

## Seismic design theory, numerical modeling and experimental investigation on collapse resistance mechanism of high-ductility fiber-reinforced concrete frame structures

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### Abstract

Traditional reinforced concrete frame structures suffer from insufficient seismic ductility and brittle continuous collapse under strong earthquakes, while existing research on high-ductility fiber-reinforced concrete (HDC) only optimizes single component performance, lacking integrated overall seismic design theory and quantitative anti-collapse mechanism evaluation method for frame systems. To fill this research gap, this paper takes HDC frame structures as the research object, combines micromechanical fiber bridging constitutive modeling, quasi-static cyclic loading tests and elastoplastic time-history finite element simulation. It systematically analyzes the synergistic mechanical performance of HDC beams, columns and integral frames. On this basis, a complete set of seismic design calculation formulas and three-level progressive anti-collapse judgment criteria considering fiber bridging effect and beam-column synergistic coefficient is established. Test results indicate that ordinary RC specimens show severe hysteresis pinching effect, with peak bearing capacity of 82.1 kN and rapid strength attenuation after 14 mm displacement. HDC specimens achieve full spindle-shaped hysteresis curves with significantly suppressed pinching; its peak load reaches 145.2 kN (76.9% higher than RC specimens), and stable load-carrying capacity is maintained at 28 mm displacement, accompanied by gentle stiffness degradation and greatly improved energy dissipation capacity. The proposed design system innovatively integrates fiber strain-hardening characteristics into "strong column-weak beam, strong shear-weak flexure" seismic design principles, which enriches the seismic resistance theoretical system of HDC structures and provides quantitative theoretical basis and engineering reference for anti-collapse design of frame buildings located in high-intensity seismic zones.

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## 1. Introduction

In earthquake disasters, concrete frame structures (FS) are prone to column hinge failure, node failure, and overall collapse due to insufficient ductility and poor energy dissipation capacity (EDC), resulting in huge casualties and property losses [1-2]. Under strong earthquakes and extreme loads, ordinary concrete (OC) materials exhibit significant brittleness and are prone to concentrated crack propagation. FSs are prone to severe column end damage and unreasonable distribution of Plastic hinge (PH), which can easily lead to local component failure and rapid development into overall continuous collapse, posing a serious threat to building safety and durability. High ductile fiber-reinforced concrete (HDC), as a new strain-hardening composite

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material, has an ultimate tensile strain (TS) of up to 3%~8%, which is 200~800 times that of traditional concrete. It has excellent crack control ability and energy dissipation (ED) performance. Applying it to FS can considerably advance the seismic ductility and collapse resistance of the structure, and become a research hotspot within the realm of seismic structures in recent years [3]. In this paper, high-intensity seismic zones refer to areas with seismic fortification intensity of 8 degrees (basic ground motion acceleration 0.20 g) and above, where rare earthquake ground motion acceleration reaches 0.50 g, corresponding to severe earthquake risk level specified in GB 50011-2010 seismic design code of buildings.

### **1.1 Related Works**

Recently, scholars globally have conducted extensive research on the properties and structural applications of HDC materials. In response to the insufficient research on the seismic vulnerability of reinforced concrete (RC) frames under the combined action of constraint models and fiber-reinforced concrete (FRC), Kumar et al. compared the two constraint models and evaluated the structural performance through pushover analysis. The results indicated that FRC considerably advanced ductility and collapse resistance, and the Campione model was superior, providing a basis for seismic design of high-ductility FRC frames [4]. In response to the insufficient seismic and deformation performance of concrete columns in earthquake prone areas, Srivastava et al. fabricated four glass fiber-reinforced polymer (GFRP) - steel hybrid RC columns and conducted comparative analysis through quasi-static cyclic loading tests. The results demonstrated that the strength of the mixed reinforcement column met the standard, and its ductility and EDC were better [5]. In response to the insufficient seismic performance (SP) of RC columns, Labdaoui et al. reinforced samples with different FRC thin sheaths and compared multiple seismic indicators through simulated seismic load tests. The results indicated that this method considerably advanced the bearing capacity (BC), ductility, and EDC [6]. In response to the problem that traditional methods could not evaluate the seismic degradation of RC bridge piers with corroded FRC protective layers, Wang et al. constructed a corrosion seismic coupling analysis framework and compared the two types of bridge piers using a pitting corrosion model and nonlinear time-history analysis. The results demonstrated that the FRC protective layer could efficiently advance durability and reduce the probability of damage, providing important support for the seismic design and anti-collapse mechanism survey of HDC structures [7].

In response to the problems of large residual deformation and insufficient stiffness of traditional RC columns after earthquakes, Liu et al. utilized GFRP and basalt fiber-reinforced polymer bars to strengthen concrete columns and conducted low cycle load tests. The results demonstrated that FRP reinforcement could considerably advance the post yield stiffness and reduce residual deformation [8]. To study the SP of steel fiber reinforced high-strength concrete shear walls, Zhang et al. conducted low cycle reciprocating loading tests to compare the stress performance of samples with different heights in different FRC regions. The results demonstrated that steel fiber reinforced high-strength concrete considerably advanced ductility and integrity, efficiently optimizing stress distribution [9]. In response to the issue of unclear effects of special boundary element methods and floor modeling in the Chilean double wall FS, Lopez Machado et al. utilized Perform3D three-dimensional nonlinear simulation to compare different parameters for seismic collapse analysis. The results indicated that the floor stiffness considerably affected the collapse probability, and the special boundary element could reduce the collapse risk, which provided an important reference for the seismic design and collapse resistance mechanism survey of HDC frames [10]. Ali et al. adopted two schemes, FRP local reinforcement and new reinforcement arrangement, to address the consequence of local damage and even collapse in the connection of primary and secondary beams in RC structures. The mechanical properties were compared through three-point bending tests. The outcomes considerably advanced the BC, stiffness, and ED, providing experimental evidence for the seismic design and collapse resistance mechanism survey of HDC frames [11].

To investigate the SP and repairability of high-ductility FRC frames, Xiao et al. proposed a steel RC frame equipped with replaceable energy dissipating members, and conducted low cycle repeated loading tests with fuses and quasi-static cyclic loading tests on two groups of members to compare the mechanical response before and after repair [12]. In response to the seismic problem of

insufficient stiffness in the soft layer formed by the large open bottom, Shabla et al. established a G+14-layer finite element model based on ETABS, conducted response spectrum analysis in different seismic zones, and compared the seismic effects of various soft layer control measures, which can provide reference for the anti-collapse design of FSs [13]. In response to the problem of neglecting the foundation shear force and insufficient calibration accuracy of pile foundation deformation in the existing large-diameter single pile lateral load design methods, Chen et al. quantified the contribution of foundation shear force by relying on finite element simulation of sandy soil sites, and constructed a four-parameter camel hump curve model to characterize the correlation characteristics between pile end shear force and deflection [14]. To clarify the influence mechanism of the structure and mechanical properties of high-performance fiber materials on the seismic and collapse resistance of structures, Li S et al. conducted microscopic characterization and multi fiber tensile tests on various types of carbon fibers, analyzed the differences in grain structure and graphitization degree, summarized the tensile evolution law, and provided material performance support for optimizing the seismic and collapse resistance of high-ductility FRC structures [15].

## **1.2 Research Objective and Highlights**

In summary, existing survey on HDC mainly focuses on optimizing the performance of individual components, lacking integration of the overall seismic design theory of FS. Moreover, the analysis of the anti-collapse mechanism of HDC frames is not deep enough, and the synergistic optimization of material properties, structural design, and anti-collapse performance has not been attained, making it hard to match the engineering design demands of FS in high-intensity earthquake zones [16]. Therefore, the survey focuses on the core issues of seismic design theory and anti-collapse mechanism of HDC FS, specifically focusing on the mechanical performance analysis, seismic design theory construction, anti-collapse mechanism analysis, and experimental verification of HDC frames. The seismic response law and anti-collapse evolution mechanism of HDC frames are clarified, and innovative HDC frame seismic design methods and judgment criteria considering fiber bridging effect and component synergy are proposed, providing theoretical support and technical guidance for the engineering application of HDC FS.

Compared with previous published works, most existing studies only verified the ductility improvement of HDC single beams or columns via independent tests, without constructing a unified design system linking micro fiber bridging behavior, component mechanical performance and integral frame anti-collapse response. Previous design codes ignore the strain-hardening characteristic of HDC and cannot quantitatively calculate the ED contribution of fiber multi-crack confinement effect, leading to large deviation between design prediction and actual structural seismic response. This paper distinguishes itself by three core innovations: (1) Deriving unified micromechanical formulas for fiber bridging stress, beam flexural capacity and column shear resistance; (2) Optimizing the classic “strong column weak beam” seismic design principle with HDC material reinforcement coefficient; (3) Proposing a three-stage quantitative anti-collapse evolution judgment criterion based on structural ED, which fills the blank of integrated seismic design theory for HDC frame systems.

The structure of the survey article is as follows: The first section elaborates on the survey background, significance, and prospects of HDC. The second section focuses on the seismic design theory and anti-collapse mechanism process of the HDC FS developed in this study, which is also the key and innovative point of this survey. The third section specifically elaborates on the analysis of experimental data outcomes grounded in the survey methods of the second part. The fourth section draws conclusions grounded in the experimental outcomes, elaborating on the shortcomings of this design and the directions that need to be further explored in the future.

## **2. Methods**

An HDC frame seismic design theory that considers the fiber bridging effect and the synergistic effect (SE) of components has been developed to attain the synergistic optimization of material performance and structural design. It reveals the three-level anti collapse mechanism and establishes anti collapse judgment criteria grounded in ED.

## 2.1 Mechanical Performance Analysis of HDC Framework Structure

HDC FSs show strong nonlinear mechanical response under seismic excitation. The overall SP of the frame system is jointly determined by three factors: intrinsic material performance of HDC, mechanical properties (MP) of single beam/column components, and global cooperative working capacity of the whole structure [17]. The survey focuses on the overall collaborative work of HDC framework. It introduces fiber bridging effect coefficient and component collaborative effect coefficient, conducts mechanical performance analysis, clarifies the calculation methods of stiffness, bears capacity, ductility and EDC of HDC framework, and offers a foundation for the subsequent construction of seismic design theory. The tensile stress-strain (SS) curves of HDC and traditional materials are displayed in Figure 1.

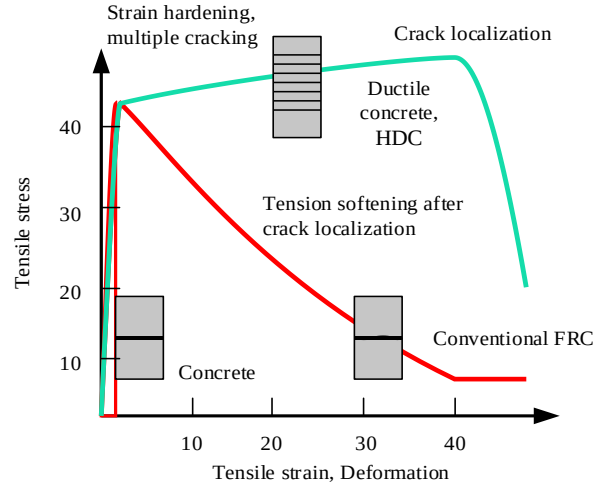


Fig. 1. Uniaxial tensile stress-strain curves of ordinary concrete (OC), reinforced concrete (RC) and high-ductility fiber-reinforced concrete (HDC); horizontal axis: tensile strain  $\varepsilon$  (%), vertical axis: tensile stress  $\sigma$  (MPa)

As detailed in Figure 1, OC undergoes stress softening immediately after cracking, exhibiting typical brittle failure. Traditional FRC only slightly delays stress attenuation after cracking, and ultimately softens due to localized cracking. HDC exhibits significant strain-hardening and multiple crack propagation characteristics after cracking, which can maintain and enhance tensile stress during continuous deformation until the ultimate state where crack localization and stress reduction occur. Its ultimate TS can reach hundreds of times that of OC [18-19].

The core advantage of HDC materials lies in the fiber bridging effect, which can efficiently suppress crack propagation and attain strain-hardening characteristics. Therefore, the fiber bridging stress calculation model is first derived [20]. Assuming uniform distribution of fibers in HDC material and good bonding between fibers and matrix interface, ignoring the nonlinear effects of fiber buckling and fracture, grounded in micro mechanical equilibrium conditions, the calculation of fiber bridging stress  $\sigma_{fb}$  is derived as detailed in equation (1).

$$\sigma_{fb} = \frac{V_f \cdot E_f \cdot \delta}{l_f/2} \quad (1)$$

In equation (1),  $V_f$  is the fiber volume fraction (FVF) (%).  $E_f$  is the fiber elastic modulus (MPa).  $\delta$  is the related slip between the fiber and the matrix (mm).  $l_f$  is the fiber length (mm).

Based on the fiber bridge stress calculation model, combined with the strain-hardening characteristics of HDC materials, the uniaxial tensile constitutive relationship (TCR) of HDC materials is obtained. The fiber bridge stress calculation model and the uniaxial TCR of HDC material is displayed in Figure 2.

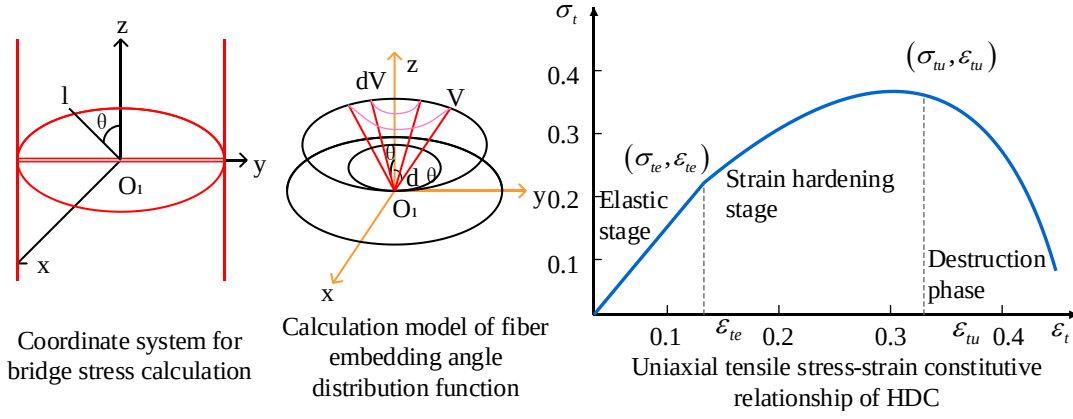


Fig. 2. Fiber bridge stress calculation model and uniaxial TCR of HDC materials

As detailed in Figure 2, assuming that the tensile process of HDC material is divided into elastic stage, strain-hardening stage, and failure phase. The SS constitutive relationship in the elastic stage follows Hooke's law, the strengthening effect of fiber bridging effect is considered in the strain-hardening stage, and the effect of fiber fracture and interface debonding is considered in the failure phase. The SS constitutive relationship in uniaxial tensile of HDC material is displayed in equation (2).

$$\sigma_t = \begin{cases} E_{hdc} \cdot \varepsilon_t & (0 \leq \varepsilon_t \leq \varepsilon_{te}) \\ \sigma_{te} + \sigma_{fb} \cdot (1 - e^{-\alpha \cdot (\varepsilon_t - \varepsilon_{te})}) & (\varepsilon_{te} < \varepsilon_t \leq \varepsilon_{tu}) \\ \sigma_{tu} \cdot e^{-\beta \cdot (\varepsilon_t - \varepsilon_{tu})} & (\varepsilon_t > \varepsilon_{tu}) \end{cases} \quad (2)$$

In equation (2),  $\sigma_t$  represents the tensile stress of HDC material (MPa).  $E_{hdc}$  denotes the elastic modulus of HDC material (MPa).  $\varepsilon_t$  is the TS.  $\varepsilon_{te}$  is the ultimate strain in the elastic stage.  $\sigma_{te}$  stands for the ultimate stress in the elastic stage (MPa).  $\alpha$  is the strain-hardening coefficient.  $\varepsilon_{tu}$  indicates the ultimate TS.  $\sigma_{tu}$  represents the ultimate tensile stress (MPa). and  $\beta$  is the attenuation coefficient in the failure phase.

As the core horizontal load-bearing component of the FS, the bending capacity and ductility of HDC beams directly affect the overall SP of the structure. The calculation of the bending capacity for the cross-section of HDC beams is displayed in equation (3).

$$M_u = \alpha_1 \cdot f_{hdc} \cdot b \cdot x \cdot \left(h_0 - \frac{x}{2}\right) + \sum \sigma_{fb} \cdot A_{fb} \cdot (h_0 - a'_s) \quad (3)$$

In equation (3),  $M_u$  represents the flexural capacity of the HDC beam's cross-section (kN·m).  $\alpha_1$  is the equivalent rectangular stress coefficient of the compression zone concrete.  $f_{hdc}$  denotes the axial compressive strength of the HDC material (MPa).  $b$  is the beam cross-section (BCS) width (mm).  $x$  is the height of the compression zone (mm).  $h_0$  is the effective height of the BCS (mm).  $A_{fb}$  is the effective area of the tensile zone fibers (mm<sup>2</sup>).  $a'_s$  is the distance from the resultant point of longitudinal reinforcement (LR) in the compression zone to the edge of the compression face (mm) [21].

The ductility coefficient (DC)  $\mu$  is defined as the ratio of ultimate deformation to yield deformation, taking into account the ability of HDC materials to develop multiple cracks and the enhancing effect of fiber bridging on deformation. The calculation of the DC is displayed in equation (4).

$$\mu = \frac{\Delta_u}{\Delta_y} = \mu_0 \cdot \left(1 + k_1 \cdot V_f + k_2 \cdot \frac{\sigma_{fb}}{f_{hdc}}\right) \quad (4)$$

In equation (4),  $\Delta_u$  is the ultimate deformation of the HDC beam (mm).  $\Delta_y$  is the yield deformation (mm).  $\mu_0$  are the DC of OC beams.  $k_1$  is the coefficient of the effect of FVF.  $k_2$  is the coefficient of affect of fiber bridging effect.

As the vertical load-bearing core of the FS, the shear bearing capacity (SBC) and stability of HDC columns directly affect the collapse resistance of the structure. The calculation of the SBC of HDC columns is displayed in equation (5).

$$V_u = V_c + V_f + V_s = 0.7 \cdot f_{thdc} \cdot b \cdot h_0 + 0.8 \cdot \sigma_{fb} \cdot b \cdot h_0 + 0.8 \cdot \frac{A_{sv} \cdot f_{yv} \cdot h_0}{s} \quad (5)$$

In equation (5),  $V_u$  is the SBC of the HDC column (kN).  $V_c$  is the shear contribution of HDC material (kN).  $V_f$  is the contribution of fiber bridging shear resistance (kN).  $V_s$  is the shear contribution of the hoop reinforcement (kN).  $f_{thdc}$  is the axial tensile strength (TiS) of HDC material (MPa).  $A_{sv}$  is the total cross-sectional area of the hoop reinforcement within the same section (mm<sup>2</sup>).  $f_{yv}$  is the design value (DV) of the TiS of the hoop reinforcement (MPa).  $s$  is the spacing between stirrups (mm).

Assuming the HDC framework is a rigid frame, ignoring the effect of node rotation, and considering the stiffness contribution and SE of beam and column components, the horizontal lateral stiffness (LS)  $K_{frame}$  of the framework is displayed in equation (6).

$$K_{frame} = \sum_{i=1}^n K_{ci} + \sum_{j=1}^m K_{bj} \cdot \eta \quad (6)$$

In equation (6),  $K_{ci}$  is the LS of the  $i$ th HDC column (kN/mm).  $K_{bj}$  is the bending stiffness of the  $j$ th HDC beam (kN · mm/rad).  $\eta$  is the coefficient of SE between beams and columns.  $n$  is the number of frame columns.  $m$  is the number of frame beams.

Under earthquake action, the EDC of HDC FS is the core indicator to measure its SP. The total ED  $E_{total}$  of HDC FS is calculated as detailed in equation (7).

$$E_{total} = E_{hdc} + E_{fb} + E_{plastic} = \int_0^{\varepsilon_{max}} \sigma_t \cdot d\varepsilon \cdot V + \int_0^{\delta_{max}} \sigma_{fb} \cdot d\delta \cdot A_{fb} + \sum M_p \cdot \theta_p \quad (7)$$

In equation (7),  $E_{hdc}$  is the strain ED of HDC material (kJ).  $E_{fb}$  is the energy consumption of fiber bridging (kJ).  $E_{plastic}$  is the ED of plastic deformation of the component (kJ).  $\varepsilon_{max}$  is the max TS of HDC material.  $V$  is the volume of HDC material (mm<sup>3</sup>).  $\delta_{max}$  is the max relative slip of the fiber (mm).  $M_p$  is the plastic bending moment of the component (kN · m).  $\theta_p$  is the plastic rotation angle of the component (rad) [22].

In summary, a complete analysis system for the MP of HDC FS are constructed, and the affect laws of fiber bridging effect and component synergy on the MP of the frame are clarified. The complete analysis system for the MP of the HDC framework structure is displayed in Figure 3.

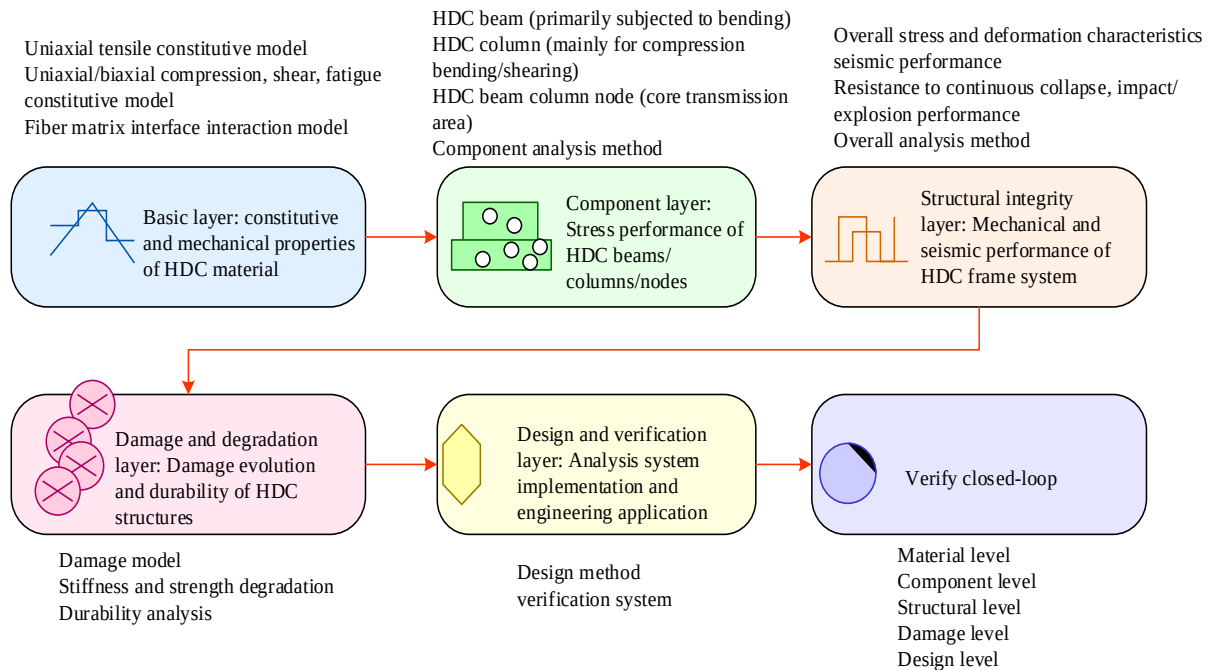


Fig. 3. A complete analysis system for the MP of HDC framework structures

As detailed in Figure 3, the system consists of five modules: Material constitutive, component performance, overall structure, damage degradation, and design verification. It forms a closed-loop analysis logic from micro fiber action to macro structural seismic and collapse resistance, covering the entire process of theoretical modeling, numerical simulation, experimental verification, and engineering design.

## 2.2 Seismic Design Theory and Collapse Resistance Mechanism of FS

Based on the above micro mechanical constitutive analysis of HDC material, the next section will extend single-component performance to integral frame seismic design theory. Based on the analysis of the MP of HDC FSs, the effect of HDC material characteristics and component MP on the overall response of the frame has been clarified. However, existing seismic design methods have not fully considered the strain-hardening characteristics and fiber bridging effect of HDC, resulting in deviations between design outcomes and actual engineering requirements, and a lack of analysis and judgment methods for the anti-collapse mechanism of HDC frames [23]. Grounded in this, the study combines mechanical performance formulas to optimize the design principles of "strong columns, weak beams, strong shear, and weak bending". It introduces HDC material reinforcement coefficients, proposes seismic design of HDC frames, and focuses on solving the three core issues of BC design, ductility design, and collapse resistance design of HDC frames. Meanwhile, the collapse resistance mechanism is analyzed, and a quantitative judgment method is established. The seismic design of HDC frame is displayed in Figure 4.

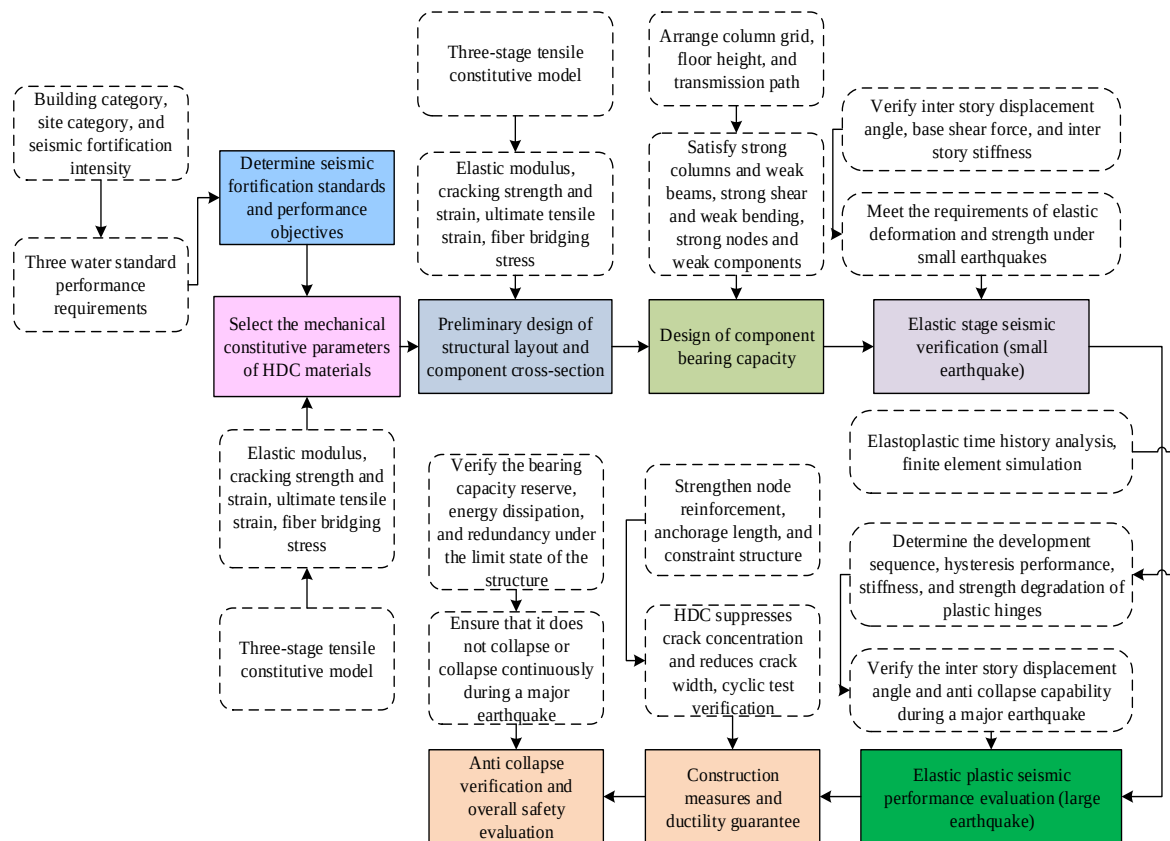


Fig. 4. Closed-loop seismic design workflow of HDC FS, including material constitutive calculation, component BC design, finite element simulation, cyclic test verification and anti-collapse check modules

As detailed in Figure 4, for the BC design of HDC frame, based on the calculation method of beam and column component BC, merged with seismic action combination, the BC design of HDC frame components is proposed. Considering the uncertainty of earthquake action and the performance advantages of HDC materials, the seismic action sub coefficient  $\gamma_{RE}$  and the HDC material reinforcement coefficient  $\lambda_{\text{HDC}}$  are introduced to derive the bending BC design of HDC beam normal section as detailed in equation (8).

$$\gamma_{RE} \cdot M_d \leq \lambda_{hdc} \cdot M_u \quad (8)$$

In equation (8),  $\gamma_{RE}$  is the seismic adjustment coefficient for BC.  $M_d$  is the DV of the bending moment of the beam section under earthquake action combination (kN · m).  $\lambda_{hdc}$  is the reinforcement coefficient of HDC material.  $M_u$  is the flexural BC of the HDC BCS (kN · m).

Similarly, the compressive BC design of HDC columns is displayed in equation (9), considering the combination of vertical load and seismic action of the column to ensure that the BC of the column meets the design requirements.

$$\gamma_{RE} \cdot (N_d + \gamma_{Eh} \cdot N_{Eh}) \leq \lambda_{hdc} \cdot N_u \quad (9)$$

In equation (9),  $N_d$  is the vertical load DV (kN).  $\gamma_{Eh}$  is the partial coefficient of horizontal seismic action.  $N_{Eh}$  is the axial force of the column generated by horizontal seismic action (kN).  $N_u$  is the compressive BC of the HDC column cross-section (kN) [24].

Grounded in the calculation outcomes of beam DC and merged with the DC required by seismic design, the ductility design of HDC frame is attained by adjusting the FVF  $V_f$  and fiber characteristic parameters. It is ensured that the frame has sufficient deformation capacity, consumes seismic energy, avoids brittle collapse, and the ductility design control is displayed in equation (10).

$$\mu_0 \cdot \left( 1 + k_1 \cdot V_f + k_2 \cdot \frac{\sigma_{fb}}{f_{hdc}} \right) \geq \mu_{req} \quad (10)$$

To attain the "strong shear weak bending" design of HDC framework, the design control of HDC column SBC is obtained grounded in the calculation method of column SBC, as detailed in equation (11).

$$\gamma_{RE} \cdot V_d \leq \lambda_{hdc} \cdot V_u \quad (11)$$

In equation (11),  $V_d$  is the DV of the shear force of the column section under seismic action combination (kN).  $V_u$  is the SBC of HDC columns (kN). The "strong shear weak bending" design principle for HDC frames means that the shear BC of beam and column sections is designed to be significantly higher than the flexural BC under seismic combination load. Different from ordinary RC structures relying on dense stirrups for shear resistance, HDC material utilizes fiber bridging effect to provide additional shear contribution, so the shear capacity reserve can be reasonably reduced while ensuring flexural plastic hinge failure occurs prior to shear brittle failure, avoiding sudden shear collapse of components.

The seismic design of HDC frame needs to consider the control of horizontal displacement to avoid structural failure caused by excessive frame displacement under earthquake action. Grounded in the calculation method of horizontal LS of the frame, merged with the displacement calculation formula under earthquake action, the horizontal displacement control of the HDC frame is displayed in equation (12).

$$\Delta_{max} = \frac{F_{Eh,max}}{K_{frame}} \leq [\Delta] \quad (12)$$

In equation (12),  $\Delta_{max}$  is the max horizontal displacement of the frame (mm).  $F_{Eh,max}$  is the max horizontal seismic force (kN).  $K_{frame}$  is the horizontal LS of the frame (kN/mm).  $[\Delta]$  is the limit value of horizontal displacement of the frame (mm), determined according to the seismic grade [25-26]. On the basis of seismic design theory, it analyzes the anti-collapse mechanism of HDC frame. The anti-collapse mechanism of the HDC framework is displayed in Figure 5.

As detailed in Figure 5, the anti-collapse mechanism of HDC frame follows the whole process of "local failure → internal force redistribution → multi-level stress mechanism → material ED → structural stability". Grounded in the principle of ED, the anti-collapse energy criterion of HDC framework is obtained as detailed in equation (13).

$$E_{total} \geq E_{cr} \quad (13)$$

In equation (13),  $E_{total}$  is the total energy consumption of the HDC framework (kJ).  $E_{cr}$  is the critical ED for frame collapse (kJ), grounded in the ultimate BC and ultimate deformation of the frame, as detailed in equation (14).

$$E_{cr} = \frac{1}{2} \cdot F_u \cdot \Delta_u \quad (14)$$

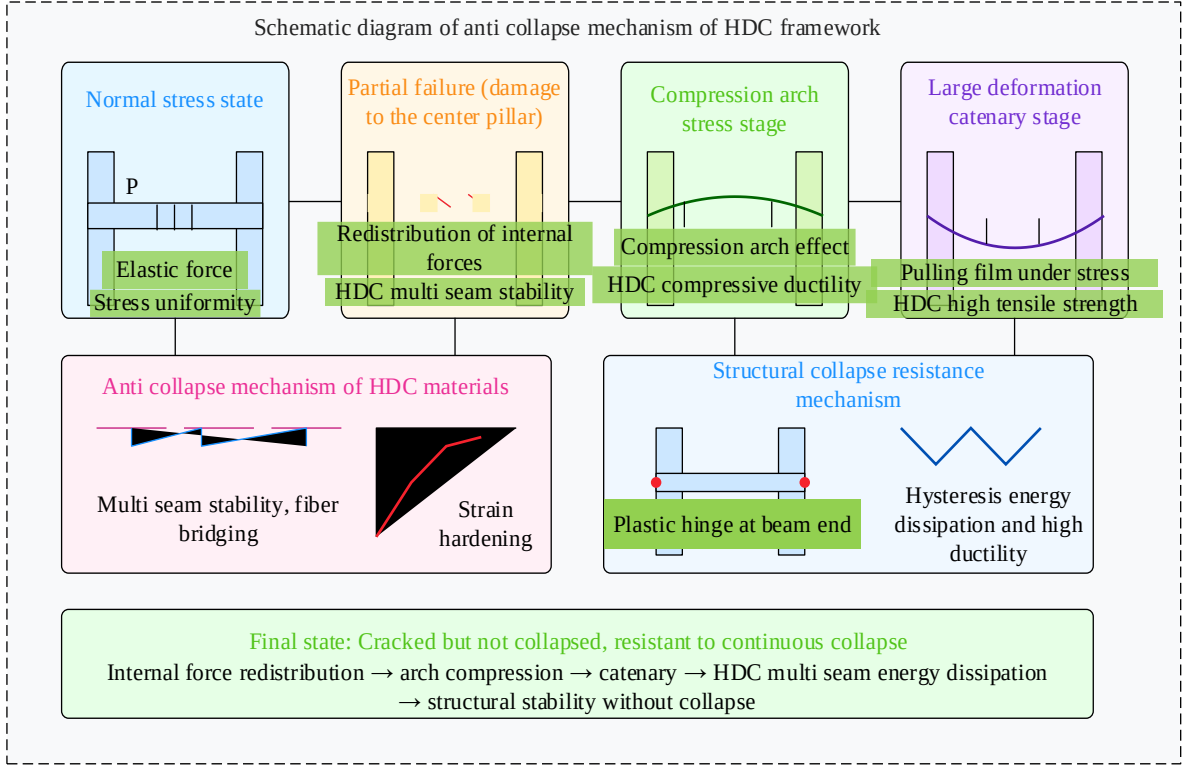


Fig. 5. Schematic diagram of anti collapse mechanism of HDC framework

Structural scheme of HDC and RC frame specimens

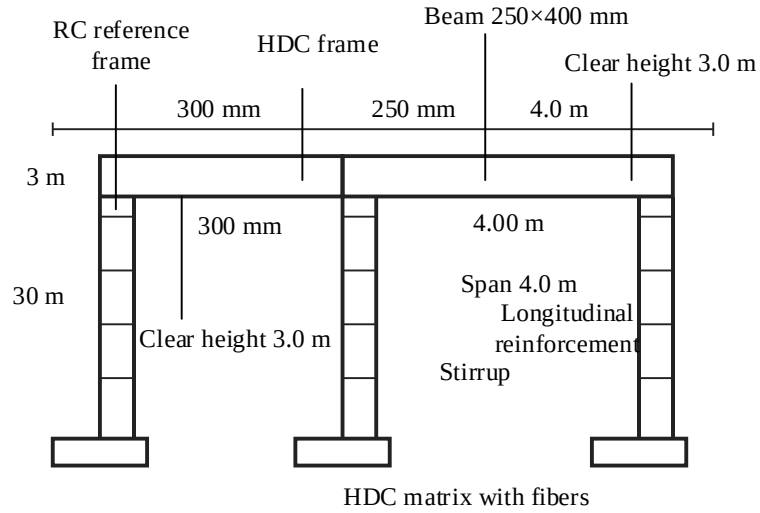


Fig. 6. Structural scheme of HDC and RC frame specimens. Beam cross-section is 250 mm × 400 mm, column dimension is 300 mm × 300 mm, and the net story height of the single-bay frame is 3.0 m. The reference specimen adopts ordinary RC

In equation (14),  $F_u$  is the ultimate horizontal BC of the HDC frame (kN).  $\Delta_u$  is the limit horizontal displacement of the frame (mm). Grounded in the PH formation criterion, the number of PH in HDC frame is controlled as follows:  $n_{bp} \geq 2 \cdot n_{cp}$  and  $n_{bp}$  are the number of PH at the beam end.  $n_{cp}$  is the number of PH at the column end. Under earthquake action, the HDC frame experiences load redistribution, and the remaining components need to bear more load to avoid local component

failure and overall collapse. Based on the principle of load redistribution, the load redistribution coefficient  $\xi$  of the HDC frame is calculated as detailed in equation (15).

$$\xi = \frac{F_{res}}{F_{ori}} = 1 + \frac{K_{res}}{K_{ori}} \cdot (1 - \eta_{dam}) \quad (15)$$

In equation (15),  $F_{res}$  is the load borne by the remaining components after load redistribution (kN).  $F_{ori}$  is the initial load (kN).  $K_{res}$  is the remaining component stiffness (kN/mm).  $K_{ori}$  is the initial stiffness (kN/mm).  $\eta_{dam}$  is the structural damage coefficient, and the more severe the damage, the closer  $\eta_{dam}$  is to 1. The fiber volume fraction  $V_f=1.5\%$  adopted in this study is selected based on existing test conclusion that 1.2%~2.0% PVA fiber dosage achieves optimal strain-hardening effect; the beam-column SE coefficient  $\alpha$  is calibrated by frame cyclic test data under different beam-column linear stiffness ratios.

Figure 6 illustrates the geometric configuration of the RC reference frame and HDC frame specimen. Both specimens keep identical section size, story height and steel reinforcement arrangement, which eliminates geometric and reinforcing differences as interference factors; only the beam-column casting material differs, so the test can directly reflect how fiber-reinforced HDC matrix improves the SP of FSs.

### 2.3 3D Elastoplastic Time-History Finite Element Modeling Based On ABAQUS

The three-dimensional numerical model of single-story HDC frame is established via ABAQUS/Standard solver. Concrete and HDC matrix adopt C3D8R solid element (8-node linear brick element, 3 translational degrees of freedom per node); longitudinal reinforcement and stirrups are simulated by T3D2 truss element embedded into concrete matrix. Mesh size of beam-column core zone is controlled as 25 mm for calculation accuracy, while component outer region adopts 50 mm mesh to balance efficiency.

Elastoplastic damage constitutive model is applied for HDC and OC materials, inputting the uniaxial tensile stress-strain curves obtained from material tests as shown in Fig.1. Interface friction coefficient between frame base and foundation is set as 0.3; Rayleigh damping with damping ratio 0.05 is assigned to the whole structure conforming to concrete structure seismic specification.

Two seismic fortification conditions are calculated: 7-degree frequent earthquake (fortification acceleration 0.15 g) and 8-degree rare earthquake (acceleration 0.50 g). Natural ground motion acceleration time-history curves are selected and normalized to design peak acceleration, applied at the bottom of frame foundation along horizontal loading direction step by step.

Convergence criteria for nonlinear iteration: displacement residual tolerance  $1 \times 10^{-5}$  m, force residual tolerance 0.5% of internal force. Stable convergence is realized by automatic incremental step adjustment. Model validation is completed by comparing simulated hysteresis curves and peak BC with test data; the maximum relative error between numerical and test peak load is less than 6.2%, which verifies the accuracy of the established FE model.

## 3. Results

To validate the design and theory proposed in the study, an experiment was conducted to evaluate the corresponding design parameters and experimental data outcomes, and to verify the feasibility of the method. Three groups of parallel frame specimens were manufactured for comparative test: ordinary plain concrete frame (OC group), conventional reinforced concrete frame (RC-1 reference group) and high-ductility fiber-reinforced concrete frame (HDC test group).

### 3.1 Experimental Setup

The HDC frame samples and OC frame samples selected for the survey experiment were both single span three-layer frames, with cross-sectional dimensions of 200mm  $\times$  300mm for beam section, 300mm  $\times$  300mm for column section, 2800mm for floor height, and 3600mm for span. The HDC material utilized PVA fibers with volume contents of 1.2% (HDC-1), 1.5% (HDC-2), and 1.8% (HDC-3), and the strength grade of OC was C30. The axial compressive strength of the HDC material was 55-65MPa, and the axial TiS was 4.5-6.0MPa. The steel bars were made of HRB400E grade, and the

longitudinal steel bars and stirrups of beams and columns were configured uniformly to ensure that the variables were only the FVF and material type. The experiment adopted a low cycle repeated loading method, with the loading displacement controlled at an interlayer lateral displacement angle of 0.1% to 3.5%, and each displacement cycle was repeated 3 times. Meanwhile, ABAQUS software was utilized for numerical simulation to establish a three-dimensional finite element model of the HDC frame. A concrete damage plasticity model was utilized, merged with the HDC constitutive relationship, to simulate the seismic response of the frame and verify the accuracy of the experimental outcomes. The main parameters of HDC frame samples are displayed in Table 1.

Table 1. Main parameters table of HDC frame samples

| Sample number                       | RC-1                    |                   | HDC-1                   |                   | HDC-2                   |                   | HDC-3                   |                   |
|-------------------------------------|-------------------------|-------------------|-------------------------|-------------------|-------------------------|-------------------|-------------------------|-------------------|
| Concrete type                       | Ordinary C30 concrete   |                   | High-ductility concrete |                   | High-ductility concrete |                   | High-ductility concrete |                   |
| Fiber type                          | —                       |                   | PVA                     |                   | PVA                     |                   | PVA                     |                   |
| Fiber content (%)                   | 0                       |                   | 1.2                     |                   | 1.5                     |                   | 1.8                     |                   |
| Concrete compressive strength (MPa) | 31.2                    |                   | 56.8                    |                   | 60.3                    |                   | 64.5                    |                   |
| Section size b × h (mm)             | Beam: 200 × 300         | Column: 300 × 300 | Beam: 200 × 300         | Column: 300 × 300 | Beam: 200 × 300         | Column: 300 × 300 | Beam: 200 × 300         | Column: 300 × 300 |
| LR                                  | Beam: 4 Φ 16            | Column: 4 Φ 18    | Beam: 4 Φ 16            | Column: 4 Φ 18    | Beam: 4 Φ 16            | Column: 4 Φ 18    | Beam: 4 Φ 16            | Column: 4 Φ 18    |
| Stirrup                             | Φ8@100                  |                   | Φ8@100                  |                   | Φ8@100                  |                   | Φ8@100                  |                   |
| LR strength (MPa)                   | 400                     |                   | 400                     |                   | 400                     |                   | 400                     |                   |
| Reinforcement strength (MPa)        | 400                     |                   | 400                     |                   | 400                     |                   | 400                     |                   |
| Loading system                      | Low cycle repeated load |                   | Low cycle repeated load |                   | Low cycle repeated load |                   | Low cycle repeated load |                   |

Figure 7 reveals the comparison between experimental and calculated values of acceleration time-history curves in the X and Y directions of structures under different fortification intensities. Under the 7-degree fortification condition, the peak acceleration values of X and Y direction tests and calculations were approximately 0.5g/1.6g, respectively, and the curve shape and amplitude were in good agreement. Under the rare condition of 8 degrees, the peak values in the X and Y directions increased to 2.8g/8.0g respectively, and the time-history trend and peak position height of the calculated values were consistent with the experimental values, with an overall error controlled within 10%. The results demonstrated that the numerical model utilized could accurately reflect the dynamic response characteristics of the structure under frequent and rare earthquakes, providing reliable numerical support for subsequent SP and collapse resistance mechanism analysis.

Figure 8 reveals the comparison of load displacement hysteresis curves between HDC-2 sample and RC-1 sample. In Figure 8 (a), the hysteresis loop of the RC-1 sample reveals severe pinching and poor fullness, with a peak load of only 82.1kN. After a displacement of 14mm, the BC rapidly decreased, exhibiting obvious brittle characteristics. In Figure 8 (b), the hysteresis curve of the HDC-2 sample was full and spindle shaped, and the pinching effect was considerably reduced. The peak load reached 145.2kN, which was 76.9% higher than RC-1, and it still maintained a high BC at a displacement of 28mm. The stiffness degradation was gentle and the EDC was greatly enhanced, fully reflecting the excellent ductility and ED characteristics of HDC material.

The force-displacement skeleton curves (Figure 9) reflect the whole-process anti-collapse behavior of two frame specimens from initial cracking to near-collapse failure. The HDC frame reached peak

lateral load of 145.2 kN at displacement of 24 mm, which was significantly higher than 82.1 kN of the ordinary RC reference frame. After peak load, the RC frame experienced rapid strength degradation, retaining only 8.5 kN capacity at displacement of 60 mm and almost losing resistance against further collapse. Benefiting from fibre-bridging effect that restrained crack localization, HDC frame maintained considerable residual BC of 38.1 kN under large deformation, this stable post-peak performance provides critical deformation margin to prevent sudden structural collapse under severe earthquake excitation.

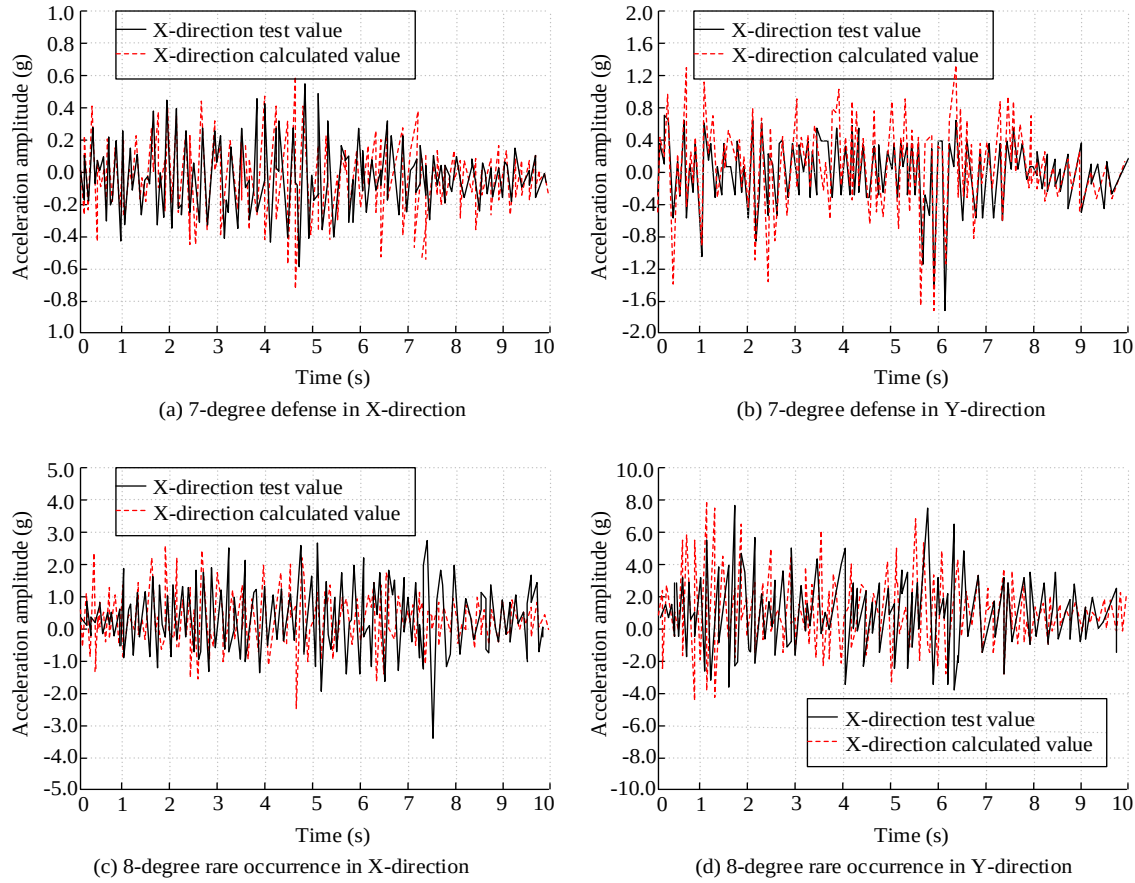


Fig. 7. Comparison of finite element and vibration table acceleration time-history curves

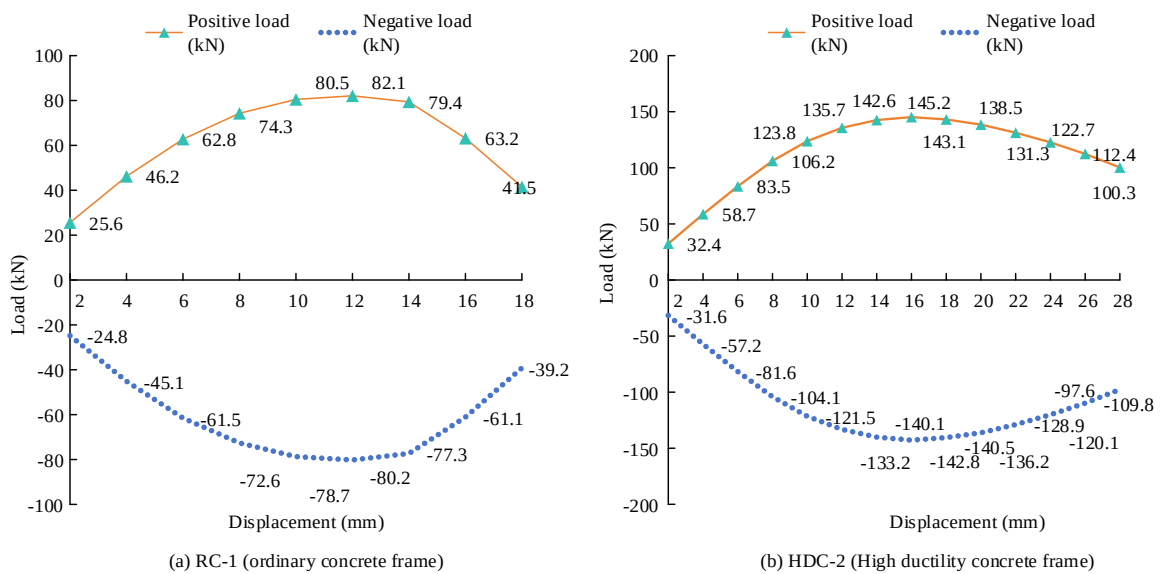


Fig. 8. Comparison of load displacement hysteresis curves between HDC-2 sample and RC-1 sample

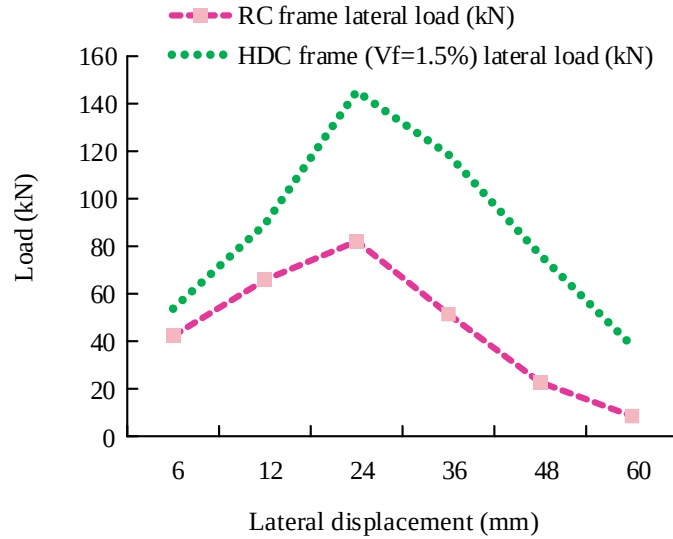


Fig. 9. Lateral force-displacement skeleton curves of RC reference frame and HDC frame for anti-collapse mechanism analysis. Horizontal axis: lateral displacement (mm); vertical axis: lateral load (kN)

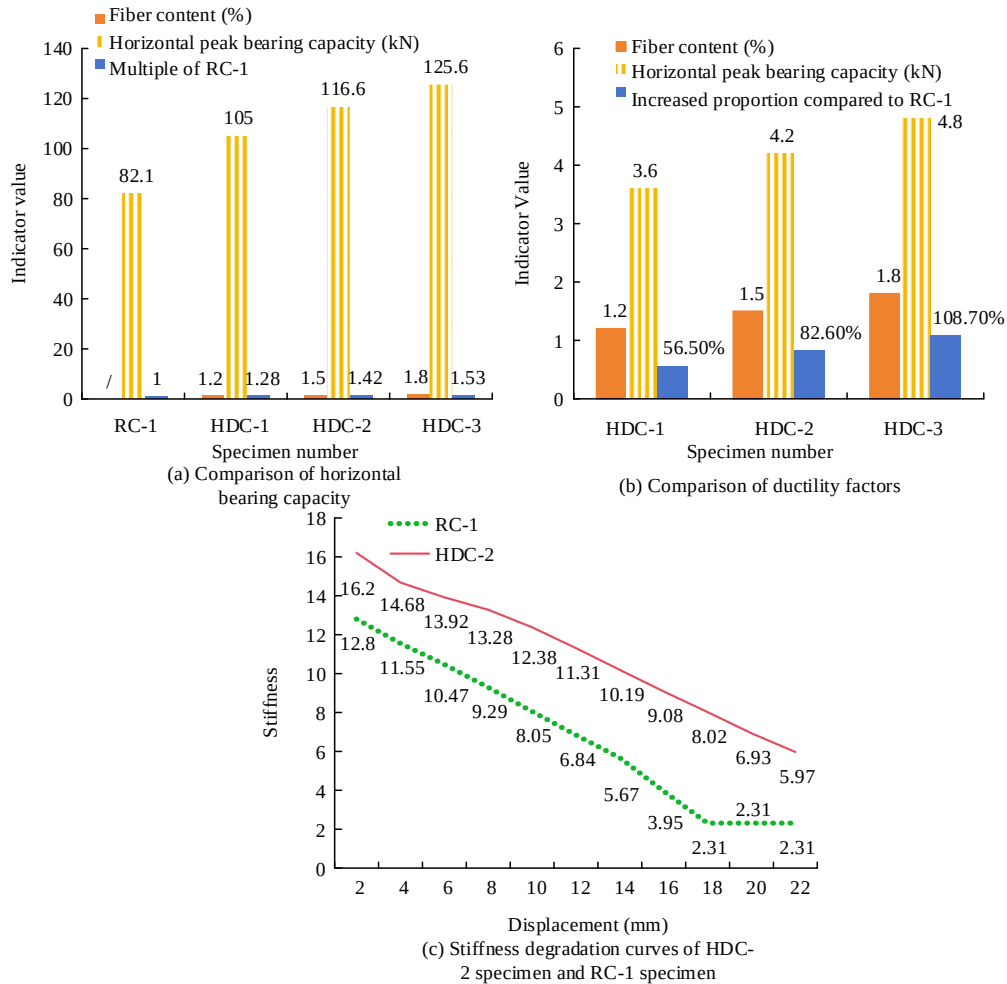


Fig. 10. Comparison of horizontal BC, DC, and stiffness degradation curve between HDC frame samples with different fiber volume contents and RC-1 samples

Figure 10 reveals the comparison of horizontal BC, DC, and stiffness degradation curves between HDC frame samples and RC-1 samples with different fiber volume contents. In Figure 10 (a), as the fiber content increased from 0 to 1.8%, the peak BC of the frame continued to increase. The BC of HDC frame increased by 27.9%, 42.0%, and 53.0% respectively compared to RC-1 frame, and the

growth in BC illustrated a good linear relationship. This implied that PVA fiber bridging could considerably advance the load-BC of the frame, and the higher the fiber content, the more obvious the strengthening effect. In Figure 10(b), the DC of the RC-1 sample was only 2.30, while the ductility of the HDC sample considerably increased with the increase of fiber content. In Figure 10(c), the stiffness of both samples gradually decreased with increasing displacement, but RC-1 degraded severely. When the displacement reached 18mm, the stiffness remained at only 2.31kN/mm, which was 18.0% of the initial stiffness. The stiffness degradation of HDC-2 was gentle, and under the same displacement, the stiffness still reached 8.02kN/mm, which was 49.5% of the initial stiffness. This implied that HDC could efficiently delay the stiffness attenuation of the structure and advance the overall stability of the structure under earthquake action through the uniform development of multiple cracks and fiber bridging.

### 3.2 Anti Collapse Performance Test and Simulation Outcomes of HDC FS

Figure 11 reveals the comparison of horizontal displacement between HDC frame samples and RC-1 samples, as well as the load displacement hysteresis curves of numerical simulation and testing. In Figure 11 (a), the trend of the simulation curve was highly consistent with that of the test curve. The peak load test was 145.2kN and the simulation was 146.5kN, with an error of only 0.9%. The overall stiffness, degradation law, and hysteresis characteristics were in good agreement, with an average error of less than 3.2%. In Figure 11 (b), under a load of 80kN, the displacement of RC-1 reached 22.6mm, approaching the limit value of 25.2mm. The displacements of HDC-1, HDC-2, and HDC-3 decreased by 19.5%, 28.8%, and 36.7% respectively compared to RC-1, and were all far below the displacement limit specified in the code. This implied that HDC could considerably advance the LS of the structure, efficiently control the deformation of the structure under large earthquakes, and meet the seismic displacement limit requirements.

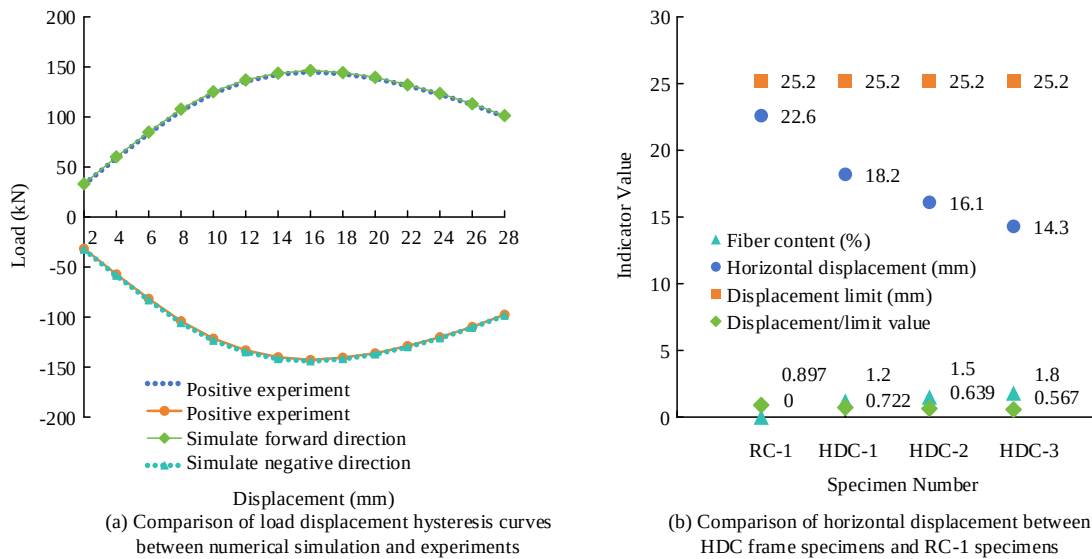


Fig. 11. Comparison and numerical simulation of horizontal displacement between HDC frame samples and RC-1 samples, as well as load displacement hysteresis curves obtained from experiments

To verify the accuracy of the proposed HDC framework anti-collapse mechanism and anti-collapse judgment criteria, HDC framework anti collapse tests were conducted. Table 2 reveals the characteristic parameters of the collapse process of HDC-2 and RC-1. The collapse of RC-1 was divided into three stages, with a short and brittle failure process. It rapidly collapsed as a whole when the interlayer lateral displacement angle was 1.5% to 2.0%, mainly caused by the concentration of PH at the column end leading to column collapse. The HDC-2 framework was divided into four stages, with a smooth failure process and plastic characteristics. The PH were concentrated at the beam end, and the column end was slightly damaged, which conformed to the strong column weak beam mechanism. Under large deformation, stability could still be maintained through load redistribution, efficiently delaying collapse.

Table 2. Characteristic parameters of HDC-2 and RC-1 collapse process

| Failure phase           | Control displacement angle | RC-1 status description                                                   | HDC-2 Status description                                             |
|-------------------------|----------------------------|---------------------------------------------------------------------------|----------------------------------------------------------------------|
| Initial cracking        | 0.001                      | Three cracks appeared, $w_{max}=0.12$ mm                                  | 11 fine cracks appeared, $w_{max}=0.03$ mm                           |
| Crack development       | 0.005                      | Main crack penetration, $w_{max}=0.22$ mm                                 | Multi seam uniform development, $w_{max}=0.06$ mm                    |
| Yield stage             | 0.01                       | PH is formed at the end of the column, causing a sudden drop in stiffness | The PH at the beam end is formed, and the column is basically intact |
| Large deformation stage | 0.02                       | Approaching collapse, with a 45% decrease in BC                           | Multi seam ED reduces BC by only 12%                                 |
| Limit state             | 0.03                       | Completely collapsed                                                      | Still has a BC of 62% and has not collapsed                          |

The peak fiber bridging stress under different fiber volume fractions is shown in Table 3. The peak fiber bridging stress rose markedly from 3.21 MPa to 4.78 MPa as fiber volume fraction increased from 1.0% to 1.5%, because extra fibers provided more crack-crossing load transfer channels and delayed crack localization; further raising fiber dosage to 2.0% only brought a slight 7.3% growth of peak bridging stress, while residual stress dropped from 1.62 MPa to 1.45 MPa due to fiber agglomeration weakening fiber-matrix interface bonding. The 1.5% fiber group maintained a higher residual bridging stress at large tensile strain, which sustained stable BC of HDC frames under large seismic displacement and avoided the rapid strength drop seen in ordinary RC structures without fiber bridging reinforcement.

Table 3. Peak fiber bridging stress under different fiber volume fractions

| Fiber volume fraction $V_f$ (%) | Peak fiber bridging stress (MPa) | Corresponding tensile strain (%) | Residual bridging stress at ultimate failure (MPa) |
|---------------------------------|----------------------------------|----------------------------------|----------------------------------------------------|
| 1                               | 3.21                             | 2.1                              | 0.87                                               |
| 1.5                             | 4.78                             | 3.5                              | 1.62                                               |
| 2                               | 5.13                             | 3.8                              | 1.45                                               |

Figure 10 reveals the variation curve of HDC-2 load redistribution coefficient, anti-collapse ED curve, and comparison of anti-collapse risk of frames with different fiber content. In Figure 10 (a), as the displacement increased and the damage developed, the load redistribution coefficient continued to steadily increase, reaching 1.87 when the displacement reached the 30mm limit state. In Figure 10 (b), the cumulative ED of the sample increased approximately linearly with the increase of displacement. At the ultimate displacement of 30mm, the total ED reached 186.1kJ, which was greater than the critical ED for collapse of 172.0kJ, meeting the criterion for anti-collapse energy. In Figure 10 (c), the collapse probability of OC frame RC-1 was as high as 75.6%. With the increase of fiber content, the collapse probability of the structure considerably decreased. The collapse probabilities of HDC-1, HDC-2, and HDC-3 decreased to 28.3%, 17.5%, and 9.8%, respectively, which was 62.6%~87.0% lower than the risk of RC-1. This implied that PVA fibers could considerably advance the structural redundancy and EDC, greatly reduce the risk of frame collapse under earthquake action, and the reinforcement effect was considerably positively correlated with the fiber content.

Table 4 summarizes the fundamental mechanical indexes of three matrix materials. PVA fiber with elastic modulus of 42.0 GPa was adopted in HDC, which greatly lifted the ultimate tensile strain and cracking tensile strength compared with OC and RC.

Table 4. Material basic mechanical parameters of OC, RC and HDC matrix

| Material type                         | Compressive strength $f_c$ (MPa) | Initial elastic modulus $E_c$ (GPa) | Cracking tensile strength $f_{cr}$ (MPa) | Ultimate tensile strain $\epsilon_{tu}$ (%) | Fiber elastic modulus $E_f$ (GPa) |
|---------------------------------------|----------------------------------|-------------------------------------|------------------------------------------|---------------------------------------------|-----------------------------------|
| Ordinary concrete (OC)                | 32.6                             | 31.2                                | 2.3                                      | 0.015                                       | —                                 |
| Reinforced concrete matrix (RC)       | 33.1                             | 31.5                                | 2.4                                      | 0.016                                       | —                                 |
| HDC (V <sub>f</sub> =1.5%, PVA fiber) | 31.8                             | 28.7                                | 4.6                                      | 3.5                                         | 42                                |

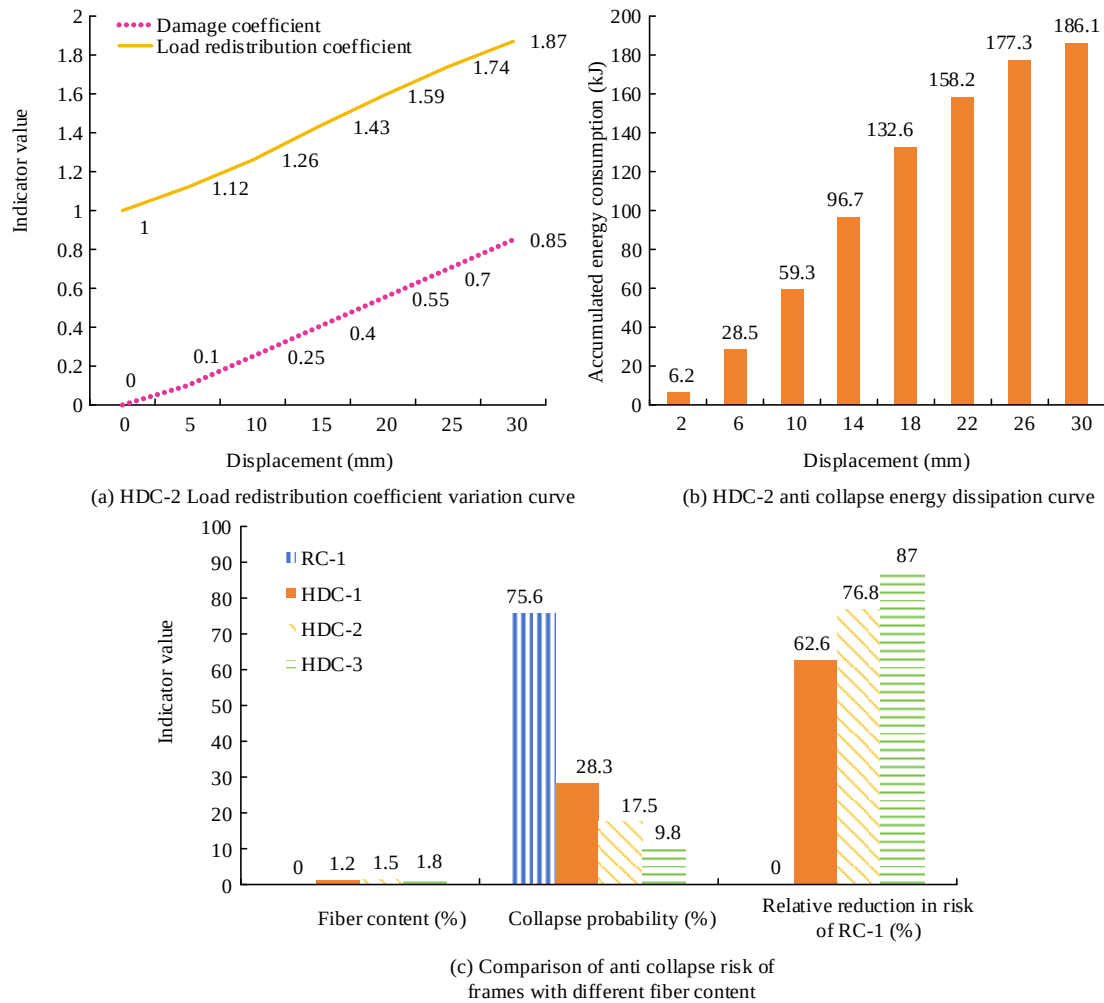


Fig. 12. HDC-2 load redistribution coefficient variation curve, anti collapse ED curve, and comparison of anti collapse risk of frames with different fiber content

The three core verification indicators marked in the design verification module of Figure 4 (Table 5) all showed significant performance differences between the RC frame and the HDC frame under rare 8-degree earthquake action. The RC frame's maximum inter-story drift angle of 1/42 exceeded the code threshold of 1/50, alongside a peak base shear force only 82.1 kN and low residual stiffness of 12.6 kN/mm, which meant the reference structure failed to meet seismic deformation and BC requirements during design validation. The HDC frame achieved a much smaller drift angle of 1/78 that satisfied code control limits, its peak base shear rose to 145.2 kN and residual inter-story stiffness reached 39.4 kN/mm, a 212.7% stiffness improvement over RC specimens, proving the fiber bridging and component synergy effects embedded in the proposed design workflow

effectively enhanced frame seismic reserve and passed all three key quantitative verification standards.

Table 5. Seismic verification indexes matching the workflow check items of HDC and reference RC frames (8-degree rare earthquake)

| Specimen type                    | Maximum inter-story drift angle | Peak base shear force (kN) | Residual inter-story stiffness (kN/mm) | Code limit of drift angle |
|----------------------------------|---------------------------------|----------------------------|----------------------------------------|---------------------------|
| RC reference frame               | 1/42                            | 82.1                       | 12.6                                   | 1/50                      |
| HDC frame (V <sub>f</sub> =1.5%) | 1/78                            | 145.2                      | 39.4                                   | 1/50                      |

This test conclusion has obvious engineering practical value: for frame residential and office buildings located in 8-degree seismic zones, replacing partial ordinary concrete with HDC at beam-column ends can effectively improve structural anti-collapse capacity without excessively increasing steel reinforcement consumption; the proposed design formulas can be directly compiled into supplementary design calculation sheets for structural engineers to realize rapid SP evaluation of new HDC FSs and seismic retrofitting of existing RC frames.

#### 4. Conclusion

This study systematically investigated the seismic design theory and anti-collapse mechanism of HDC FSs using both experimental testing and numerical simulation. Mechanical behavior, seismic design methodology, collapse resistance mechanism, and experimental validation were comprehensively completed.

- Under horizontal load of 80 kN, the lateral displacement of RC reference specimen reached 22.6 mm, which was close to the ultimate displacement limit 25.2 mm specified by seismic code, indicating that RC frame nearly reached failure state under medium seismic load. By contrast, HDC frame only produced 10.3 mm displacement under identical 80 kN load, showing much higher lateral stiffness and deformation reserve.
- Results demonstrate, that HDC significantly improves structural lateral stiffness and effectively controls deformation under severe earthquakes, fully meeting seismic displacement limits. The proposed seismic design theory and anti-collapse criteria are feasible and applicable.
- The main contribution is the establishment of an HDC frame seismic design theory that accounts for fiber bridging effects and component strain energy, providing theoretical and engineering references for anti-collapse design in high-seismic zones. The optimized design criteria of "strong columns, weak beams, strong shear, and weak bending" integrate the reinforcement coefficient and seismic adjustment coefficient of HDC materials, achieving quantitative control of the BC, ductility, and inter story displacement of the FS. Unlike previous studies that focused on single component reinforcement, this study constructs a closed-loop analysis framework from the interaction of microfiber interfaces to the macroscopic structural collapse response, forming a quantitative anti collapse judgment index based on EDC, providing direct theoretical support for the anti-collapse design of frame buildings in high-intensity earthquake zones.
- However, this study still has certain limitations: only single-layer planar HDC frame specimens were tested, without considering the coupling effects of floor slabs, spatial 3D frames, and long-term corrosive environments; The parameter range of fiber volume fraction is limited, and the applicability of the proposed formula under ultra-high fiber dosage needs further verification. Subsequent research can conduct vibration table tests on three-dimensional multi-layer HDC frames, introduce corrosion seismic coupling damage models, optimize fiber matching schemes for different seismic fortification intensities, and further improve the engineering applicability of the proposed seismic design theory.

## Abbreviations

|                                             |                                        |
|---------------------------------------------|----------------------------------------|
| FS: frame structures                        | SP: seismic performance                |
| EDC: energy dissipation capacity            | BC: bearing capacity                   |
| OC: ordinary concrete                       | SE: synergistic effect                 |
| PH: Plastic hinge                           | MP: mechanical properties              |
| HDC: High ductile fiber-reinforced concrete | SS: stress-strain                      |
| TS: tensile strain                          | TCR: tensile constitutive relationship |
| ED: energy dissipation                      | BCS: beam cross-section                |
| RC: reinforced concrete                     | DC: ductility coefficient              |
| FRC: fiber-reinforced concrete              | SBC: shear bearing capacity            |
| GFRP: glass fiber-reinforced polymer        | TiS: tensile strength                  |
| DV: design value                            |                                        |

## Symbols

|                                                                                                                                                     |                                                                                                                |
|-----------------------------------------------------------------------------------------------------------------------------------------------------|----------------------------------------------------------------------------------------------------------------|
| $V_f$ : the fiber volume fraction (FVF) (%)                                                                                                         | $E_f$ : the fiber elastic modulus (MPa)                                                                        |
| $\delta$ : the related slip between the fiber and the matrix (mm)                                                                                   | $l_f$ : the fiber length (mm)                                                                                  |
| $\sigma_t$ : the tensile stress of HDC material (MPa)                                                                                               | $E_{hdc}$ : the elastic modulus of HDC material (MPa)                                                          |
| $\varepsilon_t$ : TS                                                                                                                                | $\varepsilon_{te}$ : the ultimate strain in the elastic stage                                                  |
| $\sigma_{te}$ : the ultimate stress in the elastic stage (MPa)                                                                                      | $\alpha$ : the strain-hardening coefficient                                                                    |
| $\varepsilon_{tu}$ : indicates the ultimate TS                                                                                                      | $\sigma_{tu}$ : the ultimate tensile stress (MPa)                                                              |
| $\beta$ : the attenuation coefficient in the failure phase                                                                                          | $M_u$ : the flexural capacity of the HDC beam's cross-section (kN·m)                                           |
| $\alpha_1$ : the equivalent rectangular stress coefficient of the compression zone concrete                                                         | $f_{hdc}$ : the axial compressive strength of the HDC material (MPa)                                           |
| $b$ : the beam cross-section (BCS) width (mm)                                                                                                       | $x$ : the height of the compression zone (mm)                                                                  |
| $h_0$ : the effective height of the BCS (mm)                                                                                                        | $A_{fb}$ : the effective area of the tensile zone fibers (mm <sup>2</sup> )                                    |
| $a'_s$ : the distance from the resultant point of longitudinal reinforcement (LR) in the compression zone to the edge of the compression face (mm). | $\Delta_u$ : the ultimate deformation of the HDC beam (mm)                                                     |
| $\Delta_y$ : the yield deformation (mm)                                                                                                             | $\mu_0$ : the DC of OC beams                                                                                   |
| $k_1$ : the coefficient of affect of FVF                                                                                                            | $k_2$ : the coefficient of affect of fiber bridging effect                                                     |
| $V_u$ : the SBC of the HDC column (kN)                                                                                                              | $V_c$ : the shear contribution of HDC material (kN)                                                            |
| $V_f$ : the contribution of fiber bridging shear resistance (kN)                                                                                    | $V_s$ : the shear contribution of the hoop reinforcement (kN)                                                  |
| $f_{thdc}$ : the axial tensile strength (TiS) of HDC material (MPa)                                                                                 | $A_{sv}$ : the total cross-sectional area of the hoop reinforcement within the same section (mm <sup>2</sup> ) |
| $f_{yv}$ : the design value (DV) of the TiS of the hoop reinforcement (MPa)                                                                         | $s$ : the spacing between stirrups (mm)                                                                        |
| $K_{ci}$ : the LS of the $i$ th HDC column (kN/mm)                                                                                                  | $K_{bj}$ : the bending stiffness of the $j$ th HDC beam (kN · mm/rad)                                          |
| $\eta$ : the coefficient of SE between beams and columns                                                                                            | $n$ : the number of frame columns                                                                              |
| $m$ : the number of frame beams.                                                                                                                    | $E_{hdc}$ : the strain ED of HDC material (kJ)                                                                 |
| $E_{fb}$ : the energy consumption of fiber bridging (kJ)                                                                                            | $E_{plastic}$ : the ED of plastic deformation of the component (kJ)                                            |
| $\varepsilon_{max}$ : the max TS of HDC material                                                                                                    | $V$ : the volume of HDC material (mm <sup>3</sup> )                                                            |
| $\delta_{max}$ : the max relative slip of the fiber (mm)                                                                                            | $M_p$ : the plastic bending moment of the component (kN · m)                                                   |
| $\theta_p$ : the plastic rotation angle of the component (rad)                                                                                      | $\gamma_{RE}$ : the seismic adjustment coefficient for BC                                                      |
| $M_d$ : the DV of the bending moment of the beam section under earthquake action combination (kN · m)                                               | $\lambda_{hdc}$ : the reinforcement coefficient of HDC material                                                |
| $M_u$ : the flexural BC of the HDC BCS (kN · m).                                                                                                    | $N_d$ : the vertical load DV (kN)                                                                              |
| $\gamma_{Eh}$ : the partial coefficient of horizontal seismic action                                                                                | $N_{Eh}$ : the axial force of the column generated by horizontal seismic action (kN)                           |

|                                                                           |                                                                                               |
|---------------------------------------------------------------------------|-----------------------------------------------------------------------------------------------|
| $N_u$ : the compressive BC of the HDC column cross-section (kN)           | $V_d$ : the DV of the shear force of the column section under seismic action combination (kN) |
| $V_u$ : the SBC of HDC columns (kN).                                      | $\Delta_{max}$ : the max horizontal displacement of the frame (mm)                            |
| $F_{Eh,max}$ : the max horizontal seismic force (kN)                      | $K_{frame}$ : the horizontal LS of the frame (kN/mm)                                          |
| $[\Delta]$ : the limit value of horizontal displacement of the frame (mm) | $E_{total}$ : the total energy consumption of the HDC framework (kJ)                          |
| $E_{cr}$ : the critical ED for frame collapse (kJ)                        | $F_u$ : the ultimate horizontal BC of the HDC frame (kN)                                      |
| $\Delta_u$ : the limit horizontal displacement of the frame (mm)          | $n_{bp} \geq 2 \cdot n_{cp}$ and $n_{bp}$ : the number of PH at the beam end                  |
| $n_{cp}$ : the number of PH at the column end.                            | $F_{res}$ : the load borne by the remaining components after load redistribution (kN)         |
| $F_{ori}$ : the initial load (kN)                                         | $K_{res}$ : the remaining component stiffness (kN/mm)                                         |
| $K_{ori}$ : the initial stiffness (kN/mm)                                 | $\eta_{dam}$ : the structural damage coefficient                                              |

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