC samples and randomly distributed fiber samples. The comparison showed that the fracture modulus of the former increased by 39%. Mu ^[60] attempted to control fiber arrangement by changing the magnetic field strength and increased the tensile strength of UHPC by over 100%. Zhou ^[61] found that the three-point bending strength is linearly positively correlated with the number of reinforcing fibers near the fracture surface, while the number of effective fibers is linearly positively correlated with the fiber inclination angle.

2.1.4 Flexural Performance

Similar to the compression and tensile properties, the bending performance is also a fundamental

indicator for characterizing the properties of UHPC materials, mainly used to evaluate the bending toughness and ductility performance of UHPC. The toughness is the energy absorption ability of materials to resist deformation processes and is an important parameter used to evaluate crack resistance performance, while the flexural strength increase with the improvement of the fiber orientation (shown as Figure 2.7). Ductility refers to the ability of a component to resist deformation after reaching its ultimate bearing capacity. To reduce the brittle failure of components under external forces, the ductility performance of UHPC should be considered. The flexural hardening characteristics of UHPC make it have good strength and ductility. In practical engineering applications, there are few purely tensile components, and most of them are flexural tensile components. Therefore, UHPC with flexural hardening characteristics is of great significance for structural safety [62].

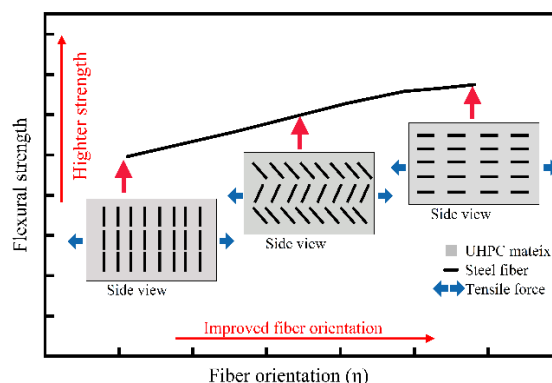


Figure 2.7 The relationships between fiber orientation and flexural strength of UHPC[63].

The type and dosage of mineral admixtures will determine the quality of UHPC matrix and thus affect the bending behavior of components. Du [64] studied the feasibility and benefits of using fly ash to prepare UHPC. The influence of different fly ash contents on the bending performance of UHPC was tested. It was found that the use of silica fume reduced the bending strength and toughness of UHPC, as silica fume delayed the hydration reaction. When the content of silica fume increases from 0 to 30%, the bending strength decreases by 36% and the toughness decreases by 36%. Wu [65] used sugarcane bagasse ash instead of cement to develop green UHPC, and analyzed the effect of sugarcane bagasse ash on the bending strength of UHPC 40 mm × 40 mm × 160 mm beams. The results showed that 20% sugarcane bagasse ash had an improving effect on the bending strength of UHPC, but the bending strength of UHPC with a 60% substitution rate was 20.7% lower than that of the control group. Wu [66] evaluated and compared the enhancement effect of nano-SiO₂

and nano-CaCO₃ on the mechanical properties of UHPC. The results showed that with the increase of nano-SiO₂ and nano-CaCO₃, the fiber matrix strength and mechanical strength of UHPC significantly increased. The 28 day bending strength of UHPC prepared with 3.2% nano CaCO₃ increased by 20% compared to the control group.

The influence of fibers on the bending performance of UHPC is also significant. Meng ^[67] studied the effects of PVA fibers, PE fibers, and calcium carbonate whiskers on the flexural properties of ultra-high performance permeable concrete. The experiment showed that the addition of PVA fibers or PE fibers significantly improved the flexural strength and ultimate deflection, and the improvement effect of PVA fibers was better than that of PE fibers. The mixed use of calcium carbonate whiskers can further improve the flexural deformation resistance. Abbas ^[68] studied the effect of different lengths and dosages of steel fibers on the splitting strength and bending strength of UHPC, and found that UHPC with shorter steel fibers exhibited better bending performance compared to long steel fibers with similar volume dosages. El Hassan ^[69] studied the effect of hybrid steel fibers on the flexural properties of alkali slag fly ash blended UHPPC. It was found that the tensile strength and bending strength of UHPC are mainly affected by the curvature and tensile strength of steel fibers. The higher the bending degree of steel fibers, the better the tensile strength, compression performance, and bending toughness of UHPC. Wang ^[25] studied the effects of steel fiber content and concrete strength on the flexural toughness of coarse aggregate UHPC, and the results showed that the toughness index was highest when the fiber content was 1.5% -2%. The fracture energy of the sample increases linearly with the increase of steel fiber content. As the bending strength increases, the fracture energy shows a trend of first increasing and then decreasing. Dong ^[70] studied the effect of microfilament steel fibers (diameter 20 μ m, length 10 mm) on the bending toughness of beams and plates with and without notches. The results showed that adding 1.5% microfilament steel fibers can increase the bending toughness of notched beams by more than twice, and increase the bending toughness of notched beams and plates by more than 80%. He ^[71] studied the bending performance of glass fiber mesh reinforced UHPC boards through bending tests on simply supported four sided plates. The results show that glass fiber mesh can effectively suppress the initiation and propagation of main cracks in UHPC boards, and make the board exhibit deformation hardening characteristics. With the increase of mesh layers, the ultimate bearing capacity and bending toughness are significantly improved.

The curing conditions are a crucial step before the final formation of UHPC, and play an important role in improving the bending toughness and ductility performance of UHPC. Ma ^[72] conducted bending fatigue tests on UHPC at different stress levels at 20 °C and -10 °C. The results indicate that with the decrease of temperature, the degradation of UHPC bending fatigue performance under cyclic loading is significantly intensified due to the cold brittleness characteristics of steel fibers in the cracked section of the specimen, and the fatigue strength is also significantly reduced. Yang ^[73] tested the bending performance of UHPC at different curing temperatures through experiments and found that the compressive strength of the specimens cured at 90 °C for 1 to 7 days decreased by about 20%, the bending strength decreased by 10%, and the fracture energy decreased by 15% compared to the specimens cured at 20° C. Hassan ^[74] studied the effect of pouring temperatures of UHPC at 10, 20, 30, and 90 °C on the compressive and flexural tensile strength of UHPC. The results showed that the development of early compressive and flexural tensile strength is highly dependent on the setting temperature. The specimen solidified at 90 °C reaches its maximum compressive and flexural strength at 7 days of age. The sample with lower curing temperature has an impact compressive strength greater than 45 MPa after 3 days of pouring. Tian ^[75] studied the effect of low-temperature curing conditions on the bending performance of recycled concrete powder ECC through experiments, and found that the ultimate deflection of ECC board was the highest at -10 °C, which was 53.6 mm. The maximum ultimate strength is 5.3 MPa at 0 °C. The influence of different factors on deflection is in the order of fly ash content, temperature, water cement ratio, and fiber content.

2.2 Fiber bridging mechanisms in UHPC

Adding fibers to concrete is one of the effective methods to improve its mechanical properties ^[76, 77]. In the past few decades, many scholars have conducted extensive research on FRC. Fibers interact with concrete through forces such as adhesion, friction, and mechanical anchoring. The presence of fibers is a key factor affecting the mechanical properties, creep, shrinkage, and cost of UHPC, what's more, there are many types of fibers that can be selected to be added to UHPC, as shown in Figure 2.8. When cracks occur in materials or structures, it can lead to strength loss and reduced toughness. The fibers existing inside the matrix can play a role in preventing cracking and bridging, and continue to bear the load after cracks occur, improving the overall ductility, toughness, and impact resistance of the structure. Fibers maintain the integrity of components before complete

failure^[78], playing a critical role in the safety design of structures. In the material cost composition of UHPC, steel fibers account for the largest proportion, reaching more than one-third of the total material cost of UHPC. Therefore, considering the influence of factors such as fiber direction, volume fraction, combination, and geometric shape, improving the reinforcement and toughening efficiency of fibers on the basis of controlling fiber content is crucial for the development of UHPC in large-scale application.



Figure 2.8 Different types of fibers used in UHPC^[79].

Fiber matrix interface bonding refers to the stress transfer process from the surrounding matrix to the fiber interface. The fiber matrix interface bonding performance is a key factor determining the tensile, bending, and compressive strength, failure mode, fracture behavior, and other mechanical properties of UHPC^[80]. When the bonding strength is lower than the tensile strength of the matrix, the fibers will pull out under lower loads, resulting in a significant decrease in the strength utilization rate of composite materials such as UHPC. On the contrary, if the bonding strength between the fiber and the matrix is too high, and the fiber itself has insufficient strength, the excessive bonding strength will cause the fiber to suddenly break and/or the matrix to peel off, making it unable to resist the development of cracks during the drawing process. Therefore, it is necessary to understand the bonding failure mode and pull-out behavior between fibers and matrix in order to further reveal the crack resistance and toughening mechanism of fibers in UHPC and improve the mechanical properties of UHPC.

The intrinsic effects of fiber matrix bonding behavior mainly include three factors: chemical/physical bonding, friction, and mechanical interlocking/anchoring^[81]. For fiber-reinforced cement-based composite materials, since there is generally no chemical change between the fibers and the cementitious matrix, the physical bonding force is usually much greater than the bonding force generated by chemical reactions. Friction is generally generated by the squeezing of uneven cement slurry and fiber surfaces at the micro or micro scale. The mechanical interlocking/anchoring effect is generated through the entanglement between fibers and the interlocking effect between fibers and aggregates.

According to the different directions of stress transmission and fiber orientation between fibers and matrix, the bonding force between fibers and matrix can be divided into shear bonding force (parallel to the interface) and tensile bonding force (perpendicular to the interface) ^[82]. Shear bonding force is one of the main factors affecting the failure mode and mechanical behavior of FRC due to the resistance of the matrix to fiber pull-out. In the elastic stage of adhesive force development, the adhesive force is distributed along the fibers, which helps to safely transmit stress. When the bonding force exceeds the gripping force (pull-out force) of the matrix on the fibers, microcracks occur at the interface between the fibers and the matrix and gradually develop. The bonding stress will be transmitted through the bridging fibers at the crack location. Once the critical value is reached, relative slip will occur between the fibers and the matrix, resulting in shear bonding stress failure. After the failure of elastic bonding force, it will enter the friction bonding stage, where the frictional force at the interface becomes the main force limiting the fiber pull-out. The magnitude of the friction bonding force is mainly influenced by the friction coefficient at the interface, fiber stress per unit length, and Poisson's ratio^[83].

Yoo ^[47] studied the effects of matrix strength and fiber type on fiber pull-out performance through experiments, and the results showed that with the increase of matrix strength, the pull-out performance of fibers was improved. Compared with straight fibers, hooked fibers exhibit higher bonding strength and tensile strength, and compared with hooked fibers, straight fibers are more effective in improving matrix strength. Wille ^[84] evaluated the tensile response of straight steel fibers in UHPC with different compositions. The sand composed of SiO_2 and ZrSiO_4 increases the roughness of the matrix around the fibers, thereby improving the friction and bonding stress of the fibers. When using smaller glass powder and hydrophilic nano- SiO_2 , the bonding performance is

improved. Yoo ^[85] studied the effect of matrix shrinkage on the pull-out behavior of straight steel fibers embedded in UHPC. The results showed that the bond strength and pull-out work between fibers and matrix decreased with the decrease of matrix shrinkage, because low matrix shrinkage reduced the radial constraint stress on fibers. In Lee's study ^[58], the influence of fiber inclination angle on pull-out load was considered. Due to the combined effects of compression friction and matrix peeling, the peak pull-out load of straight steel fibers in UHPC was highest when the fiber inclination angle was 30° or 45°. Hashimoto ^[86] studied the tensile properties of steel fiber-reinforced UHPC after crack initiation in corrosive environments and found that steel fibers treated with moderate corrosion can significantly improve the tensile and bending properties of UHPC components. Qin ^[87] pointed out that the pull-out load of inclined nylon and polypropylene fibers is a function of the inclination angle, and proposed that the pull-out of flexible fibers from the matrix is similar to a frictional sliding process, which is the combined effect of extrusion force and friction force. Zhang ^[88] found that the bond strength between hook fibers and corrugated fibers in UHPC is 20.6 MPa and 33.3 MPa, respectively, which is about 3.81 times and 6.17 times that of straight fibers.

After FRC damage and cracking, bridging fibers can prevent microcracks in the matrix from continuing to expand and withstand tensile stress. The bridging effect plays a particularly significant role in preventing matrix cracking. When the fiber bridging stress across the crack exceeds the critical frictional sliding stress or the ultimate tensile strength of the fiber itself, the fiber will be pulled out of the matrix or break. This process can cause cracking or peeling of the substrate. The failure mode of fiber fracture belongs to brittle failure, which largely depends on the difference between the strength of the fiber itself and the properties of the matrix. For UHPC with added steel fibers, the strength of the cement matrix is much lower than that of the steel fibers themselves, so the fibers will undergo pull-out failure mode. The optimal combination mode of fibers and matrix is to fully utilize the advantages of fiber strength while avoiding brittle failure of FRC materials or structures composed of brittle matrix ^[89].

The theoretical model of fiber pull-out is crucial for studying and predicting the performance of FRC. Up to now, numerous predictive models have been developed to evaluate the fiber pull-out characteristics and bonding behavior embedded in ordinary concrete or high-strength concrete matrices. For straight fiber-reinforced concrete, the proposed analysis models include the shear lag

model^[90]; Revised shear lag model ^[91, 92]; Elastic friction coupling debonding model ^[93]; Friction attenuation semi empirical model^[94]. For end hooked shaped fibers, the failure process during drawing is relatively complex, and the interfacial debonding mechanism of fibers with different geometric shapes is different. Therefore, models based on virtual work (energy conservation) ^[95], fracture mechanics ^[96], and empirical analysis ^[97] have been proposed successively.

Usually, the bonding performance between UHPC fibers and matrix depends on the characteristics of the fibers, matrix, and their interactions. These characteristics and factors include fiber geometry and surface roughness ^[98], mechanical and deformation properties of fibers ^[38], spatial orientation of fibers ^[99], matrix density ^[56], interactions between fibers (mixing effects) ^[100], etc.

Chun ^[98] observed that when polishing straight fibers along the fiber axis, the bonding strength of UHPC increased by 66%. When polishing the fiber surface vertically, the load displacement curve of fiber pull-out exhibited hardening characteristics. Frazao ^[101] conducted experiments on the pull-out characteristics of corroded fibers and found that the rough fiber surface enhanced the bonding performance. However, corrosion can also lead to a reduction in the effective area of steel fibers, which may result in significant brittle characteristics of UHPC components. It is believed that a corrosion level of 2% is the upper limit. Zhang ^[88] observed that compared with fibers with fewer ripples and double ended hooked fibers, fibers with more ripples had significantly increased pull-out resistance and pull-out energy. However, compared with single ended hooked fibers, the pull-out resistance of double ended hooked fibers was basically not improved. Yoo ^[50] studied the effects of fiber hook length and three different end hook angles (30 °, 45 °, and 60 °) on the fiber UHPC bonding behavior. The experiment found that during the drawing process, the fibers also continuously deform, and the end hook length increases from 2.5 mm to 5 mm, resulting in a 70% increase in static bonding strength. Wille ^[38] compared the pull-out characteristics of twisted fibers and smooth fibers in UHPC matrix, and the results showed that the bonding strength of twisted fibers was 4.7 times that of smooth fibers. Compared with hook fibers, twisted fibers also exhibit more obvious sliding hardening characteristics.

For UHPC reinforced with short fibers, without intervention, the fibers in the matrix are in a randomly distributed state. Fiber orientation is not always conducive to load transfer direction and perpendicular to the cracking surface^[102], but tends to be a macroscopic statistical strengthening

effect. The reinforcing and toughening effect of fibers always reaches its maximum within a certain orientation range. Yoo ^[50] verified through experiments that the maximum pull-out resistance of straight, hooked, half hooked, and twisted fibers is between a tilt angle of 30 ° to 45 °, rather than between 0 ° or 60 °. The results indicate that due to matrix peeling, the fiber sliding displacement increases with the increase of tilt angle. Lee ^[103] expressed the shear strength of straight fibers as a function of fiber inclination angle, and fitted the bond shear stress inclination angles at 15 °, 30 °, 45 °, and 60 °. It was found that the bond shear stress was highest at a fiber inclination angle of 45 °. Cao's study ^[55] showed that compared with fiber inclination angles of 45 ° and 60 °, the effect of orientation angle on pull-out load was not significant when the fiber embedding angle was 20 ° and 30 °. As the fiber inclination angle increased, the normalized pull-out energy of twisted and hooked fibers significantly decreased.

Numerous studies have shown that the advantages of single fiber reinforced FRC are limited. Attempting to incorporate different types of fibers will enhance its mechanical properties at multiple scales and different degrees of cracking ^[104]. Chun ^[100] conducted multi fiber tensile tests on different types of steel fiber UHPC. It was found that, with a constant total fiber volume fraction of 2%, when 0.5% of micro straight steel fibers were replaced by twisted fibers and hooked fibers, the average bond strength of micro straight fibers increased by 7.8% and 17.4%, respectively. Banthia ^[105] studied the blending effect of two types of steel fibers and one type of cellulose natural fiber, and conducted bending and direct shear tests. The results showed that containing two or more types of fibers can improve the performance of FRC similar to adding additives. Lawler's study ^[106] shows that compared to a single type of steel fiber reinforced concrete, steel fiber microfiber hybrid reinforced concrete has higher strength and toughness. This is because microfibers can suppress or delay the formation of macroscopic cracks, thereby improving the tensile strength of FRC. He ^[107] studied the blending effect of glass fiber (GF) and polypropylene (PP) in UHPC, and the results showed that glass fiber had a more significant reinforcing effect. Compared with PP-UHPC, GF-UHPC has increased its compressive, tensile, and flexural strength by about 20%, 30%, and 40%, respectively.

2.3 Interface bonding performance between NC and UHPC

With the extension of service time, more and more concrete structures are facing the threat of damage and deterioration, requiring maintenance and reinforcement. However, many existing

reinforcement methods have problems such as unclear improvement in mechanical properties, limited improvement in durability, or causing secondary damage to the original structure [108, 109]. Among the reinforcement methods, increasing the cross-section method has a lower cost and will not cause excessive damage to the original structure, making it a very ideal reinforcement method, and there are many test methods for the interface bonding performance as shown in Figure 2.9. At present, the new cement-based material UHPC has been well applied and developed in the field of bridge reinforcement [110], making the reinforcement layer thinner and more aesthetically pleasing while meeting mechanical properties and durability. The bonding performance between UHPC and ordinary concrete interface is the key for the UHPC reinforcement engineering [111]. Due to the complex situation that the reinforcement surface of actual structures is often subjected to tensile loads, shear loads, or a combination of multiple loads, the new old interface is prone to failure after structural reinforcement. Therefore, revealing and investigating the interface roughness characteristics on the debonding behavior and mechanical performance degradation mechanism of NC-UHPC interface is the key to improving the structural reinforcement efficiency and ensuring its safe service.

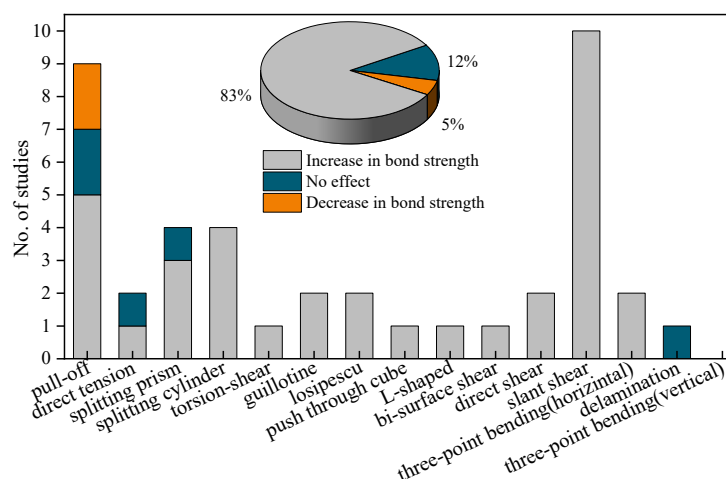


Figure 2.9 Studies for interface bonding performance test methods.

The bonding between the substrate concrete and the repaired UHPC material mainly comes from three aspects: chemical forces, thermodynamic bonding mechanisms, and macroscopic mechanical interactions at the interface. Chemical bonding force is the weak chemical bonding force generated during the solidification process of post poured repair materials on the surface of the base concrete. This chemical bonding force is generally believed to be caused by the van der Waals force generated by the hydration of cement particles on the surface of the base concrete that have not undergone

hydration reaction^[112]. The bonding effect of thermodynamic mechanism refers to the bonding force generated by the gradual filling and formation of a dense microstructure at the microscale due to the growth of hydrated calcium silicate during the setting and hardening of the cover layer concrete. Sadowski's nano-scale study on the interface region showed that the hydration products of cement, calcium silicate hydrate (C-S-H), ettringite (AFt), and boromagnesite ($\text{Ca}(\text{OH})_2$), grow anisotropically, filling the micropores on the surface of the concrete substrate and forming a mutually permeable bonding layer. The macroscopic mechanical effect of the third aspect is manifested as the mechanical adhesion force of the adhesion layer to the hardened concrete substrate, which is influenced by the characteristics of the base concrete, the properties of the repair material, and environmental conditions. Courard^[113] found that the mechanical adhesion mechanism of the interface is similar in many aspects to the interfacial bonding behavior between cement slurry and coarse aggregate, and there is a similar wall effect that leads to the formation of transition zones.

The bonding area between the concrete substrate and UHPC repair material is the weakest part of the entire reinforcement structure, so the bonding performance between the base concrete and UHPC repair material is crucial for the repair effect^[114]. The factors that affect the interfacial bonding performance can be considered from two aspects: the characteristics of the repair material and the characteristics of the substrate material. For the repair materials, these factors specifically include rheological properties, fiber type and content, strength, aggregate type and content, shrinkage characteristics, etc. For the substrate material, the influencing factors include substrate strength, surface moisture level, surface roughness, and the presence or absence of adhesive. In practical engineering, there are usually three methods to improve the bonding performance of these two materials. The first method is to increase the surface roughness of the substrate material, which includes treatment methods such as wire brushing, sandblasting, high-pressure water gun washing, and chiseling^[115, 116]. The second method is to use organic or inorganic interface agents to enhance the bonding performance of the interface^[117]. The third method is to use high-performance repair materials such as high-performance FRC to improve the interfacial bonding performance^[118].

In terms of the influence of repair material properties on bonding behavior, Feng^[119] conducted experimental research on the interface failure characteristics of UHPC wet joints and found that compared with untreated fiber specimens, fiber doped UHPC wet joint specimens have better interface bonding performance (interface tensile strength can reach 6.64 MPa). When the fiber

volume fraction is less than 2.5%, the interface tensile strength and interface fracture toughness both increase with the increase of fiber volume fraction. Long fibers have a better effect on improving the interface performance of UHPC than short fibers.

Oommen ^[120] investigated the bonding behavior between the latex modified concrete and the ordinary concrete, and improved the tensile strength of the cover layer by adding 0.5%, 1.0%, and 1.5% short carbon fiber reinforced latex modified concrete. And it was found that the increase in carbon fiber content did not significantly improve the bonding strength of the sample interface. Erhard's study ^[121] pointed out that directly laying unmodified cement mortar on the surface of dry concrete can cause a portion of the moisture in the slurry to be absorbed into the base concrete before the cement slurry undergoes sufficient hydration reaction. The post poured concrete pavement layer will not firmly adhere to the base surface, resulting in a decrease in bonding performance. This also reflects the importance of wetting the substrate surface. Diab ^[122] studied the effect of self compacting concrete cover layers with 0.1% and 0.2% polypropylene fiber content on bond strength. The experimental results show that as the flowability of self compacting concrete increases, the ability to fill the base pores is improved. After adding 0.1% and 0.2% polypropylene fibers, the bond strength values increased by 10% and 14%, respectively. Farzad ^[123] conducted direct shear, oblique shear, and three-point bending tests on composite components composed of ordinary concrete and UHPC, and found that the bonding effect between UHPC and the substrate was better than that of self compacting concrete paving. This is because UHPC can easily penetrate into the tiny pores of the substrate through surface capillary action, thereby forming high durability and high density joints. Feng ^[124] evaluated the roughness of the base concrete and measured the shear bond strength by exposing coarse aggregates. The results showed that compared with adjusting the water cement ratio and fiber type of the UHPC cover layer, the treatment method of exposing coarse aggregates can more effectively improve the interfacial bearing capacity. Beushausen ^[125] found that the hydration process of UHPC in the cover layer can cause shrinkage due to excessive evaporation. At this time, the substrate concrete may exert excessive constraints on the repair material, leading to debonding or even cracking of the cover layer. Therefore, in the early stage of bond strength development, excessive shrinkage of the cover layer should be controlled.

In terms of the influence of interface roughness, many studies have shown that different interface roughness has a significant impact on the bonding behavior of NC-UHPC interface ^[126, 127]. Tayeh

[128] found that the interface bonding, splitting tensile strength, and diagonal shear strength of sandblasted specimens increased by over 100%. Tian [129] studied the effects of engineering cement-based composite materials (ECC), polyvinyl alcohol (PVA) fiber types, and interface roughness on the tensile properties of ECC concrete interface through experiments and mechanical models. Prado [130] discussed the effect of surface treatment of high-strength concrete (HSC) on the interfacial strength of HSC-UHPC. Consider surfaces with exposed coarse aggregates, shear bonds, exposed fine aggregates, exposed fibers, and surfaces that have not undergone any special surface treatment (smooth). Hong [131] studied the effects of surface roughness of ordinary concrete substrates and different treatment methods on the interfacial bonding strength of NC-UHPC through splitting tensile tests and direct shear tests. In the groove interface perpendicular to the loading direction, the interface shear strength is significantly increased. Valikhani [115] conducted a double shear test study on the bond strength between UHPC with different surface treatments and substrate ordinary concrete. Different surface treatments were performed on 30 samples, including increasing interface roughness, reinforcing treatment, and adding adhesives. The roughness of the substrate interface was evaluated and correlated with the bonding strength between the two materials. Zanotti [118] studied the influence of different types of fibers on bonding performance using oblique shear tests. The experimental results show that using steel fiber reinforced concrete as a repair material can significantly improve the bond strength and enhance the durability of the repair structure. Al Madani [132] conducted a series of studies on the key influencing factors on the interfacial bonding strength between UHPC overlay and ordinary concrete substrate. We discussed the variable parameters that affect the interfacial bonding behavior, such as different surface roughness, curing conditions, and exposure conditions of ordinary concrete substrates.

2.4 Multi scale calculation method of concrete materials

UHPC, similar to other cementitious materials, is essentially a multi-scale material. As shown in Figure 2.10, according to the different research objects and purposes, the research scale of cement-based materials can be divided into four scales: nanoscale (10^{-9} m), microscopic (10^{-6} m), microscopic (10^{-3} - 10^{-2} m), and macroscopic (10^{-2} - 10^{-1} m) [133, 134]. Nano scale is the smallest material scale that can be measured by means of test at present, and the research object is the main hydration product of cement - calcium silicate hydrate (C-S-H) gel. Considering the computational efficiency, this level is usually not taken into account in structural analysis. At the micro scale, the research

object is cement paste, including C-S-H gel, unhydrated cement particles, $\text{Ca}(\text{OH})_2$ crystals, aluminate phase and pores [135]. At this scale, the parameter that is more sensitive to the accuracy of analysis and calculation is porosity, because the final thermal mechanical properties of cement paste depend on its own microstructure, especially the pore structure. The research objects at the microscale mainly include coarse aggregates in concrete, cement mortar, various toughened fiber materials in the matrix, as well as shrinkage cracks and interfacial transition zones (ITZ) formed inside after setting and hardening [136, 137]. At the macroscopic scale, research is mainly conducted on the mechanical behavior of different forms of structures and components in concrete, treating concrete as a uniform continuous material and describing it using the continuity constitutive equation. Multi scale modeling methods allow for the study of UHPC at each length scale, which is necessary for materials with significant heterogeneity at different length scales.

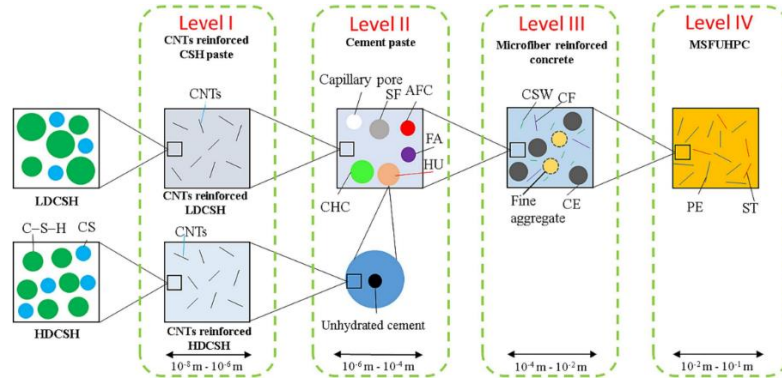


Figure 2.10 Scale division of concrete calculation^[138].

For the practical engineering or scientific problems, strictly speaking, uniform materials do not exist. The assumed continuous uniform medium material is an abstract physical conceptual model that balances the relationship between accuracy, efficiency, and cost by simplifying and approximating the real problem when an exact solution cannot be obtained, in order to obtain the best solution to the problem. For cement-based or similar composite materials, numerous factors lead to the ultimate behavior of the material being a manifestation of macroscopic statistical physics laws, and excessive pursuit of accuracy in calculation results may not necessarily achieve better results. For the homogenization approximation of composite materials, while meeting the requirements of solution accuracy, the resolution of the research problem space can be appropriately reduced by ignoring factors that have a small impact on the results, thereby improving computational efficiency.

In the 1970s, Bensoussan^[139] and Enrique^[140] introduced a mathematical alternative method for

studying heterogeneous materials, which later became known as homogenization theory. The homogenization theory and methods are generally used to analyze physical systems with two or more scales, and to link the microscale containing the second phase space with the macroscale corresponding to the overall structure. By progressively expanding the parameters of the physical field through perturbation equations and combining them with appropriate variational principles, multiple scales of parameters are linked together. The homogenization method can not only obtain the equivalent parameters of the material, but also obtain the stress and strain distribution information of the studied object. Compared to the analysis method representing volume elements (RVE), the advantage of progressive analysis method is that it does not require periodic assumptions at the macroscopic scale, and the microstructure at the macroscopic level can have randomness.

The homogenization method is a method of calculating and analyzing the equivalent mechanical properties of RVE selected from heterogeneous composite materials at the microscale. After the concept of homogenization was proposed, various models and calculation methods for predicting the equivalent properties of composite materials were developed based on Eshelby's equivalent inclusion theory ^[141], mainly including the sparse method, self consistent method, generalized self consistent method, Mori-Tanaka method developed from inclusion theory, boundary method based on variational principle, differential method, and asymptotic homogenization method ^[142]. The above methods have their own advantages in solving the equivalent properties of composite materials, and should be selected reasonably according to the specific problems.

In terms of equivalent and predictive macroscopic elastic properties of concrete, Haile ^[142] proposed a homogenization method based on multi-level micro mechanics by combining experimental research on compressive strength testing, scanning electron microscopy, and mercury intrusion porosity testing. Factors such as fiber type, volume fraction, orientation, size and volume fraction of coarse aggregates were considered to calculate and predict the elastic mechanical properties of UHPC. Li ^[143] applied principles of micromechanics to study the influence of aggregates on the properties of concrete, specifically proposing a four phases spherical model to estimate the effective bulk modulus of three-phase concrete. By simplifying the four phases spherical model into an equivalent three-phase spherical model and an equivalent two-phase spherical model, the calculation formula for the equivalent modulus was derived. The calculated effective modulus is consistent with the experimental results in existing literature. Da Silva ^[144]

described a method for determining the macroscopic elastic properties of UHPC based on nanoindentation measurement. To this end, the elastic properties of UHPC mixtures containing quartz powder fillers and fly ash were evaluated at different length scales through nanoindentation (microscale) and static and dynamic elastic modulus and compressive strength testing (macroscale). In Zhang's work ^[145], the effects of uneven interfaces and inclusion sizes were considered by introducing a spring layer interface model and a logarithmic normal size distribution function into the homogenization scheme. The temperature dependent elastic modulus of UHPC was estimated by gradually homogenizing from the matrix scale of gel to the macro scale of UHPC. Ouyang ^[146] proposed an analysis model based on component content using the Mori Tanaka multiscale homogenization method, which takes the volume fraction and elastic modulus of the microscopic phase at different scales as inputs to estimate the corresponding elastic modulus at the macroscopic scale. And the elastic modulus of cement paste, mortar, and UHPC provided in the literature were used for verification. In Gao's study ^[147], the mechanical performance parameters and volume fractions of various microscopic phases of UHPC were measured through nanoindentation and backscattered electron image analysis. The micro mechanical properties of the hardened paste were measured using the micro indentation method. In the input of homogenization, it was found that the effective modulus of pores is not 0 but needs to be set to 12 GPa. Finally, the Mori-Tanaka calculation framework was used to provide the equivalent modulus of hardened UHPC. Chen ^[148] proposed a multi-scale computational framework based on hydration kinetics to predict the macroscopic elastic modulus of UHPC. The volume evolution of each component was quantitatively calculated by the reaction degree between Portland cement and silica fume, and the elastic modulus of UHPC was accurately predicted using a multi-stage homogenization scheme.

In terms of multi-scale research on fiber-reinforced cement-based materials, Sun ^[149] studied the influence of basalt fiber length and content on the basic mechanical properties of concrete through multi-scale simulations. A damage constitutive model was established based on Mori Tanaka homogenization theory and progressive damage theory at the microscale to predict the composite material properties of basalt fiber reinforced concrete. The compression, splitting, tensile, and bending properties of basalt fiber reinforced concrete were studied through experiments and numerical simulations. Abaimov ^[150] combined Mori Tanaka homogenization theory to investigate the effects of fiber content, size effect, and fiber length on fatigue life of steel fiber reinforced

concrete. He predicted the elastic modulus of steel fiber reinforced concrete under different volume contents and conducted numerical simulations of static bending tests and fatigue tests. The simulation results were well fitted with the relevant experimental results. Dhar ^[151] analyzed the elastic modulus and compressive strength of basalt fiber reinforced concrete using the Mori Tanaka homogenization theory. Analysis shows that when the fiber content is between 0% and 0.25%, increasing the fiber content can improve the elastic modulus and compressive strength of basalt fiber reinforced concrete. Fiber agglomeration can reduce the performance of basalt fiber reinforced concrete, but the impact is very small. Li ^[152] used homogenization theory and finite element simulation to calculate the elastic modulus and Poisson's ratio of fiber-reinforced concrete, and established a theoretical model for predicting the elastic modulus and Poisson's ratio of fiber-reinforced concrete using homogenization theory. In addition, by comparing the calculation results with the microscopic random fiber numerical model, it was found that the predicted values of the homogenization theory were in good agreement with the finite element simulation values and macroscopic experimental values, which verified the accuracy of the homogenization theory model and the finite element model. Zhang ^[145] studied the effect of fiber geometry and orientation on the effective thermal conductivity of polypropylene steel fiber hybrid concrete at high temperatures through simulation and experimentation. A multi-scale thermal conductivity model was developed based on the Mori Tanaka scheme to predict the thermal conductivity of concrete as a temperature function, and the influence of aspect ratio and orientation distribution of hybrid fibers on macroscopic properties was studied.

For the interface homogenization investigation, Li ^[153] uses quasi-static homogenization projection to characterize the mechanical behavior of the effective bonding zone of rough surface adhesion. In the context of uniform projection, due to the instability during micro adhesion and debonding processes, the traction and separation relationship in the homogenized cohesive zone of rough interfaces may be highly oscillatory. Based on cohesive mechanics behavior, an analytical framework is proposed to describe the energy dissipation induced by unstable bonding on rough surfaces, revealing the influence of roughness on surface adhesion. Mohammadi ^[154] studied the homogenization of surface properties and derived the correlation between periodic and random roughness and effective surface elastic behavior. Research has found that surface residual stress is almost unaffected by roughness, while surface elastic properties undergo significant changes. Alfaro

[155] used numerical homogenization theory to correlate the discrete microscale fracture process in thin fiber epoxy layers with the microscale traction separation law. By setting interface elements with mixed failure modes between continuous elements of simulated fibers and epoxy resin, he simulated cracking within epoxy resin and debonding behavior between fibers and epoxy resin. Hirsch [156] proposed a modeling strategy for studying the failure behavior of rough interfaces in computational homogenization schemes, investigating the effects of surface roughness and different local failure types on the overall interface. Effective traction separation laws were extracted using numerical homogenization methods, and effective interface parameters were derived. Palmieri [157] proposed a multi-scale method based on micromechanics analysis of interfacial viscous failure of FRP concrete. The cohesive force relationship under macroscopic mixed modes was obtained through numerical homogenization program, and the selection process of representative volume elements was discussed. The bond failure performance under load conditions of type I, type II, and different degrees of mode mixing was analyzed. Hirsch [158] considered the bonding performance of metal and glass fiber reinforced epoxy resin at the microscale. The interface failure is controlled by the cohesive force model, and the rough interface numerical homogenization method is used to predict the effective traction separation relationship of the interface under tensile action. The simulation results indicate that an increase in interfacial roughness has a positive impact on bonding behavior.

Chapter 3 Design and Performance Testing of UHPC Materials

3.1 Overview

After collecting a series of localized raw materials, the compatibility of the mix ratio of localized UHPC materials based on the theory of maximum packing density calculation was analyzed in this chapter. Experimental research was conducted on the relationship between UHPC powder particles, aggregate particles, and workability using a multi-step optimization method. Two types of fine aggregate UHPC with fiber contents of 1% and 2%, as well as two types of coarse aggregate UHPC with fiber contents of 0.75% and 1.5%, were ultimately determined. On this basis, micro scale electron microscopy (SEM) experiments and macro scale cube compression, dog bone tension, and thin plate four point bending experiments were conducted. The SEM experiment focused on observing the micro local morphology of UHPC and the bonding status between hydration products and fibers. The mechanical performance test analyzed the effects of fiber content and coarse aggregate volume fraction on the strength and tensile hardening behavior of UHPC.

3.2 Collection of raw materials

The physical and chemical properties of raw materials have a significant impact on UHPC. Choosing appropriate localized raw materials is crucial for the development of UHPC mix proportions. Raw material screening includes evaluating and selecting the compatibility, appearance, and workability between materials. Specific tasks include: determining the density of candidate materials, such as particle density of powders and apparent density and bulk density of aggregates; Determine the compatibility and optimal content ratio between powder materials based on the Haegermann cone measurement of slurry flow expansion; Prepare cubic compression specimens for strength comparison and mix proportion selection. After successive screening, all raw materials used for this study have been determined, and the specific requirements for each raw material are as follows.

(1) Cement

According to the influence of Portland cement on concrete properties, the cement selected for preparing UHPC can meet the sulfate resistance, low aluminum tricalcium content, and good early strength conditions of CEM I 52.5N and CEM I 42.5R ^[159] of Portland cement. The cement used in this study is P·II type Portland cement with a strength grade of 52.5. The chemical composition of

cement is shown in Table 3.1, and the basic physical properties are shown in Table 3.2.

Table 3.1 Chemical Composition of Cement (wt.%)

Content	Na ₂ O	Al ₂ O ₃	Fe ₂ O ₃	K ₂ O	CaO	SiO ₂	SO ₃
	0.21	2.76	4.01	0.49	67.87	20.25	2.61

(2) Reactive powder materials

The reactive powder materials used for preparing UHPC are mainly silica fume, fly ash, and ground blast furnace slag. The activity index of these reactive additives is determined according to European standards EN 13263-1 ^[160] (silica fume), EN 450-1 ^[161] (fly ash), and EN 15167-1 ^[162] (blast furnace slag). In this study, silica fume was selected as the reactive powder in the UHPC mix, and its chemical composition and physical properties are shown in Tables 3.3 and 3.4.

Table 3.2 Basic Physical Properties of Cement.

Index	initial setting time (min)	final setting time (min)	Specific surface area (m ² /kg)	Average particle size (μm)	stability	compressive strength (MPa)		Flexural Strength (MPa)	
						3d	28d	3d	28d
	120	160	389	24.8	qualified	36.0	62.0	6.5	8.5

Table 3.3 Chemical Composition Information of Silica Fume (wt.%)

Content	Na ₂ O	Al ₂ O ₃	Fe ₂ O ₃	K ₂ O	CaO	SiO ₂	SO ₃
	0.20	0.20	0.10	0.25	0.25	98.0	0.52

Table 3.4 Basic Physical Properties of Silica Fume.

Index	density	Specific surface area (m ² /kg)	Particle size distribution (μm)		
	(kg/m ³)		D10	D50	D90
	3.03	17000	0.15	0.25	0.77

(3) Inert powder material

The inert powders used for preparing ultra-high performance concrete are mainly quartz powder and limestone powder, among which quartz powder is widely used due to its better effect. The particle size range of quartz powder required for preparing UHPC is generally between 0.45 μm and 200 μm, but due to the filling effect, finer quartz powder can perform better. The average particle size of the quartz powder used in this research is 12 μm, and the main physical and chemical performance indicators are shown in Table 3.5.

(4) Superplasticizer

Superplasticizers generally use polycarboxylate ether-based superplasticizers, and the dispersion

effect of liquid aqueous solution superplasticizers is better than that of powdery superplasticizers. Usually, superplasticizer powder is dissolved in water with a solid content of 40% and mixed to form a liquid superplasticizer with a density of about 1.05g/cm³. The superplasticizer used in this study is also a liquid superplasticizer, with a water reduction rate greater than 30%.

Table 3.5 Physical and Chemical Performance Indicators of Quartz Powder

Content	SiO ₂	Al ₂ O ₃	Fe ₂ O ₃	CaO	MgO	loss on ignition	adsorbed water
	96.28	0.34	0.034	0.081	0.26	1.10	0.17

(5) Water

The water used for UHPC preparation is tap water, which meets the requirements of the "Water for Concrete Mixing" standard (JGJ 63-2006).

(6) Fine aggregate

The fine aggregate used in the experiment is machine-made quartz sand. Select quartz sand with particle sizes ranging from 0.09 mm to 0.25 mm (fine sand) and 0.25 mm to 0.84 mm (coarse sand).

(7) Coarse aggregate

Adding an appropriate amount of coarse aggregate to UHPC can reduce its risk of early shrinkage cracking. The coarse aggregate used in UHPC in this study is crushed basalt aggregate with a particle size of 3-5 mm. The physical properties are shown in Table 3.6.

Table 3.6 Physical and Mechanical Properties of Basalt Aggregate

Cement content (%)	Apparent density (kg/m ³)	Bulk density (kg/m ³)	Tight packing density (kg/m ³)	Needle like content (%)	crushing value (%)	porosity (%)
0.5	2840	1750	1820	2.1	8.1	37.2

(8) Fiber

The fibers used in UHPC have higher elastic modulus and tensile properties compared to the matrix, while also having Poisson's ratio and thermal expansion coefficient close to the matrix.

Table 3.7 Physical Properties of Steel Fibers

Type	Length (mm)	Diameter (mm)	Aspect ratio	Elastic modulus (GPa)	density (kg/m ³)	breaking strength (MPa)	Elongation (%)
	13	0.2	65	210	7800	2850	<4%

The fiber used in this study is a straight copper plated microfilament steel fiber with a length of

13 mm and a diameter of 0.2 mm. Its physical properties are shown in Table 3.7.

The raw materials used in the study are shown in Figure 3.1.



Figure 3.1 UHPC raw material.

3.3 Mix proportion design

The significant difference between UHPC and ordinary concrete is the addition of fibers, and the distribution of fibers inside the UHPC matrix is closely related to the mixing process of UHPC preparation. Therefore, in order to fully disperse the fibers and maximize their ability to enhance and toughen the mechanical properties of UHPC, the mixing parameters, including the type of mixing equipment, feeding sequence method, and mixing time at each stage, are particularly crucial. In this study, the mix proportion development process of UHPC used the Eirich EL10 Plus strong mixer produced by Eirich Company in Germany. As shown in Figure 3.2, the mixer can provide

advanced functions such as real-time mixing speed control, mixing temperature display, and mixing power calculation. The mixer used in the preparation process is an 80 L single horizontal shaft mixer, as shown in Figure 3.3.



Figure 3.2 Eirich Strong Blender.

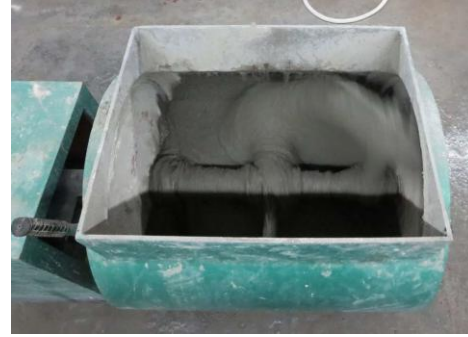


Figure 3.3 80 L single horizontal shaft mixer.

Optimization design of UHPC gradation based on the stacking model of ideal gradation curve theory. This stacking model describes the particle aggregation function $P(D_i)$ with different particle sizes D_i . In 907, Fuller [163] proposed the classic Fuller grading curve, as shown in equation (3.1):

$$P(D_i) = (D_i / D_{\max})^{1/2} \quad (3.1)$$

Where, $D_{\max}$ is the maximum particle size.

Afterwards, Andreasen and Andersen [164] developed the Fuller grading curve, which became the widely used A&A model, as shown in equation (3.2) below:

$$P(D_i) = (D_i / D_{\max})^q \quad (3.2)$$

Where, q is a parameter related to the particle size distribution, called the distribution modulus. The recommended range of values is between 0.1 and 0.5.

Later on, Funk and Dinger [165] realized that the A&A model included the case of infinitely small particle sizes and made modifications to the model, resulting in the modified A&A model (also known as the Funk Dinger model), as shown in equation (3.3):

$$P(D_i) = \frac{D_i^q - D_{\min}^q}{D_{\max}^q - D_{\min}^q} \quad (3.3)$$

This modified model has been widely applied in the design of concrete material mix proportions, and a large number of scholars have studied the distribution modulus q that affects the grading curve

[166-169]. In this study, the distribution modulus q was taken as 0.23 [170].

Considering the different functional requirements of UHPC in bridge engineering, such as collision wall repair, pavement repair, and small bridge load-bearing components, four localized UHPC material ratios were designed and determined with coarse aggregate content and fiber content as variable parameters. The specific ratio parameters are shown in Table 3.8.

Table 3.8 UHPC mix proportion.

Material/No.	FA-1	FA-2	CA-0.75	CA-1.5
cement	750	750	680	680
Silica fume	90	90	81.6	81.6
quartz flour	262.5	262.5	238	238
Superplasticizer	22.5	22.5	19.04	19.04
water	177.8	177.8	162	162
Quartz sand 1 (0.15-0.20 mm)	303.6	303.6	173.5	173.5
Quartz sand 2 (0.25-0.60 mm)	708.4	708.4	442.6	442.6
Basalt coarse aggregate	—	—	593.0	593.0
steel fibre	78	156	58.5	117

The four mix proportions are Fine Aggregate (FA) UHPC with fiber content of 1% and 2%, respectively, without coarse aggregate, numbered FA-1 and FA-2; Coarse aggregate (CA) with fiber content of 0.75% and 1.5%, with the characteristics of UHPC, numbered CA-1 and CA-2. The particle size distribution of raw materials is shown in Figure 3.4.

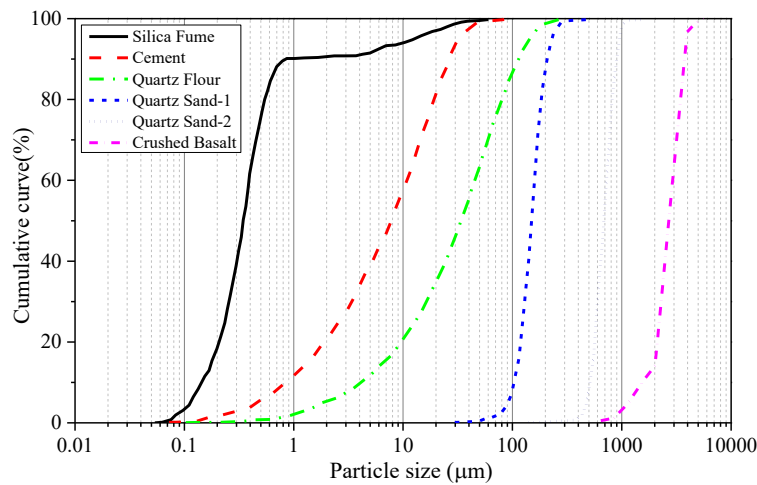


Figure 3.4 UHPC Particle Size Distribution.

3.4 Hardened UHPC mechanical performance test

3.4.1 Experimental Overview

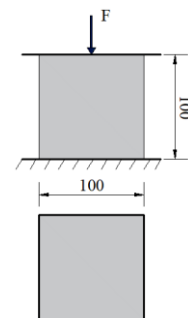
According to the four designed UHPC mix proportions, experimental studies were conducted on the modulus, compression performance, tensile performance, and bending performance of UHPC in accordance with European standards EN 12390-13 and EN 12390-3^[171, 172], Japanese standards^[173], and EN 12390-5^[174]. The prepared specimens are shown in Figure 3.5, and the test arrangement and specimen size information for the four types of tests are shown in Figure 3.6. The size of the cube compression specimen is 100 mm × 100 mm × 100 mm; The size of the cylindrical modulus specimen (height × diameter) is 200 mm × 100 mm; The dimensions of the dog bone tensile specimen (length × width × thickness) are 368 mm × 100 mm × 50 mm, with a tensile area of 50 mm × 50 mm; The dimensions of the thin plate bending specimen are 550 mm × 150 mm × 50 mm, with a loading spacing of 150 mm for four point bending and a pivot spacing of 450 mm. Three cubic compression specimens, five dog bone tensile specimens, five thin plate four point bending specimens, and three cylindrical specimens were prepared for each ratio.



Figure 3.5 Four types of UHPC specimens.



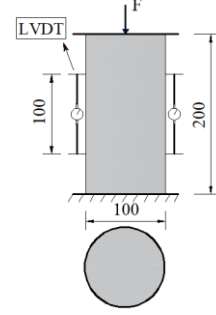
(a1) Cubic compression test



(a2) Cubic specimen size



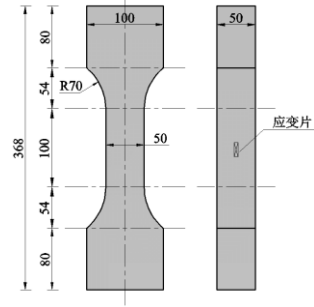
(b1) Cylinder modulus test



(b2) Dimensions of cylindrical specimens



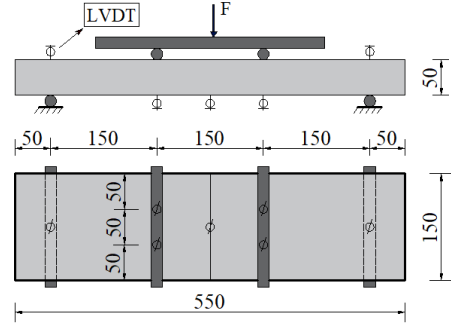
(c1) Dog bone tensile test



(c2) Size of dog bone specimen



(d1) Thin plate four point bending test



(d2) Dimensions of thin plate specimens

Figure 3.6 UHPC Mechanical Performance Test. (Unit: mm)

3.4.2 Test Program

To investigate the effects of UHPC fiber content and coarse aggregate content on its macroscopic mechanical properties, four types of tests were conducted: cube compression test, elastic modulus test, dog bone tensile test, and thin plate four point bending test. According to the standard EN 12390-13^[171], the loading speed for cube compression test and cylinder elastic modulus test is set at 0.6 MPa/s. The loading speed for the dog bone tensile test is set to 0.05 mm/min, and the loading speed for the four point bending test is set to 0.1 mm/min. At the same time, extensometers are arranged on both sides of the cylindrical specimen and the dog bone tensile specimen to measure the displacement (strain) changes during the loading process. Displacement sensors are arranged above the four-point bending thin plate fulcrum position, at the loading point, and below the mid span to accurately measure the deflection changes of the plate. All experiments were conducted

using a WAW-500J microcomputer controlled electro-hydraulic servo universal testing machine for loading, with a maximum output load of 3000 kN. All test data is collected through a synchronous acquisition system, which synchronously records load, deflection, and strain data at a frequency of 5Hz.

In order to further understand the mix effectiveness of ultra-high performance concrete and provide microscopic experimental basis for subsequent multi-scale toughening mechanism analysis, the microstructure of UHPC was analyzed using Zeiss Gemini Sigma 300/VP SEM ultra-high resolution field emission scanning electron microscopy. The scanning electron microscope has a resolution of 0.8nm @ 15kV and a magnification of 12-2e6 times. All samples were extracted from the cross-section of the samples using scanning electron microscopy, and then coated with gold (Au) to improve the conductivity of the samples during the scanning process. The prepared UHPC sample is shown in Figure 3.7.



Figure 3.7 Gold plating treatment on the surface of the sample.

3.4.3 Test Results and Analysis

3.4.3.1 Electron microscopy scanning test results

Figure 3.8 shows the SEM morphology of UHPC microstructure magnified 56-12000 times. By observing the fiber distribution in the matrix of Figure 3.8 (a) and (b), it can be seen that there is no aggregation of fibers and they are well dispersed. The bonding strength between steel fibers and UHPC matrix is good, and there are no obvious $\text{Ca}(\text{OH})_2$ crystals at the fiber pull-out point, and the transition interface is not obvious. The diameter of the internal pores in UHPC is close to the diameter of the steel fiber (0.2mm), and there are no large pores, indicating good compactness and high density. Figure 3.8 (c) shows the local microstructure of UHPC magnified 400 times. Due to the chemical shrinkage of UHPC slurry during the initial hydration stage, shrinkage microcracks are generated on the pore surface. However, the size of microcracks is relatively small compared to

the fiber diameter and aggregate particle size, so the microscopic damage caused by shrinkage during the initial setting stage of UHPC can be ignored. Figures 3.8 (d) and (f) clearly show the morphology of the interface between fine aggregate and slurry (hydrated calcium silicate) in UHPC. The slurry and aggregate are tightly combined, generating mechanical interlocking force and forming a dense fine aggregate slurry bonding zone. At the same time, the presence of C-S-H (hydrated calcium silicate) also improves the bonding between steel fibers and matrix, enhancing the role of fibers as reinforcing materials in UHPC.

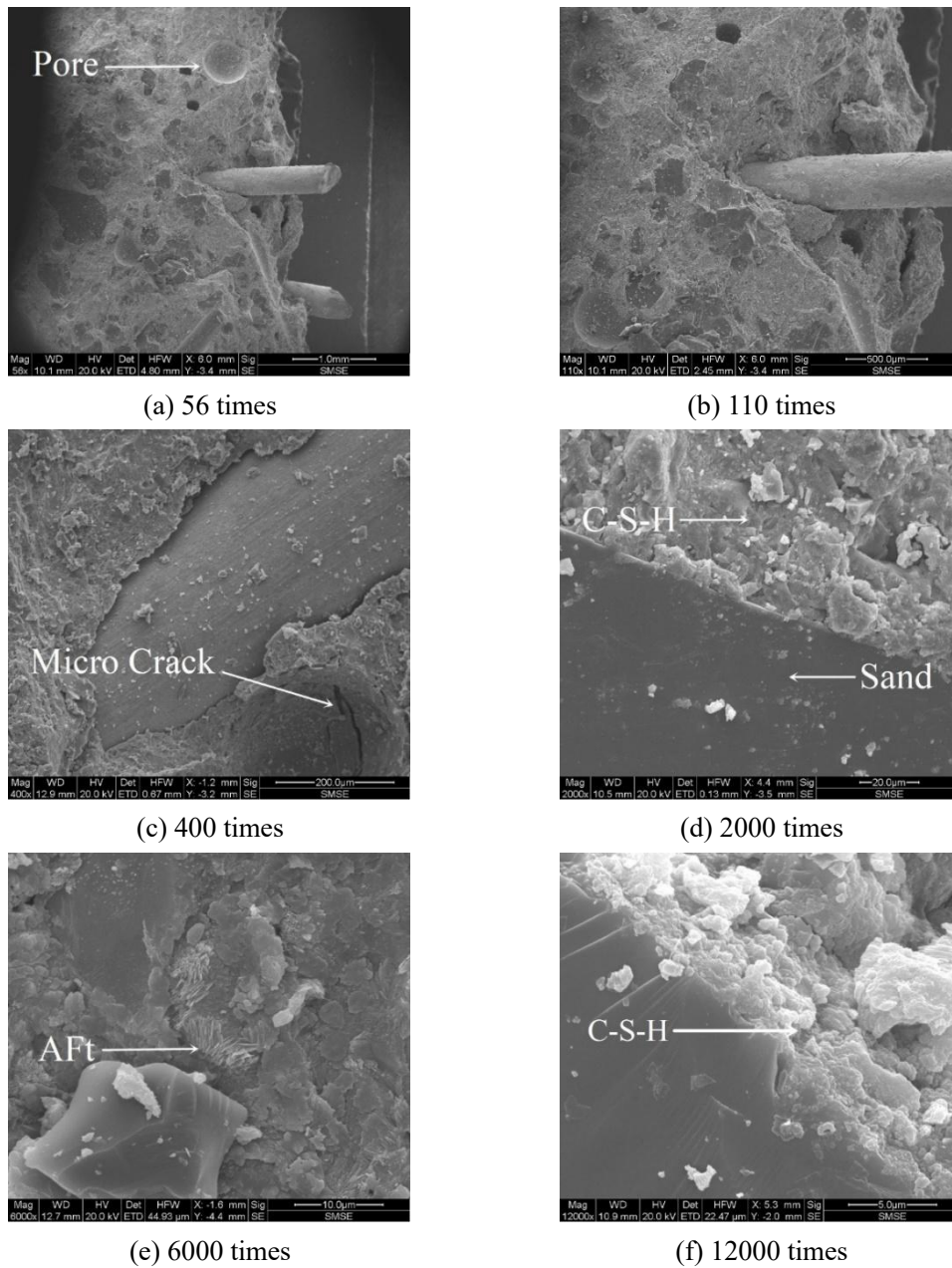


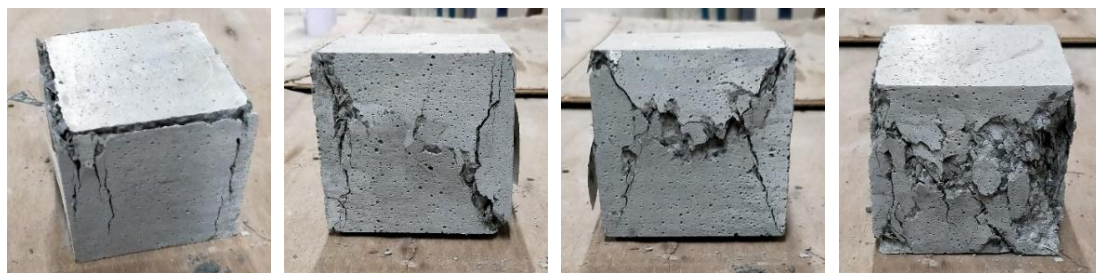
Figure 3.8 SEM image of UHPC microstructure.

Figure 3.8 (e) shows the crystal formed by the combination of cement hydration product calcium

aluminate hydrate and sulfate ions in UHPC - calcium sulfoaluminate hydrate (commonly known as ettringite, abbreviated as AFt). The chemical composition and crystal structure of AFt are basically the same as those of natural mineral ettringite. The growth rate and final length of ettringite formed by hydration in concrete are influenced by three factors: chemical factors, microstructure of the slurry, and microstructure of the concrete or mortar. Research has shown that the water cement ratio of UHPC has a significant impact on the growth of ettringite. Under the same conditions, the larger the water cement ratio of the slurry, the looser the microstructure of the cement stone, and the larger the space for the formation of ettringite. When the water cement ratio is small, the cement stone structure is more compact, the pores are smaller, and the amount of generated ettringite is less, so the crystals will undergo anisotropic growth and appear needle shaped.

3.4.3.2 Uniaxial compression test results

The failure modes of UHPC with different fiber volumes and coarse aggregates are shown in Figure 3.9. During the elastic deformation stage, no cracks appeared on the outer surfaces of the four specimens, but as the elastic ultimate strength approached, micro damage and micro cracks in the matrix began to initiate and converge. When the compressive stress exceeds the elastic stage, internal microcracks extend to the surface of the specimen, while macroscopic cracks begin to appear and propagate at the surface and corners, accompanied by slight cracking sounds, and then propagate diagonally, forming shear planes instead of vertical splitting planes. As the deformation increases, the wedge-shaped block will experience relative sliding along the shear plane, and due to the bridging and anchoring effects, steel fibers can limit excessive shear sliding.



(a) FA-1

(b) FA-2

(c) CA-0.75

(d) CA-1.5

Figure 3.9 UHPC Cube Compression Failure Mode.

As the compressive stress increases, the crack length and propagation area gradually increase. Comparing the damage and compression failure modes of four types of UHPC specimens, it was found that the cubic specimens containing coarse aggregates had a greater degree of ultimate

damage. This is because the presence of coarse aggregate causes uneven distribution of internal stress in ultra-high performance concrete during compression, and gradually forms stress concentration zones around the coarse aggregate, thereby promoting crack propagation and development, ultimately presenting a greater degree of fragmentation mode.

The compressive strength of UHPC cubes with four different mix proportions is shown in Figure 3.10. The experimental results indicate that an increase in steel fiber content will enhance the compressive strength of ultra-high performance concrete. The average compressive strength of UHPC with a fiber content of 2% is 144.3 MPa, which is 19.0% higher than the sample with a fiber content of 0.75% (121.3 MPa). It is worth mentioning that the average compressive strength of UHPC corresponding to the CA-1.5 mix ratio is 123.0 MPa, which is lower than the compressive strength of the sample without coarse aggregate (130.3 MPa) and with a fiber content of 1% (i.e. FA-1). This is because adding an appropriate amount of coarse aggregate can increase the risk of early shrinkage cracking, but when the critical value is exceeded, the ITZ damage area with lower strength between coarse aggregate and UHPC increases accordingly. The increase of initial microcracks and defects can lead to the development of cracks during loading, thereby affecting the safety of the structure. The standard deviations of the compressive strength of UHPC with four different mix proportions are 4.78 MPa, 2.42 MPa, 6.48 MPa, and 5.73 MPa, respectively, which are less than 5.5% of the corresponding average compressive strength values.

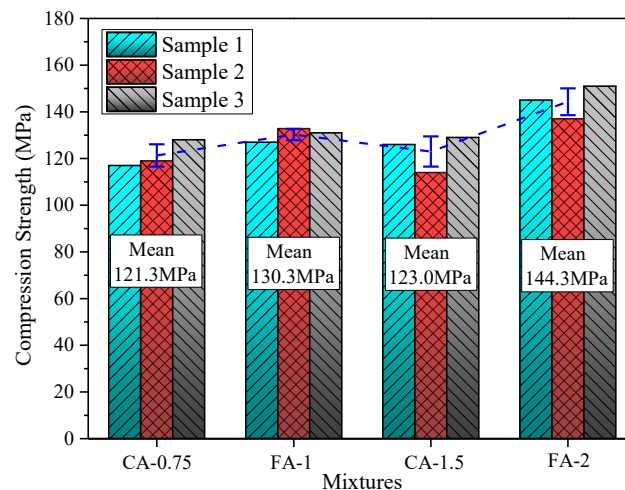


Figure 3.10 UHPC compressive strength results.

3.4.3.3 Elastic modulus test results

The modulus test takes the elastic modulus of the compressed section of UHPC under static action, generally taking the elastic modulus corresponding to 1/3 of the axial compressive strength. Record

the cyclic loading stress-strain curves for three cycles (preloading to 1/3 compressive strength), and then load at a constant loading rate until final failure. The compression failure modes of four UHPC cylinders with different mix proportions are shown in Figure 3.11.



(a) FA-1



(b) FA-2



(c) CA-0.75



(d) CA-1.5

Figure 3.11 Compression Failure Mode of UHPC Cylinder.

It can be clearly observed from the figure that the ultimate failure mode of the cylinder of fine aggregate UHPC presents an oblique single shear plane, while the cylinder failure of coarse aggregate UHPC presents multiple vertical cracks. The reason for this is that the fine aggregate UHPC has a dense microstructure inside, which can withstand external loads as a whole. Therefore, under the constraint of the upper and lower top surfaces, the internal shear stress of the specimen exceeds its allowable stress, resulting in oblique shear failure. Due to the presence of aggregate particles, a low strength interfacial transition zone (ITZ) is formed around the coarse aggregate UHPC. These ITZ are mostly composed of loose hydrated calcium silicate (C-S-H), and during the concrete setting and hardening process, many shrinkage microcracks or damages will be formed due to the shrinkage of the cement slurry, which will be dispersed throughout the entire cylindrical

specimen. When UHPC specimens are subjected to compressive loads, these low strength interface regions will first initiate microcracks. As the load increases, microcracks continue to develop and accumulate, ultimately leading to the failure of the cylindrical specimen and exhibiting a phenomenon of dispersed cracks at the macroscopic scale.

The main results of the cylindrical uniaxial compression test are summarized in Table 3.9. The results showed that the modulus of coarse aggregate UHPC was significantly higher than that of fine aggregate UHPC, and as the fiber content increased, the average elastic modulus of UHPC also increased. The Poisson's ratios of UHPC for fine and coarse aggregates are 0.18 and 0.20, respectively.

Table 3.9 Cylinder Modulus Test Results.

No.	Elastic modulus (MPa)				Poisson's ratio
	Sample 1	Sample 2	Sample 3	Average	
FA1	4.61e04	4.64e04	4.65e04	4.65e04	0.18
FA2	4.72e4	4.72e4	4.87e4	4.77e4	0.18
CA0.75	4.96e04	5.14e04	5.23e04	5.11e04	0.20
CA1.5	5.25e04	5.16e04	5.31e04	5.24e04	0.20

3.4.3.4 Uniaxial tensile test results

The fracture morphology of UHPC dog bone tensile specimens is shown in Figure 3.12. The figure shows that most of the fracture locations of the specimens are within the same cross-sectional area, with a small number of fracture locations occurring at variable cross-sectional areas. The ultimate failure of the specimens was the tensile fracture of a single main crack. Although small cracks appeared at other locations during the tensile process, their widths did not exceed 1 mm. The final cross-section was the result of the continuous development of a certain crack. No obvious diffuse cracks were observed. The fibers at the cross-section were all in the form of pull-out failure (Figure 3.13), and no pull-out occurred. The reason is that the gripping force at the interface between the matrix and fibers is insufficient to resist the stress of tensile cracking. The average ultimate tensile strength of UHPC with four different mix ratios of FA1, FA2, CA0.75, and CA1.5 is 8.52 MPa, 10.54 MPa, 5.83 MPa, and 9.40 MPa. The increase in fiber content significantly improved the tensile strength of UHPC.



(a) FA-1



(b) FA-2



(c) CA-0.75



(d) CA-1.5

Figure 3.12 Uniaxial tensile failure mode of UHPC.



(a)



(b)

Figure 3.13 Tensile failure surface.

The tensile stress-strain curves of UHPC with different fiber contents are shown in Figure 3.14. The stretching results show significant differences in the tensile properties of UHPC under different fiber contents. When the fiber content is low (fiber content $\leq 1\%$), as shown in Figure 3.14 (a) and (b), the tensile cracking failure of the specimen exhibits strain softening characteristics. This is due to insufficient bonding strength between the steel fibers at the crack location and the UHPC matrix interface. When the tensile load exceeds the peak bearing capacity, some fibers fail to bond

with the matrix and begin to experience relative slip, causing a sudden drop in bearing capacity to a certain value. Afterwards, another part of the fibers began to exert crack resistance, and the bearing capacity could be maintained at a certain value to continue to bear the load. As the number of bridging fibers at the crack initiation site decreases, the bearing capacity continuously decreases and eventually fractures occur.

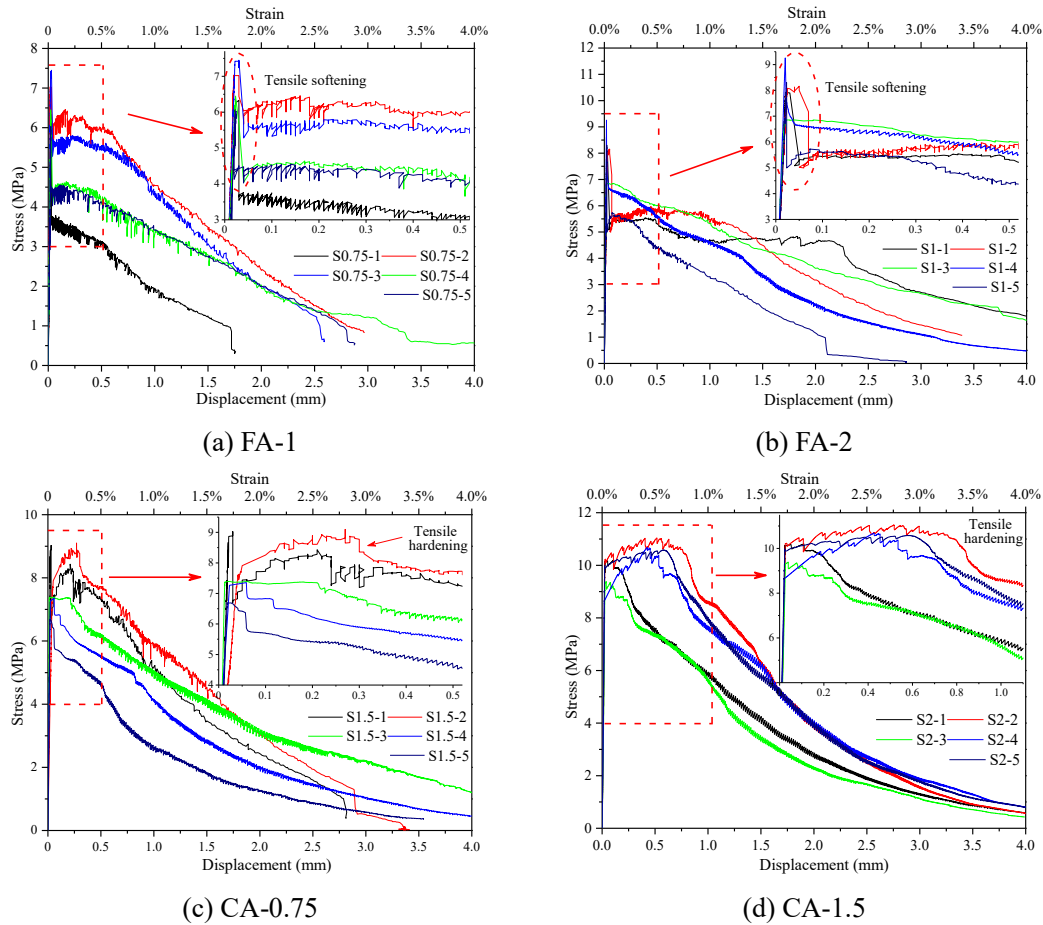


Figure 3.14 UHPC tensile stress-strain curve.

Figures 3.14 (c) and (d) show that most of the tensile curves of UHPC with fiber contents of 1.5% and 2% exhibit strain hardening characteristics. Strain hardening occurs after micro cracks appear in weak areas within the UHPC matrix. After entering the strain strengthening stage, the steel fibers distributed between microcracks begin to play a bridging role, and the stress between microcracks is transmitted through the steel fibers. The continuous extraction of steel fibers causes the curve to exhibit a "toothed" variation. The evolution of microcracks in strain hardened UHPC is shown in Figure 3.15. For strain strengthened UHPC, due to the fact that the energy required for new cracks generated during the strain strengthening process accompanied by multiple cracks is less than the energy required to penetrate existing cracks, the width of microcracks formed at the end of the

elastic stage does not continue to increase, but rather new microcracks form at other locations in the matrix. The strengthening stage continues until the bond strength of the steel fibers distributed between a certain crack inside the specimen reaches the limit value (the bond strength of the steel fibers cannot effectively restrain the growth of the crack width), and the specimen reaches the peak load. With the continuous sliding and pulling out of the steel fibers between the cracks, the stress of the specimen begins to decrease, the crack width continues to increase, and ultimately fails.

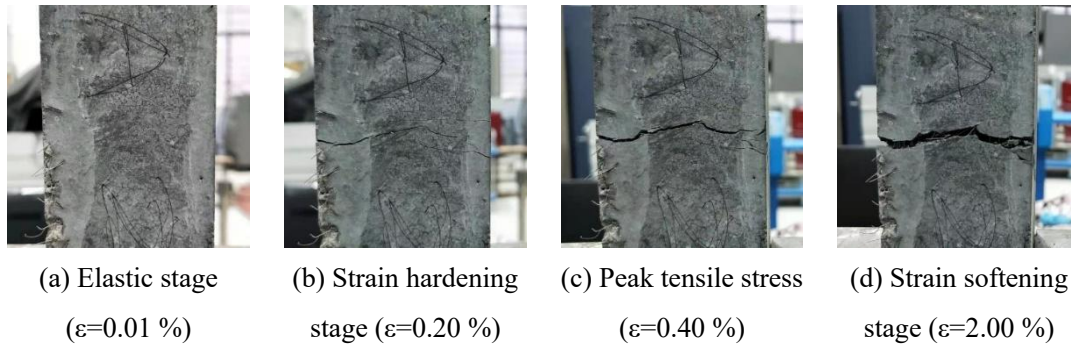


Figure 3.15 Crack Evolution of UHPC at Different Tensile Stages: (a) No Microcracks; (b) The maximum microcrack width is 1.11mm; (c) The maximum microcrack width is 2.35mm; (d) The maximum crack width is 6.48mm.

To analyze the mechanical properties of UHPC after tensile cracking under different fiber contents, the maximum tensile hardening stress/minimum tensile softening stress (P_c) within 0.5% strain after cracking was extracted and compared with the cracking load (P_k) to evaluate the degree of improvement of UHPC fiber content on tensile cracking performance. The calculation results of P_c/P_k of UHPC under different fiber contents are shown in Figure 3.16.

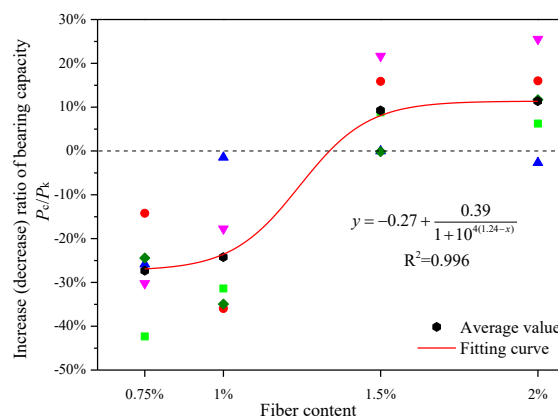


Figure 3.16 Increase (decrease) ratio of UHPC bearing capacity after tensile cracking.

As the fiber content increases, the P_c/P_k of UHPC changes from negative to positive after tensile cracking. The average P_c values and corresponding P_k ratios of 0.75%, 1.0%, 1.5%, and 2% are -

27.4%, -24.3%, 9.2%, and 11.3%, respectively. Nonlinear fitting was performed on P_c/P_k and fiber content, and the fitting equation is shown in Figure 3.16. It was found that the fiber content corresponding to $P_c/P_k=0$ is 1.33%. Therefore, it can be inferred that the critical fiber content for strain hardening and strain softening under tensile action of UHPC with a length to diameter ratio of 65 is 1.33%.

3.4.3.5 Four point bending test results

Bending performance is an important reference for evaluating the deformation of UHPC materials or structures under external loads. The bending failure results of UHPC thin plates under four different mix proportions are shown in Figure 3.17, and the bending strength results are listed in the Table 3.10.

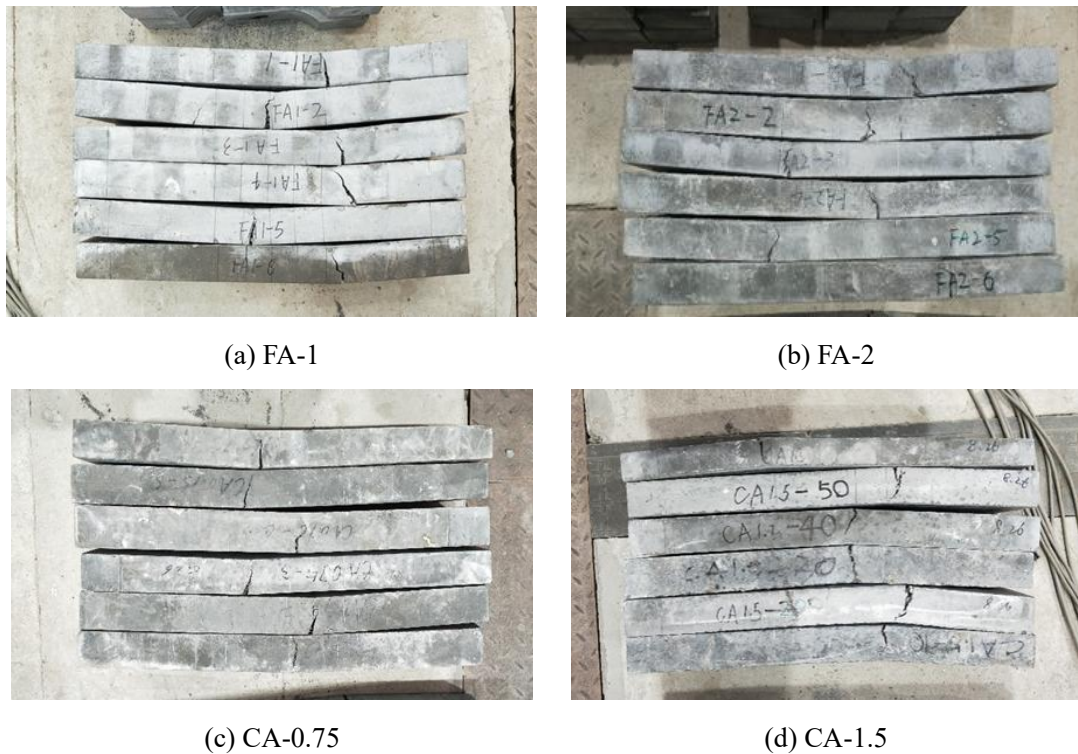


Figure 3.17 Four point bending fracture morphology of UHPC thin plate.

As shown in the figure, under the action of four point bending stress, the cracking position of UHPC board is basically within the pure bending area or near the loading point on one side. The average deflection corresponding to the peak load of the four mix proportions of UHPC boards is 1.4 mm, 1.8 mm, 1.2 mm, and 1.9 mm, respectively. The increase in fiber content not only enhances the bending strength of UHPC boards, but also increases their bending deformation capacity, improving their structural toughness and ductility index. After the UHPC board reaches the peak load during bending and unloading, the gap opening displacement slightly shrinks, and the gap

opening displacement of the UHPC board after unloading ranges from 2 mm to 7 mm.

Table 3.10 UHPC Thin Plate Bending Strength Results

No.	Initiation stress (MPa)	Peak stress (MPa)	Crack initiation stress/peak stress
FA-1	14.05	22.72	0.62
FA-2	20.85	30.45	0.68
CA-0.75	12.69	14.77	0.86
CA-1.5	21.02	24.76	0.85

3.5 Conclusion

This chapter designed four types of UHPC mix proportions based on the theory of ideal gradation curve, and conducted experimental research on the properties of the mixture and the mechanical properties after hardening. The main conclusions drawn are as follows:

(1) Designed and determined four localized material ratios for UHPC. Fine aggregate properties UHPC (FA-1 and FA-2) with fiber content of 1% and 2% respectively; Coarse aggregate UHPC (CA-1 and CA-2) with fiber content of 0.75% and 1.5%.

(2) The mechanical properties of UHPC after hardening are excellent, exhibiting good tensile strain hardening characteristics and toughening effects. Strong resistance to bending deformation, with good bending toughness and ductility characteristics.

(3) The micro electron microscopy scanning results of UHPC show that the slurry and aggregate are tightly bound, generating mechanical interlocking force and forming a dense fine aggregate slurry bonding zone. The presence of C-S-H also improves the bonding between steel fibers and matrix, enhancing the role of fibers as reinforcing phases in UHPC.

Chapter 4 Meso-scale Fiber Toughening Mechanism of UHPC in the Compression Process

4.1 Overview

The addition of steel fibers and coarse aggregates can greatly improve the toughness and compressive performance of UHPC, but it will also involve more interface transition zones (ITZ) of lower strength. This chapter combines UHPC compression tests and meso-scale random methods to reveal the compression damage evolution of UHPC components and the toughening and crack resistance mechanisms of fibers at the meso-scale through numerical simulations, the related research provides a comparison to the multi-scale calculation methods in Chapter 5. Specifically, by combining the concrete damage plasticity model (CDP model) and the microscopic random method, the two-dimensional microscopic compression damage process of four different mix proportions of UHPC was simulated. By comparing experimental and numerical results, the accuracy of the numerical analysis parameters and the feasibility of the research method were verified. The research results provide a theoretical analysis method for the safety assessment of existing UHPC structures and the optimization of UHPC materials and structures.

4.2 Meso-scale random numerical model of UHPC

UHPC, similar to concrete, belongs to cement-based materials. The macroscopic scale of concrete is considered uniform, but fundamentally, the interior of concrete is uneven. The mechanical behavior and durability characteristics of UHPC are highly dependent on the microstructure of concrete. The micro scale model is widely used to reveal the influence of concrete micro scale on its macroscopic mechanical properties and corresponding failure mechanisms. While considering the heterogeneous characteristics, it can accurately obtain the response of macroscopic homogenization, thereby providing a basis for revealing the damage failure mechanism of UHPC. At the microscopic level, concrete is considered a composite material composed of three phases: ITZ, coarse aggregate, and mortar matrix. For UHPC, fiber phase needs to be added. Due to the characteristics of porous gradient structure, the mechanical properties of ITZ differ significantly from those of mortar matrix. Therefore, it is important to separately model these phases in microscale models.

Microscopic scale simulation can be mainly divided into two methods: continuous microscopic

simulation and discrete microscopic simulation. Continuous medium micro modeling represents the microstructure of concrete as a continuous medium, mainly using finite element method for analysis. Discrete microscale modeling uses different elements (such as spheres, lattice beams, trusses, etc.) to represent concrete microscale structures, including lattice models, random particle models, M-H models, random mechanical property models, rigid spring element models, and random aggregate models [175].

The lattice model considers a continuous medium as a lattice system composed of discrete elastic rods or beam elements connected together. The lattice model is limited in its application due to the inability to consider the influence of smaller particles and the low computational efficiency in analyzing three-dimensional materials and structures. The random particle model assumes that concrete is composed of two phases of materials, aggregate and mortar, where the aggregate is spherical particles dispersed in the matrix. It is assumed that concrete is composed of triangular truss elements, and corresponding mechanical parameters are assigned to different phase elements. However, only axial forces were considered for the interaction relationship between particles, without considering interfacial shear behavior, so it cannot reflect the true failure process of concrete microstructure. The M-H model starts from the microscale and considers the randomness of aggregate particle distribution in the matrix and the randomness of mechanical properties of different components. It introduces fracture characteristics into concrete microscale elements, but this model cannot consider the macroscopic response of concrete under multiaxial stress. The stochastic mechanical property model utilizes statistical and numerical simulation methods to fully consider the heterogeneous characteristics of the spatial distribution and mechanical properties of each component material. By assigning a Weibull distribution function to the physical field, the discreteness of the microstructure can be reflected. However, this model does not take into account the randomness of the distribution of microscopic phases within concrete, and cannot explain the microscopic mechanisms of concrete damage failure and fiber toughening mechanisms under specific loads. The rigid body spring element method takes the centroid displacement of discrete elements as the basic unknown variable, approximates the displacement field of the real structure with segmented rigid body displacements, connects the elements with springs, and represents the internal stress of the structure with spring stress. Due to the fact that this method reflects the deformation process of concrete by the rigid displacement between elements, it is not suitable for

problems with complex stress and displacement boundary conditions. The random aggregate model first converts the Fuller grading curve into a two-dimensional aggregate grading curve and determines the amount of aggregate, and then generates micro aggregates in specific areas using the Monte Carlo random method. Unlike abstract stochastic mechanics models, random aggregate models belong to phenomenological models. With this idea, other microscopic phases such as voids, fibers, and ITZ in concrete can also be generated. This chapter uses microscopic random methods to generate a UHPC microscopic model that includes aggregates, steel fibers, ITZ, and matrix. Different microscopic phases are assigned different material properties, and the damage evolution during UHPC compression process and the toughening and crack resistance mechanism of fibers are studied.

4.2.1 Fiber generation

The total number of fibers can be calculated according to the geometric dimension and the volume fraction. The positions and directions of the fibers were assumed to be randomly distributed in a cube specimen. For the actual 3D space model of steel fiber concrete, the fiber volume ratio ρ_f , can be calculated by

$$\rho_f = V_f / V_0 \quad (4.1)$$

where V_f is the volume of steel fiber; V_0 is the volume of the whole concrete. Then, on the 2D plane model, the number of fibers N can be determined by,

$$N = \text{round} \left(\frac{W \times H \times \rho_f}{L \times d} \right) \quad (4.2)$$

where $\text{round}()$ is the rounding function; W and H are the width and height of the specimen; L and d are the length and diameter of the fiber.

The fiber generation adopted the MATLAB platform since the steel fibers are randomly dispersed in the concrete matrix, that is, the position and direction are random. Once the diameter, the length and the boundary information of the generation area of a fiber are given, a steel fiber distribution can be generated by the random function. During the generation process, judge sentences were used to judge and circulate the total number of fibers, whether the fibers overlap or intersect, and the minimum distance between any two fibers, until the requirements are met.

4.2.2 Generation of polygon aggregates

After the formation of the steel fibers, the next step is to generate aggregate particles within the gradation range consistent with the test. The maximum compactness gradation curve, i.e., the Fuller curve, proposed by B. Fuller is the most common 3D ideal gradation curve. Its expression is as follows:

$$P = \sqrt{D_0 / D_{\max}} \quad (4.3)$$

where D_0 is the diameter of a certain sieve; P is the proportion of aggregate of D_0 in total aggregate mass; and $D_{\max}$ is the maximum aggregate particle size.

The Walraven formula ^[176] established the relationship between the gradation and content of the aggregate in the space of the fiber reinforced concrete specimen and the area of the aggregate cut by the internal section, providing a theoretical basis for the establishment of a 2D numerical model. The Walraven formula assumes that all aggregates are spherical, and the aggregate gradation satisfies the Fuller gradation curve, as displayed in Eq.(4.3). The probability that any point in the given space is located in the aggregate smaller than radius D_x is assumed to be as follows:

$$P_c(D < D_0) = P_k \left(1.065 \left(\frac{D_0}{D_{\max}} \right)^{0.5} - 0.053 \left(\frac{D_0}{D_{\max}} \right)^4 - 0.012 \left(\frac{D_0}{D_{\max}} \right)^6 - 0.0045 \left(\frac{D_0}{D_{\max}} \right)^8 - 0.0025 \left(\frac{D_0}{D_{\max}} \right)^{10} \right) \quad (4.4)$$

where P_k is the volume ratio of the aggregate in concrete.

For a given particle gradation, the probability of aggregates with a diameter smaller than D_0 in the 2D plane can be calculated from the abovementioned Eq.(4.4). The cross-sectional area of each particle size and the number of particles can be calculated by using Eq.(4.5) and (4.6):

$$S_i = S_0 (P_c^i - P_c^{i-1}) \quad (4.5)$$

$$N_i = S_i / S_{i0} \quad (4.6)$$

where, S_0 represents the total area of concrete; S_i is the cross-sectional area of each particle size aggregate; S_{i0} indicates the area of a single aggregate of particle size; and N_i indicates the number of particles of each size of aggregate.

The specific process of generating the random distribution model of steel fibers and coarse aggregates can be described as follows: (1) The key parameters related to fibers and aggregate of a

given meso-scale numerical model were determined. (2) The endpoint coordinates and angle of the first fiber were randomly generated in the designated area at the beginning of circulation. (3) Whether the fiber overlaps and intersects with other fibers was determined. Steps (2) and (3) were repeated until the number of fibers met the requirements. (4) The fiber end point coordinate information was stored. (5) The generation of the first random aggregate in the designated area was started. (6) Whether the generated aggregate intersects any fiber and whether the spacing meets the requirements were determined. (7) The generation of aggregate particles was continued, and whether they intersect or embed with any fiber or aggregate was determined. Steps (5)–(7) were repeated. (8) Whether the total area of aggregate particles met the requirements was determined, and the coordinate information of aggregate boundary nodes was stored. The UHPC mesoscopic model generation process is displayed in Figure 4.1.

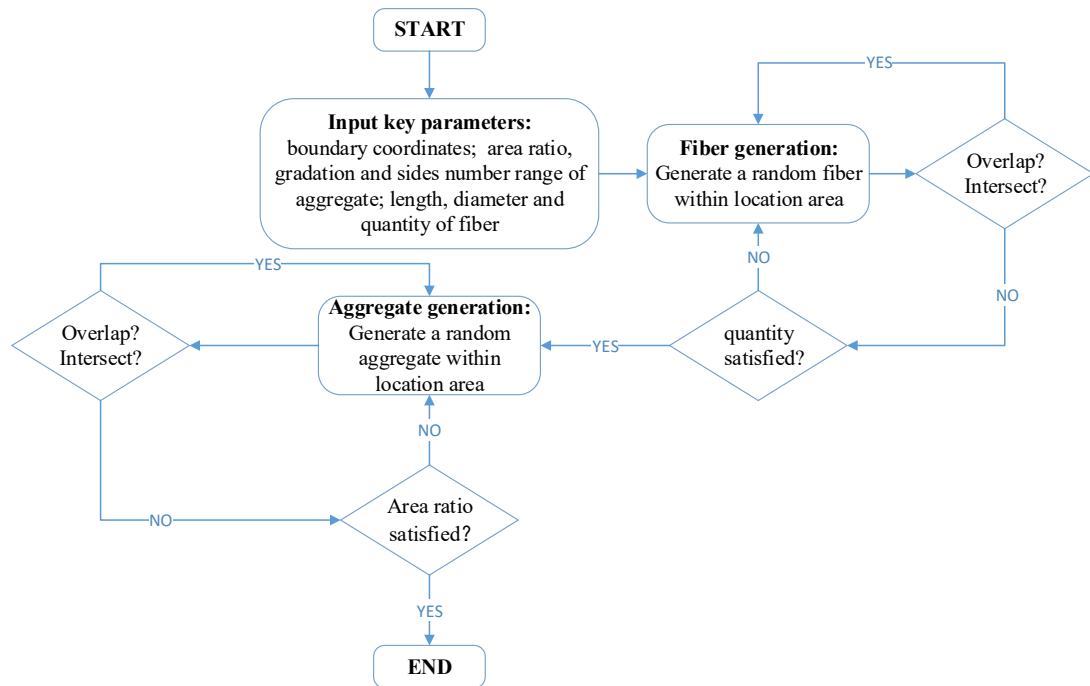


Figure 4.1 Process diagram of random aggregate and fiber generation.

4.3 UHPC microstructure material constitutive model

4.3.1 Mortar Matrix

In order to simulate the compression damage and failure process of UHPC at the microscopic level, it is necessary to establish a material model that considers the micromechanical behavior of different components. As there is currently no specific material model for UHPC, this study uses the Concrete Plastic Damage Model (CDP model) to simulate quasi-static compression of UHPC.

This model has been successfully used to simulate and analyze the dynamic response of ordinary reinforced concrete (RC) structures and the static performance of high-performance fiber-reinforced cement-based composite structural components [177]. Cao [178] and Lee [179] discussed in detail the analysis process and advantages of this model.

The CDP model is a multi axis stress coupling model based on damage plasticity, used to study the performance of concrete structures. The compression process considers strain hardening and strain softening behavior, while the tensile process only considers strain softening behavior. We adopted the non correlated plastic model with yield function proposed by Lubliner [180] and modified Lee's [179] model, while also incorporating the Drucker Prager hyperbolic function form of plastic potential with expansion angle. The scalar damage parameters (tensile and compressive separation) used in the stiffness degradation process are calculated based on tensile stress and compressive inelastic strain. Specifically, the yield function (F) and plastic potential (G) take the following forms:

$$F = \frac{1}{1-\alpha} \left(\bar{q} - 3\alpha\bar{p} + \beta \left(\tilde{\varepsilon}^{pl} \right) \left\langle \hat{\sigma}_{\max} \right\rangle - \gamma \left\langle -\hat{\sigma}_{\max} \right\rangle \right) - \bar{\sigma}_c \left(\tilde{\varepsilon}_c^{pl} \right) \leq 0 \quad (4.7)$$

$$G = \sqrt{\left(\xi \sigma_{t0} \tan \varphi \right)^2 + \bar{q}^2} - \bar{p} \tan \varphi \quad (4.8)$$

Among them, $\bar{q}$ is the von Mises equivalent stress, $\bar{p}$ is the equivalent hydrostatic pressure, and $\hat{\sigma}_{\max}$ is the maximum principal stress. The other terms are: $\bar{\sigma}_c \left(\tilde{\varepsilon}_c^{pl} \right)$ and $\bar{\sigma}_t \left(\tilde{\varepsilon}_t^{pl} \right)$ are equivalent compressive and tensile stresses corresponding to cumulative plastic strain $\tilde{\varepsilon}_c^{pl}$ and $\tilde{\varepsilon}_t^{pl}$; ξ is defined as the eccentricity of a curve approaching the asymptote; σ_{t0} is the uniaxial tensile failure stress of concrete; φ is the expansion angle under high confining pressure on the p - q plane. The other parameters α , β , and γ can be calculated by the following equation:

$$\alpha = \frac{\sigma_{b0} / \sigma_{c0} - 1}{2(\sigma_{b0} / \sigma_{c0}) - 1}; \quad 0 \leq \alpha \leq 0.5 \quad (4.9)$$

$$\beta = \frac{\bar{\sigma}_c \left(\tilde{\varepsilon}_c^{pl} \right)}{\bar{\sigma}_t \left(\tilde{\varepsilon}_t^{pl} \right)} (1 - \alpha) - (1 + \alpha) \quad (4.10)$$

$$\gamma = \frac{3(1 - K_c)}{2K_c - 1} \quad (4.11)$$

where, $\sigma_{b0} / \sigma_{c0}$ is the ratio of the initial biaxial compression yield stress to the initial uniaxial compression yield stress (default value 1.16), K_c is the ratio of the tensile meridian to the

compression meridian, and defines the shape of the yield surface in the deviation plane (default value 2/3).

4.3.2 Constitutive model for ITZ

As mentioned above, the formation of ITZ is caused by the wall effect, which accumulates smaller cement particles near the larger aggregate particles and produces a considerable porosity area near the transition zone. For UHPC, due to the low water binder ratio and the addition of ultra-fine silica fume, quartz powder, and other auxiliary cementitious materials, the filling density inside the matrix is greatly improved. Therefore, as a part of the stiffness of mortar matrix, the stiffness of interface layer is higher than that of NSC. ITZ is a potential cracking zone in concrete, and the damage of concrete often starts here. In this study, the cohesive element was used to simulate ITZ. The zero-thickness bonding element was inserted between the mortar and aggregate and also between mortar and steel fiber. The cohesive element typically used the traction-separation law to express its damage and fracture process, as demonstrated in Figure 4.2. In the rising part of linear elasticity, peak traction occurred when damage occurs, and the interface softened when damage developed.

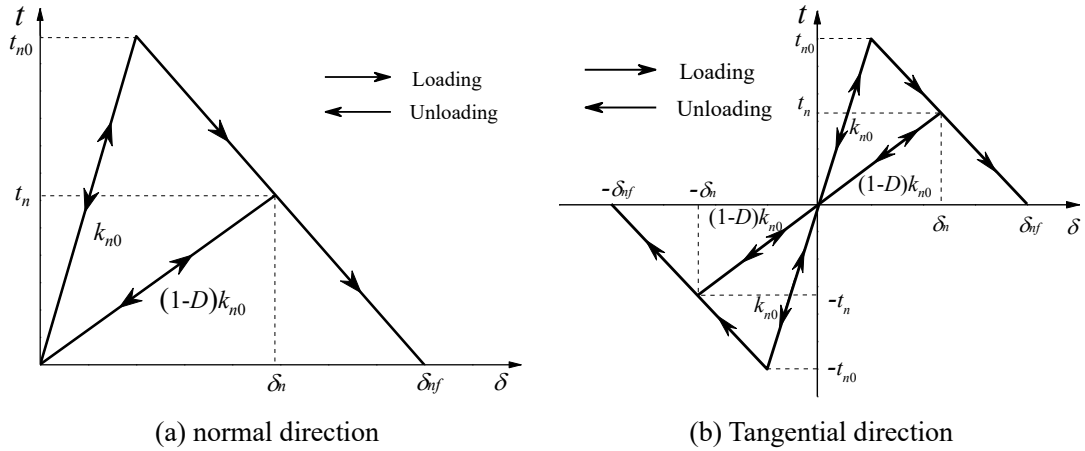


Figure 4.2 Traction-Separation curve of cohesive element

The mechanical behavior of 2D viscous element can be expressed by Eq.(4.12) or (4.13), as follows:

$$\mathbf{t} = \mathbf{K}\delta \quad (4.12)$$

$$\begin{bmatrix} t_n \\ t_s \end{bmatrix} = \begin{bmatrix} k_n & 0 \\ 0 & k_s \end{bmatrix} \begin{bmatrix} \delta_n \\ \delta_s \end{bmatrix} \quad (4.13)$$

where the discontinuity traction $\mathbf{t}$ is related to the separation displacement δ via the stiffness matrix $\mathbf{K}$; t_n and t_s are the normal and tangential traction stresses corresponding to the relative

displacements δ_n and δ_s for a given load increment; the stiffness k_n, k_s adopts the initial stiffness values of k_{n0} and k_{s0} before cracking, respectively. The selection of an appropriate value for the initial stiffness of the interface element is not simple. It must be sufficiently high to prevent excessive deformation of the viscous element in the elastic state. On the other hand, if the initial stiffness value is too high, it will lead to the morbidity of numerical problems and traction vibration before reaching the maximum value.

The damage initiation criterion and damage evolution law can be used to describe the degradation of the bonding element. The damage initiation is characterized by the damage initiation criterion. The damage criteria describing the cohesive element are as follows: quadratic nominal stress (strain) criterion, the maximum nominal stress (strain) criterion, and others. In this study, the cohesive element used the maximum nominal stress criterion demonstrated in Eq.(4.14) for constitutive control, as follows:

$$\max \left\{ \frac{\langle t_n \rangle}{t_{n0}}, \frac{\langle t_s \rangle}{t_{s0}} \right\} = 1 \quad (4.14)$$

where t_{n0} and t_{s0} represents the peak bond stresses in normal and tangential directions, respectively and $\langle \rangle$ represents the Macaulay bracket.

After damage occurs, the law of damage evolution clarifies the development process of damage. This suggests how the material stiffness is reduced. The normal and shear stiffness k_n, k_s are influenced by the scalar damage variable D according to:

$$k_n = (1-D)k_{n0}; \quad k_s = (1-D)k_{s0} \quad (4.15)$$

where scalar damage variable D is used to specify the overall damage of the element, which can be calculated by the following Eq.(4.16):

$$D = \frac{\delta_{nf}(\delta_{n,\max} - \delta_{n0})}{\delta_{n,\max}(\delta_{nf} - \delta_{n0})} \quad (4.16)$$

Where $\delta_{n,\max}$ is the maximum effective relative displacement in the loading process of cohesive element; δ_{n0} and δ_{nf} are effective relative displacements of damage initiation and complete failure, respectively. Under monotonic loading, The damage variable D develops from 0 to 1, where 0 means no damage exists in the material, and 1 means the material is completely damaged. The stress component is affected by the damage variable and is given by Eq.(4.17) and (4.18), as follows:

$$t_n = \begin{cases} (1-D)t_{n0}, & t_{n0} \geq 0 \\ t_{n0} & otherwise \end{cases} \quad (4.17)$$

$$t_s = (1-D)t_{s0} \quad (4.18)$$

4.3.3 Constitutive model for aggregates

For normal strength concrete, the strength of the internal coarse aggregate is often much higher than that of the mortar matrix. Damage and macroscopic cracks under load tend to develop along the ITZ or matrix with lower strength. Therefore, aggregates are usually treated as linear elastic materials in NSC. However, for UHPC, the increasing bulk density and the decreasing water cement ratio greatly improve the strength of matrix and ITZ, thereby expanding the cracks through the aggregate. The linear elastic constitutive model is obviously no longer applicable to the coarse aggregate in UHPC. In this study, the damage of coarse aggregate is also considered, and the CDP model is given to describe the failure characteristics during the compressive process ^[181].

4.3.4 Constitutive model for steel fibers

To reflect the real damage and failure process of UHPC, the constitutive properties of steel fiber in the meso-scale numerical model was defined as elastoplastic with the yield strength of 235 MPa. Table 4.1 summarizes the four related bond material parameters used by UHPC in the analysis. The parameters for mortar, aggregates, and ITZ were taken from the research results of Kristombu Baduge ^[182], and the steel fiber properties were taken from the literature ^[183].

Table 4.1 Material parameters for the UHPC numerical models.

Properties	Mortar	Aggregate	Steel Fiber	ITZ
Elastic Modulus (GPa)	45	90	200	32
Poisson's ratio	0.2	0.25	0.3	0.2
Compressive strength (MPa)	150	250	2850	120
Tensile Strength (MPa)	6	13	2850	48
Fracture Energy (N/mm)	-	-	-	0.015

4.4 Establishment of meso-scale model

As a composite material, UHPC can be divided into four phases on the meso-scale, including the mortar, the steel fibers, the coarse aggregates and the ITZ between the mortar and the fiber/aggregate interface. The coordinate information of the randomly distributed steel fibers and coarse aggregates can be generated with the assistance of MATLAB. Then, the distribution and location of the rest

two components of the mortar and the ITZ could be determined. In this research, the random distribution model was analyzed by ABAQUS. As Figure 5.3 demonstrates, elements of the mortar, the steel fiber, and the coarse aggregate were defined as 3-node continuous plane stress elements (CPS3), and the ITZ were defined as 4-node 2D cohesive elements (COH2D4). Python programming was used to realize the embedding of 0 thickness cohesive element.

The numerical analysis was conducted on 24 groups of specimens with various coarse aggregates and steel fiber contents, which are summarized in Table 5.2. UHPC compression simulation was solved by ABAQUS's Explicit/dynamic method, and loading was controlled by displacement. The displacement load was applied to the upper reference point (RP) and transmitted to the UHPC specimen through the analytical rigid body coupled with the reference point. The lower reference point was fully constrained and coupled with the lower rigid body. The tangential direction between the rigid body and the UHPC specimen surface was defined as frictional contact, with a coefficient of friction of 0.2 and the normal direction as hard contact. In the quasi-static analysis, the lowest mode of the structure dominates the response of the structure. After the fundamental frequency of the structure is solved by the ABAQUS, the time required for appropriate quasi-static analysis can be obtained. The final optimal solution time is 0.1s, and under this solution time setting, the kinetic energy output of the model system is less than 5% of the internal energy. The specimens are named in the form of CAm-Fn, where m represents the content of coarse aggregate, and n represents the volume ratio of steel fiber.

Table 4.2 Summary of the numerical specimens.

Coarse aggregate content (CA, %)	0, 10, 20, 30
Steel fiber content (F, %)	0.5, 0.75, 1.0, 1.5, 2.0, 2.5

4.5 Analysis of Fiber toughening mechanism

4.5.1 Comparison and verification between experimental and numerical results

The compression damage distribution nephogram of the cubic compression mesoscopic numerical simulation results with the four mix proportions are displayed in Figure 4.4, in which blue area represents the area without damage and the red represents complete damage. Under the compressive loading, the microscopic damage and the microscopic cracks in the matrix begin to sprout and expand. The cracks at the top corners and the middle of the sample penetrate to form an

oblique macroscopic shear surface along with the continuous failure of the sample, as shown in Figure 5.4(a1), (b1), (c1), and (d1).

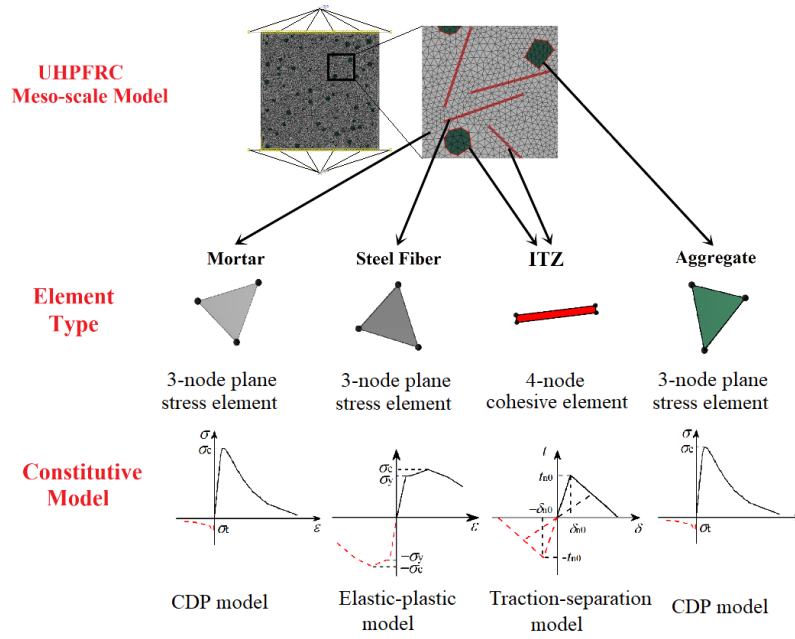


Figure 4.3 UHPC microscopic numerical model and constitutive models of various microscopic phases.

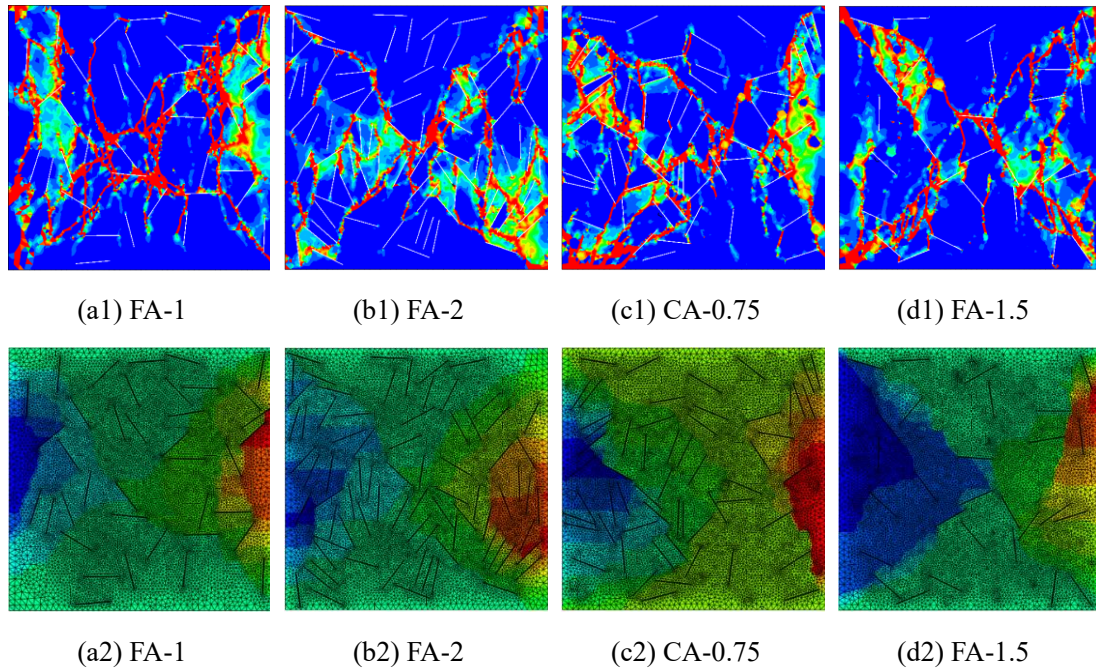


Figure 4.4 Numerical model results of UHPC with different mix proportions.

Compared with Figure 3.7, it is found that the compression shear expansion failure modes of the numerical simulation damage distribution results is highly consistent with the test results, which verifies the correctness of the numerical simulation method in this study. When damage occurs, the

ITZ area with lower strength damage first. With the increase of the load, the discontinuous damage area in the specimen expand gradually and form an oblique main shear damage plane. The modeling results of the compression damage and the final horizontal displacement nephogram of various proportions in Figure 4.4 (a2), (b2), (c2), (d2) show that the displacement boundary of the numerical model is consistent with the crack distribution of the experimental results. Additionally, the main shear plane direction of compression failure is also coincident, which also verifies the feasibility of the random distribution simulation method.

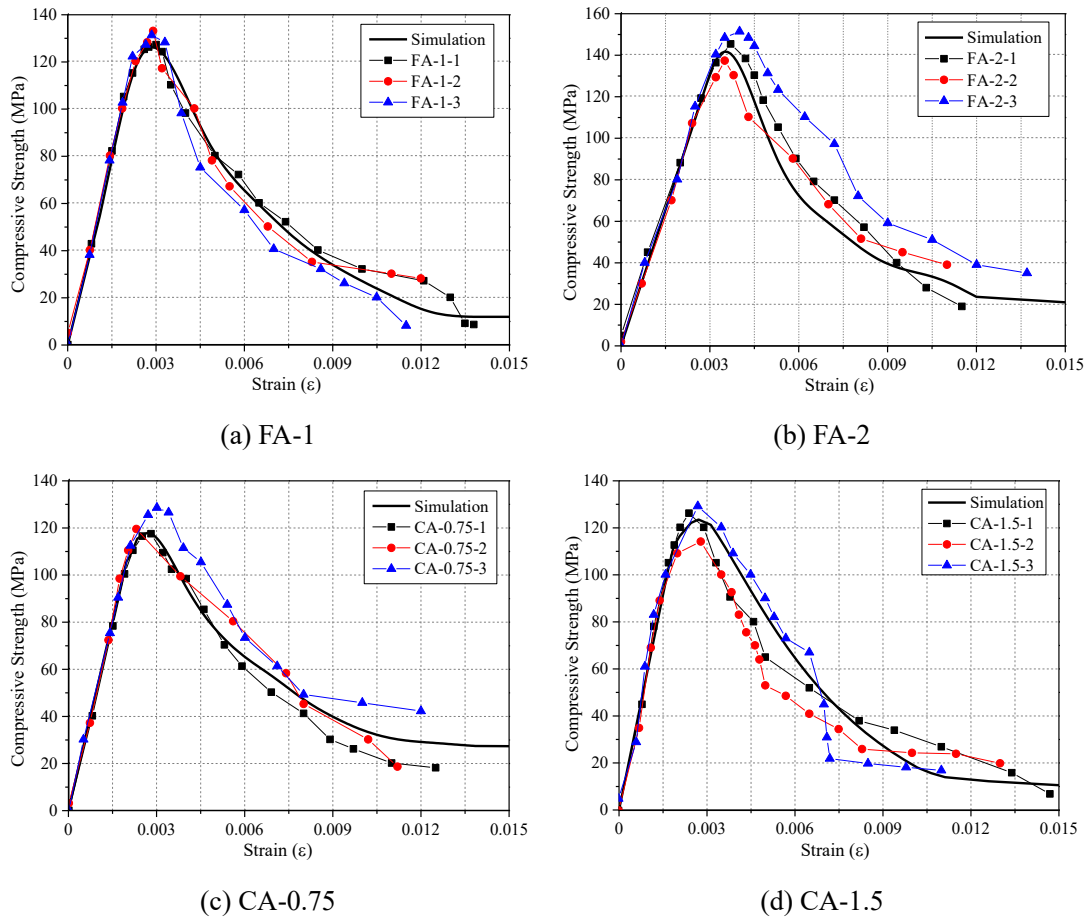


Figure 4.5 Stress strain curve of UHPC cube under uniaxial compression.

The compressive stress–strain curves of proportions FA-1, FA-2, CA-0.75, and CA-1.5 are plotted in Figure 4.5. Before the stress reached 80% of the peak value, the compressive stress–strain curves are linear. With the growing loads, the specimens start to damage and crack while the steel fibers begin to play a bridging role. As the cracks grow, the slope of the compressive stress–strain curves keep decreasing and the load gradually reaches the peak value. When the deformation of a specimen exceeds the peak strain, the bearing capacity of the specimen falls. As the axial deformation further increases, an increasing number of the steel fibers pull out from the matrix, and eventually, the

bearing capacity loses. In addition, it can be seen from Figure 4.5 that the experimental results of stress-strain curve are consistent with the numerical simulation results. The average test results of peak loads of FA-1, FA-2, CA-0.75, and CA-1.5 differed by 3.2%, 1.8%, 2.8%, and 0.4% with the corresponding numerical simulation results, respectively, which verifies the correctness of parameter selection of the mesoscopic numerical model.

4.5.2 Influence of aggregate and fiber content

Figure 4.6 displays the changes in the peak compressive stress of the UHPC meso-model with different fiber and CA contents. In the case of the same CA content, as the volume fraction of steel fibers increase, the compressive strength tends to increase. When the coarse aggregate content changes from 0%, 10%, 20% to 30%, the steel fiber content increases from 0.5% to 2.5%, and the compressive strength increases by 19.8%, 22.6%, 10.8%, and 11.9%, respectively. The addition of the steel fiber improves the performance degradation of the matrix due to the initiation and propagation of micro cracks, and greatly enhance the toughness and ductility of UHPC after cracking. When the steel fibers content remains constant, the compressive strength first increases and then decreases with the coarse aggregates content increases. An appropriate amount of coarse aggregates with great rigidity could produce a good particle skeleton effect in the matrix, and the strength significantly increases compared with the UHPC without coarse aggregates. With the increasing amount of coarse aggregates, the ITZ with lower internal strength gradually increases, which promotes damage and destruction and causes the decrease in the compressive strength.

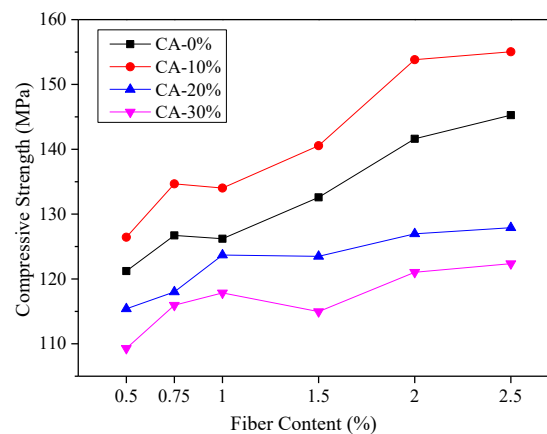


Figure 4.6 Compressive strength of UHPC under different aggregate and fiber contents.

4.5.3 Damage evolution and failure modes under compression

The evolution of compression damage was analysed. Taking CA0-F1.0 as an example, Figure 5.7

illustrates the stress–strain curve and the curves of damage ratio in mortar and ITZ. Based on the damage ratio curves, the entire process of uniaxial compression can be divided into four stages, that is the elastic non-damage stage, the initial damage stage, the accelerated damage stage and the residual stage. The stress–strain curve in the elastic non-damage stage demonstrates a linear change. The materials of each phase in UHPC are in an elastic compression state, and no damage occurs. After exceeding the proportional limit, the ITZ with weak strength as well as a small part of the mortar damaged. The proportion of damage is local distributed and small, the stress is close to the peak stress. When the compressive strain exceeds the strain corresponding to the peak stress, the compression specimen enters the third stage, the accelerated damage stage. A large amount of ITZ and mortar damage, the macroscopic cracks continue to expand and penetrate, and a large number of fibers across the cracks play a role in toughening and preventing cracks. As the stress gradually decreases, the ITZ damage ratio comes to be stable, the ITZ damage ratio hardly increases, and the mortar damage ratio reaches about 80% of the final damage ratio. The final stage that is the residual stage. The UHPC specimens at this stage lost the bearing capacity. The stress change is small, and most of the steel fibers near the main shear surface are deboned and pulled out. A small part of the mortar continues to damage due to the dislocation of the shear surface until the loading process completed.

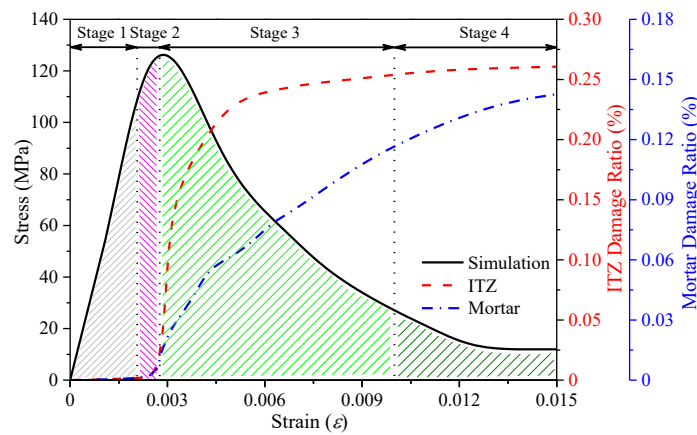


Figure 4.7 Compression stress-strain curve and damage stage division of UHPC.

Figure 4.8 shows the evolution of the mortar damage ratio under compression failure with different coarse aggregates and the fiber contents are essentially consistent. In the non-damaged elastic stage, the damage does not accumulate and the stress of mesoscopic phases are equal. After entering the initial damage stage, some part of elements initiate damage, the elastic modulus of the

element decrease, and the overall stiffness begin to decrease as part of the mortar element reaches the damage threshold. Due to the small number of damaged elements in mortar, the stress continues to increase while the growth rate slows down until reaching the accelerated damage stage. At this stage, a large number of elements rapidly damage. The proportion of mortar damage increases, the overall stiffness rapidly decreases, and the compressive stress decreases at the highest rate. The bearing capacity of the specimen rapidly losses. Upon entering the residual stage, the growth rate of the damage ratio of the mortar element slows down and gradually reaches a stable level. The specimen shows a large number of cracks that develop and penetrate, which is also revealed via macroscopic view. The bearing capacity and compressive load maintain constant since the connection of the steel fibers.

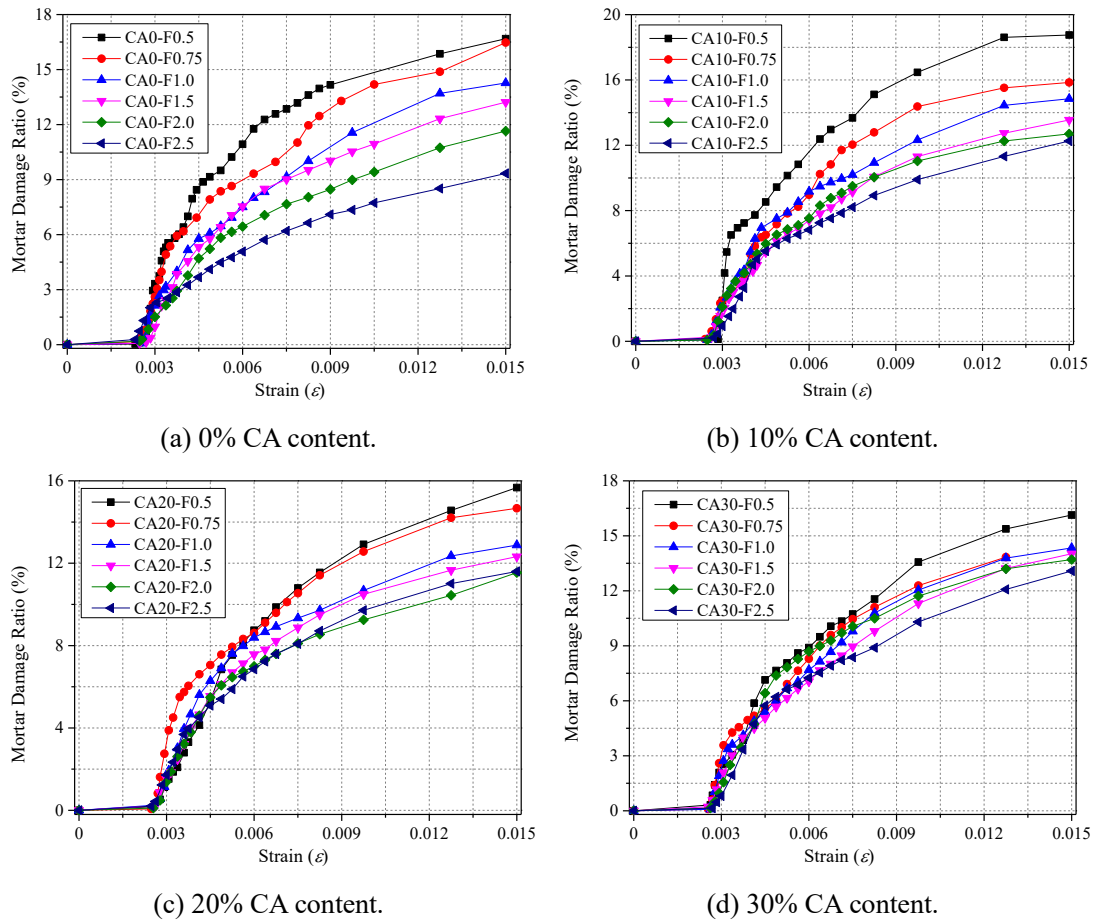


Figure 4.8 Changes in mortar unit damage ratio during compression of UHPC with different aggregate and fiber contents.

Figure 4.9 demonstrates the change of ITZ damage ratio under compression. The figure reveals an evident difference from that of mortar damage (Figure 4.8). In the elastic and the initial damage stages, the damage ratio of the ITZ elements changes in the same way as the damage ratio of the

mortar element, and no damage occurs in the elastic phase. In the initial damage stage, the damage first occurs in the low-strength ITZ region. While in the mesoscopic model, the initial damage also generally occurs in the ITZ. Although the material properties of ITZ are the same, the latter initial damage is mainly caused by stress concentration around the aggregate and the end of fiber (Figure 4.10), and then propagates along the ITZ or the matrix. When the compressive stress in the specimen is close to the peak stress, the specimen reaches the accelerated damage stage. At this time, a large number of ITZ elements destroyed. The damage rate of ITZ elements was much higher than that of mortar elements. Under the same strain, the damage ratio is greater in the ITZ than that in the mortar matrix. With further increase of compression strain, the growth rate of the damaged ITZ elements begin to decrease, because the micro cracks in the compression process of the specimen have formed a macroscopic shear band. After the stiffness of the specimen decrease, the unloading behavior occurs, therefore, in the ITZ region, the growth rate of the damage element gradually decrease and reach a constant value.

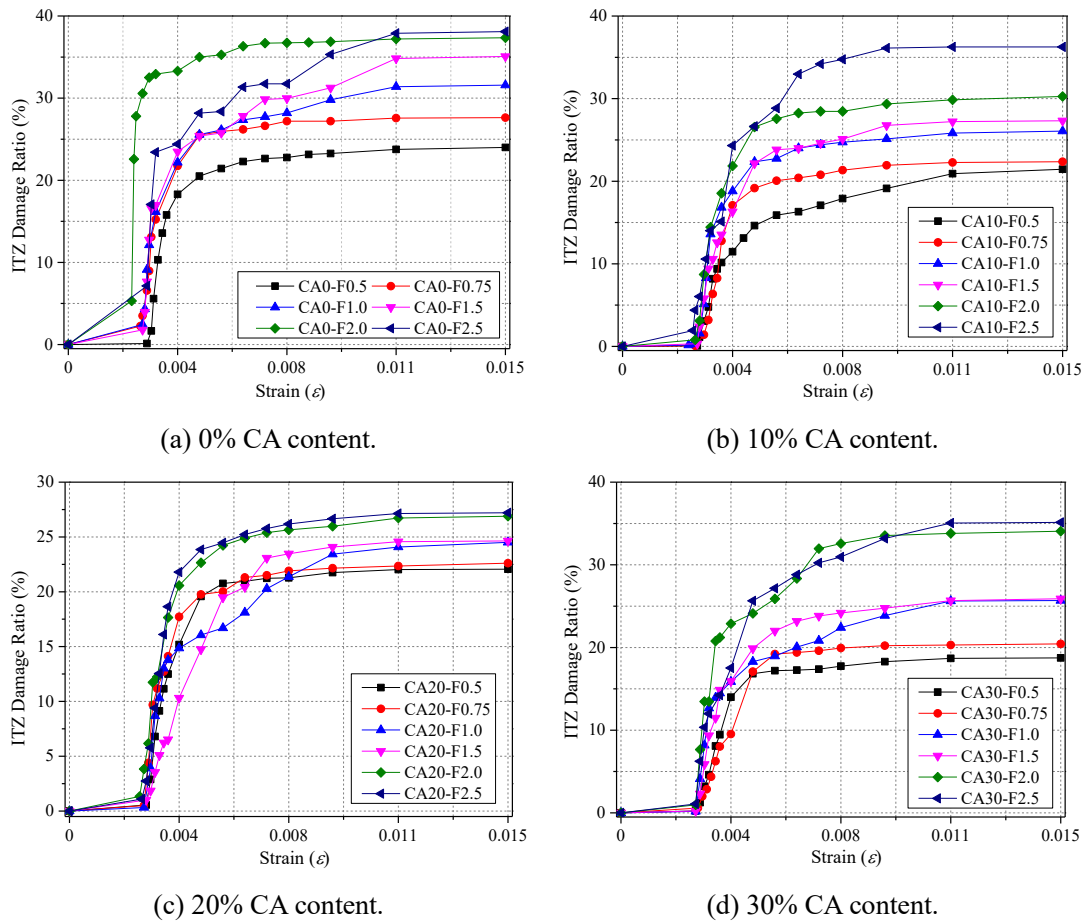


Figure 4.9 Changes in ITZ damage ratio during compression of UHPC with different aggregate and fiber contents.

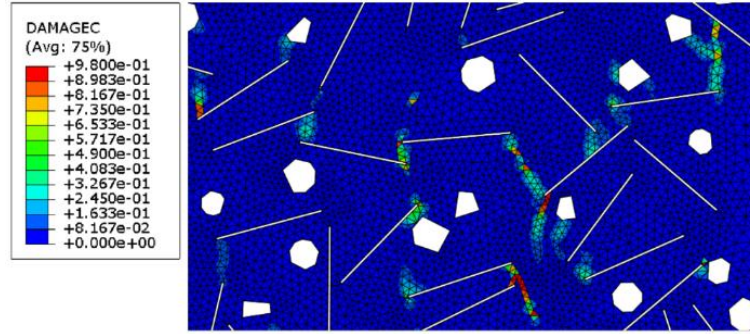


Figure 4.10 Initial compression damage caused by stress concentration around the aggregate and fiber ends.

Figure 4.11 displays the damage ratio in coarse aggregates. The figure indicates a certain difference between the CA damage ratio curves and the mortar/ITZ damage ratio curves. In the elastic stage, the aggregates were not damaged. After entering the initial damage stage, the ITZ elements and a part of the mortar elements damaged first. With the expansion of micro cracks, a small amount of aggregates occurred damage. The force is uniform throughout the compression process, because the coarse aggregate formed a good spatial framework system inside the UHPC. In the accelerated damage stage and the residual stage, the growth rate of the damage ratio in aggregates is even distributed, indicating that the skeleton function of the coarse aggregates could enhance the crack resistance throughout the loading process, especially in the post-cracking stage.

Figure 4.12 reveals the ultimate compression damage ratio of each phases of UHPC. With the increasing fiber content, the damage ratios of different phase materials present different change trends, but the change trend of the same phase material remain the same. Figure 4.12(a) indicates that the damage ratio trend of the mortar continues to decrease with increasing fiber content. With the increasing fiber content, the ability to bridge cracks is enhanced, thereby reducing the damage rate of the mortar. When the fiber contents increase from 0.5% to 2.5%, the damage ratio of the mortar with 0%, 10%, 20%, and 30% CA contents reduce by 7.3%, 6.5%, 4.1%, and 3.0%, respectively. As demonstrated in Figure 4.12(b), the ratio of ITZ damage continues to increase with increasing fiber contents, which is the opposite to the change of mortar damage in Figure 4.12(a). This caused by the difference of modulus between the fiber and ITZ and between the fiber and mortar. As a result, the damage of the ITZ element at the interface of the coarse aggregate and the fiber with mortar matrix increase during compression deformation. When the fiber content increases from 0.5% to 2.5%, the ITZ damage of the UHPC with 0%, 10%, 20%, and 30% CA content reduce

by 14.1%, 14.8%, 5.1%, and 16.38%, respectively. Figure 4.12(c) shows the final damage ratio of coarse aggregate in UHPC with the different fiber and CA contents. As the fiber content increases, the damage ratio of coarse aggregates increase first and then decrease.

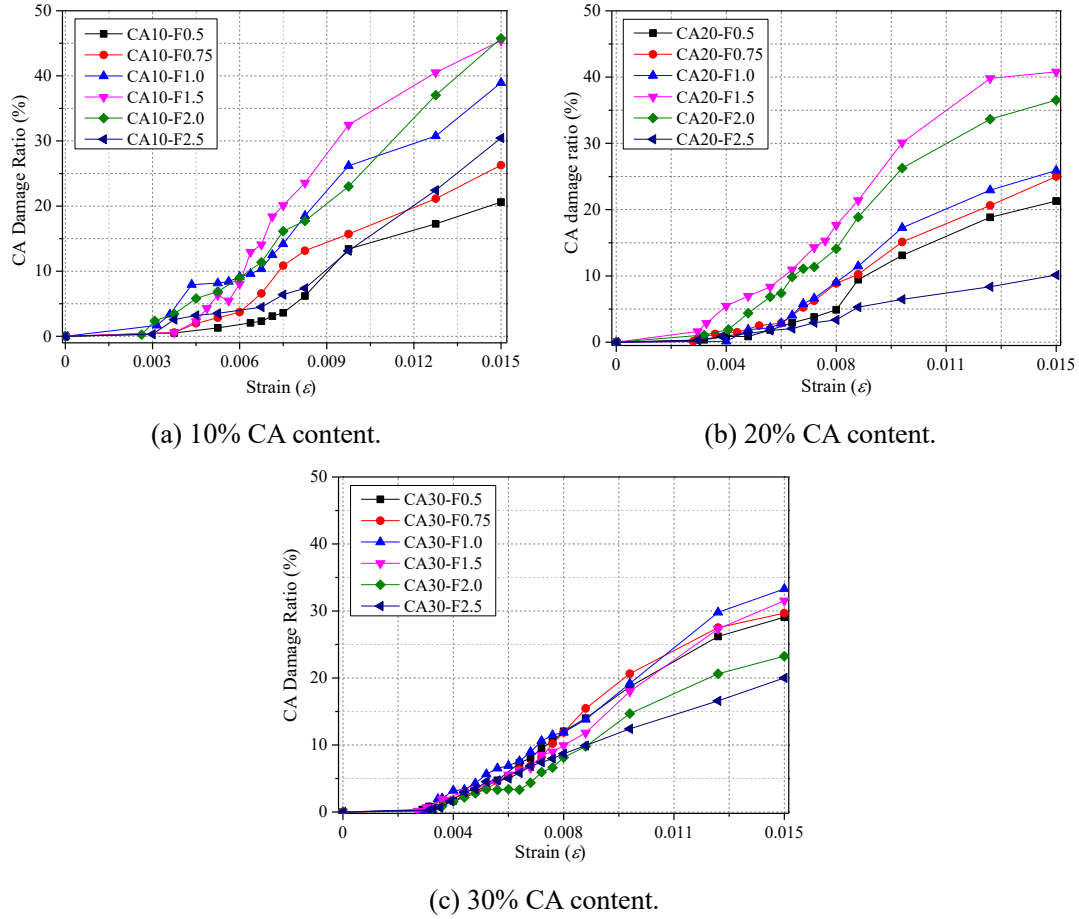


Figure 4.11 Changes in aggregate damage ratio during compression of UHPC with different aggregate and fiber contents.

The steel fiber content corresponding to the peak value of coarse aggregate damage amount is different. When the CA content increases from 10% to 30%, the maximum damage of the aggregate is 45.7%, 40.8%, and 33.3%, and the corresponding steel fiber content is 2%, 1.5%, and 1%, respectively. With the coarse aggregate content increases, a continuous particle framework system formed inside UHPC. If the fiber content also increases, then the fibers and coarse aggregates would interlock and agglomerate. Therefore, the reinforcement and crack resistance effect of aggregate cannot brought into full play, and the damage of aggregate would reduce accordingly. On the contrary, if the fiber content is small, then more coarse aggregate can exert an enhanced crack resistance effect, and the magnitude of damage would increase accordingly.

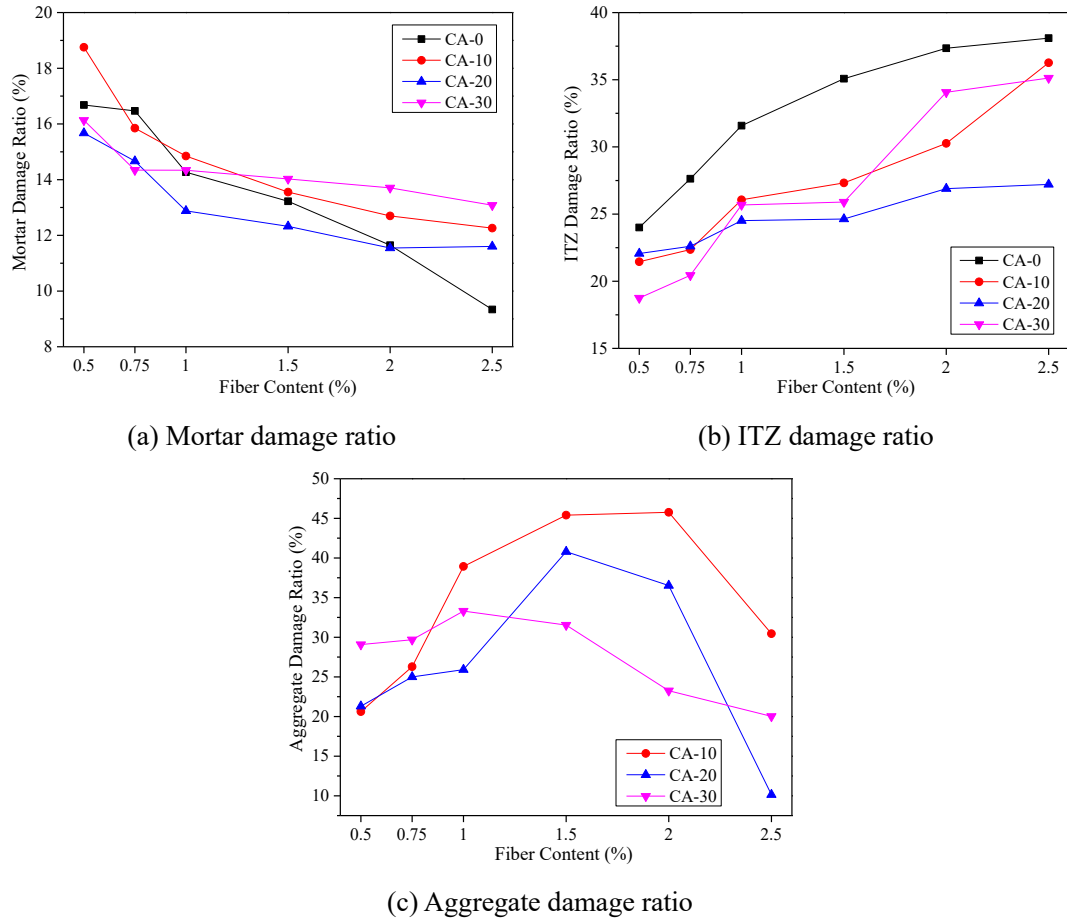


Figure 4.12 Damage ratio of different phase materials in the final compression state of UHPC.

4.5.4 Influence of the fiber orientation on toughness

Under the compression loads, the mortar and aggregate elements underwent a compression failure process while along with the tensile failure. Figure 4.13 displays the compression and tensile damage distribution of the mortar element during the compression process with a steel fiber content of 2%. After the load exceeds the elastic stage, the mortar at both ends of the fiber begin to produce tensile damage due to the difference in Young's modulus, and then, compression damage begins to occur and expand. However, the development rate of tensile damage is higher than that of the compression damage, as shown in Figure 4.13 (a1) and (a2). This phenomenon is due to the stress concentration at the fiber ends during compression deformation, leading to initial damage to the nearby mortar. The element failure is due to the transverse tensile stress of vertical compression, which causes cracks expanding in the up and down directions. As the load continues to increase, the stress release of the damaged element cause the stress redistribution in the surrounding elements, causing more elements to damage, as indicated in Figure 4.13 (b1) and (b2). The damaged elements lead to the concentration of stress in the surrounding elements, which in turn leads to the continuous

acceleration of cracks along the tip. The steel fibers began to demonstrate a large deformation, and the scattered micro cracks continue to evolve, develop, and penetrate. Therefore, the macroscopic cracks appeared, as revealed in Figure 4.13 (c1) and (c2). With the further increase in compressive strain, part of the steel fibers pulled out, and the macro cracks converged to form the main shear surface, finally, the specimen lost its bearing capacity, as shown in Figure 4.13 (d1) and (d2).

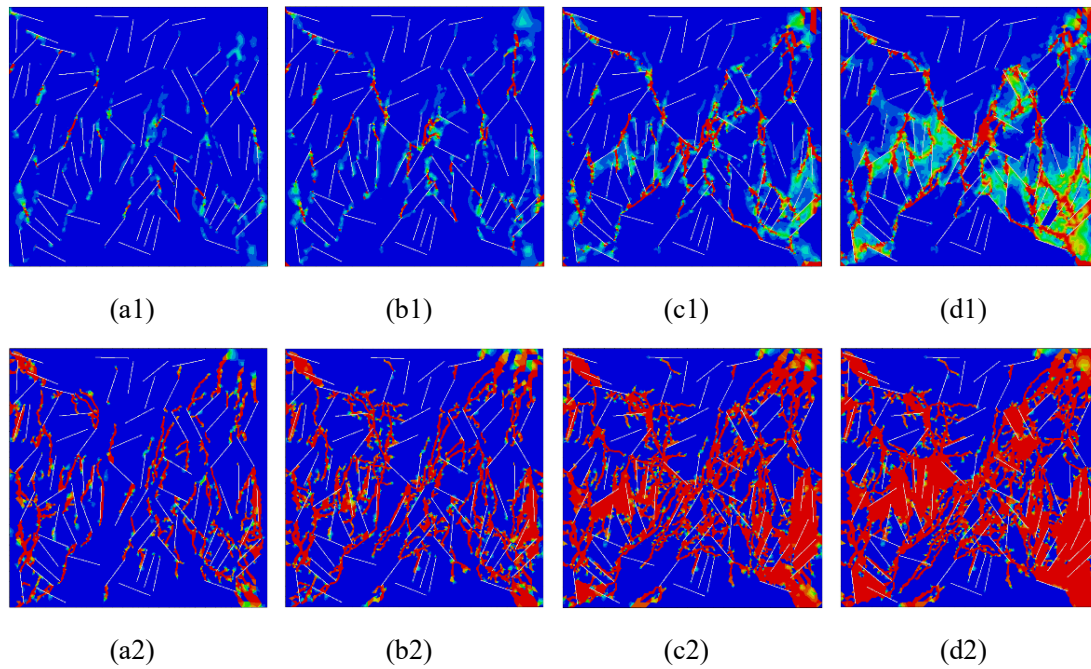


Figure 4.13 Compression/tensile damage evolution of UHPC during loading process.

The main function of the randomly distributed fibers in UHPC is to hinder the expansion of micro cracks and prevent the development of macro cracks. The difference in the fiber distribution angle and position have a great impact on the toughening and cracking resistance of concrete. Figure 4.14 demonstrates the tensile elements quantity ratio of the fibers in UHPC with steel fiber contents of 0.5%, 0.75%, 1.0%, 1.5%, 2.0%, and 2.5%. The tensile stress of a steel fiber is greater than the maximum tensile stress (6 MPa) of UHPC matrix and varied with different angles, i.e., the angles to the horizontal direction are 0° – 15° , 15° – 30° , 30° – 45° , 45° – 60° , 60° – 75° , and 75° – 90° , respectively. When the peak strain reaches 0.003, which corresponds to the average peak stress and divides the pre-peak and post-peak stages. The figure indicates that at the pre-peak stage, the angle of tensile steel fibers in the range of 0° – 60° . When the fiber content is low, i.e., the fiber content is less than or equal to 1%, the angle of the tensile fiber in the pre-peak stage is lower than 45° , as demonstrated in Figure 4.14 (a), (b), and (c). A smaller angle between the fiber and the horizontal direction results in a larger tensile ratio, indicating that the compressive stress increase the transverse

tensile stress of the specimen. In the process of damage along the vertical direction, the horizontal fibers serve a function in crack resistance. When the fiber content exceeds 1%, the angle range of the tensile fiber in the pre-peak stage became higher.

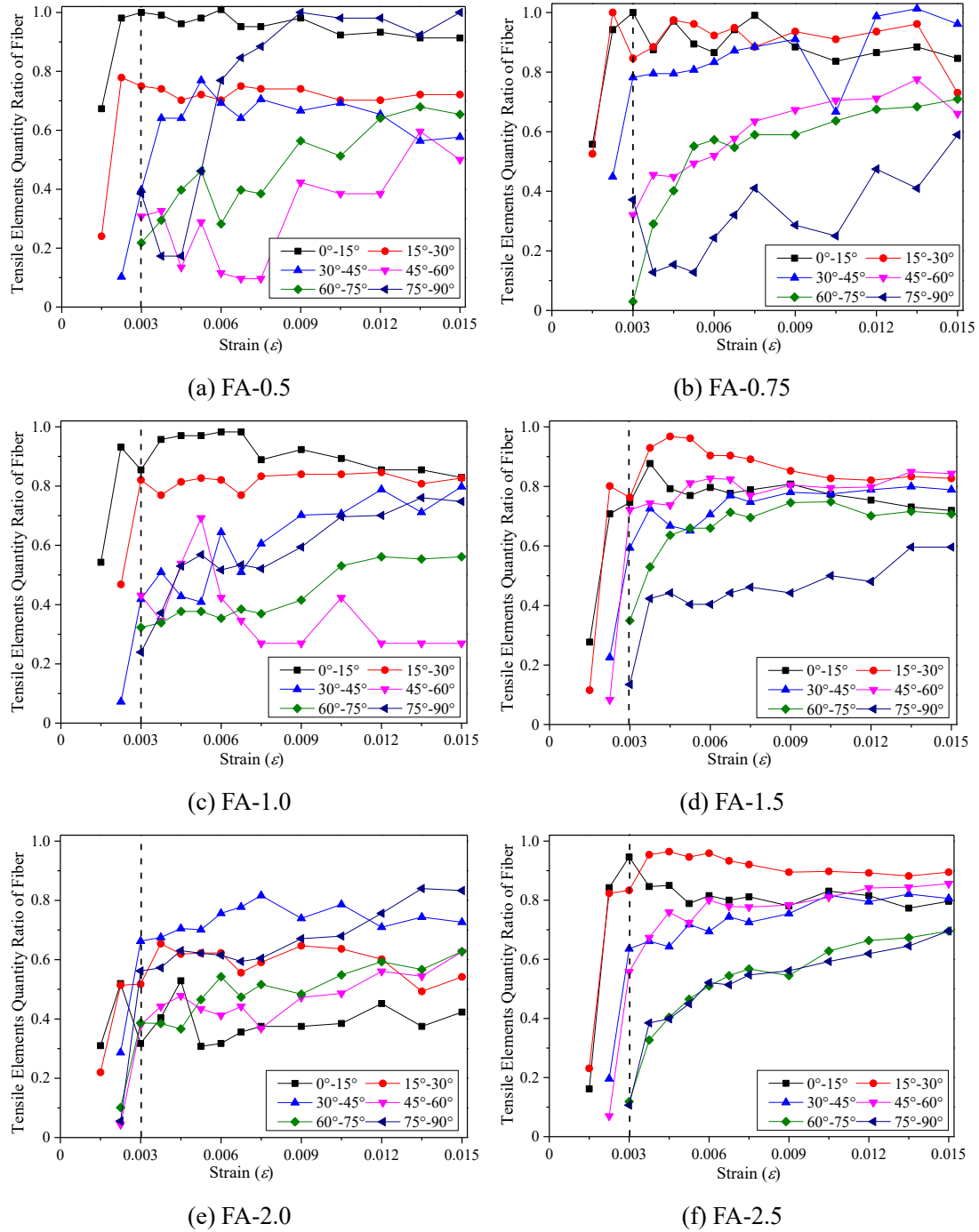


Figure 4.14 Ratio of tensile fiber quantity with different distribution angles.

However, the proportion of the tensile fiber elements with a smaller angle is still large, thereby playing a major role. After entering the post-peak stage, the micro cracks in the specimen gather to form macro cracks, and the horizontal fibers still display a toughening and crack resistance effect.

Moreover, due to the continuous extension and expansion of the cracks, the fibers with a higher angle (vertical distribution) also begin to bear the force, and the tensile elements ratio demonstrates a growing trend, which is not obvious.

Figure 4.15 shows the ratio of compressive/tensile fiber element quantity with different fiber contents. The compression element refers to a steel fiber element with a compressive stress greater than 0. A tensile element refers to a steel fiber element with a tensile stress greater than 6 MPa. The quantity ratio of compressive and tensile elements is essentially consistent. The compressive elements quantity ratio increase firstly and then decrease with the increasing strain. The peak of the ratio appears at the strain of 0.0015, which is also just before the damage occurs compared with Figure 4.9(a). This is because the cement base material and the fiber jointly bear the external force at the initial stage of the load. When the damage occurs in the specimen, the fibers spanning the damage or cracks start to serve the functions of toughening and cracking resistance. The compressive elements quantity ratio is rapidly decreasing while the tensile element quantity ratio increase, which also means that more fibers has changed from a compressed state to a crack-resistant tensile state. After the peak load, the tensile element quantity ratio and the compressive element quantity ratio of the steel fibers gradually stabilized, as the final ratio of the tensile quantity elements maintains at about 74.0% and the compressive fiber element ratio remains at approximately 9.6%.

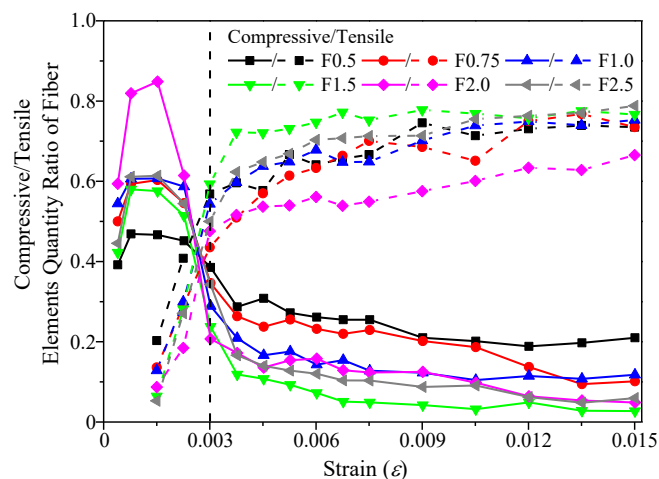


Figure 4.15 Compressed/tensile fiber quantity of UHPC with different fiber contents.

In order to reveal the stress states of the steel fibers under compression, the average stress values of the steel fibers with different fiber contents and different angle ranges were extracted. The results displayed in Figure 4.16 indicate that the steel fiber stress differs with different fiber contents while the overall fiber stress state change trends are essentially consistent. Before reaching the peak load,

the steel fiber stress shows a short-term compression trend (the stress value was less than 0) and then changes to a tension state with rapidly increased tension stress. The peak tensile stress appears after the compressive peak as the micro cracks have not expanded to macroscopic cracks when the load reach the peak value.

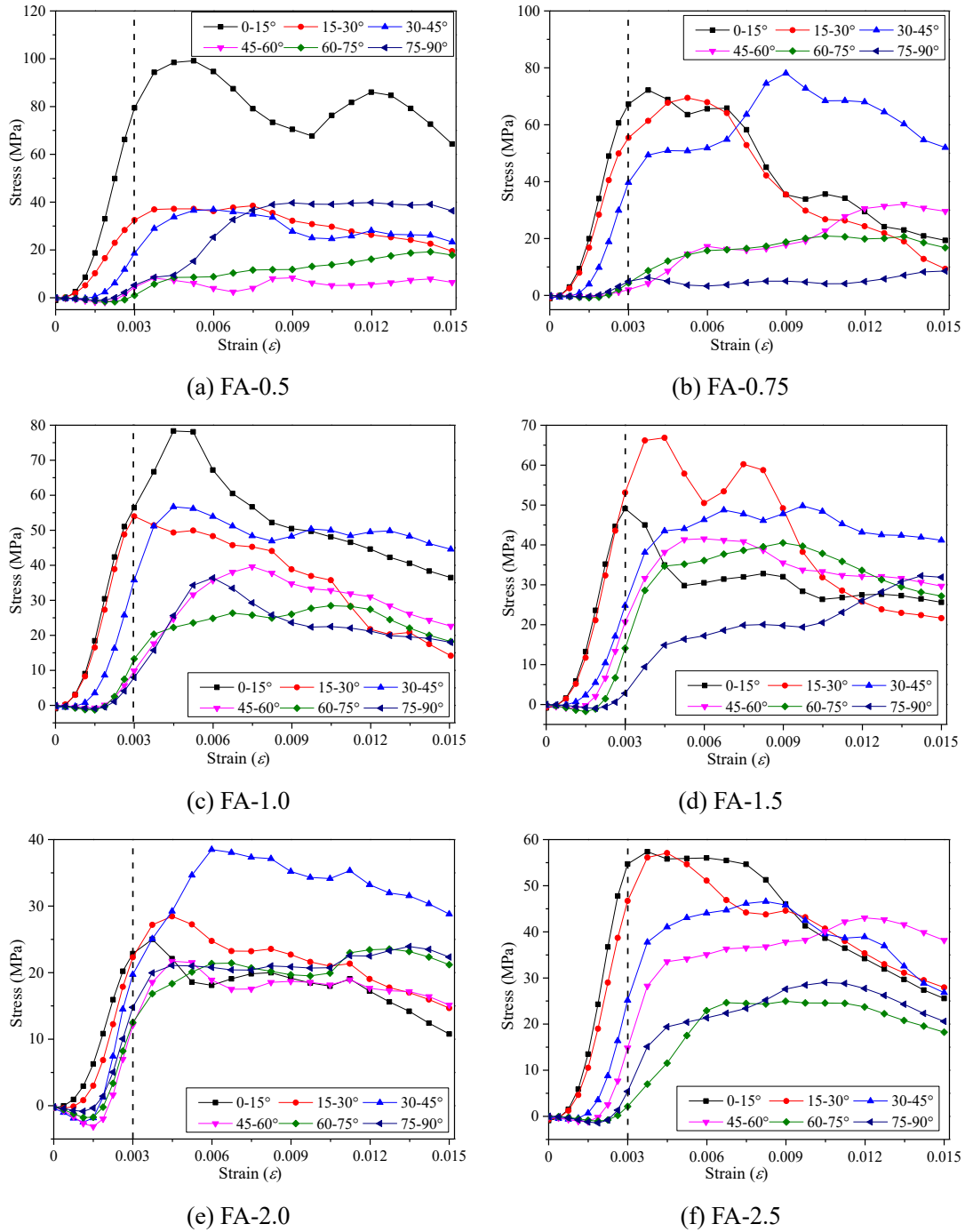


Figure 4.16 Changes in average stress of fibers at different angles.

When the strain continues to increase, the cracks converge and penetrate, forming the main shear plane. At the same time, a large number of fibers participate in bearing the force, and the bridging

stress of the fibers reach the maximum value. As the load-bearing capacity of the test specimen decrease, the stress of the steel fibers in most of the angle ranges began to decrease. A small number of the steel fibers are still bridging the cracks, and the stress continue to rise or remain unchanged. Moreover, when the angle between the steel fiber and the horizontal is around 45° , the small angle between the fiber and the horizontal led to a greater amount of the average stress value when the strain is less than 0.003. Similarly, this phenomenon is due to the fact that the vertical compressive stress arises the transverse tensile strain and limits the crack propagation along the up and down direction before the peak load. The transverse fibers are more likely to serve the functions of toughening and crack resistance than the vertical fibers, and the average tensile stress is greater. Therefore, in an actual project, the reasonable pouring method should be adopted according to the actual stress state of different components in the structure, so as to make the steel fiber arrangement in the structure play a maximum role.

Figure 4.17 shows the weighted average of the steel fiber stresses in different angle ranges in UHPC with different fiber contents. At the elastic stage and the initial damage stage, the average fiber stress changes to a tensile state after a short compression state, and the tensile stress continues to increase. When the macroscopic cracks formed, the average tensile stress of the fiber maintains a steady state; this is also the toughening stage of the fibers after UHPC cracking. When the strain almost reaches the failure strain, the average bridging stress of the fibers begins to decrease and finally the specimen loses bearing capacity.

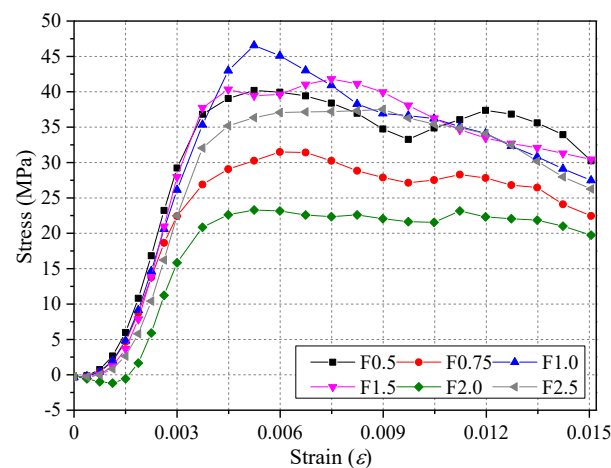


Figure 4.17 Average fiber stress of UHPC with different fiber contents.

Although the average stress of fibers in the same angle range differ greatly of with fiber contents (Figure 4.16), the difference in average stress of the whole fibers in the specimen is small (The

maximum stress difference of 0-15° fiber under the same strain reaches 80.58MPa, while the maximum average stress difference of different fiber content is 23.26MPa). The reason can be attributed to the fact that the stress value of the fibers in the specimen is larger than that in the fibers at both ends which are far away from the cracks. At the same time, the bridging stress of the vertical fiber is very low. Therefore, the average stress difference of the whole specimen is small.

4.6 Conclusion

In this chapter, the failure process of UHPC is investigated on meso-scale, experimentally and numerically. The following conclusions can be drawn:

(1) The analysis method based on the combination of the CDP model and the meso-random method can efficiently simulate the compression failure process of UHPC contains steel fibers and coarse aggregates. The experimental and the numerical simulation results are in good agreement.

(2) With the increasing fiber content (0.5%~2.5%), the compressive strength of UHPC increases. When the coarse aggregates content grows (0%~30%), the compressive strength first increases and then a reduction is observed.

(3) ITZ damage occurs earlier to the damage in mortar or aggregates, and the damage ratio development curves of ITZ are obviously different from those of mortar and aggregates. When the specimen damages and forms a macroscopic shear zone, the damage ratio of ITZ tends to become a constant value, whereas those of the mortar and aggregates continues to grow. At the final state, the damage ratio evolution of mortar keeps decreasing, the damage ratio of ITZ increases, and the damage ratio evolution of aggregates appears to go up first and then decrease with increasing fibers content.

(4) With the increase of compressive strain, most fibers change from the initial compression state to the tensile cracking resistance state after cracking. The element quantity ratio of the fiber that under compression state increases at first and then decreases, while the element quantity ratio of the fiber continues to increase under the tensile state.

(5) The analysis results of the fiber stress show that before the formation of the macroscopic shear surface in the compression loading process, the horizontally distributed fibers serve the functions of toughening and cracking resistance. After the formation of the macroscopic shear crack, the overall fiber in the UHPC participated in the force.

Chapter 5 Multi-scale Calculation Method for Mechanical Properties of UHPC

5.1 Overview

Traditionally, it is believed that macroscopic phenomenological experimental research methods are sufficient to meet the needs of material performance studies. However, with the development of UHPC materials and the precise control of target performance, traditional testing methods have become too expensive and have a long research cycle, making it difficult to meet engineering needs. Especially for UHPC formulations based on performance requirements, the required yield for each formula may not be high, and the cost of large-scale trial and error is too expensive. In addition, although some existing experimental research equipment, including scanning electron microscopy and nanoindentation technology, have been used to study the properties of UHPC materials, the microstructure formed during hydration of cement-based composite materials is very complex, making it difficult to accurately determine mechanical properties such as tensile (compressive) strength and elastic modulus through microscopic experiments. Combining the research results of chapter 4, it can also be found that although the microscopic or refined numerical simulations can achieve more accurate response descriptions of UHPC or other cement-based materials, the cost of the calculation method is obviously significant, especially for the large-scale structures. Furthermore, there is still a gap in the connection between the macroscopic and microscopic mechanical properties of materials. As a typical heterogeneous material, the macroscopic mechanical properties of UHPC are affected by the properties of the constituent materials and the ratio. It is necessary to apply alternative analysis and research models, such as multi-scale calculation and analysis methods, to establish the conversion relationship between the micro and macro properties of UHPC.

This chapter takes the multi-scale relationship between the microstructure and macroscopic mechanical properties of UHPC materials as the starting point, focusing on the influence of UHPC microstructure components and their properties on macroscopic homogenization performance. Establish a representative volume element (RVE) containing UHPC microstructural information, coupled with a progressive damage model, and solve for a multi-scale constitutive model with macroscopic mean field homogenization. Use the obtained model to calculate and verify the

macroscopic equivalent modulus, compression performance, tensile performance, and bending performance of UHPC, and predict the multi-scale performance of UHPC with different aggregate and fiber contents.

5.2 Micro-scale analysis method

Generally speaking, the macroscopic analysis scale (L) of composite materials differs greatly from the microscopic calculation scale ($L \gg l$) inside them. It is obviously impossible to fully consider the microscopic structural information and directly calculate it. Therefore, at the microscale, it is often assumed that the components of the material are arranged according to a certain periodic pattern, and representative volume elements (RVE) with representativeness are selected to theoretically calculate and analyze the macroscopic equivalent properties of non-uniform materials. In order to meet the periodic deformation coordination requirements of stress and deformation in periodic microstructures, corresponding periodic boundary conditions need to be applied on the RVE boundary. RVE is the link between macroscopic and microscopic scales. Introducing RVE can transfer the macroscopic equivalent performance evaluation of materials to the microscopic scale, simplifying the complexity of theoretical calculations and also being used for the analysis and research of discrete non-uniform materials.

The homogenization theory is generally used for computing and analyzing physical systems with two or more scales. The assumption of medium continuity in mechanics and physics can be regarded as a concrete manifestation of homogenization theory. The homogenization analysis method uses the macroscopic structure of the medium and the periodic microscopic structure characteristics of non-uniform media to describe macroscopic composite materials. The specific approach is to represent the field quantity describing the object as a function of macro coordinates and micro coordinates, and then use the ratio of macro coordinates and micro coordinates as the micro scale for parameter expansion. Transform the original problem into a uniform solution problem at both macroscopic and microscopic scales using perturbation methods. The perturbation analysis method, also known as the small parameter expansion method, is the use of perturbation method to solve the asymptotic solution of the objective equation. Usually, the physical equation is dimensionless, and then dimensionless small parameters reflecting physical characteristics are selected as perturbation amounts for power series expansion. By substituting the series into dimensionless equations, approximate equations for each level are obtained, and the coefficients of the power series

corresponding to these equations are determined. The series is truncated to obtain the asymptotic solution of the original equation. When using this method to solve the macroscopic equivalent performance of composite materials, the asymptotic solution of the equivalent performance includes the characteristics of heterogeneous microstructure, and can also obtain the microscopic stress field of the composite material disturbed by heterogeneity.

Consider a composite elastic body Ω with periodic structural characteristics, subjected to a physical force f , with a surface force t on the boundary Γ_t , and a displacement boundary condition Γ_u , where the macroscopic structure at a certain point x is viewed as a non-uniform RVE arranged periodically in space. The schematic diagram of the multi-scale homogenization process is shown in Figure 4.1.

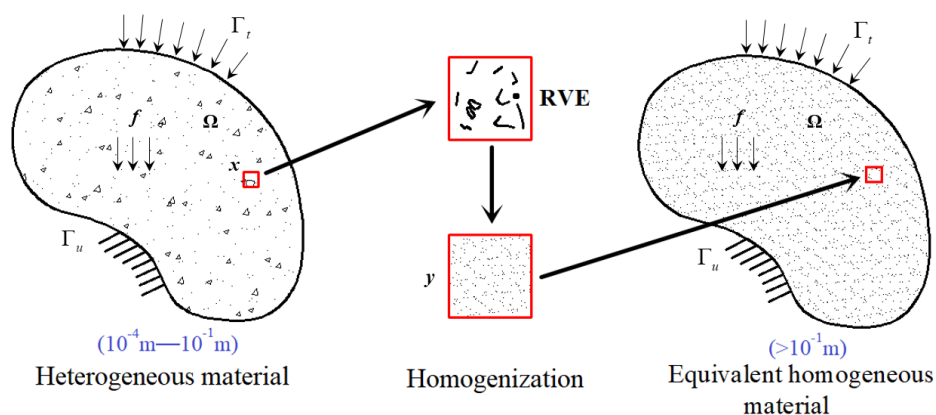


Figure 5.1 Multi scale homogenization process.

The homogenization method for the microstructure of composite materials is an important approach to evaluate their macroscopic effective performance based on the material's microscopic field and properties. Among them, solving the macroscopic equivalent stiffness and equivalent strength of RVE is a fundamental and important content. At present, the models and methods for solving the equivalent stiffness and strength of composite materials are relatively well-developed, and these models can be classified into the following categories:

(1) Direct solution (equivalent) method for composite material RVE: This method aims to obtain the equivalent stress (strain) field through analytical and numerical methods with the RVE microscopic field as the objective. The finite element solution of series model, parallel model, concentric cylinder model (applicable to FRP materials) and microscopic field all belong to this method.

(2) Approximate solving models based on the theory of single inclusion include sparse method, self consistent model, universal self consistent model, and Mori Tanaka method. This type of method is widely used to solve the macroscopic equivalent properties of composite materials with spherical and ellipsoidal inclusions.

(3) The boundary method based on the variational principle: This method is mainly based on the variational extremum solution principle to calculate the upper and lower limits of the macroscopic equivalent properties of composite materials, providing a reasonable basis for the performance design of the studied materials.

(4) Dual scale asymptotic unfolding homogenization method: This method originates from analyzing problems in physical systems with two or more long scales, progressively unfolding displacement fields at both macroscopic and microscopic scales, and applying them to periodic RVE structures.

5.2.1 Direct Equivalent Method

The direct equivalent method is based on the average of the surface (2D) or volume (3D) of the target field quantity (stress field or strain field) to directly calculate the equivalent stress or strain, and then obtain the macroscopic equivalent performance based on the relationship between macroscopic equivalent stress and macroscopic equivalent strain. The volume average of local (microscopic) stress $\bar{\sigma}_{ij}$ and strain $\bar{\varepsilon}_{ij}$ in materials can be defined as

$$\bar{\sigma}_{ij} = \frac{1}{V} \int_{\Omega} \sigma_{ij} d\Omega \quad (5.1)$$

$$\bar{\varepsilon}_{ij} = \frac{1}{V} \int_{\Omega} \varepsilon_{ij} d\Omega \quad (5.2)$$

where, Ω and V represent the RVE elastic region and volume, and the average strain energy density $\bar{\omega}$ of the elastic body can be expressed as

$$\bar{\omega} = \frac{1}{V} \int_{\Omega} \omega d\Omega = \frac{1}{V} \int_{\Omega} \frac{1}{2} \sigma_{ij} \varepsilon_{ij} d\Omega = \frac{1}{V} \int_{\Omega} \frac{1}{2} C_{ijkl} \varepsilon_{ij} \varepsilon_{kl} d\Omega = \frac{1}{V} \int_{\Omega} \frac{1}{2} S_{ijkl} \sigma_{ij} \sigma_{kl} d\Omega \quad (5.3)$$

Where, $\omega = \frac{1}{2} \sigma_{ij} \varepsilon_{ij}$ is the strain energy density, C_{ijkl} is the stiffness coefficient; S_{ijkl} is the flexibility coefficient. In addition, $\bar{\omega}$ can also be expressed as

$$\bar{\omega} = \frac{1}{2} \bar{\sigma}_{ij} \bar{\varepsilon}_{ij} \quad (5.4)$$

The relationship between the effective stiffness tensor, flexibility tensor, and average stress (strain) tensor is as follows,

$$\bar{\sigma}_{ij} = \bar{C}_{ijkl} \bar{\varepsilon}_{kl} \quad (5.5)$$

$$\bar{\varepsilon}_{ij} = \bar{S}_{ijkl} \bar{\sigma}_{kl} \quad (5.6)$$

Thus, the relationship between the equivalent stiffness tensor and strain energy is obtained as follows:

$$\bar{C}_{ijkl} = \frac{\partial^2 \bar{\omega}}{\partial \bar{\varepsilon}_{ij} \partial \bar{\varepsilon}_{kl}} \quad (5.7)$$

Assuming that the elastic body Ω consists of $n+1$ regions ($\Omega_0, \Omega_1, \dots, \Omega_n$), each region has a volume fraction of $v_0, v_1, \dots, v_n$, moreover $\sum_{r=0}^n v_r = 1$, the average stress, average strain, and average strain energy density can be expressed as

$$\bar{\sigma}_{ij} = \sum_{r=0}^n v_r \bar{\sigma}_{ij}^{(r)} \quad (5.8)$$

$$\bar{\varepsilon}_{ij} = \sum_{r=0}^n v_r \bar{\varepsilon}_{ij}^{(r)} \quad (5.9)$$

$$\bar{\omega} = \sum_{r=0}^n v_r \bar{\omega}^{(r)} \quad (5.10)$$

When RVE has uniform boundary conditions, the displacement boundary conditions include as

$$u_i^0(\Gamma) = \varepsilon_{ij}^0 x_j \quad (5.11)$$

For stress boundary conditions, there are

$$T_i(\Gamma) = \sigma_{ij}^0 n_j \quad (5.12)$$

Among them, $\varepsilon_{ij}^0, \sigma_{ij}^0$ is the constant strain (stress), x_j is the coordinate j direction component, and n_j is the normal component outside the boundary. When the stress field inside the inclusion is a uniform field, the equivalent elastic performance can also be represented by the stress (strain) concentration factor. Elastic bodies containing sparse ellipsoidal (spherical) inclusions can be represented in this way, and the linear elastic relationship in each term is:

$$\sigma_{ij}^{(r)} = C_{ijkl}^r \varepsilon_{kl}^{(r)} \quad (r = 0, 1, \dots, n) \quad (5.13)$$

$$\varepsilon_{ij}^{(r)} = S_{ijkl}^r \sigma_{kl}^{(r)} \quad (r = 0, 1, \dots, n) \quad (5.14)$$

Substituting equation (5.13) into equation (3.8) yields the equivalent stiffness tensor of the composite material as follows:

$$\bar{C} = C^0 + \sum_{r=1}^n v_r (C^r - C^0) \varepsilon^{(r)} \bar{\varepsilon}^{-1} \quad (5.15)$$

Similarly, the equivalent flexibility tensor can be obtained as:

$$\bar{S} = S^0 + \sum_{r=1}^n \nu_r (S^r - S^0) \sigma^{(r)} \bar{\sigma}^{-1} \quad (5.16)$$

For the r phase inclusion, introducing strain and stress concentration factors A^r and B^r , the relationship between the average strain $\varepsilon^{(r)}$ and stress $\sigma^{(r)}$ in the inclusion and the macroscopic equivalent strain $\bar{\varepsilon}$ and stress $\bar{\sigma}$ of the composite material is as follows,

$$\varepsilon^{(r)} = A^r \bar{\varepsilon} \quad (5.17)$$

$$\sigma^{(r)} = B^r \bar{\sigma} \quad (5.18)$$

So the equivalent stiffness and flexibility tensors of composite materials can be expressed as:

$$\bar{C} = C^0 + \sum_{r=1}^n \nu_r (C^r - C^0) \varepsilon^{(r)} A^r \quad (5.19)$$

$$\bar{S} = S^0 + \sum_{r=1}^n \nu_r (S^r - S^0) \sigma^{(r)} B^r \quad (5.20)$$

The above solving process indicates that as long as the strain (stress) concentration factor is obtained, the equivalent macroscopic properties of the composite material can be obtained. In fact, the strain (stress) concentration factor evolved from the elastic field of an infinitely large homogeneous medium containing a single ellipsoidal inclusion, which was first derived by Eshelby [141]. The strain concentration tensor, also known as the Eshelby tensor, is a fourth-order symmetric tensor that is related to the geometry of inclusions.

If the inclusions in composite materials are distributed completely randomly, they can be regarded as isotropic materials at the macroscopic scale. At this time, the equivalent properties of the material can be represented by two independent engineering constants, the equivalent bulk modulus k and the equivalent shear modulus μ . The equivalent principal stress and shear stress can be expressed as:

$$\bar{\sigma}_{kk} = 3k \bar{\varepsilon}_{kk} \quad (5.21)$$

$$\bar{s}_{kk} = 2\mu \bar{e}_{kk} \quad (5.22)$$

where, $\bar{\sigma}_{kk} = \bar{\sigma}_{11} + \bar{\sigma}_{22} + \bar{\sigma}_{33}$, $\bar{\varepsilon}_{kk} = \bar{\varepsilon}_{11} + \bar{\varepsilon}_{22} + \bar{\varepsilon}_{33}$, $\bar{s}_{ij}$ and $\bar{e}_{ij}$ is the offset of $\bar{\sigma}_{ij}$ and $\bar{\varepsilon}_{ij}$,

k and μ can be further expressed as:

$$k = k_0 + \sum_{r=1}^n \nu_r (k_r - k_0) \varepsilon_{kk}^{(r)} \bar{\varepsilon}_{ii}^{-1} \quad (5.23)$$

$$\mu = \mu_0 + \sum_{r=1}^n \nu_r (\mu_r - \mu_0) e_{ij}^{(r)} \bar{e}_{ij}^{-1} \quad (5.24)$$

5.2.2 Equivalent method based on the inclusion theory

Many classic micromechanical stiffness prediction methods are derived from Eshelby's research

on the problem of infinite media containing a single inclusion. This type of method does not use the average of field quantities, but solves the equivalent performance based on the content of inclusions, geometric dimensions, and microstructural properties. Sparse method, self consistent method, generalized self consistent method, and Mori-Tanaka method all evolved from that theory.

Assuming the uniform strain inside the ellipsoidal inclusion is $\varepsilon^{(r)}$, the intrinsic strain is ε^* , and the fourth-order Eshelby tensor is S_{ijkl}^E , the following relationship exists:

$$\varepsilon^{(r)} = S_{ijkl}^E \varepsilon^* \quad (5.25)$$

The intrinsic strain here refers to the strain that does not generate stress when the constraints around the inclusion are removed. Such strains include material thermal strain, plastic strain, and phase transition strain. The Eshelby tensor has symmetry and satisfies $S_{ijkl}^E = S_{jikl}^E = S_{ijlk}^E$.

As shown in Figure 5.2, when there is an ellipsoidal inclusion with a modulus of C^r in an infinite homogeneous medium (with a modulus of C^0), it is affected by the far field uniform stress boundary $\bar{\varepsilon}$ (or $\bar{\sigma}$). Based on the Eshelby inclusion theory, it is assumed that the ellipsoidal inclusion is transformed into the same material as the matrix, and the intrinsic strain required for the transformation is ε^* , then using the superposition principle, the average strain and average stress inside the inclusion can be obtained as follows:

$$\varepsilon^{(r)} = \left[I + S_r^E : C_0^{-1} : (C_r - C_0) \right]^{-1} : \bar{\varepsilon} \quad (5.26)$$

$$\sigma^{(r)} = C_r : \left[C_0 + C_0 S_r^E : C_0^{-1} : (C_r - C_0) \right]^{-1} : \bar{\sigma} \quad (5.27)$$

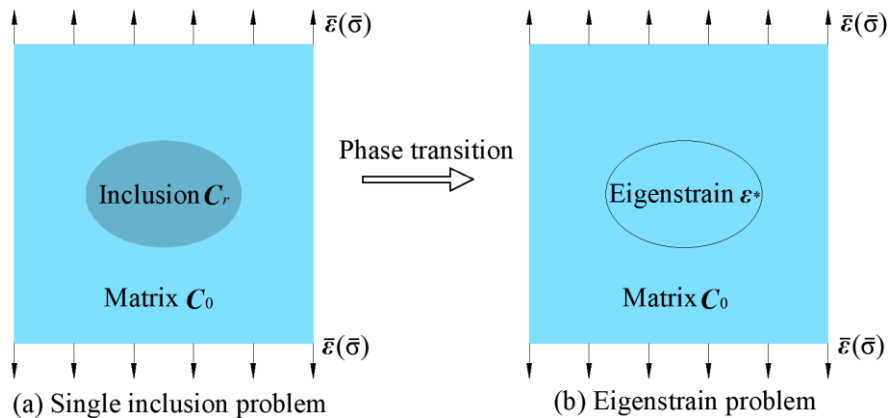


Figure 5.2 Schematic diagram of Eshelby equivalent inclusion theory.

The problem of multiple inclusions can be abstracted as solving multiple single inclusion problems. For cases where the local coordinate system of a single inclusion is inconsistent with the

macroscopic coordinate system, the physical quantities in the local coordinate system can be converted to a unified macroscopic coordinate when solving the RVE mean field quantity. For the problem of multiple inclusions, it is also necessary to consider the interactions between inclusions. There are mainly two implementation methods: one is to change the matrix modulus, and the corresponding methods include self-consistent method and generalized self-consistent method. The second method is to change the far-field load conditions, and the corresponding method is the Mori-Tanaka method.

(1) Sparse method

The sparse method is suitable for calculating composite materials with low inclusion content, in which case the influence between inclusions can be ignored. Based on this assumption, the uniform strain $\varepsilon^{(r)}$ within the r phase inclusion in the matrix is approximately equal to the strain when a single inclusion is embedded in an infinitely large elastic body. When composite materials are subjected to macroscopic uniform strain $\bar{\varepsilon}$, the macroscopic equivalent stiffness tensor of the composite material can be obtained according to (5.17), (5.19), and (5.26) as follows:

$$\bar{\mathbf{C}} = \mathbf{C}_0 + \sum_{r=1}^n \nu_r \left[(\mathbf{C}_r - \mathbf{C}_0)^{-1} + \mathbf{S}_r^E : \mathbf{C}_0^{-1} \right] \varepsilon^{(r)} \mathbf{A}^r \quad (5.28)$$

where, $\mathbf{S}_r^E$ and $\mathbf{C}_r$ are the Eshelby tensor and stiffness tensor of the r phase inclusion, respectively, and $\mathbf{C}_0$ are the elastic stiffness tensor of the matrix phase. When a composite material is composed only of isotropic materials consisting of a matrix phase and inclusions, and the reinforcing phase is randomly distributed spherical particles, then this composite material can be regarded as an isotropic material. The expressions for the macroscopic equivalent bulk modulus and equivalent shear modulus can be obtained from (5.28) as follows:

$$\bar{K} = K_0 + \frac{\nu_r (4G_0 + 3K_0)(K_1 - K_0)}{4G_0 + 3K_1} \quad (5.29)$$

$$\bar{G} = G_0 + \frac{5\nu_r G_0 (4G_0 + 3K_0)(G_1 - G_0)}{3K_0 (3G_0 + 2G_1) + 4G_0 (2G_0 + 3G_1)} \quad (5.30)$$

where, $K = E / 3(1 - 2\nu)$ is the bulk modulus, G is the shear modulus, and ν_r is the volume fraction of inclusions. When using uniform stress boundary conditions $\bar{\sigma}$, the macroscopic equivalent stiffness tensor and equivalent flexibility tensor of the composite material can be

obtained from (5.25), (5.26), and (5.27) as follows:

$$\bar{\mathbf{C}} = \left\{ \mathbf{C}_0^{-1} + \sum_{r=1}^n \nu_r \left[(\mathbf{I} - \mathbf{C}_0^{-1} : \mathbf{C}_r)^{-1} - \mathbf{S}_r^E \right]^{-1} : \mathbf{C}_0^{-1} \right\}^{-1} \quad (5.31)$$

$$\bar{\mathbf{S}} = \mathbf{S}_0 + \sum_{r=1}^n \nu_r \left[(\mathbf{I} - \mathbf{S}_0^{-1} : \mathbf{S}_r^{-1})^{-1} - \mathbf{S}_r^E \right]^{-1} : \mathbf{S}_0 \quad (5.32)$$

Similarly, if the inclusions in the composite material are spherical, the corresponding equivalent

bulk modulus $\bar{K}$ and equivalent shear modulus $\bar{G}$ can be calculated from (5.31) as follows:

$$\bar{K} = K_0 + \frac{\nu_r (4G_0 + 3K_0)(K_1 - K_0)}{K_0 (4G_0 + 3K_1) - \nu_r (4G_0 + 3K_0)(K_1 - K_0)} \quad (5.33)$$

$$\bar{G} = G_0 + \frac{5\nu_r G_0 (4G_0 + 3K_0)(G_1 - G_0)}{3K_0 (3G_0 + 2G_1) + 4G_0 (2G_0 + 3G_1) - 5\nu_r G_0 (4G_0 + 3K_0)(G_1 - G_0)} \quad (5.34)$$

When $\nu_r \rightarrow 0$, the inclusion content is very small, the calculation results of equations (5.31), (5.32),

(5.33), and (5.34) tend to approach the bulk modulus and shear modulus of the matrix, respectively.

It can be seen that the sparse method is more accurate in solving at this time. On the contrary, when

the inclusion content is high ($\nu_r > 5\%$), the results of (5.31) and (5.32), (4.33) and (5.34) are

inconsistent. This is because the premise of using the sparse method is to assume that the matrix is

infinitely large and that the macroscopic stress and strain satisfy $\bar{\boldsymbol{\sigma}} = \mathbf{C}_0 : \bar{\boldsymbol{\varepsilon}}$, which obviously does

not conform to actual physical facts. To solve this problem, the interaction between inclusions can

be considered. One way is to change the matrix modulus and replace it with the equivalent modulus

($\mathbf{C}_0 \rightarrow \bar{\mathbf{C}}$); Another way is to change the far-field load by replacing it with the average load within

the matrix ($\bar{\boldsymbol{\sigma}} \rightarrow \boldsymbol{\sigma}_0$, $\bar{\boldsymbol{\varepsilon}} \rightarrow \boldsymbol{\varepsilon}_0$).

(2) Self-consistent method

The self-consistent method assumes that after the inclusion is embedded into an infinite matrix, the stiffness coefficient of the matrix changes from $\mathbf{C}_0$ to an equivalent stiffness coefficient $\bar{\mathbf{C}}$.

By combining (5.26) and (5.31), the equivalent stiffness tensor of the self-consistent model can be obtained as follows:

$$\bar{\mathbf{C}} = \mathbf{C}_0 + \sum_{r=1}^n \nu_r (\mathbf{C}_r - \mathbf{C}_0) : \left[\mathbf{I} + \mathbf{S}_r^E : \bar{\mathbf{C}}^{-1} : (\mathbf{C}_r - \bar{\mathbf{C}}) \right]^{-1} \quad (5.35)$$

It should be noted that equation (5.35) is an implicit equation and cannot be analytically expressed.

Generally, numerical methods are needed to solve it.

If a composite material is composed only of two-phase materials and contains randomly distributed spherical particles, its equivalent modulus can be obtained from (5.35) as

$$\bar{K} = K_0 + \frac{\nu_r (4\bar{G} + 3\bar{K})(K_1 - \bar{K})}{4\bar{G} + 3K_1} \quad (5.36)$$

$$\bar{G} = G_0 + \frac{5\nu_r \bar{G} (4\bar{G} + 3K_0)(G_1 - G_0)}{3\bar{K}(3\bar{G} + 2G_1) + 4\bar{G}(2\bar{G} + 3G_1)} \quad (5.37)$$

(3) Generalized self-consistent method

In order to obtain more accurate equivalent moduli, some more precise models have been further developed based on the self-consistent model, and the representative one is the generalized self-consistent method. The generalized self-consistent method assumes that the matrix surrounding the inclusion forms a new inclusion with the original inclusion, and this new inclusion is embedded into an infinitely large matrix during calculation. The generalized inclusion method considers the microstructure characteristics of composite materials from a more refined perspective, which can more accurately calculate the equivalent performance. However, the generalized self-consistent method also has its own shortcomings, mainly reflected in the fact that it cannot be solved by the Eshelby equivalent inclusion method, and more solutions are based on specific inclusion shapes using elastic mechanics theory. This increases the difficulty of solving and reduces the universality of the method. For inclusions with complex shapes, analytical solutions cannot usually be obtained. When the inclusions in the matrix are randomly distributed in a spherical shape, the stiffness coefficient obtained by the generalized self-consistent method is:

$$\bar{K} = K_0 + \frac{\nu_r (4G_0 + 3K_0)(K_1 - K_0)}{(4G_0 + 3K_0) + 3(1 - \nu_r)(K_1 - K_0)} \quad (5.38)$$

$$A \left(\frac{G}{G_0} \right)^2 + 2B \left(\frac{G}{G_0} \right) + C = 0 \quad (5.39)$$

In the above equation, A, B, and C are coefficients for solving the shear modulus. As early as 1979, Christensen^[184] and Christensen^[185] provided a generalized self-consistent solution based on

the characteristic function, which is limited to isotropic spherical or cylindrical single inclusions. After that, Huang et al. ^[186] extended the generalized self-consistent method to anisotropic materials, ellipsoidal inclusions, multiphase inclusions, and microcracks using the energy method.

(4) Mori-Tanaka method

The derivation process of M-T method is mainly based on the approximate solution of Eshelby equivalent inclusions. Both the self-consistent method and the generalized self-consistent method consider the influence of inclusions by changing the stiffness of the matrix. The far-field stress (strain) of the matrix is equal to the uniform stress (strain) under external load. The M-T method considers the mutual influence between inclusions and matrix by changing the far-field load. The M-T method is widely used in predicting the effective performance of various composite materials.

Due to the interaction of inclusions, for the problem of multiple inclusions containing intrinsic strain (ε^*), the stress inside the matrix is relatively complex, and the average stress $\langle \sigma \rangle_m$ inside the matrix can be expressed as:

$$\langle \sigma \rangle_m = C_0 \langle \varepsilon \rangle_m = -\nu_r C_0 (S^E \varepsilon^* - \varepsilon^*) \quad (5.40)$$

where, $\langle \varepsilon \rangle_m$ is the average strain of the matrix, ν_r is the volume fraction of inclusions, and C_0 is the stiffness tensor of the matrix. Weng ^[187] further developed this method based on the concept of average stress using the M-T method. Assuming that the matrix containing inclusions is a single-phase composite material subjected to uniform stress boundary conditions (equivalent stress is σ_0), while the matrix without inclusions under the same boundary conditions has:

$$\sigma_0 = C_0 \varepsilon_0 \quad (5.41)$$

Due to the presence of inclusions, the strain field in the actual composite matrix differs from the strain field in a uniform and non-inclusion state by a perturbation strain $\tilde{\varepsilon}$, and the corresponding stress is perturbation stress $\tilde{\sigma}$. In practical composite materials, the strain and stress of the matrix become $\varepsilon_0 + \tilde{\varepsilon}$ and $\sigma_0 + \tilde{\sigma}$, satisfying that:

$$\sigma_0 + \tilde{\sigma} = C_0 (\varepsilon_0 + \tilde{\varepsilon}) \quad (5.42)$$

The strain and stress inside the inclusion deviate from the matrix. Assuming the deviation is ε' and σ' , the strain and stress inside the inclusion actually become $\varepsilon_0 + \tilde{\varepsilon} + \varepsilon'$ and $\sigma_0 + \tilde{\sigma} + \sigma'$,

and the physical equation inside the inclusion is

$$\sigma_1 = \mathbf{C}_1 (\varepsilon_0 + \tilde{\varepsilon} + \varepsilon') = \mathbf{C}_0 (\varepsilon_0 + \varepsilon + \tilde{\varepsilon} - \varepsilon') \quad (5.43)$$

and then

$$\varepsilon' = \mathbf{S}^E \varepsilon^* \quad (5.44)$$

while the disturbance strain is

$$\tilde{\varepsilon} = -\nu_r (\mathbf{S}^E - \mathbf{I}) \varepsilon^* \quad (5.45)$$

By solving the above equation, it can obtain

$$\varepsilon^* = \mathbf{H} \varepsilon_0 \quad (5.46)$$

where,

$$\mathbf{H} = [\mathbf{C}_0 + \Delta (\nu_r \mathbf{I} - \nu_0 \mathbf{S}^E)]^{-1} : \Delta \mathbf{C} \quad (5.47)$$

$$\Delta \mathbf{C} = \mathbf{C}_1 - \mathbf{C}_0 \quad (5.48)$$

The equivalent effect $\bar{\varepsilon}$ can be further transformed into:

$$\bar{\varepsilon} = \nu_1 \varepsilon_1 + (1 - \nu_1) \varepsilon_0 = \nu_1 (\varepsilon_0 + \tilde{\varepsilon} + \varepsilon') + (1 - \nu_1) (\varepsilon_0 + \tilde{\varepsilon}) = \varepsilon_0 + \nu_1 \varepsilon^* \quad (5.49)$$

after substituting (3.46), it can obtain:

$$\bar{\varepsilon} = (\mathbf{I} + \nu_1 \mathbf{H}) \varepsilon_0 \quad (5.50)$$

while the final equivalent modulus obtained as

$$\bar{\mathbf{C}} = \mathbf{C}_0 : (\mathbf{I} + \nu_1 \mathbf{H})^{-1} \quad (5.51)$$

Another widely used extension of the M-T method was proposed by Benveniste ^[188]. Starting from the strain concentration factor, assuming that it represents the strain concentration factor in the sparse state of composite materials and is independent of the volume fraction, then in the case of finite volume fraction, it is assumed that:

$$\varepsilon_1 = \mathbf{A} \bar{\varepsilon} \quad (5.52)$$

where $\mathbf{A}$ is associated with volume fraction, after it is determined, the equivalent stiffness of the composite material is:

$$\bar{\mathbf{C}} = \mathbf{C}_0 + \nu_r (\mathbf{C}_1 - \mathbf{C}_0) : \mathbf{A} \quad (5.53)$$

Assuming that the strain in the inclusion and the strain in the matrix satisfy the following

relationship:

$$\varepsilon_1 = \mathbf{G}\varepsilon_0 \quad (5.54)$$

So, the strain concentration factor under effective volume fraction can be obtained according to

$$\bar{\varepsilon} = \nu_r \varepsilon_1 + (1 - \nu_r) \varepsilon_0 \text{ as:}$$

$$\mathbf{A} = \mathbf{G} : [\nu_0 \mathbf{I} - \nu_r \mathbf{G}]^{-1} \quad (5.55)$$

Under the limit conditions of $\nu_r \rightarrow 0$ and $\nu_r \rightarrow 1$, the inclusion content satisfies:

$$\begin{cases} \mathbf{A}|_{\nu_r \rightarrow 0} = \tilde{\mathbf{A}} \\ \mathbf{A}|_{\nu_r \rightarrow 1} = \mathbf{I} \end{cases} \quad (5.56)$$

By combining (4.53) and (4.55), the strain concentration factor $\tilde{\mathbf{A}}$ can be obtained as follows:

$$\tilde{\mathbf{A}} = [\mathbf{I} - \mathbf{S}^E : (\mathbf{C}_0) : (\mathbf{C}_1 - \mathbf{C}_0)]^{-1} \quad (5.57)$$

Furthermore, the equivalent stiffness is obtained as:

$$\bar{\mathbf{C}} = \mathbf{C}_0 + \nu_r (\mathbf{C}_1 - \mathbf{C}_0) : \tilde{\mathbf{A}} : [\nu_0 \mathbf{I} - \nu_r \tilde{\mathbf{A}}]^{-1} \quad (5.58)$$

The calculation of the multi-scale mechanical properties of UHPC in this thesis is based on the coupling of M-T homogenization method and progressive damage model.

5.2.3 Differential Method

The idea of differentiation is to construct a differentiation process that gradually adds inclusions into the matrix. Adhere to the idea of cyclic iteration of ‘small addition homogenization’. Assuming that an inclusion with a volume fraction of ν_1 exists in a composite material with a volume of V_0 , the corresponding equivalent stiffness is $\mathbf{C}$. After adding inclusions with a volume of δV , the volume fraction changes from ν_1 to $\nu_1 + \delta \nu_1$, and the corresponding equivalent modulus changes as $\mathbf{C} + \delta \mathbf{C}$. Removing the volume of the matrix to maintain the original volume of the composite material, the actual content of inclusions becomes as

$$\nu_1 V_0 + \delta V - \nu_1 \delta V = (\nu_1 + \delta \nu_1) V_0 \quad (5.59)$$

The uniform stress and strain relationship of composite materials becomes:

$$\bar{\sigma} = (\mathbf{C} + \delta \mathbf{C}) \bar{\varepsilon} \quad (5.60)$$

Due to the small volume fraction of inclusions, the Eshelby equivalent inclusion method using

sparse method was used to solve the average strain concentration factor as follows:

$$\mathbf{A} = [\mathbf{I} + \mathbf{S}^E \mathbf{C}^{-1} : (\mathbf{C}_1 - \mathbf{C})]^{-1} \quad (5.61)$$

By combining (4.59), (4.60), and (4.61), the stiffness change of the composite material can be calculated as follows:

$$\delta \mathbf{C} = (\mathbf{C}_1 - \mathbf{C}) \mathbf{A} \frac{\delta V}{V_0} \quad (5.62)$$

Substituting (4.59) into the above equation, when $\delta \nu_1$ approaching infinity, the differential equation of equivalent stiffness has the following form:

$$\frac{d\mathbf{C}}{d\nu_1} = \frac{1}{1-\nu_1} (\mathbf{C}_1 - \mathbf{C}) \mathbf{A} \quad (5.63)$$

For the above equation, when $\nu_1 = 0$ and $\mathbf{C} = \mathbf{C}_0$, for spherical inclusion particles, the differential equations for the bulk modulus K and shear modulus G of composite materials can be obtained from the above equation:

$$\begin{cases} \frac{dK}{d\nu_1} = \frac{(K_1 - K)(K + K^*)}{(1-\nu_1)(K_1 + K^*)} \\ \frac{dG}{d\nu_1} = \frac{(G_1 - G)(G + G^*)}{(1-\nu_1)(G_1 + G^*)} \end{cases} \quad (5.64)$$

where

$$K^* = \frac{4}{3}G \quad , \quad G^* = \frac{G(9K + 8G)}{6(K + 2G)} \quad (5.65)$$

For the above equation, if $G = G_0$ and $K = K_0$, it can obtain the explicit solution of (4.64) as follows:

$$\begin{cases} K = K_0 + \frac{\nu_1(K_1 - K)}{1 + (1-\nu_1)\frac{K_1 - K_0}{K_1 + K^*}} \\ G = G_0 + \frac{\nu_1(G_1 - G)}{1 + (1-\nu_1)\frac{G_1 - G_0}{G_1 + G^*}} \end{cases} \quad (5.66)$$

5.2.4 Variational Method

Variational method, also known as the boundary method based on the variational principle, uses the variational extremum principle to obtain the upper and lower limits of the equivalent performance of composite materials, thereby providing a reasonable design range for the evaluation

of the equivalent performance design of composite materials.

Assuming uniform boundary conditions are applied to a region with a volume of V , the strain energy $\tilde{U}$ that satisfies the strain boundary condition ($\tilde{\epsilon}$) in that region has the following form:

$$\tilde{U} = \frac{1}{V} \sum_{r=0}^n \frac{1}{2} \int_{V_r} \tilde{\epsilon} : \tilde{C}_1 : \tilde{\epsilon} dV \quad (5.67)$$

So the true strain energy density corresponding to the equivalent strain ($\bar{\epsilon}$) can be expressed as:

$$U = \frac{1}{2} \bar{\epsilon} : \bar{C} : \bar{\epsilon} \quad (5.68)$$

According to the principle of minimum potential energy, in a permissible strain field that satisfies displacement boundary conditions and geometric conditions, the strain energy of the system is taken as the true state strain energy (minimum value), and there exists:

$$U \leq \tilde{U} \quad (5.69)$$

Assuming that the allowable stress field $\tilde{\sigma}$ of composite materials satisfies the stress boundary conditions and equilibrium equation, the residual energy of the system in this state is:

$$\tilde{P} = \frac{1}{V} \sum_{r=0}^n \frac{1}{2} \int_{V_r} \tilde{\sigma} : S_r : \tilde{\sigma} dV \quad (5.70)$$

The true residual energy density of the system represented by equivalent stress ($\bar{\sigma}$) is:

$$P = \frac{1}{2} \bar{\sigma} : \bar{S} : \bar{\sigma} \quad (5.71)$$

where $\bar{S}$ represents the equivalent flexibility tensor. According to the principle of minimum residual energy, the real stress field minimizes the residual energy of the system, and

$$P \leq \tilde{P} \quad (5.72)$$

Based on this idea, Voigt and Reuss respectively replaced the allowable strain and allowable stress with equivalent strain $\tilde{\epsilon} = \bar{\epsilon}$ and equivalent stress $\tilde{\sigma} = \bar{\sigma}$ to obtain the upper and lower limits of the equivalent stiffness of composite materials as

$$\left\{ \begin{array}{l} \bar{C} \leq \frac{1}{V} \sum_{r=1}^n \int_{V_r} C_r dV = \sum_{r=1}^n c_r C_r \\ \bar{C} \geq \left[\frac{1}{V} \sum_{r=1}^n \int_{V_r} S_r dV \right]^{-1} = \left[\sum_{r=1}^n \int_{V_r} c_r C_r^{-1} \right]^{-1} \end{array} \right. \quad (5.73)$$

The Voigt Reuss model based on variational method can effectively predict the elastic modulus of unidirectional long fiber composite materials along the fiber direction and transverse direction. The

variational method provides upper and lower limits for the equivalent stiffness of composite materials, which provides a standard for judging the correctness of the calculation results. The results of various multiscale calculations should be included in the upper and lower limits calculated by the variational method.

5.2.5 Comparison of different methods

Taking randomly distributed spherical hollow ($K_1/K_0=0$) and spherical rigid inclusions ($K_1/K_0=100$) as examples, analyze the calculation results of different uniformization methods mentioned above. The calculation results indicate that when the volume fraction of inclusions is small $c_r \rightarrow 0$, the results of the self-consistent method, generalized self-consistent method, differential method, and M-T method all tend towards the modulus of the matrix. From this, it can be seen that various analysis methods can predict the equivalent modulus of composite materials well when the inclusion content is low. The sparse method is only suitable for use with small inclusions, and when the volume content is large, the equivalent stress (strain) calculation results deviate significantly from the actual situation. The self-consistent method has the problem of decreased prediction accuracy when there is a significant difference in stiffness between the matrix phase and the reinforcing phase. Overall, the generalized self-consistent method, differential method, and M-T method can all provide ideal stiffness prediction results in the presence of low and high inclusions.

5.3 Failure criteria for composite materials

The strength theory of composite materials is the foundation and focus of studying the bearing capacity and damage failure of composite materials. Due to the strong assumptions of traditional failure criteria based on isotropic materials, it is impossible to distinguish and consider the strength differences in different directions of composite materials in practical situations. Moreover, traditional design methods rely on repeated experiments with long cycles and high costs, making them unable to be widely applied in the field of composite materials. In this situation, many failure criteria for engineering applications have been proposed one after another. Below are the failure criteria introduced separately.

(1) Maximum stress (strain) failure criterion

The maximum stress (strain) failure criterion states that for the three main directions studied in

composite materials, namely the stress (strain) components along the fiber direction, perpendicular to the fiber direction, and another orthogonal direction, as shown in Figure 4.3, as long as one of them reaches the critical value in that direction, the material will fail. The corresponding criterion is:

$$\max \left\{ \left| \frac{\sigma_1}{X} \right|, \left| \frac{\sigma_2}{Y} \right|, \left| \frac{\tau_{12}}{S_{12}} \right| \right\} = 1 \quad (5.74)$$

or

$$\max \left\{ \left| \frac{\varepsilon_1}{\varepsilon_1^u} \right|, \left| \frac{\varepsilon_2}{\varepsilon_2^u} \right|, \left| \frac{\gamma_{12}}{\gamma_{12}^u} \right| \right\} = 1 \quad (5.75)$$

where X , Y , and S_{12} are the ultimate strengths in the 1, 2 main directions and shear directions, corresponding to the tensile strength in each direction in the tensile state, and corresponding to the compressive strength in each direction in the compressive state. Similarly, ε_1^u , ε_2^u and γ_{12}^u correspond to the ultimate strains in the 1, 2 principal directions and shear directions respectively, distinguishing between tensile and compressive situations. The maximum stress (strain) criterion has the advantages of simple form and convenient use, but it ignores the interactive effects between different components, and usually predicts the strength of composite materials more accurately.

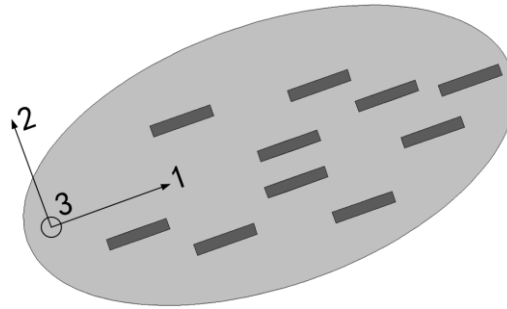


Figure 5.3 Definition of local coordinates.

(2) Tsai Hill failure criterion

The Tsai Hill failure criterion evolved from the Mises criterion, and the failure criterion is:

$$\frac{\sigma_1^2}{X^2} - \frac{\sigma_1 \sigma_2}{X^2} + \frac{\sigma_2^2}{Y^2} + \frac{\tau_{12}^2}{S_{21}^2} = 1 \quad (5.76)$$

Although the Tsai-Hill failure criterion considers the mutual influence of the three directional components of composite materials, it is only applicable to composite materials with equal or similar tensile and compressive strengths.

(3) Hoffman's failure criterion

In order to fully consider the failure behavior of composite materials under different tensile and compressive strengths, Hoffman proposed an improved form of Tsai-Hill failure criterion:

$$\frac{\sigma_1^2 - \sigma_1 \sigma_2}{X_T X_C} + \frac{X_C - X_T}{X_T X_C} \sigma_1 + \frac{\sigma_2^2}{Y_T Y_C} + \frac{Y_C - Y_T}{Y_T Y_C} \sigma_2 + \frac{\tau_{12}^2}{S_{21}^2} = 1 \quad (5.77)$$

where, X_T , X_C , Y_T and Y_C represent the tensile and compressive strength of composite materials in the two main directions, respectively. This failure criterion adds a primary term for stress, which allows for consideration of the unequal tensile and compressive strength of composite materials. When the tensile and compressive strengths are equal, the above equation degenerates into the Tsai-Hill failure criterion.

(4) Tsai-Wu Failure Criteria

In order to fully improve the prediction accuracy of composite material strength, the Tsai Wu failure criterion is proposed in a unified tensor expression:

$$F_1 \sigma_1 + F_2 \sigma_2 + F_{11} \sigma_1^2 + F_{22} \sigma_2^2 + F_{66} \tau_{12}^2 + 2F_{12} \sigma_1 \sigma_2 = 1 \quad (5.78)$$

Where, $F_1 = \frac{1}{X_T} - \frac{1}{X_C}$, $F_2 = \frac{1}{Y_T} - \frac{1}{Y_C}$, $F_{11} = \frac{1}{X_T X_C}$, $F_{22} = \frac{1}{Y_T Y_C}$, $F_{66} = \frac{1}{S_{21}^2}$, F_{12} cannot be determined

through experiments, and different determination methods can be referred to in references [189, 190].

The Tsai-Wu failure criterion is widely used due to its concise form and consideration of the interaction between stresses in all directions.

(5) Hashin Failure Criteria

The Hashin failure criterion was first proposed in 1980. The Hashin criterion is related to different types of failure modes (fiber, matrix, shear), each of which is associated with a different failure index defined by the ultimate stress function. In this model, the stiffness (constitutive matrix) of the damage point is reduced by considering the previously calculated failure mode index. The Hashin damage initiation criterion considers four possible failure modes and four corresponding indicators:

(i) fiber tensile fracture (F_f^c), (ii) fiber compression buckling (F_f^c), (iii) matrix tensile cracking (F_m^t), and (iv) matrix compression fracture (F_m^c). According to different failure modes, there are four types of failure modes: fiber tension, fiber compression, matrix tension, and matrix

compression. The values of each failure indicator are determined by solving the equations for each failure mode $F(\sigma/f)=1$. When the corresponding failure index is greater than or equal to 1, damage occurs in a specific mode. S_1^T and S_1^C is the longitudinal tensile strength and longitudinal compressive strength of direction 1, S_2^T , S_3^T and S_2^C , S_3^C is the transverse tensile strength and transverse compressive strength of directions 2 and 3, S_{12}^F , S_{13}^F and S_{23}^F is the shear strength in the corresponding plane. The specific criteria are as follows:

① Fiber tensile:

$$F_f^t(\sigma) = \frac{\sigma_{11}^2}{(S_1^T)^2} + \frac{\sigma_{12}^2 + \sigma_{13}^2}{(S_{12}^F)^2} \quad (5.79)$$

② Fiber compressive:

$$F_f^c(\sigma) = \frac{-\sigma_{11}}{S_1^C} \quad (5.80)$$

③ Matrix tensile:

$$F_m^t(\sigma) = \frac{(\sigma_{22} + \sigma_{33})^2}{(S_2^T)^2} + \frac{(\sigma_{23}^2 - \sigma_{22}\sigma_{33})}{(S_{23}^F)^2} + \frac{(\sigma_{12}^2 + \sigma_{13}^2)}{(S_{12}^F)^2} \quad (5.81)$$

④ Matrix compressive:

$$F_m^c(\sigma) = \frac{(\sigma_{22} + \sigma_{33})}{S_2^C} \left[\left(\frac{S_2^C}{2S_{23}^F} \right)^2 - 1 \right] (\sigma_{22} + \sigma_{33}) + \frac{(\sigma_{22} + \sigma_{33})^2}{4(S_{23}^F)^2} + \frac{(\sigma_{23}^2 - \sigma_{22}\sigma_{33})}{(S_{23}^F)^2} + \frac{(\sigma_{12}^2 + \sigma_{13}^2)}{(S_{12}^F)^2} \quad (5.82)$$

The Hashin failure criterion distinguishes the failure modes of fibers and matrix, while also considering the effects of different shear stresses (τ_{12} and τ_{13}) on fiber failure. After extensive research and verification, the Hashin failure criterion can reasonably reveal and describe the failure mechanism of fiber-reinforced composite materials, and the predicted results are highly consistent with the experimental results. This research adopts the 3D Hashin failure criterion for calculating the multi-scale performance of UHPC.

(6) Puck failure criterion

The mature Hashin failure criterion emphasizes that composite materials can be divided into fiber failure and matrix failure, that is, inter fiber failure. In the fiber failure mode, the maximum stress criterion is used for judgment, and in the matrix failure mode, a strength criterion expressed by the interaction of transverse stress and in-plane shear stress is established for analysis. Puck and

Schurmann revised and improved the Hashin failure criterion, namely the Puck failure criterion. The maximum stress criterion is still used for fiber failure, which is believed to be mainly caused by tensile stress parallel to the fiber direction. However, in the analysis of inter fiber failure, unlike the Hashin criterion, the Puck criterion does not use a global orthogonal coordinate system, but establishes a local coordinate system on the fracture surface, and considers the stress component of the cross-section as the main factor determining the fracture. And the matrix failure is divided into three different stress states, including transverse tension (type A), moderate transverse compression (mode B), and large transverse compression (mode C).

Assuming that directions 1, 2, and 3 respectively represent the fiber direction, the perpendicular fiber direction, and another orthogonal direction, then mode A considers the failure caused by the single or combined action of transverse tensile stress σ_n^+ or longitudinal shear stress τ_{21} in the cross-section. Due to the action of tensile stress, the fracture surfaces separate from each other. From a macro perspective, this will lead to a decrease in the elastic modulus and longitudinal shear modulus during compression. Mode B considers failure to be caused by longitudinal shear stress, which occurs on the plane where the shear stress acts. Compared with Mode A, the transverse normal stress acting together on the failure plane is compressive stress, and the crack will not open and the fracture surfaces will be squeezed together. Therefore, the stiffness decrease caused by the failure mode B between fibers is significantly lower than that of A. Failure mode B only occurs when the ratio of compressive stress to transverse compressive strength is less than 0.4. Mode C believes that if the ratio is greater than 0.4, failure will occur on a slope at a certain angle θ to the surface of the action by σ_{22} and τ_{21} . In the plane stress state, the increase in failure angle ranges from 0° in mode B to approximately 54° in mode C, and then to the pure compressive stress state ($\tau_{21} = 0$). Based on the above failure modes, the Puck failure criterion can be written in the following form:

① Fiber failure under the tensile stress:

$$\frac{1}{\varepsilon_{1T}} \left(\varepsilon_1 + \frac{\nu_{f12}}{E_{f1}} m_{\sigma f} \sigma_{22} \right) = 1 \quad (5.83)$$

② Fiber failure under the compressive stress:

$$\frac{1}{\varepsilon_{1T}} \left| \left(\varepsilon_1 + \frac{\nu_{f12}}{E_{f1}} m_{\sigma f} \sigma_{22} \right) \right| + (10\gamma_{21})^2 = 1 \quad (5.84)$$

③ Matrix failure under the transverse tensile stress:

$$\sqrt{\left(\frac{\tau_{21}}{S_{21}} \right)^2 + \left(1 - p_{vp}^+ \frac{Y_T}{S_{21}} \right)^2 \left(\frac{\sigma_{22}}{Y_T} \right)^2} + p_{vp}^+ \frac{\sigma_{22}}{S_{21}} + \frac{\sigma_{11}}{\sigma_{11D}} = 1 \quad (5.85)$$

④ Matrix failure under the moderate transverse compressive stress:

$$\frac{1}{S_{21}} \left(\sqrt{\tau_{21}^2 + (p_{vp}^- \sigma_{22})^2} + p_{vp}^- \sigma_{22} \right) + \frac{\sigma_{11}}{\sigma_{11D}} = 1 \quad (5.86)$$

⑤ Matrix failure under the large transverse compressive stress:

$$\left[\left(\frac{\tau_{21}}{2(1 + p_{vv}^-) S_{21}} \right)^2 + \left(\frac{\sigma_{22}}{Y_C} \right)^2 \right] \frac{Y_C}{(-\sigma_{22})} + \frac{\sigma_{11}}{\sigma_{11D}} = 1 \quad (5.87)$$

where, ε_{1T} and ε_{1C} are the tensile and compressive failure strains in the 1 direction, respectively; ε_1 is the normal strain in direction 1; ν_{f12} and E_{f1} are respectively the Poisson's ratio and elastic modulus of the fiber in direction 1; $m_{\sigma f}$ is the average stress amplification factor of fibers in direction 2; σ_{11} and σ_{22} are the normal stress in the orthogonal direction; γ_{21} and τ_{21} are respectively the shear strain and stress in the direction of elastic symmetry; S_{21} is the shear strength in the transverse and parallel directions of the fiber, p_{vp}^+ , p_{vp}^- and p_{vv}^- are parameters related to the angle of the fracture surface; σ_{11D} is a linearly degenerate stress value.

Although the Puck failure criterion is not as concise as the Hashin failure criterion, it provides information about the location of the failure surface, and the prediction results are relatively ideal, which has been widely applied. The failure criterion was adopted in the study of the influence of fiber orientation on the multi-scale properties of UHPC under tensile stress in Chapter 6.

5.4 Progressive failure theory

Progressive failure analysis refers to the analysis of the failure process in which the mechanical properties of materials gradually decrease, and is commonly used to explain the anisotropic damage and failure behavior of materials. After the failure index reaches the damage initiation threshold, the material degrades according to a progressive damage evolution model until complete failure occurs. Although the progressive damage model is also developed based on continuum damage mechanics,

it does not consider the initiation and propagation of individual macroscopic defects (cracks) in the material, but focuses on studying the influence of microscopic defects on the overall performance of the material.

5.4.1 Progressive damage model

As shown in Figure 4.4, considering a 2D planar object with micro voids on its surface. The presence of these micro voids reduces the object's ability to withstand external loads, which can also lead to lower apparent stresses inside the object under certain strain loads. In continuum mechanics, damage state variables are used to consider the phenomenon of stress reduction. For uniaxial tension, the effective stress $\hat{\sigma}$ and apparent stress σ are linked by the damage variable D :

$$\sigma = M \times \hat{\sigma} = (1 - D) \times \hat{\sigma} \quad (5.88)$$

where M is the damage tensor. The principle of strain equivalence states that strain is not affected by damage, so the stress-strain relationship between undamaged and damaged materials can be expressed as:

$$d\hat{\sigma} = \hat{C} : \varepsilon \quad (5.89)$$

$$\sigma = \mathbf{C}(\mathbf{D}) : \varepsilon \quad (5.90)$$

Where, $\hat{C}$ is the effective stiffness tensor, $\mathbf{C}(\mathbf{D})$ is the damaged stiffness tensor, and $\mathbf{D}$ is the damage variable. The damage effect tensor can be obtained as:

$$\sigma = M : \hat{\sigma} = (\mathbf{S}^{-1} : \mathbf{S}) : \hat{\sigma} \quad (5.91)$$

where, $\hat{\mathbf{S}}$ and $\mathbf{S}$ are the actual (after damage) flexibility matrix and the undamaged flexibility matrix, respectively.

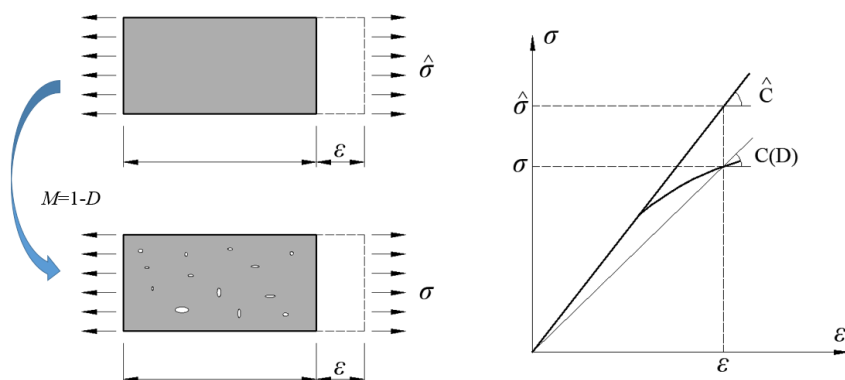


Figure 5.4 Effective stress and damage stress.

For the progressive failure modes that affect the secant stiffness of materials and reduce stress levels, the damage flexibility $\mathbf{S}(\mathbf{D})$ calculates the performance degradation of anisotropic materials by affecting the diagonal terms of the undamaged flexibility matrix $\mathbf{S}$:

$$S(D) = \begin{bmatrix} \frac{\hat{S}_{1111}}{1-D_{11}} & \hat{S}_{1122} & \hat{S}_{1133} & 0 & 0 & 0 \\ & \frac{\hat{S}_{2222}}{1-D_{22}} & \hat{S}_{2233} & 0 & 0 & 0 \\ & & \frac{\hat{S}_{3333}}{1-D_{33}} & 0 & 0 & 0 \\ & & & \frac{\hat{S}_{1212}}{1-D_{12}} & 0 & 0 \\ & sym & & & \frac{S_{2323}}{1-D_{23}} & 0 \\ & & & & & \frac{S_{1313}}{1-D_{13}} \end{bmatrix} \quad (5.92)$$

When damage occurs, the apparent Poisson's ratio becomes

$$\nu_{12} = \frac{S_{1122}}{S_{1111}} = \frac{\hat{S}_{1122}}{\hat{S}_{1111}} \times (1 - D_{11}) \neq \hat{\nu}_{12} \quad (5.93)$$

For the general cases, uniaxial tension can lead to the generation of non-zero transverse effective stress, so the damage tensor $\mathbf{M}$ is not diagonal.

5.4.2 Damage initiation and evolution criteria

Progressive failure uses a function (damage evolution index) corresponding to strain or equivalent stress to describe the initiation of material damage:

$$f = f(\varepsilon, \hat{\sigma}) = f_{ini} \quad (5.94)$$

The evolution of damage is represented by the law of damage evolution, and its differential form is:

$$\dot{D} = \phi(\dot{f}, f) \quad (5.95)$$

The mechanical properties allowed by thermodynamics require a range of damage variables from 0 to 1 and a non negative damage rate $\dot{D}$ at all times. The damage variable is calculated using the failure index function:

$$D = \max_{\tau \in [0, t]} \phi[f(\tau)] (\phi \in [0, 1]) \quad (5.96)$$

There are many forms of damage evolution functions, usually including linear functions, power-law functions, instantaneous damage functions, and exponential damage functions.

(1) Linear damage function

This evolutionary law requires four parameters, $f_{\min}$, $f_{\max}$, $D_{\max}$ and D_{final} to be defined, and the failure function form is:

$$\varphi(f) = \begin{cases} 0 & f < f_{\min} \\ D_{\max} \times \frac{f_{\max}}{f} \times \frac{f - f_{\min}}{f_{\max} - f_{\min}} & f_{\min} \leq f < f_{\max} \\ D_{\text{final}} & \text{other} \end{cases} \quad (5.97)$$

When $f_{\min} = 1$ and $D_{\max} = 1$, the evolution criterion becomes the bilinear damage evolution criterion.

(2) Power-law damage function

This evolutionary law requires the definition of five parameters: $f_{\min}$, $f_{\max}$, α , $D_{\max}$ and D_{final} . The failure function form is:

$$\varphi(f) = \begin{cases} 0 & f < f_{\min} \\ D_{\max} \times \frac{f^{\alpha} - f_{\min}^{\alpha}}{f_{\max}^{\alpha} - f_{\min}^{\alpha}} & f_{\min} \leq f < f_{\max} \\ D_{\text{final}} & \text{other} \end{cases} \quad (5.98)$$

when $f_{\min} = 0$, $\alpha = 1$, $D_{\max} = D_{\text{final}} = 1$, the power-law damage function degenerates into a linear damage function.

(3) Heaviside damage function

The evolution law requires the definition of two parameters, $f_{\min}$ and $D_{\max}$, and the failure function form is:

$$\varphi(f) = \begin{cases} 0 & f < f_{\min} \\ D_{\max} & \text{other} \end{cases} \quad (5.99)$$

This form of damage function corresponds to the brittle failure mode. When the failure index reaches $f_{\min}$, the constitutive curve shows a sudden drop and then continues to develop with a slope of $E \times (1 - D_{\max})$.

(4) Exponential damage function

This evolutionary law requires the definition of four parameters, $f_{\min}$, α , β and $D_{\max}$, and the failure function form is:

$$\varphi(f) = \begin{cases} 0 & f < f_{\min} \\ D_{\max} \times \left(1 - \exp\left(-\frac{f^{\alpha\beta} - f_{\min}^{\alpha\beta}}{e\beta}\right) \right) & \text{other} \end{cases} \quad (5.100)$$

When $f_{\min} = 0$, $\alpha = 1$, $D_{\max} = 1$, the exponential damage function corresponds to the famous Weibull stress-strain curve. For the multi-scale calculation of the mechanical properties of UHPC in this chapter, the exponential damage evolution function was adopted.

5.5 Establishment of the multiscale numerical model

Based on the Mori Tanaka homogenization method, the mechanical parameters of UHPC matrix with different coarse aggregate contents were first predicted. The coarse aggregate particles were icosahedral with a particle size of 4 mm. After the first step of equivalence, considering the influence of fiber content, a cylindrical fiber inclusion with a diameter of 0.2 mm and a length of 13 mm was added to the generated first step RVE, and a continuous progressive damage model was coupled to determine the constitutive model of UHPC under different fiber contents. Based on the calculation results, generate an applicable Jacobian matrix from the obtained UHPC constitutive relationship. Finally, the generated Jacobian matrix is transformed into the UMAT subroutine of ABAQUS to analyze the mechanical properties of UHPC under different loads at a macro level. The analysis and establishment process of the numerical model is shown in Figure 5.5.

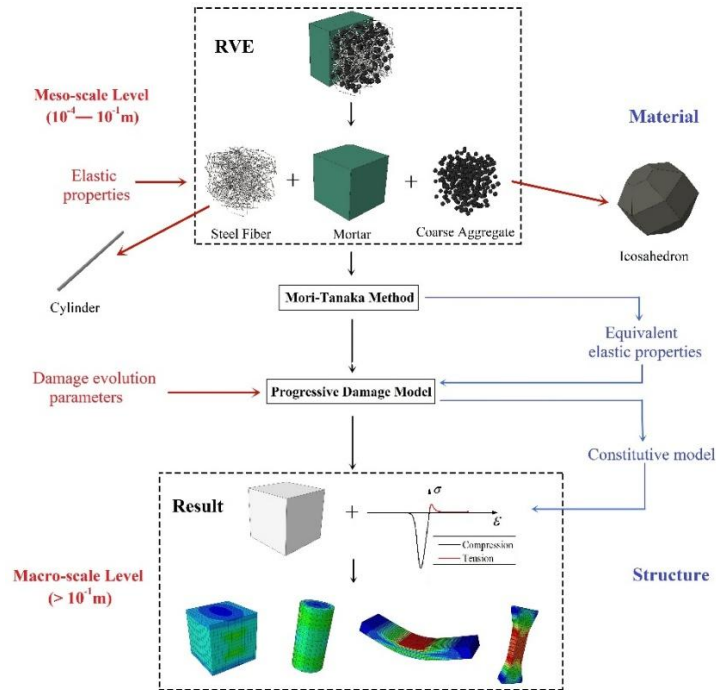


Figure 5.5 Multi scale analysis process of UHPC mechanical behavior.

Coupling the micro and macro performance of UHPC to achieve multi-scale simulation analysis. The mechanical performance parameters of UHPC phases are shown in Table 5.1, and the multi-scale calculation parameters are shown in Table 5.2. The geometric dimensions of the numerical model are the same as the actual component dimensions. After meshing the finite element model, the element is defined as a reduced integral 8-node continuous brick element (C3D8R), and an enhanced hourglass control is used to limit the expansion of the hourglass mode. Use ABAQUS/Standard to calculate and analyze the stress process of UHPC. Using displacement control loading, create reference points coupled with boundary motion, and apply corresponding displacement/constraint boundary conditions to the reference points.

Table 5.1 Mechanical Performance Parameters of UHPC Components

	Modulus (GPa)	Possion's ratio	Density (kg/m3)
Matrix	45.0	0.20	2460
Aggregate	70.0	0.20	2890
Fiber	200	0.30	7800

Table 5.2 UHPC Multi-scale Calculation Parameters

Strength Parameter (direction)	Symbol	Strength (MPa)
Longitudinal Tensile (1)	S_1^T	8.5
Longitudinal Compressive (1)	S_1^C	145.0
Transverse Tensile (2, 3)	$S_2^T = S_3^T$	8.5
Transverse Compressive (2, 3)	$S_2^C = S_3^C$	145.0
Shear (1-2, 1-3)	$S_{12}^F = S_{13}^F$	49.5
Shear (2-3)	S_{23}^F	49.5

5.6 Multi scale performance prediction and analysis of UHPC

5.6.1 Compression performance of UHPC

Figures 5.6 (a) and 5.6 (b) show the compression test and numerical simulation results of UHPC with FA-2 and CA-1.5 mix proportions, respectively. The results showed that the average compressive strength of FA-2 was 135.5 MPa, while the simulation result was 139.2 MPa, with an error of 2.7%; The average compressive strength of CA-2 is 144.3 MPa, with a simulation result of 147.1 MPa and an error of 1.9%. It can be seen that the compression stress-strain curve of numerical simulation is highly consistent with the experimental data. The multi-scale damage model and

parameter selection used in this research are effective for simulating the compression process of UHPC with (or without) coarse aggregates. The main source of error in multi-scale calculations is the deviation between real material parameters and calculated parameters, generally, this error is acceptable. Usually, under compressive loads, concrete has three cracking locations: a) in the matrix, b) in coarse aggregates, and c) at the interface between the matrix and aggregates. For UHPC, the strength of hardened paste and ITZ has been significantly improved, which is considered to be higher than the inherent strength of coarse aggregates. The compression failure mode in the experiment also indicates that the fracture surface will not only propagate through the hardened slurry or ITZ, but also through the aggregate. Due to the reinforcing effect of coarse aggregate, the average compressive strain corresponding to the peak compressive load of CA-1.5 UHPC is 0.00395, which is lower than the average compressive strain corresponding to the peak compressive load of FA-2, which is 0.00490.

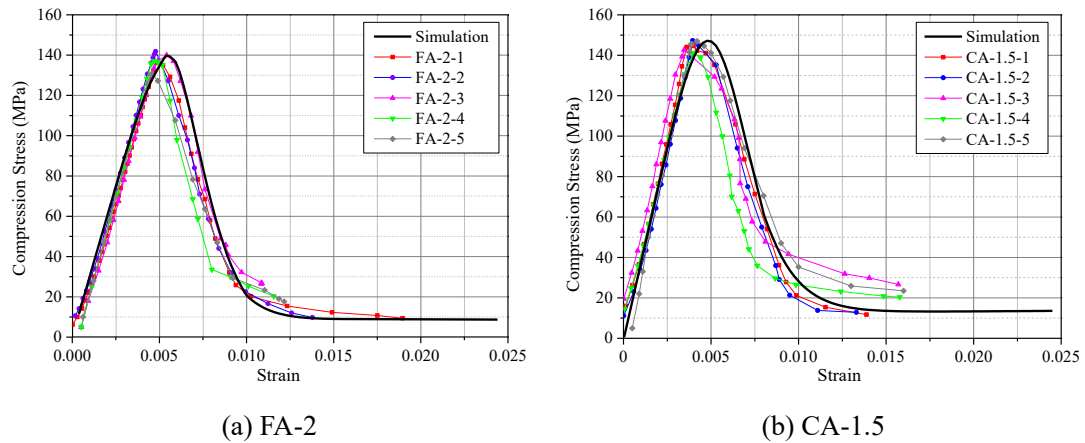


Figure 5.6 Comparison between UHPC compression test and numerical simulation.

The elastic modulus of UHPC specimens with different coarse aggregate and steel fiber contents is shown in Figure 5.7. The results indicate that with the increase of basalt coarse aggregate and steel fiber content, the elastic modulus of UHPC continues to increase. When the steel fiber content changes from 0% to 5%, the elastic modulus of the specimens corresponding to 0% CA to 50% CA increases by 13.39%, 12.55%, 11.61%, 10.61%, 9.56%, and 8.43%, respectively.

The content of coarse aggregate changed from 0% to 50%, and the elastic moduli corresponding to 0% SF to 50% SF increased by 22.03%, 19.88%, 18.95%, 18.12%, 17.39%, and 16.69%, respectively. Due to the high stiffness of coarse aggregates and the good continuity of particle skeleton, the elastic modulus of UHPC can be effectively improved. Although the elastic modulus

of steel fibers is much larger than that of coarse aggregates (about three times), due to their large geometric aspect ratio, they can only exert their maximum effect when subjected to force along the fiber direction. Comparatively speaking, coarse aggregates have a more significant increase in the elastic modulus of UHPC.

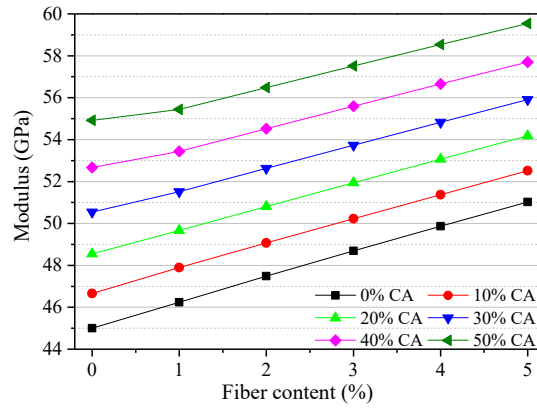
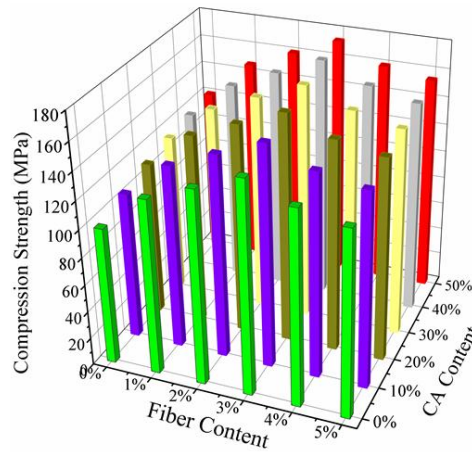


Figure 5.7 Elastic modulus of UHPC with different basalt aggregates and fiber contents.

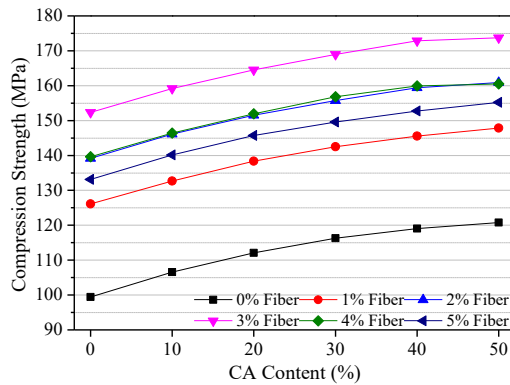
The compressive strength of UHPC specimens with different coarse aggregate and steel fiber contents is shown in Figure 5.8. As shown in the figure, the content of coarse aggregate and steel fiber has a significant impact on the compressive strength of UHPC. Figures 5.8 (a) and (b) indicate that with the same SF content, the compressive strength increases continuously with the increase of CA content, but the growth rate decreases continuously. The average compressive strength of UHPC with 10% CA is 138.51 MPa, which is 6.87 MPa higher than the average compressive strength of UHPC without coarse aggregate (0% CA) at 131.64 MPa, with a strength improvement rate of 5.2%. The average compressive strength of 50% CA UHPC is 153.17 MPa, which is 1.58 MPa higher than the average compressive strength of 151.59 MPa of 40% CA UHPC, with a strength improvement rate of 1.0%. This is because with the increase of basalt coarse aggregate content, although the strength of UHPC is improved, the size of the internal aggregate matrix interface area (ITZ) of UHPC is also gradually increasing. ITZ is an important reason for the decrease in damage strength of UHPC.

Figure 5.8 (c) shows that with the same CA content, the compressive strength of UHPC increases first and then decreases with the increase of SF content, with the highest compressive strength observed at a 3% content. The average compressive strength of 1% SF UHPC is 138.83 MPa, which is 26.47 MPa higher than the average strength of 112.36 MPa of 0% SF UHPC, with a strength improvement rate of 23.5%; The average compressive strength of 2% SF UHPC is 13.31 MPa higher

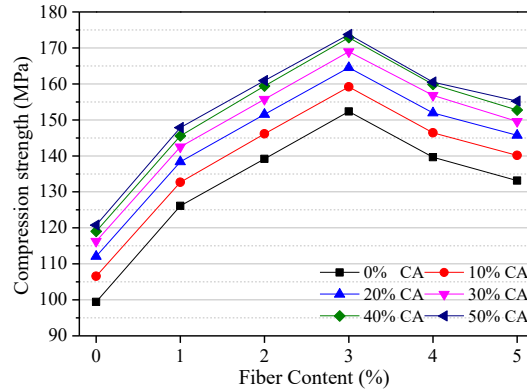
than that of 1% SF UHPC, with a strength improvement rate of 9.6%; The average compressive strength of 3% SF UHPC is 13.11 MPa higher than that of 2% SF UHPC, with a strength improvement rate of 8.6%. When the steel fiber content exceeds 3%, the strength begins to decrease. This is because when the fiber content is too high, the steel fibers inside UHPC will agglomerate, directly leading to a decrease in strength. The above analysis shows that when the steel fiber content is increased from 0% to 1%, the efficiency of improving the strength of UHPC is the highest. When the steel fiber content is around 3%, the fiber spatial distribution inside the specimen reaches saturation. Increasing the fiber content further will result in a continuous decrease in strength.



(a) Distribution of compressive strength.



(b) Compressive strength of UHPFRC with different CA content.



(c) Compressive strength of UHPFRC with different fiber content.

Figure 5.8 Compressive strength of UHPFRC with different CA and fiber content.

5.6.2 Tensile properties of UHPC

The tensile stress-strain curves of UHPC with different mix proportions are shown in Figure 5.9. It can be seen that the tensile stress-strain curve of the same mix proportion is basically consistent with the numerical simulation results. When the tensile stress reaches its peak, the tensile specimen

fails. The difference between the peak load of FA-1 tensile simulation at 9.25 MPa and the average peak load of tensile test at 8.75 MPa is 5.7%; The peak load of FA-2 tensile simulation is 11.82 MPa, and the average peak load of tensile test is 11.36 MPa, which is 4.0%; The peak load of CA-0.75 tensile simulation is 7.17 MPa, and the average peak load of tensile test is 6.83 MPa, which is 4.9%; The peak load of CA-1.5 tensile simulation is 10.91 MPa, and the average peak load of tensile test is 10.40 MPa, which is 4.9%.

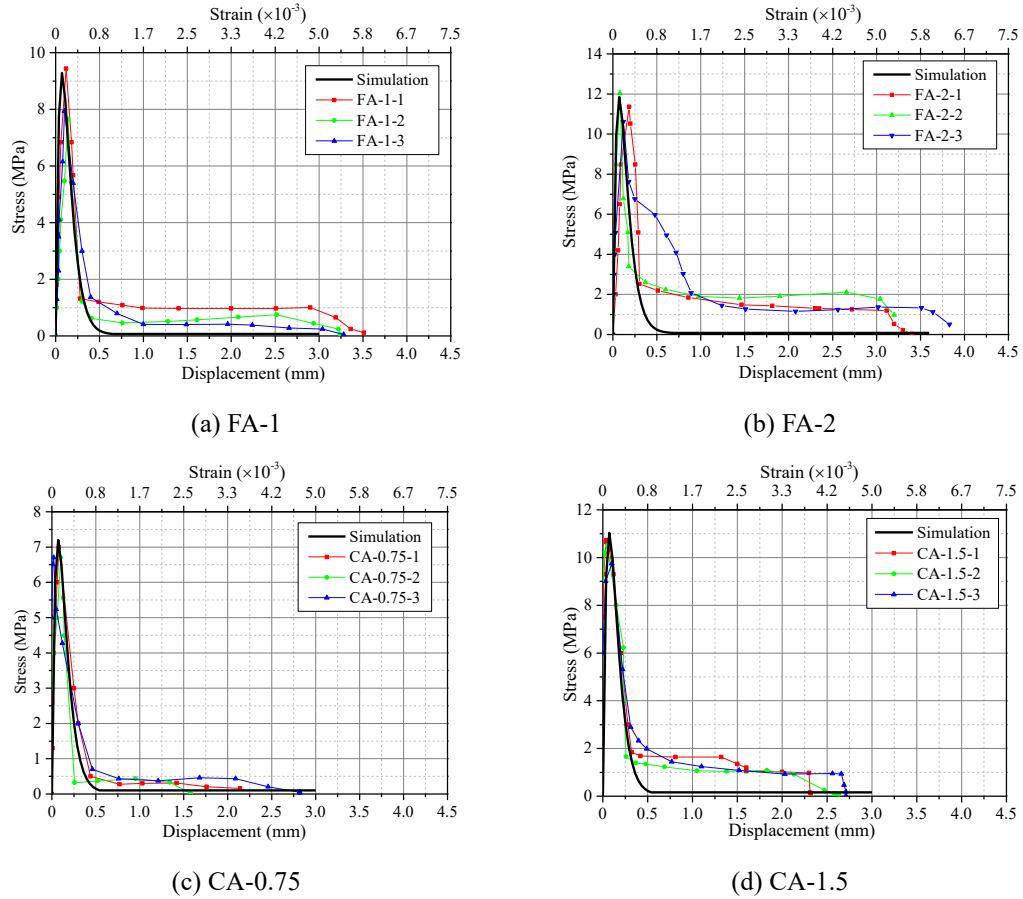


Figure 5.9 Comparison between UHPC tensile test and numerical simulation.

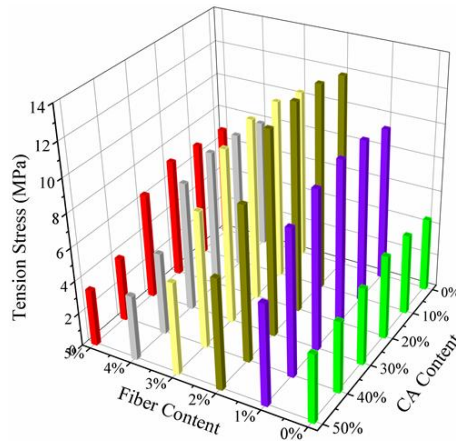
In the study of Li ^[191] indicates that the occurrence of dispersed tensile cracks during the tensile process of UHPC depends on the strength of the first crack, which in turn depends on the size of the crack. Comparing FA-1 (Figure 5.9 (a)) and FA-2 (Figure 5.9 (b)) without coarse aggregate, it was found that the average residual stress after tensile failure of FA-2 (taking the stress value corresponding to a tensile displacement of 2.5 mm) was 0.98 MPa, which was greater than FA-1's 0.57 MPa. Similarly, when the tensile displacement of the two types of UHPC containing coarse aggregates is 2.0 mm, the average residual tensile stress value of CA-1.5 (Figure 5.9 (d)) is 0.72 MPa, which is greater than the 0.27 MPa of CA-0.75 (Figure 5.9 (c)).

Like ordinary concrete, UHPC does not undergo plastic deformation before cracking, but exhibits elastic tensile behavior before reaching linear ultimate strain. It is generally believed that the elastic modulus of ultra-high performance concrete in tension is the same as that in compression. The fibers play an anchoring role after the matrix cracks, and the stress of UHPC does not immediately become zero after reaching the linear maximum strain. The tensile force borne by the matrix and steel fibers before cracking is borne by the steel fibers after cracking. As the crack size increases, the steel fibers are gradually pulled out, and the tensile strength of the material decreases, approaching failure. Due to the high strength characteristics of steel fibers, the matrix of UHPC often fails earlier, as the bond strength between the cementitious material and fibers also reaches its ultimate value before crack development. The strength, aspect ratio, volume fraction, and shape of steel fibers can all lead to different tensile constitutive relationships of UHPC. According to the research results on the tensile properties of UHPC, Naaman ^[192] classified the tensile properties of UHPC materials into three types: strain hardening, low strain hardening, and strain softening. Meanwhile, based on the mechanical properties of UHPC strain hardening and bending hardening, the critical fiber content for strain hardening and bending hardening were derived, and it was found that the critical fiber content for bending hardening was lower than that for strain hardening. The results show that: (1) ultra-high performance concrete materials with strain hardening characteristics also have bending hardening characteristics; (2) Ultra high performance concrete materials with strain softening have both flexural hardening and flexural softening properties; (3) Compared with strain softening and bending softening, UHPC with strain hardening and bending hardening characteristics is more suitable for practical engineering.

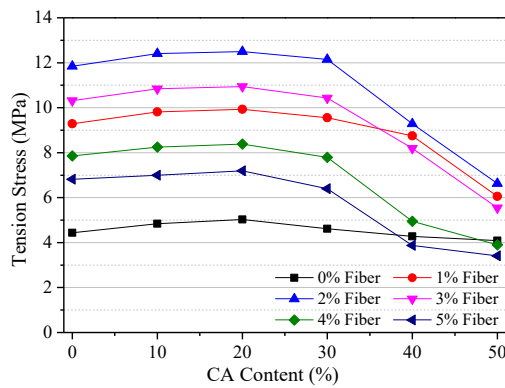
The direct tensile strength results of UHPC with different coarse aggregate and steel fiber contents are shown in Figure 5.10. As shown in the figure, the CA and SF contents also have a significant impact on the tensile strength of UHPC. Unlike compressive strength (Figure 5.8 (b)), Figure 5.10 (b) shows that with the same SF content, the tensile strength does not increase continuously with the increase of CA content, but shows a certain slow increase followed by a rapid decrease. The tensile strength corresponding to 20% CA reaches its maximum.

The variation of CA content in UHPC without fiber (0% SF) is slightly different from that of UHPC with fiber. Due to the lack of bridging stress of steel fibers, its tensile strength is lower, and the change in CA content has little effect on its tensile strength. The maximum tensile strength value

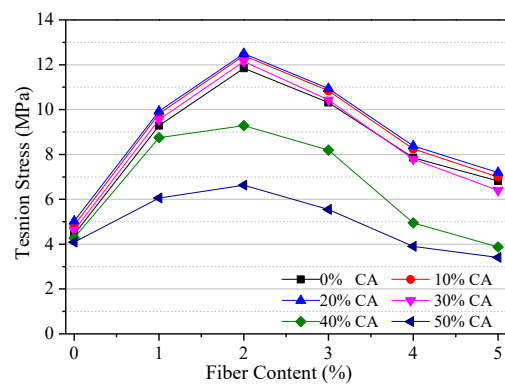
of UHPC with 20% CA content is 5.03 MPa, which is 0.59 MPa higher than the tensile strength of UHPC with 0% CA content, which is 4.44 MPa. When the CA content is 20%, the tensile strength of UHPC with 1% to 5% SF is 0.64 MPa, 0.66 MPa, 0.63 MPa, 0.53 MPa, and 0.37 MPa higher than the corresponding 0% CA strength, respectively. When the CA content is greater than 30%, the tensile strength decreases sharply, and the tensile strength of UHPC with 5% SF content is lower than that corresponding to 0% SF. This is because coarse aggregates can cause damage to the microstructure of UHPC, leading to a decrease in the bonding stress between fibers and matrix. Meanwhile, as the content of coarse aggregate further increases, there will be severe interlocking between fibers and coarse aggregate, making it difficult to evenly disperse the fibers. Therefore, the microstructure of UHPC specimens is more complex.



(a) Distribution of tension strength.



(b) Tension strength of UHPFRC with different CA content.



(c) Tension strength of UHPFRC with different fiber content.

Figure 5.10 Tension strength of UHPFRC with different CA and fiber content.

Figure 5.10 (c) shows the effect of changes in SF content on the tensile strength of UHPC when the CA content is the same. With the increase of SF content, the tensile strength of UHPC also shows

a similar change pattern as shown in Figure 5.8 (c), that is, the strength will first increase and then decrease. Unlike compressive strength, the peak tensile strength of UHPC occurs at a fiber content of about 2%. When the fiber content increases from 0% to 1%, the tensile strength of UHPC corresponding to 0% to 50% increases by 109.2%, 102.6%, 97.4%, 106.7%, 104.4%, and 48.2%, respectively; When the fiber content increases from 1% to 2%, the tensile strength of UHPC corresponding to 0% to 50% increases by 27.4%, 26.5%, 25.8%, 27.2%, 6.2%, and 9.4%, respectively. This also indicates that the addition of fibers is extremely important for improving the tensile strength of UHPC, and increasing the fiber content within a reasonable range will greatly enhance the tensile performance of UHPC.

5.6.3 Bending performance of UHPC

5.6.3.1 Bending strength analysis

Figure 5.11 compares the experimental results and numerical simulation results of the four point bending load deflection curve of UHPC board. As shown in the figure, the numerical simulation results are highly consistent with the experimental results for the four mix proportions, and the load deflection curve test results for each mix proportion envelop the numerical simulation results. The average peak loads of the four point bending test results of UHPC plates with FA-1, FA-2, CA-0.75, and CA-1.5 ratios were 7.90 kN, 7.93 kN, 8.28 kN, and 8.32 kN, respectively. The corresponding numerical simulation peak loads were 7.98 kN, 7.86 kN, 8.37 kN, and 8.14 kN, with a maximum error of 2.2%. This indicates that the progressive damage model can accurately predict the four point bending failure process of UHPC plates under different microscopic compositions. UHPC is similar to ordinary concrete. According to the evolution law of its bearing capacity after the initial crack load, the typical bending mechanical behavior curve of UHPC includes two types: deflection hardening type and deflection softening type. In order to better study the influence of different fiber contents on the bending mechanical properties of UHPC boards, according to ASTM ^[193], international scholars have proposed evaluation indicators mainly based on linear stage proportional limit and bending ultimate strength parameters. When the ultimate bending strength is greater than the proportional ultimate strength in the linear stage, UHPC exhibits a deflection hardening type, while conversely, it exhibits a deflection softening type. After the flexural softening of UHPC reaches its linear limit, the load immediately decreases and failure occurs; After the linear limit point, UHPC with flexural hardening will undergo a significant flexural hardening process, and the load

that the component can withstand can continue to increase, but the growth rate is relatively slow. After reaching the ultimate load, the position of the main crack is determined and further expanded, thus entering the softening section. The flexural hardening characteristics of UHPC give it good strength and ductility. In practical engineering applications, pure tensile members are relatively rare, while most are flexural members. Therefore, UHPC with flexural hardening characteristics is of great significance for flexural members. The four point bending test results of UHPC board show that the fiber bridging effect causes the bending ultimate strength value to be greater than the proportional ultimate load value in the linear stage, and the fiber toughening effect is significant, which has a good improvement effect on the performance of UHPC.

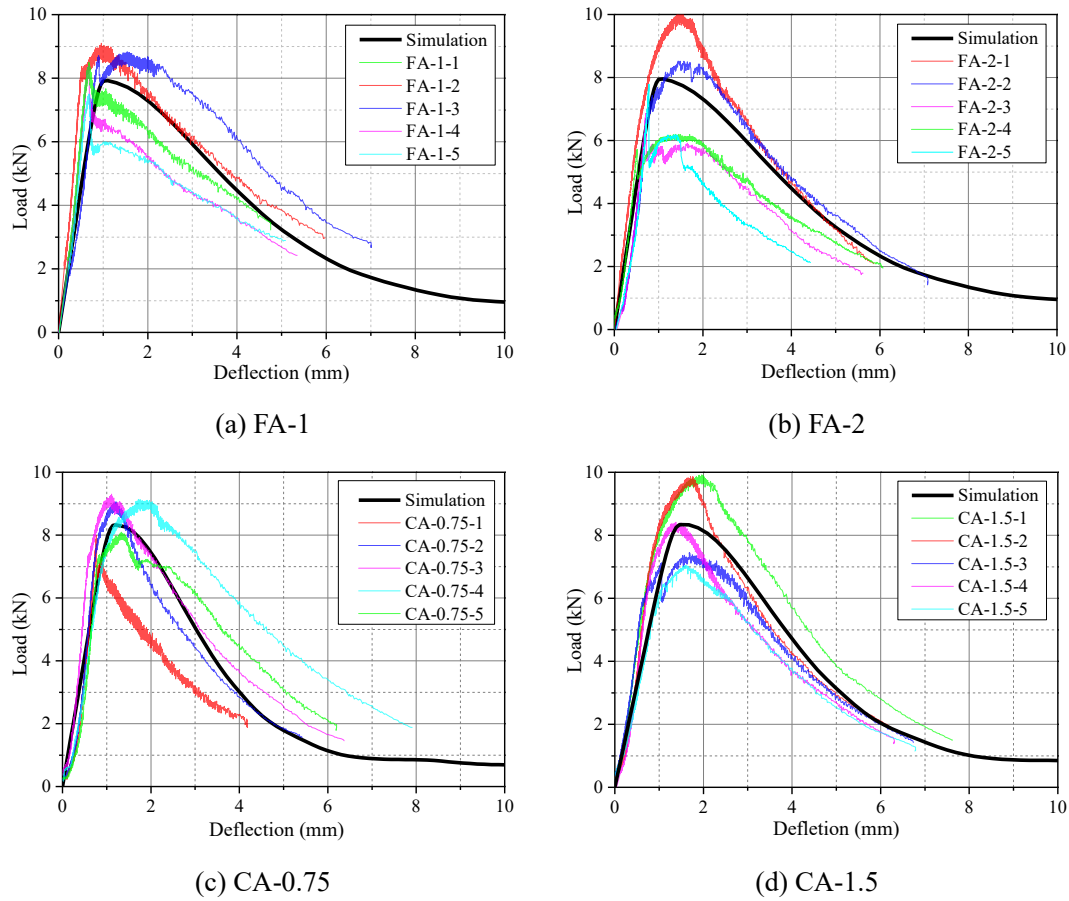
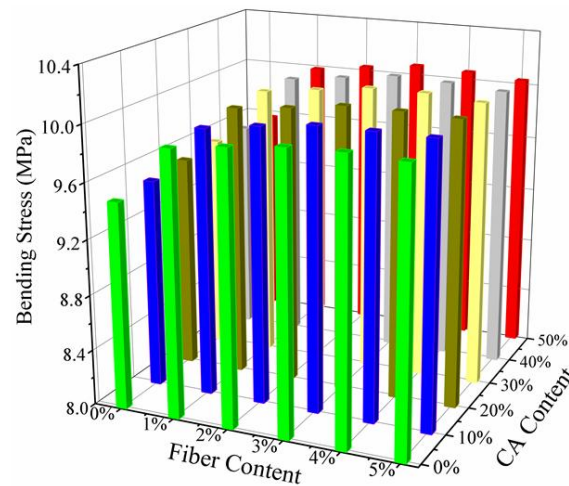


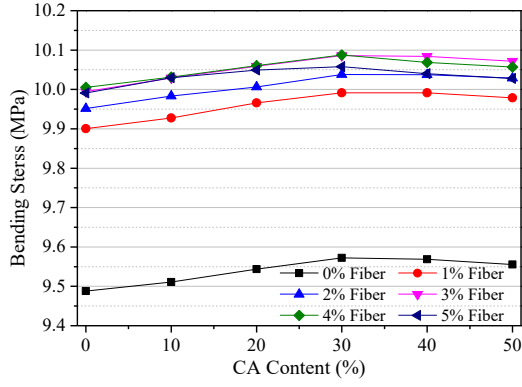
Figure 5.11 Comparison between UHPC bending test and numerical simulation.

The influence of different fiber and coarse aggregate contents on the bending strength of UHPC boards is shown in Figure 5.12. From Figures 5.12 (a) and (b), it can be seen that the bending strength of UHPC without steel fibers is significantly lower than that of UHPC with steel fibers added. UHPC with the same fiber content shows a trend of first increasing and then decreasing with the increase of coarse aggregate content, but the strength change is relatively small. When the

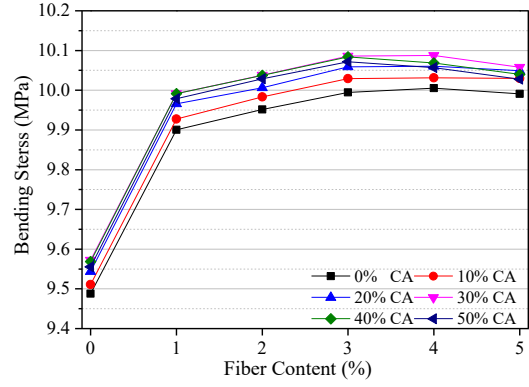
amount of coarse aggregate is small, the UHPC matrix cementitious material is relatively more abundant, the distribution of fibers is more uniform, and the effective utilization rate of fibers is higher. The bonding force between the UHPC matrix and steel fibers still plays a significant role. At the same time, the interlocking effect between coarse aggregates hinders the development of cracks, which can to some extent compensate for the negative effect of the decrease in fiber utilization caused by the addition of coarse aggregates. When the CA content reaches 30%, the bending stress of the thin plate with 0% to 5% fiber content is the highest at a CA content of 30%, which is 0.07 MPa to 0.09 MPa higher than the bending stress of UHPC with 0% CA. After the CA content exceeds 30%, the reason for the decrease in bending stress is also due to the sharp decline in the workability of UHPC, uneven fiber distribution, severe agglomeration of steel fibers, and the reduction of cementitious materials, which leads to a decrease in the bonding effect between fibers and matrix, thereby reducing the bending performance of UHPC. The effect of fiber content on the bending stress of UHPC thin plates is shown in Figure 5.12 (c). As the fiber content increases, the bending stress also follows a pattern of first increasing and then decreasing, which is similar to the effect of fiber content on the tensile properties of UHPC.



(a) Distribution of bending strength.



(b) Bending strength of UHPFRC with different CA content.



(c) Bending strength of UHPFRC with different fiber content.

Figure 5.12 Bending strength of UHPFRC with different CA and fiber content.

5.6.3.2 Bending toughness analysis

The toughness of material is the capability of absorbing energy under load until failure, which is usually expressed as the area surrounded by stress-strain curve or load-deflection curve, also known as toughness (R). In this study, when the deflection is 5 times of the peak load corresponding to the deflection, the area surrounded by the load-deflection curve is taken as the toughness, and its expression is as follows:

$$R = \int_0^{\delta} F dx = \int_0^{5\delta_{pk}} F dx \quad (5.101)$$

where R is the toughness, F is the load, and δ_{pk} is the deflection corresponding to the peak load.

Taking the peak load point of the bending load-displacement curve of UHPC slab as the boundary, the front of the peak point is the stage of stable crack growth, and the area enclosed by the load-displacement curve and the coordinate axis is called the pre-crack toughness; the back of the peak point is the process from the unstable development of the micro-crack to the obvious crack failure, and the area enclosed by the load-displacement curve and the coordinate axis is called the post crack toughness. The total toughness, pre-peak toughness and post peak toughness of the bending load-deflection curve of UHPFRC plates with different coarse aggregate content and fiber volume fraction are calculated, and the results are shown in Figure 5.13 and Figure 5.14.

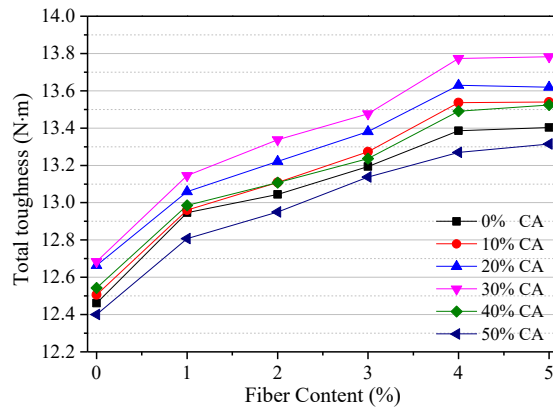


Figure 5.13 Bending toughness of UHPC slab.

It can be seen from Figure 5.13 that the total bending toughness of UHPC slabs increases with the increase of fiber volume fraction, and the changing regularity is similar to that of bending strength (Figure 5.14). The higher the fiber volume fraction, the larger the area surrounded by the load deflection curve, that is, the greater the toughness of the specimen. The total toughness of UHPFRC with 5 % fiber is 7.55 % ~ 8.66 % higher than that with 0 %. It can be seen that the addition of steel fiber effectively increases the toughness of the specimen. After the UHPFRC specimen has micro-cracks, the released deformation energy is first used for fiber debonding instead of supporting the crack to continue to expand, so the fracture process is delayed and the toughening effect is achieved. When the fiber volume fraction reaches 4 %, the whole effect of the fiber in the specimen is weakened, which results in the growth rate of total toughness value to slow down or basically no longer increase.

The pre-peak toughness calculation results are shown in Figure 5.14 (a). The difference in pre peak toughness of UHPC under different coarse aggregate and fiber contents is small, and the influence of coarse aggregate and fiber content on it is not significant. The reason can be attributed to the fact that in the stage before peak load, although the fiber volume fraction and aggregate content have a certain influence on the modulus of UHPC, the difference in load deflection curve area is not significant. Therefore, the overall range of change in toughness before peak load is small, and the change pattern is not obvious. The post peak toughness (also known as post fracture toughness) is shown in Figure 5.14 (b).

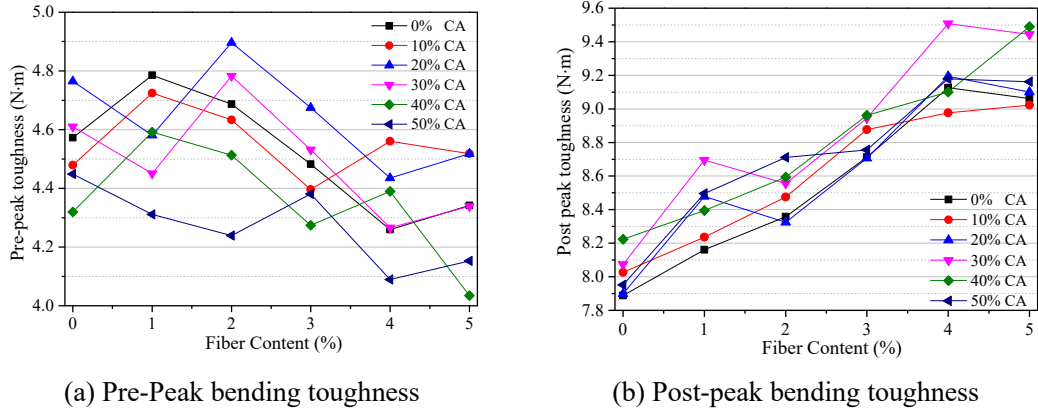


Figure 5.14 Relationship between pre-peak and post-peak bending toughness of UHPC slab.

When considering the toughness change of UHPC components under load, the deformation performance, that is, the ductility also should be considered. Ductility refers to the ability of components to resist deformation after reaching the ultimate bearing capacity. The ductility of UHPC should be considered in order to reduce the brittle failure under external forces, which is of great significance to the safety of structures. The ductility of concrete slab or beam can be characterized by displacement ductility index (μ_δ) and curvature ductility index ($\mu_{1/r}$). The curvature ductility index is the ratio of the curvature corresponding to the ultimate moment $(1/r)_u$ to the yield tensile curvature $(1/r)_y$. The calculation method of displacement ductility index is similar, but vertical displacement of mid-span section is used instead of curvature. For ductility analysis, ultimate moments or loads are defined when the structure does not have more capacity to support the load. In this paper, the displacement ductility index is used for ductility analysis, and the final stage load value is defined as 75 % of the maximum load in the post peak curve.

Figure 5.15 show the relationship between ductility index and fiber volume fraction and CA content, respectively. it shows that with the fiber volume fraction in the range of 0 %~5 %, the displacement ductility index increases with the increase of fiber volume fraction, and the ductility index of 5 % fiber increases by 12.29 %~18.90 % compared with the ductility index of 0 % fiber, which indicates that the bending ductility deformation performance of UHPFRC slab has been improved. The UHPC slab has good deformation performance after cracking, and has greater ductility near the failure load. Figure 5.15 shows that with the increase of CA content, the bending ductility of UHPC slab changes little, which is because the main factor for the improvement of ductility performance is the bridging effect of fiber after cracking, while the coarse aggregate affects the uniform distribution of fiber in the component, and the interlock of fiber and aggregate makes the internal stress state of UHPC more complex, therefore, the improvement of the ductility performance by coarse aggregate is not obvious.

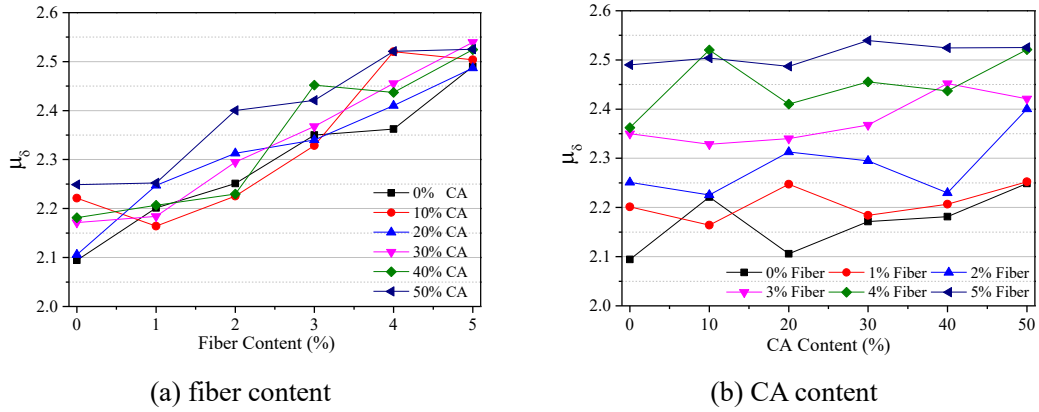


Figure 5.15 Relationship between the ductility with fiber and aggregate content.

5.7 Conclusion

In this paper, the mechanical properties of UHPFRC with various coarse aggregate contents and fiber volume fraction were investigated by a series of tests and multi-scale numerical simulation. The following conclusions can be drawn:

(1) A multi-scale analysis method of UHPFRC is established. At the meso-scale level, the mesoscopic mechanical properties of UHPFRC are equivalent by Mori-Tanaka homogenization method. Then the three-dimensional Hashin failure criterion and exponential damage evolution function, which are widely used in fiber-reinforced composites, are assigned to RVE. Finally, the mechanical properties of RVE are input into the finite element numerical model of UHPFRC on the macro-scale to analyze its damage and failure process under different loads, so as to couple the mesoscopic and macroscopic properties of UHPFRC and realize the multi-scale simulation analysis.

(2) Comparing the results of UHPFRC finite element simulation with the test results, it is found that the load (stress) - displacement (strain) curves of compression, tension and four-point bending are in good agreement with the corresponding test results. The maximum errors between the mean peak stress of compression, tension and four-point bending tests and the corresponding numerical simulation results are 2.7 %, 5.7 % and 2.2 %, respectively. The feasibility of using multi-scale simulation to analyze the damage and failure behavior of UHPFRC is verified.

(3) With the increase of steel fiber (0 % ~ 5 %) and coarse aggregate (0 % ~ 50 %), the tensile strength, compressive strength and bending strength of UHPFRC increase first and then decrease. Therefore, in order to improve the application level of UHPFRC in practical engineering, the steel fiber volume fraction should be controlled at 2 % ~ 3 % and the coarse aggregate content should not be more than 30 %.

(4) The results of four-point bending of UHPFRC slabs show that the total toughness of 5 % fiber volume fraction is 7.55 % ~ 8.66 % higher than that of 0 %, and the post peak toughness by 12.41 % ~ 16.97 %. The ductility index of 5% fiber volume fraction is 12.29 % ~ 18.90 % higher than that of 0 %. Compared with the coarse aggregate, the increase of fiber has a more obvious strengthening and toughening effect on the bending toughness and post cracking ductility of UHPFRC, which can effectively improve its mechanical properties after cracking.

Chapter 6 Multi-scale Analysis on the Tensile Properties of UHPC

6.1 Overview

The most significant feature that distinguishes UHPC from normal concrete is its higher tensile strength and tensile hardening behavior, which are attributed to the presence of internal fibers. In addition to the type, volume content, geometric size, and shape of fibers, fiber orientation characteristics also have a significant impact on the tensile properties of UHPC. This chapter establishes a multi-scale relationship between fiber orientation and macroscopic mechanical parameters at the microscale level based on the orientation distribution characteristics of fiber distribution in UHPC. Meanwhile, the influence of fiber orientation characteristics on the tensile mechanical properties of UHPC was described and evaluated using fiber orientation tensor. Establish a progressive damage analysis model for UHPC tensile testing considering fiber orientation, based on the Puck failure criterion. Analyzing the fiber distribution characteristics near and far from the UHPC tensile fracture surface, revealing the influence of fiber orientation on the UHPC tensile fracture process. On this basis, the tensile strength of UHPC with different fiber contents and orientation tensors was predicted. After comparing with the experimental results, the feasibility of this analysis method for UHPC was verified.

6.2 Description of fiber orientation

6.2.1 Description of the orientation tensor

The tensor description of the fiber orientation characteristics of composites is given. In the reconstruction of ODF, lower even-order tensor included the higher even-order tensor, and the reconstructed ODF was obtained by the closure approximation. Furthermore, the mean-field homogenization performance of UHPC was calculated by the two-step homogenization method coupling M-T model and Voigt-Reuss model. Finally, combined with Puck failure criterion and exponential damage evolution criterion, the tensile strength of UHPC can be predicted.

For a two-phased composite material, it is usually assumed that all the inclusions share the same geometric size and arrangement direction. For cement-based short fiber reinforced composites like UHPC, the fiber orientation distribution may change greatly during the casting process. During the homogenization process, fibers in a single RVE are also distributed randomly and irregularly. Therefore, the ODF have to be reconstructed with the proper descriptions of fiber orientation before

the homogenization.

In the spherical coordinate system, the direction of the single inclusion can be described by the unit vector $\mathbf{p}$, as shown in Figure 6.1. θ is the angle between projection in x_1 - x_2 plane and the coordinate axis x_1 , and φ is the inclination angle between the vector $\mathbf{p}$ and the axis x_3 .

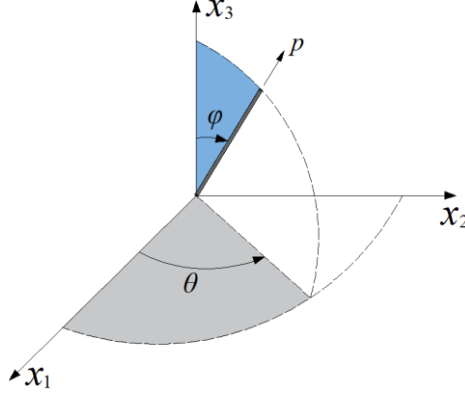


Figure 6.1 Vector diagram of the single fiber.

The ODF $\psi(\mathbf{p}, \mathbf{x}, t)$ is the orientation probability density function of parameters $\mathbf{p}$, $\mathbf{x}$ and t , where $\mathbf{x}$ represents the axis direction and t represents time. $d\mathbf{p}$ is an element of area of the unit sphere U . Due to axial symmetry of $\mathbf{p}$, it can be obtained that

$$\psi(\mathbf{p}) = \psi(-\mathbf{p}) \quad (6.1)$$

Harris^[194] gives the evolution equation of ψ :

$$\frac{D\psi}{Dt} = -\frac{\partial}{\partial p_i}(\psi \dot{p}_i) \quad (6.2)$$

where $\partial / \partial p_i$ represents the divergence operator on U ; $\dot{p}_i$ denotes the derivative of p_i ; (p_1, p_2, p_3) is the non-independent variable; $\|\mathbf{p}\|=1$, so, $\partial / \partial p_i$ is constrained. However, this restriction does not cause problems because when $\|\mathbf{p}\|=1$, $\dot{p}_i$ is orthogonal to p_i . The evolution equation of ψ at any time on U can be obtained by replacing $\partial / \partial p_i$ with $(\partial / \partial p_i - p_i p_j \partial / \partial p_j)$.

What's more, in order to ensure the continuity of ψ at any time t , the continuity condition must be satisfied. When UHPC is casting, the ODF ψ of the fiber can be regarded as a function of flow rate, so it needs to satisfies

$$\dot{p}_i = \frac{1}{2}(1+\lambda)\frac{\partial v_i}{\partial x_j}p_j - \frac{1}{2}(1-\lambda)\frac{\partial v_j}{\partial x_i}p_j - \lambda d_{kl}p_i p_j p_k p_l - C_i \dot{\gamma} \frac{\partial \ln \psi}{\partial p_i} \quad (6.3)$$

where λ is the parameters related to fiber geometry; $C_i \dot{\gamma}$ is the rotary diffusivity; C_i is the random effect coefficient of fiber interaction; $\dot{\gamma}$ is a scalar of strain rate tensor. The last term

modifies the Jeffery model, making it is able to consider the interaction of fiber-fiber and fiber-flow.

Because there are 6 independent parameters in the 3D case, the method that uses $\psi(\mathbf{p}, \mathbf{x}, t)$ to describe the orientation state is not convenient. By utilizing the orientation tensor description ^[195-197], (which is successive and more detailed summaries of Ψ), this problem can be solved. The n th dyadic power of $\mathbf{p}$ can be record as $\mathbf{p}^{\otimes n}$, then, the n th inclusion orientation tensor $\mathbf{a}_n$ can be calculated by averaging the dyadic power on U ,

$$\mathbf{a}_n = \int_U \psi(\mathbf{p}, \mathbf{x}, t) \mathbf{p}^{\otimes n} d\mathbf{p} \quad (6.4)$$

According to the symmetry and regularization of the fiber distribution function, $\mathbf{a}_0$ is the constant 1.

The n th order orientation tensor is calculated by the n th order central moment of the ODF. So the odd-order orientation tensors are cancelled, the second and fourth-order tensor $\mathbf{a}_{ij}$ and $\mathbf{a}_{ijkl}$ are

$$\mathbf{a}_{ij} = \int_U \psi p_i p_j d\mathbf{p} \quad (6.5)$$

$$\mathbf{a}_{ijkl} = \int_U \psi p_i p_j p_k p_l d\mathbf{p} \quad (6.6)$$

Introducing the symmetric operator S , for the general tensor $\mathbf{b}$, $S(\mathbf{b})$ is calculated by averaging all the transpose matrices $\mathbf{b}$, from which the expressions of $S(\mathbf{b}_{ij})$ and $S(\mathbf{b}_{ijkl})$ can be written as

$$S(\mathbf{b}_{ij}) = (\mathbf{b}_{ij} + \mathbf{b}_{ji}) / 2 \quad (6.7)$$

$$S(\mathbf{b}_{ijkl}) = (\mathbf{b}_{ijkl} + \mathbf{b}_{ijlk} + \mathbf{b}_{ikjl} + \mathbf{b}_{jikl} + \dots) / 24 \quad (6.8)$$

There are 24 items in Eq.(6.8), from the symmetry,

$$\mathbf{a}_{ij} = S(\mathbf{a}_{ij}) \quad (6.9)$$

$$\mathbf{a}_{ijkl} = S(\mathbf{a}_{ijkl}) \quad (6.10)$$

Since $\mathbf{p}$ is the unit vector, then the regularization condition has

$$\mathbf{a}_{ii} = 1 \quad (6.11)$$

$$\mathbf{a}_{ijkk} = \mathbf{a}_{ij} \quad (6.12)$$

According to Advani ^[197], the deviation can be expressed as

$$f_{ij} = \mathbf{a}_{ij} - \frac{1}{N} \delta_{ij} \quad (6.13)$$

$$f_{ijkl} = a_{ijkl} - \frac{6}{4+N} S(\delta_{ij} a_{kl}) + \frac{3}{(4+N)(2+N)} S(\delta_{ij} \delta_{kl}) \quad (6.14)$$

in which,

$$S(\delta_{ij} a_{kl}) = (\delta_{ij} a_{kl} + \delta_{ik} a_{jl} + \delta_{il} a_{jk} + \delta_{jk} a_{il} + \delta_{jl} a_{ik} + \delta_{kl} a_{ij}) / 6 \quad (6.15)$$

$$S(\delta_{ij} \delta_{kl}) = (\delta_{ij} \delta_{kl} + \delta_{ik} \delta_{jl} + \delta_{il} \delta_{jk}) / 3 \quad (6.16)$$

$N = 2$ or 3 stands for the 2D or the 3D cases. The deviation parts of the orientation tensors are independent, completely symmetric and trace is zero,

$$f_{ij} = f_{ji} \text{ and } f_{11} + f_{22} + f_{33} = 0 \quad (6.17)$$

So the set $\{f_{11}, f_{22}, \dots, f_{23}\}$ has only five equations. Similarly, the set $\{f_{ijkl}\}$ has nine independent equations, and

$$f_{ii} = f_{ijjk} = f_{ijklmm} = 0 \quad (6.18)$$

By expanding $\psi(\mathbf{p})$ into Fourier series based on spherical function, the ODF of the fibers can be reconstructed. Considering the transformation properties of fibers' rigid body rotation, the irreducible even rank tensors are expressed as "state" variables, then the modified Fourier representation of $\psi(\mathbf{p})$ has the following form ^[198]

$$\psi(\mathbf{p}) = A_0 \cdot 1 + A_{ij} f_{ij} f(p_i p_j) + A_{ijkl} \cdot f_{ijkl} f(p_i p_j p_k p_l) + \dots \quad (6.19)$$

In the above equation, $f(p_i p_j)$ and $f(p_i p_j p_k p_l) \dots$ are the deviation parts, and the definition is similar to $f_{ij}, f_{ijkl} \dots$ in Eq.(6.14) and (6.15). While, $A_0, A_{ij}, A_{ijkl}, \dots$ are the expansion coefficients, they also have the properties of complete symmetry and no trace, and can be expressed by the expression determined by $\psi(\mathbf{p})$, N and tensor order s . The general term of expansion coefficient can be expressed as

$$A_{ij\dots}_s = \frac{1}{2^{N-1} \cdot \pi} \cdot \frac{\prod_{r=0}^s (2r+1)}{s!} \int_U \psi(p) f_{ij\dots}_s dp \quad (6.20)$$

where $N = 2$ or 3 also represents 2D or 3D problems, and s is the highest tensor order involved in the corresponding expansion coefficient. Then the expansion coefficients of order 0, 2 and 4 of 3D problems in Eq.(6.19) are shown in Eq.(6.21) - (6.23) respectively,

$$A_0 = \frac{1}{4\pi} \int_U \psi(p) dp \quad (6.21)$$

$$A_{ij} = \frac{1}{4\pi} \cdot \frac{3 \times 5}{2} \int_U \psi(p) f_{ij} dp \quad (6.22)$$

$$A_{ijkl} = \frac{1}{4\pi} \cdot \frac{3 \times 5 \times 7 \times 9}{2 \times 3 \times 4} \int_U \psi(p) f_{ijkl} dp \quad (6.23)$$

When the orientation tensor of the fiber is determined, the evolution equation can be obtained by averaging the orientation. According to the method provided by Advani ^[199], the mixed convective derivatives of the orientation tensors containing the fiber geometry information could be obtained by multiplying Eq.(6.3) and integrating on U ,

$$\dot{a}_{ij} = -2\lambda a_{ijkl} d_{kl} - 2C_I \dot{\gamma} N f_{ij} \quad (6.24)$$

$$\dot{a}_{ijkl} = -4\lambda a_{ijklmn} d_{mn} - 4C_I \dot{\gamma} \left((N+2) f_{ijkl} + \frac{3N}{N+4} S(f_{ij} \delta_{kl}) \right) \quad (6.25)$$

where $d_{ij} = (\partial v_i / \partial x_j + \partial v_j / \partial x_i) / 2$ is the strain tensor rate, v_i is the velocity component of the fiber. Obviously, Eq.(5.24) and (5.25) are embedded in each other. The trace of the fourth-order mixed convection derivative is exactly the second-order mixed convection derivative, and the second-order mixed convection derivative has no trace. Only one equation needs to be considered in a given convection model, however, the fourth-order tensor is embedded in the second-order tensor expression, which is embedded in the sixth-order tensor, etc. To solve this problem, it is necessary to express the second-order tensor as the basic quantity of fourth-order tensor to reconstruct the ODF. This method is the closure approximation.

6.2.2 Closure approximation

Considering the fact that the actual ODF of a composite material is usually unknown when the inclusions are non-aligned. In addition, the homogenization process of composites can also be based on the knowledge of the orientation tensors rather than the specific ODF. The closure approximation methods ^[200-205] are developed to try to reconstruct the missing high-order information, since the lower order orientation tensor only contains local information of ODF. The hybrid closure approximation has accurate solutions in both random and completely aligned cases, it was proposed to eliminate the disadvantages of linear and quadratic closures, after mixing the linear and quadratic forms, the expressions can be written as

$$a_{ijkl} = K \left[-\frac{3}{(4+N)(2+N)} S(\delta_{ij} \delta_{kl}) + \frac{6}{4+N} S(a_{ij} \delta_{kl}) \right] + (1-K) [a_{ij} a_{kl}] \quad (6.26)$$

where $K = N^N \det(a_{ij})$. The hybrid closure equation contains the same terms of the linear and quadratic. The difference is that the multiplication coefficient is a function of the invariant of a_{ij} rather than a constant.

6.3 Two-step homogenization method

A two-step homogenization process was then adopted to establish the link between the fiber orientation information on meso-scale and the macroscopic elastic properties, as shown in Figure 6.2. Firstly, the RVE was decomposed into a group of particles, and each particle was homogenized based on M-T model. Then, the RVE containing homogenized particles was homogenized in the second step by the Voigt-Reuss model.

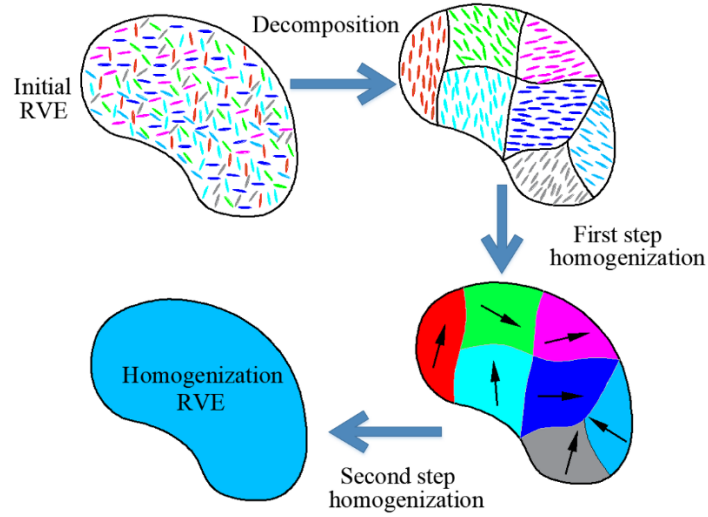


Figure 6.2. Two-step homogenization process.

6.3.1 Homogenization for two-phase composites

Considering a region ω composed of two kind of isotropic materials with volume V . The volume fraction of matrix is $\nu_m = V_m / V$, and the volume fraction of non-aligned inclusions is $\nu_f = V_f / V = 1 - \nu_m$, then the uniform stress $\bar{\sigma}$ and uniform strain $\bar{\epsilon}$ can be defined as

$$\bar{\epsilon} = V_m \bar{\epsilon}_m + V_f \bar{\epsilon}_f \quad (6.27)$$

$$\bar{\sigma} = V_m \bar{\sigma}_m + V_f \bar{\sigma}_f \quad (6.28)$$

where $\bar{\sigma}_m(\bar{\epsilon}_m)$ and $\bar{\sigma}_f(\bar{\epsilon}_f)$ represent the homogenization stress (strain) of the matrix and the inclusions in RVE, and the average stress (strain) of the matrix approximately equal to the far-field stress (strain) $\sigma_0(\epsilon_0)$,

$$\bar{\varepsilon}_m \approx \varepsilon_0 = \bar{\varepsilon} \quad \text{and} \quad \bar{\sigma}_m \approx \sigma_0 = \bar{\sigma} \quad (6.29)$$

Considering Eq.(6.27), (6.28) and the Hooke's law, the following relation can be obtained

$$\bar{C} = C^m + V_f [C^f - C^m] B \quad (6.30)$$

where $\bar{C}$ is the effective stiffness tensor; C^f and C^m are the effective stiffness tensors of the inclusion and the matrix; B is the strain concentration tensor relating the homogenized strain in the fiber ($\bar{\varepsilon}_f$) to the strain (ε_0) due to the remote stress (σ_0). Combining Eq. (6.27) (6.28) and Eq. (6.31) proposed by Benveniste^[206] for the relationship between the fiber strain and matrix strain expressed by concentrated tensor T , the expression of B can be obtained, as shown in (6.32),

$$\varepsilon_f = T \varepsilon_m \quad (6.31)$$

$$B = T(V_m I + V_f T)^{-1} \quad (6.32)$$

then, $\bar{C}$ can be calculated by taking formula (6.32) to (6.30)

$$\bar{C} = C^m + V_f [C^f - C^m] T(V_m I + V_f T)^{-1} \quad (6.33)$$

Then, all the stiffness coefficients of the composite can be determined by Eq.(6.33), and the macroscopic stiffness coefficients depend on the strain concentration tensor T .

6.3.2 Mori-Tanaka model

It is assumed that there is an infinite uniform linear elastic region Ω with the elastic coefficient of C^m , there is an inclusion γ with elastic coefficient C^f inside, and the eigenstrain records as ε^* . The strain caused by load σ_0 acting on infinite uniform linear elastic region Ω containing inclusion is ε_0 , the non-uniform stress and strain caused by inclusion γ are expressed as σ and ε respectively, the real stress and strain are $\sigma_0 + \sigma$ and $\varepsilon_0 + \varepsilon$, and the stress fields in γ and $\Omega - \gamma$ regions can be expressed as^[141]

$$\sigma_0 + \sigma = C^f (\varepsilon_0 + \varepsilon) \quad \text{in } \gamma \quad (6.34)$$

$$\sigma_0 + \sigma = C^m (\varepsilon_0 + \varepsilon) \quad \text{in } \Omega - \gamma \quad (6.35)$$

Therefore, the elastic stiffness in γ and $\Omega - \gamma$ is C^m , consider assigning an eigenstrain to the domain γ , the stress field of region $\Omega - \gamma$ is constant, and that of region γ is expressed as

$$\sigma_0 + \sigma = C^m (\varepsilon_0 + \varepsilon - \varepsilon^*) \quad \text{in } \gamma \quad (6.36)$$

where σ and ε are the disturbance stress and strain caused by eigenstrain ε^* , and the strain ε and eigenstrain ε^* are related by the Eshelby's tensor P as

$$\varepsilon = P\varepsilon^* \quad (6.37)$$

With the properly selected eigenstrain ε^* , Eq.(6.34) and (6.36) are equal and expressed as

$$C^m(\varepsilon^0 + \varepsilon - \varepsilon^*) = C^f(\varepsilon^0 + \varepsilon) \quad (6.38)$$

then, we can get Eq.(6.39) and (6.40)

$$(C^m - C^f)(\varepsilon_0 + \varepsilon) = C^m \varepsilon^* \quad (6.39)$$

$$C^{m-1}(C^m - C^f)(\varepsilon_0 + \varepsilon) = \varepsilon^* \quad (6.40)$$

the fourth-order symmetric tensor I is introduced and combined with Eq.(6.37) instead of ε^* ,

$$PC^{m-1}(C^m - C^f) = I - \varepsilon_0 / (\varepsilon_0 + \varepsilon) \quad (6.41)$$

and then,

$$PC^{m-1}(C^m - C^f) = I - \bar{\varepsilon}_m / \bar{\varepsilon}_f \quad (6.42)$$

so the following relation can be derived

$$\bar{\varepsilon}_f [I - PC^{m-1}(C^m - C^f)] = \bar{\varepsilon}_m \quad (6.43)$$

The relationship between the concentration tensor T and Eshelby's tensor is obtained by combining Eq.(6.43) with Eq. (6.31)

$$T = [I - PC^{m-1}(C^m - C^f)]^{-1} \quad (6.44)$$

joint Eq.(6.44) and Eq.(6.33), the final form of M-T model can be expressed as

$$\bar{C} = C^m + V_1 [C^1 - C^0] [I - PC^{m-1}(C^0 - C^1)]^{-1} (V_0 I + V_1 T)^{-1} \quad (6.45)$$

6.3.3 Voigt- Reuss model

In the second step of the homogenization, the macroscopic stiffness tensor $\bar{C}$ can be calculated by the simple interpolation, which based on the Voigt lower limit and Reuss upper limit mean field homogenization models,

$$\bar{C} = \frac{1}{2} C_{Voigt} + \frac{1}{2} C_{Reuss} \quad (6.46)$$

and the macroscopic stiffness tensors C_{Voigt} and C_{Reuss} can be obtained by,

$$C_{Voigt} = v_1 C_1 + (1 - v_1) C_0 \quad \text{and} \quad C_{Reuss} = [v_1 C_1^{-1} + (1 - v_1) C_0^{-1}]^{-1} \quad (6.47)$$

Then, the macroscopic stress-strain relationship after homogenization can be expressed as

$$\bar{\sigma} = \bar{C}\bar{\varepsilon} \quad (6.48)$$

6.4 Failure criterion and degradation model

The composite material is different from the general metal material, the structure is complex, and the bearing capacity is composed of the fiber and matrix. Therefore, when one region reaches the failure point, other regions can continue to bear the load because of the structural particularity, so the degradation mode of composite materials should be progressive and continuous damage. To describe the damage mechanical behavior of UHPC on multi-scale, it is essential to investigate the damage evolution on the basis of the progressive damage theory. Compared with the Hashin failure criterion, the Puck criterion does not adopt the global orthogonal coordinate system, but established the local coordinate system on the fracture surface, and considers that the stress component of the cross section is the main factor determining the fracture. The matrix failure is divided into three failure modes [207]. Since the Puck failure criterion has been widely used in composites.

Table 6.1 Material parameters for calculation.

Parameter	Symbol	Value
Young's Modulus (1)	E_1	45000 MPa
Young's Modulus (2/3)	$E_2 = E_3$	44500 MPa
Shear Modulus (1-2,1-3)	$G_{12} = G_{13}$	18700 MPa
Shear Modulus (2-3)	G_{23}	18400 MPa
Poisson's ratio	μ	0.2
Longitudinal Tensile Strength (1)	S_1^T	8.5 MPa
Longitudinal Tensile Strength (2/3)	$S_2^T = S_3^T$	8.0 MPa
Longitudinal Compressive Strength (1)	S_1^C	145 MPa
Longitudinal Compressive Strength (2/3)	$S_2^C = S_3^C$	135 MPa
Shear Strength (1-2)	S_{1-2}^S	49.5 MPa
Shear Strength (1-2, 1-3)	S_{1-2}^S	47.5 MPa
Fracture Energy (1)	G_1	6.67 N·mm
Fracture Energy (2/3)	$G_2 = G_3$	6.10 N·mm

The degradation mode of the material is that the material degenerates immediately when it reaches the failure point, and the bearing capacity of the material reduces to a degree that cannot bear the load at all. Degradation models usually include linear degradation model, exponential degradation model, polyline degradation model and others. Considering the influence of the fiber

orientation on the material properties, the elastic properties and strength of the UHPC in the direction 1 are taken from the study of Yu, and the fracture energy G_I is taken from the study of Liu [208]. The elastic properties, strength and fracture energy in 2/3 direction are appropriately reduced to reflect the performance differences in different directions. All the material parameters of UHPC in this study are shown in Table 6.1.

6.5 UHPC fiber orientation test measurement

The four types of UHPC mix proportions designed with fiber contents of 0.75%, 1%, 1.5%, and 2% (numbered S0.75, S0.75, S0.75, and S0.75 respectively) are detailed in Table 3.8. Copper plated microfilament steel fibers with a diameter of 0.4 mm, a length of 13 mm, and a tensile strength of 2500 MPa are used. As shown in Figure 6.3, the pouring process uses a pouring hopper to ensure that the fiber distribution is along the direction of force on the specimen. Five tensile specimens were prepared for each ratio of UHPC and tested after 28 days of curing under standard curing conditions.

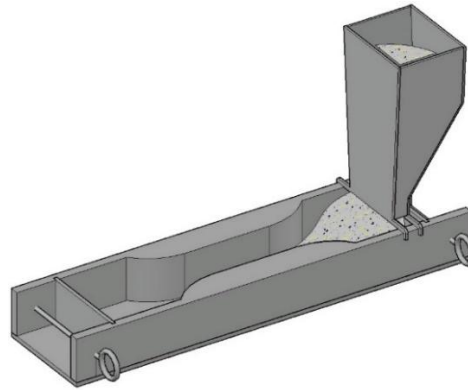


Figure 6.3 Schematic diagram of the pouring process of the UHPC specimen.

For the measurement of fiber orientation in concrete, several evaluation methods have been successfully used in the existing literature, such as scanning electron microscopy analysis [209], digital image analysis [210], computed tomography scanning and X-ray scanning [211] et. al. Although computed tomography can provide 3D images for the fiber orientation analysis, its analysis process is very complex and time-consuming. At the same time, digital image processing technology has been very mature, more fine details can be recognized by combining the high-resolution images and appropriate processing technology. The distribution of fiber orientation in tensile specimens was investigated by the image processing method. For each group of the specimens, two cross-section samples of the tensile specimen were prepared after the tensile test, one is near the fracture plane

(Section 1) and the other is far away from the fracture surface (Section 2), as Figure 6.4 illustrated. The fiber distribution images of the cutting surface of those samples were collected by *VHC-2000C super depth of field 3D microscope system*, and then analyzed.

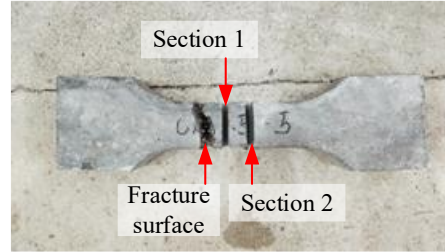


Figure 6.4 Position of the cross-section samples.

In order to accurately measure and extract the fiber distribution parameters of the section images, the following process is adopted,

- (1) Converting the original image to a grayscale image;
- (2) Transforming the grayscale image into a binary image by the threshold target detection method;
- (3) Extracting the length of the major and minor axis of the ellipse fiber section according to the pixel coordinate parameters of the elliptical fiber section on the cutting surface;
- (4) Calculating the inclination angle φ between the single fiber and the cross section, which defined as,

$$\cos \varphi = \frac{d_f}{L} \quad (6.49)$$

where d_f is the diameter of the fiber, and L is the major axis of the ellipse. The extraction and calculation process of the fiber orientation parameters is also explained in Figure 6.5.

The above image processing and fiber orientation parameter calculation are realized by *rgb2gray* (gray conversion), *graythresh* (binary threshold calculation), *im2bw* (binary conversion) and *regionprops* (region attribute extraction) commands of *MATLAB* platform. Figure 6.6 shows the section images and binary images of the cross-section samples of the four UHPC proportions. Figure 6.7 gives the pixel value and its relative increase ratio of the single fiber when the inclination angle changes from 0° to 75° . the pixel value of 0° , 15° , 30° , 45° , 60° and 75° is 380, 394, 439, 537, 760 and 1468, while the corresponding pixel value relative increase ratio to 0° is 0%, 3.5%, 13.4%, 29.3%, 50.0% and 74.1%, therefore, in the collected images, the pixel values of fiber sections with different inclination angles can be distinguished and has enough pixel density to ensure the accuracy

of fiber orientation calculation.

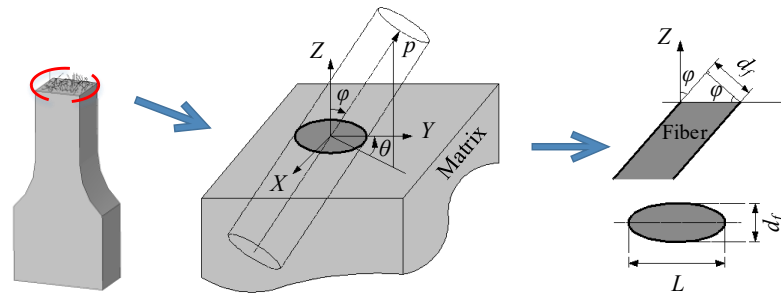


Figure 6.5 Extraction and calculation process of fiber orientation parameters.

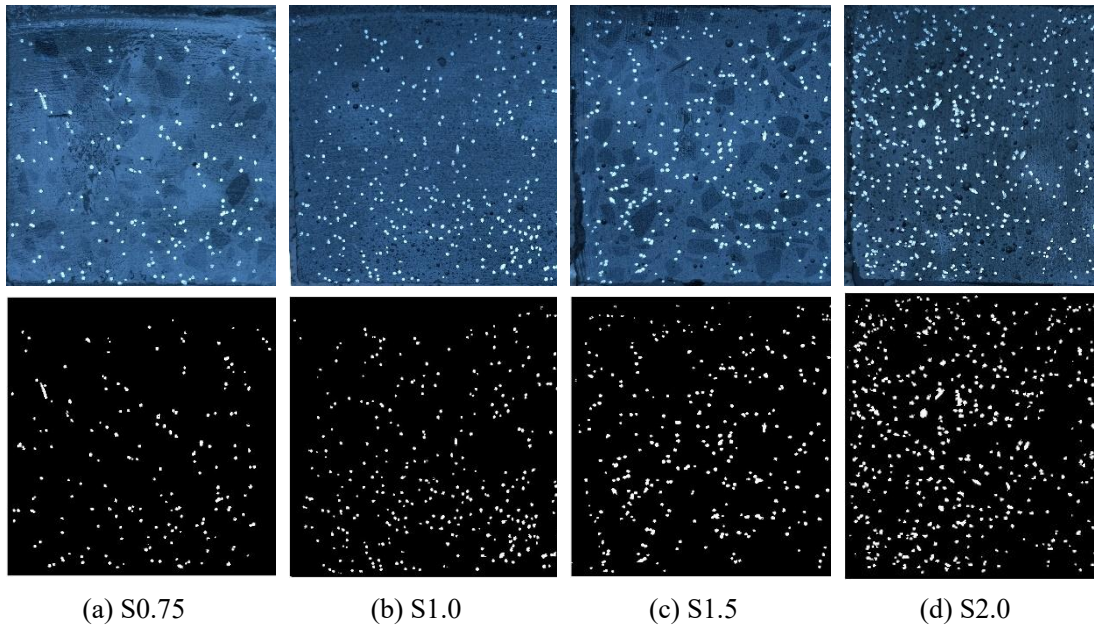


Figure 6.6 Cross section image and binary image of UHPC with different fiber contents.

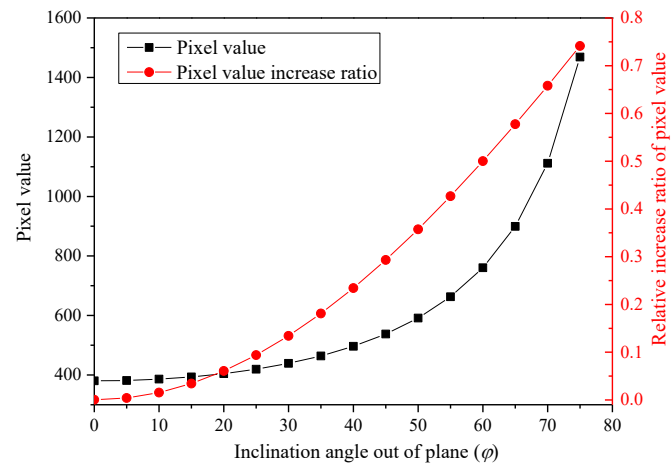


Figure 6.7 Pixel value and its relative increase ratio of the single fiber under the different inclination angles.

6.6 Multiscale Performance Verification and Prediction

6.6.1 Results of the fiber determination

The fiber distribution of the cross-section samples were extracted, and the inclination angle out of plane φ , and the transverse angle in-plane θ , were calculated. In Figure 6.8, the fiber orientation distribution of the Section 1 samples (near the fracture surface) are given. There are obvious differences in the distributions of the angle θ and the angle φ . The distribution of the angle φ is more concentrated than that of the angle θ . With the increase of the fiber volume fraction, the distribution of the angle φ becomes more concentrated at about 35° , and the distribution of the angle θ presents a random distribution.

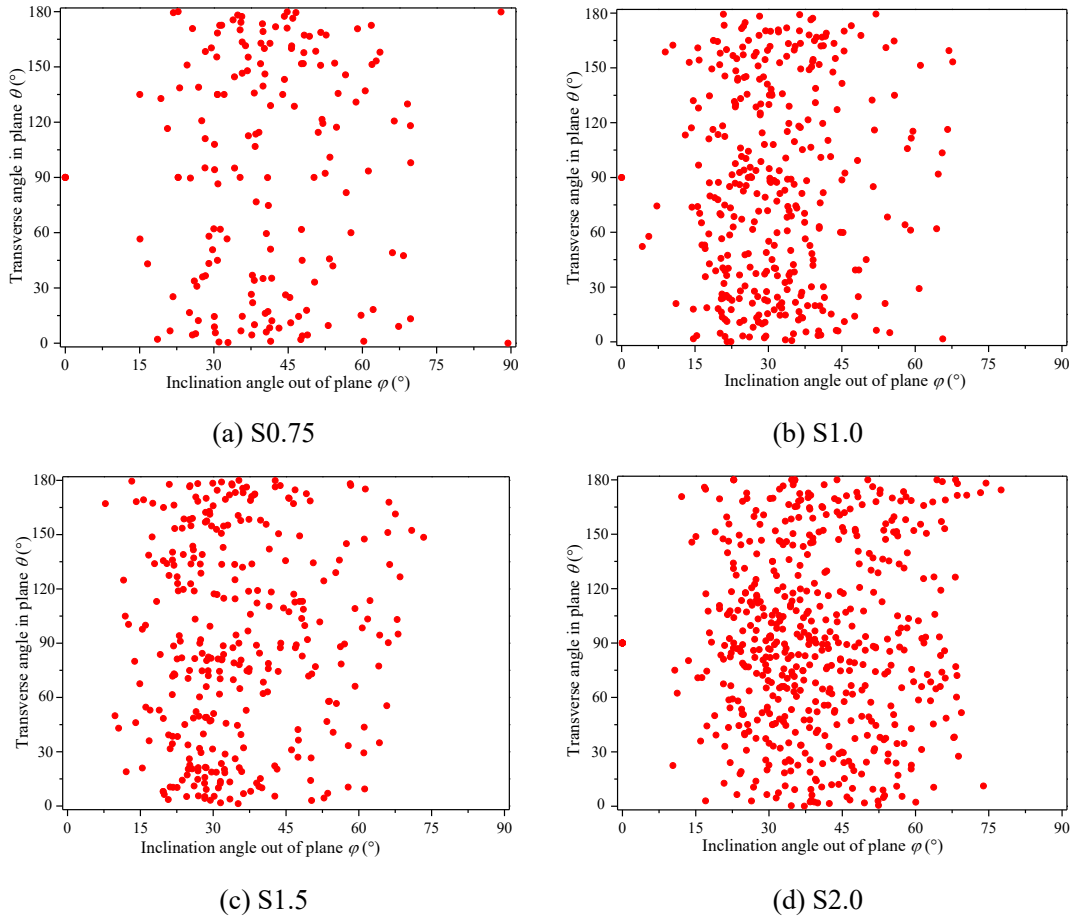


Figure 6.8 Fiber orientation distribution near fracture surface of the UHPC tensile specimens with different fiber content.

The statistical results of the number of fibers are shown in Figure 6.9. The value range of the fiber inclination angle θ is $[0, \pi/2]$, if the fiber length is L_f , and the range of the fiber center to the cross section is $[0, (L_f/2)\cos\theta]$, then the number of fibers per unit area of the section (N_b) can be

expressed as,

$$N_b = N_{1D} \int_0^{\frac{\pi}{2}} \int_0^{\frac{L_f}{2}} p(l)p(\theta) \cos \theta dl d\theta \quad (6.50)$$

where N_{1D} is the number of fibers per unit area of one-dimensional ($N_{1D}=V_f / A_f$), $p(l)$ is the distribution of the fiber length out of the cross section, $p(\theta)$ is the distribution probability function of inclination of the fiber. When the fibers are 3D distributed, $p(l)=2/l_f$, while $p(\theta)=2/\pi$, then N_b with the different fiber volume fraction can be calculated. The theoretical fiber number of the section with the different fiber volume fractions is close to the corresponding actual fiber number. Most of the samples show the phenomenon that the number of fibers in Section 2 is greater than that in Section 1. It indicates that the number of the fiber near the fracture surface where tensile fracture occurs in UHPC is relatively small, or the distribution orientation is unfavorable for resisting the tensile stress. Therefore, the bridging effect is relatively weak at this position. When the tensile stress is close to the crack initiation stress, the accumulated damage and micro-cracks caused by the tensile action in the specimen are relatively uniform dispersed in the interior or surface of the specimen. With the growing load, the micro-cracks in the disadvantageous area of the fiber spatial distribution will form the macroscopic cracks, and causing the bearing capacity to drop suddenly.

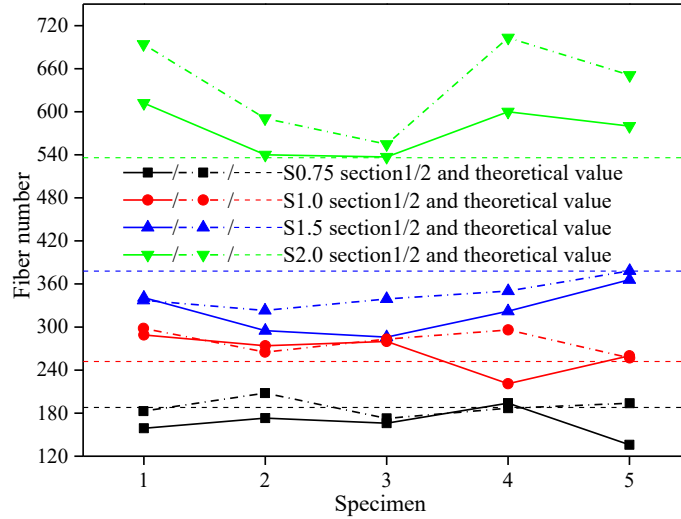


Figure 6.9 Theoretical and actual number of the fiber in the sample section.

6.6.2 Characteristics of the fiber orientation distribution

When casting UHPC, the fiber will flow with the paste, resulting in various forms of distribution. According to M. Xia's research ^[212], the orientation distribution of the fiber can be described by a probability density function (PDF) $p(\theta, \varphi)$, which is defined as

$$p(\theta, \varphi) = \frac{\{\sin \theta / 2\}^{2P_1-1} \{\cos \theta / 2\}^{2Q_1-1}}{\int_{\theta_a}^{\theta_b} \{\sin \theta / 2\}^{2P_1-1} \{\cos \theta / 2\}^{2Q_1-1} d\theta} \times \frac{\{\sin \varphi\}^{2P_2-1} \{\cos \varphi\}^{2Q_2-1}}{\int_{\varphi_a}^{\varphi_b} 2 \{\sin \varphi\}^{2P_2} \{\cos \varphi\}^{2Q_2} d\varphi} \quad (6.51)$$

where, θ_a , θ_b , φ_a and φ_b are the inferior and superior limits of the inclination angle, and $0 \leq \theta_a \leq \theta_b \leq \pi$, $0 \leq \varphi_a \leq \varphi_b \leq \pi/2$. P_1 , Q_1 and P_2 , Q_2 are the shape parameters of the PDF, and they are all greater than or equal to 0.5.

It is clear that the inclination angle φ (out of the cross section) is the key factor to the tensile properties. When only the distribution of the out of plane angle φ is considered, Eq.(6.51) is simplified to a marginal PDF to express the orientation distribution in the tensile direction,

$$g(\varphi) = \frac{\{\sin \varphi\}^{2P} \{\cos \varphi\}^{2Q}}{\int_{\varphi_a}^{\varphi_b} \{\sin \varphi\}^{2P} \{\cos \varphi\}^{2Q} d\varphi} \quad (6.52)$$

To analyze the influence of the shape factors P and Q , the $g(\varphi)$ curves are plotted in Figure 6.10 with (P, Q) equals to (0.5, 8), (1.5, 7), (2.5, 6), (3.5, 5), (4.5, 4), (5.5, 3), (6.5, 2) and (7.5, 1). The shape factors P and Q decide the curve shape, that the larger P and Q correspond to the thinner curve is and the higher the peak value, while the smaller P and Q correspond to the flatter curve.

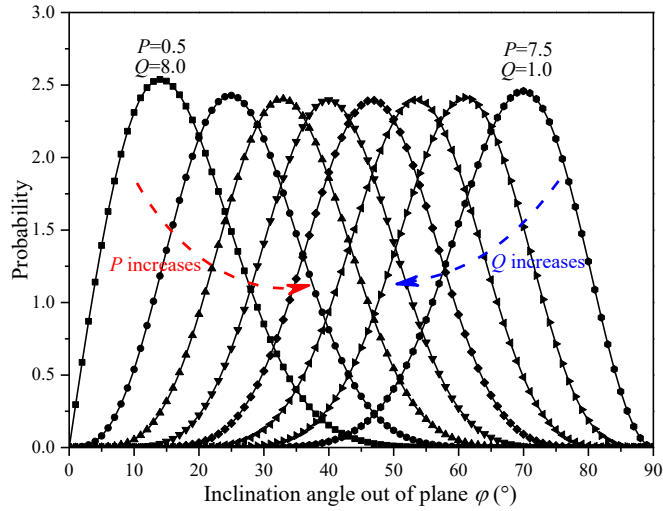


Figure 6.10 The evolution of the fiber orientation PDF with different shape factors.

Then the maximum probability value of $g(\varphi)$ and the expectation value of the inclination angle φ are calculated when P and Q vary from 0.5 to 8 respectively. The results are shown in Figure 6.11 and Figure 6.12. With the single increase of P or Q , the maximum probability value and the corresponding expectation value of the inclination angle φ both show the increasing trend. Additionally, the maximum probability value changes approximately linearly with the increase of P

or Q , while the increase rate of the expectation value decreases with the increase of P or Q . When $P=Q$, the expectation value of the inclination angle out of plane is 45° . When $P > Q$, the expectation value is greater than 45° . The larger the difference between P and Q , the larger the expectation value is and the closer the inclination angle is to 90° . On the contrary, when $P < Q$, the larger the difference between P and Q , the closer the expectation value is to 0° , which means that the fiber tends to be consistent with the ideal distribution.

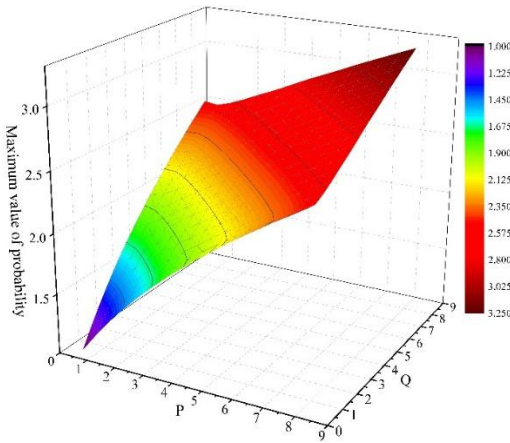


Figure 6.11 The maximum of probability with different P and Q .

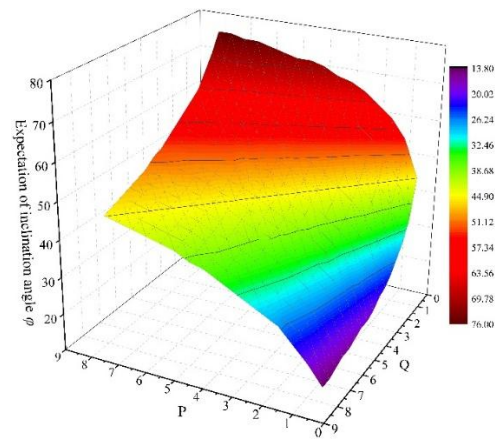


Figure 6.12 The expectation of inclination angle with different P and Q .

The fiber orientation distribution curves of the out of plane angle φ are fitted by Eq.(6.61), as seen in Figure 6.13. The characteristics of ‘left skewed distribution’ can be observed in samples with different fiber contents, which means that the fiber distribution tends to be consistent and perpendicular to the fracture surface. However, the two sections of a same specimen show obvious differences.

It can be found that the expectation value of the inclination angle φ is lower in the Section 2 sample than that in the corresponding Section 1 sample, while the maximum probability value of the Section 2 sample is higher than that of the Section 1 sample. This phenomenon indicates that the fiber distribution at Section 2 provides a better bridging effect on resisting micro-crack propagation of UHPC. Therefore, the tensile fracture of UHPC specimen tends to occur at the ‘weaker’ Section 1. Besides, it can be found that with the increasing of the fiber content, the probability density function is more ‘flat’, indicating that the larger quantity of the fiber may interfere the consistent distribution effect of the fiber in the pouring process, and reduce its crack resistance and toughening effect.

For each UHPC proportion, the average value and the deviation of the expectation value of the inclination angle φ are calculated based on the ODF curves, and the results are shown in Figure

6.14 The average value at Section 2 is less than that at Section 1. With the increasing fiber content, the average expectation value at both Section 1 and Section 2 increase, indicating that large fiber content is not conducive to the formation of the ideal fiber orientation distribution.

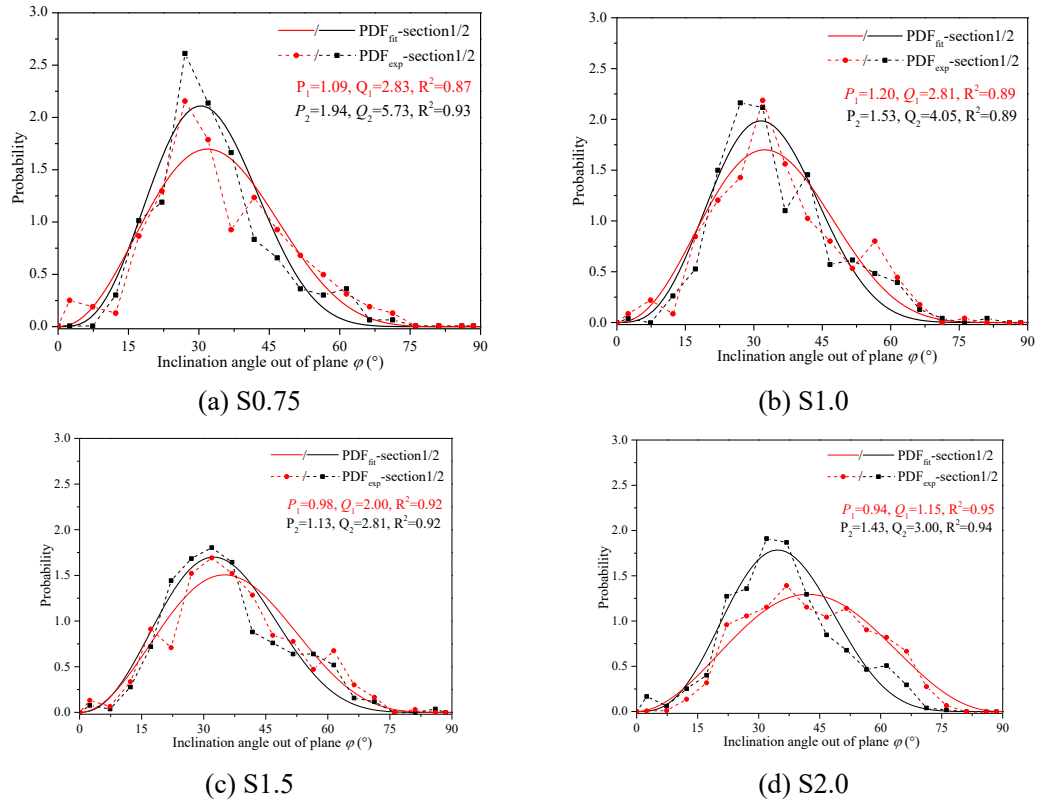


Figure 6.13 Experimental and fitting orientation distribution of UHPC.

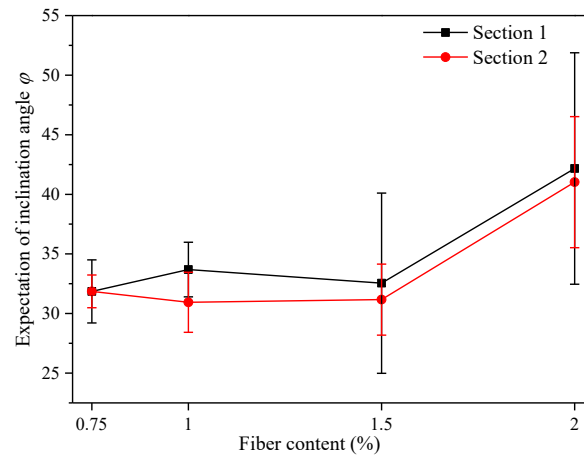


Figure 6.14 Average expectation and deviation of inclination angle φ with different fiber content.

For a certain UHPC proportion, the deviations of Section 1 samples are larger than those of the Section 2 samples, and the deviations of Section 1/2 samples with the fiber content of 0.75%, 1%, 1.5% and 2% are 2.6/1.4, 2.5/2.3, 7.5/3.0 and 9.7/5.5 respectively. Addition of the steel fiber reduces

the fluidity of UHPC paste, and it becomes more difficult to form of an aligned fiber distribution, expressing by the larger deviation of fiber distribution.

6.6.3 Relationship between fiber orientation distribution and tensile properties

Based on the extraction and calculation of the fiber orientation parameters of UHPC, the PDF of the fiber is obtained, and then the orientation tensor a_{ij} is calculated. The calculation results of section 1 with four UHPC proportions are shown in Table 6.2. The tensor directions 1, 2 and 3 represent the tensile direction along the specimen (Z axis), the vertical tensile direction (X axis) and another orthogonal direction (Y axis), respectively. The orientation tensors with the same subscript are similar for all the specimens with different fiber contents, and the tensor a_{11} plays a leading role in the tensile properties. With the increased fiber content, the fluidity of UHPC decreases and the consistency of the fiber distribution is reduced, resulting in the decrease of a_{11} .

Table 6.2 The orientation tensor of UHPC with different fiber content.

No.	a_{11}	a_{22}	a_{33}	a_{12}	a_{13}	a_{23}
S0.75	0.675	0.139	0.186	0.242	0.337	0.148
S1.0	0.656	0.159	0.186	0.247	0.333	0.154
S1.5	0.647	0.166	0.187	0.250	0.330	0.158
S2.0	0.626	0.183	0.191	0.260	0.327	0.169

However, in most cases, the ODF of an actual composite material is unknown and cannot be accurately measured by experiments. It is necessary to reconstruct the ODF through the closure approximation according to the only second-order tensor. To verify the applicability and feasibility of this method for UHPC, the ODF of the fiber was reconstructed by the hybrid closure approximation method according to the orientation tensor. Based on the reconstructed ODF information and the modified RSA algorithm ^[213], the mesoscopic RVE with different fiber content can be generated. The results are shown in Figure 6.15. Segurado's study ^[214] shows that macro equivalent performance can be better obtained when the RVE is applied with periodic boundary conditions. Therefore, a certain amount of fibers passing through the RVE surface are cut and transferred to the opposite surface. The reconstructed ODF, the corresponding experimental ODF and the fitting results are shown in Figure 6.16. It is obvious that the reconstructed ODF is close to the corresponding experimental results. The difference of the inclination expectation between the reconstructed and the corresponding experimental results is less than 7%, and the maximum

probability value difference is less than 6.7%, which verifies the applicability of this method for UHPC.

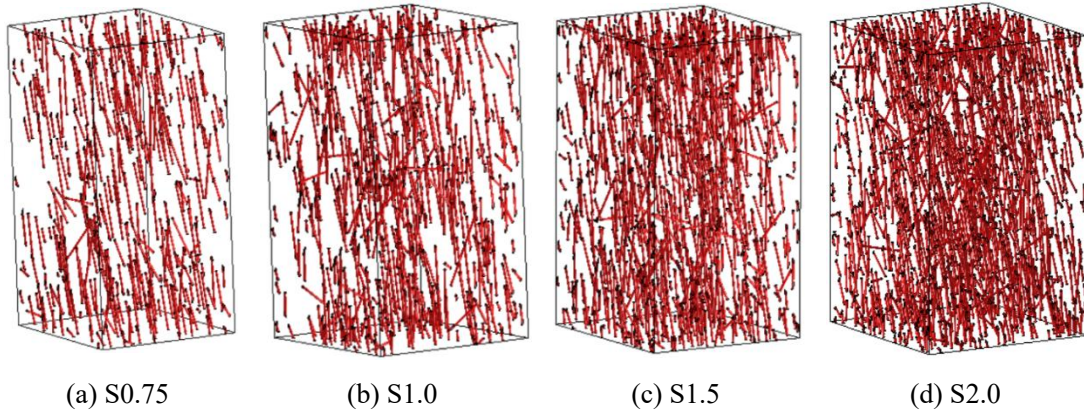


Figure 6.15 RVE of UHPC reconstructed by mixed closure approximation.

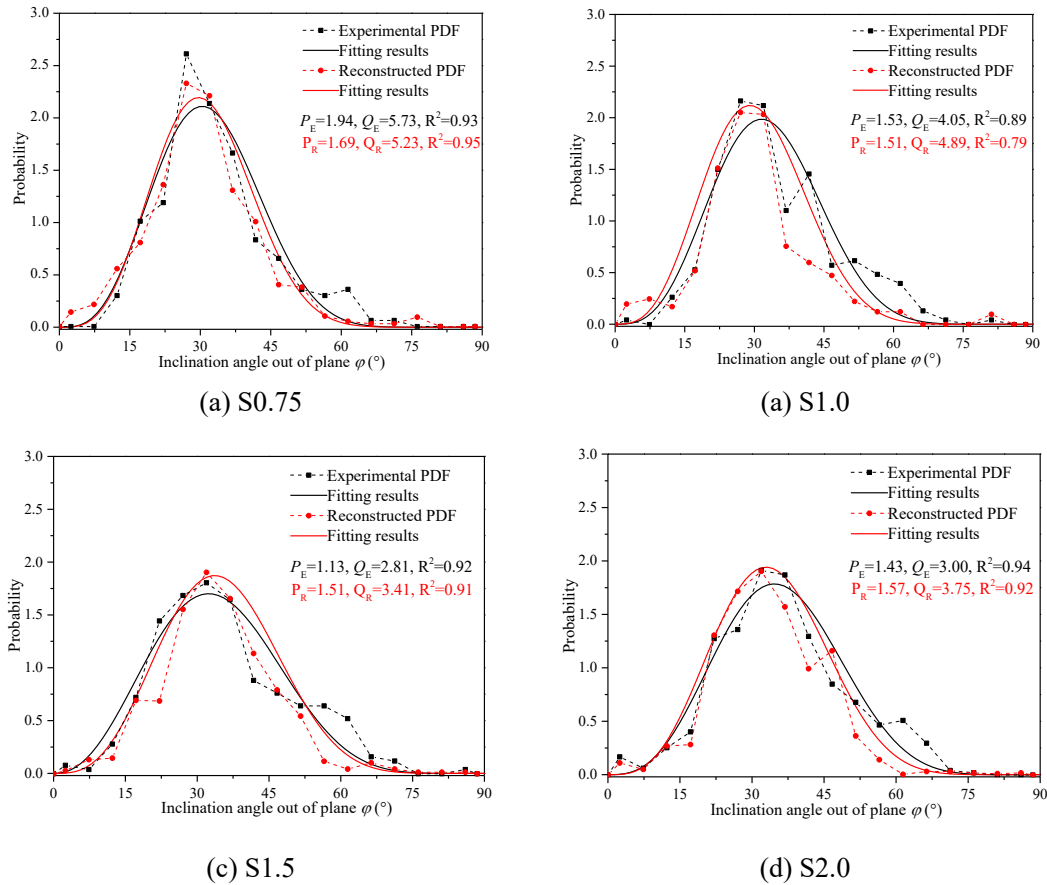


Figure 6.16 Comparison between the reconstructed ODF and experimental ODF.

The fiber orientation tensors a_{11} , a_{22} and a_{33} are calculated with the fiber inclination interval of 10° . It is also assumed that the in-plane distribution angle θ is a random distribution, so, $a_{22} = a_{33}$. The results of equivalent elastic modulus E_{11} , E_{22} ($E_{22} = E_{33}$) and shear modulus G_{12} , G_{23} ($G_{13} = G_{23}$) of UHPC with the fiber content of 0.5%, 0.75%, 1.0%, 1.5%, 2.0%, 2.5% and 3.0%

calculated by reconstructed RVE are shown in Figure 6.17. The results show that the Young's modulus and shear modulus of UHPC increase with the growing fiber contents, and the greater the fiber content is, the greater the effect of the orientation tensor on the equivalent properties of UHPC is (the change range of curve increases). As the expectation of inclination angle along the tensile direction grows (the orientation tensor decreases), the elastic modulus in the tensile direction decreases first and then increases slowly. The larger the fiber content arises the more obvious change trend (Figure 6.17 (a)). The reason for this change lies in the reduced Modulus in the tensile direction when the alignment of fibers becomes worse. Due to the inclination of the fiber, the in-plane equivalent modulus component perpendicular to the tensile direction increases (Figure 6.17 (b)), resulting in an increasing trend of E_{22} . When the tensile modulus (E_{11}) decreases to a certain value, the reinforcing effect of fibers begin to change from the tensile direction of the specimen to the direction that perpendicular to the tensile direction. The transverse reinforcing effect of the fiber in turn enhances the mechanical properties of the tensile direction, so E_{11} appears an upward trend when it decreases to a certain value.

Changes in the equivalent shear modulus G_{12} and G_{23} are shown in Figure 6.17 (c) and (d). When the fiber distributed consistently (the fiber inclination angle is close to 0°), the shear direction 12 is parallel to the fiber aligned direction and the shear modulus is the minimum. When the shear failure occurs in the 12 surface, most of the shear resistance is provided by the matrix. With the fiber inclination angle increases, the shear strength of the fiber gradually play a role, so the shear modulus G_{12} gradually increases. In addition, the shear modulus G_{23} firstly increases and then decreases with the change of orientation tensor a_{11} . When the orientation tensor a_{11} is equal to 1, the shear modulus G_{23} is close to G_{12} under the different fiber content, which also means that the shear strength of the direction 23 is mainly provided by the matrix when the fiber aligned perfectly. Similarly, when the fiber inclination angle increases, the strengthening effect of the fiber improves the shear properties in the 23 direction, and G_{23} increases gradually. While after the fiber angle exceeds 60° , the characteristics of the transverse arrangement of the fibers began to have the effect, and the contribution of the fibers to the shear properties of the 23 direction gradually weakened, so G_{23} gradually decreased.

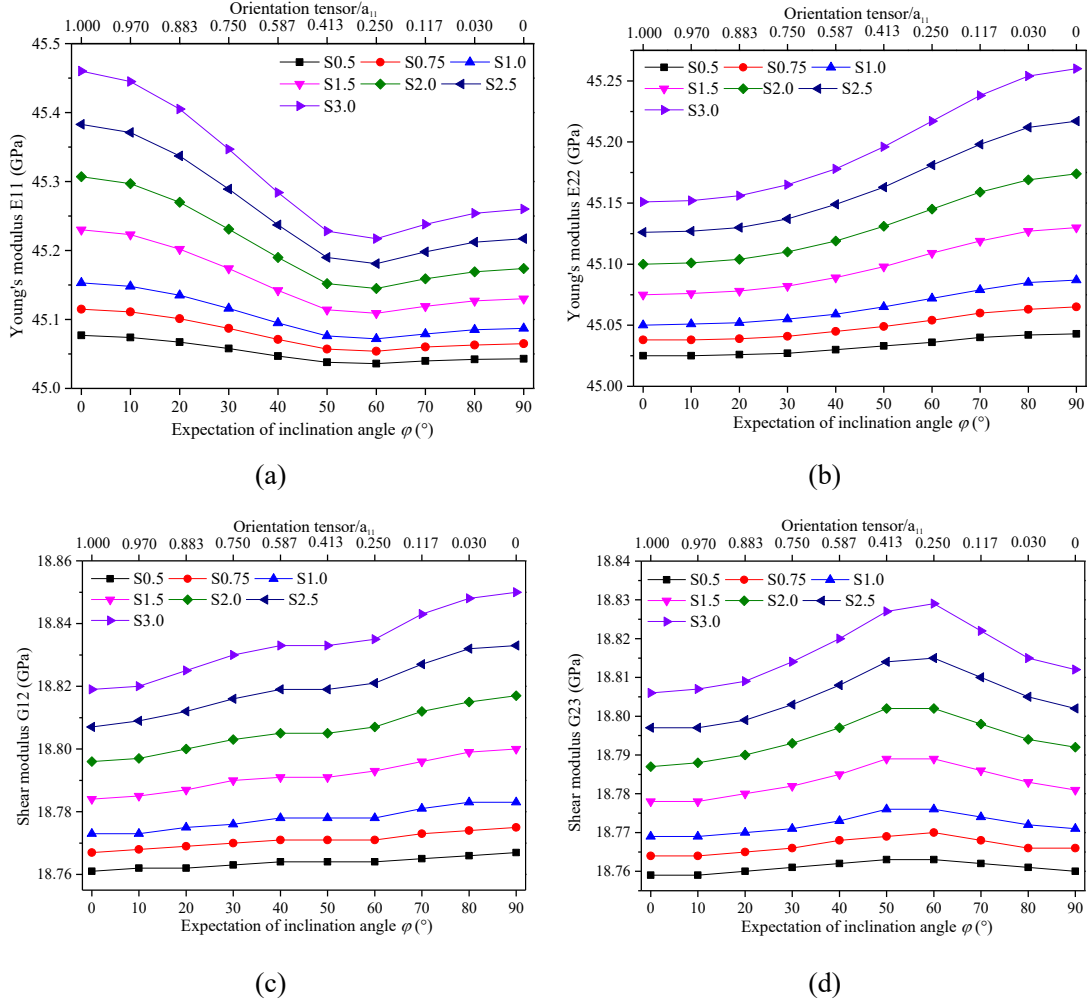


Figure 6.17 Equivalent properties of UHPC with the different fiber contents and orientation tensors.

Influence of the fiber contents and the orientation tensors on the equivalent elastic properties of UHPC is predicted quantitatively by the hybrid closure approximation and the two-step homogenization method. Regression analysis was carried out according to the calculation results of equivalent properties. Since the regression analysis contains two variables, the multiple regression analysis was used. The change trend of E_{11} , E_{22} , G_{12} , and G_{23} with various fiber contents and orientation tensors were investigated, and the Eq.(6.53) was used for fitting,

$$f = a\gamma\sin(x/b) + cy + d \quad (6.53)$$

where, a , b , c and d are the fitting parameters of the regression function, f is the elastic or shear modulus, x is the orientation tensor a_{11} , and γ is the fiber volume content. The fitting results are shown in Table 6.3. It is obvious that the regression equation can efficiently fit the relationship between the fiber content, the orientation tensor and the equivalent elastic constant of UHPC.

Table 6.3 Fitting results of equivalent elastic properties.

Parameters	a	b	c	d	R^2
E_{11}	-92510	-1420	72.19	45000	0.94
E_{22}	-118400	7137	73.85	45000	0.95
G_{12}	-138600	-15280	32.41	18750	0.99
G_{23}	-505700	132100	23.94	18570	0.96

After obtaining the elastic properties of UHPC related to the fiber orientation tensor, the effect of fiber orientation on the multi-scale tensile properties was further studied. Coupled with Puck failure criterion, the macroscopic tensile properties of UHPC were calculated by using the user defined material subroutine (UMAT) in ABAQUS. The results of the average experimental and multi-scale predicted tensile strength are shown in Figure 6.18. The average test strength indicates the average strength of the same fiber content, and so as the average of the fiber orientation tensor. The experimental results show that with the increase of the fiber content, the average orientation tensor decreases (the expectation of inclination angle increases). The experimental results of the average tensile strength are close to the multi-scale predicted results under the corresponding fiber contents and the orientation tensors.

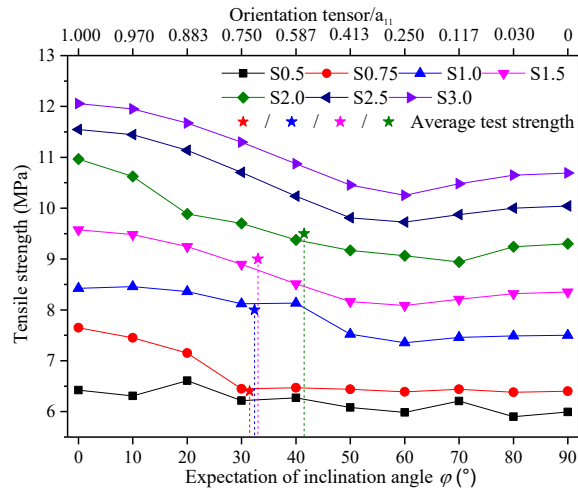


Figure 6.18 Average experimental and multi-scale predicted tensile strength of UHPC with different fiber contents and orientation tensors.

The change of tensile strength with the different fiber orientation tensor is similar to that of Young's modulus E_{11} . When the fiber content is low, the toughening effect is weak, so the fiber orientation tensor has little effect on the tensile strength. When the quantity of fibers increases, the improvement effect is enhanced. At the same time, the effect of the fiber orientation is also more

and more significant, which can be obtained from the change in tensile strength. Figure 6.18 shows that when $V_f \geq 1\%$, the tensile strength firstly decreases and then increases with the decrease of orientation tensor. To give full play to the maximum performance advantages of the materials and improve the structural performance, the influence of the fiber orientation characteristics should also be considered while maintaining the optimal fiber content. On the basis of optimizing the relevant construction technology, the excellent performance of UHPC can be maximized.

6.7 Conclusion

In this paper, the effect of the fiber orientation on the tensile properties of UHPC was investigated by uniaxial tensile test and multi-scale numerical analysis. The main conclusions are as follows,

(1) With the increase of the fiber content, the tensile stress-displacement curves of UHPC changed obviously from strain softening to strain hardening. The critical fiber content of strain hardening and strain softening is about 1.33% for UHPC, which reinforced with the fiber aspect ratio of 32.5.

(2) The fiber distribution characteristics of UHPC show that the PDF of the fiber inclination angle presents a characteristic of 'left skewed distribution'. The expectation of the inclination angle of fibers far from the fracture surface (Section 2) is less than that near the fracture surface (Section 1), and the maximum probability of fiber distribution in Section 2 is greater than that in Section 1. This indicates that UHPC tensile fracture tends to occur in the weak plane with poor aligned distribution. The ODF fitting morphology shows that the increase of the fiber number will influence the effect of the consistent distribution of the fiber, and weakening the crack resistance and toughening effect.

(3) The ODF distribution reconstructed by the hybrid closure approximation is close to the corresponding experimental results. Among them, the difference of the expectation of the inclination angle is no more than 7%, the maximum probability difference is less than 6.7%, which verifies the applicability of this method for UHPC.

(4) Young's modulus E_{22} and shear modulus G_{12} increase with the decrease of orientation tensor a_{11} . The change of the reinforced direction makes the tensile modulus E_{11} decrease first and then increase, while the shear modulus G_{23} increase first and then decrease. With the decrease of orientation tensor, the tensile strength of UHPC also shows a trend of decreases first and then increases, which becomes more and more obvious with the increase of the fiber contents.

Chapter 7 Investigation on the NC-UHPC Interface Bonding Performance

7.1 Overview

The use of UHPC reinforcement and repair of damaged normal concrete (NC) structures is currently an effective and rapid reinforcement technique. This chapter aims to investigate the debonding failure performance of UHPC (repair material) and NC (substrate) under tensile and shear loads. Considering the complex interface geometry at the microscale, a multi-scale analysis method for the mechanical behavior of NC-UHPC interface was established. The failure of the base mortar is described by coupling the elastoplastic damage model with gradient enhanced damage evolution, and the interface is represented by the cohesive zone model (CZM). UHPC is characterized by the second-order mean-field homogenization (MFH) method. After extracting the roughness feature parameters of the interface, an RVE corresponding to the roughness is established to analyze the load transfer and debonding failure behavior, thereby obtaining an effective traction separation curve. On this basis, the tensile and shear separation mechanisms of NC-UHPC interface were revealed, and the effects of interface roughness and strength on macroscopic shear and tensile strength were evaluated. The improvement of interface roughness and base concrete strength has a positive impact on the mechanical properties of the interface.

7.2 Multiscale interface analysis model between NC and UHPC

This chapter firstly expounds on the material constitutive model, and then gives the general framework for homogenization analysis of a random rough interface region. A four-phase NC-UHPC interface RVE with embedded aggregates was investigated, which configures the response of non-uniform microstructure. The effective TSL and the interface characteristics were obtained by homogenization and then used for macroscale analysis. The elastic-plastic damage model was adopted for the NC matrix, taking the damage into consideration, and the mesh dependence of the local continuous damage model was solved by the damage evolution in form of localizing gradient-enhancement. The randomly distributed polygonal aggregate was generated in the NC matrix, which is assumed to be elastomer without failure. The second-order homogenization model coupled with progressive damage was adopted for UHPC.

7.2.1 Constitutive models

7.2.1.1 Constitutive model of NC material

The elastic response of NC matrix can be determined by the elastic stage free energy $\hat{\Psi}(\mathbf{b}^e) = \hat{\Psi}(\lambda_1^e, \lambda_2^e, \lambda_3^e)$, where λ_i^e is the principal stretches of $\mathbf{b}^e$. The Cauchy stress tensor $\boldsymbol{\sigma}$ is determined with the Kirchoff stress tensor $\boldsymbol{\tau}$ as $\boldsymbol{\sigma} = 1/J \boldsymbol{\tau}$, and $J = \det(\partial \phi / \partial \mathbf{X}) > 0$, in which ϕ describes the motion of the material body, and $\mathbf{X}$ is the material position vectors. Here, the general phenomenological ansatz of the Ogden type ^[215] is adopted,

$$\Psi = \sum_{p=1}^N \frac{\mu_p}{\alpha_p} (\hat{\lambda}_1^{\alpha_p} + \hat{\lambda}_2^{\alpha_p} + \hat{\lambda}_3^{\alpha_p} - 3) + \frac{\kappa}{4} (J^2 - 2 \ln J - 1) \quad (7.1)$$

where μ_p , α_p , κ represent the material parameters, $\hat{\lambda}_i$ are the isochoric principal stretches according to Flory ^[216] and a deviatoric and volumetric part of the additive split of Ψ .

The parabolic yield criterion is used to describe the asymmetric tensile compression behavior of materials after yield (plastic flow), and the yield function can be described as,

$$f = 3J_2 + (\sigma_c - \sigma_t) I_1 - \sigma_t \sigma_c \quad (7.2)$$

where $J_2 = 1/2 \hat{\mathbf{s}} : \hat{\mathbf{s}}$ is the second invariant of the effective Cauchy stress deviator $\hat{\mathbf{s}} = \hat{\boldsymbol{\sigma}} - 1/3 I_1 \mathbf{I}$, in which $\hat{\boldsymbol{\sigma}} = 1/J \hat{\boldsymbol{\tau}}$ is the effective Cauchy stress tensor, $I_1 = \text{tr}(\hat{\boldsymbol{\sigma}})$ is the first invariant, $\mathbf{I}$ is the identity tensor, and σ_t and σ_c is the tensile and the compressive yield strength respectively.

The evolution of the elastic left Cauchy-Green deformation tensor is governed by the non-associative flow rule,

$$-\frac{1}{2} (\mathcal{L}_{\mathbf{v}} \mathbf{b}^e) \cdot (\mathbf{b}^e)^{-1} = \dot{\lambda} \frac{\partial Q}{\partial \hat{\boldsymbol{\sigma}}} \quad (7.3)$$

where $\mathcal{L}_{\mathbf{v}} \mathbf{b}^e$ represents the Lie derivative according to Coleman ^[217]. With the plastic multiplier $\dot{\lambda}$ according to the loading-unloading conditions $\dot{\lambda} \geq 0$, $f \leq 0$, $\dot{\lambda} f = 0$ and the consistency condition $\dot{f} = 0$. The plastic potential is given as,

$$Q = 3J_2 + \alpha I_1^2 \quad (7.4)$$

where α can be expressed with the initial plastic Poisson's ratio ν_p ^[218]

$$\alpha = \frac{9 - 18\nu_p}{2 + 2\nu_p} \quad (7.5)$$

Then, the tensile and compressive yield strengths σ_t and σ_c in Eq.(7.2) are defined by the evolution equations,

$$\dot{\sigma}_t = h_t(\varepsilon^p)\dot{\varepsilon}^p \quad \text{with} \quad \sigma_t(0) = \sigma_t^0 \quad (7.6)$$

$$\dot{\sigma}_c = h_c(\varepsilon^p)\dot{\varepsilon}^p \quad \text{with} \quad \sigma_c(0) = \sigma_c^0 \quad (7.7)$$

where σ_t^0 and σ_c^0 are the initial yield stress, h_t and h_c is the hardening modulus. The isotropic hardening is controlled by the equivalent plastic strain and defined by Melro ^[218],

$$\dot{\varepsilon}^p = k\dot{\lambda} \sqrt{\frac{\partial Q}{\partial \boldsymbol{\sigma}} : \frac{\partial Q}{\partial \boldsymbol{\sigma}}} \quad (7.8)$$

where $k = 1 / (1 + 2\nu_p^2)^{1/2}$ depends on the initial plastic Poisson's ratio ν_p .

It is widely known that the finite element solution of the standard damage model heavily depends on the spatial discretization. Therefore, the nonlocal gradient-enhanced damage form^[219, 220] is introduced, which effectively eliminates the disease conditioned mesh dependence in the numerical analysis of the damage. The localizing model considers isothermal linear elastic deformation and adopts a scalar damage variable d to describe the material degradation. The equivalent strain variable ε_{eq} describing the macroscopic deformation state is defined. The local gradient model introduces an additional morphological variable $\bar{\varepsilon}_{eq}$ to characterize the rapid fluctuation process of micro level under each macro material point, which effectively alleviates the spurious damage growth of the standard gradient model ^[221]. In the local gradient model ^[222], the free energy density Π_b can be expressed as,

$$\Pi_b = \int_{\Omega} \left(\frac{1}{2} (1-d) \boldsymbol{\varepsilon} : \mathbb{E} : \boldsymbol{\varepsilon} \right) d\Omega + \int_{\Omega} \left(\frac{1}{2} h (\varepsilon_{eq} - \bar{\varepsilon}_{eq})^2 + \frac{1}{2} g h c \nabla^2 \bar{\varepsilon}_{eq} \right) d\Omega \quad (7.9)$$

where, $\mathbb{E}$ is the fourth-order elasticity tensor, h is the coupling modulus, c is the gradient parameter, g is the interaction function, and Ω denotes an arbitrary domain.

The first term on the right of Eq.(7.9) gives the energy density in the damaged state, and the second term is the density functional considering the interaction between micro and macro scales

and the interaction between micro cracks (defects) themselves. Therefore, the total free energy density $\hat{\psi}$ can be expressed as,

$$\hat{\psi} = \frac{1}{2}(1-d)\varepsilon : \mathbb{E} : \varepsilon + \frac{1}{2}h(\varepsilon_{eq} - \bar{\varepsilon}_{eq})^2 + \frac{1}{2}ghc\nabla^2\bar{\varepsilon}_{eq} \quad (7.10)$$

With the help of the Coleman-Noll procedure, the constitutive relations can be obtained as,

$$\sigma = \frac{\partial \psi}{\partial \varepsilon} = (1-d)\mathbb{E} : \varepsilon + h(\varepsilon_{eq} - \bar{\varepsilon}_{eq}) \frac{\partial \varepsilon_{eq}}{\partial \varepsilon} \quad (7.11)$$

$$\bar{\sigma} = \frac{\partial \psi}{\partial \bar{\varepsilon}_{eq}} = -h(\varepsilon_{eq} - \bar{\varepsilon}_{eq}) \quad (7.12)$$

$$\bar{\xi} = \frac{\partial \psi}{\partial \nabla \bar{\varepsilon}_{eq}} = ghc\nabla \bar{\varepsilon}_{eq} \quad (7.13)$$

The second term in Eq.(7.11) represents the coupling effect between the micro and macro scales. Eq. (7.12) and (7.13) describe equivalent coupling stress and the interaction within the micro cracks themselves, respectively. They should satisfy the following governing equations for the localizing gradient damage model consisting of the equilibrium equation and the micro force balance equation,

$$\nabla \cdot \sigma = 0 \quad (7.14)$$

$$\bar{\sigma} = \nabla \bar{\xi} \quad (7.15)$$

Then the following governing equations in the strong form for localizing gradient model can be obtained as,

$$\nabla \cdot \sigma = b \quad \text{in } \Omega \quad (7.16)$$

$$\bar{\varepsilon}_{eq} - \varepsilon_{eq} = \nabla \cdot (gc\nabla \bar{\varepsilon}_{eq}) \quad \text{in } \Omega \quad (7.17)$$

where b is the body force, then it is necessary to satisfy the Neumann and Dirichlet boundary conditions in Eq.(7.18) and (7.19), also the following condition regarding the $\bar{\varepsilon}_{eq}$ in Eq.(7.20),

$$\sigma \cdot n = t^* \nabla \bar{\varepsilon}_{eq} \cdot n = 0 \quad \text{on } \partial\Omega_t \quad (7.18)$$

$$u_x = u^* \quad \text{on } \partial\Omega_u \quad (7.19)$$

$$\nabla \bar{\varepsilon}_{eq} \cdot n = 0 \quad \text{on } \partial\Omega_f \quad (7.20)$$

where t^* and u^* are the prescribed traction force and displacement on the corresponding portions on the boundary $\partial\Omega_t$ and $\partial\Omega_u$.

The local gradient model assumes that the failure process area decreases with the accumulation

of damage d . Then the interactive function $g(d)$ is defined as,

$$g(d) = \frac{(1-R)\exp(nd) + R - \exp(-n)}{1 - \exp(n)} \quad (7.21)$$

in which, R and n are the constants. When $g(d)=1$, the energetic interactions between micro-cracks themselves take effect over a constant domain, the localizing gradient model regresses to the standard gradient model. Then the equivalent strain scale ε_{eq} can be derivate considering the modified von Mises as,

$$\varepsilon_{eq} = \frac{k-1}{2k(1-2\nu)} I_1 + \frac{1}{2k} \sqrt{\frac{(k-1)^2}{(1-2\nu)^2} I_1^2 + \frac{2k}{(1+\nu)^2} J_2} \quad (7.22)$$

where ν is the Poisson's ratio, k is the ratio of compressive strength to the tensile strength, I_1 is the first invariant of the strain tensor, and J_2 is the second invariant of the deviatoric strain tensor.

To avoid the reversibility of damage, a load history related parameter κ is defined as equal to the maximum value of the nonlocal scalar $\bar{\varepsilon}_{eq}$,

$$\kappa(t) = \max \{ \bar{\varepsilon}_{eq}(\tau) | 0 \leq \tau \leq t \} \quad (7.23)$$

where t and τ are the total and instantaneous times respectively concerning the deformation history. Then, the exponential form of the damage variable d can be deduced with the history variable κ as,

$$d(\kappa) = \begin{cases} 0 & \kappa \leq \kappa_0 \\ 1 - \frac{\kappa_0}{\kappa} [(1-\alpha) + \alpha \exp(-\beta(\kappa - \kappa_0))] & \kappa > \kappa_0 \end{cases} \quad (7.24)$$

where κ_0 is the damage threshold level, α is the residual strength, and β is a softening curve shape parameter controls the slop.

The elastic-plastic response coupling the gradient-enhanced damage evolution constitutive model can be implemented in the general finite element solution package of ABAQUS, which is defined as a user subroutines according to the strategy presented by Seupel et al.^[223]. The parameters corresponding to the NC material used in this study are shown in Table 7.1.

Table 7.1 Calculation parameters of the NC material.^[224]

Elastic Modulus (MPa)	Poisson's ratio	Damage threshold	Residual strength	Softening curve shape parameter
$E = 3.45\text{e}4$	$\nu = 0.3$	$\kappa_0 = 1.2\text{e}-4$	$\alpha = 0.99$	$\beta = 300$
Ratio of compressive to tensile strength	Coupling modulus	Residual interaction parameter	Material parameter	Gradient parameter (mm ²)
$k = 10$	$h = E \times 10^{-9}$	$R = 0.005$	$n = 5$	$c = 30$

7.2.1.2 Constitutive model of UHPC material

For a fiber reinforced composite like UHPC, the mean-field homogenization (MFH) is an effective way to obtain the macro-scale properties. The purpose of MFH is to calculate the approximate estimation the stress and the strain field of each phase by the volume averaged, and also the properties of RVE at the macro scale. The widely used MFH methods, such as the Voigt-Reuss model, the self-consistent method, and the Mori-Tanaka method are the incremental or the incrementally affine formulations, which are all the first-order based. For the inelastic composites (elasto-plastic or elasto-viscoplastic), it means that the comparison tangent operator for each phase is calculated from the volume average of the corresponding variable field. This average is called the first statistical moment for each corresponding field. The second-order homogenization contains not only the first statistical moment of each corresponding variable field but also the second statistical moment. The second moment is related to variance. The latter enriches statistical information rather than just a simple average and ensures better prediction results.

Considering the Taylor expansion around a point $\underline{X}_c$ of a finite materials vector $\underline{\Delta X}$ at the second- order complemented by a micro-fluctuation $\underline{w}$,

$$\underline{\Delta X} = \underline{F}_M \cdot \underline{\Delta X} + \frac{1}{2} \underline{G}_M : \underline{\Delta X} \otimes \underline{\Delta X} + \underline{w}(\underline{\Delta X}) \quad (7.25)$$

where, $\underline{X}_c$ is the geometrical center of the RVE, i.e. $\int_{\Omega_{RVE}} \underline{X} d\Omega = 0$, $\underline{\Delta X} = \underline{X} - \underline{X}_c$, $\underline{\Delta X} = \underline{X} - \underline{X}_c$.

The second and third-order tensors are $\underline{F}_M = \frac{\partial \phi(\underline{X}_c)}{\partial \underline{X}}$ and $\underline{G}_M = \frac{\partial}{\partial \underline{X}} \left(\frac{\partial \phi(\underline{X}_c)}{\partial \underline{X}} \right)$, and the

macroscopic deformation from the undeformed state to the deformed state is defined as $\phi(\underline{X})$.

The displacement of the geometrical center of the RVE is $\underline{u}_c = \underline{x}_c - \underline{X}_c$, Then, the displacement field can be derived as,

$$\underline{u} = \underline{x} - \underline{X} = \underbrace{\underline{u}_C + (\underline{F}_M - \underline{I}_d) \cdot \underline{\Delta X} + \frac{1}{2} \underline{G}_M : \underline{\Delta X} \otimes \underline{\Delta X}}_{-\underline{u}^M} + \underline{w}(\underline{\Delta X}) \quad (7.26)$$

where, $\underline{u}^M$ is the macro-scale displacements. According to Kouznetsova^[225, 226], the macro-scale first and second-order gradients are defined as,

$$\begin{cases} \underline{F}_M = \frac{1}{V} \int_{\Omega_{RVE}} \underline{F}_m d\Omega \\ \underline{G}_M \cdot \underline{J} + \frac{1}{2} \underline{G}_M : \underline{J} \otimes \underline{I}_d = \int_{\partial\Omega_{RVE}} \underline{x} \otimes \underline{\Delta X} \otimes \underline{N} d\Gamma \end{cases} \quad (7.27)$$

where $\underline{N}$ is the normal direction unite vector of the RVE, and $\underline{J} = \int_{\Omega_{RVE}} \underline{\Delta X} \otimes \underline{\Delta X} d\Omega$ is the second moment of the undeformed RVE at point $\underline{X}_C$, $\underline{F}_m$ is the microstructural deformation gradient tensor.

In the framework of the second-order homogenization, these relations are used to set up a generalized macroscopic relation linking the average stress and higher-order stress tensors to the first and second gradients of the transformation. The implementation of the second-order homogenization shows that concerning a first order formulation, the second-order theory brings significant improvement when three conditions are met, (a) fiber reinforcement, (b) high stiffness contrast between fibers and matrix, (c) the elastic-plastic matrix exhibits little hardening^[227]. For UHPC, it is very consistent with these premises, and the second-order homogenization calculation framework can be used to predict the macro performance of UHPC reasonably. The specific finite element discretization and solution strategies can be found in reference^[228, 229]. UHPC calculation parameters can be selected from the reference^[99] and listed in Table 7.2, in which, directions 1, 2 and 3 indicate that along the fiber direction, perpendicular to the fiber direction and another orthogonal direction, respectively.

Table 7.2 UHPC material parameters for calculation.^[99]

Parameter	Value
Young's Modulus (1)	45000 MPa
Young's Modulus (2/3)	44500 MPa
Shear Modulus (1-2,1-3)	18700 MPa
Shear Modulus (2-3)	18400 MPa
Poisson's ratio	0.2
Longitudinal Tensile Strength (1)	8.5 MPa
Longitudinal Tensile Strength (2/3)	8.0 MPa

Longitudinal Compressive Strength (1)	145 MPa
Longitudinal Compressive Strength (2/3)	135 MPa
Shear Strength (1-2)	49.5 MPa
Shear Strength (1-2, 1-3)	47.5 MPa
Fracture Energy (1)	6.67 N·mm
Fracture Energy (2/3)	6.10 N·mm

7.2.1.3 Constitutive model of interface

The interface between NC and UHPC is described with the cohesive zone model (CZM). Its strength, energy release rate, and specific evolution behavior of elements can be determined by the TSL. Generally, the TSL shape has no significant influence on the overall mechanical behaviors [230], and the bilinear TSL as Eq.(6.28) is adopted,

$$\mathbf{t}_c = (1 - D_{CZM})(k_n \langle \llbracket u_n \rrbracket \rangle_+ \mathbf{n}_c + k_s \llbracket u_s \rrbracket \mathbf{s}_c) + k_n \langle \llbracket u_n \rrbracket \rangle_- \mathbf{n}_c \quad (7.28)$$

where k_I and u_I are the elastic stiffness and separation, while $I=n$ and $I=s$ represents the normal and shear direction, respectively. $\llbracket \cdot \rrbracket$ represents the jump operator across the interface. $\mathbf{n}_c$ and $\mathbf{n}_s$ is the local interface orientation. The Macaulay bracket $\langle x \rangle_{\pm} = \frac{1}{2}(x \pm |x|)$ can avoid the interface penetration under compression loading. The damage variable D_{CZM} ranges between 0 indicate an undamaged state and 1 indicates complete failure. The damage is initialized after the linear slope and triggered by the quadratic stress criterion,

$$\left(\frac{\langle t_n \rangle}{t_n^0} \right)^2 + \left(\frac{t_s}{t_s^0} \right)^2 = 1 \quad (7.29)$$

where the t_I is the traction and t_I^0 is the cohesive strength. After traction exceeds the critical traction, a linear decrease of the elastic stiffness k_I follows up to full debonded determined by the power law criterion,

$$\left(\frac{G_I}{G_{cl}} \right)^{\alpha} + \left(\frac{G_{II}}{G_{cII}} \right)^{\alpha} = 1 \quad (7.30)$$

in which, G_I is the pure mode energy release rate and G_{cl} is the critical energy release rate, $I = I, II$ represents the mode I and mode II failure of the interface, and the exponent α is to weight the mixed mode behavior. The separation can be expressed as $\delta_m = \sqrt{\langle u_n \rangle^2 + u_s^2}$, and the

damage driving value is,

$$D_{CZM} = \frac{\delta_m^f (\delta_m^{\max} - \delta_m^0)}{\delta_m^{\max} (\delta_m^f - \delta_m^0)}, \quad \delta_m^{\max} = \max_{\tau \leq t} [\delta_m(\tau)] \quad (7.31)$$

The mixed mode separation at damage onset δ_m^0 and the separation δ_m^f at the fully damaged state. The specific parameters adopted for the interface material are listed in Table 7.3.

Table 7.3 Calculation parameters for ITZ [231].

Density (kg/mm ³)	Elastic Modulus (MPa)	Poisson's ratio	Tension strength (MPa)	Compression strength (MPa)	Fracture energy (N/mm)
2.2e-6	15	0.2	3	35	0.015

7.2.2 Homogenization of the interface

As shown in Figure 7.1, on the macro-scale, considering a domain Ω^M with the external boundary Γ^M , which is configured with the displacement $\mathbf{u}^M$ and traction $\mathbf{t}^M$ at Γ_u^M and Γ_t^M , respectively. The interface layer embedded in Ω^M is defined as Γ_{coh}^M . For a point of the macro-scale interface Ω^M , it can be regarded as a corresponding mesoscale domain of Ω^m , and it is considered to be the RVE of the research object of interface debonding behavior. It is assumed that the width of the mesoscale representative domain is b , the mesoscopic debonding failure path is Γ_{coh}^m , and the external boundary Γ^m of the mesoscale domain is symmetrical at the left (Γ_L^m) and the right (Γ_R^m) side.

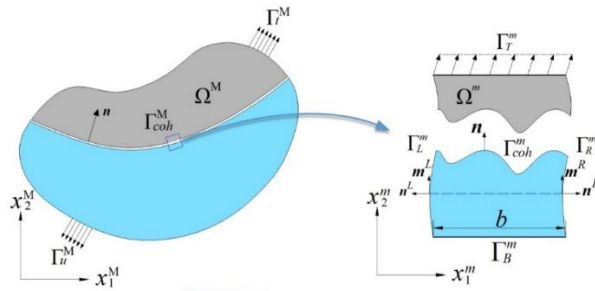


Figure 7.1 Transformation of the macroscopic domain Ω^M and the mesoscopic domain Ω^m .

According to the homogenization concept proposed by Hill^[232, 233], the average of the deformation gradient $\mathbf{F}$ over the RVE domain is,

$$\langle \mathbf{F} \rangle = \frac{1}{V_0} \int_{\Omega^m} \mathbf{F} dV \quad (7.32)$$

The volume average of Piola-Kirchhoff stress $\mathbf{P}$ ^[234] is given as,

$$\langle \mathbf{P} \rangle = \frac{1}{V_0} \int_{\Omega^m} \mathbf{P} dV \quad (7.33)$$

Then the connection between the mesoscale and macro scale can be expressed as,

$$\langle \mathbf{P} \rangle : \langle \dot{\mathbf{F}} \rangle = \langle \mathbf{P} : \dot{\mathbf{F}} \rangle \quad (7.34)$$

For the large deformation behavior of the thin layer and interface, the macro scale traction separation relationship proposed by Hirschberger et al.^[235] is adopted, which relates the macroscopic interface kinematics $\bar{\phi}$ and interface traction $\bar{\mathbf{t}}$ as,

$$\langle \mathbf{F} \rangle = \mathbf{I} + \frac{1}{h_0} \llbracket \bar{\phi} \rrbracket \otimes \bar{\mathbf{N}} \quad \text{and} \quad \bar{\mathbf{t}} = \bar{\mathbf{N}} \cdot \langle \mathbf{P} \rangle \quad (7.35)$$

where h_0 is the RVE initial height, $\bar{\mathbf{N}}$ is the initial unite normal vector on the Γ_{coh}^M , combining Eq.(7.34) and (7.35), an equivalent condition for the interface can get as,

$$\bar{\mathbf{t}} \cdot \llbracket \dot{\bar{\phi}} \rrbracket = h_0 \langle \mathbf{P} : \dot{\mathbf{F}} \rangle \quad (7.36)$$

To satisfy the equivalent condition in Eq.(7.36), the hybrid boundary conditions proposed by Hirschberger are applied^[235]. The boundary conditions are divided into two parts: the prescribed displacements on the top and bottom faces of an RVE, and the periodic displacements with antiperiodic fluxes on the left and right faces. The displacement and traction boundary conditions are expressed as,

$$\mathbf{u} \Big|_{u \in \Gamma_u^m} = (\bar{\mathbf{F}} - \mathbf{I}) \cdot \mathbf{X} + \tilde{\mathbf{u}} \quad (7.37)$$

$$\mathbf{t} \Big|_{t \in \Gamma_t^m} = \bar{\mathbf{N}} \cdot \mathbf{P} \quad (7.38)$$

Specifically, $\tilde{\mathbf{u}}$ denotes the fluctuation component of displacement satisfies $\tilde{\mathbf{u}}^{1,2+} = \tilde{\mathbf{u}}^{1,2-}$ and $\tilde{\mathbf{u}}^{3+} = \tilde{\mathbf{u}}^{3-} = 0$, while the traction components satisfy $\mathbf{t}^{1,2+} = -\mathbf{t}^{1,2-}$. For the nonlocal field quantities $\bar{\varepsilon}$, the homogeneous Neumann boundary condition is used as the material boundary condition. The symmetric boundary condition is consistent with the corresponding symmetric displacement field,

$$\bar{\varepsilon}^{1,2+} = \bar{\varepsilon}^{1,2-} \quad (7.39)$$

$$p = \mathbf{J} \mathbf{F}^{-l} \cdot \nabla \bar{\varepsilon} \cdot \bar{\mathbf{N}} \quad (7.40)$$

with the constraints, i.e. $p^{1,2+} = -p^{1,2-}$ and $p^{3+} = p^{3-} = 0$ on the RVE boundary.

Then After incorporating the volume average of the mesoscopic Piola-Kirchhoff stress in Eq. (7.33) and the antiperiodic traction in Eq. (7.38), the equivalent traction in Eq. (7.36) can be further converted by the hybrid boundary conditions as,

$$\bar{\mathbf{t}} = \frac{1}{b} \int_{\Gamma_T^m} \mathbf{t} d\Gamma_i^m \quad (7.41)$$

With the area b of the positive boundary Γ_T^m of the undeformed RVE. With the homogenization of the mesoscopic traction over the top surface of the RVE, the equivalent traction for arbitrary microstructures can be performed.

7.2.3 RVE generation

The generation process of a 2D NC-UHPC RVE is as follows. (1) The R_q values corresponding to different treatment methods were obtained by scanning the rough surface of substrate concrete. (2) The equivalent random field surface curve is generated according to the real roughness value and the method proposed by Temizer^[236]. After calculating the autocorrelation length of each curve, it is determined that the plane size of RVE is 10 mm × 10 mm, and the thickness is set as 1 mm with the default value. (3) The random aggregate generation method proposed by Li^[237] is used to generate 2D periodic fine aggregates distributed inside NC, and the generated interface curve is transformed into NURBS description, which corresponds to the mesh discretization of the numerical model. (4) The RVE is discretized and 0-thick cohesive elements are generated at the interface between NC and UHPC. After being imported into ABAQUS, different material properties are assigned separately.

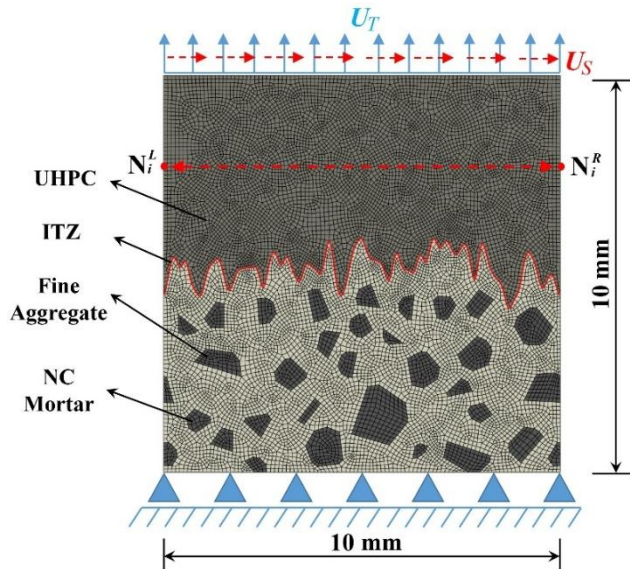


Figure 7.2 Meso-scale RVE with the rough interface of NC-UHPC.

It is noted that the elastoplastic constitutive model coupled gradient-enhanced damage is defined by the user-defined program subroutine UMAT. Finally, a python script is executed to impose periodic boundary conditions on both sides of the RVE, as shown in Figure 7.2. Specifically, node pairs are defined at the node positions with the same height on the left and right interfaces of RVE, and their degrees of freedom can be defined by Eq. (7.42),

$$U_{(x,y)}^{N_L} - U_{(x,y)}^{N_R} = 0 \quad (7.42)$$

where, $U_{(x,y)}^{N_L}$ and $U_{(x,y)}^{N_R}$ is the displacement of any node on the left boundary surface and the corresponding node on the right boundary surface, respectively.

7.3 NC-UHPC interface separation test

The interfacial debonding failure performance between NC (substrate) and UHPC (coating) was experimentally investigated by pull-off tests on 72 drilled-core samples and bi-shear tests on 36 samples, considering 2 strength grades of the substrate concrete and 6 kinds of interface roughness.

7.3.1 Materials

As listed in Table 7.4, raw materials of the normal concrete include two kinds of powder materials, the P.C 42.5 cement and the fly ash, the natural river sand with the particle size range of 0-4.75 mm, the broken basalt with the particle size range of 4.75-19.00 mm, and the superplasticizer. The proportion of UHPC refers to our previous research [183]. The three kinds of powder materials are the P II-52.5 cement, the quartz powder and the silica fume. Besides, the superplasticizer, the quartz sands with two particle sizes, the crushed basalt, and the microfilament steel fiber are also included. The mechanical properties of NC and UHPC are listed in Table 7.5. The UHPC overlay was cast after 28 days of standard curing of the substrate concrete. The tests were carried out after another standard curing of 28 days.

Table 7.4 NC and UHPC mixtures (kg/m³).

	Raw materials	Amount
NC(C40/C50)	Cement (42.5R)	350.0/420.0
	Fly ash	95.0/90.0
	Water	175.0/170.0
	Superplasticizer	7.1/8.2
	Sand	750.0/680.0

	Coarse aggregate	1040.0/1060.0
	Cement (52.5R)	680.0
	Silica fume	81.6
	Quartz flour	272.0
	Quartz sand (0.15-0.20 mm)	169.8
UHPC	Quartz sand (0.25-0.60 mm)	433.6
	Superplasticizer	17.0
	Water	159.8
	Basalt aggregate	581.0
	Steel fiber	117.0

Table 7.5 Mechanical properties of the NC and UHPC (MPa).

Item	NC-C40	NC-C50	UHPC
Cubic compressive strength	45.7	56.1	135.5
Tensile strength	2.9	3.2	9.1
Young's modulus	33500	34520	44500

7.3.2 Interface preparation

In this research, six technical treatment solutions were used to prepare the NC substrate surface, gaining six kinds of interfacial roughness, as shown in Figure 7.3. The treated substrate surface named the slight roughing, the heavy roughing, and the prick roughing were prepared after the initial setting of NC (Figure 7.3 (a) - (c)). The other three kinds of chiseled samples with the chiseling spacing of 2 cm, 4 cm, and 6 cm respectively were prepared after 28 days of curing (Figure 7.3 (d) - (f)). The roughness of the treated substrate surface was measured by a 3D digital microscope with the super depth of field. The measured root mean square depth R_q of the six surfaces is 0.41 mm, 1.31 mm, 0.91 mm, 0.66 mm, 0.47 mm, and 0.95 mm, respectively.

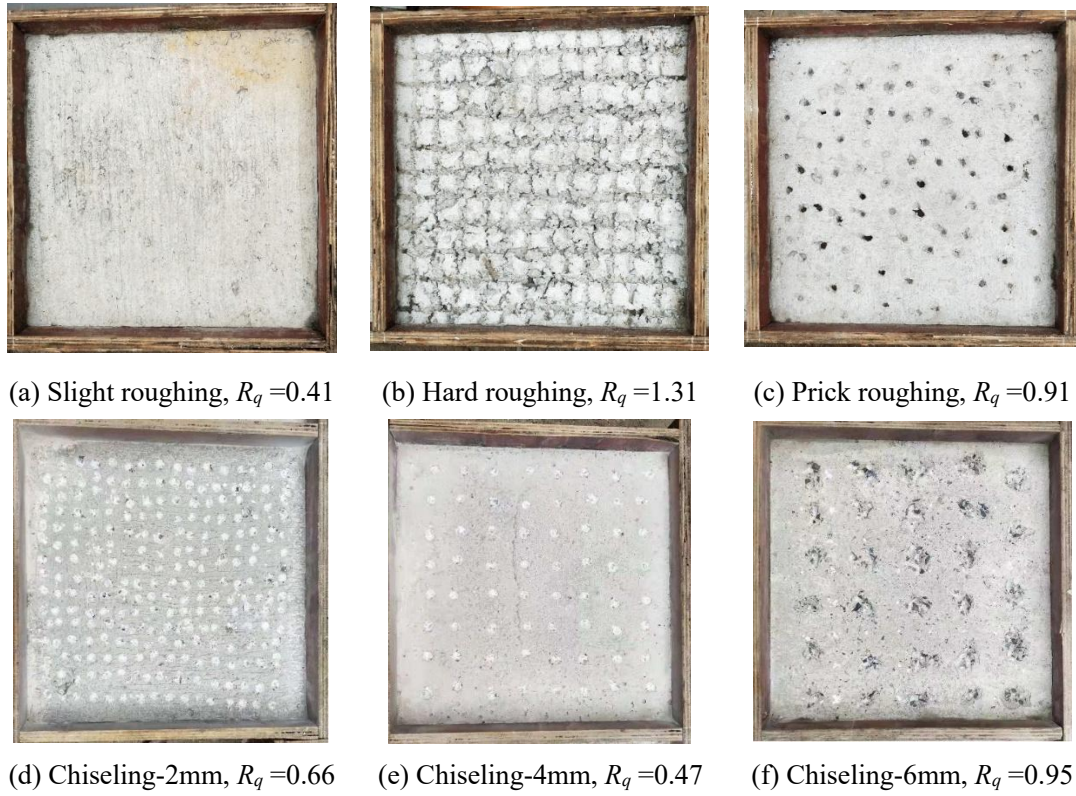
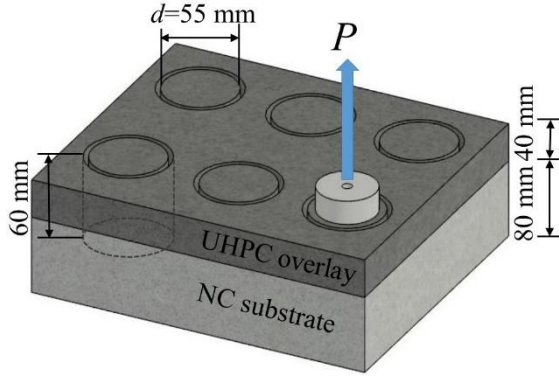


Figure 7.3 Treated surface of the substrate. (Unit. mm)

7.3.3 Test setup

The pull-off test and the bi-surface shear test are the widely adopted experimental methods to evaluate the interfacial performance between cement-based materials under pure tension and under pure shear [115, 238, 239]. In this study, a total of 12 combinations of interface roughness and substrate strength grade were considered, and 6 groups of pull-off tests and 3 groups of bi-shear tests were carried out for each combination. Figure 7.4 shows the test setup of the pull-off test (PO). The core samples are drilled on the composite slab with an 80 mm thick NC substrate and 40 mm thick UHPC overlay. Referring to the international standard C1583/C1583M^[240], the diameter of a drilled core is 55 mm, and the drilled depth is 60 mm. The samples were centrally pulled off at the rate of 85 N/s till failure, and the peak loads were recorded. An HC-6000C tester produced by Suzhou Zhonglutong Test Instrument Co., Ltd.) was adopted during the test.

The bi-surface shear test (BS) setup is shown in Figure 7.5. The overall size of the cubic sample is 150 mm × 150 mm × 150 mm, of which the NC thickness is 100 mm and the UHPC thickness is 50 mm. The cubic sample is uniaxial compressed. A preloading to 10 kN was carried out to check the test system. After the preloading, the loading was applied at the rate of 0.2 mm / min until failure, and the values of the loading and the displacement were recorded synchronously.

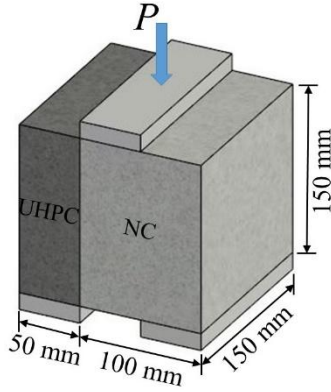


(a) layout and dimensions of the pull-off test.



(b) testing specimen.

Figure 7.4 Test setup of the pull-off test (PO).



(a) layout and dimensions of the bi-surface shear test.



(b) testing specimen.

Figure 7.5 Test setup of the bi-surface shear test (BS).

7.3.4 Extraction of surface roughness

The roughness of the bonding surface has a significant impact on the behavior of the NC-UHPC interface. Therefore, an appropriate quantitative method is needed to characterize the roughness and geometric characteristics of the bonding surface. For the concrete materials, their natural surfaces or treated surfaces (roughening, high-pressure water spraying, etc.) often show highly random roughness characteristics. To characterize the effect of roughness on the bond failure behavior of NC-UHPC composite components, in this study, the widely used root mean square roughness R_q is used to characterize the surface roughness,

$$R_q = \sqrt{\frac{1}{A} \iint_A h^2(x,y) dx dy} \quad (7.43)$$

where, A is the calculation area, (x,y) represents the position corresponding to the surface point

with the depth of h .

Considering the convergence problem of 3D numerical calculation, the actual 3D surface is simplified to the 2D plane, as shown in Figure 7.6 (a). Accordingly, R_q is expressed as,

$$R_q = \sqrt{\frac{1}{L} \int_0^L (h(x) - \bar{h})^2 dx} \quad (7.44)$$

where, the calculation area A degenerates into the sample length L , and $h(x,y)$ becomes $h(x)$. Furthermore, the height profile spacing is evaluated by the correlation length L_c , and the normalized autocorrelation function (ACF) represents the correlation degree with a shift copy of itself, which can be expressed as,

$$C(\tau) = \lim_{L \rightarrow \infty} \frac{1}{\sigma^2 L} \int_0^L h(x)h(x+\tau)dx \quad (7.45)$$

where σ is the standard deviation and τ is shift length, while $\tau=0$ corresponding to the depth profile is completely correlated to itself and $C(\tau)=1$. As the shift spacing increases, the autocorrelation value decreases, and the correlation length l_{ACF} is used as a critical spacing parameter. Generally, the l_{ACF} corresponding to $C(\tau)=0.5$ is taken as the threshold characteristic length of the surface roughness, therefore, a dimensionless roughness parameter R related to l_{ACF} can be expressed as,

$$R = \frac{R_q}{l_{ACF}} \quad (7.46)$$

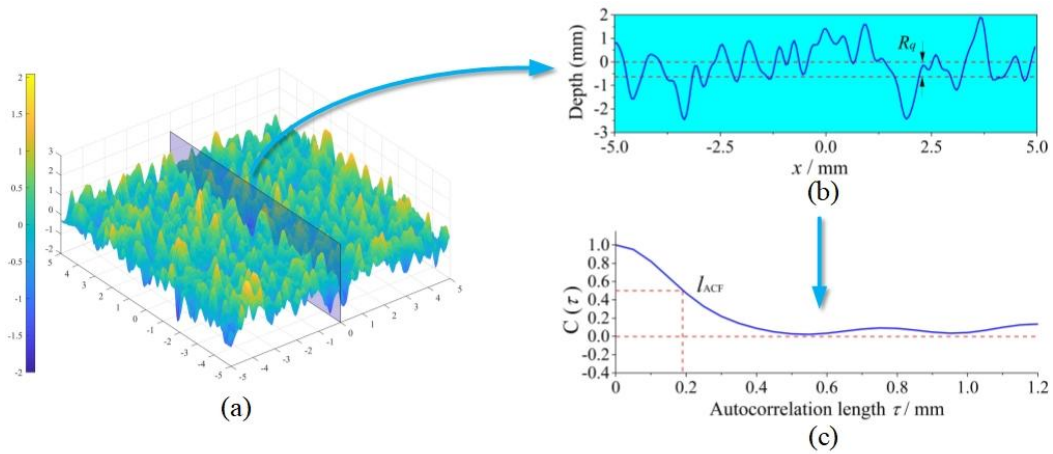


Figure 7.6 Section extraction process of the 3D roughness surface, (a) random 3D surface, (b)

2D depth distribution, (c) normalized ACF calculation result.

7.4 Analysis of NC-UHPC debonding failure results

7.4.1 Interface roughness analysis

Six types of random 2D rough surfaces were generated by the Python script, and the surface roughness parameters R_q are consistent with the measured values of the six kinds of treated substrate surfaces shown in Figure 6.2. The generated surfaces are named as R041, R047, R066, R091, R095, and R131 according to the R_q values and their profiles are plotted in Figure 6.7. It is obvious that the maximum height of profile difference of the surface increases with the increase of roughness, and the values for the above surfaces are 1.77 mm, 1.99 mm, 3.19 mm, 3.52 mm, 4.36 mm, and 6.39 mm. The corresponding autocorrelation functions are shown in Figure 6.8. The length l_{ACF} of the autocorrelation curve corresponding to $C(\tau)=0.5$ is 3.5 mm, 3.9 mm, 4.5 mm, 5.9 mm, 6.1 mm, and 7.1 mm respectively. Therefore, the mesoscale RVE size of 10mm can reflect the roughness characteristics of different surfaces well. The Gaussian distribution fitting results of the surface profile distribution histogram with different roughness are shown in Figure 7.9. It shows that with the increase of roughness, the expectation of the arithmetic mean deviation of the surface depth is basically unchanged, and the variance of surface depth distribution σ^2 is equal to R_q of surface roughness characteristics, which increases with the increase of the roughness. The difference between R_q and Gaussian fitting variance σ^2 of these six random surfaces with different roughness is 4.4 %, 5.0 %, 1.9 %, 3.4 %, 2.6 %, and 0.2 % respectively.

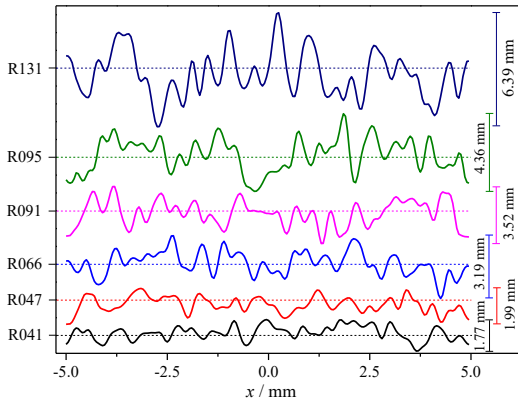


Figure 7.7. Surface profiles of the various surfaces.

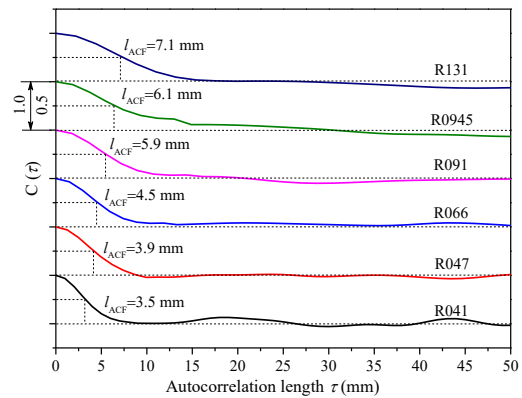


Figure 7.8. Autocorrelation functions for the various surfaces.

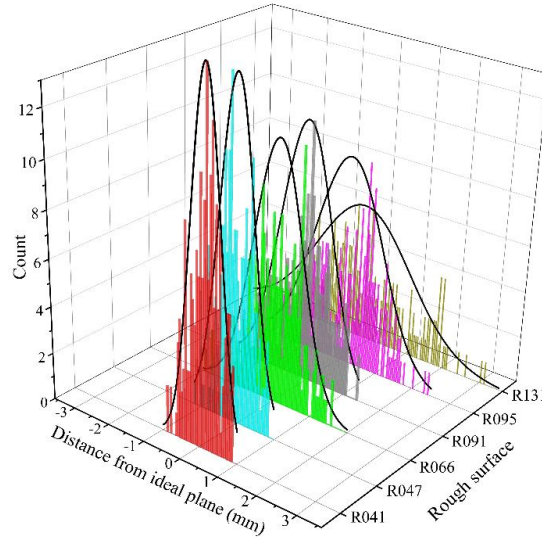


Figure 7.9. Surface depth distribution histogram and Gaussian fitting results.

7.4.2 Effective T-S results and damage distribution under tension

7.4.2.1 Effect of interface roughness

The effective TSL curves with different surface roughness and substrate concrete strengths are given in Figure 7.10. With the increase of roughness, the shape of the effective TSL curve changes. The curves firstly experience the linear rising stage, and the rising slopes increase as the substrate concrete strength increase. With low roughness ($R_q = 0.41$ mm or $R_q = 0.47$ mm), the effective TSL is close to the bilinear variation while nonlinearity becomes obvious with high roughness, especially in the post-peak stage. The reason lies in that some parts of the NC substrate and the UHPC near the interface also bear tensile stress with high roughness. As interfacial failure, the NC near the interface also degrades nonlinearly, resulting in the nonlinearity of the effective TSL. On the other hand, the rough bonding interface makes the stress distribution more complex and uneven, leading to the nonlinear performance of the effective TSL. Figure 7.11 compared the simulated tensile traction of the NC-UHPC interface and the experimental results. The effective tensile strength shows a strong linear correlation with the surface roughness. The numerical tensile traction is wrapped by the experimental results correspondingly, which verifies the feasibility of the numerical calculation method. The linear correlation coefficient between the tensile strength and the surface roughness is more than 80%. The tensile strength of the specimen with interface roughness $R_q = 1.31$ mm is 100 % and 64 % higher than that of the specimen with roughness $R_q = 0.41$ mm, respectively with C40 and C50 substrate concrete.

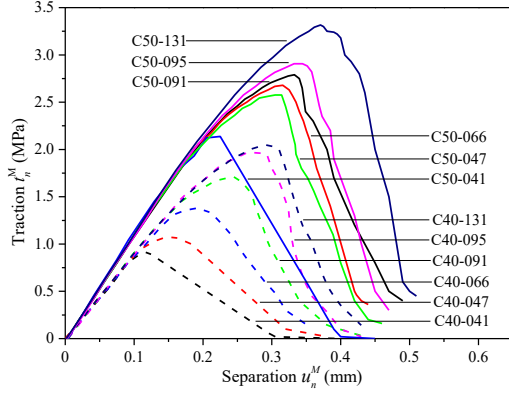


Figure 7.10. Effective tensile TSL with different surface roughness and substrate concrete strengths.

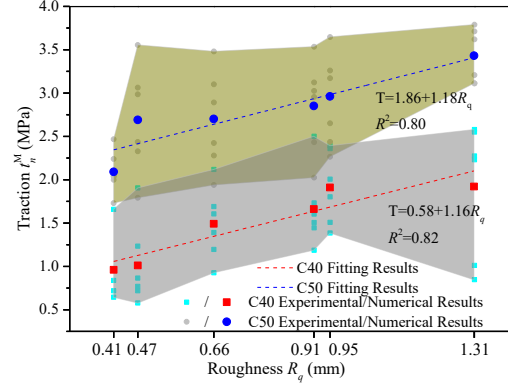


Figure 7.11. Effective tensile traction of the homogenized RVE with different surface roughness and substrate concrete strengths.

The damage distribution in the mesoscale RVE under tension is shown in Figure 7.12. The damage of UHPC and NC is represented by a unified damage legend, and the cohesive element of interface damage is shown by a red line. It is clear that the damage distribution significantly varies with different roughness. The damaged area in NC is small with low roughness. For an RVE with $R_q = 0.41$ mm, the maximum damage of NC tension is $d = 0.83$, and the number of the damaged elements at the interface with $d > 0.95$ accounts for 96%. In contrast, the UHPC is not damaged. Similarly, the number of damaged elements at the interface with $d > 0.95$ accounts for 92% and no damage is observed in UHPC when $R_q = 0.66$ mm. It is indicated that the damage of the RVE under tension is determined by the bond failure with low roughness since the strength of ITZ is lower than that of NC and UHPC. As the roughness increases, as shown in Figure 7.12(c), microscopic damage begins to initiate in the final tensile state, which is the consequence of the stress concentration in the local area caused by the increase of the uneven degree of interface stress. In addition, the cohesive element also has discontinuous damage distribution due to the uneven stress distribution. As the roughness goes up even higher, Both the damage areas in NC and UHPC become larger, as shown in Figure 7.12 (d), (e), and (f). It can be seen from the damage distribution results that the NC damage distribution is mainly concentrated on both sides of the 'peak' of the rough surface, while there is no damage at the top of the NC surface peak. Similarly, the damage of UHPC also occurs far away from its' peak ', and most of the damage position of UHPC is located at the same height as the boundary of the NC damage area. For an NC-UHPC pull-off specimen with high roughness, the failed interface presents an irregular fracture surface in which UHPC and NC are embedded with each other, which has been observed in both the numerical and the experimental

work(Figure 7.13). It is because of the increased interface boundary length along with the same size of the RVE. Although the strength of ITZ is low, after the length increases, the external work that overcomes interface damage is greater than that overcomes UHPC damage with a short path in the process of tensile failure. Therefore, fracture failure also appears in the UHPC part.

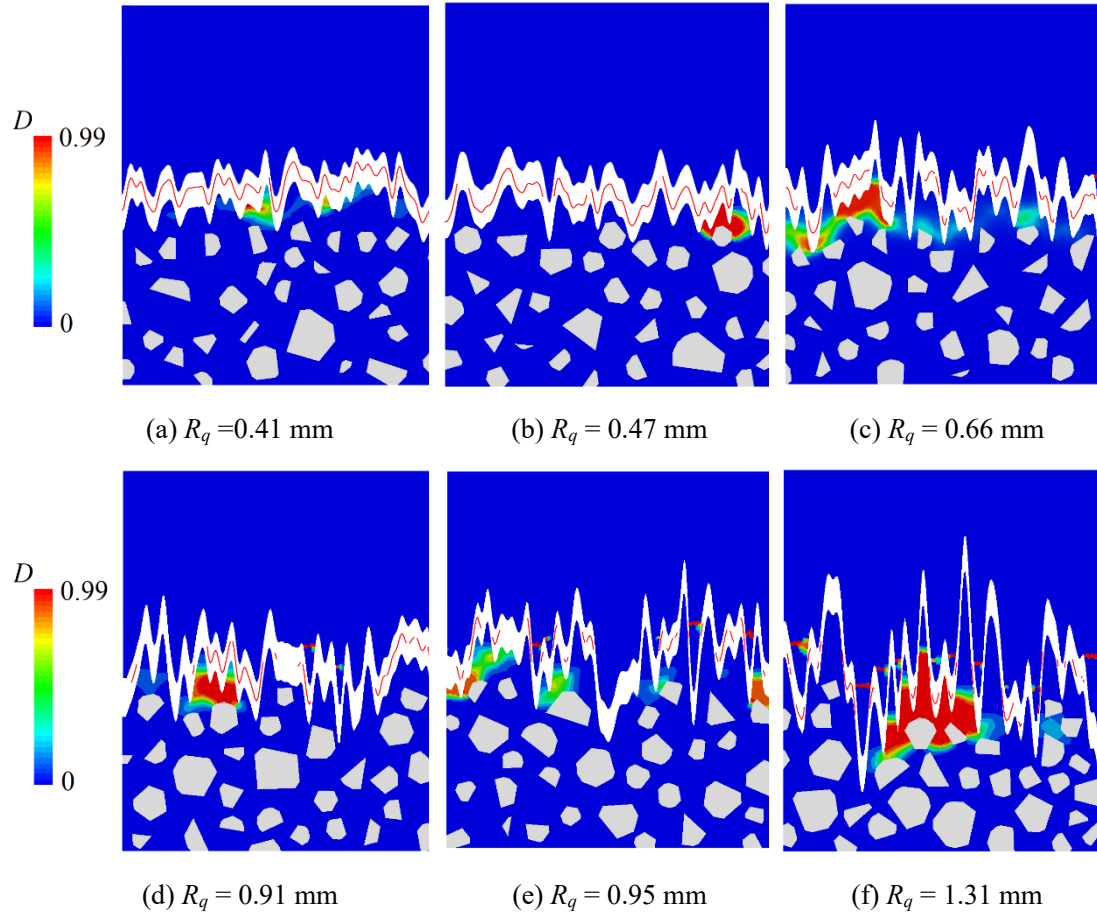


Figure 7.12. Damage distribution of the mesoscale RVE under tension.



Figure 7.13. NC-UHPC tensile failure surface.

7.4.2.2 Effect of interface strength

Taking the sample with interface roughness $R_q = 0.47$ mm as an example, the tensile damage distribution in the RVEs is shown in Figure 7.14. It can be seen that the tensile damage distribution in an RVE changes significantly with the increase of the interface strength. When the interface strength is low, damage only occurs in the interface area, as shown in Figure 7.14 (a). Accordingly, it can be seen from the effective TSL results in Figure 7.15 that the effective peak tensile traction is the smallest, and the TSL fits the obvious bilinear curve, which is consistent with the TSL constitution defined in the interface element.

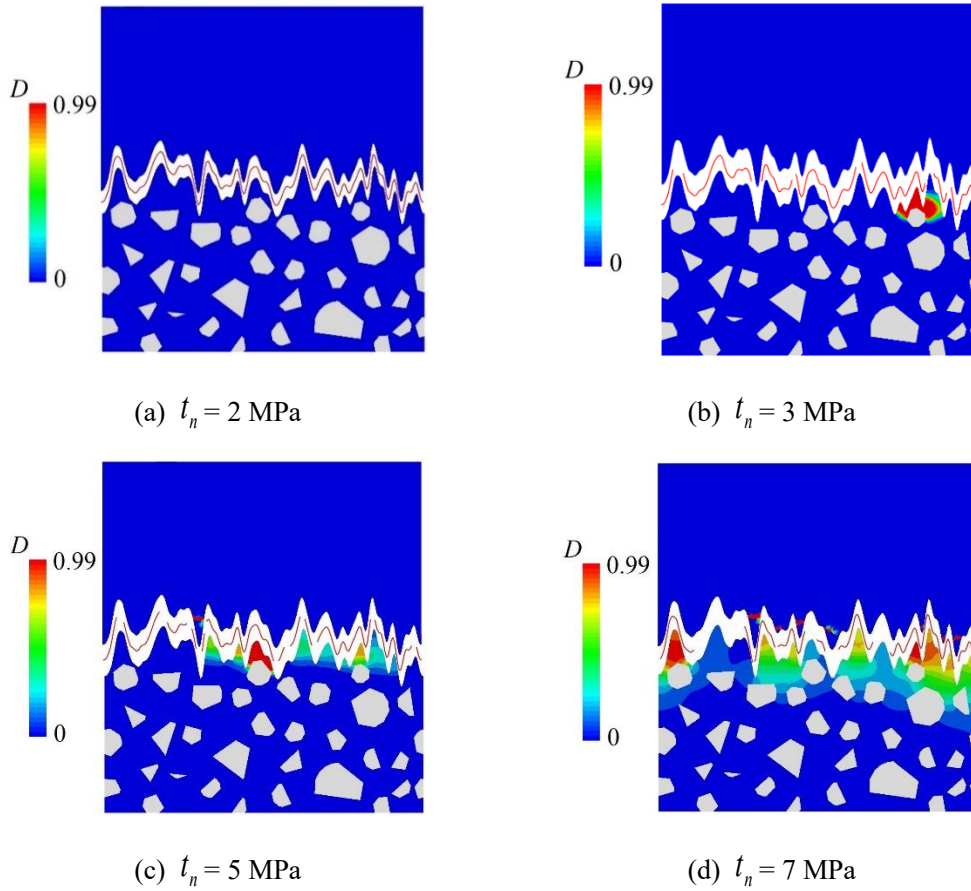


Figure 7.14 Tensile damage distribution of the RVE with roughness $R_q = 0.47$ mm for different interface strengths.

When the interface strength increases to 3MPa, the non-uniformity of local stress distribution causes discontinuous damage distribution in the interface element, and a nonlinear transition appears at the peak tensile traction in the TSL. As the interface strength continues to increase, the damaged area in NC enlarges and distributes in multiple 'peak' areas of the NC rough interface. Also, damage in UHPC generates and enlarges. The effective TSL no longer shows linear decline after the peak

but a sharp decrease. The UHPC damage distribution in the RVE with $t_n=7$ MPa is similar to that in Figure 7.12 (f), and the effective post-peak part of TSL also shows the nonlinear degeneration with the largest tensile strength. Compared with the corresponding effective tensile traction 1.78 MPa of the 2 MPa interface strength, When the interface strength becomes 3 MPa, 5 MPa, and 7 MPa, the effective shear traction is 2.45 MPa, 3.59 MPa, and 5.42 MPa respectively.

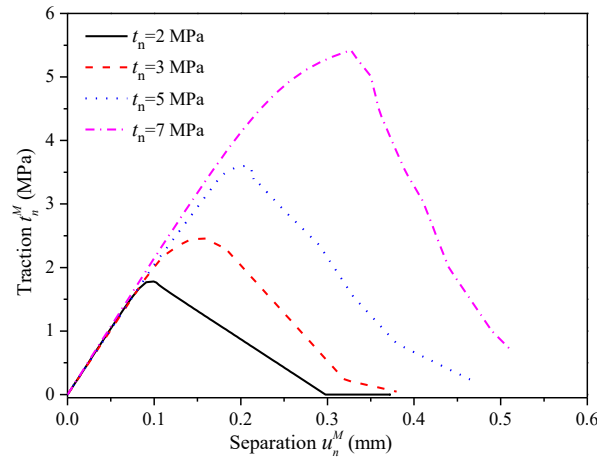


Figure 7.15. Effective traction-separation results of the RVE with different interface strengths.

7.4.3 Effective T-S results and damage distribution under shear

7.4.3.1 Effect of interface roughness

The effective TSL under shear is shown in Figure 7.16. Similar to those under tension, the NC strength benefits to the stiffness in the linear rising stage. When the maximum shear traction is reached, the effective TSL curves vary with the interface roughness in the post-peak stage, more significantly than those under tension. When the roughness is small, ($R_q = 0.41$ mm or $R_q = 0.47$ mm) the shear effective traction decreases rapidly after a short nonlinear transition near the peak, resulting in quasi-brittle shear failure. As the roughness increases, the interlocking effect between NC and UHPC increases and more NC contributes to the shear resistance, leading to the significant nonlinear fluctuation of the effective TSL curves in the post-peak stage. For the interface with high roughness, the non-uniformity of stress under shear is higher than that under tension, thus there is a significant fluctuation in the effective TSL curve after the peak. When the critical interface shear dislocation displacement is reached, a quasi-brittle failure happens at the interface.

Figure 7.17 compares the simulated and the experimental results of the effective shear traction with different NC strengths and different roughness. The test results envelop the numerical results

where the effective shear traction value increases with the increase of roughness, verifying the feasibility of this multi-scale numerical method. The linear fitting degree between the numerical effective traction calculation results and the roughnesses corresponding to C40 and C50 substrate strength is 88% and 73% respectively. Compared with the shear traction of $R_q = 0.41$ mm, the shear traction t_s^M with roughness $R_q = 1.31$ mm is increased by 202 % and 121 % respectively of the C40 and C50 substrate strengths.

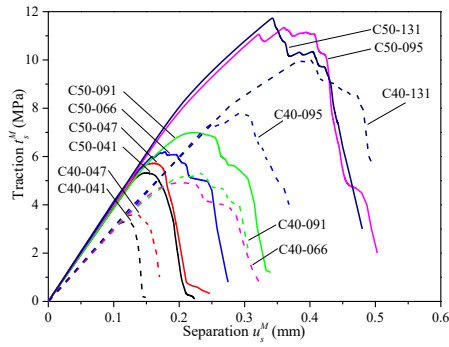


Figure 7.16 Effective TSL with different surface roughness and substrate concrete strengths.

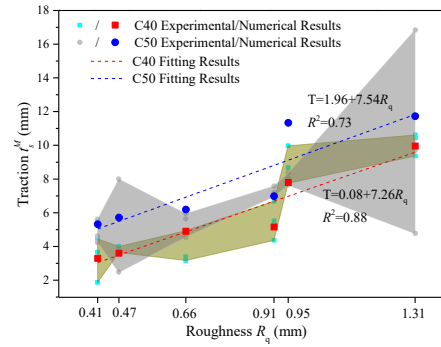


Figure 7.17 Effective shear traction of the homogenized RVE with different surface roughness and substrate concrete strengths.

The damage distribution of the RVE under shear is shown in Figure 7.18. The damage distribution under shear is different from that under the tensile stress. The damage in NC near the interface presents a horizontal band distribution under shear, while the damage under tensile is mainly due to the failure of the interface element, and the damage in NC occurs in one or several areas because of stress concentration. When subjected to shear loading, the roughness affects the width of the NC damage distribution band. When the roughness is low, most energy of the external force is offset by the failure of the low strength cohesive element in the RVE. Compared to the case with $R_q = 1.3$ mm, the area of NC bearing shear force is relatively small with $R_q = 0.41$ mm. This failure mechanism (failure dominated by the cohesive element) corresponds to the quasi-brittle shear failure characteristics of the effective TSL corresponding to $R_q = 0.41$ mm and $R_q = 0.47$ mm in Figure 7.16. When the roughness goes higher, the irregular interface morphology causes the high non-uniformity of stress distribution. At the same time, the random distribution of aggregate in NC will also affect the damage distribution in NC matrix near the interface. As shown in Figure 7.18 (c), the increase of roughness will also cause UHPC initiates damage, and then enhance the nonlinear characteristics in the post-peak stage of the effective TSL curve. With the increase of

interface roughness, the uneven distribution of shear stress near the interface increases. Due to the high strength of UHPC, the location of NC shear damage is concentrated in the horizontal position below the interface. The NC matrix embedded in UHPC is not damaged, so the final shear failure surface is relatively flat. As the bi-surface shear failure results of $R_q = 0.41$ mm and $R_q = 1.31$ mm are shown in Figure 7.19 the fracture surface of the specimen with the larger roughness is significantly rougher after shear failure, while the shear fracture surface of the specimen with smaller roughness is relatively smoother, which is in good agreement with the numerical simulation results.

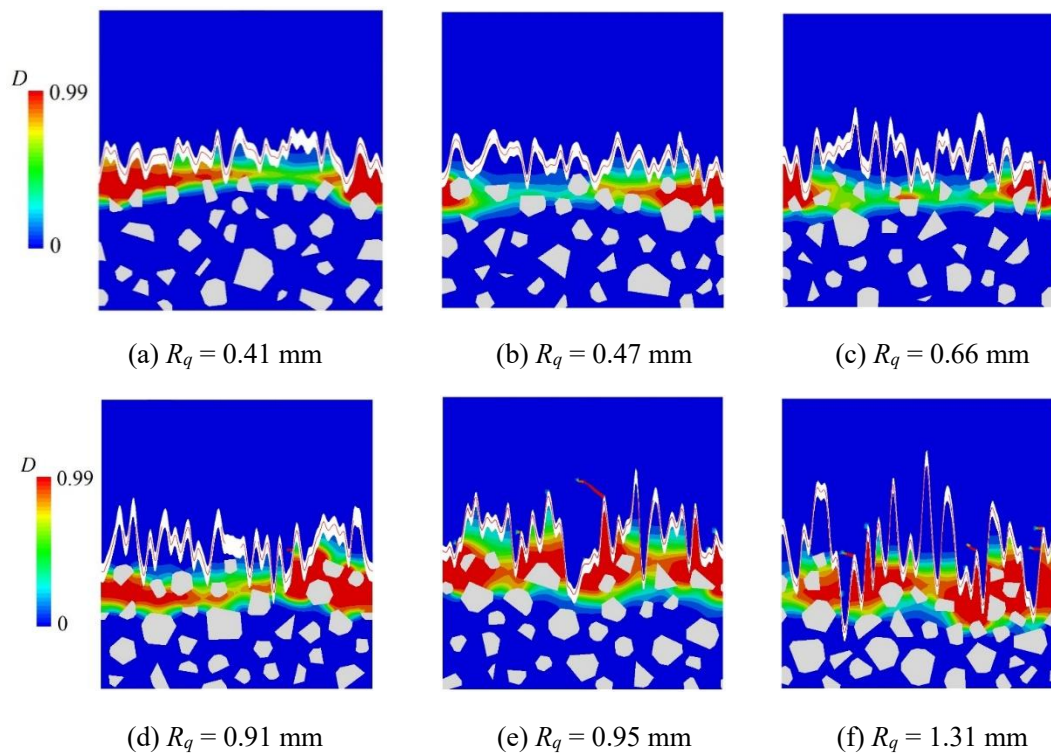


Figure 7.18 Damage distribution of the mesoscale RVE with shear stress.

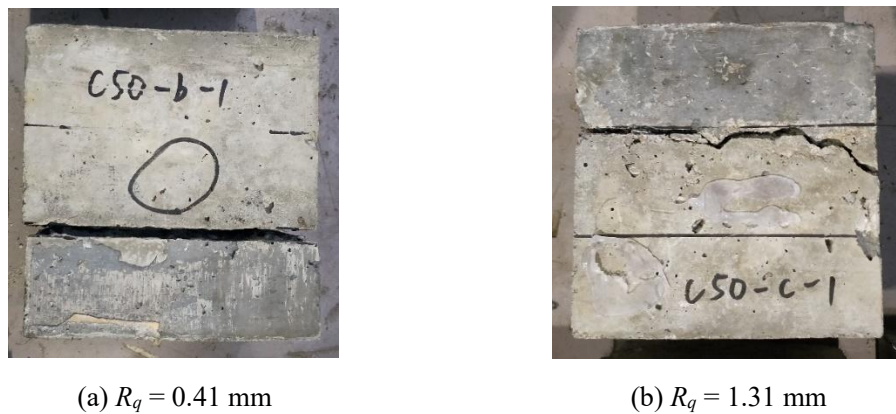


Figure 7.19. NC-UHPC bi-surface shear failure results with the different interface roughness.

7.4.3.2 Effect of interface strength

The distribution of the damage in the RVE with different interface strengths is shown in Figure 7.20, taking the specimen with roughness $R_q = 0.47$ mm as an example. It can be seen that almost all the interface elements are damaged under shear when the interface strength ranges from 2 MPa to 3 MPa (the NC matrix strength). What's more, the distribution of aggregates affects the stress distribution as well as the damage distribution.

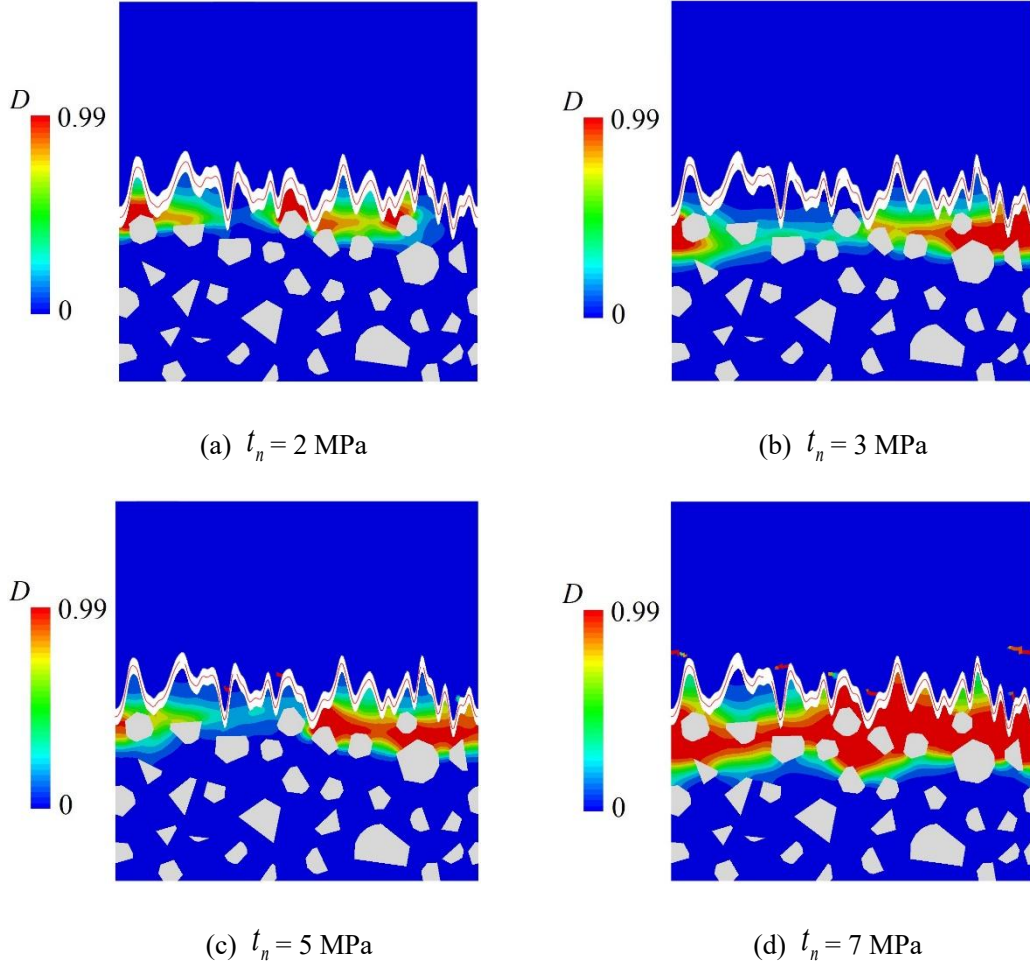


Figure 7.20. Shear damage distribution in the RVE with roughness $R_q = 0.47$ mm and different interface strengths.

As indicated in Figure 7.21, the effective TSL curves with low interfacial strength show obvious shear brittleness, which is dominated by the interface failure. As the interface strength increases to 3 MPa, the damaged band area in NC is formed at the end of the shear failure. Since the strength of the cohesive element is equal to that of the NC matrix, the damage variable of the cohesive elements and the NC embedded in the high strength UHPC does not reach 0.95, and the UHPC is also not damaged. When the interface strength exceeds the NC matrix strength and reaches 5MPa, the shear

damage band formed at the final failure becomes more and more obvious, as shown in Figure 7.20 (c). Meanwhile, the resistance contribution by NC increases and the nonlinearity of the effective TSL curves becomes more significant, which is also strengthened by the initiation of damage in UHPC. When the interface strength reaches 7MPa close to the UHPC tensile strength, it could be regarded as adding an interface binder to improve the interface bonding performance. The width of the final shear damage band is significantly larger than that in the case with lower interface strength. Simultaneously, the number of cracks in UHPC also increases, and the effective TSL curves show a larger traction value. Compared with the corresponding effective shear traction 4.15 MPa of the 2 MPa interface strength, When the interface strength becomes 3 MPa, 5 MPa, and 7 MPa, the effective shear traction is 5.72 MPa, 6.86 MPa, and 8.89MPa respectively.

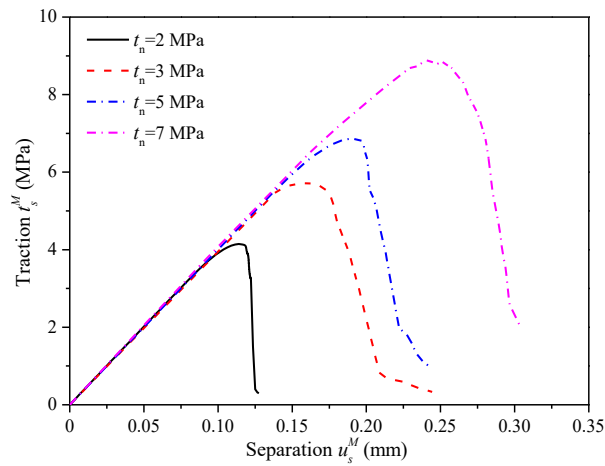


Figure 7.21 Effective traction-separation results of the RVE with different interface strengths.

7.5 Conclusion

The mechanical behavior of the interface zone of cement-based composites is very complex, and its performance is of great significance to ensure the reliability of the structure during service. This work takes the debonding failure behavior of the interface between NC and UHPC as the research object, and reveals the failure mechanism under the tensile and the shear loads, considering the influence of the interface roughness and the substrate concrete strength. Some meaningful conclusions obtained are as follows,

(1) Both the traction values under tension and shear are positively correlated with the interface roughness. As the surface roughness increases from 0.41 mm to 1.31 mm, the tensile traction values grow up by 100 % and 64 % respectively with substrate made of C40 concrete and C50 concrete. Meanwhile, the shear traction values grow up by 202 % and 121 % respectively.

(2) The shape of the effective TSL curves significantly changes with the increasing surface roughness. The TSL evolution is dominated by the interface failure with low roughness, and the curve shape is approximately bilinear. As the roughness increases, the non-uniformity of stress distribution becomes significant, and the curve shows obvious post-peak nonlinearity.

(3) the effective shear TSL shows obvious quasi-brittle failure characteristics. When the surface roughness rises, the interlocking effect between NC and UHPC is enhanced, resulting in significant nonlinear fluctuation of the effective TSL curve in the post-peak stage.

(4) Both of the effective TSL curves under tension and under shear are highly affected by the interfacial strength. The effective TSL curve with low interfacial strength shows obvious brittleness characteristics while that with high interfacial strength shows nonlinear characteristics in the post-peak stage.

(5) The maximum effective traction values experience an increase as the interfacial strength becomes higher. Taking the case with the roughness parameter of 0.47 mm as an example, the effective traction increases by 204 % under tension and by 114 % under shear when the interface strength varies from 2 MPa to 7 MPa.

Chapter 8 Investigation on the Bending Performance of RC-UHPC Composite Beams

8.1 Overview

The application of UHPC in the field of bridge structures is constantly expanding, and high-performance structural systems based on UHPC materials are also rapidly developing. This chapter focuses on the macroscopic bending performance of RC-UHPC composite beams and proposes a corresponding multi-scale analysis method for the performance of composite structures. Adopting the Mori Tanaka model with coupled progressive damage to evaluate the multi-scale macroscopic mechanical behavior of UHPC materials. The complex interface geometry structure at the microscale is described by a homogenization framework, and after obtaining effective mechanical responses, it is endowed with a macroscopic flat interface structure. Through experimental comparative research, the three-point bending failure modes, load deflection, and load interface slip curves of RC-UHPC composite beams under different interface roughness were verified and predicted. The research results found that the interface microstructural characteristics have a significant impact on the overall response of RC-UHPC composite components. The larger the roughness of the interface, the smaller the degree of interface damage under the same load level. The increase in interface roughness can enhance the overall collaborative stress performance of RC-UHPC composite beams, thereby reducing the risk of interface cracking.

8.2 Multi-scale analysis model for RC-UHPC composite structure

For the analysis of the bending performance of NC-UHPC composite beams, the microscale structural characteristics and damage failure process of the interface region RVE during loading are related to the macroscopic mechanical response. Therefore, it is necessary to establish a numerical homogenization computational framework for the RC-UHPC interface. In this chapter, following the method proposed by Matous^[241], the macroscopic and microscopic scales are linked through a homogenization process based on the Hill energy principle. The RVE boundary of the RC-UHPC interface region is endowed with local periodic boundary conditions. In order to incorporate real RC-UHPC interface geometry information into numerical calculations, the roughness of the composite beam interface was measured, and NURBS spline difference descriptions for numerical calculations were generated using Matlab programs, corresponding to the mesh discretization of the

numerical model. The specific interface homogenization calculation process in this chapter is the same as in Section 7.2, and will not be repeated here. Using the Mori Tanaka model coupled with progressive damage in Chapter 4 to evaluate the multi-scale macroscopic mechanical behavior of UHPC laminates. The RC foundation beam is analyzed and calculated using the widely used concrete damage plasticity model in Chapter 5. The specific calculation parameters can be found in the parameter lists of each section, and will not be listed one by one in this chapter. The multi-scale analysis and calculation process of RC-UHPC composite beams are shown in Figure 8.1.

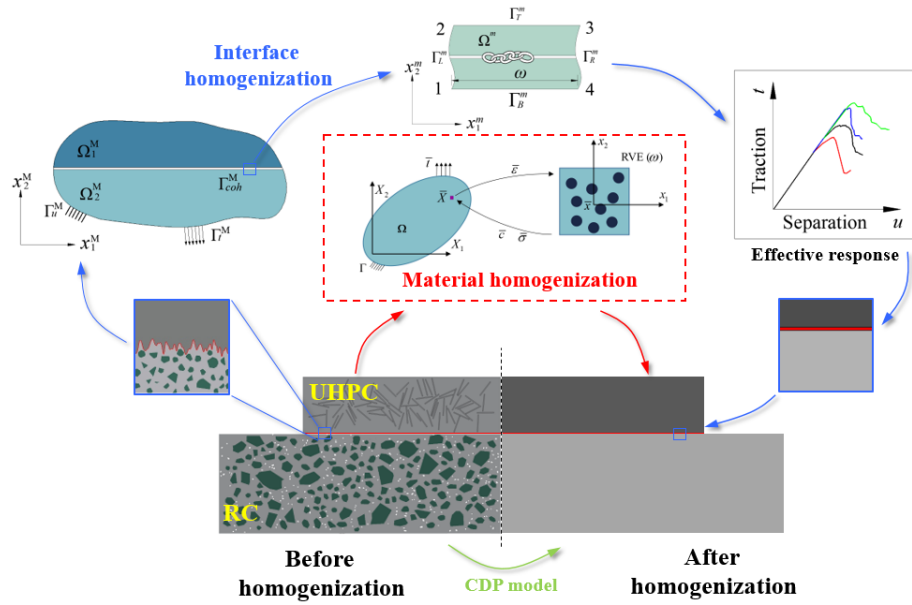


Figure 8.1 Multi-scale analysis process of RC-UHPC composite beam.

8.2.1 Materials

The raw materials of NC include P.O. 42.5 grade cement, fly ash, river sands, crushed basalts, water, and liquid polycarboxylic acid superplasticizer. The maximum particle size is 4.75 mm for the sand, and 10 mm for the basalts. The water reduction rate of the superplasticizer is 30 %. For the UHPC, the cementitious materials conclude P II. 52.5 grade cement, silica fume, and quartz powder. Two kinds of quartz sands with particle sizes of 0.15-0.20 mm and 0.25-0.60 mm were adopted as the fine aggregates. The crushed basalt with a particle size of 3-5 mm was taken as the coarse aggregates. The length and diameter of the steel fiber are 13 mm and 0.2 mm. Besides, water and superplasticizer were also included. In Table 8.1, the mix proportion of NC and UHPC are listed. The compressive strength is 56.1 MPa for NC and 135.5 MPa for UHPC. The tensile strength of the UHPC is 12.8 MPa.

The $\phi 6$ HPB235 rebar was used as the stirrup, and the $\phi 8$ HPB235 rebar was used as the erect

reinforcement. The yield stress is 235 MPa, and the ultimate stress is 310 MPa. The $\phi 16$ HRB335 rebar was used as the tensile bars with a yield strength of 335 MPa and an ultimate strength of 510 MPa. The elastic modulus of all the steel bars is 210 GPa.

Table 8.1 Mix proportion and properties of NC and UHPC.

Raw Materials	NC (kg/m ³)		UHPC (kg/m ³)	
Powder	Cement (42.5R)	420.0	Cement (52.5R)	680.0
	Fly ash	90.0	Silica fume	81.6
			Quartz flour	272.0
Water		170.0		159.8
Superplasticizer		8.2		17.0
Fine aggregate	River sand (0 -4.75 mm)	680.0	Quartz sand (0.15-0.20 mm)	169.8
			Quartz sand (0.25-0.60 mm)	433.6
Coarse aggregate	Broken basalt (4.75-19 mm)	1060.0	Broken basalt (3-5 mm)	581.0
Fiber	-		Steel fiber	117.0
Compressive strength	56.1 MPa		135.5 MPa	
tensile strength			12.8 MPa	

8.2.2 Preparation of composite girders with roughed interfaces

The composite girder is 1000 mm long, 100 mm wide, and 200 mm high, which is composed of a reinforced concrete substrate and a UHPC overlay, as shown in Figure 8.2. The height of the RC part is 140 mm, where longitudinal reinforced bars were designed, $\phi 16$ HRB335 at the bottom corners and $\phi 8$ HPB235 at the top corners. The distance from the bottom rebar to the bottom edge is 25 mm. The stirrup spacing is 60 ~ 80 mm. The length and thickness of the UHPC overlay are 700 mm and 60 mm, respectively.

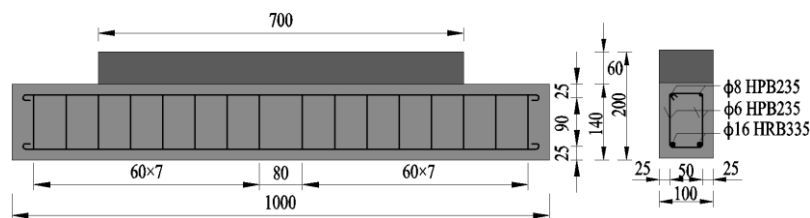


Figure 8.2 RC-UHPC composite beam size and the arrangement of the reinforced bars (mm).

The NC was first cast into the mold with an embedded reinforcement cage. After the initial setting of NC, the top surface was roughed by specific toothed steel plates, gaining three types of interfaces with different roughness as shown in Figure 8.3. The width of the teeth is 5 mm, and the depth is 3

mm, 5 mm, and 7 mm respectively showed in Figure 8.3. Correspondingly, the tested girders were named by interface types B3, B5, and B7. The roughness parameter of root mean square depth, R_q was measured to feature the treated interfaces, and it can be calculated as,

$$R_q = \sqrt{\frac{1}{A} \iint_A z^2(x, y) dx dy} \quad (8.1)$$

where A is the test area, and $z(x, y)$ is the values of peaks and valleys measured from the optimized plane. The values of R_q are 0.470 mm, 0.780 mm, and 1.095 mm, respectively. After 28 days curing of the RC part (humidity > 95%, temperature 20 ± 2 °C), the upper UHPC overlay was cast. Then, another 28-day curing was carried out before the test.

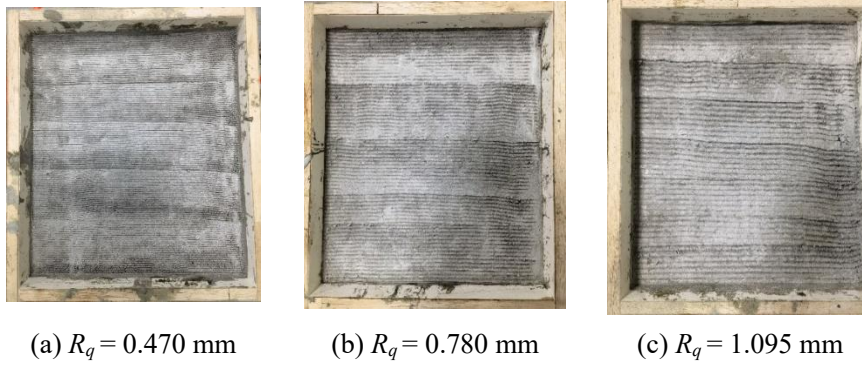


Figure 8.3 Diagrammatic sketch of the roughness interface.



Figure 8.4
Toothed steel
plates.

8.2.3 Test setup

In the test, the composite girder is simply supported with a span of 680 mm, and a linear load is applied to the surface of the UHPC overlay at mid-span, as seen in Figure 8.5. The load was controlled by displacement with the rate of 1 mm/min. During the test, three vertical linear variable differential transformers (LVDT) are arranged at the supporter section and the mid-span to measure the deflection, and two horizontal LVDTs are arranged at the two ends of the overlay to measure the interface slip. The cracking patterns were also recorded at an interval of 50 kN and the ultimate stage.

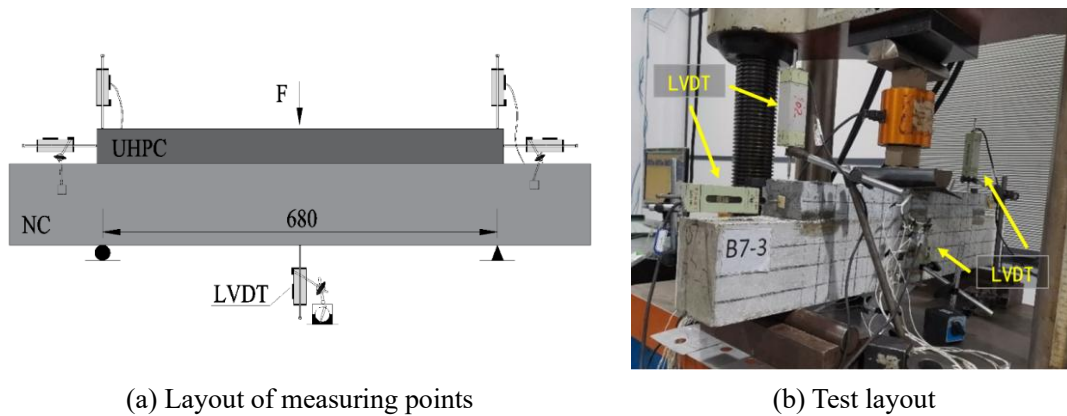


Figure 8.5 Three-point bending test arrangement of RC-UHPC composite beam.

8.3 Bending performance of RC-UHPC composite beams

8.3.1 Test results and numerical simulation verification

8.3.1.1 Load-deflection relationships and Failure modes

The three-point bending test was carried out on nine groups of composite girders, divided into three series as B3, B5, and B7 based on the interface treatment. In addition, corresponding numerical simulations were conducted. Figure 8.6 displays the load-deflection relationships obtained by both the experimental and the numerical work. The curves of the girders with different interface types share a similar tendency that a curve experiences three stages: (1) the linear-elastic stage, (2) the cracking stage, and (3) the failure stage. In the linear-elastic stage, the stress is below the elastic limit of the material, and the load-deflection curve grows approximately linearly. The slope of the curve is determined by the elastic stiffness. At this stage, the numerical results are in good consistence with the tested results. As the initial cracking generates at around 50 kN, the linear-elastic stage ends and the cracking stage starts, and the cracking loads of the three types of specimens are basically equal. The slope is lower than that of the linear-elastic stage since the development of cracks. When approaching the yielding point at a load slightly lower than 200 kN, the significant reduction of the slope marks the beginning of the failure stage. The collaboration of the UHPC pavement and the RC girder improves the ductility of the composite specimen. At this stage, obvious plastic deformation could be observed that the deflection increases by a large margin with the fluctuating or slightly increasing loading value. At the ultimate failure, the sudden interface separation at one end or flexural-shear failure occurs while accompanied by a sharp decline of the loading capacity. The numerical simulation can predict the tendency of the load-deflection relationship in the last two stages well. The corresponding numerical bearing capacity of the RC-

UHPC composite beam is 200.79 kN, 201.44 kN, and 204.70 kN. The average bearing capacity of the composite beam measured in the test is 199.62 kN, 206.51 kN, and 196.41 kN respectively, with the deviation of 5 kN, 5 kN, and 9 kN. The test results involve the numerical simulation results, and the errors between the test results and the corresponding numerical simulation results are 0.59%, 2.45%, and 4.22% respectively. The maximum error is less than 5%, which verifies the feasibility of the research method in this paper.

The typical cracking patterns corresponding to two failure modes are provided in Figure 8.6, one with interface separation, and the other without interface separation. On the side surface of the damaged tested girders, main diagonal cracks penetrated from near the upper loading point to near the lower supports. Besides, a few vertical cracks developed in the RC part. In three B7 girders, two B5 girders, and one B3 girder, the interface separation occurs at one end of the girder, as seen in Figure 8.6 (b) and (c). For the other tested girders, there is no interface separation, while the width of the main diagonal cracks is greater.

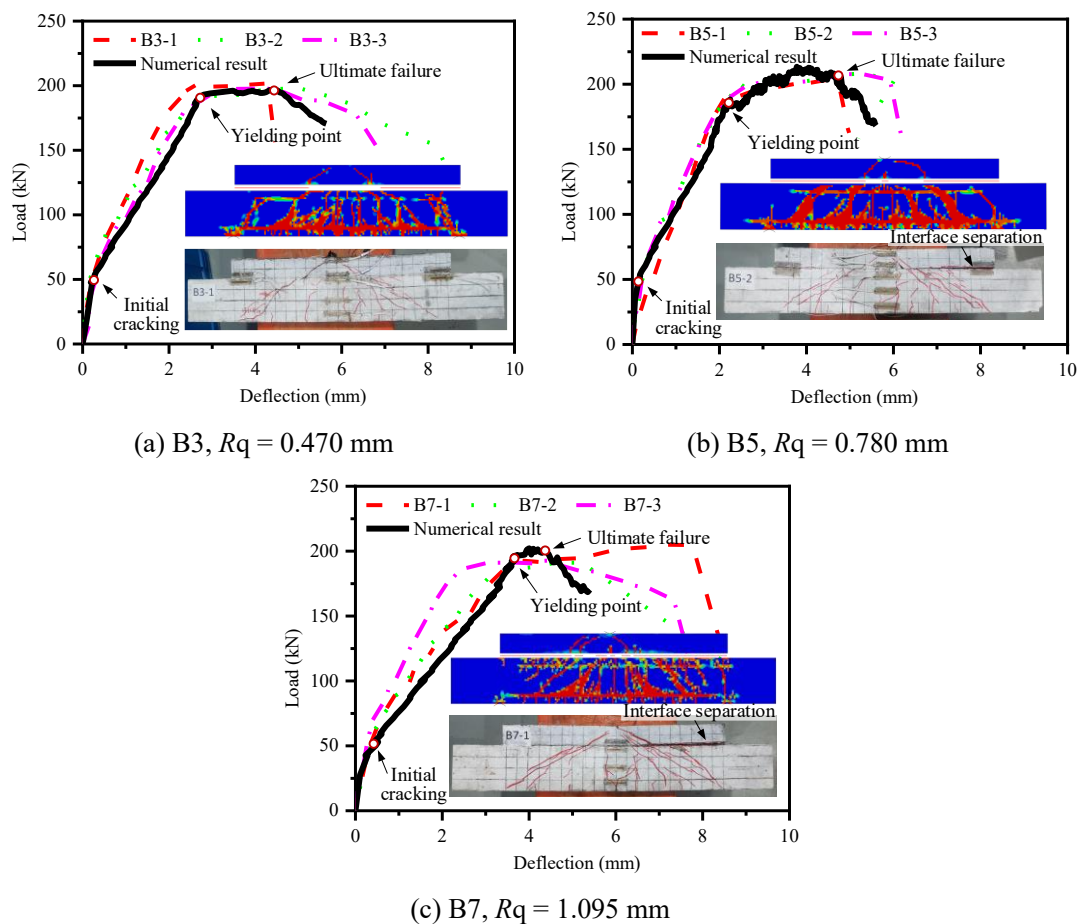


Figure 8.6 Load-deflection curve comparison between the test and numerical results.

The corresponding numerical results are also shown in Figure 8.6. The reason for the different

dominating failure modes lies in the various interface roughness. The stiffness of the RC part and the UHPC overlay differs, resulting in different stress states at the two sides of the interface. The higher roughness means the lower flatness of the interface and the higher stress concentration in the uneven interface, further enlarging the differences in stress. Thus, the interface with higher roughness is prone to instantaneous brittle failure. The damage distribution obtained by the numerical simulation is consistent with the experimental results that obvious oblique cracks accompanied by some vertical cracks as well as obvious debonding at the interface are predicted. The established numerical model can accurately predict the three stages and characteristics of the load-deflection response and the damage mode, and the comparison of the failure process effectively verifies the feasibility of the numerical analysis method.

8.3.1.2 Failure process

To further investigate the failure mechanism of the interfacial debonding behavior, the development of damage in the girder was analyzed by the numerical method, taking the B7 type girder as an example. In Figure 8.7, the tensile damage distribution at 25 %, 50 %, 75 %, and 100 % of the ultimate loading are shown.

At the loading of 25 % P_k , the girder works in the linear-elastic stage. Thus, only miserly local compression damage occurs near the support positions in the simulation, and no visible cracking has been detected in the tested girder. As the loading goes up to 50 % P_k , minor diagonal cracks appear in the RC part, and debonding occurs between the tension reinforcement and the surrounding concrete near the mid-span. Besides, the sub-fraction of the interfacial bonding elements starts to damage, leading to tensile damage at the bottom of the UHPC overlay. When the load increases to 75% P_k , the diagonal cracks continue to extend and develop, and vertical tensile cracks develop from the bottom of the RC part. The neutral axis moves up, and the tensile stress is mainly resisted by the reinforcement. With the increasing discontinuous damage in the interface bonding elements, the cooperation between the RC beam and the UHPC overlay reduces. When the load reaches 100 % P_k , the diagonal cracks penetrate through the UHPC and RC parts, and the vertical cracks fully extend. The interface bonding elements on both ends reach the ultimate shear stress and fail, indicating the interface separation and the ultimate failure of the girder.

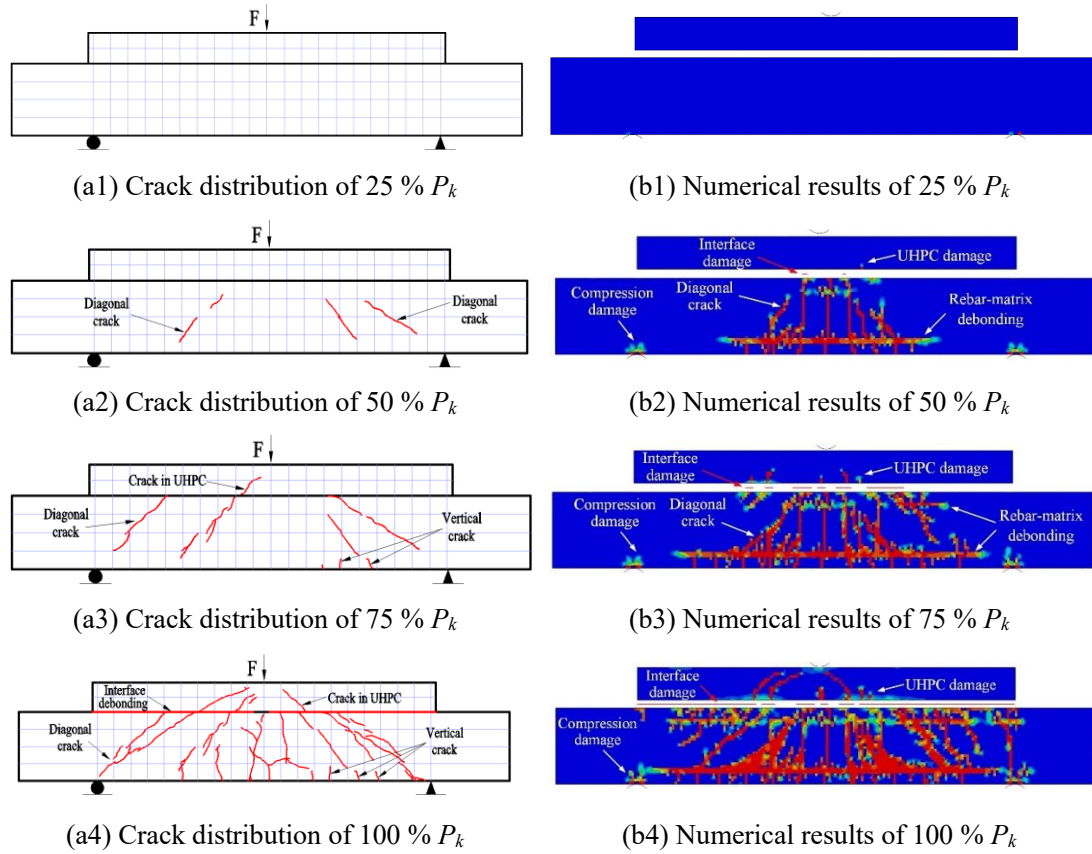


Figure 8.7 Crack distribution of RC-UHPC beams under different load levels.

8.3.1.3 Load-interface slip relationships

The load-interface slip relationship of three groups of specimens is shown in Figure 8.8. The load-slip curves can be divided into three stages: (1) bonding non-slip stage; (2) slip development stage; (3) interface separation stage. After starting the loading process and in the bonding non-slip stage, there is basically no slip at the end of the specimen. The shear resistance of the interface between RC beams and UHPC overlay is mainly contributed by the chemical bond between the layers and the substrate concrete and the interlocking action of the rough interface. This stage corresponds to the linear evolution stage in the load-slip curve (before the initial interface slip). When reaching the interface crack load, the interface slip increases and the interface shear stiffness begins to decline, and the load-slip curve enters the second stage while the significant slip begins to appear. The loading value corresponding to the average initial interface slip for B3, B5, and B7 beams is 75 % P_k , 86.7 % P_k , and 62.4 % P_k , separately. In this stage, the chemical bonding force at the interface disappears, and the interlocking effect and friction force begin to become significant. The development of horizontal cracks is restricted and the rapid development of slip is delayed. Compared with the load-deflection curve in Figure 8.8, when the load-deflection curve entered the

yielding stage, the bearing capacity still slightly increased. At this time, the interfacial slip is significantly increasing, and the vertical shear force decreases. A large number of fine cracks have been initiated at the interface. When the interface is cracked and enters the third stage, the gap between the rough interface that was originally in close contact also appears due to the opening of the interface. In this stage, although the interface shear resistance demand is reduced, the residual shear-bearing capacity of the interface is not enough to resist the further expansion and aggregation of the micro-cracks. The sliding of the interface develops rapidly until the horizontal cracks penetrate, resulting in a relatively steep decrease in the final load-slip curve. Eventually, the brittle failure of the interface occurred, and this is reflected in the sudden decrease in the load-deflection curve after exceeding the ultimate bearing capacity.

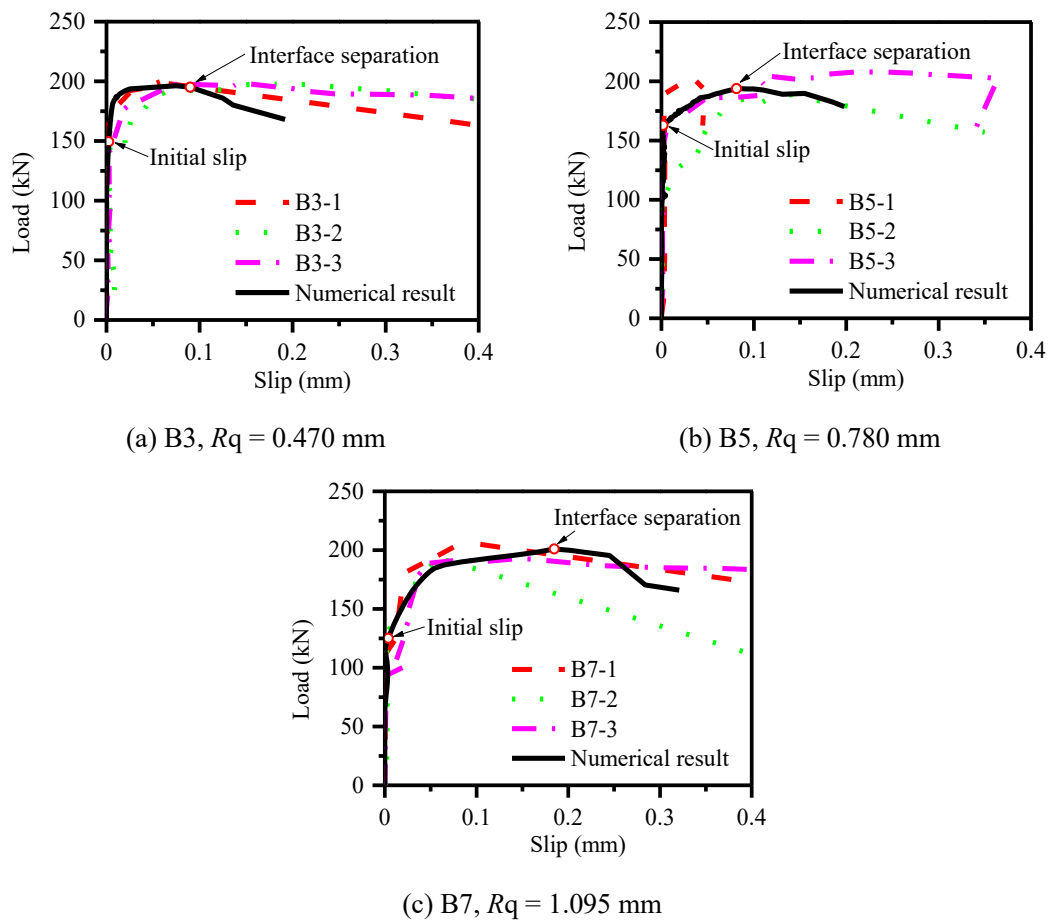


Figure 8.8 Comparison of load-interface slip curves between the test and numerical results.

During the whole loading process, the numerical simulation results of the interface slip development of the three groups of specimens are similar to the test results, and they still have sufficient interface shear resistance before the final bending failure, indicating that the shear resistance of the bonding interface of the RC-UHPC composite beam with rough and no shear stud

configuration is still excellent, which can ensure the collaborative stress between the prefabricated components and the post-cast UHPC overlay, and can fully exert the mechanical properties of the material.

8.3.2 Influence of interface morphological characteristics

Based on the multi-scale calculation method of interface homogenization in sub-section 3.3, the equivalent traction-separation curve of the NC-UHPC interface as well as the interface strength could be obtained. For the studied NC-UHPC interfaces B3, B5, and B7 with the interface roughness of $R_q = 0.470$ mm, 0.780 mm, and 1.095 mm, the traction-separation curves are displayed in Figure 8.9. The interface strength is defined as the maximum traction value, and the interfacial strength for the above-mentioned interface is 2.41 MPa, 2.78 MPa and 3.15 MPa, respectively. With the growing roughness, the equivalent strength of the interface increases, and the nonlinearity of the post-peak performance enhance as well. The reason lies in the varying range of interface damage with different roughness. With low roughness, an RVE in the interface region is flat with relatively low strength, thus the damage mainly occurs in the ITZ region when exposed to loading. The equivalent macroscopic performance of the interface region, especially the post-peak performance, is significantly influenced by the bilinear constitutive relationship of the interface RVE. With high interface roughness, the roughed RVE gains relatively high strength, and the shear failure at the interface region also involves damage in the NC and UHPC nearby. This causes the nonlinear behavior of the traction-separation curve of the interface after the peak load.

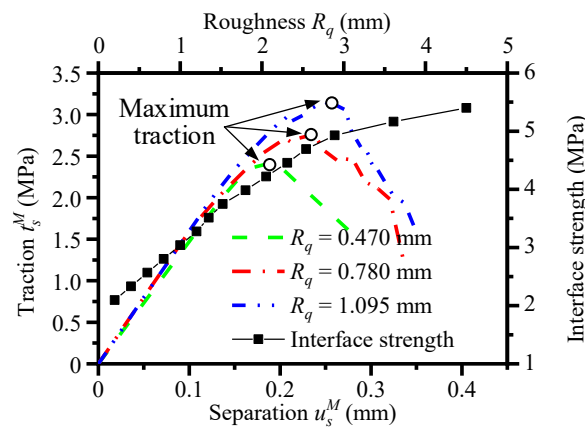


Figure 8.9 The relationship between the interface strength and the roughness.

To further investigate the influence of interface morphological characteristics on the interfacial strength, the interfacial bonding performance of 15 groups of interfaces with different roughness

was simulated by numerical method. The relationship between the interface strength and the roughness is shown in Figure 8.9. As the interface roughness R_q increases from 0.2 mm to 3.0 mm, the interface strength goes up from 2.10 MPa to 5.40 MPa. The curve climbs linearly when R_q is less than 2 mm, and the growing ratio gradually reduces when R_q exceeds 2 mm.

Then the simulated interfacial performance was assigned to the RC-UHPC girder model, and the mechanical behavior was studied. Figure 8.10 shows the relationship between the peak load resisted by the composite girder and the interface strength. It indicates that with the increasing interface strength, the peak load applied to the composite girder raises, and the increasing ratio gradually increases. The relation curve could be fitted by an exponential function with the correlation coefficient reaching 0.99, describing the positive effect of the interface strength on the girder capacity. Also, the relationship between the peak load resisted by the composite girder and the interface roughness is demonstrated in Figure 8.11. The relationship between the calculated peak load and the interface roughness does not show an exponential relationship similar to that between the peak load and the interface strength (Figure 8.10). However, the positive effect of the roughness on the girder capacity could be described by an approximately linear relationship with the correlation coefficient of 0.96.

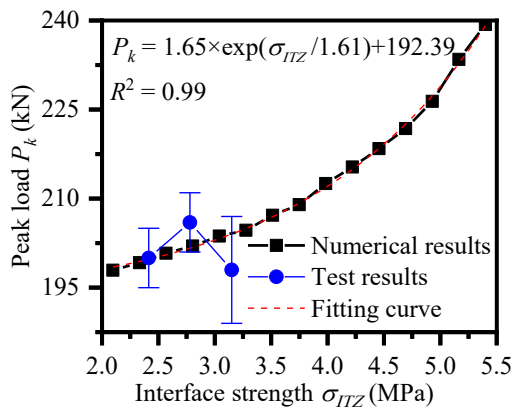


Figure 8.10 Relationship between the peak load P_k and the interface strength σ_{IIZ} .

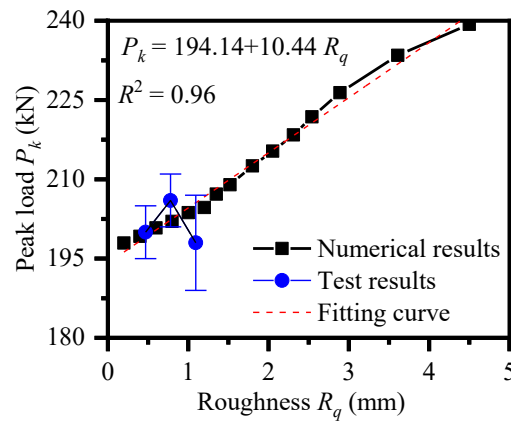


Figure 8.11 Relationship between the peak load P_k and the interface roughness R_q .

8.3.3 Evolution of stress states in the interface region

The evolution of shear stress states in the NC-UHPC interface of a composite girder was investigated by comparing the shear stress distributions at various loading stages. The shear stress along the horizontal direction was extracted for the composite girders with the interface of B3, B5, and B7 at the loading values of 25 %, 50 %, 75 %, and 100 % P_k .

As seen in Figure 8.12, the distribution of shear stress in the interface is similar for the tested girders with different interface roughness and different loading levels. The maximum shear stress appears in the interface near the mid-span and gradually decreases towards the support to zero. The shear stress level of the interface between the RC beam and UHPC overlay increases with the increase of the load level. From initial loading to 75 % P_k , cracks and damage developed and accumulated in the RC part, causing changes in local stress, and this also contributes to the fluctuation of the curve. When approaching the peak load, the concrete has severely cracked, and the load is mainly borne by the rebar. Thus, the fluctuation amplitude of the curve reduces. At the ultimate load, the maximum shear stress along the interface is 2.19 MPa, 2.70 MPa, and 3.02 MPa for B3, B5, and B7, respectively, reaching 91 %, 96 %, and 97 % of the calculated strength, indicating that shear stress dominates the interface failure. The evolution of the shear stress state in the interface corresponds to the damage evolution of the interface bonding elements in Figure 8.6. The damage first appears near the mid-span region with the higher shear stress. As the load increases, the damage range gradually extends toward the ends of the girder.

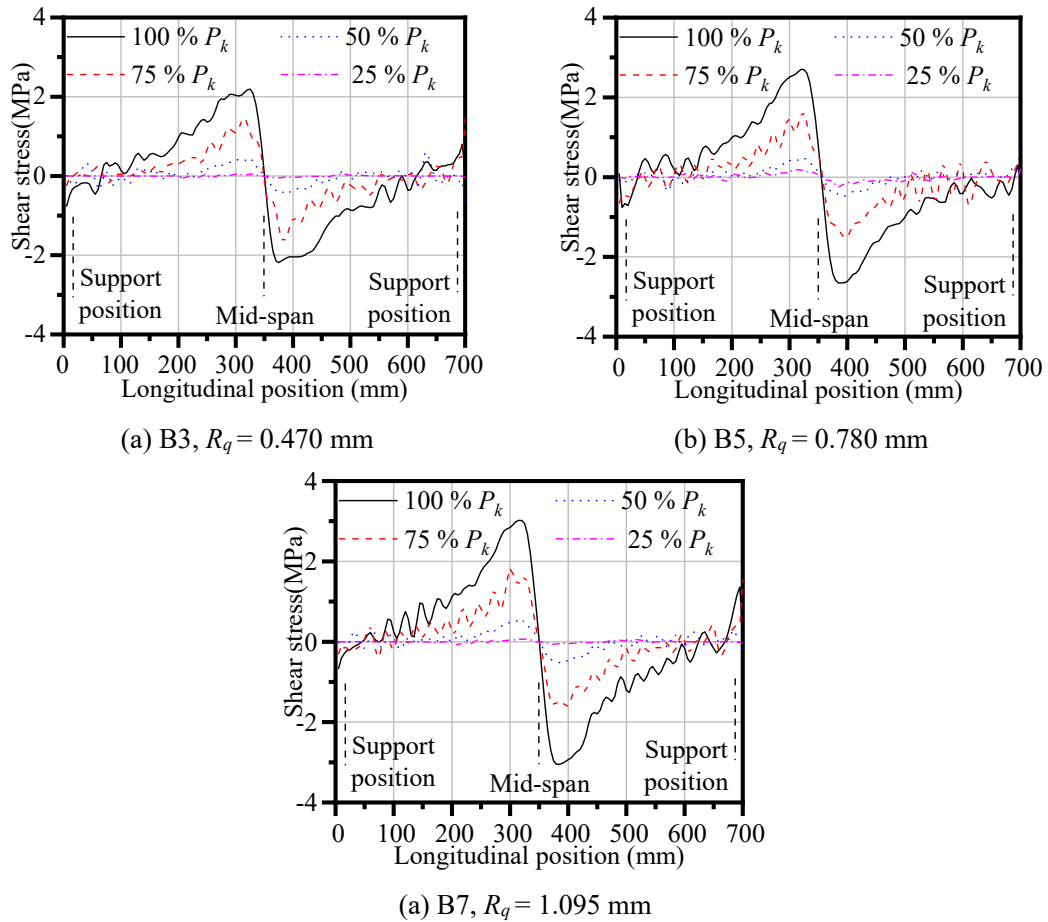


Figure 8.12 Evolution of interfacial shear stress.

The comparison of interface shear stress distribution for 15 series of interface roughness under ultimate load is shown in Figure 8.13. It indicates that with the increase of the interface roughness, the maximum value of the interface shear stress near the mid-span increases continuously, and the distribution of shear stress is consistent. This is because after the increase of the roughness, the corresponding mesoscopic equivalent RVE shear strength of the interface increases, and the peak shear strength of the interface increases continuously. Figure 8.14 shows the relationship between the maximum shear stress of the interface and the corresponding roughness of the RC-UHPC composite beam under the peak bearing capacity. It can be seen from the figure that the maximum shear stress at the interface varies linearly with the roughness. When the interface roughness increases from 0.2 mm to 3.0 mm, the maximum shear stress at the interface increases from 2.04 MPa to 5.20 MPa, which increases 2.55 times.

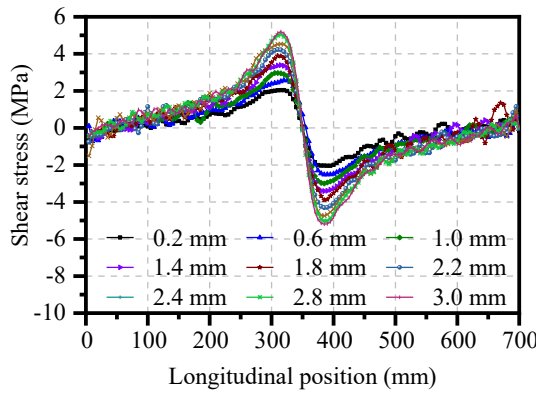


Figure 8.13 Shear stress distribution in the NC-UHPC Interface with different interface roughness.

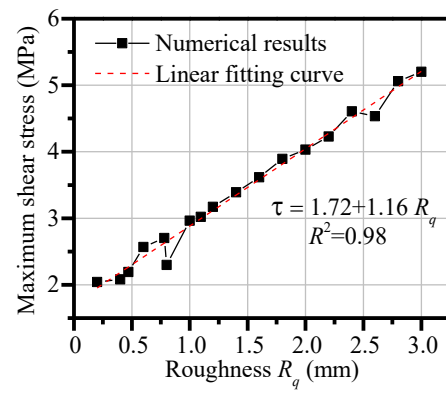


Figure 8.14 Relationship between the maximum shear stress and the interface roughness.

8.3.4 Evolution of damage in the interface

To reveal the evolution of the interfacial damage, the statistical results of the damage ratio of interface elements under different load levels and interface roughness are shown in Figure 8.15. The load level includes 70 %, 80 %, 90 %, and 100 % P_k . The interface damage factor is defined as the degree of modulus degradation of the interface element. The statistical interval of interface element damage is 0~0.25, 0.25~0.50, 0.50~0.75, and 0.75~1.0 respectively. The damage factor ranging from 0 to 0.25 indicates a low level of damage, while the damage factor ranging from 0.75 to 1.0 indicates severe damage.

As the load increases, the proportion of severely damaged elements gradually increases. At 70% P_k , the damage degree of the interface element is relatively low, and the element damage factor is

mainly within 0~0.25. Except for the three groups of interfaces with roughness less than 0.47, which have interface elements with damage factors exceeding 0.5, the damage indices of other groups are all below 0.5. When the load increases from 70% P_k to 100% P_k , the proportion of interface elements with severe damage significantly increases. Taking the 0.2 mm roughness interface specimen as an example, the proportion of elements with a damage factor above 0.75 increased from 20.2% to 68.2%, while the proportion of elements with a damage factor less than 0.25 decreased from 51.4% to 0.

The distribution of the interfacial damage factor varies with different roughness. Generally, when the interface roughness is low, the interface elements are prone to gain higher damage factor values, and the proportion of severely damaged elements goes down with higher interface roughness. The distribution of the interface damage factor for specimens with the interface roughness of 0.2 mm and 3.0 mm are compared. At 70 %, 80 %, 90 %, and 100 % P_k , the proportion of elements with a damage factor above 0.75 of the 3.0 mm roughness specimen reduces by 20.3 %, 35.4 %, 50.3%, and 47.0%, respectively; while the proportion of elements with a damage factor below 0.25 increased by 48.6%, 75.9%, 90.7%, and 73.9%, respectively. The distribution of the interfacial damage factor varies with different roughness. The increase in interface roughness can improve the overall cooperative stress performance of RC-UHPC composite beams, thus reducing the risk of interface cracking.

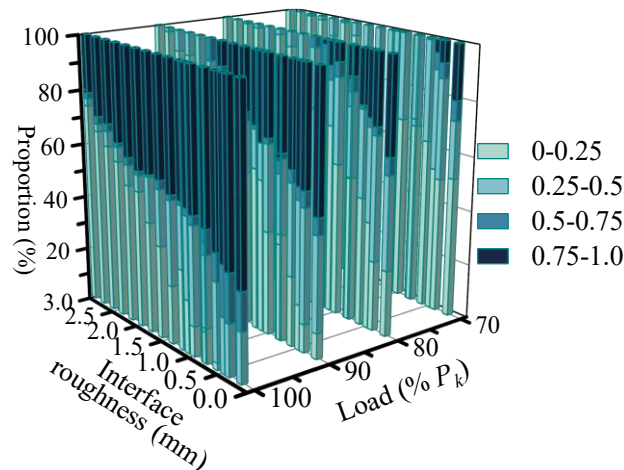


Figure 8.15 Statistical results of interface element damage.

8.4 Conclusions

In this chapter, the three-point bending test was conducted on UHPC-RC composite girders, and a multi-scale numerical simulation framework was proposed to reveal the influence of the

interface morphology on the meso-scale to its bonding behavior on the macro-scale. The following conclusions can be drawn:

(1) A multi-scale analysis method for the performance of UHPC-RC composite beams is proposed. This method can take into account the influence of material micro components on its macroscopic properties as well as the influence of interface micro morphology on the interface bonding performance.

(2) Horizontal shear stress controls the failure of the NC-UHPC interface. The maximum shear stress appears near the mid-span and gradually decreases towards the end of the girder. The distribution of shear stress is fluctuated due to the changes in local stress distribution caused by concrete damage and cracking.

(3) The interface roughness has a positive effect on the interface shear strength. When the roughness increases from 0.2 mm to 4.5 mm, the interface strength increases by 1.2 times. Therefore, interface roughness also has a significant positive effect on the overall composite beam. The bearing capacity of a composite girder is exponentially positively correlated with the interface shear bond strength and linearly positively correlated with the interface roughness.

(4) The greater the interface roughness, the greater the maximum shear stress at the interface near the mid-span. The maximum shear stress at the interface varies linearly with the roughness. When the interface roughness increases from 0.2 mm to 3.0 mm, the maximum shear stress at the interface increases from 2.04 MPa to 5.20 MPa, which is 2.55 times higher.

(5) The roughness of the interface is beneficial for reducing the degree of damage within the interface elements, thereby improving the collaboration between the UHPC overlay and the RC part and reducing the risk of interface separation.

Chapter 9 Conclusions and Suggestions for Future Research

9.1 Conclusion

UHPC is a typical composite material with multiple scale characteristics and complex microstructure. The extremely high strength, excellent resistance, and wide adjustable performance range make it increasingly widely used in the reinforcement of new bridge structures and existing structures. Based on the summary and analysis of previous research on the static mechanical properties and multi-scale calculation methods of ultra-high performance fiber-reinforced concrete materials and their interface degradation behavior, combined with composite material micromechanics analysis methods and progressive damage failure theory, relevant representative volume elements were established to establish a connection between the macroscopic properties and microscopic structural response of UHPC. Based on the multi-scale mean field homogenization method, the equivalent performance containing microstructural information can be obtained, which enables more accurate analysis of the performance evolution mechanism of ultra-high performance concrete materials and structures during service life. The main research achievements and innovative points of the thesis are as follows:

1. Based on the meso-scale random modeling method of concrete, a numerical model containing the mesoscopic phases of UHPC was established, and the compressive performance and damage evolution law of UHPC coupled with plastic damage model were analyzed. the differences in damage changes of aggregates, mortar, and interface zone during the compression process of UHPC with different fiber and coarse aggregate contents were revealed. At the microscale, the stress changes of fibers at different distribution angles were calculated, and the intrinsic relationship between fiber distribution angles and toughening and crack resistance mechanisms was revealed. The results show that as the fiber content increases (0.5% -2.5%), the compressive strength of UHPC increases. When the content of coarse aggregate increases (0% -30%), the compressive strength first increases and then decreases. In the final state, the damage proportion of mortar continuously decreases, the damage proportion of ITZ increases, and the damage proportion of aggregate first increases and then decreases with the increase of fiber content. The results of fiber stress analysis indicate that horizontally distributed fibers have more significant toughening and crack resistance functions before the formation of macroscopic shear planes during compression loading. After the

formation of macroscopic shear cracks, almost all fibers in UHPC participate in the stress.

2. Compared with the framework of micro-scale numerical simulation method, A multi-scale analysis method for ultra-high performance concrete composite materials was proposed based on the Mori-Tanaka homogenization method and progressive damage theory. At the macro scale, the micro equivalent properties of ultra-high performance concrete materials are input into the finite element model corresponding to numerical simulation, and the micro material properties are coupled with the macro finite element model. By comparing with experimental results, the effectiveness of the method was verified, and the macroscopic mechanical properties of ultra-high performance concrete were predicted under different fiber and coarse aggregate contents. The research results show that the maximum errors between the average peak stress of compression, tension, and four point bending tests and the corresponding numerical simulation results are 2.7%, 5.7%, and 2.2%, respectively. The four point bending test results of UHPC board show that the total toughness of 5% fiber volume fraction is 7.55% to 8.66% higher than that of 0%, and the post peak toughness is 12.41% to 16.97% higher than that of 0%. Compared with coarse aggregates, the increase of fibers has a more significant enhancing and toughening effect on the bending toughness and post cracking ductility of ultra-high performance concrete, which can effectively improve its mechanical properties after cracking.

3. A multi-scale tensile performance analysis method for UHPC considering fiber orientation distribution has been developed to improve its tensile performance. Firstly, the tensor of fiber orientation was used to describe, analyze, and extract the characteristic parameters of fiber distribution in the cross-section of the tensile specimen. The non-uniform fiber direction distribution function was reconstructed through a mixed closure approximation. The RVE after fiber reconstruction was established, and the elastic properties of ultra-high performance concrete considering fiber orientation were calculated using a two-step mean field homogenization method. By combining the Puck failure criterion, the tensile strength of UHPC with different fiber contents and orientation tensors was successfully predicted. With the increase of fiber content, the tensile stress-strain curve of UHPC changes from strain softening to strain hardening. The fitting results of fiber orientation distribution indicate that an increase in fiber quantity will affect the distribution of fibers and weaken the crack resistance and toughening effect. The Young's modulus E_{22} and shear modulus G_{12} increase as the orientation tensor a_{11} decreases. The change in direction of enhancement

causes the tensile modulus E_{11} to decrease first and then increase, while the shear modulus G_{12} increases first and then decreases.

4. A multi-scale analysis method for the debonding behavior of the interface between the normal concrete and UHPC has been developed to address the reinforcement application issues of repairing existing concrete structures with UHPC. Considering the influence of complex interface geometry on bonding failure behavior at the microscale, the corresponding RVE was established for six different interface roughness parameters. Through numerical homogenization calculations, the load transfer and debonding failure behavior at the interface were studied, and equivalent traction separation curves under tensile and shear forces were obtained. When the surface roughness increases from 0.41 mm to 1.31 mm, the RVE tensile traction force corresponding to C40 and C50 base concrete increases by 100% and 64%, respectively, and the shear traction force increases by 202% and 121%, respectively. For a micro RVE with a roughness of 0.47 mm, when the interfacial strength increases from 2 MPa to 7 MPa, the effective tensile traction force increases by 204% and the effective shear traction force increases by 114%. The post peak nonlinearity of the equivalent traction separation curve significantly increases with the increase of roughness.

5. The multi-scale analysis method of material and interface bonding behavior has been applied to the bending performance analysis of reinforced UHPC composite beams. The complex interface geometry at the mesoscale was described through a homogenization process, and after obtaining the effective mechanical responses, the macroscopic flat interface elements are assigned. Through experimental comparative research, the three-point bending failure modes, load deflection, and load interface slip curves of RC-UHPC composite beams under different interface roughness were verified and predicted. The research results found that the interface microstructural characteristics have a significant impact on the overall response of RC-UHPC composite components. The larger the roughness of the interface, the smaller the degree of interface damage under the same load level. The increase in interface roughness can enhance the overall collaborative stress performance of RC-UHPC composite beams, thereby reducing the risk of interface cracking.

9.2 Suggestions for Future Research

The application of multiscale theory in the material structure analysis of UHPC is a rich, promising, and challenging research topic. Combining with the research contribution of this thesis, further research and improvement can be carried out from the following aspects:

1. The multi-scale research on UHPC involved in this work belongs to the static or quasi-static mechanical problems, and further research is encouraged on the multi-scale performance under dynamic impact and complex environmental conditions.

2. For the research and application of multi-scale interface performance, it is necessary to conduct more complex performance evaluation and experimental research under stress, and the analytical models of relevant multiscale theories also need to be further improved.

3. Fiber orientation has a significant impact on the performance of UHPC. For the practical engineering applications, reasonable and efficient construction techniques should be proposed to control the fiber distribution more accurately.

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