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Article

Analytical Study on Reinforced Concrete Columns and Composite Columns under Lateral Impact

Xianhui Li ^{1,2}, Yao Yin ¹, Tieying Li ¹, Xiang Zhu ^{3,*} and Rui Wang ¹

¹ College of Civil Engineering, Taiyuan University of Technology, Taiyuan 030024, China

² College of Architectural Engineering, Shanxi Vocational University of Engineering Science and Technology, Jinzhong 030600, China

³ School of Electric Power, Civil Engineering and Architecture, Shanxi University, Taiyuan 030013, China

* Correspondence: zhuxiang@sxu.edu.cn

Abstract: This study investigates the lateral impact responses of reinforced concrete (RC) and composite columns through dynamic nonlinear analysis using LS-DYNA. The simulation results were first validated against experimental results performed earlier on four different cross sections. The finite element analysis results show that the simulation results of LS-DYNA can predict the experimental results well and can be used for further parametric analysis. The overall impact resistance of the four new composite columns is significantly better than that of RC columns. Among the composite columns, the solid concrete-filled double steel tube (S-DS) column has the best impact resistance with higher impact plateau force and smaller mid-span deflection under the same test conditions. It was found that the impact response process of all types of composite columns was similar. Finally, parametric analysis of the composite columns is performed to study the influence of load, material and other related parameters on the impact response of the composite columns. The results provide new information on the impact response of composite columns and the influence of materials and load parameters. The study provides a basis for the design and analysis of composite columns under lateral impact loading.

Keywords: RC columns; composite columns; lateral impact; numerical simulation; whole-process analysis; parameter sensitivity analysis

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1. Introduction

The collision load effect on structural components has to be considered in structural design and sometimes considered in whole life cycle analysis. Relevant reports on collision accidents are increasing year by year [1–5]. It is well-known that the compressive strength of concrete is much higher than its tensile strength. Furthermore, the compressive strength is enhanced under bi-axial or tri-axial restraint. For structural steel, the tensile strength is high while the shape may buckle locally in compression. In concrete-filled steel tubular members, steel and concrete are used such that their natural and most prominent characteristics are taken advantage of. The confinement of concrete is provided by the steel tube, and the local buckling of the steel tube is improved due to the support of the concrete core. To ensure the safety of buildings and other structures under impact loading, concrete-filled steel tube (CFST) composite columns were used to improve the impact resistance of new structural columns. In the past few decades, CFST has been extended to other forms of composite columns [6–8], such as R-ST columns, S-ST columns, H-DS columns and S-DS columns. These composite columns are expected to have improved impact resistance to reduce the losses caused by accidental collisions.

The mechanical properties of composite columns under impact have been studied by a number of researchers. Aghdamy et al. [9,10] studied the impact resistance of hollow concrete-filled double steel tube columns using parameter sensitivity analysis. Results show that the deflection is significantly affected by axial load. The impact velocity, slenderness ratio and thickness-to-diameter ratio of steel tube are the key factors in determining the

dynamic response of CFST members as these are the parameters that determine the impact energy of the loads and stiffness of the members. Zhu et al. [11,12] studied the impact resistance of steel-reinforced concrete (SRC) columns and steel-reinforced concrete-filled circular steel tube (SRCFST) columns. The results show that the impact resistance of SRCFST columns is better than that of SRC columns under the same mass. Wang et al. [13] studied the behaviors of CFST under lateral impact by experimentation and finite element methods. The results show that the critical failure energy increases with an increase in the constraint factor, and the axial force has a significant influence on the lateral displacement of specimens. Xu et al. [14] and Wang et al. [15] studied the impact behaviors of carbon fiber-reinforced polymer (CFRP)-reinforced RC columns by experimental and numerical methods. The results show that CFRP-reinforced RC columns can effectively change the lateral impact failure mechanism from shear to bending, which can effectively reduce impact damage. Han et al. [16–18] studied the impact resistance of a CFST column and a CFST composite column by experimental and finite element methods. The results showed that the CFST column had good impact resistance, while the concrete in the outer layer of the composite column was seriously damaged; inner steel tube only had limited flexural deformation. Wang et al. [19,20] carried out lateral impact tests on 31 hollow concrete-filled double steel tube columns using a drop hammer device. The results showed that CFST members had good ductility under impact loading, and the sandwich steel tube had an obvious restraint effect on concrete. Goldston et al. [21] investigated the behavior of glass fiber-reinforced polymer (GFRP) bar-reinforced high-strength concrete and ultra-high-strength concrete under impact loading. Swesi et al. [22] studied CFRP strengthening on the response of RC columns under lateral impact loads.

Recently, a total of 30 columns (6 RC columns and 24 composite columns) were tested under low-velocity impact by Zhu et al. [23]. The test results show that the change in the axial compression ratio has an important effect on the impact resistance of specimens. In this paper, LS-DYNA software is used to describe the nonlinear finite element analysis (FEA) of RC and composite columns under transverse impact. The FEA model is validated with experimental results. Then, the whole process of impact damage on the composite column is analyzed. The validated model is then used to conduct an extensive parametric study to investigate the effects of impact height, impact mass, unconfined compressive strength of concrete, yield strength of outer and inner steel tubes, diameter–thickness ratio of outer and inner steel tubes, influence of H-shaped steel on impact force and mid-span residual deflection.

2. Finite Element Model

2.1. Composite Column Parameters

The validation of the models is based on an impact test of composite columns [23]. This numerical study simulates the impact resistance of composite columns with five different cross sections (shown in Figure 1A) under axial compression ratios of 0, 0.3 and 0.5, resulting in 15 combinations, which are calculated as N_0/N_u , where N_0 is the axial force applied in the test and N_u is the axial ultimate capacity of the specimen obtained from references [24–27]. The parameters of the columns are shown in Table 1 and the dimensions of the different sections are shown in Figure 1A [23]. To compare the impact resistance of members with different section types, the composite columns and reinforced concrete columns are designed to have the same outer diameters and use the same amount of steel. The outer diameter of the steel tube is 114 mm and the wall thickness is 2 mm. For solid concrete-filled double steel tube (S-DS) and hollow concrete-filled double steel tube (H-DS) sections, the inner circular steel tube diameter is 50 mm and the wall thickness is 2.5 mm. The size of the cross-sectional structural steel of S-ST columns (Figure 1(Af)) is 57 mm $\times$ 14 mm $\times$ 2 mm $\times$ 3 mm ($h \times b \times t_1 \times t_2$). The longitudinal reinforcement diameter (D) of steel tubular-encased reinforced concrete (R-ST) and reinforced concrete (RC) members is 6 mm, and the stirrup diameter (d) is 4 mm. There is a 300 mm long plastic hinge zone at both ends of the specimen, where the stirrup spacing is 100 mm. Stirrup spacing is 200 mm outside the plastic hinge zone. The member length is 1800 mm for all.

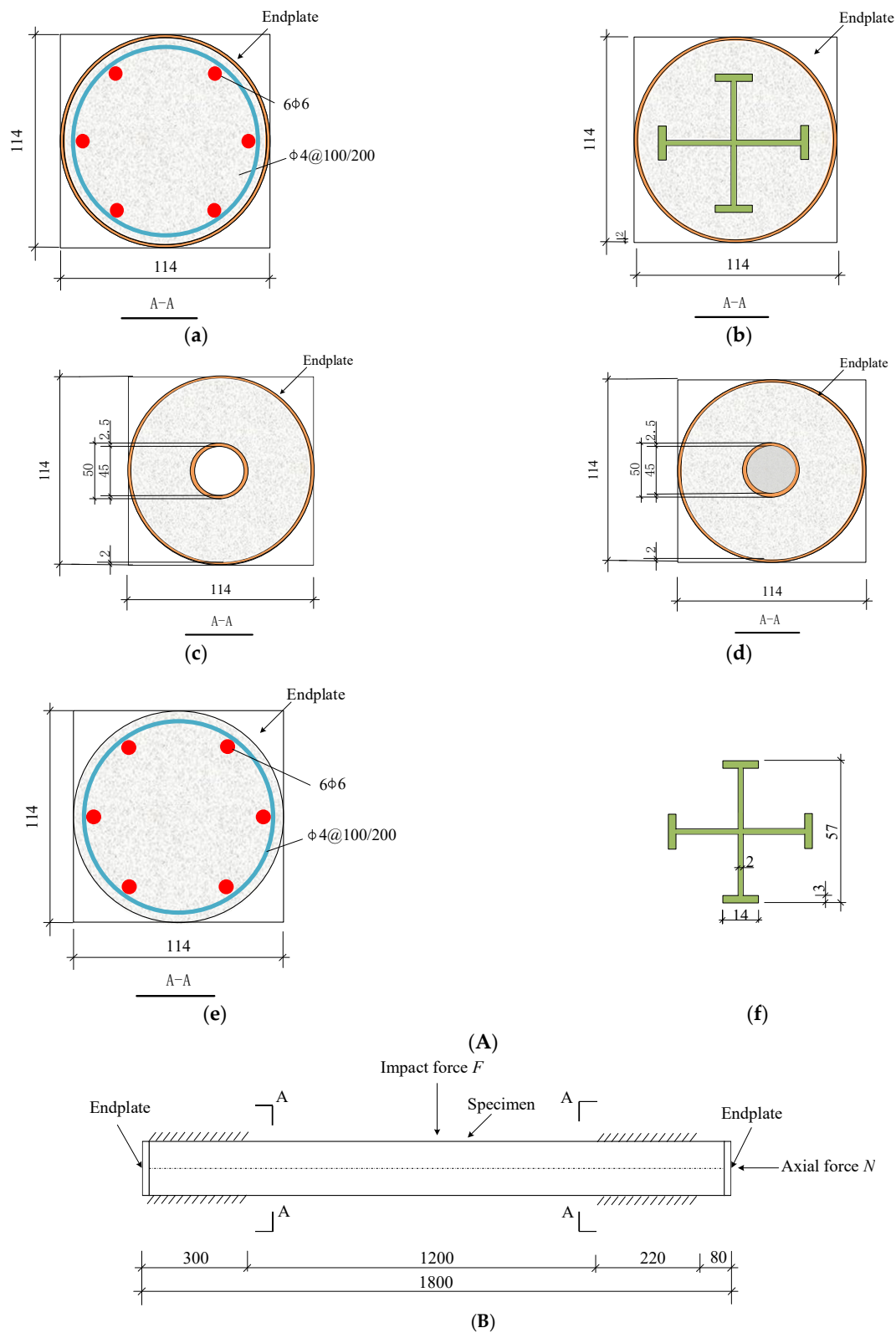
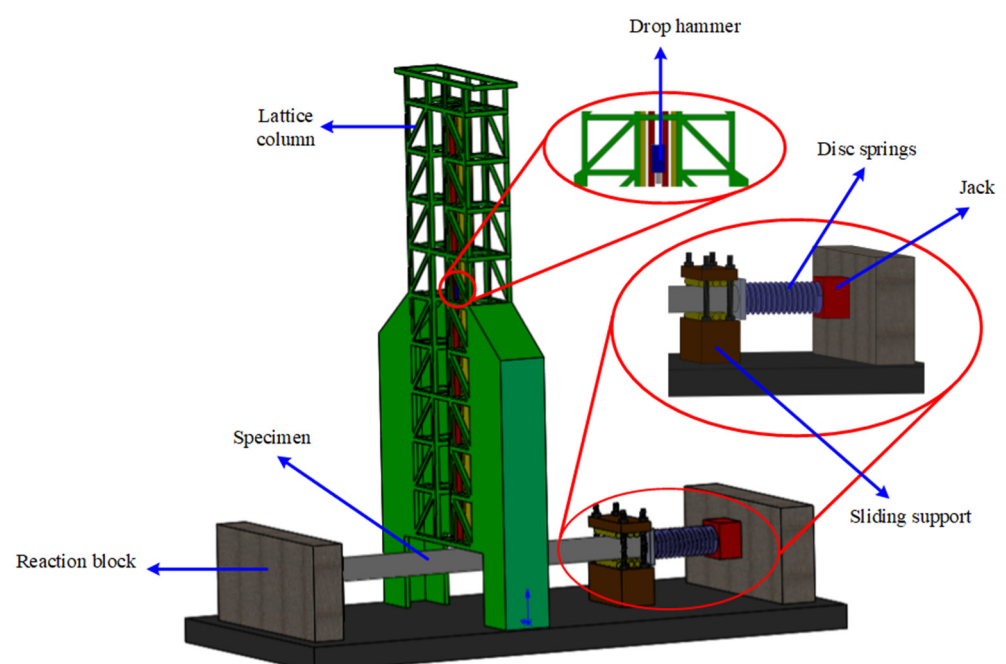


Figure 1. (A) Dimensions of the specimens (unit: mm). (a) Section of R-ST columns; (b) Section of S-ST columns; (c) Section of H-DS columns; (d) Section of S-DS columns; (e) Section of RC columns; (f) Section dimensions of the double H-shaped steel. (B) Plan view of the specimens.

Table 1. Specimen information table.

The Name of the Specimen	ID	N_0 (kN)	v_0 (m/s)	m_0 (kg)	F_{stab} (kN)		t_d (ms)		Δ_{res} (mm)	
					Test	FEA	Test	FEA	Test	FEA
Reinforced concrete (RC)	RC1	0	4.43	206.65	27.40	30.56	46.10	44.72	50.90	37.70
	RC2	59	4.43	206.65	22.45	22.09	45.45	31.52	80.88	63.93
	RC3	99	4.43	206.65	23.30	11.78	61.15	46.04	—	—
Steel tubular-encased reinforced concrete (R-ST)	R-ST1	0	9.39	206.65	135.24	129.33	23.60	21.19	56.72	60.82
	R-ST2	150	9.39	206.65	126.13	106.30	24.40	23.91	65.06	66.99
	R-ST3	250	9.39	206.65	104.70	97.66	27.55	24.33	73.87	77.53
Steel-reinforced concrete-filled steel tube (S-ST)	S-ST1	0	9.39	206.65	131.45	109.26	23.35	27.45	63.65	59.12
	S-ST2	190	9.39	206.65	118.69	88.68	24.90	29.90	75.51	69.22
	S-ST3	315	9.39	206.65	88.97	75.26	31.15	33.75	89.86	83.57
Hollow concrete-filled double steel tube (H-DS)	H-DS1	0	9.39	206.65	144.22	122.80	22.30	23.20	52.14	53.55
	H-DS2	170	9.39	206.65	129.50	108.11	23.40	24.25	70.05	61.67
	H-DS3	285	9.39	206.65	112.96	105.96	25.55	24.05	80.17	75.67
Solid concrete-filled double steel tube (S-DS)	S-DS1	0	9.39	206.65	125.34	112.95	21.50	24.25	57.82	51.14
	S-DS2	215	9.39	206.65	133.90	110.50	23.20	23.45	68.17	58.85
	S-DS3	355	9.39	206.65	113.27	112.80	24.65	22.85	76.37	69.54

The impact mass of the impact test is 206.65 kg, and the impact height of the hammer is 4.5 m, corresponding to an impact speed of 9.391 m/s. When the impact velocity is 9.391 m/s, the damage to the RC component is serious and has no analytical value, so the impact velocity of the RC component is set to 4.43 m/s. The test equipment is illustrated in Figure 2. During the test, the column is allowed to deform freely along the axial direction. The axial load was maintained at designed axial load ratios of 0, 0.3 and 0.5. The boundary conditions of the composite column in the test are fixed at both ends for rotations, and the effective length (L_0) of each specimen is 1200 mm (shown in Figure 1B).

**Figure 2.** Test setup.

2.2. Finite Element Mesh and Contact Definition

In the finite element model, the outer and inner steel tubes and H-shaped beams are simulated with four-node SHELL163 elements, and the thickness of the steel tube has five integral points to capture the local bending of the steel tube. The filled concrete and drop hammer were modelled using eight-node SOLID164 elements. Through the study of mesh convergence, the appropriate density and mesh element are determined to ensure the simulation has sufficient accuracy and efficiency, and ensure that the hourglass energy is less than 5% of the total impact energy [28]. Figure 3 shows the typical finite element model with a minimum mesh size of 5 mm.

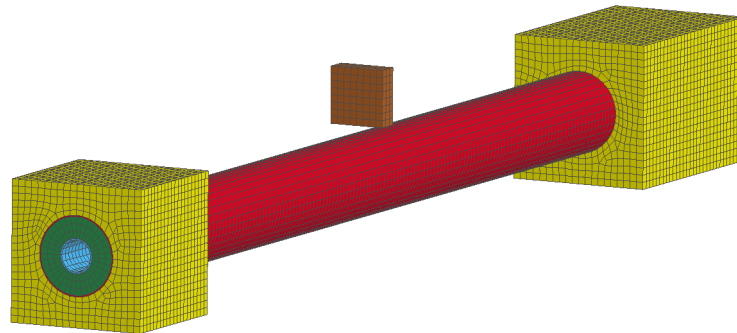


Figure 3. Schematic diagram of finite element model.

In the tests, it was observed that there is no slip between concrete and steel tubes, steel bars and H-shaped beams. Thus, the slip between steel tubes, steel bars and concrete is not considered in the simulation and the fluid–structure interaction (*CONSTRAINED LAGRANGE-IN-SOLID) method is adopted to bond the steel bars and concrete. All other contacts adopt automatic face contact (*CONTACT_AUTOMATIC_SURFACE_TO_SURFACE). The friction factor of the Coulomb friction model was taken as 0.15.

2.3. Materials Properties

Concrete is modelled by continuous cap material (*MAT-CSCM-CONCRETE), which is a cap model with a smooth intersection between the shear yield surface and the hardening cap. The initial damage surface coincides with the yield surface. Rate effects are modelled with viscoplasticity. The material model is widely used in numerical simulation due to its simplified input parameters and good simulation results. The unconfined compressive strength of a cylinder used in the simulation is 32.94 MPa, which is calculated as the average compressive strength of a concrete cube measured on the test day multiplied by 0.79 [29]; this coefficient is caused by the different calculation methods of the compressive strength of concrete in China [24] and the United States [30]. The size of the concrete aggregate is set to 5 mm.

Steel bars and steel plates are modelled by kinematic hardening plasticity with the option of including rate effects (*MAT_PLASTIC_KINEMATIC). This model is suited to model isotropic and kinematic hardening plasticity. The strain rate is accounted for using the Cowper and Symonds model [31] which scales the yield stress with a factor. It is a very cost-effective model and is available for beam (Hughes–Liu and Truss), shell and solid elements. The Young's modulus (E_s), yield strength (F_y), ultimate tensile strength of steel (F_u) and final failure strain (δ) of the material model parameters were obtained from the experimental test listed in Table 2, and the values of strain rate parameters D and p were set as 40.4 and 5, respectively [30].

Table 2. Material information sheet.

Steel Type	Diameter of Steel Bar or Thickness of Steel Plate (mm)	F_y (MPa)	F_u (MPa)	E_s (MPa)	δ
Stirrup	4	603.5	770.5	2.23	0.104
Longitudinal bar	6	529.2	681.1	2.17	0.206
Outer steel tube	2	330.1	396.7	2.21	0.192
Inner steel tube	2.5	330.4	398.4	1.75	0.235
Web of H-shaped steel	2	295.1	356.2	1.95	0.317
Flange of H-shaped steel	3	295.3	362.4	1.92	0.322

The drop hammer is modelled by rigid-body material (* MAT_RIGID). Mechanical properties of materials and degrees of freedom (excluding degrees of freedom in the direction of impact) are constrained. The impact velocity, impact mass and contact size of the drop hammer and specimen are consistent with the test.

3. Validation of the FEA Model

Figure 4 shows a comparison of predicted and observed failure modes for RC columns and composite columns. The residual deformations of the model include the overall flexural deformation, local buckling deformation of the outer steel tube and fractures. A more detailed comparison in terms of forces and displacements is presented in this section.

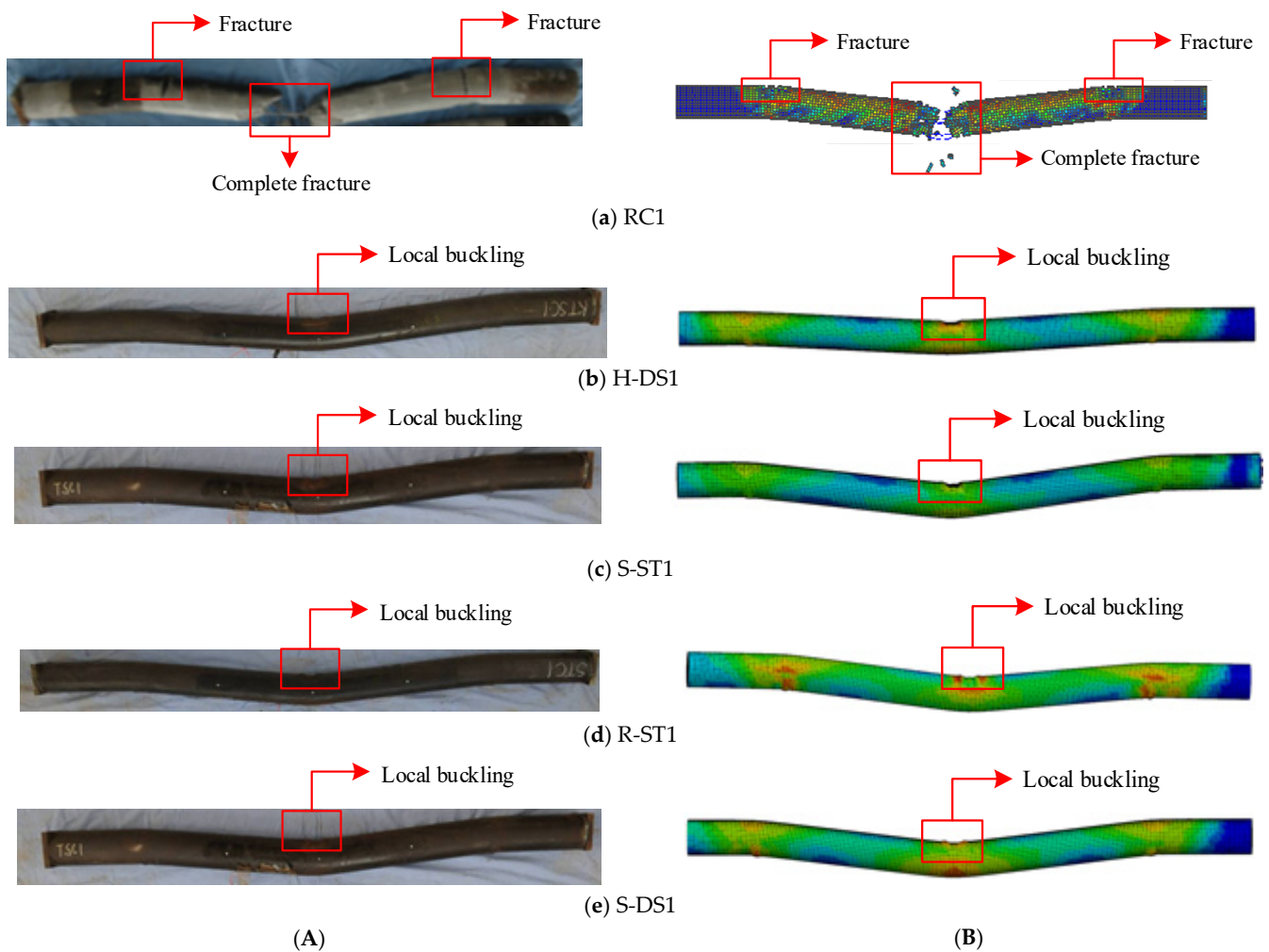


Figure 4. Comparison of observation and prediction of typical failure modes of RC columns and composite columns. (A) Observed in the test and (B) Predicted in FEA.

Figure 5 shows the comparison between the predicted and measured impact force–time histories of five composite column specimens. It can be seen that, except for the RC column, all composite sections fit well with the test results. This is because the RC column suffered severe impact damage and the CSCM concrete material model of LS-DYNA cannot accurately simulate concrete damage.

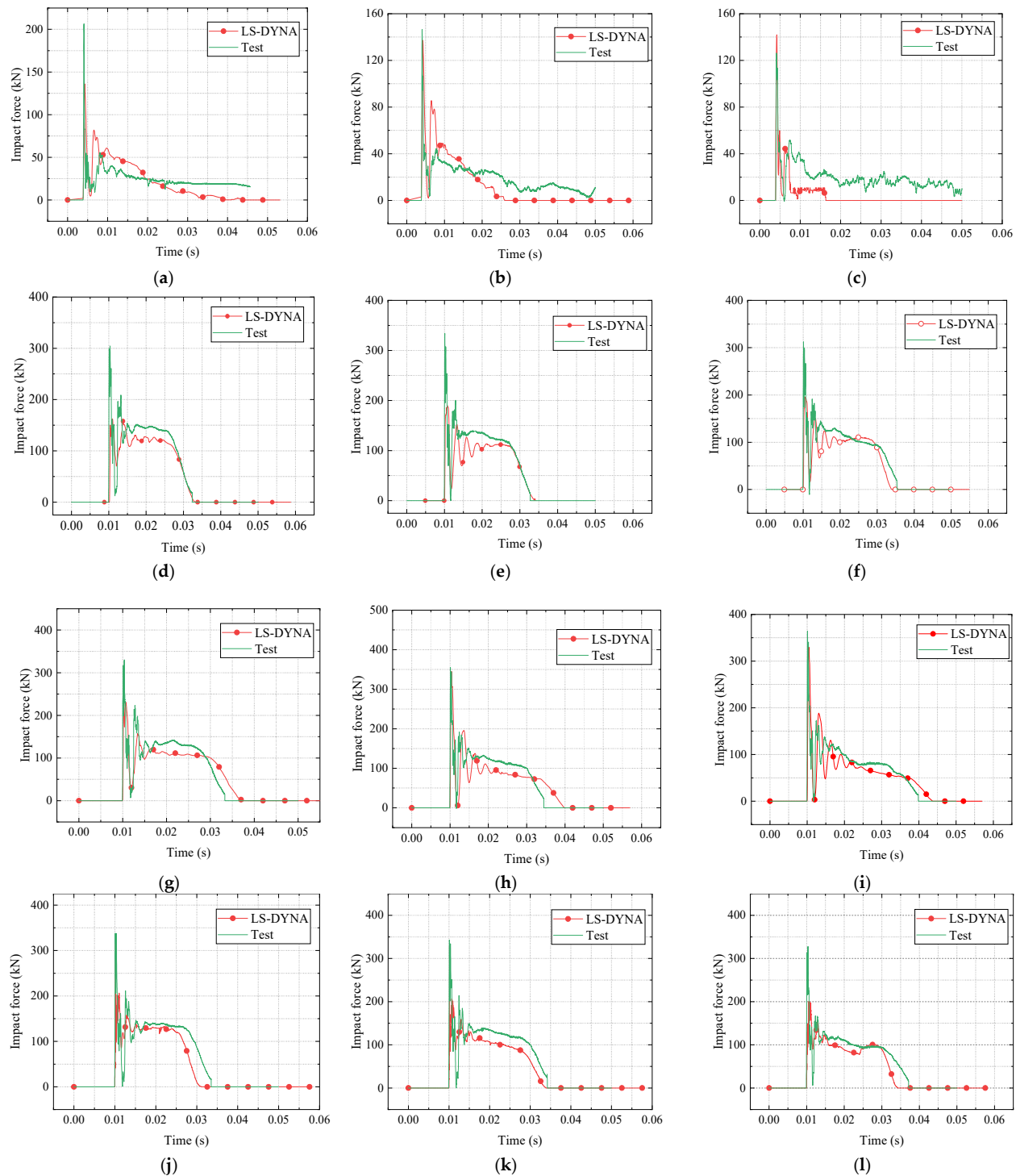


Figure 5. Cont.

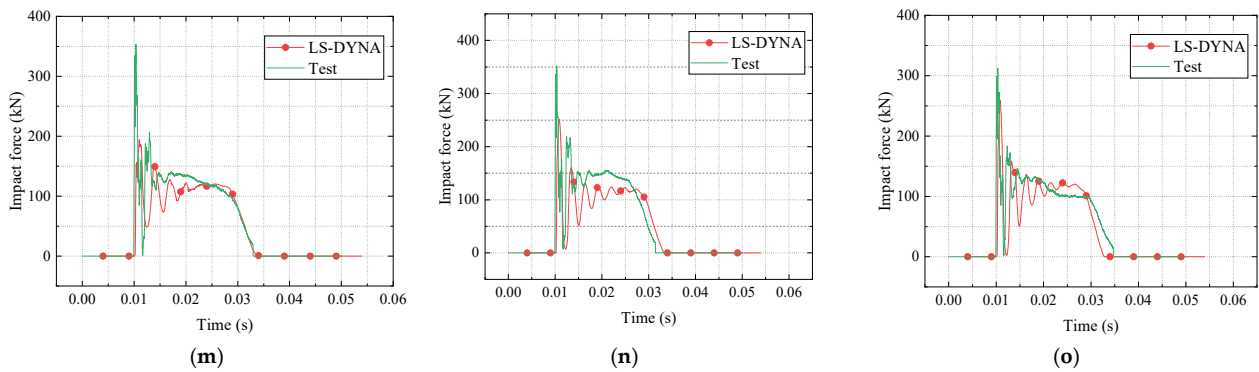


Figure 5. Impact force–time history curve. (a) RC; (b) RC0.3; (c) RC0.5; (d) H-DS; (e) H-DS0.3; (f) H-DS0.5; (g) S-ST; (h) S-ST0.3; (i) S-ST0.5; (j) R-ST; (k) R-ST0.3; (l) R-ST0.5; (m) S-DS; (n) S-DS0.3; (o) S-DS0.5.

Figure 6 shows the comparison of predicted and measured mid-span displacement–time histories of five column specimens with different cross sections. It can be seen that, consistent with the impact force–time history curves, the mid-span displacement of the simulated RC column deviates from the test result due to the limited capacity in simulating severe concrete damage.

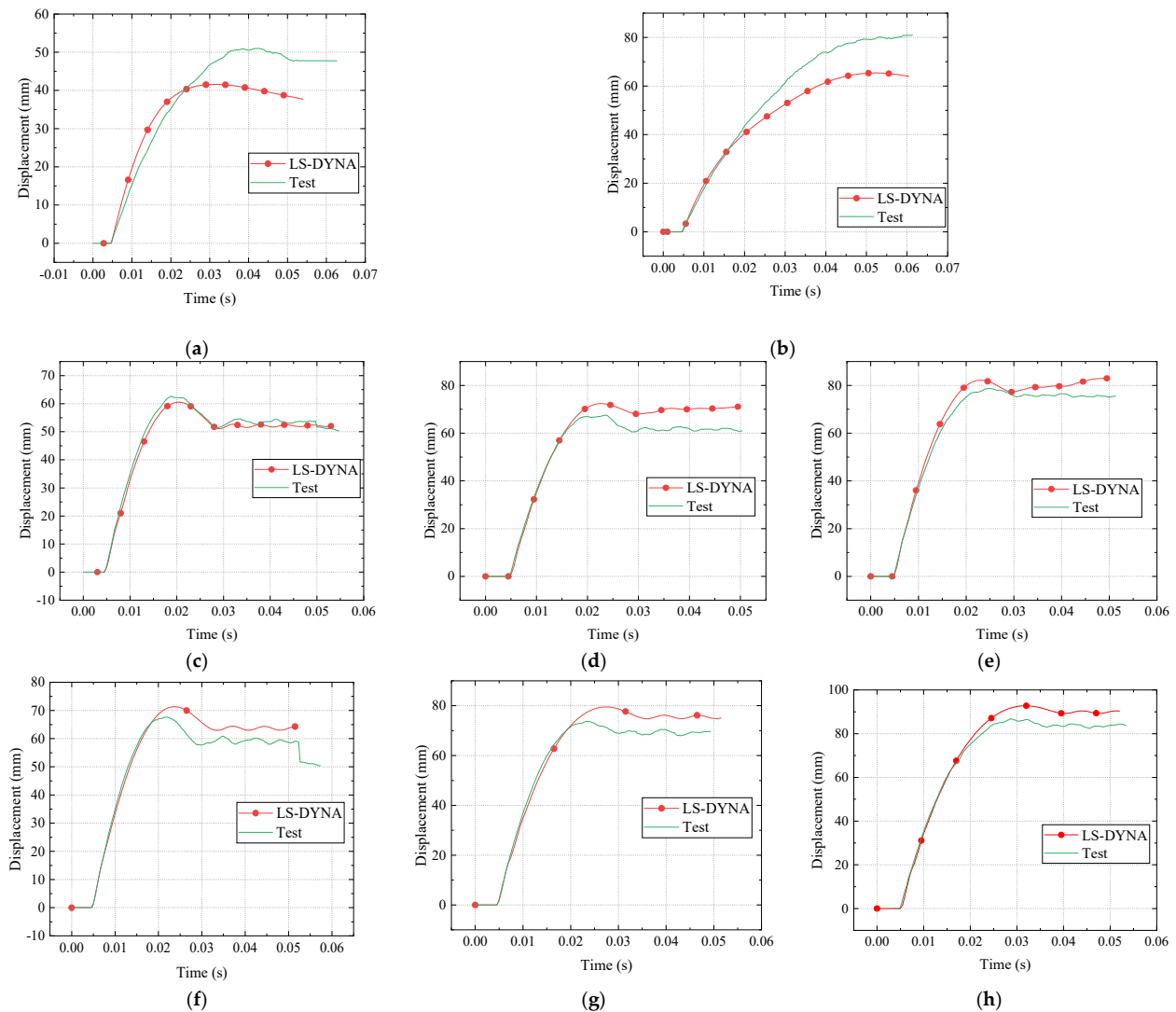


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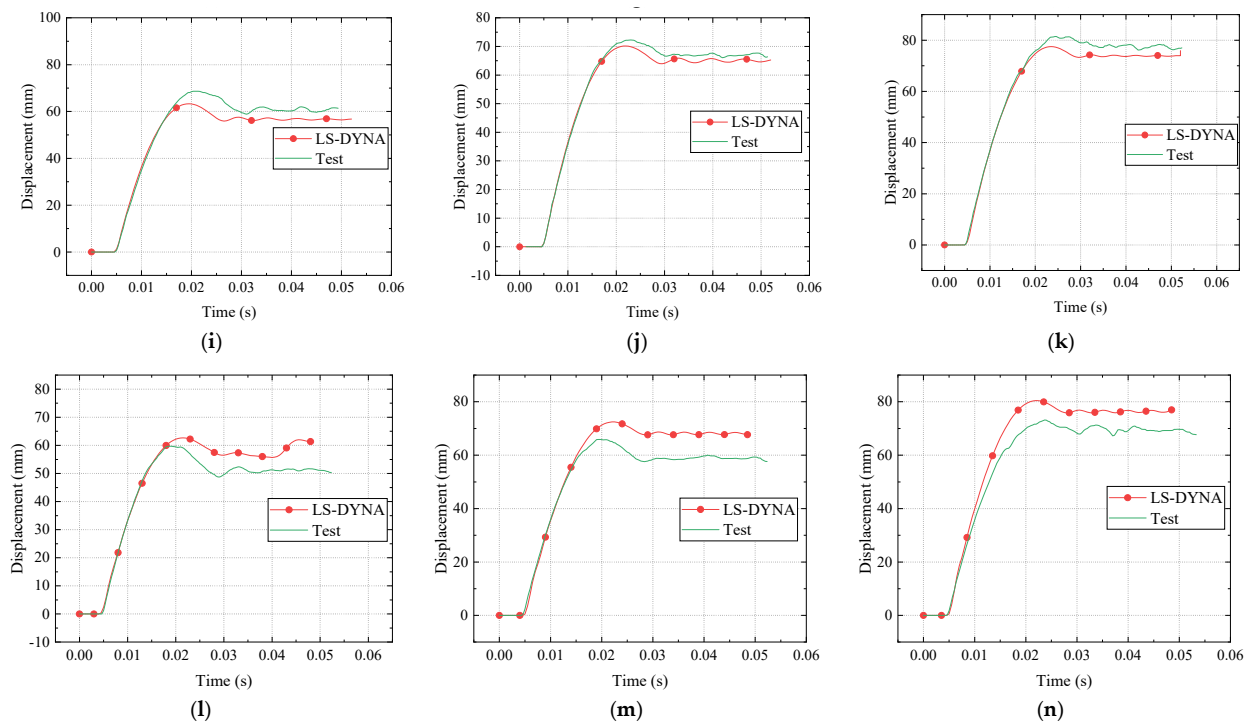


Figure 6. Mid-span deflection–time history curve. (a) RC; (b) RC0.3; (c) H-DS; (d) H-DS0.3; (e) H-DS0.5; (f) S-ST; (g) S-ST0.3; (h) S-ST0.5; (i) R-ST; (j) R-ST0.3; (k) R-ST0.5; (l) S-DS; (m) S-DS0.3; (n) S-DS0.5.

It can be seen from Figures 5 and 6 that in these composite columns, the existence of axial force reduces the impact resistance of specimens within the studied ranges. S-DS columns provide better impact-resistant performances; this is because the existence of the internal and external steel tubes enhances the effect on concrete constraints, thus enhancing the impact resistance of the component.

Figure 7 shows the difference between the numerical simulation and experimental results under various conditions. It can be seen that the difference between most numerical simulation results and experimental results is within 15%. Thus, a good agreement is generally achieved between the predictions and the test results. In Figure 7, F_{stab} is the average of the impact force in the platform stage, t is the time from impact initiation to complete unloading and δ is the global residual deformation at mid-span.

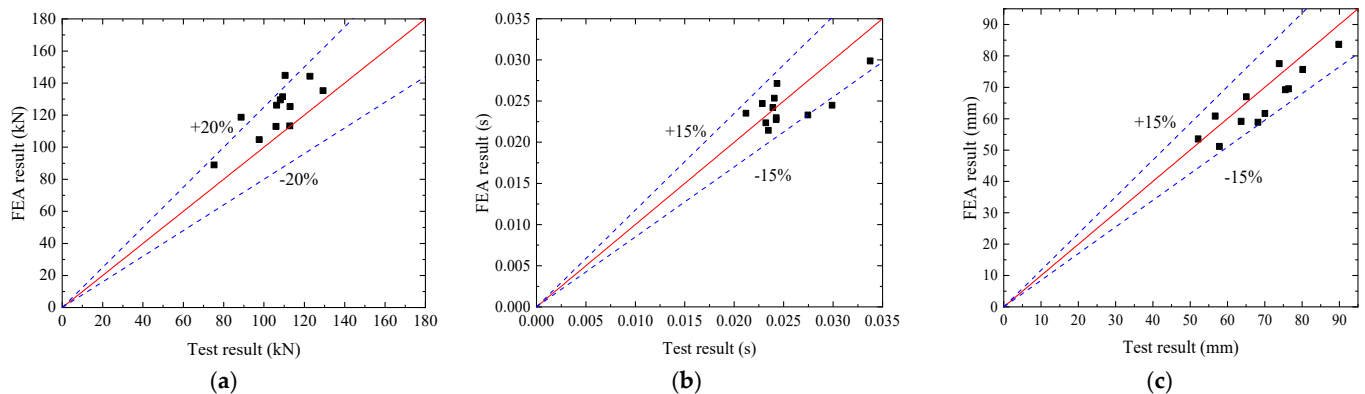


Figure 7. Comparisons between the plateau values (F_{stab}), load durations (t) of the impact forces and the mid-span deflections (Δ). (a) Plateau value (F_{stab}); (b) Load duration (t); (c) Mid-span deflection (Δ).

4. Impact Mechanism Analysis

4.1. The Impact Process

Figure 8 shows impact force (F), mid-span deflection (Δ), drop hammer impact velocity (V_0) and component mid-span velocity (V_m)–time histories. In order to facilitate the study of the characteristics of the impact process, the curves are normalized to be dimensionless by F/F_{max} , Δ/Δ_{max} , V_0/V_{0max} and V_i/V_{imax} , respectively. The drop hammer velocity (V_0) and specimen velocity (V_i) are negative in the downward direction. Similarly, deflection (Δ) is defined as positive in the downward direction. Although the sections of the specimens were different and the axial forces applied were different, the impact force–time history curves of the specimens had similar stages throughout the impact, which are: peak stage, plateau stage and attenuation stage. Due to the limitation of article length, this paper only presents the normalized time history curves of each specimen without axial force. The stages are summarized as follows:

- (1) Peak stage (OA): When the drop hammer touches the specimen, the impact force increases rapidly from zero to the peak value. The specimen demonstrates a downward velocity which rapidly increases to the peak value, which is equivalent to the impact velocity. At the same time, the velocity of the drop hammer decreases. During this stage, a small lateral displacement occurs in the specimen;
- (2) Plateau stage (AB): After the impact force reaches its peak, the specimen velocity decreases to a certain extent, and the velocity of the drop hammer fluctuates slightly. It is the difference in the two velocities that causes the sharp decrease in the impact force. Then, the drop hammer increases the specimen velocity again, and the corresponding impact force also increases after several fluctuations. After several fluctuations, the value remains stable. Subsequently, the drop hammer velocity stays consistent with the mid-span velocity of the specimen, and gradually decreases to zero;
- (3) Attenuation stage (BC): At this stage, the drop hammer and the specimen begin to rebound, and the rebound velocity of the drop hammer is greater than that of the specimen, resulting in a decline in the impact force. When the specimen is separated from the drop hammer, the impact force drops to zero. After the specimen reaches the maximum mid-span displacement, the mid-span displacement of the specimen begins to decrease due to the rebound. With the recovery of elastic displacement, the mid-span displacement of the specimen remains stable and recovers to the residual deflection.

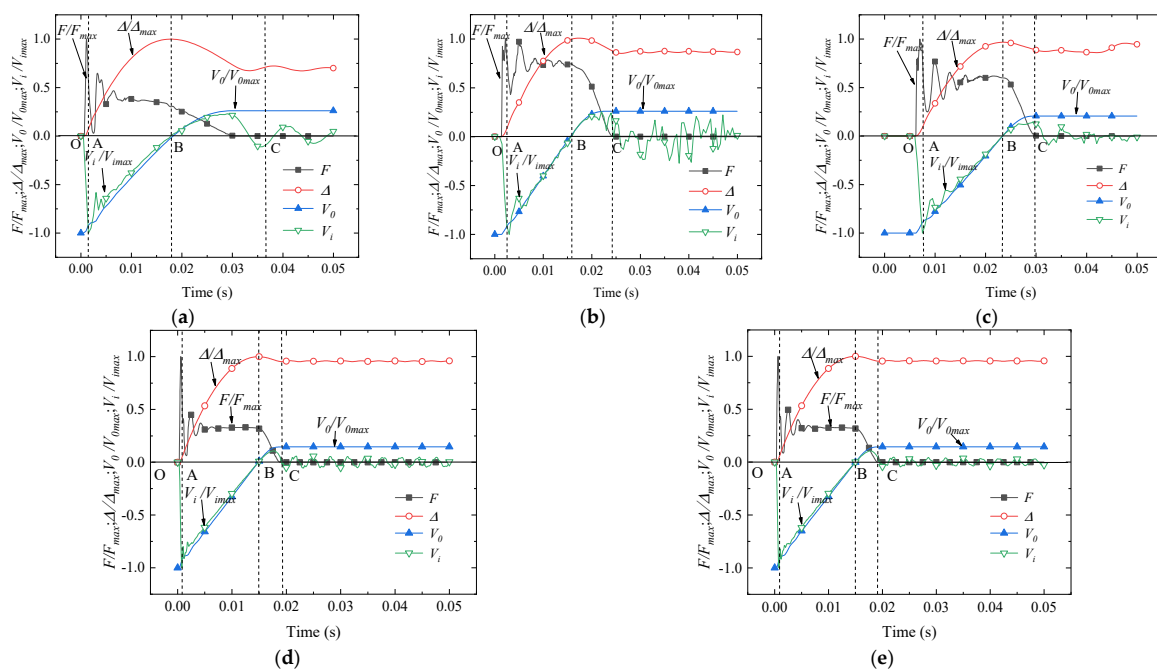


Figure 8. Normalized time history curve. (a) RC; (b) H-DS; (c) S-DS; (d) S-ST; (e) R-ST.

4.2. Deformation Mode

Figure 9 shows the equivalent plastic strain of RC, H-DS, S-DS, S-ST and R-ST components impacted by a drop hammer. The red area indicates high plastic strain. It can be seen from the figure that plastic deformation mainly occurs at the impact location and the support locations where there are maximum bending moments. The maximum deformation position of each component is similar, appearing in the middle and lower part of the span and the upper part of the support. The failure modes of internal and external steel tubes and concrete are similar.

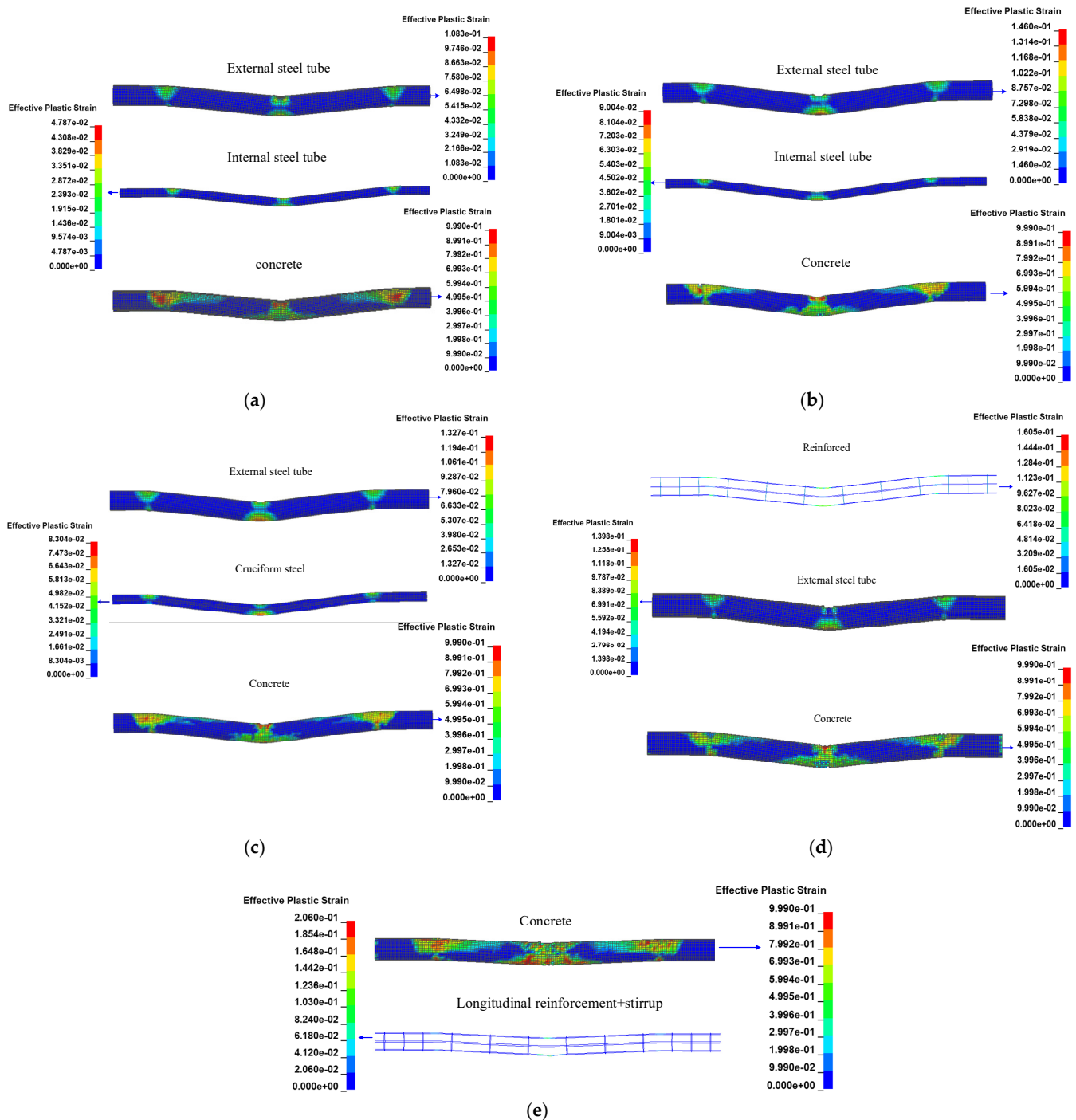


Figure 9. Plastic strain of members of different cross sections. (a) H-DS; (b) S-DS; (c) S-ST; (d) R-ST; (e) RC.

When comparing the different sections, the RC member has the most severe damage with excessive deformation which indicates that the impact resistance of RC columns is limited. For the H-DS member, it was found that the plastic strain of the inner steel tube is much smaller than that of the outer steel tube, which indicates that the energy absorbed by the inner steel tube is smaller than that of the outer steel tube. The maximum plastic strain of the outer steel tube is much smaller than that of the RC column, and the failure zone of concrete is also obviously smaller than that of the RC column, which indicates that the impact resistance of H-DS columns is better than that of RC columns. For S-DS columns, maximum plastic strains of outer and inner steel tubes are increased. For the S-ST member, the maximum plastic strain of the outer steel tube is greater than that of the double H-shaped steel, mainly because the double H-shaped steel is closer to the neutral axis, resulting in a slightly lower maximum plastic strain than that of the outer steel tube. For the R-ST member, the maximum plastic strain of steel bars is greatly improved compared with that of RC columns, which is mainly because the plastic deformation of the external steel tube is the main energy dissipation mechanism, making the deformation of steel bars and concrete smaller than that of RC columns.

4.3. Energy Dissipation

Figure 10 shows the internal energy–time history curves of RC, H-DS, S-DS, S-ST and R-ST components. As can be seen from Figure 10, for RC members, the energy absorbed by concrete is slightly greater than that absorbed by steel bars in the early stage of impact, mainly because at this time, concrete produces a large number of cracks due to impact and, thus, absorbs a large amount of energy. In the post-impact period, due to the concrete crack failure resulting in its main energy being absorbed by steel bars, at this time, the energy absorbed by steel bars is far greater than the energy absorbed by concrete. Due to the bending deformation of the specimen, the role of the stirrup is limited and it can hardly absorb energy. For H-DS and S-DS members, their deformation is nearly the same; the energy absorbed is similar. The main energy is absorbed by steel tubes, of which the outer steel tube absorbs most of the energy, about 4.7 kJ (52.24% of the impact energy), followed by concrete, about 2.1 kJ (22.95% of the impact energy), and the inner steel tube absorbs the least energy, about 1.0 kJ (10.93% of the impact energy). For S-ST members, most of the energy is absorbed by the external steel tube, about 5.43 kJ (59.34% of the impact energy), followed by concrete, about 2.23 kJ (24.37% of the impact energy), and the energy absorbed by double H-shaped steel is the lowest, about 1.11 kJ (12.13% of the impact energy). For R-ST members, the main energy dissipation is still absorbed by the external steel tube, about 5.43 kJ (59.34% of the impact energy), followed by concrete, about 2.19 kJ (23.99% of the impact energy), then the energy absorbed by longitudinal reinforcement, about 1.0 kJ (10.93% of the impact energy), and finally the energy absorbed by stirrups, about 2.75 J (0.003% of the impact energy).

Based on the above analysis, the amounts of energy absorbed by steel bars and concrete are roughly equal in the early stage of RC members, and when concrete cracks, steel bars become the main energy dissipation mechanism. For R-ST members and other composite columns, the plastic deformation of external steel tubes is the main energy dissipation mechanism, followed by concrete, and the energy absorption of internal steel tubes, steel bars and double H-beams is the lowest.

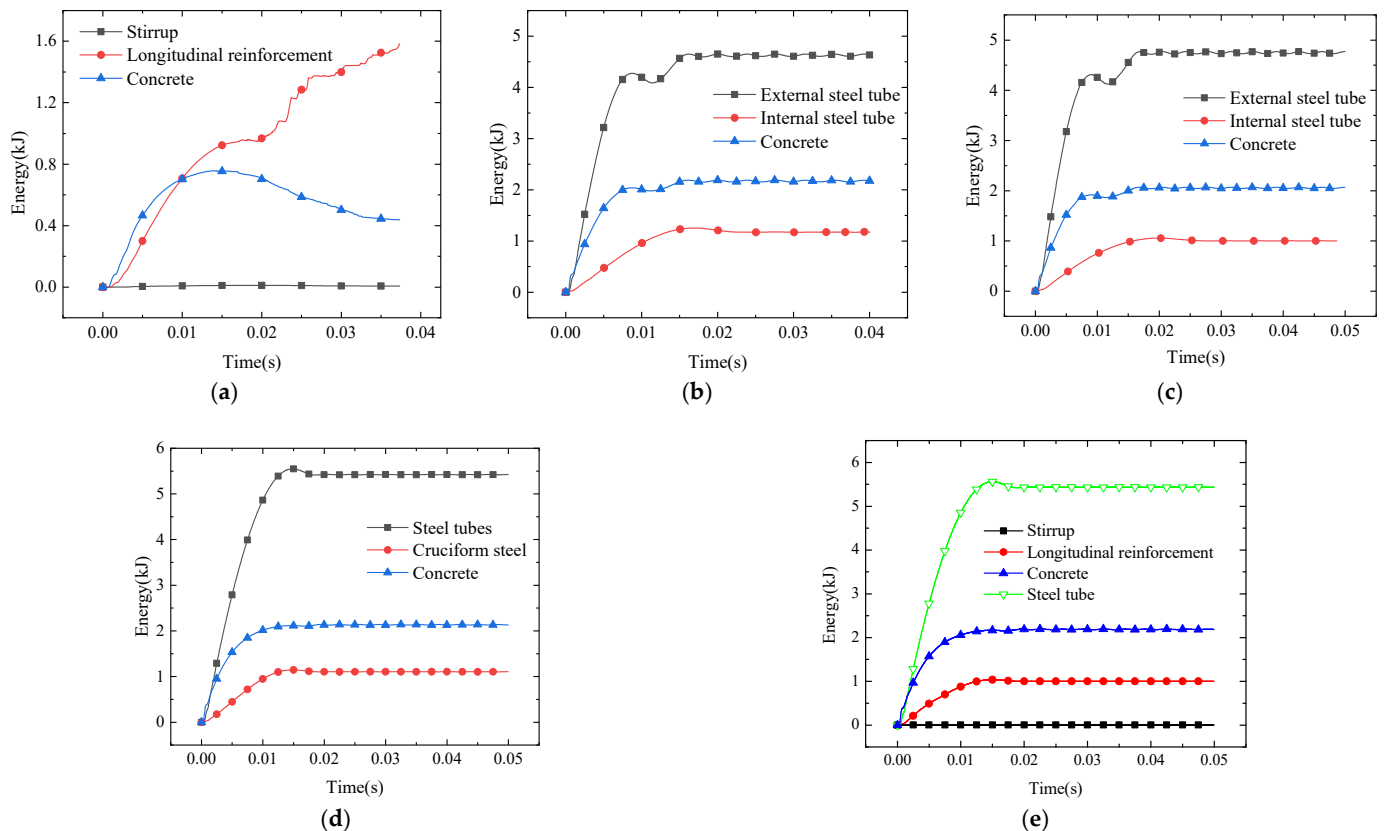


Figure 10. Energy–time history curve. (a) RC; (b) H-DS; (c) S-DS; (d) S-ST; (e) R-ST.

5. Parametric Sensitivity Analysis

Following the experimental test, a number of parameters are altered by a percentage to investigate the sensitivity. In the test, the mass is 206.65 kg, the velocity is 9.39 m/s, the compressive strength of concrete is 41.69 MPa, the yield strength of outer steel tubes is 330.1 MPa, the yield strength of inner steel tubes is 330.4 MPa, and the yield strength of steel bars is 529.2 MPa. The yield strength of double H-shaped steel is 295.2 MPa, the diameter–thickness ratio of outer steel tubes is 57, the diameter–thickness ratio of inner steel tubes is 20, the diameter of the longitudinal steel bar is 6 mm and the flange and web width–thickness ratios of double H-shaped steel are 4.67 and 12.25, respectively. These parameters are changed by -40% , -20% , $+20\%$ and $+40\%$, and the corresponding changes in impact peak value, impact duration and deflection peak value are compared to conduct sensitivity analysis.

Figure 11 shows the influence of various parameters on the peak impact force, impact duration and deflection peak of H-DS members. The compressive strength of concrete, the diameter–thickness ratios of outer steel tubes and inner steel tubes and the impact velocity have a great influence on the peak impact force. With an increase in the compressive strength of concrete, the peak impact force increases. The diameter–thickness ratio of outer steel tubes and inner steel tubes is negatively correlated with the peak impact force; the greater the diameter-to-thickness ratio, the smaller the peak force. Impact velocity has the greatest influence in the decreasing region, but fluctuates in the increasing region. The impact mass and yield strength of outer and inner steel tubes have little effect on the impact peak force.

The diameter–thickness ratio, impact mass and impact velocity of the outer and inner steel tubes have a certain influence on the impact duration. With an increase in the steel tube diameter–thickness ratio, the wall thickness of steel tubes decreases, and bending stiffness and local stiffness are reduced, resulting in reduced impact force. According to the theorem of momentum, it is known that impact duration increases. The concrete strength

and the yield strength of outer and inner steel tubes have the least influence on the impact duration. With an increase in the yield strength of steel tubes, the impact plateau value of H-DS columns increases, and the impact duration decreases.

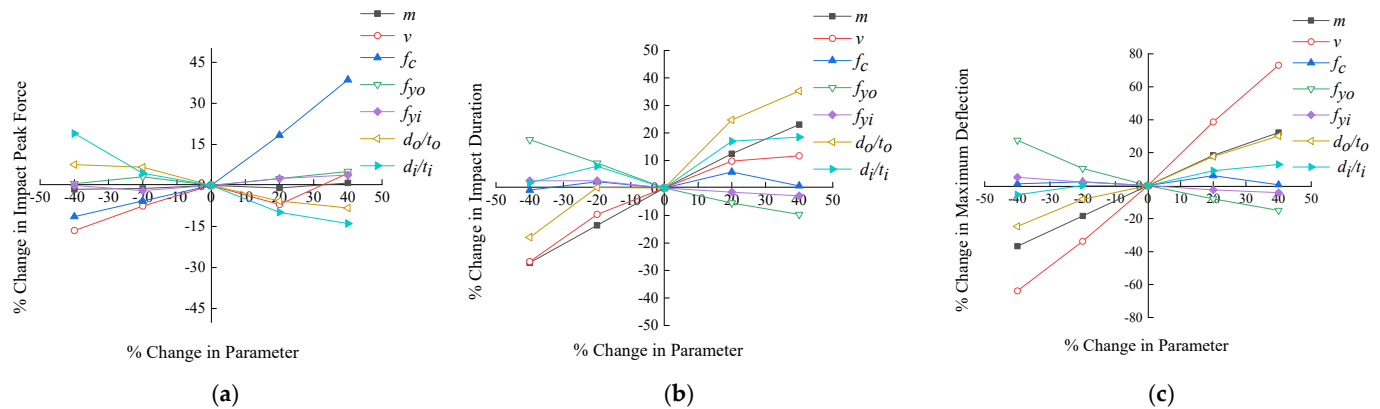


Figure 11. Sensitivity of H-DS corresponding to changes in structure-related variables. (a) Peak impact force; (b) Impact duration; (c) Maximum deflection.

Impact velocity has the greatest influence on the peak deflection, followed by impact mass and diameter–thickness ratio of the outer steel tube, diameter–thickness ratio of the inner steel tube, compressive strength of concrete and yield strength of outer and inner steel tubes. This is mainly because the impact energy is proportional to the quadratic power of velocity, so a larger displacement occurs. Changing the diameter–thickness ratio of the outer steel tube also has a great influence on the peak deflection, which is mainly because the bending stiffness of the structural column changes with an increase or decrease in outer steel tube wall thickness, while the inner steel tube has a small influence on the deflection due to the small moment of inertia. Change in other parameters do not effectively affect the bending stiffness of the structure column, so the influence on impact response is small.

From Figure 12, for S-DS columns, the compressive strength and impact velocity of concrete have the greatest influence on the peak impact force; the outer steel tube, the diameter–thickness ratio of the inner steel tube and the compressive strength of concrete have the greatest influence on the impact duration; and the impact velocity, impact mass and the diameter–thickness ratio of the outer steel tube have the greatest influence on the peak deflection.

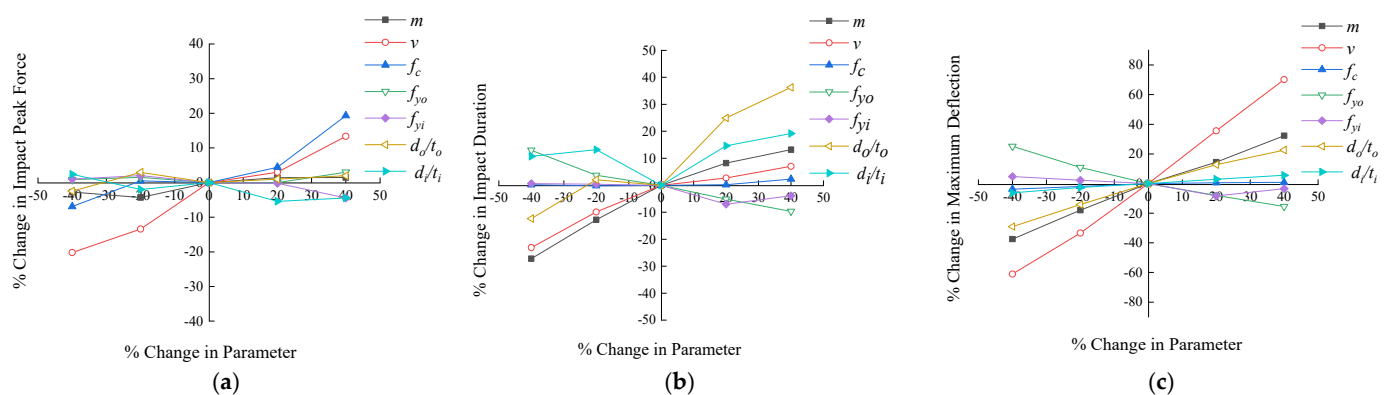


Figure 12. Sensitivity of S-DS corresponding to changes in structure-related variables. (a) Peak impact force; (b) Impact duration; (c) Maximum deflection.

Figure 13 shows the influence of various parameters of S-ST columns on impact performance. As can be seen from the figure, the impact velocity and diameter–thickness

ratio of the outer steel tube have greater influences on peak impact value, concrete's compressive strength and double H-shaped steel have slightly greater influences, while steel's yield strength and impact mass have the least influence. This is because the peak impact force is mainly related to the impact velocity and local stiffness. The most effective way to increase the peak impact force is to increase the impact velocity and the stiffness of the external steel tube in direct contact with the drop hammer. Concrete's compressive strength also has a certain influence on the local stiffness. For impact duration, the impact mass and diameter–thickness ratio of the outer steel tube have a major influence, while the yield strength and width–thickness ratio of double H-shaped steel have the least influence; this is because bending stiffness is the most important factor affecting impact endurance. Because the H-shaped section's moment of inertia is small, it is difficult to change the bending stiffness of the section; therefore, it has little effect on impact duration.

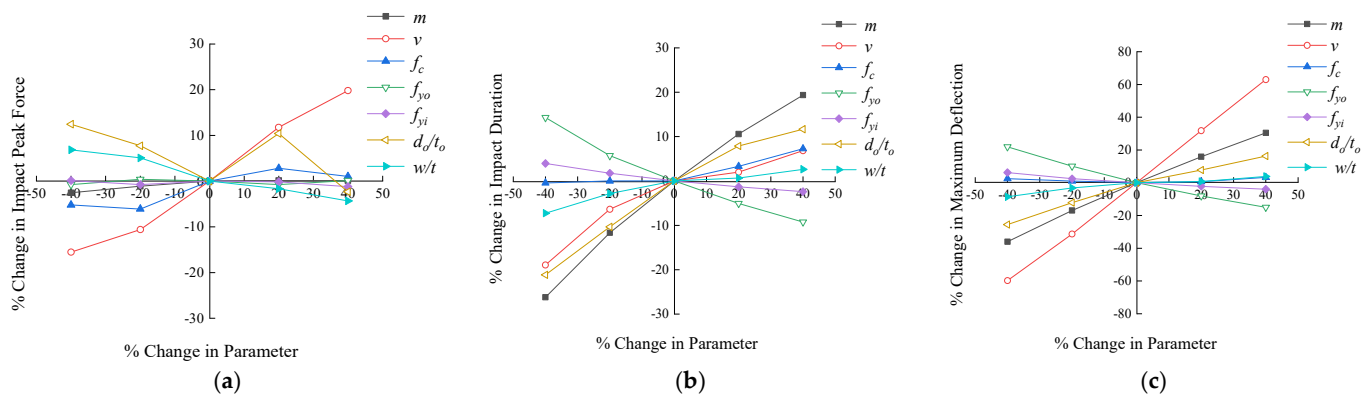


Figure 13. Sensitivity of S-ST corresponding to changes in structure-related variables. (a) Peak impact force; (b) Impact duration; (c) Maximum deflection.

Figure 14 shows the changes in energy dissipation of R-ST columns under various parameter changes. As can be seen from the figure, for the peak impact force, the factors that have a greater impact are the diameter–thickness ratio of the outer steel tube and the impact velocity, while other factors have a lesser impact, which is the same as the influence factors of the other three components. For impact duration, the most influential factors are the diameter–thickness ratio of the outer steel tube, impact velocity and impact mass, followed by the diameter of the steel bar, concrete's compressive strength and yield strength of steel bar. For the peak deflection value, the impact velocity and the diameter–thickness ratio of the external steel tube have the greatest influence, followed by the impact mass and the diameter of the steel bar, and the compressive strength of concrete and the yield strength of the steel bar have the least influence.

To summarize, the impact velocity and the diameter–thickness ratio of the outer steel tube have a greater influence on the energy dissipation of the structural column regardless of the cross section, while the influence of the inner steel tube and the double H-beam is the least. The impact velocity mainly increases the impact energy, resulting in greater impact force and impact displacement. The diameter–thickness ratio of the outer steel tube mainly changes the bending stiffness of the section of the structure. Because the outer steel tube is arranged at the outermost part of the section, it provides a larger moment of inertia, thus improving the bending performance of the specimen. Through the above analysis, it can be found that in terms of improving the flexural stiffness of the outer steel tube, changing the diameter–thickness ratio is obviously more effective than changing the yield strength of the component. When the compressive strength of concrete is increased, the interaction with the outer steel tube is strengthened and buckling of the outer steel tube can be prevented. Because the inner steel tube and the double H-shaped steel are closer to the neutral axis, the deformation is smaller than that of the outer steel tube, so the impact deformation cannot be completely deformed, and the impact on the bending stiffness of the section is

less. Therefore, the impact energy dissipation parameters of the specimen are not sensitive to the parameters related to the inner steel tube being near the neutral axis and the double H-beam steel.

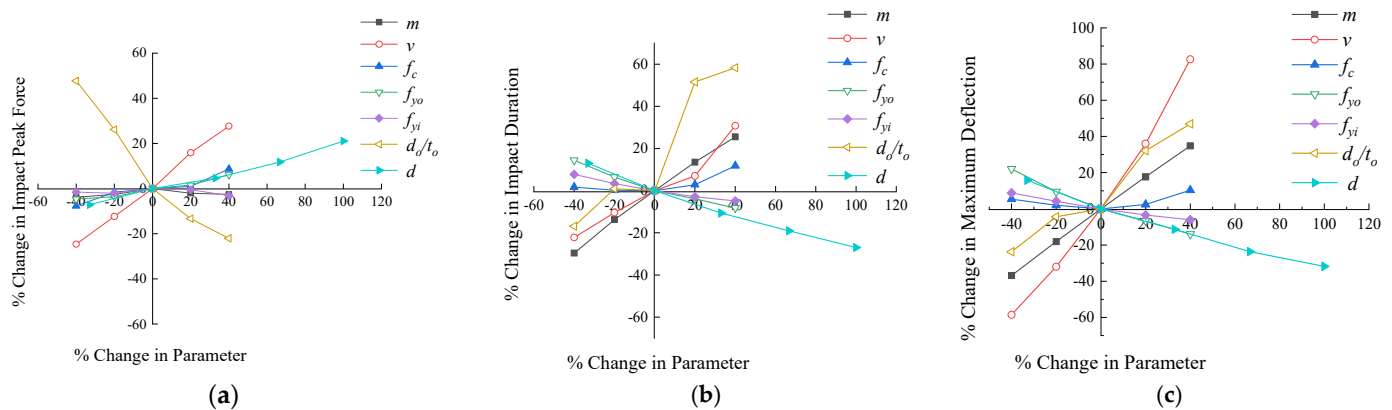


Figure 14. Sensitivity of R-ST corresponding changes to change in structure-related variables. (a) Peak impact force; (b) Impact duration; (c) Maximum deflection.

6. Conclusions

In this paper, the impact resistance of reinforced concrete columns and composite columns is studied on the basis of numerical simulation. Within the scope of parameter research in this paper, the following conclusions are drawn:

- (1) The impact resistance of composite columns is better than that of RC columns. The axial force weakens the impact resistance of the composite column regardless of its cross-sectional form;
- (2) In all composite columns, the impact process is similar, and the impact force–time curve shows three stages: peak stage, plateau stage and attenuation stage. Among them, S-DS columns have the best impact resistance, showing a higher impact platform value and a smaller peak deflection under an impact with the same energy;
- (3) The simulation results of LS-DYNA fit well with the experimental results. The software is used to analyze the impact process of composite columns, and it is found that the outer steel tube is the main energy dissipation component of these components, which can absorb 50%–60% of impact energy, followed by concrete (about 24% of impact energy), the inner steel tube and the double H-shaped steel (about 12% of impact energy);
- (4) LS-DYNA nonlinear finite element software was used to analyze the sensitivity of composite columns. Although the cross-sectional forms of different composite columns were different, the sensitivities of peak impact force, impact duration and peak deflection to parameter changes were similar. The impact velocity and diameter–thickness ratio of the outer steel tube have a great influence on the energy dissipation of the structural column, while the influence of the inner steel tube and the double H-shaped steel is the least, and the influence of concrete is in the middle;
- (5) To increase the impact resistance, the best solution is to increase the thickness of the outer steel tube, followed by the yield strength of the outer steel tube and the compressive strength of concrete, and the least economic solution is to improve the impact resistance of components through the mechanical parameters of the inner steel tube.

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Article

Experimental Study on Shear Performance of Post-Tensioning Prestressed Concrete Beams with Locally Corroded Steel Strands

Rihua Yang ^{1,2}, Yiming Yang ^{1,*}, Peng Liu ^{3,*} and Xinzhong Wang ¹

¹ School of Civil Engineering, Hunan City University, Yiyang 413000, China

² Hunan Engineering Research Center of Structural Safety and Disaster Prevention for Urban Underground Infrastructure, Hunan City University, Yiyang 413000, China

³ School of Civil Engineering, Central South University, Changsha 410013, China

* Correspondence: yangyiming@hncu.edu.cn (Y.Y.); 2015038@csu.edu.cn (P.L.)

Abstract: To study the effect of the local corrosion of prestressed steel strands on the shear failure mode and shear bearing capacity of concrete beams, unilateral steel strands in four post-tensioning prestressed concrete (PC) beams are corroded, and the shear test of four PC beams are performed. Moreover, a simplified calculation method for the shear bearing capacity of concrete beams with diagonal steel bars is proposed considering the effect of cross-sectional reduction of steel bars, the degradation of mechanical properties, and the cross-sectional damage of concrete. Results shows that the crack propagation mode and failure mode are unrelated to the corrosion of prestressed steel bars when the shear span ratio of beam is the same. The shear capacity of the beam decreases with the increase of corrosion rate, but the decreasing rate is lower than the increasing rate of the corrosion rate. The growth rate of stirrup stress is much greater than that of load after concrete tension and compression loss cracking, and the yield of stirrup can be used as a sign of the ultimate bearing capacity of the beam. In addition, by comparing the experimental and numerical simulation results, the proposed simplified calculation method for the shear bearing capacity of concrete beams is of high accuracy.

Keywords: prestressed concrete beam; shear bearing capacity; corrosion; numerical simulation

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1. Introduction

Post-tensioning and pre-tensioning are the two most commonly used technologies for tensioning prestressed steel tendons in concrete structures [1]. Compared with pre-tensioning PC structures, post-tensioning PC structures have better structural performance and are more suitable for the curved arrangement of steel tendons in long-span structures [2]. As a result, post-tensioning PC structures has become the most widely used structural form in long-span bridge construction. However, with the increase of service time, various diseases such as corrosion of steel tendons, reduction of the effective area of steel tendons, and shedding of protective layers gradually appear [3–5]. This will not only lead to the degradation of structural safety performance but also reduce the normal service performance of the bridge [6,7]. Compared with bending failure, the shear failure of structure is mostly brittle, and there are almost no predictable symptoms. This means that the damage caused by the shear failure of corroded PC components is more serious [8,9]. Therefore, it is very important to study the degradation mechanism of shear mechanical properties of corroded PC structure and reasonably evaluate the residual shear capacity.

At present, some scholars have carried out research on the shear performance of corroded concrete structures, but they mainly focus on ordinary reinforced concrete (RC) specimens. For example, based on the bearing capacity test results of corroded reinforced concrete specimens, Ye [10] found that the structural shear strength decreased gradually

with the increase of the corrosion degree of stirrups. Zhang [11] studied the corrosion-induced spalling of cover concrete and its effects on shear behavior of RC beams, and the results showed that both the concrete damage and bond degradation do not significantly affect the shear resistance of reinforced concrete beams at a small degree of corrosion. Based on the experimental and numerical analysis results, Huang [12] believed that the shear resistance of corroded RC beams in terms of initial stiffness, cracking load, ultimate bearing capacity, post-peak behavior and energy dissipation capacity is significantly degraded, and this degradation becomes more obvious with the increase of corrosion degree. Lu [13] studied the influence of different concrete strength grades and stirrup arrangement on the shear strength of RC structure, and found that the shear strength of structures is more sensitive to the change of stirrup yield strength. Compared with the RC structure, the stress characteristics of the PC structure and the elements that provide shear resistance inside the structure are significantly different, which leads to a more complex variation law of the shear resistance of the PC structure. Therefore, it is necessary to study the shear resistance of corroded PC structures.

To reasonably evaluate the shear performance of concrete structures, some scholars have proposed a calculation model of the shear capacity of corroded concrete members based on the experimental and theoretical analysis results. By refining the shear design model provided by American Concrete Institute (ACI), Soltani [14] proposed a model to estimate the reduced shear capacity for RC beams. Based on the modified compression field theory, Spinella [15] proposed a calculation model of the ultimate shear force of reinforced concrete beams with corroded stirrups. El-Sayed [16] presented a proposed design procedure for quantifying the residual shear strength of RC beams with corrosion-damaged stirrups. Da [17] developed a calculation model for the ultimate shear capacity of reinforced coral aggregate concrete beam by considering the influence of high-strength coral aggregate concrete and stirrup corrosion. Although the above models provide meaningful references for reasonably evaluating the degradation of shear capacity of corroded concrete structures, most of these models are proposed based on the assumption of uniform corrosion. In service concrete structures, the spatial variability of material properties, structural dimensions, and environmental parameters as well as grouting defects lead to uneven corrosion of steel strands, which makes the existing shear capacity calculation model no longer applicable. Therefore, it is necessary to further propose a shear capacity model that considers the effect of local corrosion of steel strands.

The main purpose of this paper is to study the change law of the shear capacity of PC structure considering the effect of local corrosion and to propose the corresponding theoretical calculation model. First, the crack development, failure mode, deflection change and ultimate bearing capacity development law are analyzed based on local corrosion test of PC member bearing capacity. Second, a simplified calculation model of shear capacity is proposed considering the effect of local corrosion of steel strands. Finally, the shear resistance of locally corroded PC structures is studied based on numerical simulations. Among them, studying the effect of local corrosion on the shear capacity of PC structures and proposing a calculation method of shear performance of PC structures considering the effect of local corrosion are the novelties of this paper.

2. Test Profile

A total of four post-tensioning PC beams, numbered as B1, B2, B3, and B4, are designed in this experiment. The test beams are 3000 mm in length and 160 mm × 450 mm in cross-sectional dimensions. The thickness of concrete cover is 30 mm. Prestressed tendons are made of seven wire twisted steel strands with a nominal diameter of 15.2 mm, and the controlled tensile stress is 1395 MPa. In addition, the 2Φ25 HRB400 reinforcing steel bars is used as ordinary tensile bars, 2Φ10 HRB400 steel bars are used as erection bars, and Φ6 HRB400 steel bars are selected as stirrups. The average measured cubic compressive strength of concrete is 42.2 MPa. The cross-sectional dimensions of test beams and the

concrete layout of steel bars are shown in Figure 1, and the mechanical property parameters of ordinary and prestressed steel bars are presented in Table 1.

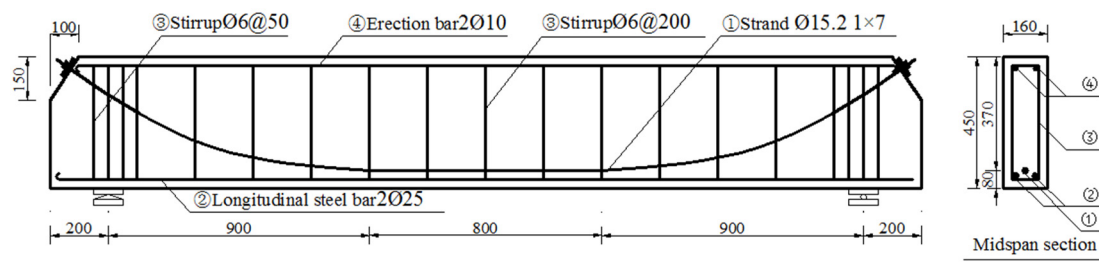


Figure 1. Dimensions and reinforcement of test beams (unit: mm).

Table 1. Mechanical parameters of steel strands and ordinary steel bars.

Type	Diameter/mm	Elastic Modulus/GPa GPAGPa	Yield Strength/MPa	Ultimate Strength/MPa
Steel strand	15.2	195	1412.4	1876.3
Ribbed steel bar (HRB400)	6	200	412.5	561.2
Ribbed steel bar (HRB400)	10	200	403.9	551.6
Ribbed steel bar (HRB400)	25	200	424.1	574.8

Among these test beams, the B1 beam is not corroded, while the bending-shear segment at the side of B2 to B4 beams are corroded by electrochemical rapid corrosion method. In this experiment, the 5% NaCl solution is adopted. After completing the rapid corrosion, the static load tests of all beams are carried out, as shown in Figure 2. As a result, the spacing between center lines of supports for simply supported beams is 2.6 m, the pure bending segment is 1.0 m, and the shear span ratio is 2.1. Here, the multi-stage loading method is implemented through a jack. The loading level of the test beam before cracking is 5 kN, and the loading level after cracking is adjusted to 10 kN. The loading rate above two processes are 5 kN/min. When the actual loading value is close to the ultimate load, the loading level is adjusted to 5 kN, and the loading rate at this time is set to 2 kN/min. Here, the loading is stopped when the concrete is crushed or unstable deformation occurs, and then the structure is considered to be damaged at this time. Finally, after the static load test, the corroded steel bars inside the structure were taken out, and the corrosion rate of steel strands is determined by using the relatively accurate sectional profile method [18]. The corrosion time and corrosion rate of steel strands in different beams are presented in Table 2.

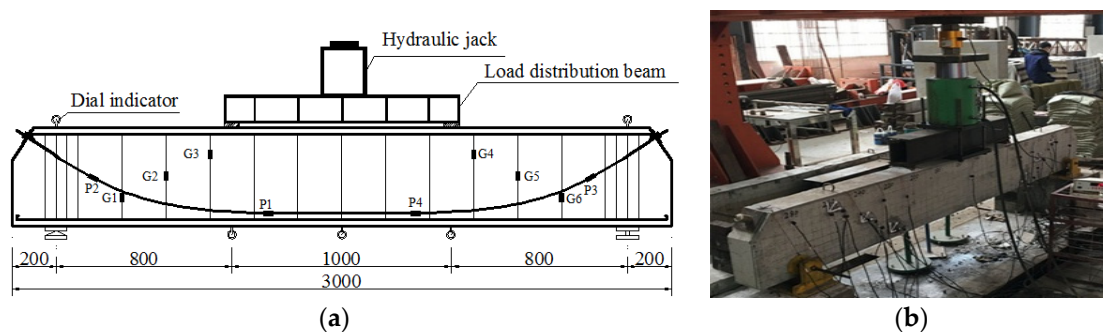


Figure 2. Loading diagram and measuring point layout. (a) Loading and measuring point layout (unit: mm), (b) Loading photo.

Table 2. Corrosion time and corrosion rate η .

Beam No.	B1	B2	B3	B4
Corrosion time (<i>d</i>)	0	3	6	10
Area corrosion rate η (%)	0	7.9	19.4	31.7

3. Shear Performance Analysis

3.1. Crack Propagation

Although steel strands varied in corrosion rate, the crack propagation modes of test beams are basically consistent. Under the loading action, a vertical bending crack firstly appears in the pure bending segment between loading points. As the load continues to grow, multiple diagonal cracks pointing to loading points successively appear in the bending-shear segment at the corrosion treatment side, while diagonal cracks appear later at the side without corrosion treatment. Due to the difference between steel strands in corrosion rate, the cracking load of test beams is varied. The load value upon the appearance of the first bending crack and that upon the appearance of the first diagonal crack in B1–B4 are listed in Table 3. It could be seen that with the increase of corrosion rate, both the load upon the appearance of the first bending crack and that upon the appearance of the first diagonal crack in B1–B4 are gradually reduced, and both bending load and diagonal cracking load are inversely proportional to the change in corrosion rate. The more seriously the prestressed steel bars are corroded, the greater their effective cross-sectional area loss. The more serious the concrete bonding degradation, the greater the loss of the effective prestress, and the earlier the test beam cracked under the loading action. At the corrosion rate of 7.9%, 19.4%, and 31.7%, the loads leading to the initiation of bending crack and diagonal crack in test beams declines by 11.3%, 23.9%, and 40.8%, and 10%, 19% and 37.8%, respectively, in comparison with the uncorroded beam. This means that the bending crack is more sensitive to corrosion rate.

Table 3. Main test results.

No.	η /%	P_{cr1} /kN	P_{cr2} /kN	P_u /kN	θ /°	w /mm
B1	0	71	90	298	30	11.2
B2	7.9	63	81	281	36	11.7
B3	19.4	54	73	271	41	12.3
B4	31.7	42	56	252	35	12.9

Note: η % denotes the corrosion rate of steel strands in test beams, P_{cr1} stands for the load corresponding to the first bending crack, P_{cr2} represents the load corresponding to the first diagonal crack, P_u is the ultimate load, θ is the dip angle in the horizontal direction of the principal diagonal crack, and w is the midspan deflection under the ultimate load.

The crack distribution in B1–B4 under the ultimate load is given in Figure 3. For all test beams, the principal diagonal crack leading to the failure is a bending-shear crack, accompanied by avulsion phenomenon of longitudinal steel bars close to the beam bottom. The number of cracks in all test beams is within 11 to 12, the beam spacing is basically identical, the height distribution of each crack shows no obvious changes relative to the comparison beam (basically equivalent), and the crack distribution in the concrete beams is not evidently influenced by the corrosion of the prestressed steel bars.

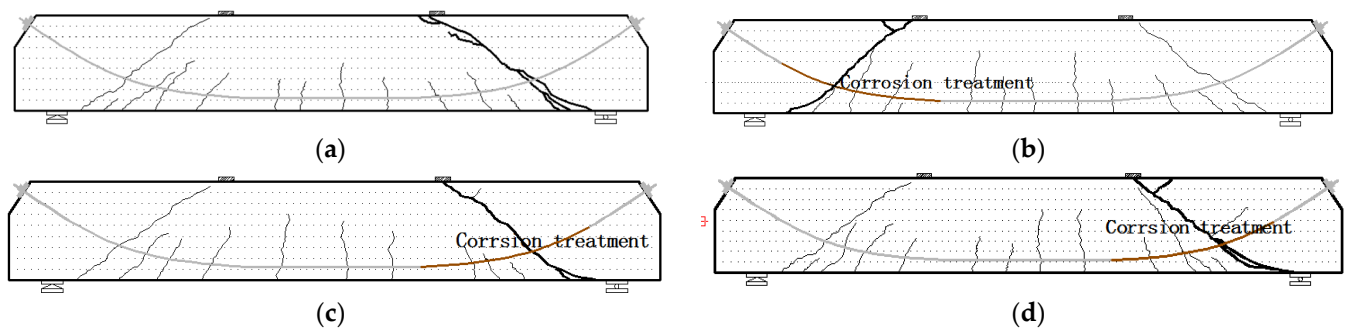


Figure 3. Crack distribution under the ultimate loading state. (a) B1 beam; (b) B2 beam; (c) B3 beam; (d) B4 beam.

3.2. Failure Mode

Despite different corrosion rates of steel strands, the failure modes of test beams are basically identical. In the initial loading stage, the pure bending segment is the first to start cracking (vertical bending tensile crack). With the increase in the load, the bending-shear segments at both sides are subjected to 3–5 diagonal cracks. As the load continued to increase, a web-shear diagonal crack pointing to the loading point firstly appears nearby the support of the bending-shear segment at the corrosion side, while the diagonal crack appears at the uncorroded side slightly later. Subsequently, the load is further increased, the vertical crack in the pure bending zone no longer develops towards the beam top, but the web-shear diagonal crack nearby the support at the corroded side develops simultaneously towards two directions and extended beneath the beam-top loading point, becoming a critical diagonal crack. The width of the principal diagonal crack increases with the load. When the ultimate load is reached and cannot be further increased, the test beam made a cracking sound and a transverse crack appears in the concrete nearby the lower part of load board at the top of the diagonal crack. As a result, the concrete is crushed and peels off. Moreover, the crack width in the zone between erection bars and steel strands at the lower end of the crack continued to grow, a horizontal avulsed crack appears on the upper surface of erection steel bars, and test beams are subjected to failures. The failure modes as shown in Figure 4. The failure modes of test beams do not go through any obvious change due to the corrosion of the steel strands, all of which experienced typical shear-compression failures.

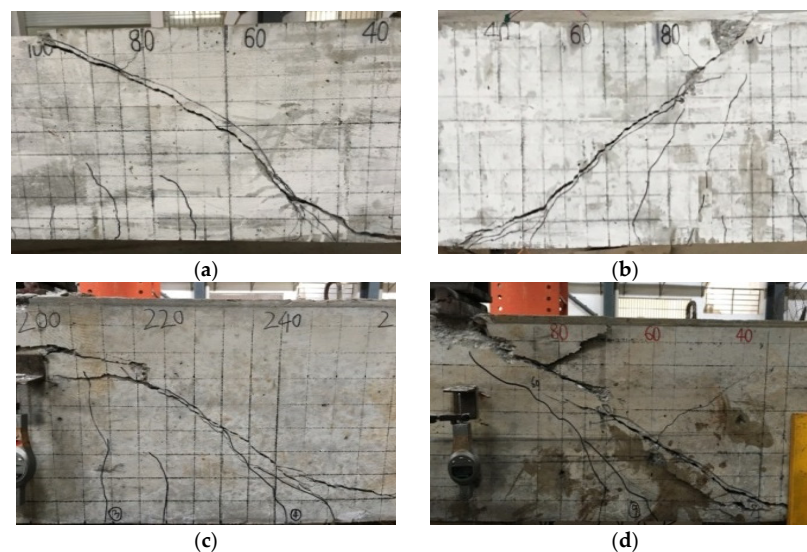


Figure 4. Failure sections of B1–B4. (a) B1 beam; (b) B2 beam; (c) B3 beam; (d) B4 beam.

3.3. Deflection Analysis

Deflection is the main index reflecting the overall stiffness of a structure. In this study, the deflections at midspan under the action of each stage of load are selected to appear in Figure 5. It appears that the curves presented indicate three-stage characteristics:

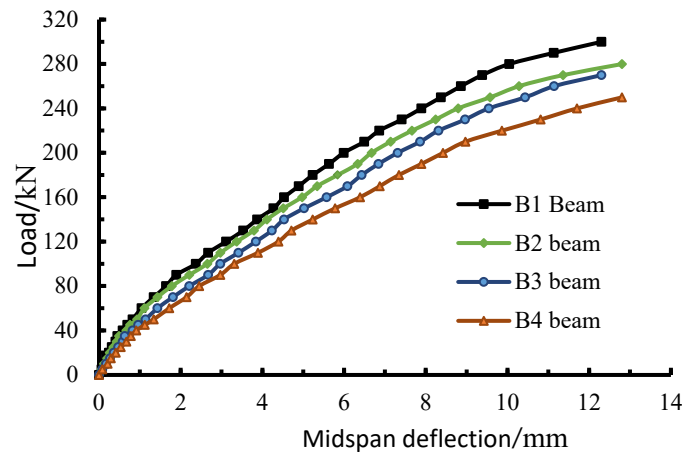


Figure 5. Load–midspan deflection curves.

In the initial loading stage, the midspan deflection of each test beam is basically linearly correlated with the load and showed elastic structural characteristics, and the deflection difference is very minor. This is because each test beam is in an elastic deformation stage, and the structural stiffness is mainly controlled by concrete. At this time, the structure is not cracked yet, and the overall stiffness difference is not large. After the load reaches the cracking load, the slope of the load–deflection curve of each test beam declines. The growth rate of deflection is higher than that of the load, and the structure is inelastic, indicating that the structural stiffness declines due to cracking. However, the stiffness decrements of different test beams are inconsistent. The load–deflection curves of B1 and B2 are basically overlapped in this stage, but the slope of the load–deflection curves of B3 and B4 is apparently smaller than that of B1. This indicates that the decline degree of structural stiffness is associated with the corrosion rate of steel strands. The reason is that due to the corrosion of steel strands, the structure cracked in advance in comparison with B1. In this case, the concrete with a cracking cross section exit from the work, and thus the concrete stress is redistributed. With the appearance of cracks, the bonding performance between steel strands and concrete is damaged, between which slipping occurs with the degradation of the overall stiffness.

When the load approaches the ultimate load, the structure is evidently plastic; namely, the load growth is small but the structural deflection increases rapidly. Compared with the B1 test beam, the ultimate midspan deflections of the B2–B4 beams increase by 2.7%, 8.0%, and 11.6%, respectively.

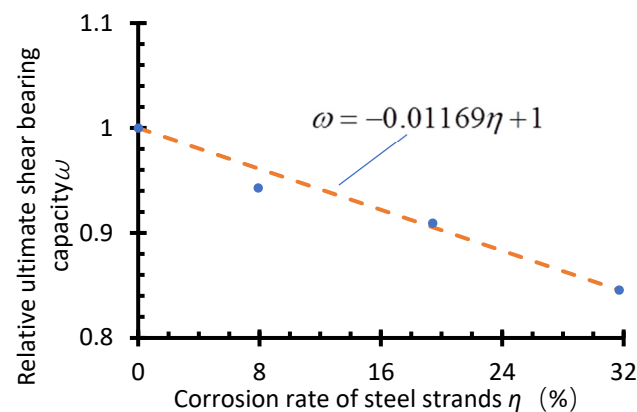
3.4. Ultimate Load

The ultimate loads of the test beams (Table 4) are significantly influenced by the corrosion of steel strands, which could result in the degradation of structural shear bearing capacity. With the increase in the corrosion rate, moreover, the structural shear bearing capacity is degraded more significantly. In this test, the shear bearing capacity of the test beams decreases by 5.8%, 9.1%, and 15.5%, respectively, at the steel strands' area corrosion rate of 7.9%, 19.4%, and 31.7%. All test beams experienced a brittle shear failure. The relative ultimate shear strength ω is defined as the ratio of the ultimate load of corroded beam to that of uncorroded beam. The area corrosion rate η and relative ultimate shear strength ω of test beams are listed in Table 4.

Table 4. Corrosion rate η and relative shear strength ω of test beams.

Test Beam No.	B1	B2	B3	B4
Corrosion rate/ η %	0	7.9	19.4	31.7
Ultimate load (KN)	298	281	271	252
Relative ultimate shear strength ω	1	0.9430	0.9094	0.8456

The data in Table 4 are linearly fitted to obtain η - ω relations, as shown in Figure 6. Due to the corrosion of the prestressed steel bars, the shear bearing capacity of component declines, and the area corrosion rate η shows a good linear relation with the relative ultimate shear strength ω .

**Figure 6.** ω - η curves.

4. Calculation of Ultimate Shear Bearing Capacity

4.1. Simplified Computational Analysis of Shear Bearing Capacity

In accordance with Code for Design of Concrete Structures (GB 50010-2020) [19], the oblique cross-sectional shear bearing capacity of PC flexural members configured with stirrups and bent-up bars can be calculated through the following equations:

$$V_u = V_{cs} + V_p + 0.8f_y A_{sb} \sin \alpha_s + 0.8f_{py} A_{pb} \sin \alpha \quad (1)$$

$$V_{cs} = 1.75 f_t b h_0 / (\lambda + 1) + f_{yv} A_{sv} h_0 / s \quad (2)$$

where V_u is the shear bearing capacity of component and V_{cs} represents the shear bearing capacity provided by stirrups and concrete. V_p denotes the shear bearing capacity provided by the prestress, which is 0, since the structure will reach the ultimate bearing capacity when cracking, and it is stipulated in the code that the contribution made by the prestress component to shear resistance should not be considered. A_{sb} and A_{pb} represent the cross-sectional areas of bent-up bars and prestressed steel bars, respectively. α_s and α_p are the included angle of the longitudinal axis of the component with bent-up bars and bent-up prestressed steel bars on the oblique section, respectively. λ is the shear span ratio of each test beam; f_t stands for the design axial tensile strength of concrete, which can be calculated as per $f_t = 0.26 f_{cu}^{2/3}$. f_{cu} is the cubic compressive strength of concrete, f_{yv} is the design tensile strength of stirrups, A_{sv} is the cross-sectional area of stirrups, s represents the spacing of stirrups, b and h_0 denote the rectangular cross-sectional width and without effective height, respectively.

According to the test beam design and the existing literature [20], the shear bearing capacity of test beams is expressed as

$$V_u = V_c + V_s + V'_p \quad (3)$$

$$V_c = \left(\frac{0.08}{\lambda - 0.3} + \frac{100\mu}{\lambda f_c} \right) \times f_c b h_0 \quad (4)$$

$$V_s = \frac{f_{yv} A_{sv} Z}{s} (\cot \theta + \cot \alpha_s) \quad (5)$$

$$V'_p = f_{py} A_{pb} \sin \alpha_p \quad (6)$$

where V_c , V_s and V'_p represent the shear bearing capacity of concrete, stirrups and diagonal steel strands, respectively. μ denotes the longitudinal reinforcement ratio, and f_c is the compressive strength of concrete, taken as $f_c = 0.76 f_{cu}$. Z is the arm of force, taken as $0.875 h_0$, θ stands for the included angle between principal diagonal crack and beam axis, α_s is the included angle between stirrups and beam axis, α_p represents the included angle between steel strands on the principal diagonal rupture plane and beam axis. It could be seen from the test results that stirrups already yielded upon a failure, so the actual yield value is taken during the stirrup calculation, and the standard value (1860 MPa) is selected for steel strands.

In Equations (3) and (4), the corrosion of steel strands is not yet considered. The design parameters of B1 are substituted into such equations to calculate the shear bearing capacity as 293.8 kN, whose error relative to the test value is -1.4% , indicating a high accuracy.

The corrosion of steel strands influences the structural shear bearing capacity mainly in two ways: On the one hand, the corrosion causes the cross-sectional damage of steel strands and reduces the strength of the steel strand. On the other hand, the corrosion led to the bonding degradation and uncoordinated strain between steel strands and concrete and stress hysteresis of prestressed steel bars. Consequently, steel strands fail to give full play to their strength, and they may not reach the standard strength upon the structural shear-compression failure. Hence, the effect of corrosion should be considered in order to correctly evaluate the shear bearing capacity of components with corroded steel strands. Given this, the concept of strength utilization rate γ_c [21] of the corroded prestressed steel bars for the PC beams is introduced to calculate the shear bearing capacity of the test beams with corroded steel strands. Through the strength utilization rate γ_c , the actual effective prestress of corroded steel strands under the ultimate load can be solved as:

$$T'_{pc} = \gamma_c (1 - 0.0981\eta) f_{pk} A_{pc} \quad (7)$$

$$A_{pc} = (1 - \eta) A_p \quad (8)$$

$$\gamma_c = \begin{cases} 1 & \xi_\eta \leq 0.23 \\ 1 - k(2.24\xi_\eta - 1.61\xi_\eta^2 - 0.41) & \xi_\eta > 0.23 \end{cases} \quad (9)$$

$$k = 0.062\eta + 0.051 \quad (10)$$

$$\xi_\eta = \frac{(1 - \eta)T_e}{f_c b h_p} \quad (11)$$

where T'_{pc} is the actual prestress of corroded prestressed steel bars, f_{pk} is the standard strength of uncorroded prestressed steel bars, γ_c is the strength utilization rate; η is the area corrosion rate, A_{pc} is the actual effective area of prestressed steel bars, A_p is the cross-sectional area of uncorroded prestressed steel bars, k is the corrosion influencing factor, and h_p is the distance from the gravity center of prestressed steel bars to the concrete top.

Then, Equation (1) is corrected according to Equations (7)–(10), thereby acquiring the ultimate shear bearing capacity of test beams as follows

$$V'_u = V_c + V_s + T'_{pc} \sin \alpha_p \quad (12)$$

The calculation results for the ultimate bearing capacity of the test beams according to Equation (12) are presented in Table 5.

Table 5. Comparison between test results and theoretical calculation results of B1–B4.

No.	η (%)	θ	$\sin \alpha_p$	k	r_c	T'_{pc}/KN	V'_p/KN	V_c/KN	V_s/KN	V_u/KN	V_{u1}/KN	V_{u1}/V_u	V_{ue}/KN	V_{ue}/V_u
B1	0	30	0.271	0.051	0.9872	255.2	74.5	149.1	75.0	298	293.5	0.985	286.5	0.961
B2	7.9	36	0.221	0.056	0.9866	233.1	68.1	148.6	75.0	281	275.1	0.979	281.1	1.000
B3	19.4	41	0.210	0.063	0.9860	201.6	58.9	147.9	75.0	271	265.5	0.978	268.0	0.989
B4	31.7	35	0.223	0.071	0.9859	168.8	49.3	147.1	75.0	252	259.7	1.031	260.3	0.995

Note: $\eta\%$ is area corrosion rate; V_u is the test ultimate load; V_{u1} is ultimate load obtained through simplified calculation; V_{ue} is the ultimate load obtained through numerical analysis and calculation.

As seen in Table 5, the average error between test value and theoretical value of shear bearing capacity is 2.2%. The value obtained through simplified calculation as per the code coincides with the test value very well, thus verifying the reasonability of the simplified calculations.

4.2. Numerical Simulation Analysis of Shear Performance

A finite element calculation model of each damaged concrete beam is established via ABAQUS. The structure is constructed with a separated-type model. Therein, concrete is simulated using reduced integral hexahedral elements (C3D8R) and expressed by a constitutive concrete damage plasticity model. Load boards and supports are simulated using rigid elements (R3D4) to avoid stress concentration, and boundary conditions are set and loading is performed through the rigid body reference point (RP). Ordinary steel bars and prestressed steel bars are both simulated using truss elements (T3D2), and strong hardening constitutive relation models [22,23] such as double broken lines are chosen.

The bond-slip between steel strands and concrete is simulated using nonlinear spring elements (SPRING2), specifically by setting elements parallel and perpendicular to the steel bars at the co-node between concrete and steel strands. To simulate the rupture of steel strands, the elements are inactivated and activated by using the model change in the interaction module. In Stage 1, all structural elements are inactivated, and the elements in the corresponding stage are activated according to experimental steps, thus completing the simulation of the whole test process.

The calculated value and measured value of ultimate bending load of each test beam are listed in Table 5. It could be seen that the relative error between test value and the value obtained through numerical analysis and calculation was small, with the maximum error of only 3.9%, and all calculated values relatively agree with the test values.

The test value and the calculated value of midspan deformation of each beam under different load conditions are displayed in Figure 7. It could be observed from Figure 7 that the test value is relatively consistent with the calculated value, which further verifies the effectiveness of the calculation model.

To compare the concrete damage status and the stress state of stirrups under the same stage of loading action and theoretically analyze the influence of the corrosion of steel strands on structural shear bearing capacity, the load is taken as 252 kN by reference to the ultimate bearing capacity of each beam. The concrete damage and cracking conditions of beams under a tensile-compressive state as well as the stress state of stirrups are exhibited in Figures 8–10. It can be observed that in each test beam, tensile cracks are firstly generated at the bottom of the midspan pure bending segment. As the load continued to grow, and bending-shear diagonal cracks are formed in the bending-shear segment and pointed to the support along the loading point. It can be judged that the failure mode of the beam is not changed due to the increase in the corrosion rate of steel strands. That is, the shear-compression failure is a dominated factor. Relative to the comparison beam B1, the increasing corrosion rate of steel strands exerts a minor influence on the tensile damage and cracking of concrete in the pure bending segment but generates a great impact on the tensile-compression damage and cracking of concrete in the bending-shear segment. Meanwhile, as the load increases, the stress of stirrups develops at a high speed after the tensile-compressive damage and cracking of concrete. Under the action of 252 kN, the maximum stress of comparison beam B1, B2, B3, and B4 are 197 MPa, 200 MPa, 342 MPa,

and 476 MPa, respectively. The greater the corrosion rate of steel strands, the earlier the stirrups would enter a yield state. The stress growth rate of stirrups is much greater than the load increasing rate, and meanwhile, the diagonal crack induced by the tensile-compressive damage of concrete already run through the whole beam body. By comparing the finite element simulation results (Figures 8 and 9) with the experimental results (Figure 3), we can find that the crack development laws of the two cases are consistent. This further proves the effectiveness of the calculation model proposed in this paper. Therefore, the yield of stirrups can serve as a sign for beams to reach their ultimate bearing capacity in practical applications.

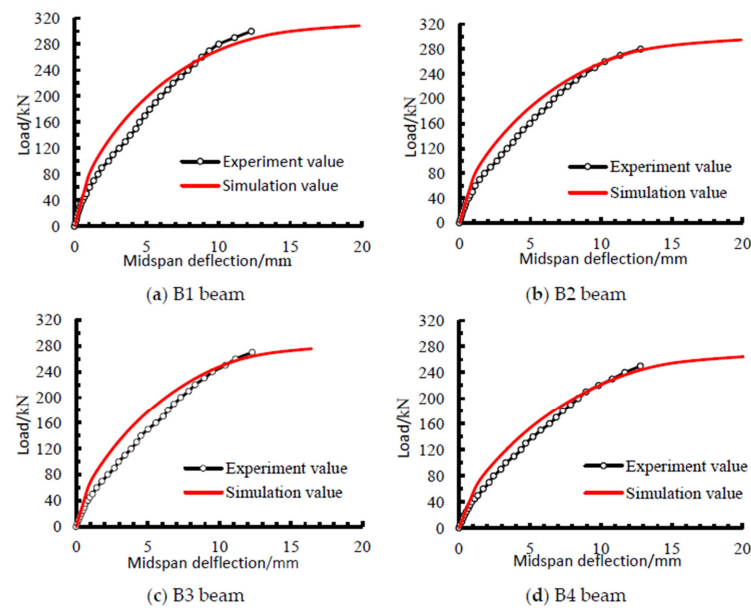


Figure 7. Comparison of midspan load–deflection curves between test results and calculation results.

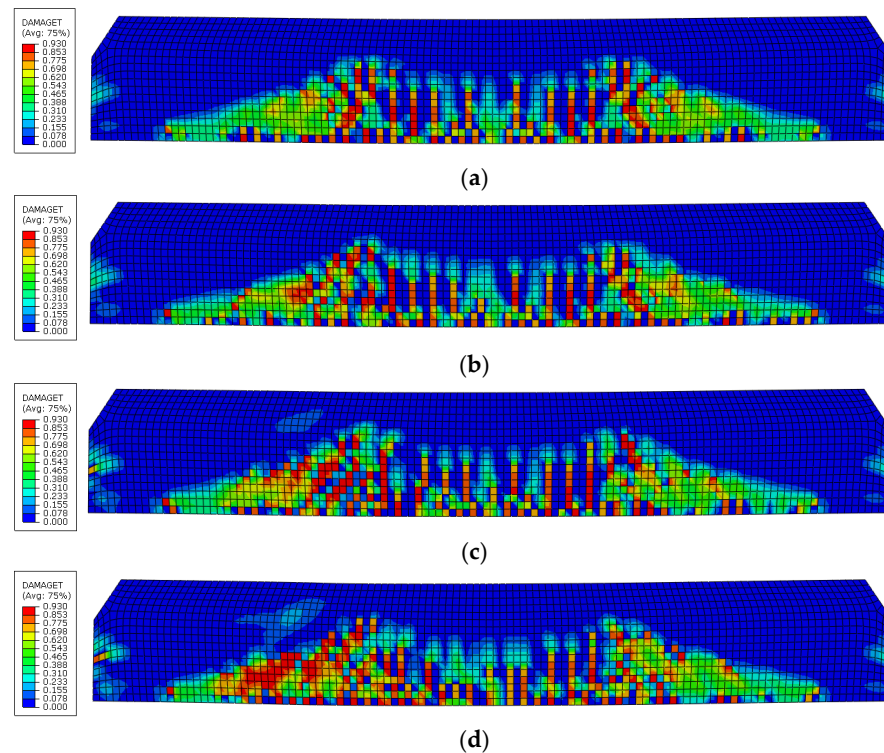


Figure 8. Tensile damage and cracking of concrete in test beams. (a) B1 beam; (b) B2 beam; (c) B3 beam; (d) B4 beam.

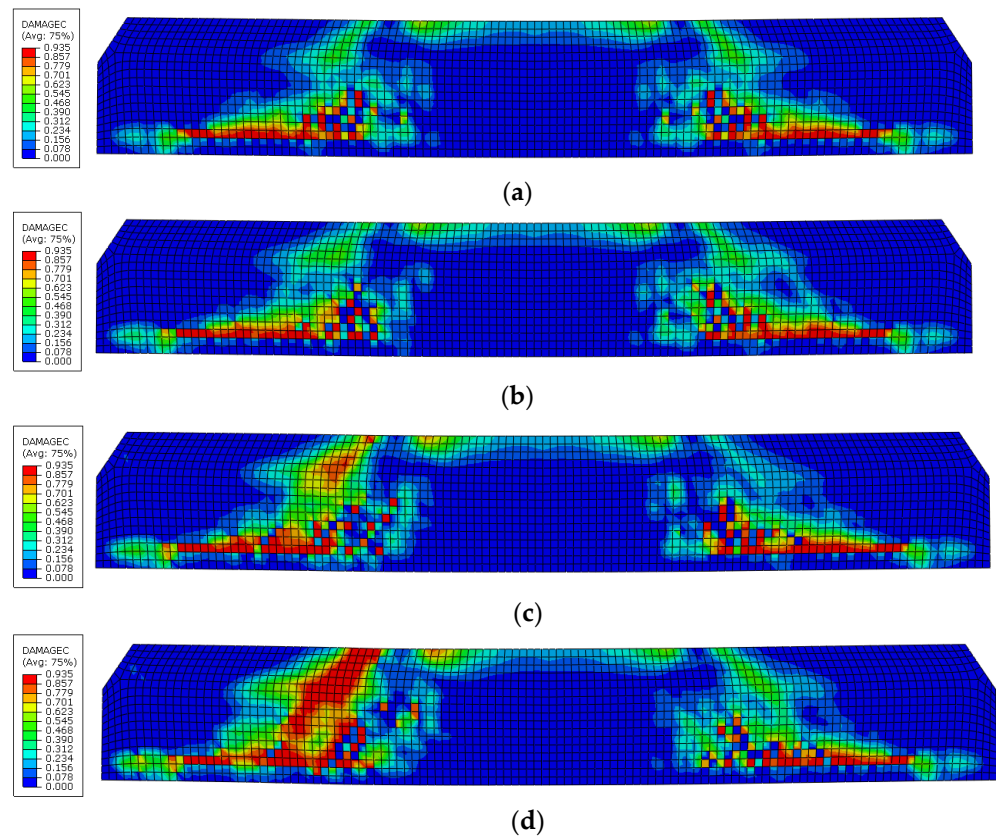


Figure 9. Compressive damage and cracking of concrete in test beams. (a) B1 beam; (b) B2 beam; (c) B3 beam; (d) B4 beam.

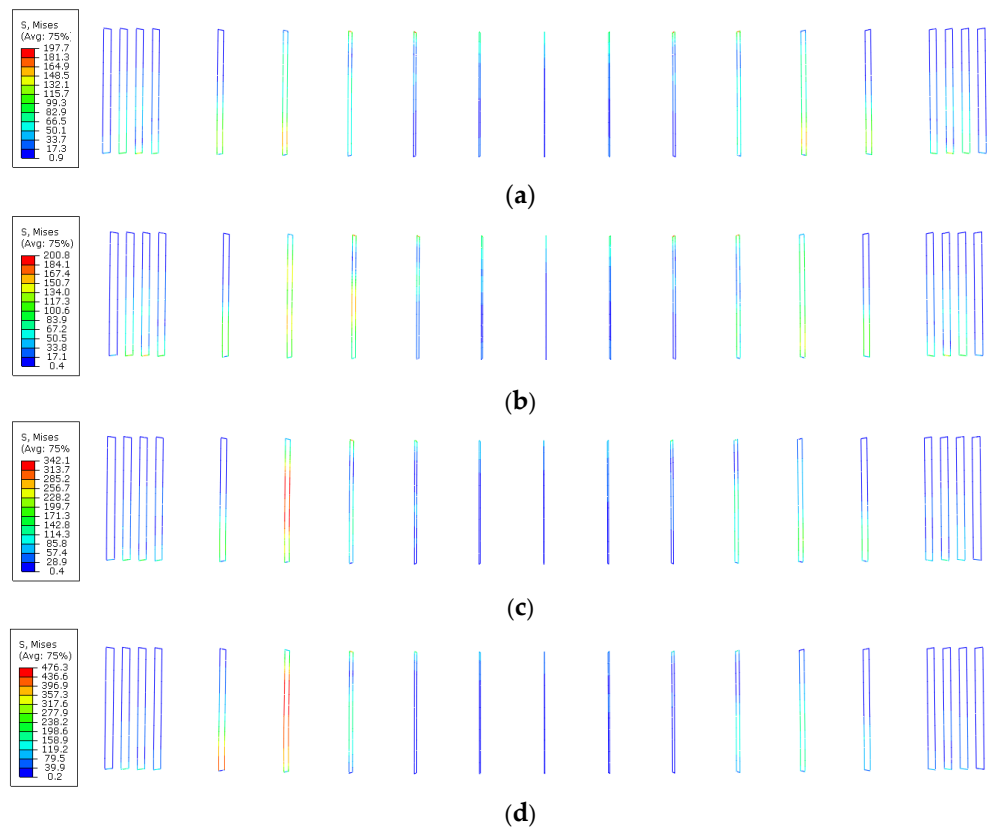


Figure 10. Stress diagram of stirrups. (a) B1 beam; (b) B2 beam; (c) B3 beam; (d) B4 beam.

5. Conclusions

In this paper, the crack development, failure mode, deflection change, and ultimate shear bearing capacity development law of the PC structure with locally corroded steel strands were analyzed, and the calculation model of ultimate shear bearing capacity considering the effect of local corrosion was also proposed. The conclusions are summarized as follows:

(1) The crack development mode and failure mode of PC structure are not affected by the corrosion of prestressed steel strands, but its shear bearing capacity gradually decreases with the increase of the corrosion rate of the steel strand section, and the corrosion rate of prestressed steel strand has a linear relationship with the degradation of the relative ultimate shear strength.

(2) The calculation method of shear capacity of PC structure considering the effect of local corrosion proposed in this paper has high prediction accuracy, and the average error of all prediction results in this experiment is only 2.6%.

(3) The larger the cross-section corrosion rate of the steel strand, the earlier the stirrup enters the yield state. In the service state evaluation of the corroded PC structure, the yield of stirrups can be used as a sign of the ultimate shear bearing capacity of the structure.

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Article

Test and Mesoscopic Analysis of Chloride Ion Diffusion of High-Performance-Concrete with Fly Ash and Silica Fume

Huang Tang ^{1,*}, Yiming Yang ^{1,*}, Jianxin Peng ^{2,*}, Peng Liu ^{3,*} and Jianren Zhang ²

¹ Hunan Engineering Research Center of Structural Safety and Disaster Prevention for Urban Underground Infrastructure, School of Civil Engineering, Hunan City University, Yiyang 413000, China; tanghuang@hncu.edu.cn

² School of Civil Engineering, Changsha University of Science and Technology, Changsha 410114, China; jianrenz@hotmail.com

³ School of Civil Engineering, Central South University, Changsha 410114, China

* Correspondence: yangyiming@hncu.edu.cn (Y.Y.); jianxinpeng@csust.edu.cn (J.P.); 2015038@csu.edu.cn (P.L.); Tel.: +86-0737-4628297 (Y.Y.); +86-0731-85258698 (J.P.)

Abstract: High performance concrete (HPC) is a kind of concrete with mineral admixtures, which has better resistance ability to chloride ions diffusion than ordinary concrete. In the present study, the authors carried out a chloride ion diffusion experiment for the HPC with fly ash and silica fume. The influence of the water–binder ratio (W/B), binary (Portland cement–Fly ash (PC-FS) and Portland cement–Silica fume (PC-SF) and ternary (Portland cement–Fly ash–Silica fume (PC-FA-SF)) combinations on the concrete compressive strength and chloride ion diffusion was investigated. It was found that the compressive strength of normal concrete and HPC increase with the decrease in the W/B , the ratio of W/B decreasing value to strength increasing value for normal concrete is 0.74, and for the HPC is 0.20, so the influence of the W/B on the concrete strength for normal concrete was obviously more than the HPC. The influence of the contents of the SF or FA on developing the concrete strength was limited. The concrete compressive strength of ternary combination specimens decreases with the increase in FA or SF when the content of the other mineral admixture SF or FA remained unchanged. The ternary combination was more efficient in prohibiting chlorides ingress insider the specimens than the binary combination. The mesoscopic simulation and the tested value of the chloride ion under the same depth was close, the average ratio of simulation value to tested value was 0.91. The aggregate shape and distribution also had a negligible influence on chloride diffusivity in the HPC, but the chloride ion concentration increased with the increase in aggregate size.

Keywords: high performance concrete (HPC); chloride ion diffusion; mesoscopic simulation; fly ash; silica fume

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1. Introduction

Many scholars have studied how adding other materials such as carbon nanotubes [1], basalt fiber [2] and mineral admixtures into normal concrete can improve the mechanical performance and durability. High performance concrete (HPC) is a kind of concrete with mineral admixtures, has high amounts of cementitious material and low water–cement ratio [3]. The addition of an appropriate amount of mineral admixture in the HPC can refine and improve the pore size distribution of the hardened cement paste, reduce the number of large pores, and increase the number of small and connected ones, which can improve the density of concrete and ultimately achieve the purpose of improving the performance of the concrete [4], so the HPC has been widely used in the erosion environment for its stronger resistance to all kinds of deleterious ions than ordinary concrete, in these ions, the aggressive of chloride ion concentration are the most obvious [5], so it is meaningful to explore the chloride ion diffusion mechanism in the HPC.

Many studies have confirmed that mineral admixtures in the HPC can improve its ability to resist chloride ion penetration compared with normal concrete, and there are two main reasons for the resistance of HPC to chloride ion penetration: firstly, the permeability of concrete mixtures, which is decreased because of the existence of the mineral admixtures. Secondly, the chloride ion binding capacity of the cementitious paste, which increases with calcium silicate hydrate gel formation and increased alumina levels inducing more chloride ion to be fixed as Friedel's salt [6]. As a common admixture, many authors focused on the durability of HPC with fly ash (FA). Sengul and Tasdemir [7] investigated the 50% replaced FA and 50% replaced finely ground granulated blast furnace slag on rapid chloride permeability. Chalee and Jaturapitakkul [8] used the original and classified FA used as a partial replacement of Portland cement at different weights of binder to investigate the effect of W/B and FA fineness on chloride diffusion coefficient (D_c) of concrete under a marine environment. The author concluded that the inclusion of pozzolans is more effective than decreasing the W/B to reduce the chloride permeability of concrete. Moffatt et al. [9] presented the durability performance of concrete incorporating high volumes of FA exposed to a harsh marine environment for 19 to 24 years. The depth of chloride penetration was found to be in excess of 100 mm for the concrete specimens without FA, whereas the presence of FA significantly decreased the depth of penetration to approximately 30 and 40 mm in specimens containing either normal density or lightweight aggregate, respectively. The above studies mainly focused on the influence of simply FA content on the durability performance of HPC, and a few studies deeply investigated the chloride permeability of HPC if other mineral admixtures as partial replacements of Portland cement.

At the same time, other authors explored the chloride ion penetration behavior in the HPC with other mineral admixtures. Cabrera and Nwaubani [10] measured the effective chloride ion diffusion coefficients of Portland cement (PC), PC-15% pulverized fuel ash (PFA), PC-15% metakaolin (MK) pastes using a chloride ion diffusion cell. Both the PC-MK and the PC-PFA pastes gave lower chloride ion diffusion coefficients than the PC paste, and the former gave particularly low values. Thomas et al. [11] and Hooton et al. [12] determined the chloride ion diffusion resistance of concrete replacement of PC with high-reactivity MK (HRM). The apparent diffusion coefficients were reduced on increasing the exposure time and on decreasing the W/B and showed marked decreases with increasing HRM content. Coleman and Page [13] also determined that the MK produces a significant increase in chloride ion binding capacity of the paste when used as a partial cement replacement. Seleem et al. [6] investigated the seawater resistance of concrete incorporating silica fume (SF), ground granulated blast furnace slag (GGBS), and metakaolin (MK) as an addition to cement in binary and ternary combinations. The authors confirmed that all kinds of pozzolanic materials are efficient in reducing the permeability of concrete far below the control one. Meanwhile, silica fume is the most efficient in this regard. The ternary blend (PC-MK-SF) is superior to all other mixtures in producing impermeable concrete. Geetha et al. [14] found that copper slag was marine application which could improve compressive strength and flexural strength and reduce sorptivity and chloride ion penetrability. Harilal et al.'s [15] work reported a novel high performance green concrete which has a better resistance ability of chloride ion than normal concrete. Jiang et al. [16] analyzed the influence of crack self-healing degree on chloride ion transport process, finding that chloride ions not only transport vertically along the crack, but also propagate in the horizontal direction of the crack. Bachtar et al. [1] provided information about the mechanism and the influence of prepared high-performance concrete mixed with either freshwater or seawater and made to cure in freshwater, seawater, or air. Ding et al. [17] compared the different effects of FA and slag on anti-rebar corrosion ability of high strength concrete with chloride ion. FA and SF are the most common mineral admixtures used in the HPC, few above studies focused on the chloride ion diffusion behavior for HPC contained FA and SF. Though Khan [18] investigated the chloride ion penetration and diffusion for high performance concrete pulverized fuel ash and silica fume with various W/B, environmental factors such as temperature, humidity were not considered, and the

influence of cementitious material combination (FA and SF) on the chloride ion diffusion needs to be further explored.

The corrosion of steel bar induced by chloride erosion is the main factor leading to the degeneration of structure mechanical performance. In recent years, many studies have focused on the mechanical property of structure after being corroded [19–25], at the same time, many studies have investigated chloride diffusion behavior in multiple phases of normal concrete by numerical simulations in mesoscopic level [26–30]. The HPC has the high amounts of cementitious material and low water–cement ratio [31], the chloride ion diffusion process in the HPC will be different with the normal concrete. However, the mesoscopic level research for the chloride ion diffusion process of HPC is few, scholars mainly focus on the material behavior of the HPC using mesoscopic model. Guo et al. [31] proposed a two-dimensional mesoscopic lattice model of high-performance concrete which accounts for fatigue damage. Huespe et al. [32] investigated the failure mode of high-performance fiber reinforced concrete composites (HPFRC), through proposing a finite element methodology. Forquin et al. [33] developed a mesoscopic numerical model for the ultra-high performance fiber reinforced concrete to investigate the dynamic tension behavior. Feng et al. [34] investigated the mechanical compression properties of UHPC through a 3D mesoscale model. However, the mesoscopic study of the chloride ion diffusion process of HPC containing FA and SF is limited.

In order to investigate the influence of different content combinations of FA and SF on the chloride permeability on HPC, in the present study, the authors carried out a chloride ion diffusion experiment for the HPC. The influence of W/B, binary (PC-FA and PC-SF) and ternary (PC-FA-SF) combinations on the concrete compressive strength and chloride ion diffusion was investigated. The chloride ion diffusion coefficient was modified and the chloride ion diffusion model of HPC was revised based on Fick's second law considering the W/B, temperature, humidity and time. The number simulation of chloride ion diffusion at the mesoscopic level for HPC with FA and SF was carried out, and the influence of aggregate shape, distribution, size of HPC on the chloride ion diffusion is investigated.

2. Experimental Programs

2.1. Detail of Specimens

All specimens were divided into two groups in the experiment. Group I was mainly designed for investigating the influence of W/B, compressive strength on the chloride ion diffusion behavior in the HPC. Group II was mainly designed for analyzing the influence of different mixing methods of mineral admixtures on the chloride ion diffusion behavior.

Twenty-four standard cubic specimens with dimension of 150 mm × 150 mm × 150 mm are made in the group I as shown in Table 1. Eight mix proportions were designed, three specimens were made for every mix proportion. The mix proportions are shown in Table 1. The HPC80 and HPC100 adopted P52.5 ordinary Portland cement, others adopted P42.5 ordinary Portland cement. The coarse aggregate adopted macadam, and the max particle size was 20 mm continuous grading. The fine aggregate adopted fine sand, and the fineness modulus was from 2.5 to 3. The water-reducing rate of the water reducer was from 20% to 25%. The slump of concrete was controlled from 18 cm to 20 cm.

There are twelve specimens in the group II as shown in Table 2. The cement variety and the content of aggregate were the same for all specimens. The specimens A-60 were the HPC without any mineral admixtures. The specimens FA-10, FA-20 and FA-30 were mixed with 10%, 20% and 30% (in weight) FA, respectively. The specimens SF-5 and SF-10 were mixed with 5% and 10% silica fume, respectively. Other specimens were mixed with both FA and SF, the contents of mineral admixtures are shown in Table 2.

Table 1. Group I specimens mix proportion.

Specimen	Water (kg/m ³)	Cement (kg/m ³)	FA (kg/m ³)	SF (kg/m ³)	Fine Ag- gregate (kg/m ³)	Coarse Aggregate (kg/m ³)	Super Plasticizer (kg/m ³)	FA Ratio	SF Ratio	W/B
NC50 ¹	136.35	396	0	0	837.4	964.25	7.13	—	—	0.34
NC60	122.27	444	0	0	791.82	1008.91	8.44	—	—	0.28
NC70	108.145	488	0	0	747.3	1052.56	9.76	—	—	0.22
HPC50 ²	123.135	396	64	35	795	1005.86	8.91	13%	7%	0.25
HPC60	119.63	409	66	36	783.34	1017.03	9.71	13%	7%	0.23
HPC70	115.215	421	69	37	769.56	1030.22	10.54	13%	7%	0.22
HPC80	144	501	81	44	647	1096	8.76	13%	7%	0.21
HPC100	129	516	84	45	607	1137	9.03	13%	7%	0.20

Note: ¹ Normal concrete. ² High performance concrete.

Table 2. Group II specimens mix proportion.

Specimen	Water (kg/m ³)	Cement (kg/m ³)	FA (kg/m ³)	SF (kg/m ³)	Fine Ag- gregate (kg/m ³)	Coarse Aggregate (kg/m ³)	Super Plasticizer (kg/m ³)	FA Ratio	SF Ratio	W/B
A-60	97.6	488	0	0	745	1044.5	10.5	—	—	0.2
FA-10	97.6	439.2	48.8	0	745	1044.5	10.5	10%	—	0.2
FA-20	97.6	390.5	97.6	0	745	1044.5	10.5	20%	—	0.2
FA-30	97.6	341.3	147	0	745	1044.5	10.5	30%	—	0.2
SF-5	97.6	463.2	0	24.6	745	1044.5	10.5	—	5%	0.2
SF-10	97.6	439.5	0	49.2	745	1044.5	10.5	—	10%	0.2
SF-15	97.6	414.2	0	73.8	745	1044.5	10.5	—	15%	0.2
F10-S5	97.6	415.0	48.8	24.4	745	1044.5	10.5	10%	5%	0.2
F20-S5	97.6	366.2	97.8	24.3	745	1044.5	10.5	20%	5%	0.2
F30-S5	97.6	316.8	146.8	24.6	745	1044.5	10.5	30%	5%	0.2
F10-S10	97.6	390.3	48.9	48.8	745	1044.5	10.5	10%	10%	0.2
F20-S10	97.6	341.3	98.2	48.5	745	1044.5	10.5	20%	10%	0.2
F30-S10	97.6	292.7	146.7	48.6	745	1044.5	10.5	30%	10%	0.2

2.2. Compressive Strength Test

The specimens were made using the standard maintenance method according to the Code for design of concrete structures (GT 50010-2010) [35]. All specimens were de-molded after 24 h and then cured in a standard curing chamber at 20 ± 2 °C and 97–100% relative humidity for 28 days. After curing, the standard cube compressive strength test was carried out to obtain the cube compressive strength.

2.3. Chloride Penetration Test

As shown in Figure 1, five surfaces (②–⑥ surfaces) of the specimens were painted using epoxy resin, and the last surface (① surface) was an erosion face. The process of painting epoxy resin is shown in Figure 1. All of the specimens were immersed in a sodium chloride solution with concentration of 3.5% (in weight) for two months, as shown in Figure 2a, the environmental temperature was 25. After that, the specimens were removed from the solution pool and dried naturally for seven days. An electric drill was used to obtain the concrete powders, as shown in Figure 2b. The drilling hole, 16 mm in diameter,

was located in the center of specimen surface. The concrete powders were drilled with an interval from 5 mm to 30 mm away from the exposed surface (i.e., 5 mm, 10 mm, 15 mm, 20 mm, 25 mm and 30 mm depths from the surface). The chloride concentration of the concrete powders was determined by using the Rapid Chloride Test (RCT)-500 system produced by Germann Instruments company in Copenhagen, Denmark, as shown in Figure 2c. The RCT method was verified as having a good correlation with these methods recommended by ASTM C1202 [36] and AASHTOT260-97 [37].

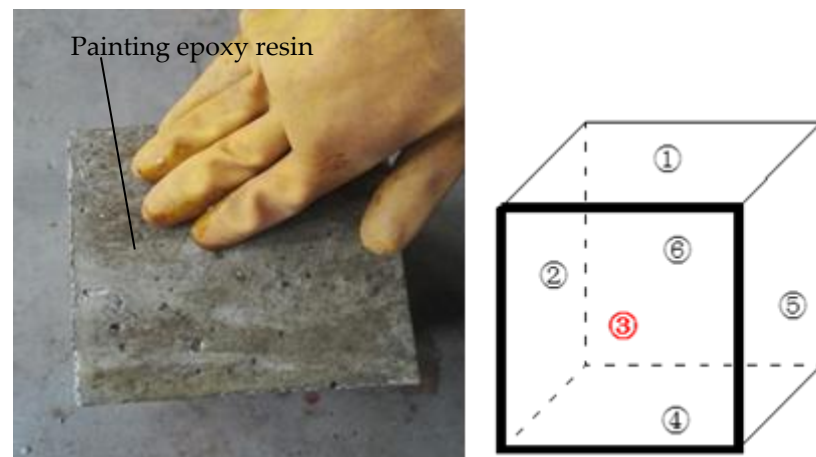


Figure 1. Process of painting epoxy resin.

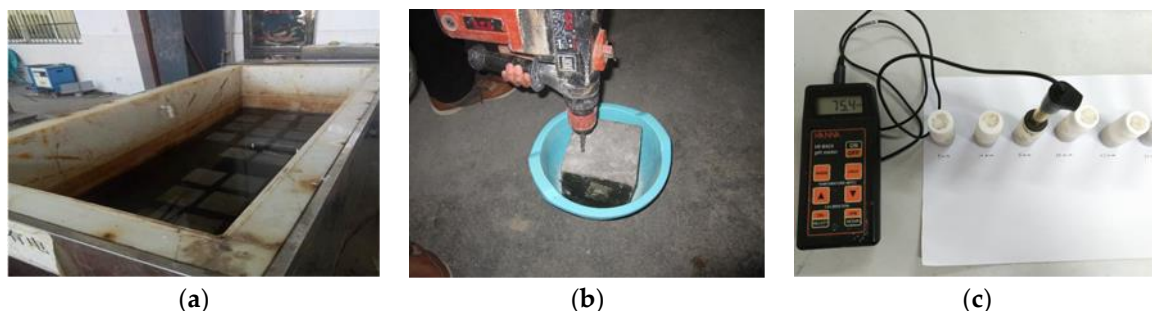


Figure 2. Chloride penetration test set-ups: (a) specimens immersing; (b) drilling holes; (c) RCT test.

3. Experimental Result Analysis

The influence of the W/B and the mixing method of FA and SF on the compressive strength and chloride concentration of HPC is discussed through experimental results.

3.1. Influence of the W/B

3.1.1. Concrete Compressive Strength

Figure 3 shows the compressive strength of Group I specimens, the value in the bracket is the W/B. As seen in the Figure 3, no matter normal concrete or the HPC, the compressive strength of specimens increases with the decrease in the W/B. For the normal concrete, the W/B decreases 35.3% from 0.34 to 0.22, the concrete strength increases 47.6% from 47.59 MPa to 70.26 MPa. For the HPC, the W/B decreases 20% from 0.25 to 0.2, the concrete strength increases 95.9% from 54.08 MPa to 105.98 MPa. The ratio of W/B decreasing value and strength increasing value for normal concrete is 0.74, but for the HPC is 0.20, so the influence of the W/B on the concrete strength for normal concrete is obviously more than the HPC.

Sengul and Tasdemir [7], Chindaprasirt et al. [38] present that the compressive strength of concrete with mineral admixtures is smaller than the normal Portland concrete under the same W/B. This result also can be verified by the compressive strength of specimens

OC70 and HPC70. However, the compressive strength of specimens HPC 60 and HPC50 is 3.5% and 13.5% larger than specimens OC60 and OC50, respectively. This can be attributed to the smaller W/B.

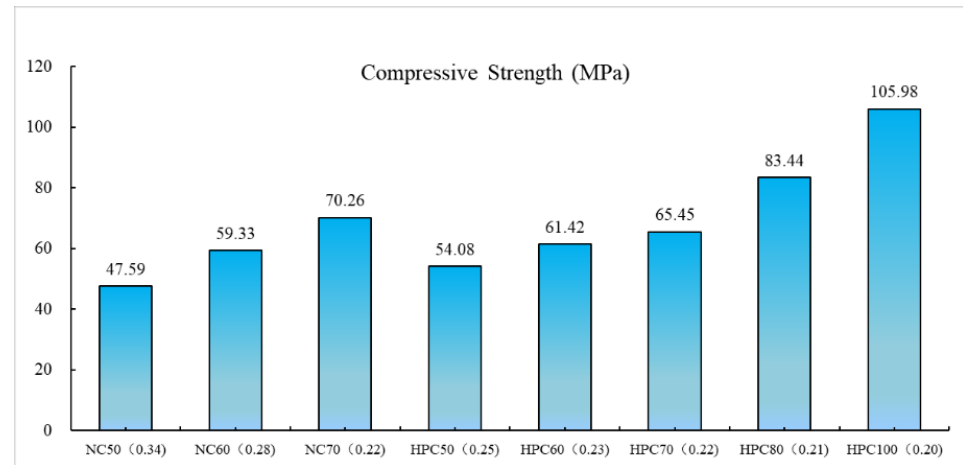


Figure 3. Compressive strength of the Group I specimens.

3.1.2. Chloride Ion Concentration (Permeability)

The W/B is an important factor impacting the chloride ion diffusion in the HPC. The chloride concentrations of part specimens were obtained as shown in Figure 4. For both the normal concrete and the HPC, from a depth of 7.5 mm to 17.5 mm, the influence of W/B on the chloride ion concentration is most significant; under the same diffusion depth, the chloride concentration decreases with the increase in W/B. The lower W/B reduces the amount of capillary pores and subsequently affects the resistance of concrete against chloride transport [6]. The concentration gradient of chloride ion is gradually decreased in the deeper regions of the concrete, and especially for depths beyond 22.5 mm, the influence of the W/B is very small.

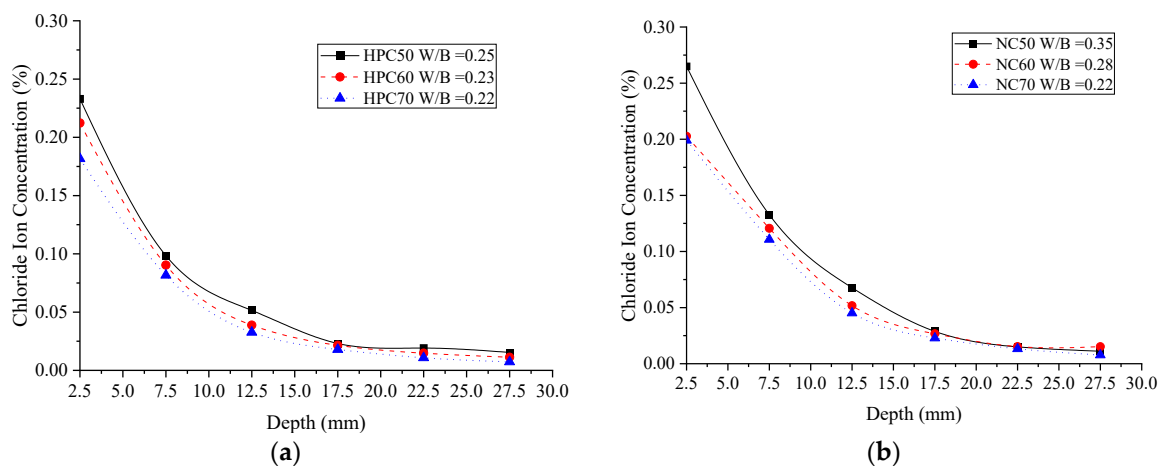


Figure 4. Chloride ion concentration of the specimens: (a) Normal concrete; (b) HPC.

Figure 5 shows the chloride ion concentration distribution of the normal concrete and HPC. From depth 0 mm to 7.5 mm, the chloride ion concentration of HPC decreases substantially compared to the normal concrete, the chloride ion concentration of specimen HPC 100 is about 16% of the specimen NC50 at depth 7.5 mm. The pozzolanic reaction is relatively slow compared to the hydration of Portland cement, which is the main reason of high early age permeability of the concretes containing pozzolanic materials. At later ages, however, the pozzolanic materials can be more effective in improving concrete properties

due to the pozzolanic reaction continuing at a higher rate for a longer period, so the chloride ion concentration of the HPC decreases slower than the normal concrete from depths of 7.5 mm to 17.5 mm.

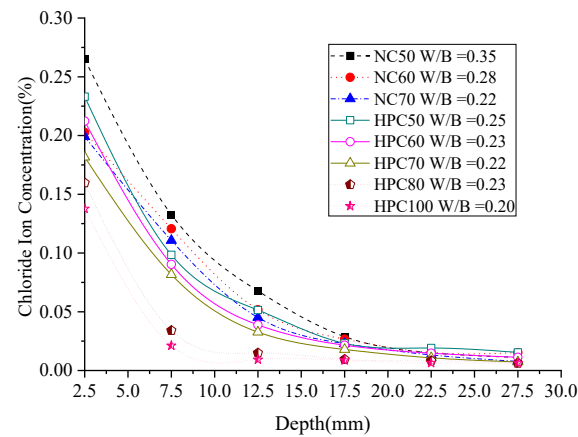


Figure 5. Chloride concentration distribution of Group I specimens.

3.2. Influence of the Mixing Method of Mineral Admixtures

3.2.1. Concrete Compressive Strength

The concrete compressive strengths of Group II specimens are presented in Figure 6. For binary combination specimens SF5, SF10 and SF15, the concrete compressive strength increases 11.93% when the content of the silica fume increases from 5% to 15%. However, the concrete compressive strength of specimen SF15 is 5.69% smaller than specimen SF10. The same situation appears for the compressive strength of specimens FA10, FA20 and FA30. The experimental results conclude that the contents of the SF or FA developing the strength is limited. Figure 7 shows the concrete compressive strength of specimens with different FA content obtained by Jerath and Hanson [39], which also concludes that the concrete compressive strength at the same age firstly increases, then decreases with the FA content increases.

Interestingly, it can be found from the strength of specimens F10-S5 to F30-S5 and F10-S10 to F30-S10, the compressive strength of specimens F10-S5 to F30-S5 decreases from 8.9% to 11.5% with the increase in FA when the content of the silica fume keeps unchanged. The compressive strength of specimens F10-S10 to F30-S10 decreases from 5.6% to 22.2% with the increase in SF when the content of the FA keeps unchanged.

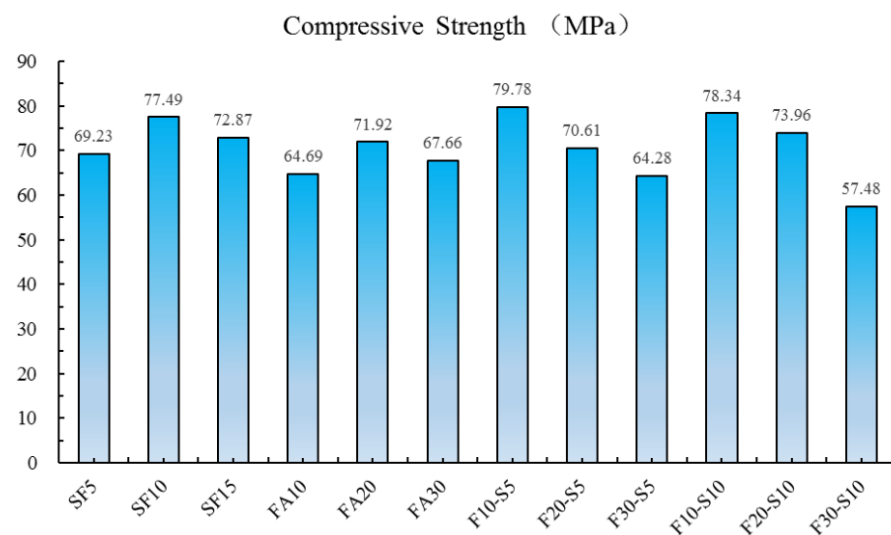


Figure 6. Compressive strength of the Group II specimens.

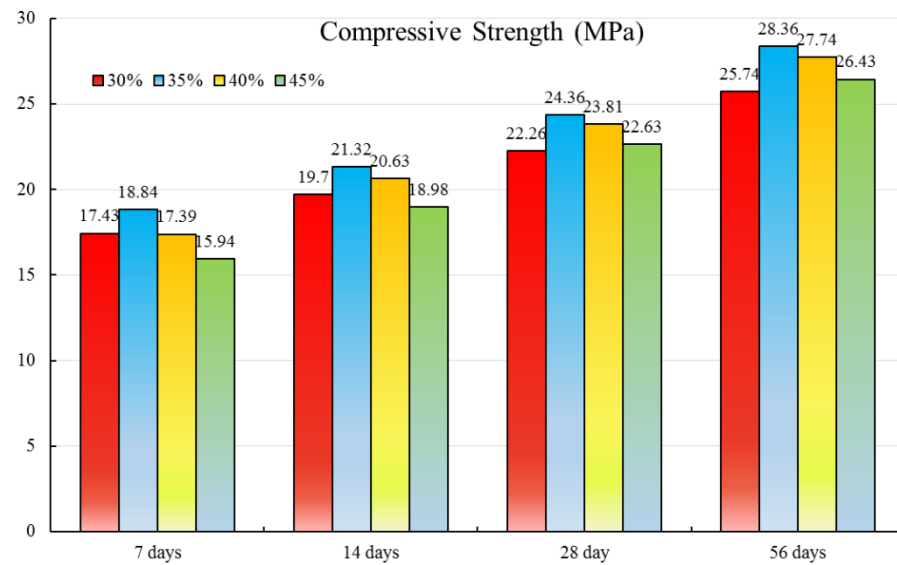


Figure 7. Compressive strength of the specimens in Jerath and Hanson's test.

3.2.2. Chloride Ion Concentration

It has been shown in laboratory and field studies that FA can mitigate corrosion by hindering chloride penetration as well as decreasing the content of free chlorides. The use of FA improves the distribution of pore size and pore shape of concrete, and the presence of C_3A in FA can absorb more chloride ions to form Friedel's salt ($C_3A \cdot CaCl_2 \cdot 10H_2O$) [40]. Moreover, fly ash reacts pozzolanically with calcium hydroxide to produce calcium-silicate hydrate (C-S-H), which absorbs more chloride ions and block the ingress path [9,40]. The SF is also more efficient in prohibiting chloride ingress inside the HPC as well as at the surface layer than round granulated blast furnace slag and metakaolin [6]. In this study, utilizing the specimens in Table 2, the influences of binary blends (Portland cement–fly ash (PC-FA) and Portland cement–silica fume (PC-SF)) and ternary blends (Portland cement–fly ash–silica fume (PC-FA-SF)) on the chloride permeability are investigated: the experimental results are shown in the Figures 8 and 9. In Figure 8, the chloride ion concentration of the specimens with FA or SF is smaller than normal concrete when the diffusion depth is smaller than 17.5 mm, but the chloride concentrations have no obvious difference beyond 17.5 mm. As shown in Figure 8a), the chloride ion concentration decreases with the FA increases from depth 2.5 mm to 12.5 mm. A similar situation also appears in Figure 8b: the chloride ion concentration decreases with the growth of the SF content from a depth of 2.5 mm to 7.5 mm.

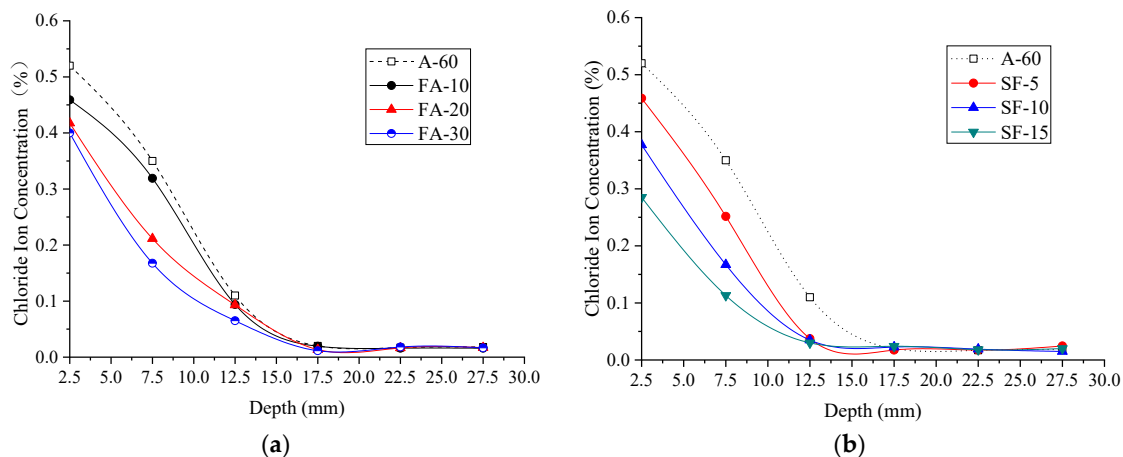


Figure 8. Chloride ion concentration of specimens with binary blends: (a) Portland cement–fly ash; (b) Portland cement–silica fume.

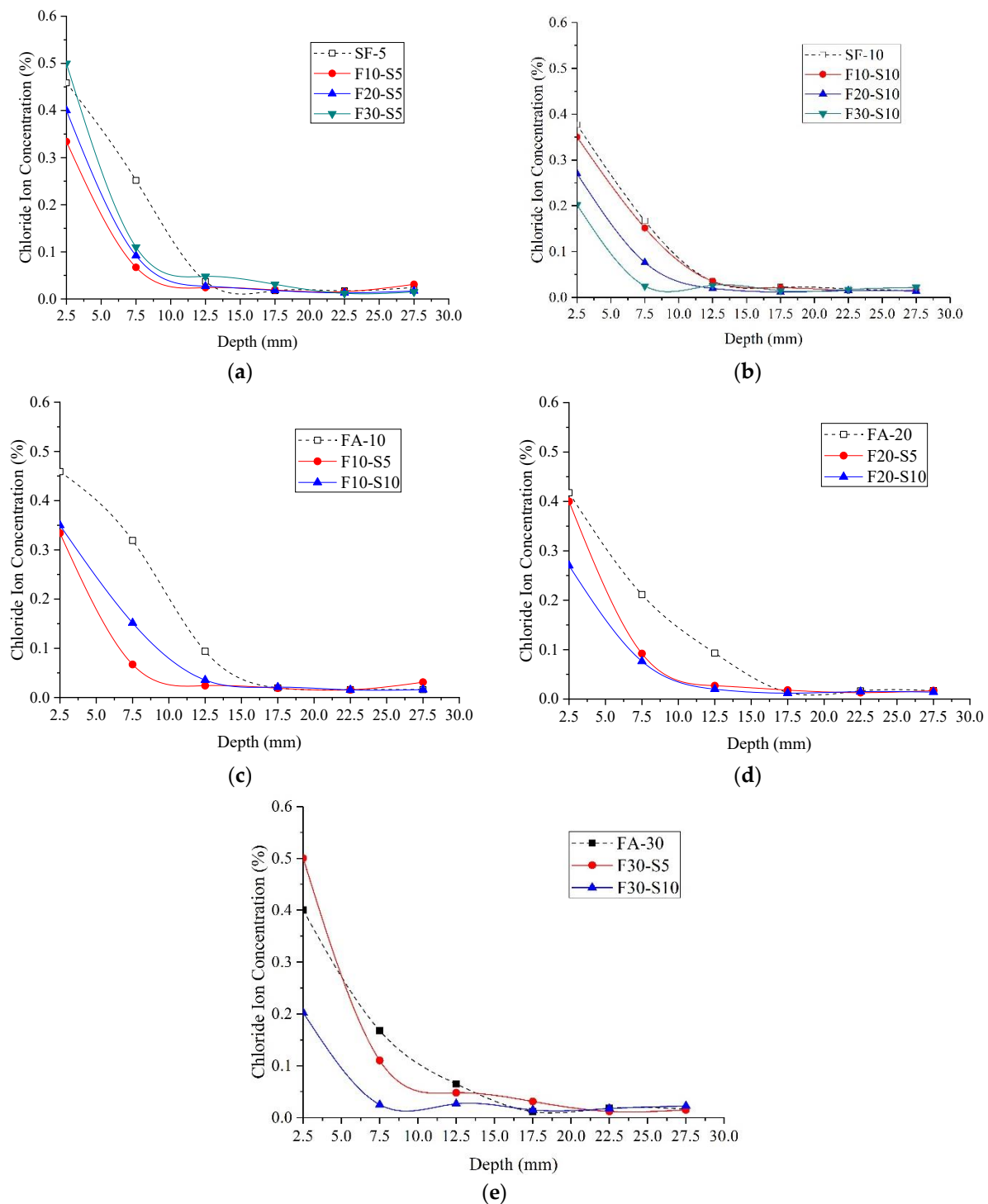


Figure 9. Chloride ion concentration of specimens with binary and ternary blends: (a,b) Portland cement–silica fume with Portland cement–fly ash–silica fume; (c–e) Portland cement–fly ash and Portland cement–fly ash–silica fume.

The profiles of chloride ion concentration of specimens with ternary blends are shown in Figure 9. When the diffusion depth is less than 12.5 mm (In Figure 9a,b,e) or less than or equal to 12.5 mm (In Figure 9c,d), the chloride ion concentration of specimens with ternary blends is smaller than specimens with binary blends under the same depth, which concludes that the ternary combination of PC-FA-SF has more efficient in prohibiting chloride's ingress inside the specimens than the binary combination of PC-FA or PC-SF.

Figure 9a presents the chloride ion concentration of specimens with different contents of FA and 5% SF. Keeping the silica fume content unchanged, the order of chloride ion concentration is F10-S5 < F20-S5 < F30-S5. When the depth is 12.5 mm, the chloride ion concentration of specimen F10-S5 is 50% less than specimen F30-S5. This situation is different from the chloride ion concentration distribution for the specimen containing only FA or SF. Interestingly, in Figure 9b, when the content of SF increases to 10%, the chloride ion concentration decreases with the FA increases under the same depth. In Figure 9c–e, the similar situation above is obviously more. When the content of FA is 10% (Figure 9c), the chloride ion concentration decreases with the SF increases. However, when the content of FA is 20% and 30% (Figure 9d,e), the chloride ion concentration decreases as the silica fume increases when the depth less than 17.5 mm. The above situation is attributed to the total content of mineral admixtures increasing, which leads to diffusion resistance improvement for improving the density of concrete.

4. Modification of the Chloride Ion Diffusion Model

4.1. Ideal Model-Fick's Second Law

Buenfeld et al. [41] considered that the progress of chloride transportation in the concrete is the unsteady diffusion process. For Fick's second law, in unit time and unit volume, the mass of material flowing out is different from that flowing into the internal structure of concrete, and the concentration of any point within it varies with time and space; this progress is also the unsteady process. Many scholars [42,43] concluded that the chloride ion diffusion process can be described by Fick's second law:

$$\frac{\partial C}{\partial t} = D \frac{\partial^2 C}{\partial x^2} \quad (1)$$

When the boundary condition is: $C(0, t) = C_s$, $C(\infty, t) = C_0$; the origin condition is $C(x, 0) = C_0$, above equation can be solved as:

$$C(x, t) = C_0 + (C_s - C_0) \left[1 - \operatorname{erf} \left(\frac{x}{2\sqrt{Dt}} \right) \right] \quad (2)$$

where $C(x, t)$ is the chloride ion concentration of the inner part concrete at time t and distance x , C_0 is the chloride ion origin concentration, C_s is the surface chloride ion concentration, D is the diffusion coefficient, $\operatorname{erf}(z)$ is error function, $\operatorname{erf}(z) = 2/\sqrt{\pi} \int_0^z e^{-z^2} dz$.

Fick's second law is an ideal model, which sets three suppositions: (1) the concrete is a semi-infinite homogeneous material; (2) the chloride ion does not react with concrete during the diffusion process; and (3) the diffusion coefficient is the constant. However, many studies have already concluded that: firstly, the concrete has micro-cracks and a void pore, and it is not a completely homogeneous material. Secondly, a chloride ion will bind with the cementitious paste and form Friedel's salt. At last, the chloride ion diffusion coefficient is not constant but time-dependent. The structure of HPC is more complicated than normal concrete because of the mineral admixture, the chloride diffusion process is affected by many factors. In this study, the diffusion coefficient was modified considering the influence of the water–bind ratio, time, temperature, humidity and deterioration effect of concrete, and a modified model is proposed.

4.2. Modified Diffusion Model

4.2.1. Relationship between the Diffusion Coefficient and W/B

It was reported that the diffusion coefficient was time-dependent, and it was affected by W/B, temperature, humidity, the combination affection between concrete and chloride ion, etc. Due to having a different composition compared with normal concrete, the diffusion coefficient should be modified. As shown in Figure 10, the normalized chloride diffusion coefficients of the HPC were obtained in this experiment and research results from

experimental results obtained by Chalee and Jaturapitakkul [8], Moffat et al. [9], Thomas and Matthews [44] and Shafq [45].

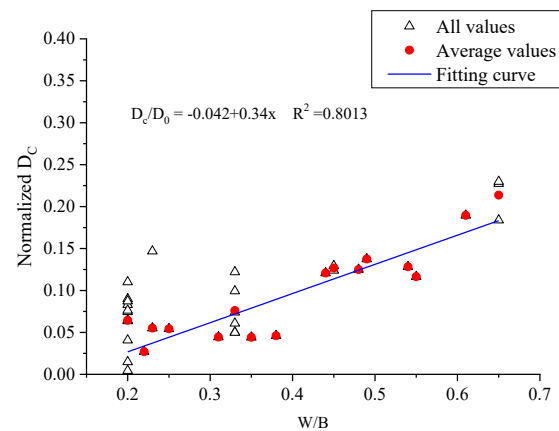


Figure 10. Chloride diffusion coefficient of concrete after different W/B.

Some scatters exist in the calculated chloride diffusion coefficients at different depths for each specimen. This could be attributed to the measurement errors of chloride concentration during the experiment. The corresponding average values, however, have relatively small scatters for the specimens with the same W/B. It can be shown in Figure 10 that the chloride diffusion coefficient shows the linear relationship with W/B, the diffusion coefficient of normal concrete is from 4.7 times to 12 times as compared with the HPC when the W/B is from 0.2 to 0.65. A coefficient is also developed to consider the W/B on the chloride diffusion coefficient based on the experimental data, which is expressed as:

$$R_C = \frac{D_C}{D_0} = -0.042 + 0.34W/B \quad (3)$$

where R_C is the coefficient considering the effect of the W/B, which is the diffusion coefficient ratio of HPC to normal concrete. D_C and D_0 are the diffusion coefficient of HPC and normal concrete, respectively.

4.2.2. Relationship between the Diffusion Coefficient and Time, Temperature

The relationship between chloride ion diffusion coefficient $D(t, T)$ and time, temperature can be presented from the experimental results obtained by Mangat and Molloy [46]. The experimental results showed that the time and temperature effect should be considered to more accurate of chloride diffusion coefficient in concrete, so the chloride diffusion coefficient can be expressed as:

$$D(t, T) = D_0 f_t f_{T_b} \quad (4)$$

$$f_t = \left(\frac{t_{\text{exp}}}{t - t_{\text{build}}} \right)^m \quad (5)$$

where $D(t, T)$ was the chloride diffusion coefficient for the real structures, f_{T_b} was the temperature correction coefficient. f_t was the time correction factor, which can be expressed by Equation (5). t_{exp} was the experimental total time, t was the calculated time, t_{build} was the construction completion time. m was the experimental parameter, which was obtained based on the different kind of cement. For normal Portland cement, m was in the range of 0.2–0.3, and for HPC with fly ash and slag, the higher values (0.5–0.7) could be used [41].

The chloride diffusion coefficient should also consider the temperature effect, which can be presented as [47]:

$$f_{T_b} = \exp \left[\frac{U}{R} \left(\frac{1}{T_{28}} - \frac{1}{T_b} \right) \right] \quad (6)$$

where U was the activated energy during the diffusion process of chloride ion in concrete and was taken as 35,000 J/mol, R was the gas constant and taken as 8.314 J/(mol.k), T_{28} was the absolute temperature corresponding to a curing of 28 days and was generally taken as 293 K, and T_b was the average absolute temperature related to calculation.

4.2.3. Relationship between the Diffusion Coefficient and Humidity

Bitaraf and Ammadis [48] pointed out that the transport of chloride ions in concrete must take pore water as continuous medium. The more pore water and the larger the section size, the smoother the diffusion of chloride ions would be. Therefore, the water content or saturation of concrete determined the number of chloride ion transport channels. At the same time, the relative humidity could be used to express the water content or saturation of concrete. When the water content inside concrete was relatively low, the chloride ion diffusion channel decreased, and its diffusion speed slowed down accordingly. Therefore, the humidity is also a factor that affects the diffusion of chloride ions: the relationship between chloride ion diffusion coefficient and humidity could be expressed as:

$$D_h = D_0 \left[1 + \frac{(1-h)^4}{(1-h_c)^4} \right]^{-1} \quad (7)$$

where D_h is the diffusion coefficient of chloride ion when the humidity is h , h_c is the critical relative humidity, and the value is 75% at the atmospheric environment, 90% at the splash zone and 95% at the tidal range zone.

In conclusion, the model of diffusion coefficient of chloride ion considering water-cement ratio, time, temperature, and humidity can be shown

$$D(t) = R_c D_0 \exp \left[\frac{U}{R} \left(\frac{1}{T_{28}} - \frac{1}{T} \right) \right] \left[1 + \frac{(1-h)^4}{(1-h_c)^4} \right]^{-1} \left(\frac{t_{\text{exp}}}{t - t_{\text{build}}} \right)^m \quad (8)$$

4.2.4. Initial Value Condition

The pouring and curing process of concrete is not eroded by chloride ion, so the chloride content inside the concrete is zero:

$$C_0(x, 0) = 0 \quad (9)$$

4.2.5. Boundary Condition

The surface chloride ion concentration of concrete is not the fixed value, but changes with time. Liu et al. [49] proposed that the relationship between surface chloride ion concentration and time can be obtained using followed equation:

$$C_s(t) = C_{\text{ref}} (1 - e^{-\alpha t}) \quad (10)$$

where C_{ref} was surface chloride concentration at the reference time t , and α was the regression parameter.

5. Random Aggregate Model of Concrete

5.1. Basic Theory

In aggregate gradation theory, the particle size and the gradation are the most important factors. In existing research, the definition of aggregate particle size is still difficult to unify and lack of practicability. Therefore, the sieving method is used to determine aggregate particle size, and the grain size can be calculated based on the upper and lower limits.

Aggregate gradation includes two aspects: particle size and the relative number of particles in each gradation. The sieving curve is used to determine aggregate gradation. Although this method can obtain the size and distribution of aggregate, it can be more

vivid through numerical characterization. Commonly the modulus of fineness and specific surface area are the numerical indicators.

The existing research results show that the concrete gradation theory can be divided into two types: continuous gradation and discontinuous gradation. If the content of aggregates with different particle sizes is close to reasonable continuous gradation curve, the strength and maximum compactness of the concrete will reach the best effect according to this gradation. In this paper, the Walraven's model [50] is used to describe the aggregate gradation, which translate the spatial spherical aggregate to the flat round aggregate based on the Fuller curve. Based on the method of probability statistics, Walraven used the cumulative distribution function to transform the three-dimensional gradation curve into the probability that an aggregate diameter $D < D_0$ for any point on an internal section, where the probability can be shown as:

$$P_c(D < D_0) = P_k[1.065(D_0/D_{\max})^{0.5} - 0.053(D_0/D_{\max})^4 - 0.012(D_0/D_{\max})^6 - 0.0045(D_0/D_{\max})^8 - 0.0025(D_0/D_{\max})^{10}] \quad (11)$$

where D_0 was sieve diameter, $D_{\max}$ was maximum aggregate particle size, P_k was the percentage of aggregate volume in the total volume, the value was 0.75. The probability P_c of the different aggregate gradation of the two-dimensional section of the specimen could be obtained by replacing the boundary value of the three-dimensional aggregate content P_K with the diameter of the aggregate at all levels in formula (11), and then the two-dimensional gradation curve could be obtained.

5.1.1. Circular Aggregate Generation and Release Process

The release process of circular aggregate is introduced in this paper firstly. The radius of the circular aggregate is assumed to be R_i , and its corresponding circular coordinate is (x_i, y_i) , then the calculation method of the geometric model of the aggregate is as follows:

- (1) Defining the radius and the centroid coordinates of aggregate (x_i, y_i) . The x_i and y_i need to satisfy the following requirements: $x_i = \frac{d}{2} + (a - d) \cdot \eta$, $y_i = \frac{d}{2} + (b - d) \cdot \eta$. Where, a and b are the side length of release area, respectively, $a = b = 150$ mm in this paper. η is a random number between 0 and 1.
- (2) Overlapping judgment. This process is determining whether the generated i aggregate overlaps with the generate $i - 1$ aggregate. If un-overlapped, the area of the aggregate is calculated and the information of the aggregate is recorded; if overlapped, turning back to the step (1) and recoding the value. The basis of overlap judgment is: If the interval length d between two circular aggregate centroid is larger than the sum of their radius, namely $d > R_1 + R_2$, it can be concluded that the circular aggregate with radius R_1 does not overlap with the circular aggregate with radius R_2 .
- (3) Judging of the aggregate area. If the condition $\Sigma A_i \geq A_0 \cdot a \cdot b$ is not met, the researcher should be turn back to step (1) and continue to release. If the above condition is met, the aggregate is finished. Where A_0 is the area fraction of the calculated aggregate.

5.1.2. Polygonal Aggregate Generation and Release Process

In this paper, the generation process of polygonal random aggregate is realized as follows: Firstly, based on the simulation of circular aggregate, the diameter of random circle is assumed as the particle size of polygonal aggregate, and the location and size of the aggregate are determined by using the randomly generated circle. Secondly, a polygonal base aggregate is generated randomly in the random circle, and the area of the polygon is greater than or equal to 0.75~1.1 times of the area of the random circle. Finally, the longest edge is set as the reference edge, the polygonal random aggregate model is established by using the rule that the reference edge generates new points and determines whether the new points meet the outward extension condition.

5.1.3. Examples of Random Aggregate

The content of aggregate in each tested block was calculated based on the different mix proportion of test specimens in Table 1 and then the geometric model of circular aggregate was generated by MATLAB 2021a programming language according to the random number theory using the Monte Carlo method.

For a circular aggregate, the area S (volume fraction of aggregate) of a certain particle size in the two-dimensional model is calculated according to Equation (11) according to the size and grade of aggregate particle size, and then the amount of aggregate corresponding to each diameter is further determined. In the section of the specimen, the position of randomly generated aggregate centroid was firstly determined, and then circular aggregate is randomly generated according to the corresponding aggregate diameter at that position. Finally, the aggregate of all particle sizes is released, that is, the establishment of circular random aggregate model was completed.

The mixing ratio of each specimen is shown in Table 1. According to the mixing ratio of each specimen in test, the volume percentage ratio of aggregate to concrete P_k can be obtained, as shown in Table 3.

Table 3. Proportion of aggregate volume to concrete for each strength grade.

Specimen	Mass of Aggregate	Volume of Concrete	Volume of Aggregate	Proportion of Aggregate Volume
NC50	6.08056875	0.003375	0.002252063	0.67
NC60	6.07746375	0.003375	0.002250913	0.67
NC70	6.074510625	0.003375	0.002249819	0.67
HPC50	6.077919375	0.003375	0.002251081	0.67
HPC60	6.07624875	0.003375	0.002250463	0.67
HPC70	6.074274375	0.003375	0.002249731	0.67
HPC80	5.882625	0.003375	0.00217875	0.65
HPC100	5.886	0.003375	0.00218	0.65

Specimens NC50 and HPC80 are used to build the randomly geometric of circular and polygon aggregates. Table 4 shows the distribution probability of aggregate particle size of specimens NC50 and HPC80. Combining the proportion of aggregate volume with the Equation (11), the probability P_c that aggregate diameter D less than D_0 in any cross section is calculated, then the aggregate number for every diameter is also confirmed. Figure 11 show the circle aggregate model of specimen NC50 and HPC80.

Table 4. Distribution probability of particle size of each aggregate in the specimens NC50 and HPC80.

Specimen	D_0	22	18	12	5
NC50	$D_0/D_{\max}$	1	0.81	0.55	0.23
	P_c	0.65	0.61	0.51	0.33
HPC80	$D_0/D_{\max}$	1	0.81	0.55	0.23
	P_c	0.63	0.59	0.49	0.32

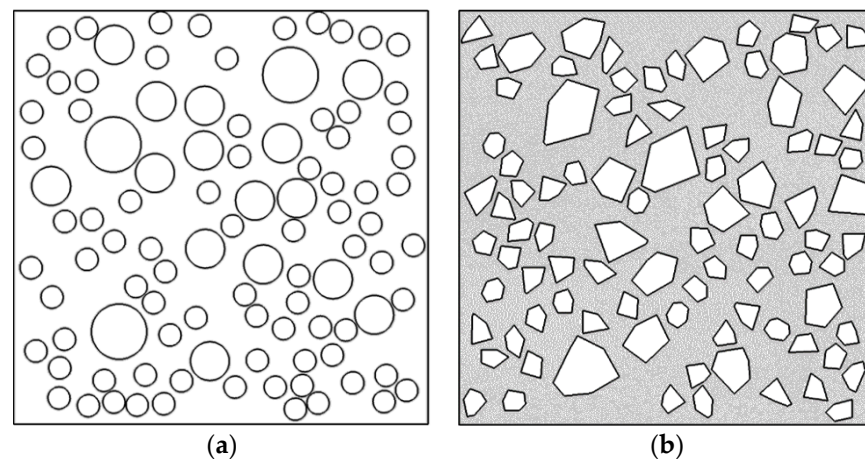


Figure 11. Circle aggregate model of specimens: (a) NC50; (b) HPC80.

6. Mesoscopic Simulation of Chloride Ion Diffusion

Simulation Method

Software COMSOL Multiphysics 5.1 provides modules for describing and analyzing various physical fields which including chemical transfer, fluid flow, and dynamic and solid mechanics. The Transport of Diluted Species Interface in this software is suitable to simulate the material transfer process under the consideration of convection–diffusion and diffusion processes, so the chloride ion diffusion process in high performance concrete in this paper is simulated using this module. The simulation steps are shown as follows:

- i. Choosing the Transport of Diluted Species Interface in the COMSOL software and importing the two-dimensional concrete random aggregate geometric model proposed in Part 4 into the software and set the cell.
- ii. Defining the parameter value in transfer equation and diffusion coefficient model based on the Equation (11).
- iii. Defining the boundary condition C_s , diffusion coefficient and initial value condition C_0 .
- iv. Dividing the mesh.
- v. Defining a two-dimensional transversal and extracting the chloride ion concentration on the transversal lines of 2.5 mm, 7.5 mm, 12.5 mm, 17.5 mm, 22.5 mm, and 27.5 mm, respectively.

Dividing the Mesh

Of the above steps, dividing the mesh is the most important. In this paper, concrete is regarded as a three-phase composite composed of aggregate, mortar, and the transition layer between them. The COMSOL software is used to carry out the grid-process for the two-dimensional random aggregate model generated in Part 4, and then the two-dimensional random aggregate finite element model is established. According to the research result from Basheer et al. [51], the thickness of transition layer is from 0 μm to 10 μm , in this paper, the thickness of the transition layer in the three-phase random aggregate model is 75 μm . The finite element model is shown in the Figure 12.

The process of mesh generation in COMSOL Multiphysics is defined by mesh sequence, which consists of operation characteristics and attribute characteristics. Among them, operation features are mainly for mesh type, copy mesh, refine mesh, or transform mesh, while attributed features are mainly for mesh type related features such as size, distribution, and proportion. Therefore, when creating a mesh in software, the operation characteristics of the mesh need to be defined first to generate or modify the corresponding geometry of the mesh. Then, the local attribute characteristics is defined. When adding local attribute characteristics in the software, the selected local attribute characteristics will appear on the corresponding sub-nodes under the operation characteristics node, especially when the

selected target is the same. The global attribute node is automatically overwritten by the local attribute feature node.

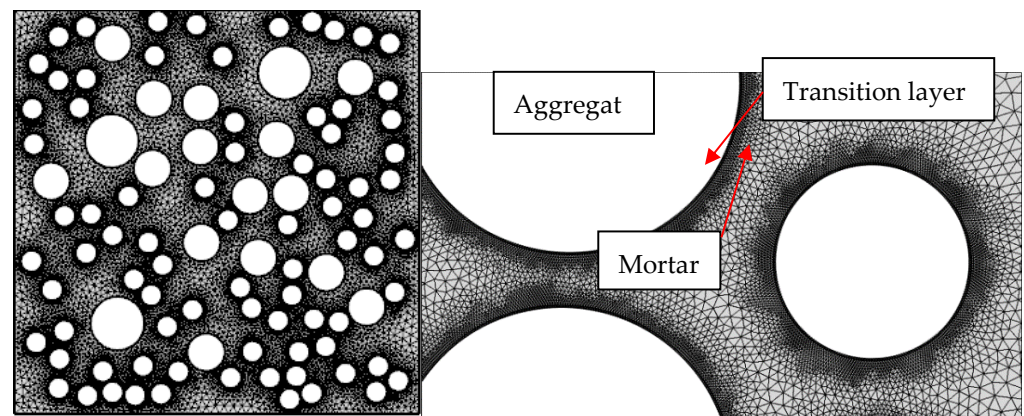


Figure 12. Random aggregate finite element model.

7. Microscopic Model Validation and Parameters Analysis

7.1. Model Validation

In order to verify the reliability and accuracy of the microscopic model in this paper, the measured concentration value in the experiment is compared with simulation value. In the specific test process, due to the presence of chloride crystals on the surface of the specimen and chloride salts in the micro-pores, it is impossible to accurately determine the chloride ion concentration on the concrete surface. Therefore, the chloride ion concentration at 2.5 mm is regarded as the chloride ion degree on the concrete surface, that is, chloride ion begins to diffuse into the concrete interior from 2.5 mm. Figure 13 shows the chloride ion concentration of test and microscopic simulation at different depth. Generally, it can be found from Figure 13 that the simulation and the tested value of the chloride ion concentration under the same depth is close, the average ratio of simulation value to tested value is 0.91. The simulation and tested chloride ion concentration of the specimens HPC60, HPC80, and HPC100 under the depths of 7.5 mm and 12.5 mm is slightly different, it may be the reason that the mortar content is not uniform.

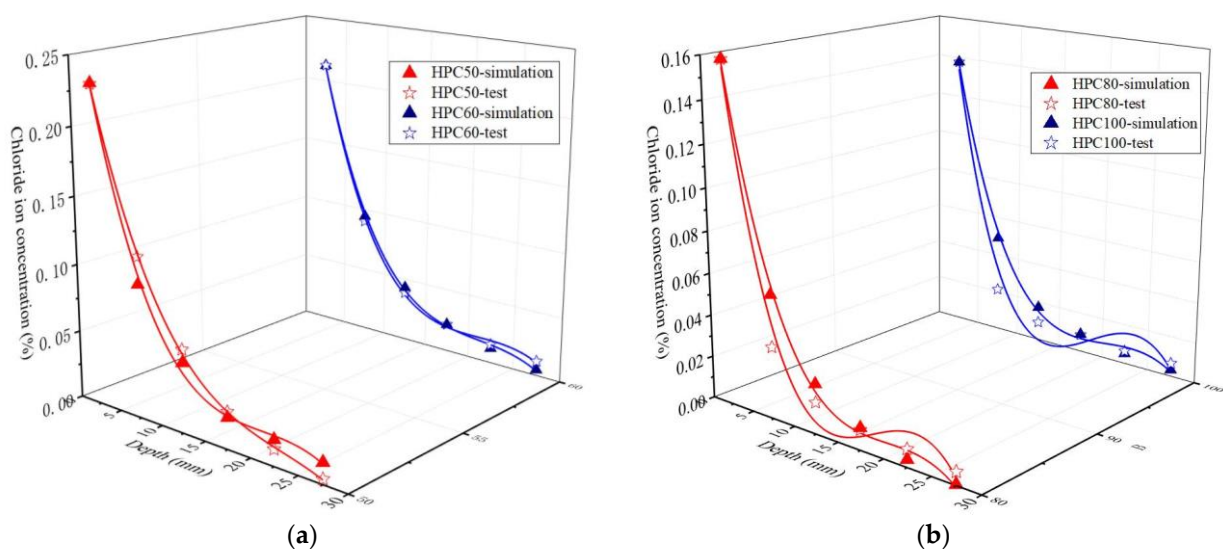


Figure 13. Chloride ion concentration of test and microscopic simulation at different depth: (a) HPC50 and HPC60; (b) HPC80 and HPC100.

7.1.1. Effect of Aggregate Shape on Chloride Ion Permeability

In the mesoscopic model, the content, particle size, and position of aggregate are kept the same, and the shape of aggregate is changed to study its influence on the diffusion of chloride ion in concrete. The surface chloride ion concentration is 0.6, the thickness of transition layer is 75 μm , and the diffusion time is 88 days. The numerical simulation results of specimens HPC50 with circle and square aggregate are shown in Figure 14. Figure 14 shows the chloride ion concentration of specimen HPC50 with different aggregate shapes, the chloride ion concentration at the same depth in the Figure 15 is the average value of all chloride ion concentrations at a boundary.

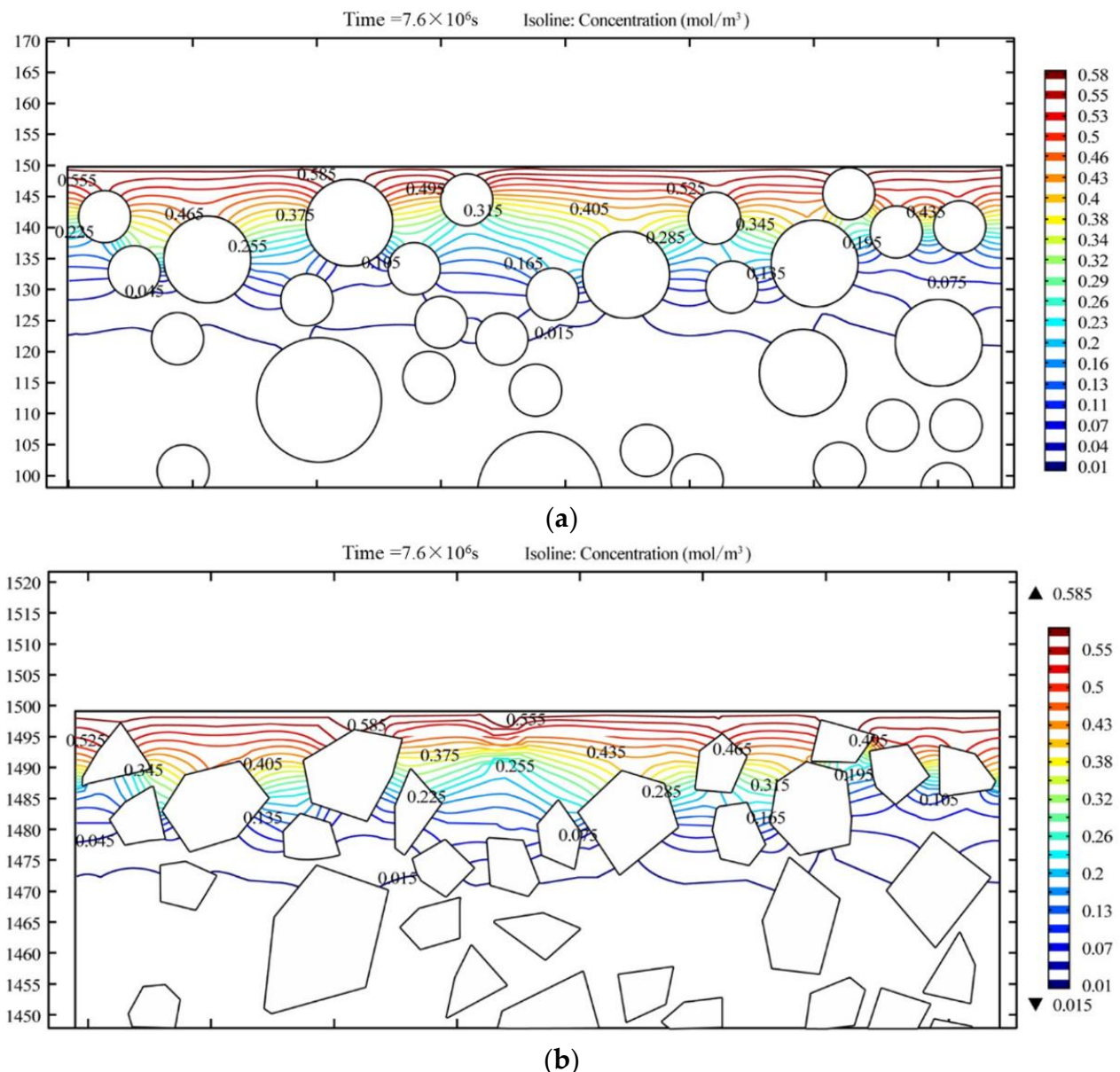


Figure 14. Numerical simulation results of specimens HPC 50 with circle and square aggregate: (a) Circle aggregate; (b) Square aggregate.

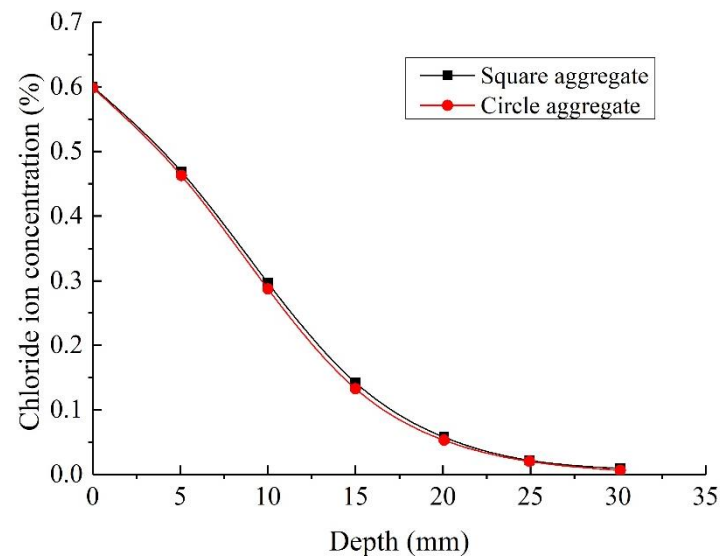


Figure 15. Chloride ion concentration of specimen HPC50 with different aggregate shape.

It can be found from Figure 15 that no matter the circle aggregate or square aggregate, the average chloride ion concentration under the depth from 0 mm to 30 mm is close. The average deviation ratio of chloride ion concentration between square aggregate and circle aggregate is 10%. Figure 14 shows that the aggregate shape effects the chloride ion concentration around the aggregate. From the above analysis it can be concluded that the aggregate shape has little influence on the chloride ion diffusion process in the concrete inside. This conclusion is same with normal concrete in Du's study [27]. Du et al. [27] found that aggregate shape has a negligible influence on chloride diffusivity in concrete.

7.1.2. Effect of Aggregate Distribution on Chloride Ion Permeability

Due to the influence of shape of aggregate on the chloride ion diffusion in concrete is not obvious, the circle aggregate is taken as an example in the microscopic model of HPC50. The content and particle size of aggregate are kept the same, and only the distribution of the aggregate is changed to study its influence on the chloride ion diffusion in concrete. The surface chloride ion concentration is still 0.6, the transition layer thickness is still 75 μm , and the erosion time is still 88 days. The numerical simulation results are shown in Figure 16. The average chloride ion concentration of specimen HPC50 with different aggregate distributions under the same depth is presented in Figure 17.

It can be seen from Figure 16 that aggregate only affects the transport path and director of chloride ions in concrete, and has little effect on the average chloride ion concentration at the same depth. Figure 17 also shows that the chloride ion concentrations between the different aggregate diffusion are close except under the 15 and 20 mm. The average deviation ratio of chloride ion concentration between Position 1 and Position 2 is 18.9%. Above finding is also similar with normal concrete in Du et al.'s study [27].

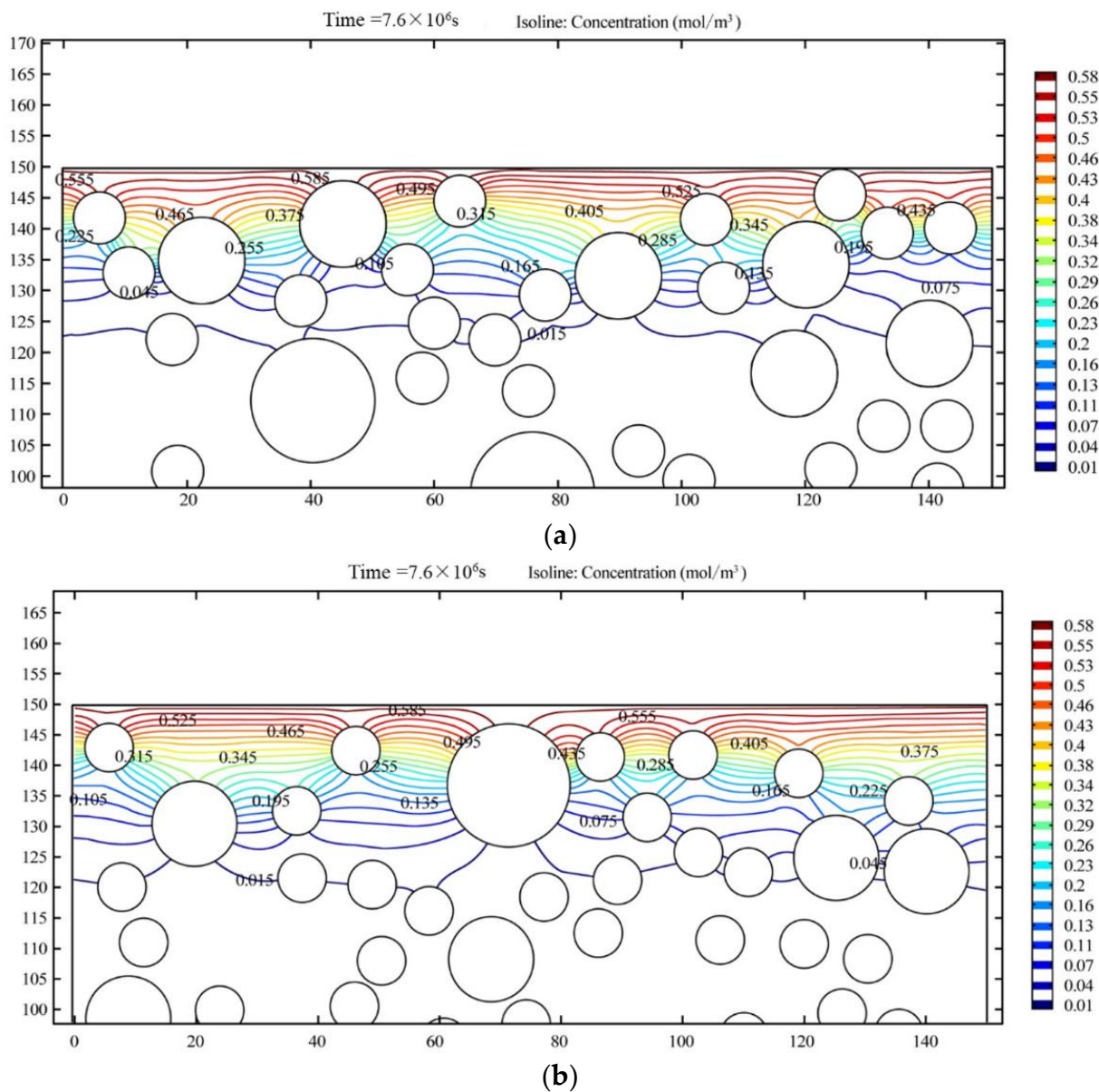


Figure 16. Numerical simulation results of specimens HPC 50 with different aggregate distribution: (a) Position 1; (b) Position 2.

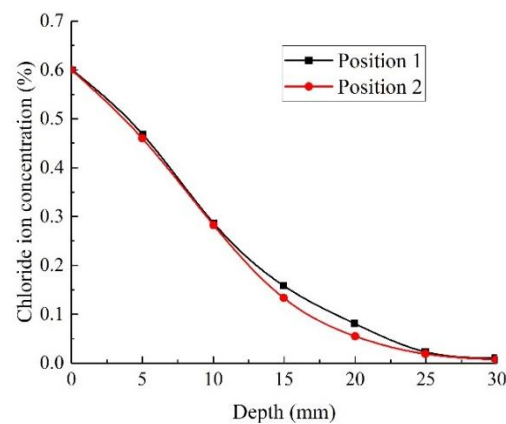


Figure 17. Chloride ion concentration of specimen HPC50 with different aggregate distribution.

7.1.3. Effect of Aggregate Size on Chloride Ion Permeability

Keeping the content of aggregate unchanged, the influence of aggregate size on the chloride ion concentration has been studied. The aggregate sizes in Figure 18a are 20 mm, 14 mm and 7.5 mm (Size 1), which are 30 mm, 15 mm, and 7.5 mm (Size 2) in Figure 18b. The surface chloride ion concentration is still 0.6, the transition layer thickness is still 75 μm , and the erosion time is still 88 d. The chloride ion distribution and average chloride ion concentration of specimen HPC50 with different aggregate size under the same depth are presented in Figure 19.

It can be seen from Figure 18 that when the aggregate size increases, and the proportion of aggregate remains unchanged, the aggregate number in concrete specimens will decrease, and the spacing between adjacent aggregates increases, which will be more conducive to the diffusion of chloride ions in the concrete.

Figure 19 also shows that when aggregate accounts is unchanged, only the particle size of aggregate is changed, the chloride ion concentration increases with the increase in aggregate size. With the increase in diffusion depth, the chloride ion concentration difference also increases gradually, the concentration difference is maximum when the diffusion depth is 15 mm. When the diffusion depth is greater than 15 mm, the concentration difference decreases, and the concentration tends to be equal due to at the depth 30 mm due to the limit of diffusion depth.

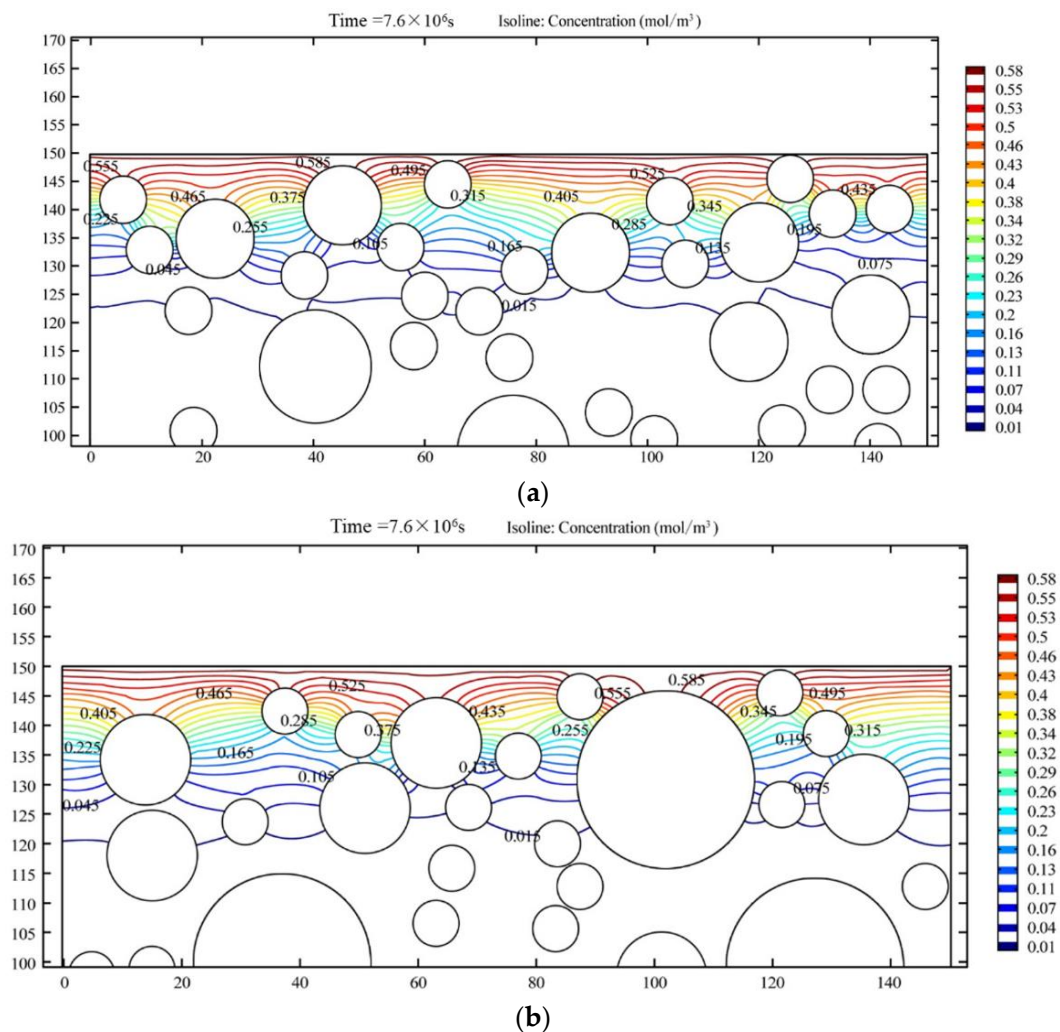


Figure 18. Chloride ion concentration distribution of specimen HPC50 with different sizes: (a) Size 1; (b) Size 2.

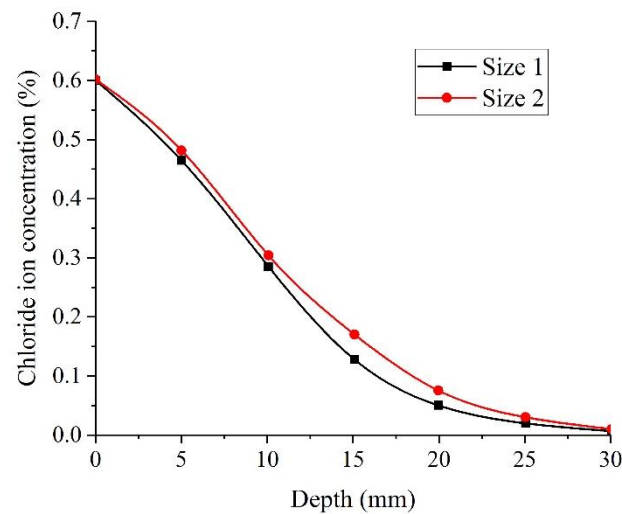


Figure 19. Chloride ion concentration of specimen HPC50 with different aggregate size.

8. Conclusions

In this paper, the chloride ion diffusion experiment for the HPC is carried out. The influence of W/B, binary (PC-FA and PC-SF), and binary (PC-FA-SF) combinations on the concrete compressive strength and chloride ion diffusion is investigated. The chloride ion diffusion coefficient is modified and the chloride ion diffusion model of the HPC is revised based on the Fick's second law considering the W/B, temperature, humidity, and time. A mesoscopic model is developed to investigate chloride diffusivity in concrete, the simulation results are compared with previous test data. The influence of aggregate shape, distribution, and size of HPC on the chloride ion diffusion is investigated. Some meaningful conclusions are obtained as follow:

- (1) The influence of the W/B on the concrete strength for normal concrete is obviously more than the HPC. The contents of the SF or FA developing the compressive strength is limited. The concrete compressive strength of ternary combination specimens decreases with the increase in FA or SF when the content of the other mineral admixture SF or FA keeps unchanged.
- (2) No matter normal concrete or the HPC, from depth 7.5 mm to 17.5 mm, the influence of W/B on the chloride ion concentration is most significant, under the same diffusion depth, the chloride concentration decreases with the increase in W/B. The ternary combination of PC-FA-SF is more efficient in prohibiting chlorides ingress inside the specimens than the binary combination of PC-FA or PC-SF.
- (3) The mesoscopic simulation and the tested value of the chloride ion under the same depth is close, the average ratio of simulation value to tested value is 0.91. The aggregate shape and distribution have a negligible influence on chloride diffusivity in the HPC, but the chloride ion concentration increases with the increase in aggregate size.

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Article

Analysis of the Mechanical Properties of a Bidirectional Laminated Slab with Shear Keys

Ming Li ¹, Zhijun Zhou ^{1,*}, Qian Wu ^{1,*}, Wei Yan ¹ and Shuang Yuan ²

¹ School of Civil Engineering, Shenyang Jianzhu University, Shenyang 110168, China

² School of Construction Engineering, Tianjin University, Tianjin 300072, China

* Correspondence: me572742890@163.com (Z.Z.); 18624028257@163.com (Q.W.);
Tel.: +86-024-24691800 (Q.W.)

Abstract: The load-bearing capacity of a bidirectional laminated slab with shear keys arranged in a rectangular grid (BCSWSK) was investigated to determine its mechanical properties, using load tests on a fabricated laminated slab and finite element models. The shear force distribution across the shear keys was measured and analysed, and the effects of different parameters were identified. The interfaces between the cast bottom slab and the shear keys were strongly bonded, as was the interface between the precast bottom slab and the cast-in-place upper slab, despite it being a secondary concrete pouring. The shear force was distributed similarly along the X direction (columns) and the Y direction (rows) of the shear key arrangement. The shear forces along the Y direction were greater and reached a maximum value sooner, but the differences between columns were mostly less than 10%. A square cross-section is recommended for the shear keys. The number of rows, number of columns, and the cross-sectional area of the shear keys are the main factors influencing the mechanical properties of the composite slab, but as they individually increased, they reached a point at which further increase had little effect. Similarly, after a certain number of shear keys were used, the concrete grade cast in situ, the shear key concrete grade, and the friction coefficient between the precast bottom slab and the cast-in-place upper slab had little influence on the BCSWSK.

Keywords: laminated slab and plates; composite slab and plates; shear key; influencing factor; element modelling

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1. Introduction

A laminated concrete slab consists of a prefabricated bottom slab (PBS) and a cast-in-place concrete upper slab (CCUS). Its advantages over conventional cast-in-place slabs are ease of construction, green and low-carbon technology, and high-production efficiency. It is an important horizontal load-bearing member of assembled concrete structures and has been widely used in housing construction [1]. The most commonly used forms of laminated slabs are steel-truss-laminated slabs (STLSs) (Figure 1a) and ribbed laminated slabs (RLSs) (Figure 1b). Of which, the steel truss is made automatically by machine. An in-depth study shows that, in conjunction with the wider uses of STLSs, the increased use of steel reinforcement in them leads to greater slab thickness [2], resulting in higher construction costs. The ribs of RLSs extend for the full length of the slab, which can be problematic in crossing pipelines. Furthermore, the PBSs of them must be temporarily supported during construction due to their limited stiffness. To overcome the problems associated with the use of these types of laminated slabs, a concrete-laminated slab with shear keys (CSWSK) was proposed to connect the PBS to the CCUS (Figure 2a). The shear keys can also be connected to temporary I-beams to increase the stiffness of the PBSs to realise support-free construction (Figure 2b).

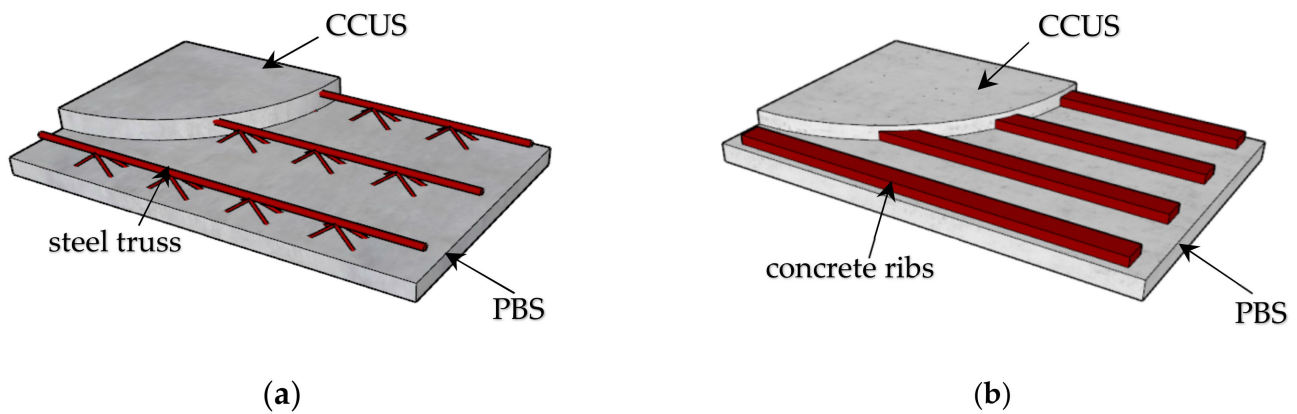


Figure 1. The most commonly used forms of laminated slab: (a) STL; (b) RL.

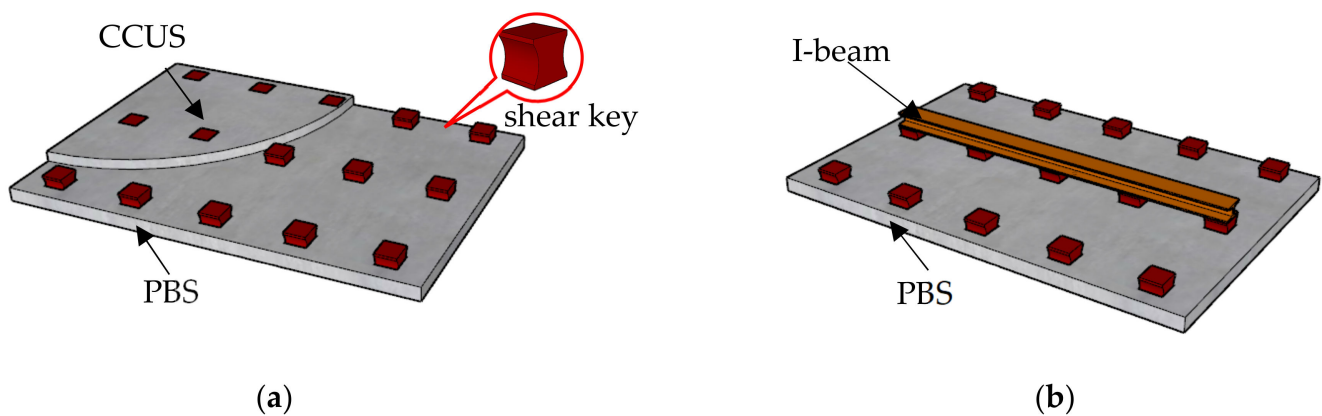


Figure 2. CSWSK and precast bottom slab with I-beam bolted to it: (a) CSWSK; (b) PBS with I-beams bolted to it.

There are large bodies of systematic research on each [3–11]. Liu et al. [12] confirmed the practicality of using STLs by empirical testing; Ma et al. [13] investigated its mechanical properties in construction and in service by testing and numerical modelling; Zhao et al. [14] analysed the influence of parameters, such as concrete strength and truss height and diameter, on the mechanical properties of STLs using ANSYS software; and Cheng [15] developed a short-term stiffness model for STLs from theoretical analysis. Wu et al. [16] confirmed the practicality of using RLSs by empirical testing; Qian et al. [17] experimentally investigated its mechanical properties, such as the flexural-bearing capacity and the shear resistance of the laminated surface, in construction and in service; Shi et al. [18] analysed the effects of parameters, such as rib height, and rib width and reinforcement rate, on the mechanical properties of RLSs using a finite element method and experimental testing; and Huang et al. [19] developed deflection and bending moment equations for RLSs in service, using load superposition and an orthotropic and anisotropic plate theory. However, for CSWSKs, only we have conducted a series of studies on its unidirectional mechanical properties [20,21], and this paper continues that series by investigating the mechanical properties of bidirectional laminated slabs with shear keys (BCWSKs), including the flexural-bearing capacity, damage modes, and influencing factors, using experimental tests and finite element modelling. A flowchart of this study is shown in Figure 3.

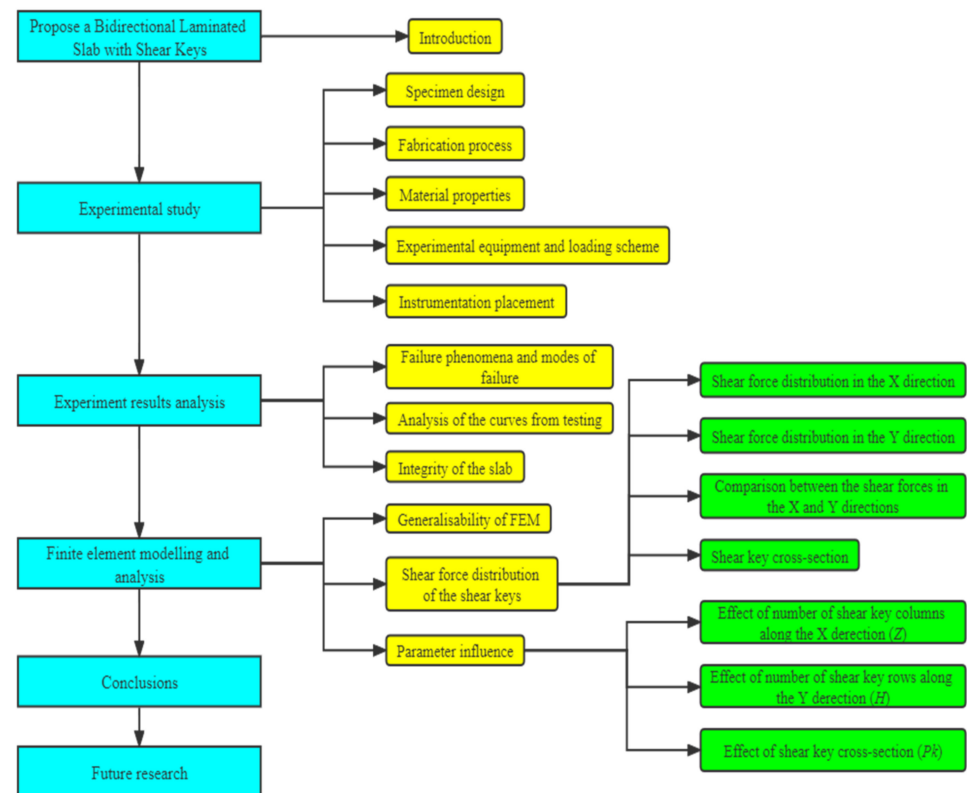


Figure 3. Flowchart of this study.

2. Experimental Study

One BCSWSK was designed and fabricated, according to standards GB (50010-2010) [22] and JGJ (1-2014) [23], to be incorporated into an existing structure. The shear keys were designed according to finite element modelling, of which the generalisability was not verified. It was tested in the Civil Engineering Structure Test Centre (Shenyang, China) of Shenyang Jianzhu University.

2.1. Specimen Design

The dimensions and form of the shear keys used in the BCSWSK are shown in Figure 4. The vertical faces were concave arcs, and the top and bottom faces were flat. The concave arc faces were designed to prevent the PBS and the CCUS from separating in the direction perpendicular to them. The concrete grade of the PBS and the CCUS was C25, and the depth/thickness of each was 50 mm. The concrete grade of the shear keys was C30; this was greater than the strength of the two slabs to prevent the keys from failing before the slab failed. The PBS is shown in Figure 5. The reinforcements had a diameter = 8 mm and tensile yield strength = 335 MPa. Field curing was adopted for the specimen.

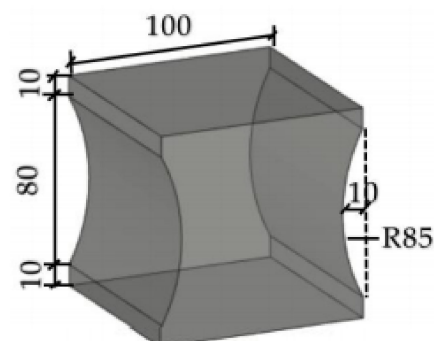


Figure 4. Dimensions and shape of shear key.

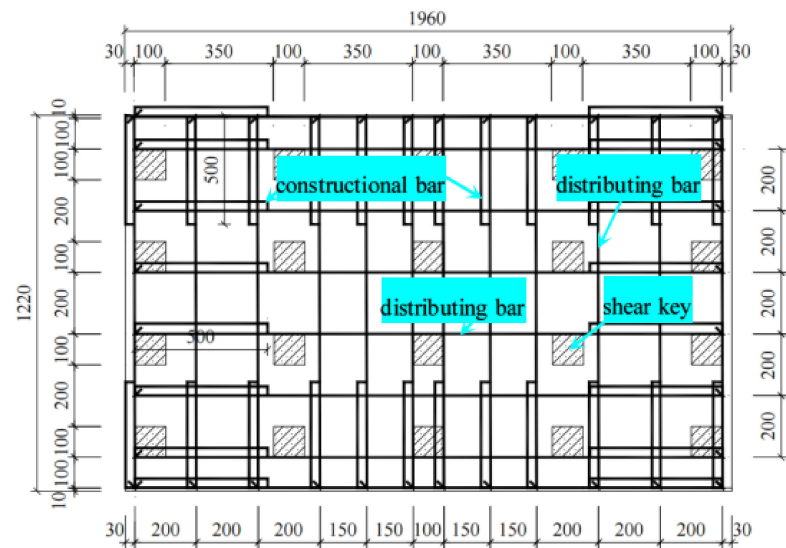


Figure 5. PBS of BCSWSK.

2.2. Fabrication Process

The fabrication process of the BCSWSK was as follows. The shear keys were made by cutting a long columnar concrete strip. The steel-reinforcing bars and shear keys were positioned in the mould, and concrete was poured to form the precast bottom slab (Figure 6a). After 28 days, the concrete of the CCUS was poured on top of the PBS to complete the lamination. The finished BCSWSK is shown in Figure 6b.

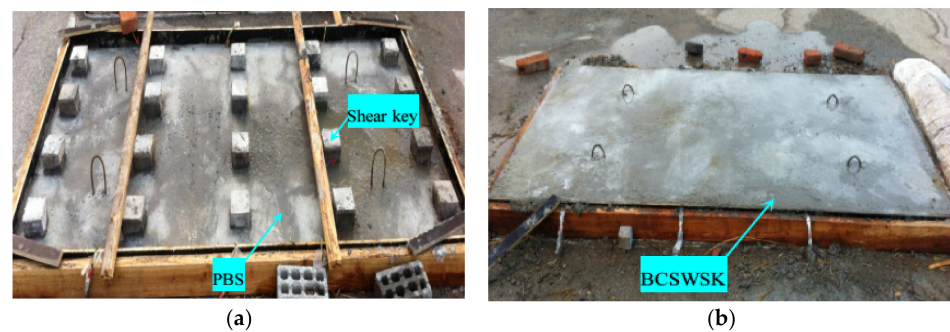


Figure 6. The production process for BCSWSK: (a) Precast bottom slab of BCSWSK; (b) BCSWSK.

2.3. Material Properties

The material properties of the steel bar and concrete were tested separately in the test rigs shown in Figure 7a,b. The compressive strength of the concrete (f_{ck}), the yield strength of the steel bar (f_y), the ultimate strength of the steel bar (f_u), and the corresponding Young's modulus ($E_c(E_s)$) are shown in Table 1.



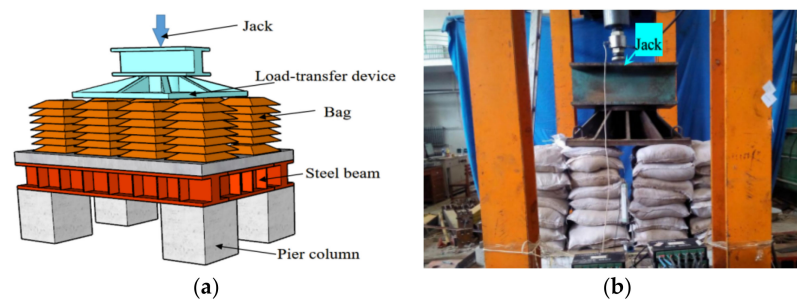
Figure 7. Steel bar and concrete materials tests: (a) Test rig for tensile strength of steel bar (HRB335); (b) Test rig for compressive strength of concrete.

Table 1. Measured mechanical property indexes of concrete and steel bar.

Batch	Material	f_{ck} (MPa)	f_y (MPa)	f_u (MPa)	$E_c(E_s)$ (MPa)
Precast bottom slab	Concrete	21.4	–	–	2.80×10^4
Post pouring concrete	Concrete	26.8	–	–	2.80×10^4
Shear keys	Concrete	27.6	–	–	3.00×10^4
HRB335	Steel bar	–	350	460	2.0×10^5

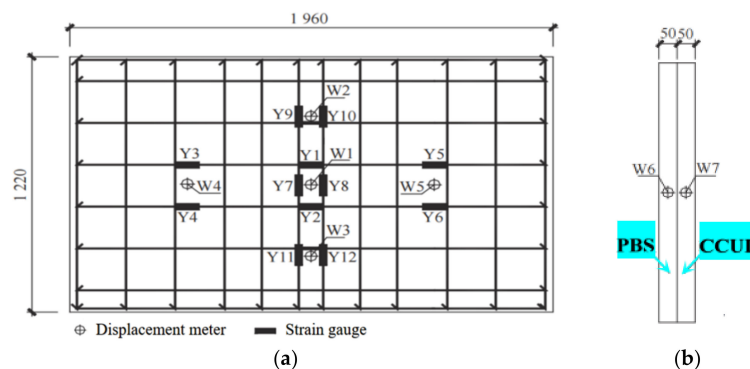
2.4. Experimental Equipment and Loading Scheme

The testing used a combination of sandbag loading and jack loading; the concentrated load exerted by the jack was transmitted to the slab by the sandbags. The loading device is shown in Figure 8. The slab was supported as follows. Two adjacent sides of the slab were supported by the immovable hinges of a triangular steel pipe, and the other two adjacent sides were supported by the rolling hinges of a cylindrical steel pipe. A steel slab was set between the bottom of the precast concrete slab and the hinge supports to avoid local compression failure. Seven layers of sandbags were stacked on the top slab surface, with eight bags per layer and each sandbag weighing 25 kg. A load-transfer device was placed on top of the sandbags. A jack was applied to the steel mat to apply the test load. For the loading system, the initial, maximum, and final settlement values were 0, 500, and 199 kN.

**Figure 8.** Loading system: (a) Schematic; (b) Photo.

2.5. Instrumentation Placement

Five displacement meters designated W1 to W5 were arranged symmetrically at the midpoints and the quarter points of the length and width of the slab bottom to measure the vertical displacement of the BCSWSK under vertical load. Strain gauges designated Y1 to Y12 were attached close to the midpoint and quarter points of the reinforcement to measure the strain of the tensile reinforcement at the bottom of the slab (Figure 9a). Displacement gauges designated W6 and W7 were arranged at the centre of the precast and laminated slabs to measure the relative displacement between the precast bottom slab and the cast-in-place concrete (Figure 9b).

**Figure 9.** Placement of displacement meters and strain gauges for BCSWSK testing: (a) Placement of displacement meters and strain gauges at the bottom of the slab; (b) Placement of displacement meters at the centre of the slab.

3. Test Results

3.1. Failure Phenomena and Modes of Failure

The failure process of the BCSWSK was as follows. The first crack appeared on the side of the specimen when it was loaded to 80 kN; it was roughly vertical. When loaded to 160 kN, the cracks on the side of the slab extended to the slab bottom and developed to the mid-span. When loaded to the measured ultimate load (213 kN), destructive deformation occurred. The maximum crack appeared at the mid-span, where the crack width exceeded 1.5 mm, and the number of cracks also began to increase, indicating that the slab had failed (Figure 10).

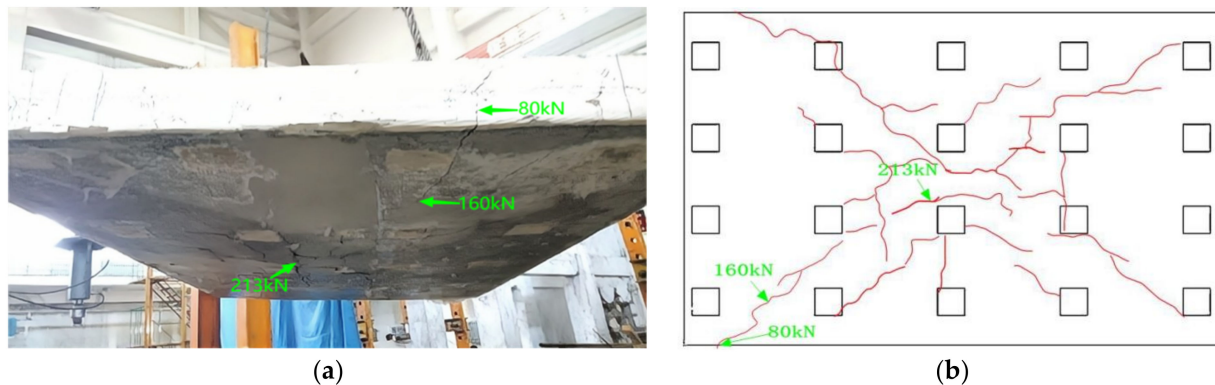


Figure 10. Crack patterns of BCSWSK after testing: (a) Photo; (b) Maps of failure patterns.

3.2. Analysis of the Curves from Testing

The load–strain curve of the tensile steel bar is shown in Figure 11, and the load–displacement curve of the concrete at the mid-span is shown in Figure 12.

It can be seen from the load–strain curve of the tensile-reinforcement steel bar of the BCSWSK (Figure 11) that, before the first crack occurred, the increase in load was approximately linearly proportional to the change in strain of the tensile steel bar, and the strain values were relatively small. As the load increased until cracks appeared in the specimen, the strain of the tensile reinforcement increased significantly, and there was a turning point in the curve. When the load increased to 160 kN, the tensile steel bar yielded, and the strain of the steel bar reached the yield value. It can be seen from Figure 12 that the load–displacement curve of the BCSWSK included an elastic section, an ascending inelastic section, and no descending section. The behaviour is similar to that of the cast-in-place slab, which indicates the integrity of the lamination [24].

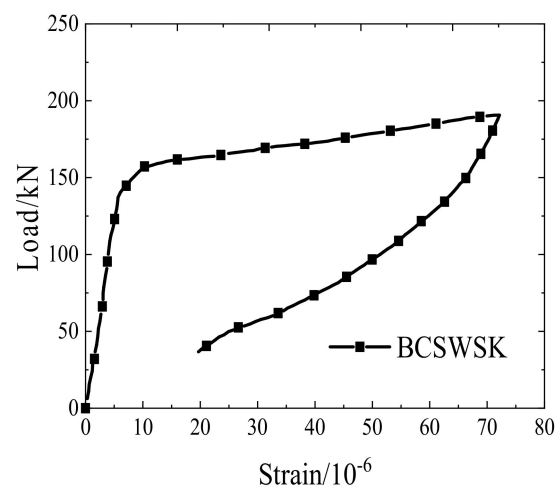


Figure 11. Load–strain curve of the tensile steel bar at mid-span.

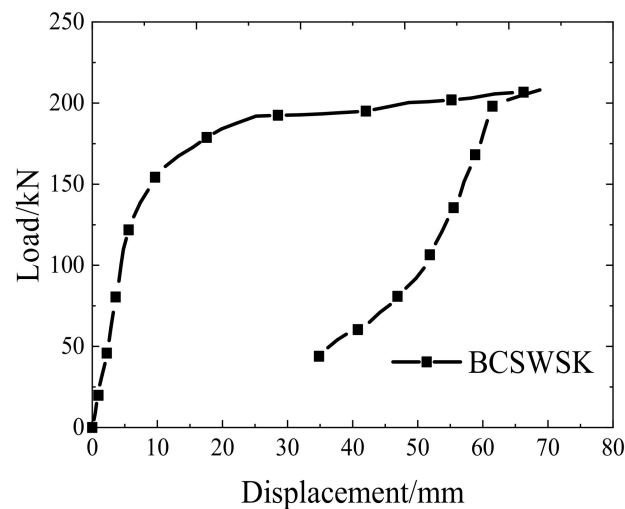


Figure 12. Load–displacement curve of the laminated concrete at mid-span.

3.3. Integrity of the Slab

In the fabrication of the BCSWSK, 28 d after the initial cast, there was a second concrete pour of the laminated slab, introducing concrete above the PBS and between the shear keys. Analysing the strength of the connection between the two was a key objective of the experiment. There was no disconnection cracking at the interfaces of the precast slab and the shear keys, which indicates that the precast slab and the shear keys were firmly connected and well-integrated (Figure 13). The horizontal displacement gauges (W6 and W7) showed that the relative displacement of the two slabs was almost zero, which also indicates the integrity of the PBS and the CCUL. It can also be seen that the crack pattern presents the failure characteristics of a two-way slab, which further shows its good integrity.



Figure 13. Failure of the precast bottom slab.

4. Finite Element Modelling and Analysis

Although the experimental results provide valuable information, they are limited to the material studied. Finite element analysis was used to generalise the results of the study. The main objectives of the numerical analysis were: (1) to determine the generalisability of the finite element model (FEM); (2) to determine the shear force distribution of the shear keys; and (3) to determine the effects of the main influencing parameters of the mechanical behaviour of the BCSWSK.

4.1. Generalisability of FEM

The load-bearing behaviour of the laminated concrete slab was modelled using ABAQUS. A concrete plastic-damage (CPD) model [24] was adopted for the constitutive relations of concrete. The equation of the compressive σ – ε curve used for it is:

$$\sigma_c = \begin{cases} \frac{E_c^2 f_{c,r} \varepsilon}{\left(\frac{E_c^2 \varepsilon_{c,r}}{(E_c^2 \varepsilon_{c,r} - f_{c,r})^2} - (\varepsilon_{c,r} E_c^3 \varepsilon - E_c f_{c,r} \varepsilon) \left[1 + \left(\frac{\varepsilon}{\varepsilon_{t,r}} \right)^{\frac{E_c \varepsilon_{c,r}}{E_c \varepsilon_{c,r} - f_{c,r}}} \right] \right)} \\ \frac{f_{c,r} \varepsilon}{\varepsilon_{c,r} E_c \varepsilon \left[(0.157 f_{c,r}^{0.785} - 0.905) \left(\frac{\varepsilon}{\varepsilon_{t,r}} - 1 \right)^2 + \frac{\varepsilon}{\varepsilon_{t,r}} \right]} \end{cases} \quad (1)$$

$$\varepsilon_{c,r} = \left(700 + 175 \sqrt{f_c} \right) \times 10^{-6} \quad (2)$$

where $f_{c,r}$ is the representative value of concrete, $\varepsilon_{c,r}$ is the corresponding strain, E_c is the damage evolution parameters, and f_c is the standard compressive strength of the concrete.

A bifold model [25] was adopted for the constitutive relations of reinforcement. The equation of the σ – ε curve used for it is:

$$\sigma = \begin{cases} E\varepsilon & \varepsilon \leq \varepsilon_y \\ f_y + 0.01E(\varepsilon - \varepsilon_y) & \varepsilon > \varepsilon_y \end{cases} \quad (3)$$

where f_y is the yield strength of steel bar and ε_y is the corresponding strain.

The PBS, shear keys, and CCUL were all C3D8R elements, and the steel bar was a T3D2 element incorporated into the concrete as an embedded region. The boundary conditions, loading modes, and meshing of the model are shown in Figure 14. The interface between the PBS and the CCUL was friction contact. The Newton iterative method was used to solve the FEM equations.

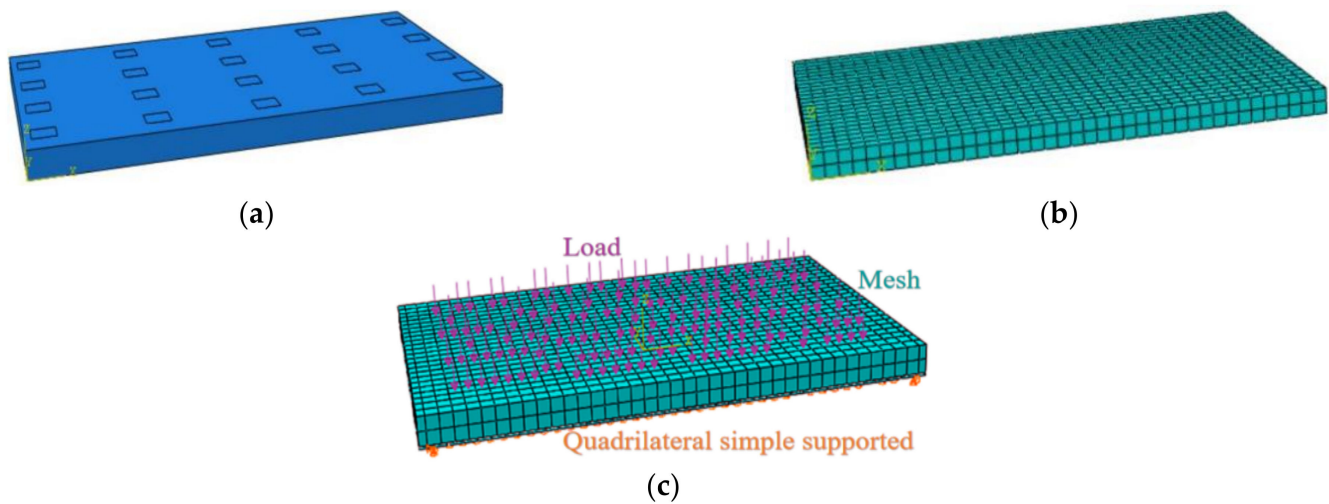


Figure 14. Boundary conditions, loading, and mesh generation of the model: (a) Unmeshed; (b) Load mesh preparation; (c) Load mesh.

The load–displacement curves and the load–strain curves of the reinforcement given by the FEM were compared with the test results (Figure 15). It can be seen that the simulated curve and the test curve were almost identical; the maximum difference was <1.5%, thus verifying the generalisability of the FEM.

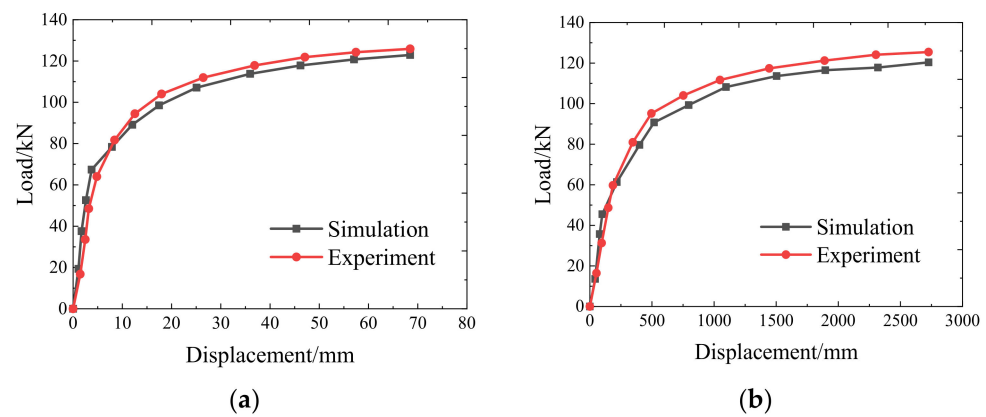


Figure 15. ABAQUS model and experimental results for comparison: (a) Load–displacement curves at mid-span; (b) Load–strain curves of steel bars.

4.2. Shear Force Distribution of the Shear Keys

Under vertical load, there was a relative misalignment between the PBS and the CCUL of the BCSWSK, which produced shear force in the shear keys. There were some differences between the keys in shear force along both the X and Y directions. A BCSWSK FEM, designated FJ1, was designed to enable us to analyse the shear distribution among the shear keys. The slab was grade C25 concrete, and the shear keys were grade C30. The remaining dimensions and physical parameters were identical to those of the test specimen. An identical loading process was simulated.

The shear force distribution within FJ1 under uniform vertical load was the same as for the original cast specimen because of the same symmetrical distribution of the shear keys along the X and Y directions. The shear keys in a quarter of the area of the slab were therefore selected for analysis, and they were numbered for convenience of description, as shown in Figure 16a.

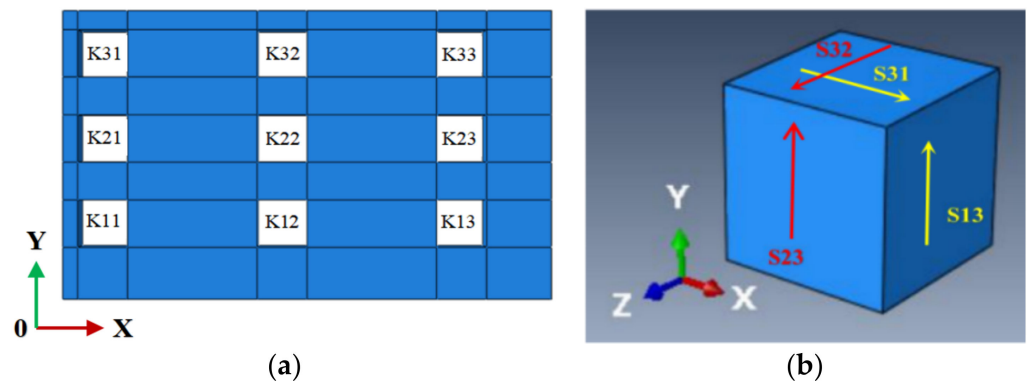


Figure 16. Number of shear keys and shear stress directions: (a) Number of shear keys in the quarter area of FJ1; (b) Shear stress directions on one shear key.

The shear keys were divided into eight elements. The shear stress directions of an element are shown in Figure 16b. X and Y are the length and width directions of FJ1, respectively. S13 is the shear stress along the Z direction in the YZ plane, and S23 is the shear stress along the Z direction in the XZ plane. According to the reciprocal theorem for shear stresses in the vertical plane, $S31 = S13$ and $S32 = S23$. The total shear force of a shear key at the contact surface of the PBS and the CCUL of FJ1 can be obtained: its value along the X direction is the product of S31 and its cross-sectional area, and its value along the Y direction is the product of S32 and its cross-sectional area. The relationship between the total load applied to FJ1, and the total shear force of each shear key in the X direction and the Y direction, can be plotted, as shown in Figure 17 (X) and Figure 18 (Y).

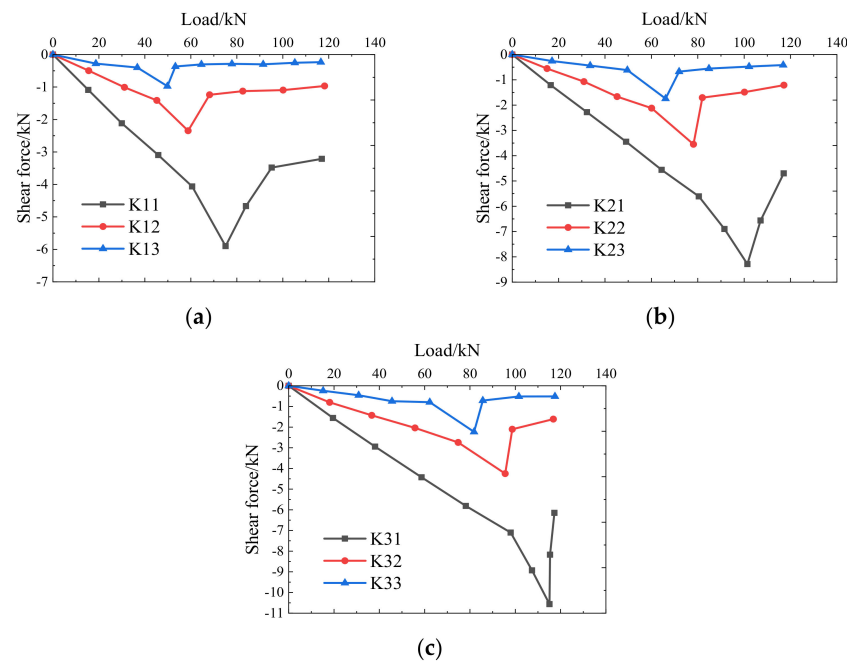


Figure 17. Load–shear force curves of the three groups of shear keys along the X direction: (a) K11–K13; (b) K21–K23; (c) K31–K33.

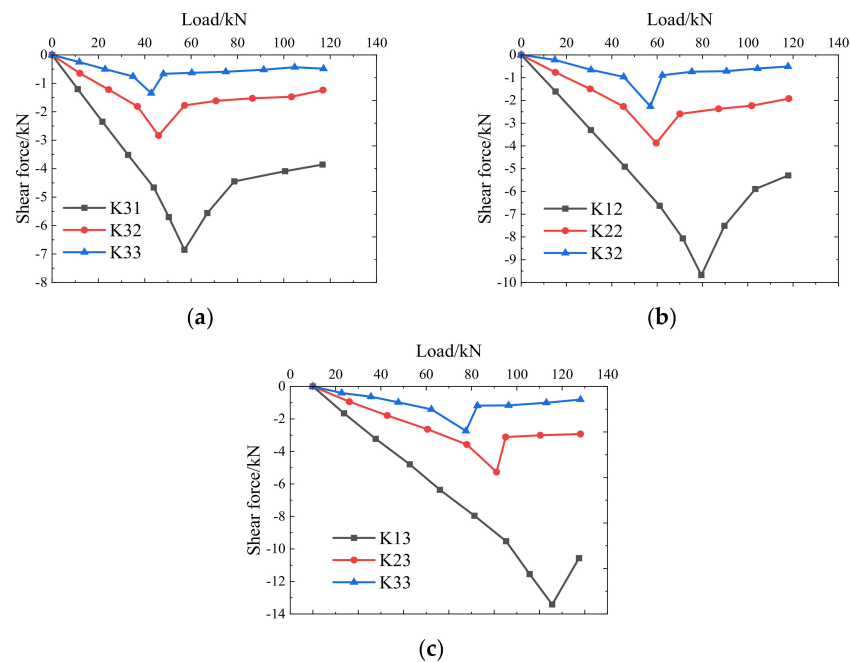


Figure 18. Load–shear force curves of the three groups of shear keys in the Y direction: (a) K11–K31; (b) K12–K32; (c) K13–K33.

4.2.1. Shear Distribution in the X Direction

Figure 17 shows the load–shear force curves of shear keys K11–K13, K21–K23, and K31–K33. It can be seen that for, all three groups of shear keys, the shear force increased linearly as the vertical load increased, and suddenly decreased when the shear force reached a maximum value. This behaviour indicates that the failure was brittle. The maximum shear force differed between the three groups: it was lowest near the mid-span and highest near the support. This is because the shear keys near the mid-span were primarily affected by bending moments. The vertical loads were different when the shear force reached its maximum value, so the shear keys did not all fail at the same time. As

the vertical load increased, failure gradually developed from the shear keys in the region with a larger bending moment near the mid-span, to the supported region with a smaller bending moment.

A comparison of the maximum shear force between the three groups of shear keys shows that, although the load–shear curves of the three groups were very similar, the shear values of K31–K33 were the greatest, and the shear values of K21–K23 and K11–K13 were the least. This was mainly because, under a vertical uniform load, FJ1 deformed along the diagonal in the area nearest the support. Deformation increased towards the centre of the slab, as did the misalignment between the precast bottom slab and the cast-in-place slab. The group of shear keys K31–K33 was closest to the centre of the slab, so resistance to the shear force generated by the misalignment was greatest for this group.

4.2.2. Shear Force Distribution in the Y Direction

The load–shear force curves for the three groups of shear keys K11–K31, K12–K32, and K13–K33, in the Y direction of FJ1, are shown in Figure 18. It can be seen that, similar to the X direction, the shear force increased linearly as the applied vertical load increased and suddenly decreased when it reached a maximum value. In each of the three groups of shear keys, the shear force was at a maximum near the support and at a minimum near the mid-span of the slab. In each group, the load values differed when the shear force reached the maximum value. A comparison of the maximum shear values shows that the shear resistance of K13–K33 was greatest because they were close to the centre of the slab.

4.2.3. Comparison between the Shear Forces in the X and Y Directions

Figure 19 shows the load–shear force curves of K11, K22, and K33 in the X and Y directions. It can be seen that the maximum shear force was greater in the Y direction than in the X direction, and that the shear force in the Y direction reached the maximum value first. This is because the bending moment of FJ1 was greater in the short-span direction than in the long-span direction, and so the bending moment had a greater effect on the shear force of the keys in the Y direction. Thus, it would be better to give the shear keys a rectangular cross-section with the longer side parallel to the short span when designing a BCSWSK to make them more effective in resisting shear.

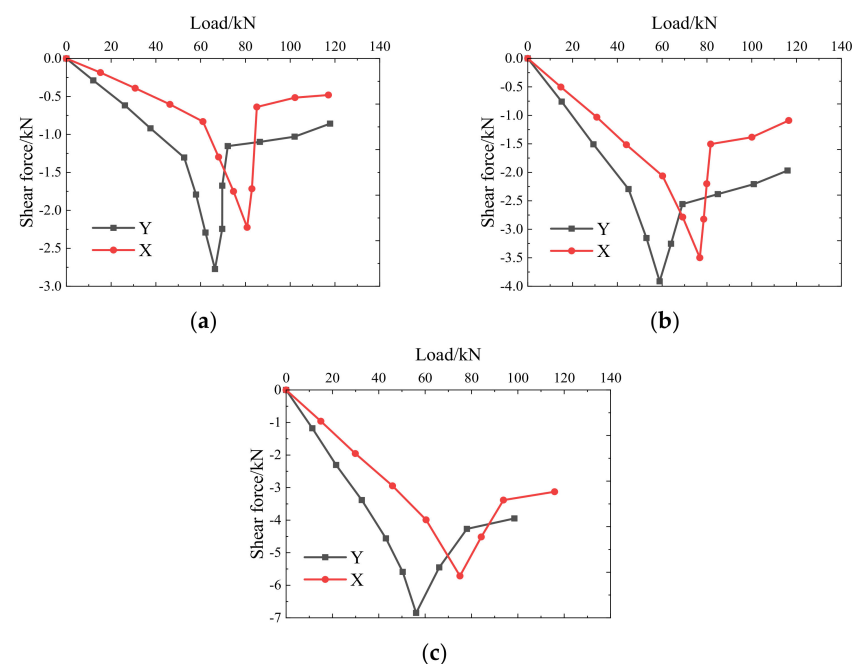


Figure 19. Load–shear force curves of different shear keys along the X and Y directions for comparison: (a) K11 along the X and Y directions; (b) K22 along the X and Y directions; (c) K33 along the X and Y directions.

4.2.4. Shear Key Cross-Section

The preceding analysis indicates that a rectangular shear key cross-section would be better than a square cross-section in terms of resisting shear. To clarify this issue, rectangular cross-section shear keys with different dimensions were modelled in the FEM. Figure 20 shows the different load–displacement curves obtained for the keys with respective dimensions in the X and Y directions: (L_{XY}) 50×200 mm, 100×100 mm, and 200×50 mm. Other dimensions and physical parameters were identical to those of FJ1. It can be seen that the bearing capacity was greatest for the 50×200 mm key until the displacement was about 53 mm, but after this point, the bearing capacity became greatest for the 200×50 mm key. However, the difference between the three curves was slight, mostly $<10\%$, so the square cross-section is recommended for the shear keys.

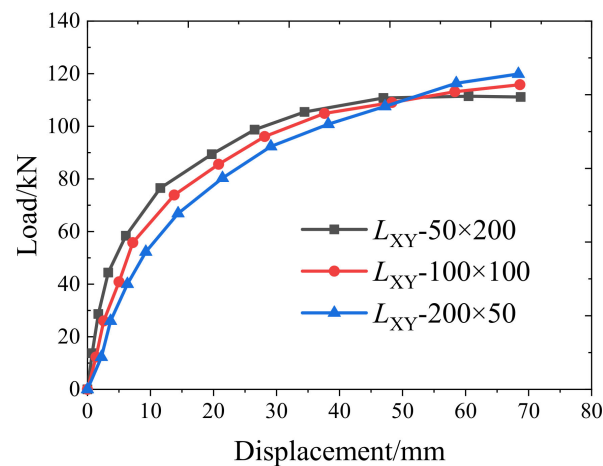


Figure 20. Load–displacement curves of BCSWSKs with different shear-key shapes for comparison.

4.3. Parameter Influence

Twenty-five laminated slabs were modelled to allow us to analyse the influence of different factors on the load–displacement curves of BCSWSKs, as shown in Table 2. All dimensions and physical parameters were identical to those of model FJ1, except for the parameters that were varied experimentally. The simulations show that changing the friction coefficient, the grade of the cast-in-place concrete, and the grade of the concrete used with shear keys had very little effect on the mechanical properties of BCSWSKs, mostly $<3.2\%$, and their load–displacement curves almost coincided, but changing the other parameters had clearly observable effects, as shown in Figures 21–23.

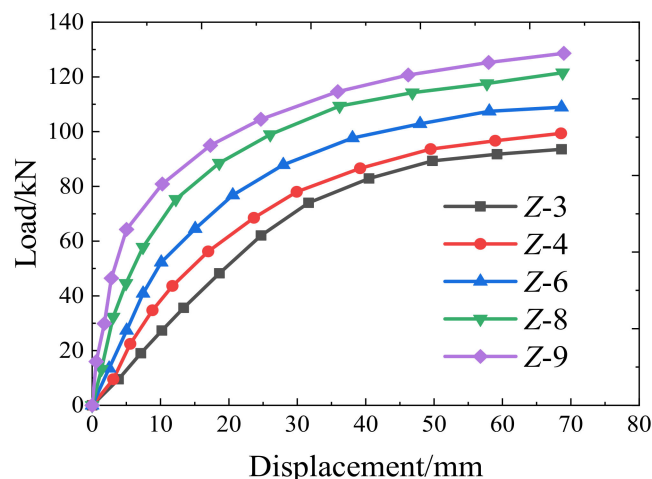


Figure 21. Load–displacement curves of BCSWSKs with different numbers of columns of shear keys for comparison.

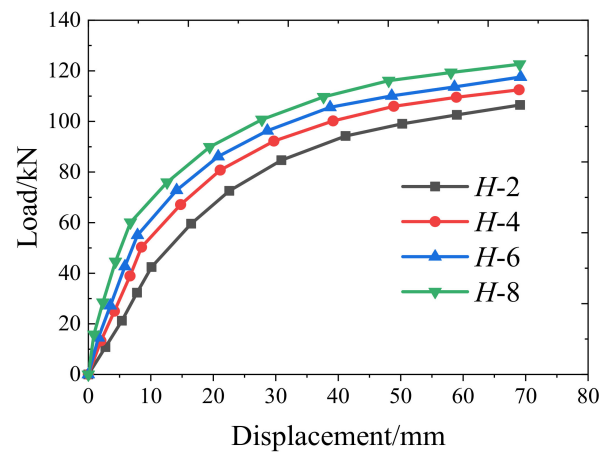


Figure 22. Load–displacement curves of BCSWSKs with different numbers of rows of shear keys for comparison.

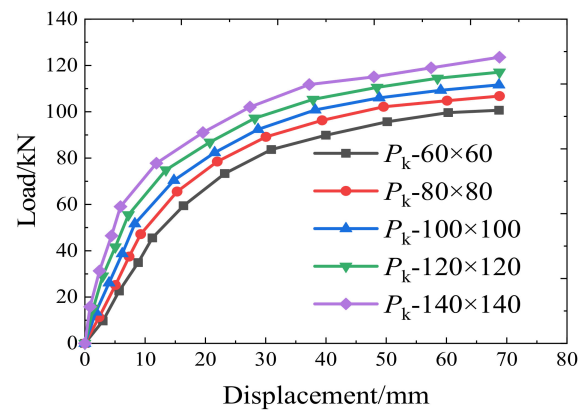


Figure 23. Load–displacement curves of BCSWSKs with different shear key dimensions for comparison.

Table 2. List of laminated slabs modelled.

Model Designation	Number of Shear Key Columns Z	Number of Shear Key Rows H	Shear Key Cross-Section Dimensions P_k (mm)	Concrete Grade		Friction Coefficient f	Yield Load (kN)	Yield Displacement (mm)
				Cast-in-Place Concrete S_i	Shear key S_k			
$f-0$	6	4	100 × 100	C25	C30	0	77.2	16.7
$f-0.4$	6	4	100 × 100	C25	C30	0.4	77.4	16.6
$f-0.6$	6	4	100 × 100	C25	C30	0.6	77.8	16.4
$f-0.8$	6	4	100 × 100	C25	C30	0.8	78.3	16.2
S_i-20	6	4	100 × 100	C20	C30	0.8	77.5	16.0
S_i-25	6	4	100 × 100	C25	C30	0.8	78.3	16.2
S_i-35	6	4	100 × 100	C35	C30	0.8	79.3	16.5
S_k-20	6	4	100 × 100	C25	C20	0.8	77.9	16.4
S_k-30	6	4	100 × 100	C25	C30	0.8	78.3	16.2
S_k-40	6	4	100 × 100	C25	C40	0.8	78.9	16.0
Z-3	3	4	100 × 100	C25	C30	0.8	70.4	23.5
Z-4	4	4	100 × 100	C25	C30	0.8	74.7	19.4
Z-6	6	4	100 × 100	C25	C30	0.8	78.3	16.2
Z-8	8	4	100 × 100	C25	C30	0.8	80.7	14.0
Z-9	9	4	100 × 100	C25	C30	0.8	81.7	12.5
H-2	6	2	100 × 100	C25	C30	0.8	72.4	22.4
H-4	6	4	100 × 100	C25	C30	0.8	75.7	18.2
H-6	6	6	100 × 100	C25	C30	0.8	78.3	16.2
H-8	6	8	100 × 100	C25	C30	0.8	79.5	15.0
$P_k-60 \times 60$	6	4	60 × 60	C25	C30	0.8	70.6	23.2
$P_k-80 \times 80$	6	4	80 × 80	C25	C30	0.8	75.4	18.8
$P_k-100 \times 100$	6	4	100 × 100	C25	C30	0.8	78.3	16.2
$P_k-120 \times 120$	6	4	120 × 120	C25	C30	0.8	79.6	15.3
$P_k-140 \times 140$	6	4	140 × 140	C25	C30	0.8	81.0	13.0

4.3.1. Effect of Number of Shear Key Columns along the X Direction (Z)

Figure 21 shows the load–displacement curves of BCSWSKs for different numbers of shear key columns ($Z = 3, 4, 6, 8, 9$). Figure 21 and Table 2 show that the bearing capacity increased as the number of shear key columns increased. When the number of columns increased from three to nine, the yield load increased by 16.1% and the yield displacement decreased by 47%. When the number of columns increased from eight to nine, the yield load increased by 1.2% and the yield displacement decreased by 10.8%. These results show that increasing the number of columns increases the yield load and decreases the yield displacement of the BCSWSK, but this trend is not so pronounced after a certain point. This reduction in effectiveness occurs because, at some point, the number of shear keys in the slab is such that the relative misalignment between the PBS and the CCUL is very small, so increasing the number of shear keys has little effect.

4.3.2. Effect of Number of Shear Key Rows along the Y Direction (H)

Figure 22 shows the load–displacement curves of BCSWSKs for different numbers of rows of shear keys ($H = 2, 4, 6, 8$). Figure 22 and Table 2 show that the bearing capacity increased as the number of shear key rows increased. When the number of rows increased from two to eight, the yield load increased by 9.8% and the yield displacement decreased by 33.2%. When the number of rows increased from six to eight, the yield load increased by 1.5% and the yield displacement decreased by 7.5%. These results show that increasing the number of rows of shear keys increases the yield load and decreases the yield displacement of the BCSWSK, but this trend is not so pronounced after a certain point. The reason for this is the same as that for the increase in the number of columns. However, a change in the number of rows has a relatively small effect on the bearing capacity of BCSWSKs compared to a change in the number of columns.

4.3.3. Effect of Shear Key Cross-Section (P_k)

Figure 23 shows the load–displacement curves of the BCSWSK with different shear key cross-sections ($P_k = 60 \times 60, 80 \times 80, 100 \times 100, 120 \times 120$, and 140×140 mm). It can be seen from Figure 23 and Table 2 that bearing capacity increased as the shear key cross-section increased. When the cross-section increased from 60×60 mm to 140×140 mm, the yield load increased by 19.5% and the yield displacement decreased by 44%. When the cross-section increased from 120×120 mm to 140×140 mm, the yield load increased by 1.8% and the yield displacement decreased by 1.5%. These results show that increasing the cross-section areas of the shear keys increases yield load and decreases the yield displacement of BCSWSKs, but this trend is not so pronounced after a certain point. The reason for this is the same as that for the effect of the number of columns.

5. Conclusions

In this research, experimental testing and FEMs were used to analyse shear force distribution and the factors that influence the mechanical properties of BCSWSKs under the action of uniform vertical load. In view of the test results and analysis, the following conclusions can be drawn.

- (1) The loading scheme of stacking and jacking to apply an equivalent uniform load is appropriate. The bonds at the interfaces between the PBS and the shear keys, and between the PBS and the CCUL, are strong despite the secondary concrete pouring for the upper slab.
- (2) The shear force distribution among its shear keys is similar in the X and Y directions. The shear force increases linearly as the load increases and decreases suddenly when the load reaches a certain point, namely when the shear keys show brittle failure. The shear force is greater in the Y direction than in the X direction and reaches a maximum value earlier. The difference in shear force distribution among shear keys between the two directions is small, mostly <10%. A square cross-section is recommended for the shear keys.

- (3) The number of rows, number of columns, and cross-sectional area of the shear keys are the main factors that influence the mechanical properties of BCSWSKs, of which the yield load and yield displacement varies by 9.8%~19.5% and 33.2%~47%, respectively. The effectiveness of all three factors decreases as the factors increase and, after a certain point, the effect is minimal, mostly <7.5%. After a certain number of shear keys have been placed, the cast-in-place concrete grade, the shear key concrete grade, and the friction coefficient between the precast bottom slab and the cast-in-place slab have little influence on the mechanical properties, mostly <3.2%.

6. Future Research

This study investigated the mechanical properties of BCSWSKs using load tests on a fabricated laminated slab and finite element models. The shear force distribution across the shear keys was measured and analysed, and the effects of different parameters were identified. These provide much reference for the design of BCSWSKs in the service stage. However, their mechanical properties in the construction stage was not studied, such as the bending properties of PBSs and PBSs with I-beams bolted to them. As such, more research should be conducted on them for their engineering practicality.

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Article

Experimental Study on Flexural Bearing Characteristics of Ceramsite Concrete Beams after Creep at Different Curing Ages

Hongbing Zhu ^{1,2,*}, Jingyi Chen ¹, Yuxin Wu ¹, Jiapeng Li ^{1,3}, Zhenghao Fu ¹ and Peng Liu ⁴

¹ School of Urban Construction, Wuhan University of Science and Technology, Wuhan 430065, China

² Institute of High-Performance Engineering Structure, Wuhan University of Science and Technology, Wuhan 430065, China

³ The National Prefabricated Construction Industry Base, Huasen Architecture and Engineering Design Consulting Company, Shenzhen 518000, China

⁴ School of Civil Engineering, Central South University, 22 Shaoshan Road, Changsha 410075, China

* Correspondence: zhuhongbing@wust.edu.cn

Abstract: Ceramsite concrete is increasingly used for structural applications due to its lightweight, high-strength, and high-temperature resistance advantages. However, research into the postcreep properties of ceramsite concrete has yet to keep pace with other areas. Therefore, this paper aims to investigate the flexural bearing properties of ceramsite concrete beams after creep and make recommendations for future research. To study the flexural load-bearing performance of ceramsite concrete beams after creep, a comparison test was carried out on four ceramsite concrete beams subjected to 200 d creep loading and four beams subjected to static load without creep loading for four curing ages. The results showed that, at first, the deformation capacity of the ceramsite concrete beams would increase after creep and the toughness and energy absorption capacity would also strengthen. Secondly, the creep could improve the ultimate bearing capacity of the ceramsite concrete beams. It was an approximate 10% increase after 200 d creep. Then, the stiffness, toughness, and ultimate bearing capacity of the ceramsite concrete beams would increase gradually with the increase of the curing age after the creep. In addition, the ultimate bearing capacity of the ceramsite concrete beams increased gradually with the increase of the curing age. The growth rate was faster in the early stage, slower after 120 d, and slower and more stable after 228 d. Finally, creep could cause the maximum crack width and depth of the ceramsite concrete beams decreased and the number of cracks would grow with the curing age. The conclusions obtained in the study provide a theoretical basis for the design of light aggregate structures for creep.

Keywords: ceramsite concrete beam; crack; creep; curing ages; ultimate bearing capacity

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1. Introduction

In recent years, as the spans of buildings and bridges have increased, the self-weight of structures is approaching the limits of what ordinary concrete can withstand. As a result, reducing the weight of such systems has attracted increasing research attention [1].

Ceramsite concrete can reduce the weight of ordinary concrete by 20% to 40% under the same load-bearing conditions [2,3]. Due to its lightweight, high strength, and high temperature resistance [4], it has been used in high-rise buildings and bridge projects and is of high research value. In practical engineering, most ceramsite concrete structures are subjected to long-term loading, causing creep in the structure. Jiang [5] found that ceramsite concrete has greater creep values than ordinary concrete, which can be up to 1.3 times greater. Under long-term loading conditions, this can lead to large creep deformations and relaxations. On the one hand, this may cause the structure to tilt and deform. On the other hand, excessive structural deformation may change the structure's mechanical properties and seriously affect its safety. Both aspects deserve attention.

For them, the former relates to the laws of concrete creep and the latter to the effects of concrete creep. Currently, many studies reflect concrete creep, and the creep values of ceramsite concrete are higher than those of ordinary concrete of the same age. The tensile creep of highly porous ceramsite concrete will be lower than that of low-absorbent ceramsite concrete, and plain ceramsite concrete [6,7]. In addition, the loading age of the structure [8–10], the curing age [11] and curing temperature [12], the concrete's collapse [13] and sand content [14], dosing of steel fibres [15,16], dosing of polymeric fibers [17], admixture [18], and the relative humidity of the working environment in which the concrete elements are placed [19,20] have all been shown to be influential factors in creep. As research continued, Zhu, Glanville [21,22] and others used experimental and theoretical hypotheses on creep to propose a creep model containing loading age and loading time parameters, respectively. Compared with the studies on the creep law of concrete, there are fewer studies involving the structure's mechanical properties after creep. Dong, Rong, and others [23–25] have suggested that long-term loading causes stress relaxation in concrete beams and reduces the stress at the tip of concrete cracks, resulting in an increase in the residual bearing capacity of the concrete beam and an improvement in crack development. However, Liu [26] obtained the opposite test results. He found that reclaimed concrete beams under long-term loading exhibited postductility and that secondary loading reduced their flexural bearing capacity. Furthermore, Saliba and Omar [27,28], among others, suggested that the development of microcracks in concrete during creep can reduce concrete residual strength. It is challenging to provide scientific references for practical engineering. Therefore, the effect of creep on the mechanical properties of ceramsite concrete structures needs to be urgently investigated. In addition, in existing studies, extending the loading age of plain concrete beams (i.e., increasing the curing age) reduces the effect of creep on the structure [8]. This gives good ideas for improving the mechanical properties of ceramsite concrete structures after creep. This is also a technical issue that deserves attention.

Analyzing the above, ceramsite concrete has good prospects for engineering applications because of its lightweight and high-strength properties. However, the effect of creep on its mechanical properties has yet to be unified, which does not provide a scientific reference for practical engineering. In addition, although extending the age of curing can reduce the effect of creep on plain concrete beams, it is still being determined whether it can improve the mechanical properties of ceramsite concrete beams after creep. Clarifying and improving the effect of creep on the mechanical properties of ceramsite concrete structures is a significant issue. Therefore, this study aims to clarify the effect of creep on the flexural properties of ceramsite concrete beams. On this basis, the influence of curing age on flexural bearing characteristics of ceramsite concrete beams after creep is found. In order to achieve these objectives, the following work was carried out in this paper. (i) Eight ceramsite concrete beams with four types of curing ages were produced. Then, the loading of 30% flexural ultimate bearing capacity was carried out on one group of them (four beams with different curing ages) for 200 days. Next, the two groups of the beam were tested under various loads by step loading, and the cracking load, failure load, and crack development of the beam were recorded. (ii) Based on the deformation law, ultimate bearing capacity, beam damage, and crack distribution, the influence of long-term creep on the flexural performance of ceramsite concrete beams was analyzed by comparing them with the experimental beams of the same age without creep loading. (iii) We drew the influence curve of ultimate bearing capacity and crack distribution curve of ceramsite concrete beams after creep with different curing ages during a static load test and fitted the data to analyze the influence of curing ages on mechanical properties of ceramsite concrete beams after creep.

2. Experimental Materials and Basic Mechanical Properties

2.1. Experimental Materials

So that related workers can repeat the experiment in the future, all materials used are listed as follows.

- (1) The coarse aggregate is 900-grade crushed stone shale granules produced by Yichang Guangda Ceramic Granules Products Co. Prewetting ceramsite before concrete mixing can effectively improve the workability of concrete. Its physical properties are shown as Table 1. The ceramsite is prewetted for 4 h, and then dried for 12 h to remove the water on the ceramsite surface (shown in Figure 1).

Table 1. Physical properties of shale ceramsite.

Particle Size (mm)	Volume Density (kg/m ³)	Apparent Density (kg/m ³)	Compressive Strength of Concrete Cylinder (MPa)	Water Absorption in 1 h (%)
5~20	814	1517	6.8	2.42

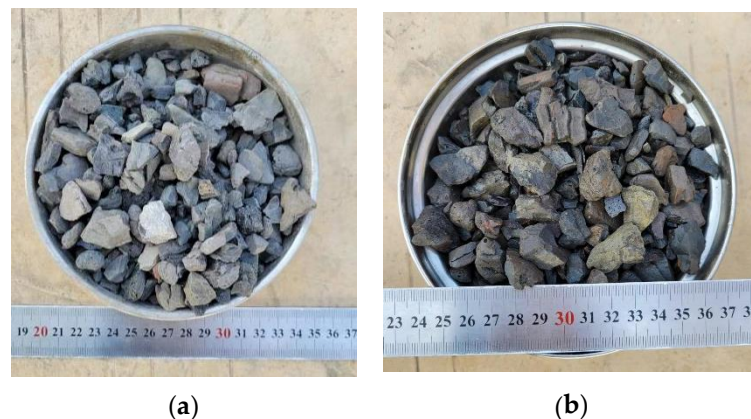


Figure 1. Shale ceramsite. (a) Before wetting process. (b) After wetting process.

- (2) Fine aggregate was ordinary river sand, which was sieved before the test to avoid the influence of impurities in the sand. The filter sieve used for the test was a 4.75 mm side length square hole sieve. The physical properties are shown in Table 2.

Table 2. Physical properties of sand.

Volume Density (kg/m ³)	Apparent Density (kg/m ³)	Mud Ratio (%)	Fineness
1570	2650	≤2	2.7

- (3) The cement was P.O. 42.5 ordinary silicate cement produced by Hubei Huaxin Cement Company Limited, Wuhan, China and its physical indices all met the requirements of the test specification, as shown in Table 3.

Table 3. Basic physical index of cement.

Density (g/cm ³)	Mineral Composition of Clinker				Fineness(Sieve Allowance by 80 μm Square Hole)/%
	C3S	C2S	C3A	C3AF	
3.15	45	25	12	8	6.5

- (4) The water-reducing agent is HsC polycarboxylat-type superplasticizers produced by Qingdao Hongxia. The technical specifications of the water-reducing agent are shown in Table 4.

Table 4. Technical index of the water-reducing agent.

Color	PH	Relative Density	Solid Content (%)	Water-Reducing Rate (%)
Pale yellow	6~8	1.08 ± 0.02	40	25–35

(5) The water was Wuhan tap water.

2.2. Mix Proportion

The strength of the concrete prepared for this test was based on the actual bridge project and the proposed strength class was LC40 ceramsite concrete. Referring to the technical standard for applying lightweight aggregate concrete (JGJ/T 12-019) in China [29], LC40 was used as the target of the trial formulation. Based on the research of the former research group [30], the mix proportion of ceramsite concrete with the strength grade of LC40 was determined after several trial mixes, as shown in Table 5. On the basis of this mix proportion, the density of ceramsite concrete was $1942 \text{ kg}\cdot\text{m}^{-3}$.

Table 5. Mixed proportion of ceramsite concrete beam specimens (unit: kg/m^3).

Cement	Ceramsite	Sand	Water-Reducing Agent	Water	Sand Ratio
510	554	730	5.1	152	43%

2.3. Basic Mechanical Properties of Concrete

According to GB/T50081-2019 Standard for Physical and Mechanical Properties of Concrete [31], the compressive strength test used $100 \text{ mm} \times 100 \text{ mm} \times 100 \text{ mm}$ cube test blocks. The axial compressive strength test used $100 \text{ mm} \times 100 \text{ mm} \times 300 \text{ mm}$ prismatic test blocks. Flexural strength test use $100 \text{ mm} \times 100 \text{ mm} \times 400 \text{ mm}$ prismatic test blocks. The test blocks are made from the mix proportions shown in Table 5. Tests were carried out after 28 d of curing to obtain their cubic compressive strength, axial compressive strength and flexural strength values. The specimens cured for 28 d were also tested for static compressive modulus of elasticity and the static compressive modulus of elasticity of ceramsite concrete was measured. The results are shown in Table 6.

Table 6. Basic mechanical indices of ceramsite concrete after 28-d maintenance.

Mechanical Parameters	Strength			Test Results
	Specimen No. 1	Specimen No. 2	Specimen No. 3	
Compressive strength/MPa	40.04	38.77	42.47	40.43
Flexural strength/MPa	6.92	7.05	6.43	6.80
Elasticity modulus/GPa	24.45	24.01	24.18	24.21

3. Design of the Test

This research aimed to study the change in the flexural capacity of ceramsite concrete under creep. Based on this goal, 12 specimens were divided into three groups.

The ceramsite concrete beams used in the test were designed depending on the principle of “strong shear and weak bending”. Their cross-section size were $100 \text{ mm} \times 150 \text{ mm}$ and span length L were 1500 mm. In addition, their net span L_0 were 1400 mm. The longitudinal reinforcement of the beam was 24. The erection reinforcement was 48, type HRB400. the hoop reinforcement was $\phi 6$, type HPB300. Furthermore, the thickness of the concrete protective layer was 20 mm, as shown in Figure 2.

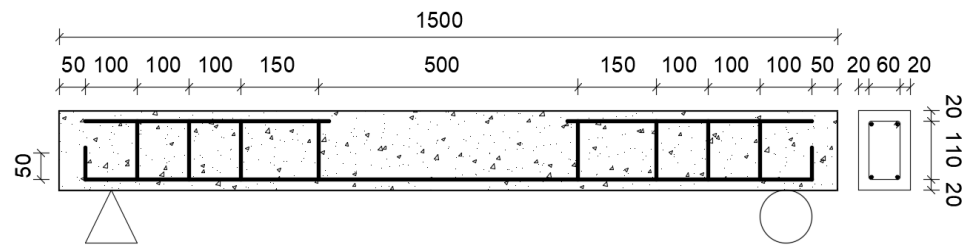


Figure 2. Structure of ceramsite concrete beam (unit: mm).

A total of 12 test beams were poured. To confirm the creep loading value of the test beam at each curing age, four beams were tested for the ultimate bending-bearing capacity under static load at 28 d, 60 d, 90 d, and 120 d of natural curing, respectively. As a reference group for the creep test beams, four beams were subjected to static load damage tests after 228 d, 260 d, 290 d, and 320 d of natural curing. The last four beams were subjected to 200 d of creep loading after 28 d, 60 d, 90 d, and 120 d of natural curing. Moreover, test setup for sustained load for reference Li [16], the four-point loading method was used for surcharge loads (shown as Figures 3 and 4).



Figure 3. Snapshots of loading. (a) Front view. (b) Side view.

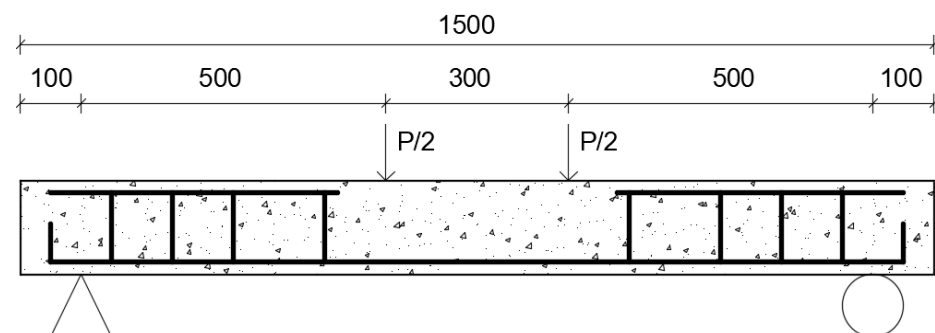


Figure 4. Diagram of loading (unit: mm).

This experiment takes into account laboratory specifics and safety issues, placing two test beams side by side. As the loading forces required for each beam are different, the following approach was taken for the experiment in order to combine the data from Table 7 with Figure 3. (i) First, we weigh and prestack each weight. We place a jack at the loading point of the distribution beam and then place the weight evenly on the distribution beam so that the force acting on the jacks are the same as the loading force shown in Table 7. (ii) Secondly, we mark in detail the distribution beam and the location of the jacks reaction points. (iii) Thirdly, the jacks are removed and the support, distribution beams and weight

are placed in the marked positions. Referring to existing studies [14,32,33], the loading stress is taken to be 30% of the ultimate load of the specimen beams. the spanwise deflection of the specimen beams are measured with dial indicators during the test and the spanwise deflection values of the specimen beams are recorded periodically.

Table 7. Loading parameters of test beam.

Group No.	Curing Age/d	Loading Duration/d	Purpose	Ultimate Bearing Capacity/kN	Loading Force/kN	Ultimate Bearing Capacity after Creep/kN
L0D1	28	0	Used to confirm creep loading force.	48.0	—	—
L0D2	60			50.0	—	—
L0D3	90			52.5	—	—
L0D4	120			53.5	—	—
L1D1	228	0	The reference group of creep test beams.	55.9	—	—
L1D2	260			56.4	—	—
L1D3	290			56.8	—	—
L1D4	320			57.2	—	—
L2D1	228	200	Control group to study the influence of curing age.	—	14.40	61.9
L2D2	260			—	15.00	61.8
L2D3	290			—	15.75	62.8
L2D4	320			—	16.05	63.1

After the creep test, the flexural bearing capacity of ceramsite concrete beams was tested. The test was carried out by using four-point loading and the loading schematic is shown in Figure 5. Strain gauges were applied at the beam's midspan before the loading started. The strain values were measured during loading to verify the flat section assumption. First, we check whether there was a defect in the sensitive grid was checked and the multimeter was used to check whether there was a short-circuit phenomenon. Then, the surface of the concrete where the strain gauges were to be applied was polished with abrasive paper along the 45° direction of the strain gauges, cleaned with alcohol and scribed and positioned. Finally, strain gauges were applied. In addition, a layer of AB glue was evenly applied to the strain gauges to protect them. Before the loading test, they were allowed to dry naturally for one day.

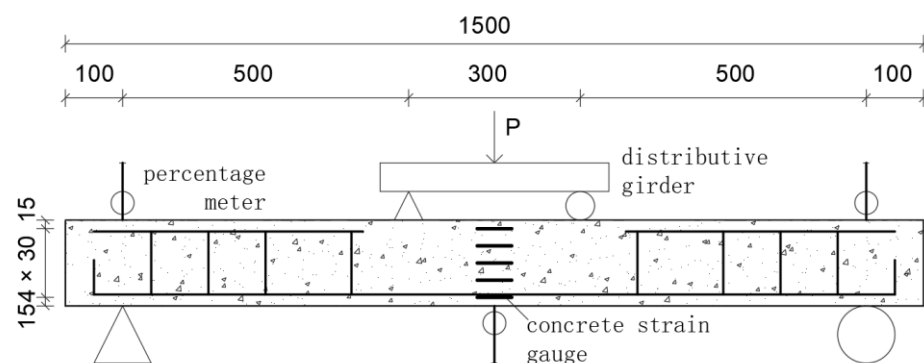


Figure 5. Diagram of static load test (unit: mm).

After the test preparation, a percentage meter was installed at the support and midspan of the ceramsite concrete beam to measure the midspan deflection. According to GB/T 50152-2012 Standard for Test Methods of Concrete Structures [34], preloading was carried out before formal loading. Moreover, the location of the specimens and instruments were checked to see if they were normal. The preload was carried out three times with continuous specimen position adjustment and instrumentation during the test. After passing the inspection, the official loading was performed.

The static load loading equipment of the test beam was a 200-t electrohydraulic servo pressure tester. The concrete strain value was collected by a static resistance strain gauge and the deflection was measured by a percentage meter. In addition, step-by-step loading was adopted. Each step's loading amount was 5 kN and the loading speed was 0.5 kN/s. When loading to 20 kN, the loading amount was adjusted to 2 kN per stage and the loading speed was set in 0.2 kN/s. When loading to 40 kN, the loading amount was adjusted to 1 kN per stage. The loading speed was controlled at 0.1 kN/s. Although the load value reached the ultimate bearing capacity of the ceramsite concrete beam, the pressure tester unloaded automatically. During the test, the loading time of each load level was controlled at 1–3 min. The midspan deflection and strain values imported into the computer were recorded and the crack development was observed. The field loading diagram is shown in Figure 6.

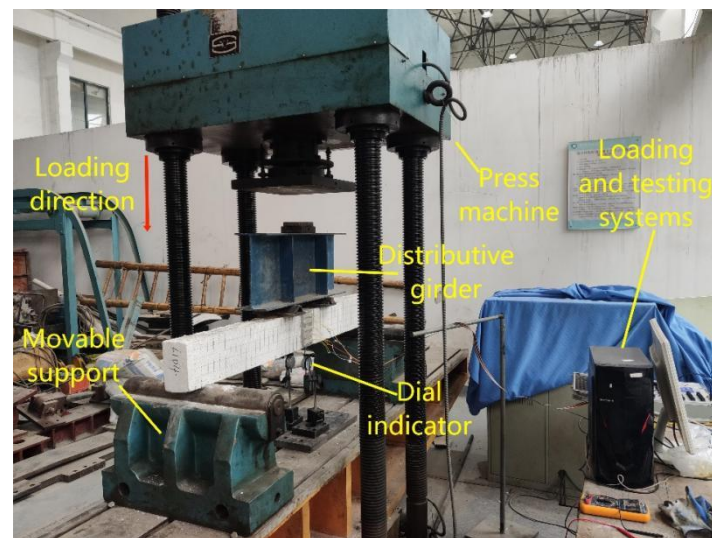


Figure 6. Snapshot of static load test.

Specimen beams L1D1, L1D2, L1D3 and L1D4 were subjected to direct static load damage tests when they were curing to their respective initial loading ages. For specimen beams L2D1, L2D2, L2D3, and L2D4, were first subjected to a 200-d period of creep loading at a value of 0.3 P_u when they were curing to their respective initial loading ages. Then, they were subjected to static load damage tests after the creep loading period.

4. Experimental Results and Analysis

4.1. Effect of Ceramsite Concrete Beam Performance after Creep

Figure 7 shows a typical damage pattern of ceramsite concrete beam. The cracks response of the beams were captured by the naked eye during the tests. It can be seen that all beams exhibit a similar cracking pattern whether they experience creep or not. That is, the first cracks in all volumes were produced in the bending region at the beginning of the loading. As the load is further increased, the cracks extend to the compressive side and additional cracks are produced in the shear zone. Compared with L1-type beams, L2-type beams displayed more cracks and less crack spacing. Moreover, the maximum crack width generated was also smaller during failure. In terms of the failure process, with the similar to ordinary concrete beams, both types of beams also experienced three stages. They were elastic working stage, elastic-plastic working stage and specimen failure. Figure 8 shows the load midspan deflection curves of beams L1D1 and L2D1 at static load loading. As can be seen from Figure 8, the direction and the changing trend of the load-mid span deflection curves of the ceramsite concrete beams were approximately the same, regardless of whether they had experienced creep loading. The load-deflection curves of the ceramsite concrete beams were essentially linear before the damage, with obvious yield points and roughly similar slopes of the curves. It exhibited that the preapplication of long-term loads did not

result in an improved ability of the ceramsite concrete beams to limit their deformation. However, compared with L1D1 and L2D1 showed a higher strength of 11% but 40% lower stiffness. The midspan deflections in the yielding and failure states were raised by 85% and 82%, respectively. It was shown that the preapplication of long-term loads led to enhance flexural strength and reduced static stiffness of ceramsite beams. Furthermore, ceramsite concrete beams that had experienced creep had more significant midspan deflection than beams that had not experienced creep when they reached yielding or failure. as referred to in reference [35].

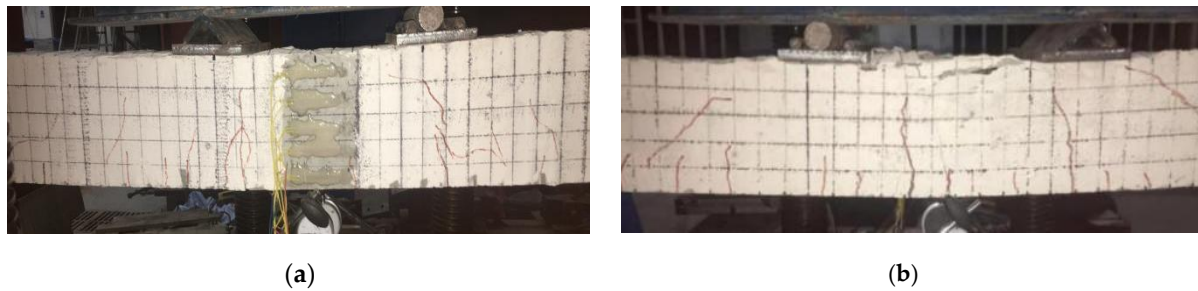


Figure 7. Ceramsite concrete beam damage pattern. (a) L1 beam damage pattern. (b) L2 beam damage pattern.

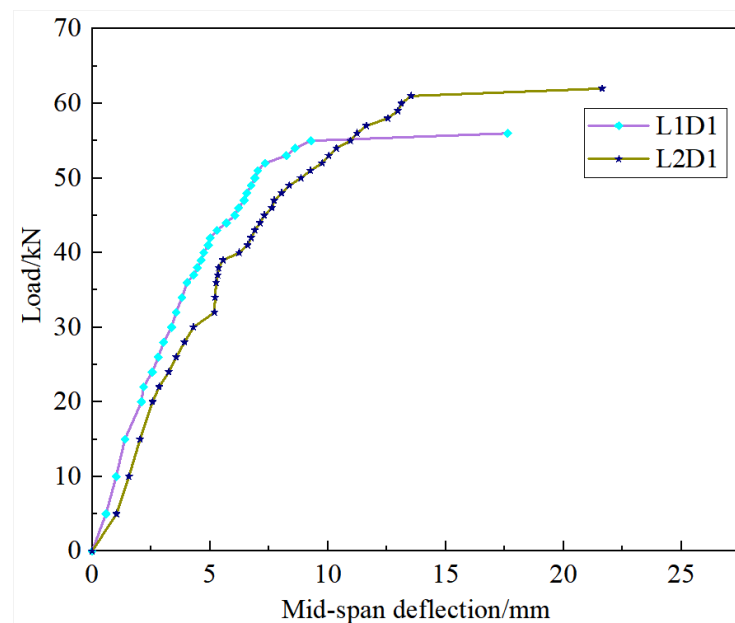


Figure 8. Load-deflection curve of L1D1, L2D1.

Figure 9 suggests the histogram of the ultimate bearing capacity of each beam. As shown in Figure 9, the ultimate bearing capacity of L2 beams enhanced by about 10% compared with L1 beams at the same curing age. It illustrated that long-term creep had an enhancing effect on the ultimate bearing capacity of ceramsite concrete beams. There were two plausible explanations for this experimental phenomenon [28,36]. On the one hand, it was caused by strengthening the concrete pressure zone during the long-term loading process. On the other hand, the stress relaxation reduced the stress at the tip of the concrete crack, resulting in the ultimate bearing capacity of the concrete beam being increased.

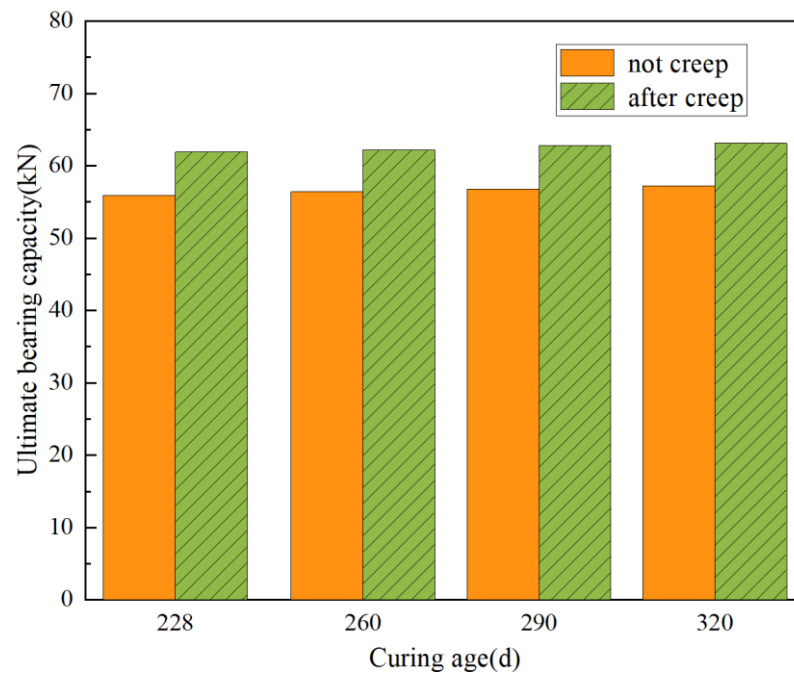


Figure 9. Ultimate bearing capacity of ceramsite concrete beams.

Figure 10 displays the distribution of cracks in ceramsite concrete beams. In the process of static loading, small vertical cracks appeared in the beam when it reached the cracking load. With continuously growing load values, the crack width, depth, and the number of cracks in the beams gradually increased. When beam reached the ultimate load, maximum cracking completely destroyed it.

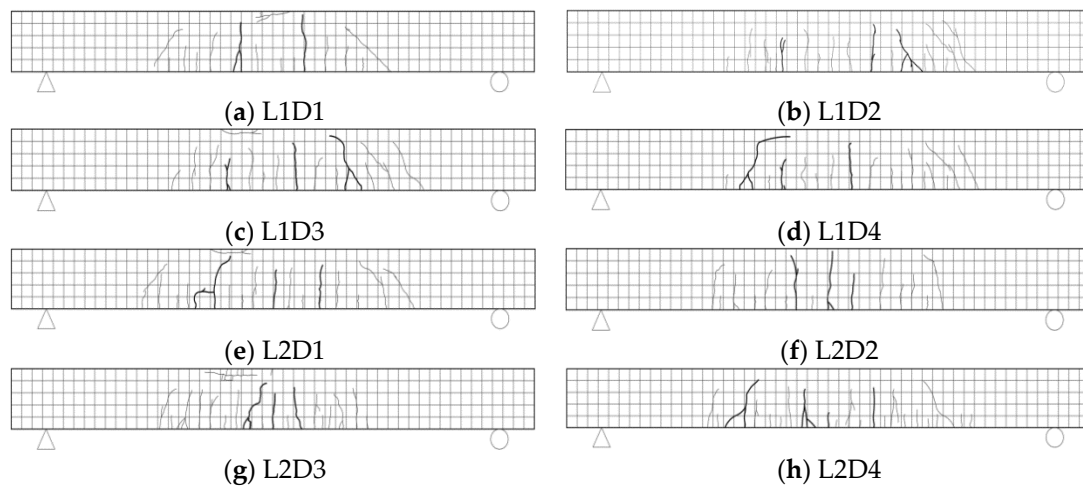


Figure 10. Distribution of cracks in ceramsite concrete beams.

Comparing the crack development of each test beam in Figure 10, it can be seen that the long-term creep effect caused the maximum crack width, crack depth, and average crack spacing to decrease when the ceramsite concrete beam (group L2) was damaged. Moreover, the number of cracks would increase. Because long-term loading causes stress relaxation in the concrete in the tension zone, stresses at the crack tip were reduced. In addition, cracks in the tension zone were difficult to develop when larger loads are applied. These could lead to that the maximum crack width and crack depth of ceramsite concrete beams were not significant when subjected to bending damage. When the ceramsite concrete beam reached the cracking load, although the crack width and the crack depth could not

be further developed, the number of cracks would continue to increase. In addition, the average crack spacing remains the same [37].

Figure 11 exhibits the load-span deflection curves for the L1 and L2 series test beams when applying static load damage. As shown in Figure 11, the midspan deflection of the test beams in both the yielded and damaged states at the same age after casting with static load damage had a significant increase after creep.

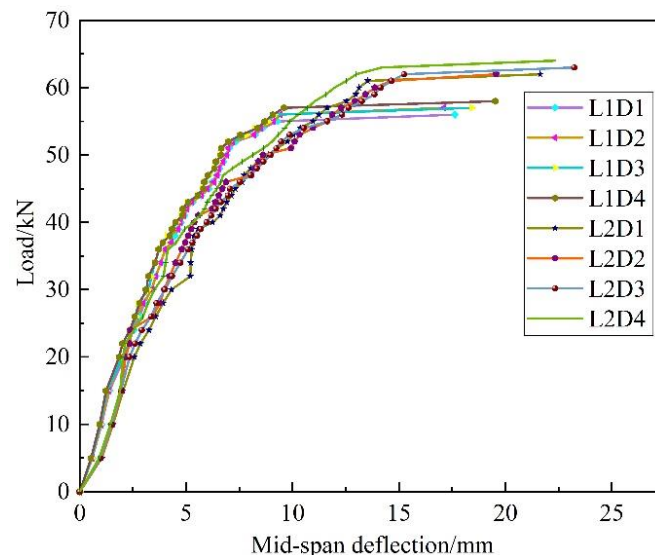


Figure 11. Load midspan deflection curve of ceramsite concrete beam in static load test.

It can also be seen from Figure 11 that the flexural stiffness of a beam undergoing creep enhanced as the age of curing increased. The deflection in the span corresponding to the transformation from elastic to plastic deformation became progressively larger and the deflection at ultimate load failure increased further. This indicated that the stiffness and ductility of the test beams gradually raised with increasing curing age. The deformation of the test beams at the same age of applied static load damage indicated that the change in spanwise deflection at static load yielding elevated slightly with increasing natural curing time for the same initial loading age. Moreover, the spanwise deflection at damage of the specimens raised significantly with increasing initial loading age [38]. All of the above studies demonstrated that the curing age affected the flexural performance of ceramsite concrete beams after creep. However, there was less research involving the effect of the curing age on the creep of ceramsite concrete beams, so it needed to be studied urgently.

4.2. Effect of Age of Maintenance on Ceramsite Concrete Beams after Creeping

As shown in Figure 12, the ceramsite concrete beams had a sizeable initial deflection deformation at loading. With the increase of the loading age, although the initial deflection value and the creep deflection value in the same holding time reduced gradually, the decreasing trend was less and less obvious. This was in accordance with Knaack [39] who proposed that the creep deflection decreases with the increase of the maintenance age.

As shown in Figure 13, as the age of loading increases, the reduction rate tended to zero. At this point, the total deflection reduction of the ceramsite concrete beam no longer raised significantly with the age of loading. This indicated a limit to the effect of the age of curing on the creep performance of the ceramsite concrete beams. As the load-holding time increased, the midspan deflection of the ceramsite concrete beams gradually increased. In the early stages of loading, the creep deflection rose at a faster rate and then slowly plateaued. The rate of change of the creep deflection with loading time varied for different loading ages of the ceramsite concrete beams. For the 28-d loading age, the deflection values enhanced fastest and the final total deflection values were the highest. As the loading age increased, the growth rate of the creep deflection of the ceramsite concrete beam at the

same load-holding time slowed down and the total deflection tended to reduce. At 120-d loading age, the growth rate of the creep deflection and the final deflection value of the ceramsite concrete beam at the same load-holding time was also the smallest. From the above analysis, the ceramsite concrete's strength, shrinkage creep and long-term structural deflection improved with increasing loading age.

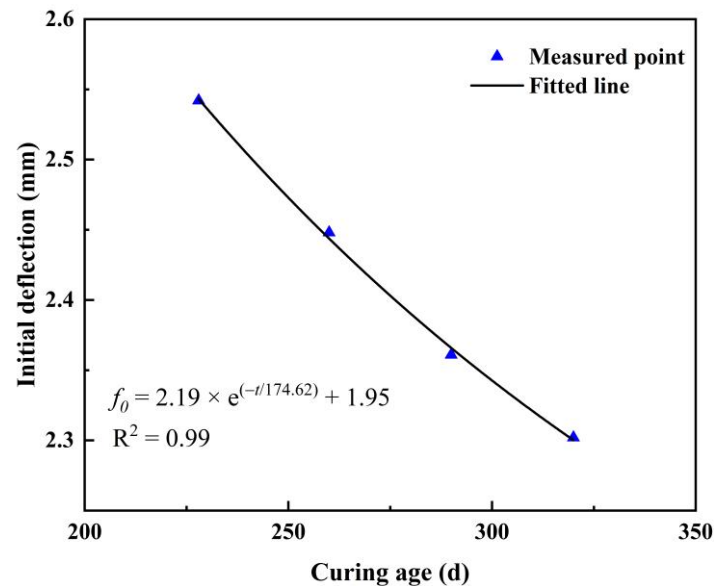


Figure 12. Effect of age of maintenance on the initial deflection.

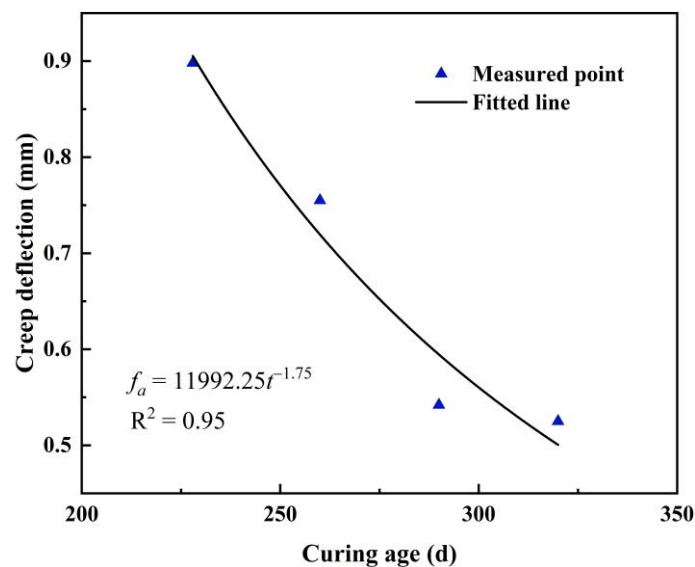


Figure 13. Effect of age of maintenance on creep deflection.

As displayed in Figure 14, The girders of group L2 were all loaded with 200 d of creep. The ultimate bearing capacity of the beams with a 320-d maintenance age increased by 1.81% compared with the beams with a 228 d maintenance age. Therefore, the age of curing also affected the ultimate bearing capacity of the ceramsite concrete beams subjected to creep. However, the extent of this effect was smaller at higher curing ages and the trend of increase in ultimate bearing capacity was not apparent.

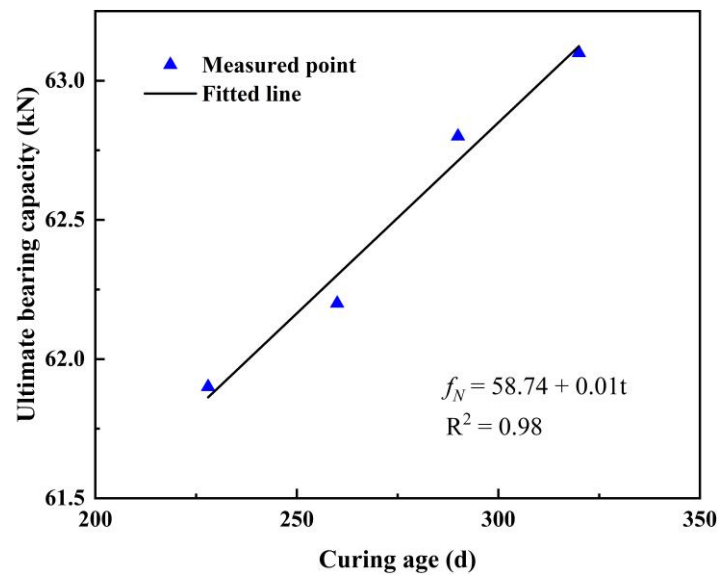


Figure 14. Ultimate bearing capacity curves at different maintenance ages.

Figures 15 and 16 and Table 8 suggest that as the age of curing increased, the number of cracks in the postcreep ceramsite concrete beams subjected to flexural compression damage gradually raised. In addition, it was with smaller maximum crack widths, crack depths, and average crack spacing. The ceramsite concrete beams with long curing periods had many dense cracks after bending and compression damage. Comparing with the test results of L2D1 and L2D4 beams showed the crack development pattern of the ceramsite concrete beams after creeping. This was because as the loading age of the ceramsite concrete beams increased, the internal and external moisture was continuously evaporated. In addition, the brittleness and strength class of the ceramsite concrete gradually enhanced. As a result, the number of cracks in the ceramsite concrete beams increased. The maximum crack width, depth, and average crack spacing decreased when the ultimate bearing capacity damage was reached. This indicated that as the age of maintenance increased, cracks in the beam were more likely to develop when subjected to external loads. The existing test results corroborated the crack-development pattern [40].

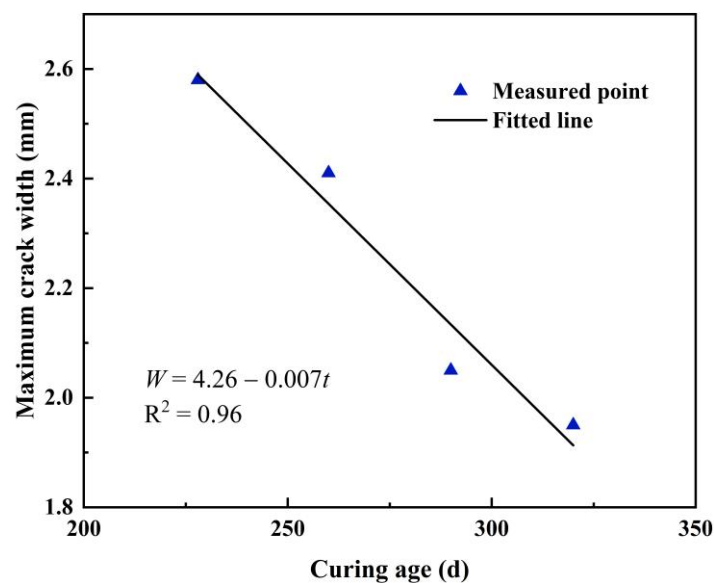


Figure 15. Maximum crack width curve after creep.

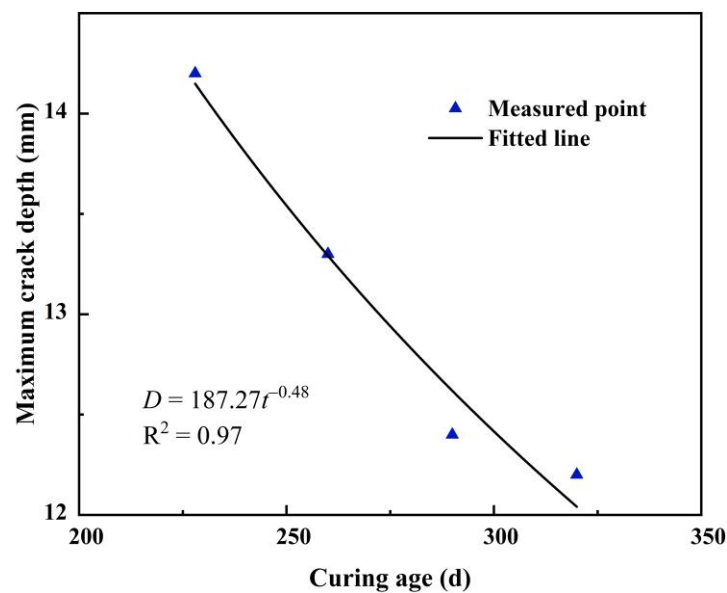


Figure 16. Maximum fracture depth curves after creep.

Table 8. Measured crack data.

Specimen Beam Grouping Number	Extreme Load Condition			Maximum Crack Depth at Damage (cm)	Average Crack Spacing at Damage (cm)
	Load (kN)	Maximum Crack Width (mm)	Number of Cracks		
L1D1	55.9	2.99	14	14.1	4.4
L1D2	56.4	2.73	16	13.5	4.3
L1D3	56.8	2.55	14	13.6	3.9
L1D4	57.2	2.41	17	13.2	3.6
L2D1	61.9	2.58	16	14.2	4.0
L2D2	61.8	2.41	17	13.3	3.6
L2D3	62.8	2.05	20	12.4	3.3
L2D4	63.1	1.95	22	12.2	3.2

5. Conclusions

Based on the results and discussion presented above, the following conclusions can be obtained.

- (1) Creep will increase the deformation capacity of ceramsite concrete beams and enhance the toughness and energy-absorption capacity. L2D1 exhibits higher flexural strength than L1D1, with a reinforcement value of 11%.
- (2) The law of load-bearing capacity of the ceramsite concrete beams was unaffected by creep loading. However, the ultimate load-bearing capacity of the ceramsite concrete beams was promoted by creep loading. The ultimate load-bearing capacity of the test beams increased by about 10% after 200 d of creep loading at the same curing age.
- (3) Creep loading affected the development of cracks. The long-term creep effect reduced the maximum crack width and crack depth. Moreover, it increased the number and average crack spacing of cracks in ceramsite concrete beams.
- (4) With the increase of loading age, the initial deflection value and the creep deflection value in the same holding time also gradually decreased. In addition, the decreasing trend was less and less noticeable. As the age of loading increases, the reduction rate tended to zero.
- (5) The age of curing also affected the ultimate bearing capacity of ceramsite concrete beams influenced by creep. The growth rate of the ultimate bearing capacity is faster in the early stages of curing ages, slower after 120 d, and slower and more stable in

the 228–320 d range. The ultimate load capacity of L2D4 has increased by only 1.81% compared to L2D1.

- (6) The age of loading affected the ability of the ceramsite concrete beams to develop cracks after creep. The shorter the age of loading, the fewer the cracks in the ceramsite concrete beams subjected to bending damage, the greater the maximum crack width, the depth of the cracks and the average crack spacing. The longer the loading age, the more dense the cracks in the concrete beams subjected to flexural damage.

In the future, it will be planned to pour more ceramsite concrete beams. A combination of experiments and numerical simulations will be used to study the effect of curing age on the load-bearing characteristics of creep-loaded ceramsite concrete beams. The results of the study will provide a scientific reference for engineering design. In addition, sulfate erosion affects the creep of ceramsite concrete beams, which will be viewed in the subsequent research directions.

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Article

Effect of the Location of Broken Wires on Prestressed Concrete Cylinder Pipes under Working Pressure

Zhu Zhang ¹, Tongchun Li ^{1,*}, Lanhao Zhao ¹ and Huijun Qi ²

¹ College of Water Conservancy and Hydropower Engineering, Hohai University, Nanjing 210098, China

² College of Computer and Information, Hohai University, Nanjing 210098, China

* Correspondence: ltchhu@163.com

Abstract: The precise analysis of the overall mechanical performance of prestressed concrete cylinder pipes (PCCPs) with broken wires is a complicated problem. In this article, powerful finite element numerical means are applied to solve this problem. Firstly, the advantages and shortcomings of the current prestress simulation methods in the finite element analysis (FEA) literature are discussed, and the initial stress method based on a novel single-spring joint element method to model the pre-stress of wires is proposed. Then, different distributions of broken zones, including random broken distributions, are developed for two typical PCCP pipes with different wire-wrapped layers. The established nonlinear FEA model considers actual service processes such as manufacturing, installation and operation to investigate the mechanical behavior of two typical PCCP pipes with different breakage distribution regions and breakage ratios, and in particular, the overall mechanical behavior of a pipe with random breakage is determined first. To verify the accuracy of the proposed pre-stress simulation method and the established nonlinear finite element model, the overall mechanical response of a pipe before wires broke is compared with the results obtained via the specifications. The computed results under the corresponding breakage assumptions are consistent with the conclusions in the existing literature, providing important guidance for pipeline management and operation.

Keywords: finite element analysis; broken wires; random distribution; initial stress method

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1. Introduction

Prestressed concrete cylinder pipes (PCCPs) are broadly applied in urban water-diversion engineering projects. They have many advantages such as a good ability to bear high internal water pressure, high resistance to earthquakes and outstanding economic values. Their first application was in France, and they were then introduced to America in 1942. Since then, more than 22,000 miles of PCCPs have been in service in North America [1]. Recently, they have been applied to many water-diversion projects in China such as the Shenyang Dahuofang Water Conveyance Project, the East of Shenzhen Water Supply Project and the South-to-North Water Transfer Project. Four components, concrete, mortar coating, cylinders and pre-stressing wires, make up PCCP pipes. Concrete has high-strength properties and is the main component of the pipes, providing a smooth surface to maintain a low roughness coefficient and reduce hydraulic loss. The steel cylinder mainly plays an anti-seepage role and has a certain load-bearing capacity. The spiral-wound pre-stressing wire is the main load-bearing part, providing uniform pre-compressive stress to withstand the internal water pressure. The mortar coating serves as an essential material to protect the high-strength steel wires from corrosion by groundwater or soil around the pipeline for more than 50 years [2]. There are two major kinds of PCCPs: Lined-Cylinder Pipes (LCPs) and Embedded-Cylinder Pipes (ECPs) [3]. The wires of LCPs spirally wrapped on a steel cylinder. ECPs high-strength steel wires helically wrapping on the outer concrete of a tube core (see Figure 1), which are put to use in water transmission projects on a large scale at home and abroad with larger internal diameters. The overall mechanical performance of

ECPs is very important and has naturally been the focus of special attention from scholars and researchers.

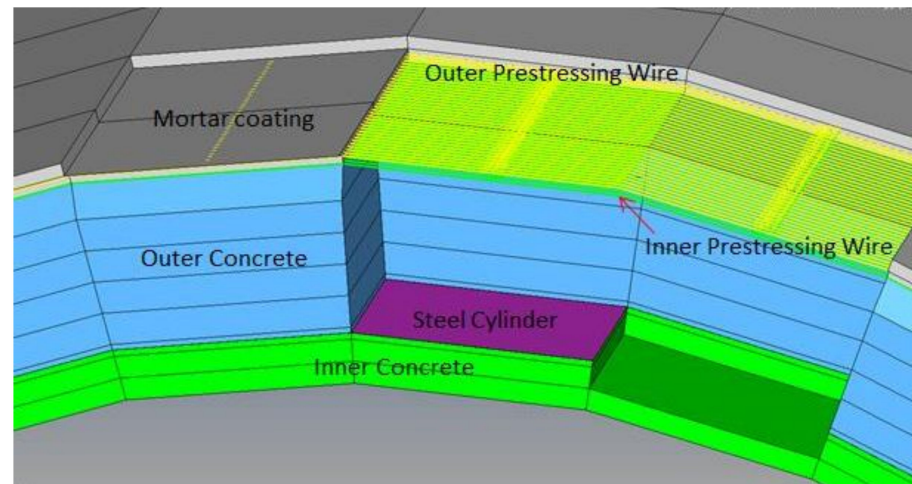


Figure 1. ECP components in ECP finite element model.

PCCP pipes are buried under the ground, so the surrounding groundwater and soil may corrode the pipes, resulting in the general deterioration of function and reductions in service life [4]. They are considered reliable and safe, with the low percentage of wire breakages, according to a recent study [5]. It showed that this percentage was only 3.7%. If they burst, however, this will cause catastrophic ruptures and leaks, resulting in serious losses of life and property. The well-known failure mode of PCCPs is bursting damage (see Figure 2), which is essentially caused by the corrosion and hydrogen embrittlement of wires [6–8]. Currently, wire break-off detection and leak detection methods are broadly used to evaluate the status of PCCPs regularly or in real time. However, it is difficult to determine the accurate number and location of broken wires through the mainstream electromagnetic eddy current inspection methods. In addition, common technologies such as acoustic emissions, which is the only technology applied in long pipelines, fail to detect leaks [9].



(a)



(b)

Figure 2. Bursting pipe: (a) after the bursting event, (b) during the bursting event.

The substitution of an existing partially broken pipe with a whole undamaged pipe is not an economical approach [10], so it is very essential to assess the residual structure integrity of an ECP with broken wires. Numerical simulations, prototype tests and qualitative decision methods are generally used to assess the condition of pipes with broken wires.

Numerical simulations are widely used, which can provide further directions before an ECP fails or bursts. You [11] developed a simplified structural mechanics model based on the classical theory of cylindrical shells. Zarghamee et al. [12] calculated the circumferential thrust and bending moment of a tube wall due to broken wires and verified the results via finite element analysis (FEA). Though a simplified model and finite element method (FEM) model could be easily established, because it neglects the stiffness of wires, it does not reflect real life. Zarghamee et al. [13] considered two different geometries of a pre-stress loss zone. One was axisymmetric, in the form of a band, and the other was rectangular. Afterwards, he simulated the cracking process and the interlocking behavior of a cracked concrete core and investigated the strength of a cracked concrete outer core under the pressure of an expanding steel cylinder using the nonlinear FEA method [14]. The stiffness of wires, the earth load and pipe loads were calculated using the initial pipe design standard.

Gomez et al. [15] calculated the stress status of the middle zone of a pipe with different numbers of broken wires. Diab et al. [16] used both two-dimensional (2D) and three-dimensional (3D) models to present the mechanical behavior of the remaining structure via elastic FEA with three distribution zones. Ge et al. determined the effect of the adhesion quality of the mortar coating on the overall performance of PCCPs by only conducting elastic analysis [17]. The pipe performance as the water pressure and number of broken wires increased was studied by Alavinasab et al. [18,19]. Then, Hajali et al. [20,21] compared the effects of broken wire wraps in the middle of a pipe with those at a joint. In addition, Xiong et al. [22] proposed a new model called the wire-wrapping model to simulate pre-stress. Hu et al. [23] presented a novel bar-wrapping calculation model under pre-stress conditions for a new type of water conveyance pipe called a bar-wrapped cylinder concrete pressure pipe. Zhai et al. [24–26] considered two distributions (removing all of the broken wire and considering the partial loss of pre-stress through simplified contact interactions) in broken pipes and analyzed the strengthening effect via carbon fiber reinforced polymer (CFRP). Dong et al. [27] defined a cohesive contact interaction to simulate the delamination of the mortar coating in broken pipes.

Four methods are generally used to simulate the pre-stress of steel wires in FEA: the equivalent load method, the initial strain method, the equivalent cooling method, and the wire-wrapping model. Most researchers adopted the equivalent load method [11–20], which regards the pre-stress of steel wires as a form of radial load. The simulation is simple and direct, without consideration of the location of steel wires, and it is easy to converge. However, the stiffness of wires is neglected, causing distortion of the simulation results. In the initial strain method conducted by Lin [28], pre-stress was applied to steel wire by applying an initial strain to the wire. In this method, the pre-stress loss usually cannot be considered since it is represented by stress. Most researchers [24–26,29,30] used the equivalent cooling method. This method constructed wire elements to apply pre-stress and take the wire stiffness into account by increasing the temperature field and equalizing the initial pre-stress to the cooling temperature of the wire. However, cooling the wire keeps it in a compressed state, which is contrary to the initial tensile state seen in practice. In addition, if the nonlinear behavior of the material and contact is considered in the nonlinear FEA, adding the temperature field undoubtedly makes it a more complex nonlinear coupled problem, which increases the solution difficulty and reduces convergence. Xiong et al. [22] put forward the wire-wrapping model, which is very complex and inconvenient to apply.

A few scholars have carried out experimental analysis studies on pipes with broken wires. Hu [29] and Shen [31] conducted a three-edge bearing test under an external load test with a 4 m-diameter, single-layer, wire-wrapped pipe. Dou et al. [32] carried out an internal hydraulic test on a PCCP with an inner diameter of 2.6 m and compared the repairing effect with a PCCP reinforced with restoration materials.

Hu et al. [33] developed an internal pressure test for a full-scale, single-layer PCCP with an inner diameter of 2.8 m and a broken wire percentage of 0% (intact), 5% and 10%. Luo et al. [34] established the mathematical evaluation model of safety risk for PCCP pipelines by adopting the analytic hierarchy process (AHP) method.

However, due to the complexity of ECPs, it is difficult to further simulate the mechanical behavior of pipes with broken wires by considering construction process simulation, nonlinear constitutive models of materials and contact interaction. Due to the test costs and technical difficulty, it is difficult to carry out prototype field tests on large-diameter pipes with broken wires in actual operational status. Therefore, the most practical method is to evaluate the mechanical behavior of a broken pipe under actual operating conditions by using the numerical simulation method. In this paper, a location hypothesis of broken wires is presented, and a random broken case induced by congenital manufacturing defects, such as hydrogen embrittlement, is considered for the first time. Meanwhile, to avoid the shortcomings of modeling the pre-stress of wires, a pre-stress simulating method called the initial stress method, based on a novel single-spring joint element method, is proposed. By considering material nonlinearity, contact nonlinearity and the bond–slip relationship in reinforced concrete (RC) structures, and by choosing a proper pre-stress simulation method, we used the FEM method to model two large-diameter ECP pipes with different bar-wrapping layers. The results achieved from nonlinear FEA are analyzed and compared.

2. Location Hypothesis of Broken Wires

The deterioration process of PCCPs with clustered broken wires can be divided into four stages, as shown in Figure 3 [27]. If the number of broken wires continues to increase, the mortar coating will peel off, and the prestressed wires will pull away, revealing visible cracks on the concrete core. Finally, the steel cylinder yields and bulges out, and the outer concrete core falls off [27]. In fact, each wire layer of an ECP is a consistent pre-stressing wire helically wrapping around the tube core, which is meshed into bar elements with dependent circumferential distribution. If a wire breaks at any point, the whole circle of steel wire will release force based on the related test and will be removed in the model [29].

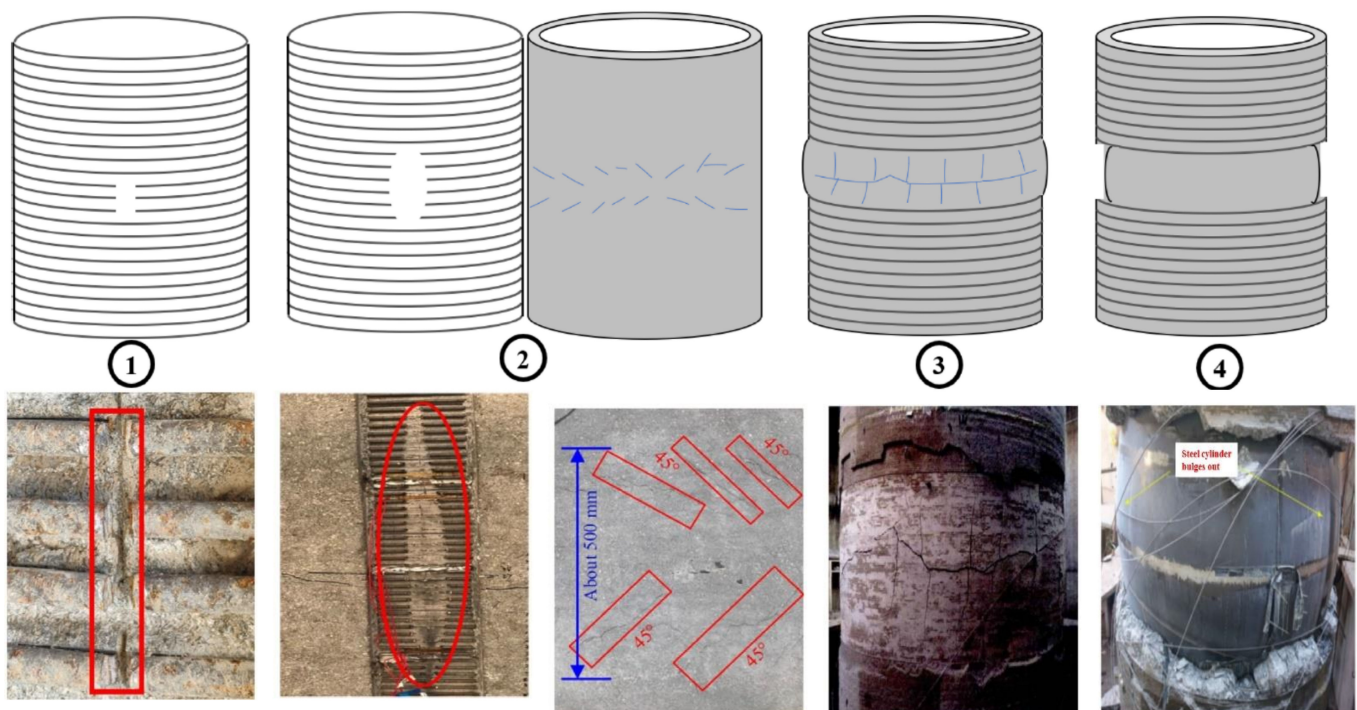


Figure 3. ECP components in ECP finite element model [27].

For pipes with two wire-wrapped layers, it is assumed that they break continuously in two layers at a joint (Figure 4a), the middle location (Figure 4b) and randomly in any location (Figure 4e). Because the existing breakage detection technology cannot detect whether a breakage occurs in the outer layer or the inner layer of a double-layer ECP, and accounting for the fact that stress in the outer layer is more than that in the inner

layer, the scenarios of a continuous break in the outer layer at a joint (Figure 4c) and the middle location (Figure 4d) in a double-layer ECP are considered. For pipes with a single layer, it is presumed that they break consistently at the middle, joint and random locations (not drawn).

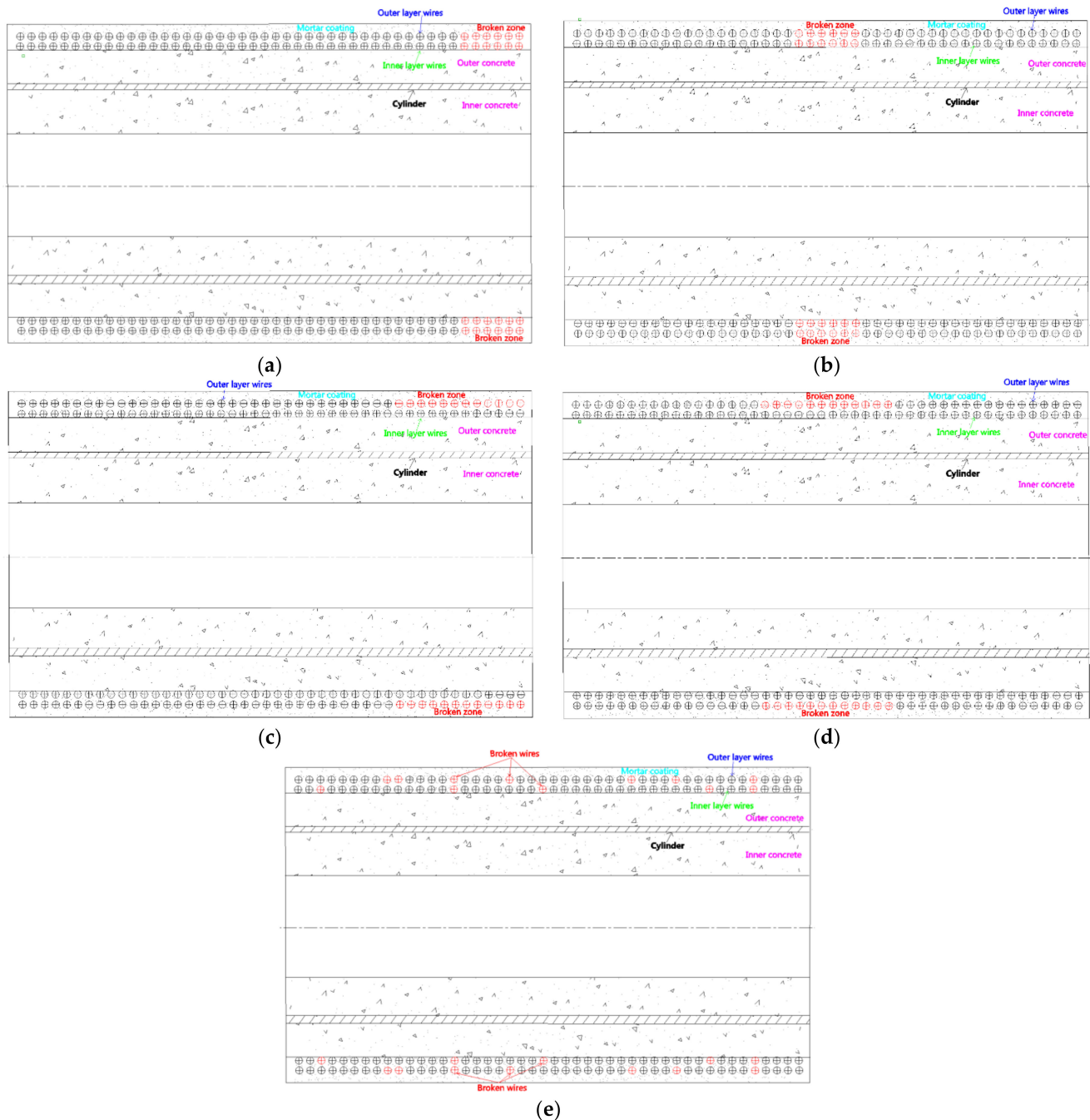


Figure 4. Location of broken wires: (a) two layers at joint, (b) two layers at middle, (c) outer layer at joint, (d) outer layer at middle, (e) random distribution.

3. Pre-Stress Simulating Methods

To avoid shortcomings when modeling the pre-stress of wires, a new pre-stress simulating method called the initial stress method, based on a novel single-spring joint element method, is proposed. In the novel single-spring joint element method, the tangential action between bars and concrete is modeled by the tangential spring, and the normal direction

degree is compelled to be equal (see Figure 5). The related principles and derivation arrangements can be seen in the literature [35].

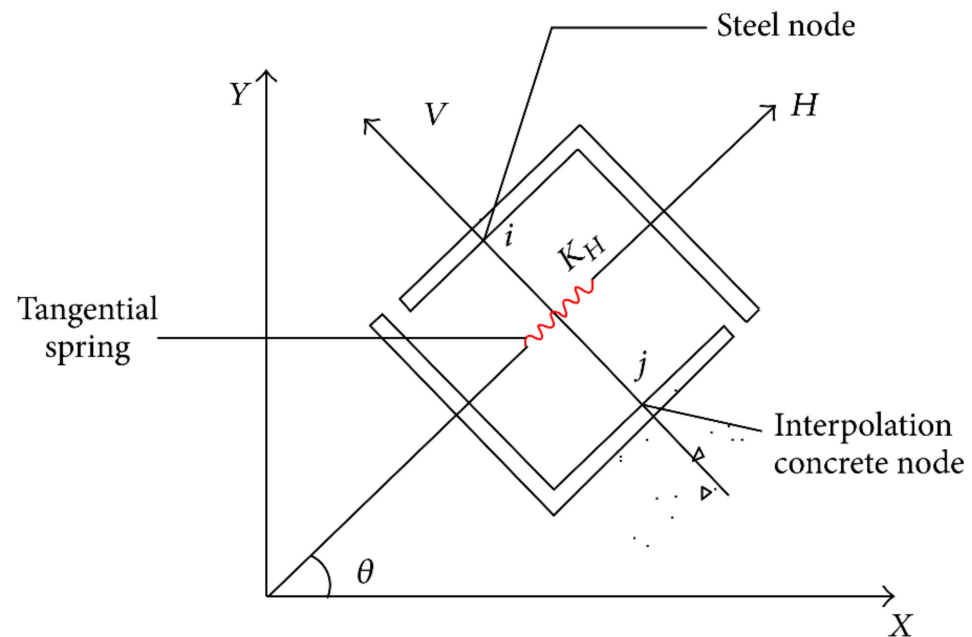


Figure 5. Single-spring joint element model for 2D problem [35].

The incremental equilibrium equation for the novel single-spring joint element method is as follows:

$$\begin{bmatrix} K + r^T K_s r & -r^T K_s \\ -K_s r & K^* + K_s \end{bmatrix} \begin{Bmatrix} \Delta u \\ \Delta u^* \end{Bmatrix} = \begin{Bmatrix} \Delta F \\ \Delta F^* \end{Bmatrix} \quad (1)$$

where K , u , F , K^* , u^* , F^* are the stiffness matrix and the displacement and load vector in the global and the nodal local coordinate system, respectively. K_s is the tangential stiffness

matrix, where $K_s = \begin{bmatrix} k_s & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & 0 \end{bmatrix}$ and $k_s = \frac{\partial \tau}{\partial s} \pi D l$, and D and l are the diameter and length

of the reinforcement, respectively. r represents the coordinate conversion matrix between the global and the local coordinate system of the reinforcement node, and $\tau - s$ represents the bond-slip curve obtained from experiments.

When solving the initial stress field of a PCCP from the initial stress of prestressed wire, Equation (1) can be rewritten as follows:

$$\begin{bmatrix} K + r^T K_s & -r^T K_s \\ -K_s r & K^* + K_s \end{bmatrix} \begin{Bmatrix} u \\ u^* \end{Bmatrix} = \begin{Bmatrix} F_0 \\ F_0^* \end{Bmatrix} \quad (2)$$

where $F_0^* = -\int_l \mathbf{B}^T \sigma_0 ds$, $\mathbf{B}$ is the strain matrix of wire elements, σ_0 is the initial stress vector of the wire elements and $F_0 = r^T F_0^*$.

Because the wire-wrapping process is completed at once, Equation (2) is not solved by the increment method, but by the total iterative method. The simulating method is easy and direct with good convergence. Most importantly, the pre-stress loss can be readily considered.

4. Finite Element Model and Material Properties

4.1. Finite Element Model

Two classes of ECP grid models based on the China South–North Water Transfer Project with 4 m diameter were established. The initial design parameters of the two pipes are summarized in Table 1, and the established finite element models are shown in Figure 6.

For pipe No.1, the total number of elements was 123,488; for pipe No.2, 89,504 elements were used. We established five pipes along the axial direction and chose the middle pipe among the left pipeline pipes as a research target to minimize the boundary effect. Meanwhile, we simulated the pre-stressing wires as bar elements and modeled the cylinder with plate elements. The top surface of the FEA model was unconstrained, and normal fixed constraints were applied to other faces.

Table 1. The design parameters of pipes.

Pipe	Working Pressure (MPa)	Buried Depth (m)	Thickness of Tube Core (m)	Wrapping Area (mm ²)	Wire Diameter (mm)	Thickness of Cylinder (mm)
No.1	0.8	10.0	350	5497	7	2
No.2	0.4	3.0	320	2628	7	2

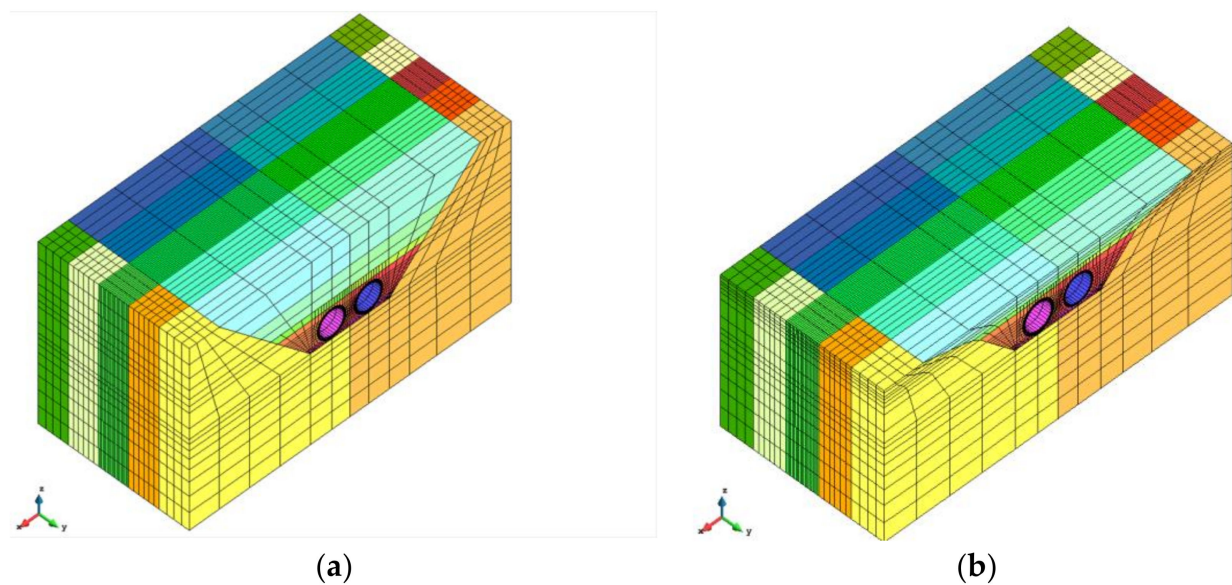


Figure 6. FEM model of pipe: (a) pipe No.1, (b) pipe No.2.

The contact interaction between pipes and soil and pipe segments were modeled. Meanwhile, the single-spring joint element method was chosen to model the interaction effect of bars and concrete. The ideal elastic–plastic model was adopted as the constitutive relationship of a cylinder based on the Mises yield criterion and supposing the cylinder has complete bond–slip ability with concrete due to its thin thickness. Three broken wires ratios were set to 10%, 20% and 30%.

4.2. Material Properties

The material properties are presented as Table 2.

Table 2. Material properties of pipes.

Material	Grade	Density (kg·m ^{−3})	Young's Modulus (GPa)	Poisson's Ratio	Compressive Strength (MPa)	Yield Strength (MPa)	Tensile Strength (MPa)
Concrete	C60	2323	28.6	0.167	48.0	—	4.03
	C50	2323	27.0	0.167	40.0	—	3.67
Mortar	C60	2243	25.3	0.167	37.9	—	3.58
Wires	—	7833	191	0.29	—	1334.5	1570.0
Cylinder	—	7833	207	0.29	—	227.5	310.0

For pipe No.1, concrete is C60 grade, and the other one is C50.
The basic material properties of the soil can be seen in Table 3.

Table 3. The basic material properties of soil.

Kind	Density (kg·m ⁻³)	Young's Modulus (MPa)	Poisson's Ratio
Foundation	2039	80.0	0.25
Bedding	2039	40.0	0.29
Backfill	2039	50.0	0.35

A four-parameter failure criterion [35,36] was adopted, which took the stress–strain curve reported in research document [3,22] as the damage constitutive curve of concrete and mortar to obtain the damage value. The Dun-Chang E-B model [37,38] was used as a constitutive model for soil. The parameters of the Dun-Chang E-B constitutive model for backfill soil are presented as Table 4. We employed the Goodman joint element [39] to model the contact effect.

Table 4. Dun-Chang E-B constitutive model coefficients of soil.

Kind	Nondimensional Parameter (K)	Nondimensional Index (n)	Failure Ratio (R _f)	Experimental Parameter (K _b)	Experimental Parameter (m)	Cohesion Force (Unit: Pa)	Internal Friction Angle (Unit: °)
Foundation	215.5	0.92	0.613	151.01	0.75	5.0 e3	30
Bedding	232.6	0.90	0.613	165.05	0.75	3.0 e3	30
Backfill	198.4	0.89	0.613	132.27	0.75	4.0 e3	30

We chose the nonlinear bond–slip constitutive curve developed by Di [40] due to the absence of bond–slip testing curves between high-strength concrete and plain wires, which were taken from the test of ordinary concrete and crescent wires, as presented in Equation (3):

$$\tau = 6.59 \times 10^2 s - 2.13 \times 10^4 s^2 + 0.22 \times 10^6 s^3 \quad (3)$$

where τ is the bond stress (MPa) and s is the slip displacement (mm). The bond stress was set only one-tenth of the value in Di's curve by taking the fact that plain wires were applied in this project into consideration. Meanwhile, the constitutive curve of wires can be seen in this paper [22].

5. FEA Results

In practice, a pipe is produced, then placed for a long time in an outdoor space and finally installed. After a series of water pressure tests, the pipe is applied in normal use; then, wires start to break. Combining the analyzing accuracy and calculation efficiency, based on the birth–death method, the FEA considering manufacturing, installation and operation was divided into eight steps:

- (1) The tube core and inner layer wires are involved in the calculation, and the initial stress of inner layer wires is applied.
- (2) The outer layer of wires is added to the calculation, and the initial stress is applied.
- (3) The concrete shrinkage and creep factors are calculated according to the specification [3] and taken into account in FEA due to the long-time storage in outdoor space.
- (4) Bedding soil and bottom backfill soil are added to the calculation, and meanwhile, the pipe weight is applied.
- (5) Middle backfill soil is added to the calculation.
- (6) The remaining backfill soil is added to the calculation.
- (7) Apply internal water pressure.
- (8) Broken wires are removed from the calculation based on different breakage hypotheses. The backfill image of the FEA model for pipe No.1 is presented in Figure 7.

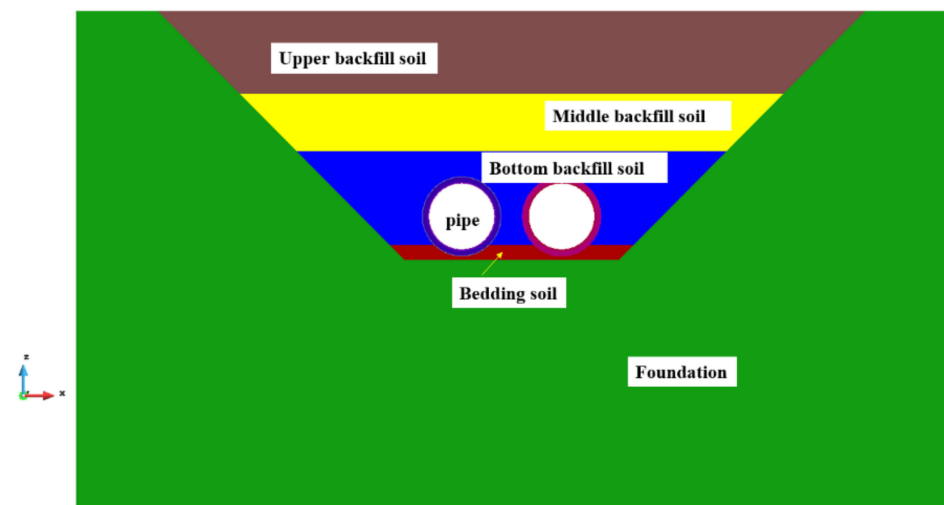


Figure 7. The backfill diagram of FEA model (pipe No.1).

For pipe No.2, step (2) and step (5) were neglected, since there was only a single layer and a low embedment depth.

The factors of creep, shrinkage and wire relaxation were determined by the specification [3] and are shown in Table 5.

Table 5. Wire-relaxation, creep and shrinkage factors.

Pipe	Wire-Relaxation Factor (Outer Wire Layer)	Wire-Relaxation Factor (Inner Wire Layer)	Shrinkage Factor	Creep Factor
No.1	0.127	0.108	6.6617×10^{-5}	1.177
No.2	—	0.082	9.7186×10^{-5}	1.182

The circumferential stress range of concrete for pipe No.1 in step (3) was 10.82 MPa to 12.63 MPa with even distribution, displayed in Figure 8. However, the specification [3] assumes that the cross-sectional stress of the entire tube core is 12.09 MPa. We present the results of pipe No.1 by the standard method and FEA in Table 6 to verify the accuracy of the finite element calculation before wires are broken. The results indicate that various indexes are basically consistent for both methods. Altogether, based on the proposed pre-stress simulation method, the 3D nonlinear FEA shows great consistency with the standard method in simulating ECPs before wires are broken, so the proposed FEA model is suitable for use to evaluate the overall performance of ECPs.

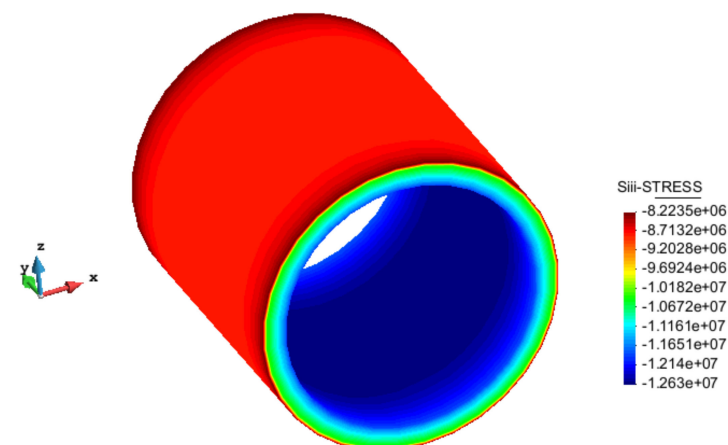


Figure 8. The circumferential stress of the tube core after wire-wrapping and outdoor storage (pipe No.1).

Table 6. Main analysis results of pipe No.1 before wires broke.

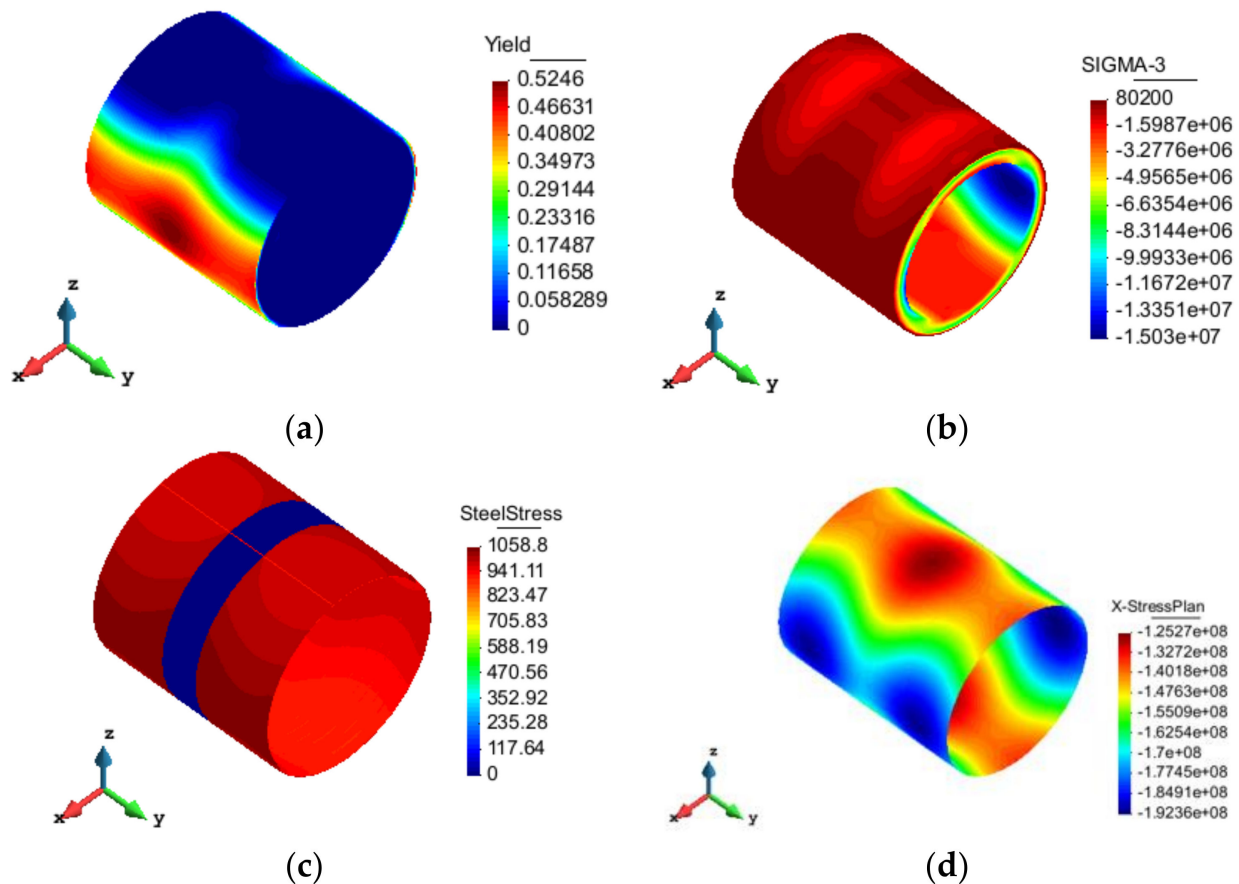
Method	Circumferential Stress in Concrete after Outdoor Storage (MPa)	Strain in Mortar Coating	Strain in Outer Concrete	3rd Principal Stress in Inner Concrete (MPa)	Stress in Wires (MPa)	Stress in Cylinder (MPa)
Standard method	−12.09	0.00056	0.00004	19.7	1024.1	−117.9
FEA	−10.82~−12.63	0.00035	0.000157	17.3	1039.7	−138.9

5.1. Finite Element Results of Pipe No.1 after Breaking Wires

Various scenarios combine the breakage ratios with distributional locations. Several results regarding the 10% broken wire ratio were selected to describe the state of pipes with broken wires.

5.1.1. Case 1: 10% Breakage Ratio and Breaking at the Outer Pre-stressing Layer in the Middle of Pipe

The resulting nephogram is shown in Figure 9.

**Figure 9.** Nephogram of case 1: (a) damage, (b) 3rd principal stress, (c) wire stress, (d) cylinder stress.

The maximum damage value of the mortar is 0.52, which increases by 0.16 compared with the pre-breaking stage. The damage to the outer concrete starts growing at the springline in the broken area, with the maximum value of 0.24. The maximum stress in the outer-layer wires increases to 1058.8 MPa, with the increment of 19 MPa compared with the pre-breaking stage. Meanwhile, the stress in the inner wires near the pre-stressing loss zone also increases, which increases to around 11.0 MPa at the springline. Once the outer-layer wire in the middle of pipe is broken, the stress in the unbroken and inner wire on the side of the broken area will increase, and the stress in the cylinder will rise. The damage on the

outer concrete will expand but not penetrate along the whole circumferential direction, and the inner concrete will not be damaged temporarily. The internal water load borne by the broken wire will be redistributed to the above materials.

5.1.2. Case 2: 10% Breakage Ratio and Breaking at the Outer Pre-stressing Layer at the Joint of Pipe

The resulting nephogram is shown in Figure 10. It shows that the maximum damage value of mortar is 0.55, which increases by 0.19 compared to the pre-breaking stage. At the springline in the broken area, the damage to the outer concrete starts growing, showing the maximum value of 0.30. The maximum stress in the outer-layer wires reaches 1058.9 MPa with the increment of about 19 MPa. At the same time, the stress in the inner wires and the stress in the cylinder around the pre-stressing loss zone increase. The damage to the outer concrete will not only expand but also penetrate along the whole circumferential direction, and the inner concrete is also damaged.

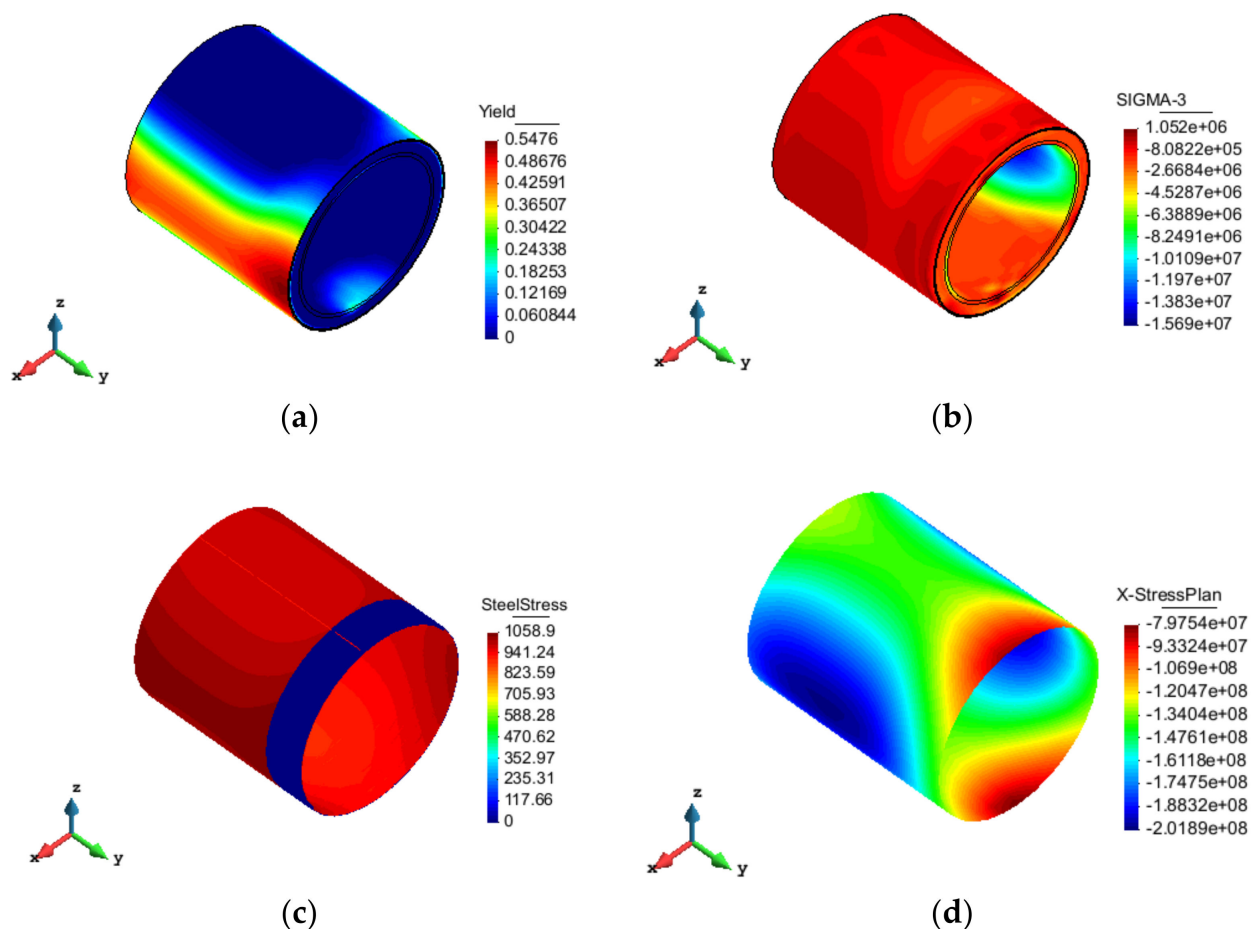


Figure 10. Nephogram of case 2: (a) damage, (b) 3rd principal stress, (c) wire stress, (d) cylinder stress.

5.1.3. Case 3: 10% Breakage Ratio and Breaking at Both Outer and Inner Layer in the Middle of Pipe

The obtained nephogram is shown in Figure 11. The maximum damage value of the mortar is 0.53. The damage to the outer concrete emerges at the springline in the broken area but does not penetrate along the whole circumferential direction, with the maximum value of 0.29. Damage also occurs at the invert and crown inside the pipe in the pre-stressing loss zone. The maximum stress in the outer wires approaches 1060.4 MPa. Meanwhile, the stress in the inner wires and the stress in the cylinder around the pre-stressing loss zone increase as well.

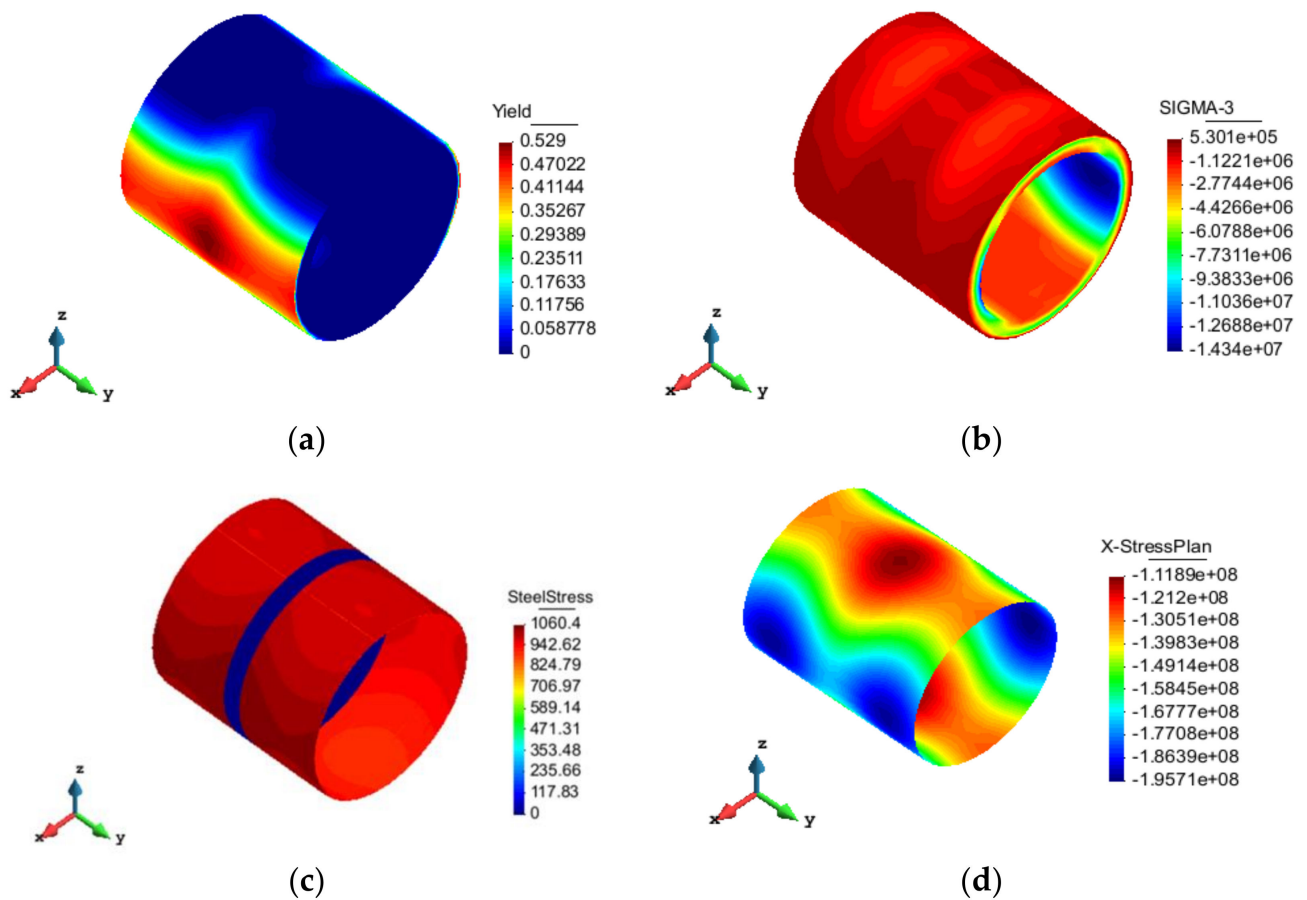


Figure 11. Nephogram of case 3: (a) damage, (b) 3rd principal stress, (c) wire stress, (d) cylinder stress.

5.1.4. Case 4: 10% Breakage Ratio and Breaking at Both Outer and Inner Layer at the Joint of Pipe

The resulting nephogram is presented in Figure 12. The maximum damage value of the mortar is 0.57. At the springline in the broken area, the damage to the outer concrete emerges, exhibiting the maximum value of 0.36. Damage occurs at the invert and crown inside the pipe in the pre-stressing loss zone. The maximum stress in the outer wires reaches 1076.4 MPa. In addition, the stress in the inner wires and the cylinder around the pre-stressing loss zone also increases.

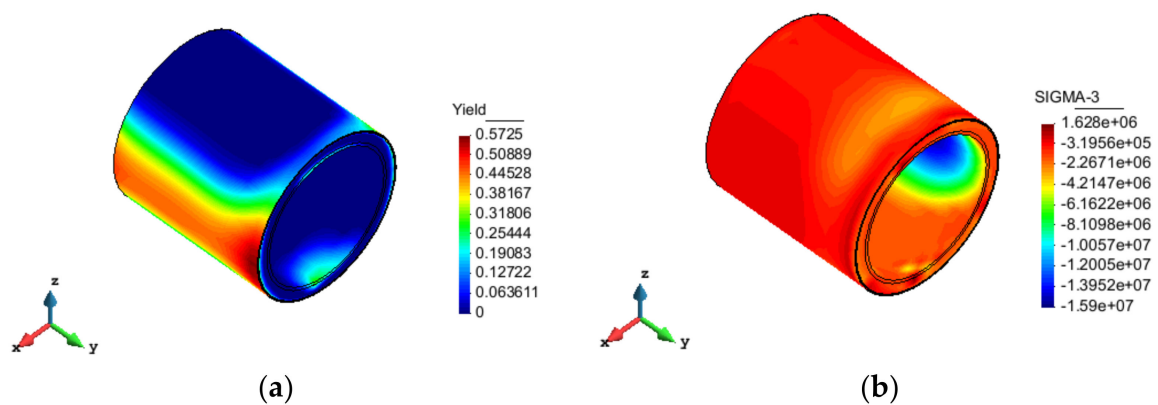


Figure 12. Cont.

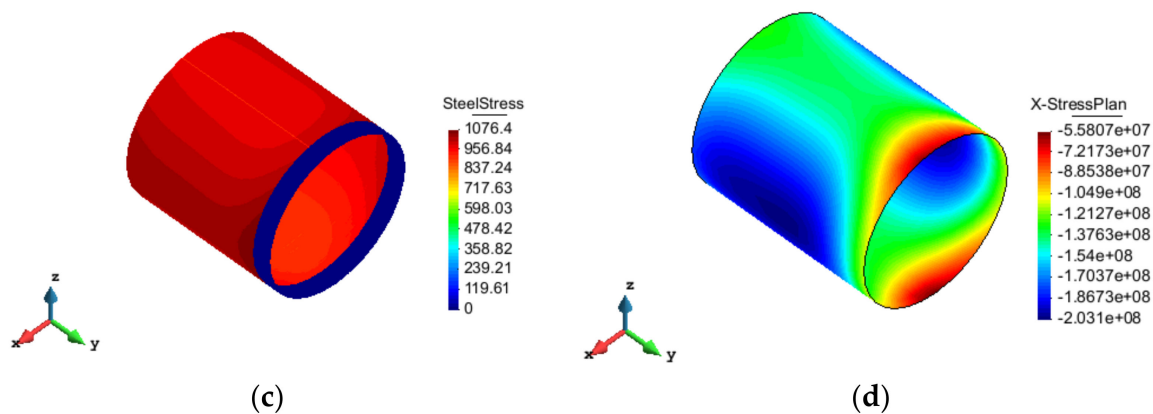


Figure 12. Nephogram of case 4: (a) damage, (b) 3rd principal stress, (c) wire stress, (d) cylinder stress.

5.1.5. Case 5: 10% Breakage Ratio and Breaking at Random Location

The obtained nephogram is illustrated in Figure 13. The mortar presents a maximum damage value of 0.39 with a small increment compared with the pre-breaking stage. The damage to the outer concrete emerges at the springline and penetrates along the whole circumferential direction, with the maximum value of 0.22. A small amount of damage to the inner concrete occurs at the invert and crown. The damage distribution in concrete and mortar is uniform along the axial direction. The maximum stress in the outer wires reaches 1057.1 MPa with the increment of 17.4 MPa. In addition, the stress in the cylinder also rises. Because the breakage ratio is small with a relatively scattered broken area, when the random broken situation occurs once, the stress in the wires next to the broken area increases, and the cylinder stress changes a little.

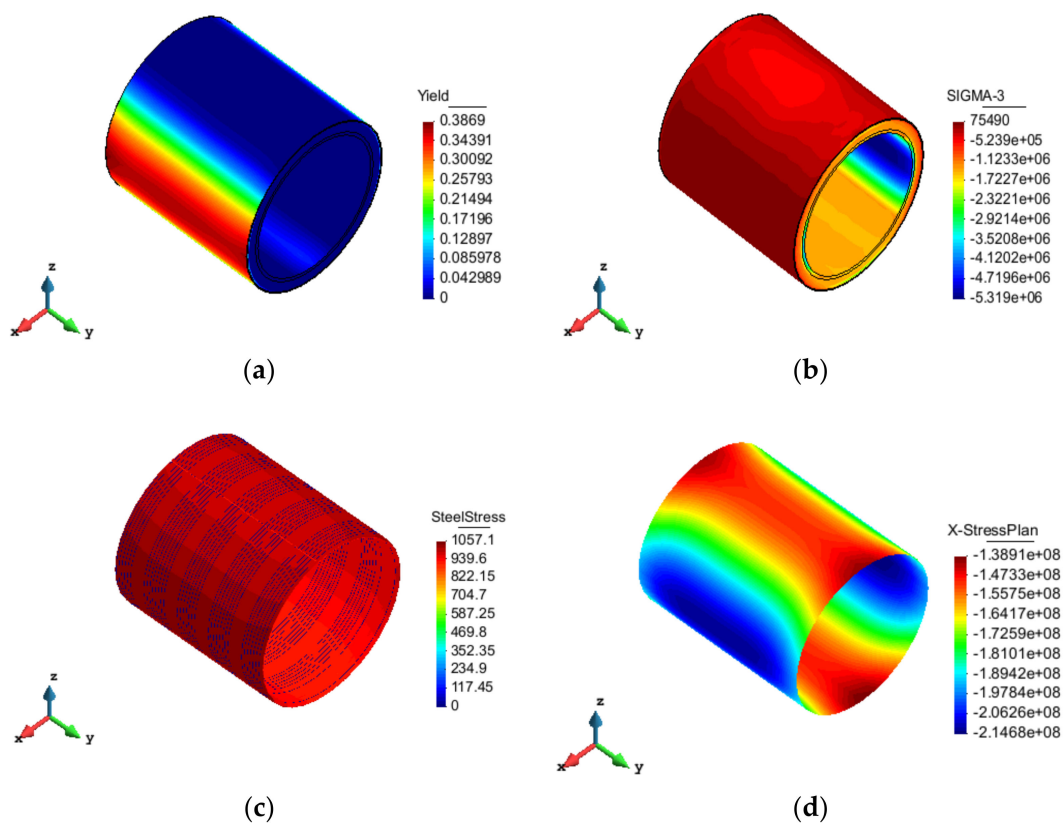


Figure 13. Nephogram of case 5: (a) damage, (b) 3rd principal stress, (c) wire stress, (d) cylinder stress.

5.2. Finite Element Results of Pipe No.2 after Breaking Wires

5.2.1. Case 6: 10% Breakage Ratio and Breaking in the Middle of Pipe

It can be seen from Figure 14 that the maximum damage value of mortar is 0.39, with an increment of 0.22 compared with the pre-breaking stage. At the springline in the broken area, the damage to the outer concrete emerges and penetrates along the whole circumferential direction, exhibiting the maximum value of 0.20. Damage occurs at the invert and crown inside the pipe around the pre-stressing loss zone with the value of 0.10. The maximum stress in the outer-layer wires grows to 1031.1 MPa. Meanwhile, the stress in the inner wires and the stress in the cylinder around the pre-stressing loss zone are also enhanced. The damage to the outer concrete will not only expand but also penetrate along the whole circumferential direction, and the inner concrete is also damaged.

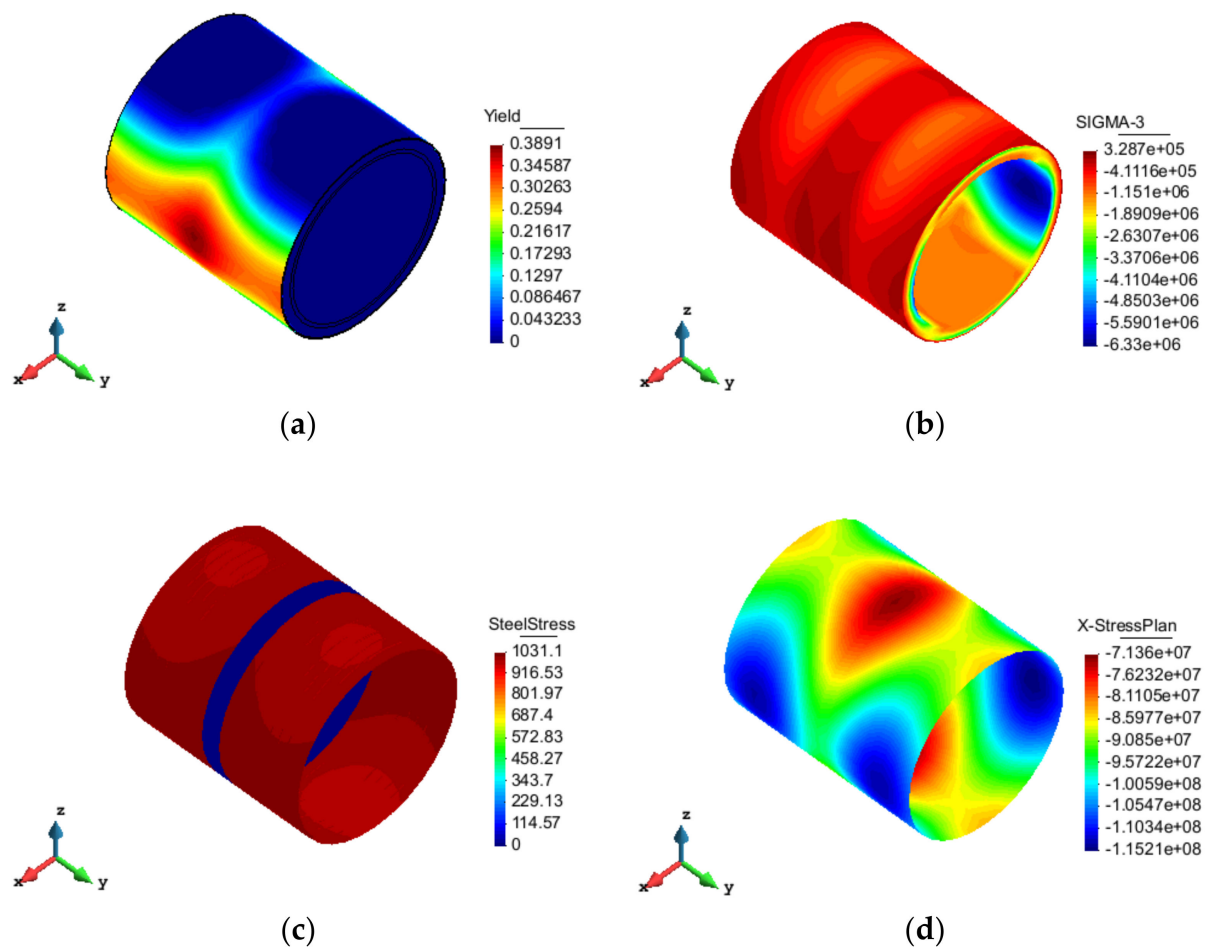


Figure 14. Nephogram of case 6: (a) damage, (b) 3rd principal stress, (c) wire stress, (d) cylinder stress.

5.2.2. Case 7: 10% Breakage Ratio and Breaking at the Joint of Pipe

The resulting nephogram is displayed as Figure 15. The maximum damage value of mortar is 0.42, with an increase in value of 0.25 compared with the pre-breaking stage. The damage to the outer concrete emerges at the springline in the broken area and penetrates along the whole circumferential direction, presenting the maximum value of 0.28. Damage can be observed in the invert and crown inside the pipe around the pre-stressing loss zone with the value of 0.12. The maximum stress in the outer-layer wires increases to 1037.4 MPa, along with the increased stress in the inner wires, the third principal stress in the inner concrete and the stress in the cylinder around the pre-stressing loss zone. The damage to the outer concrete will not only expand but also penetrate along the whole circumferential direction, and the inner concrete is also damaged.

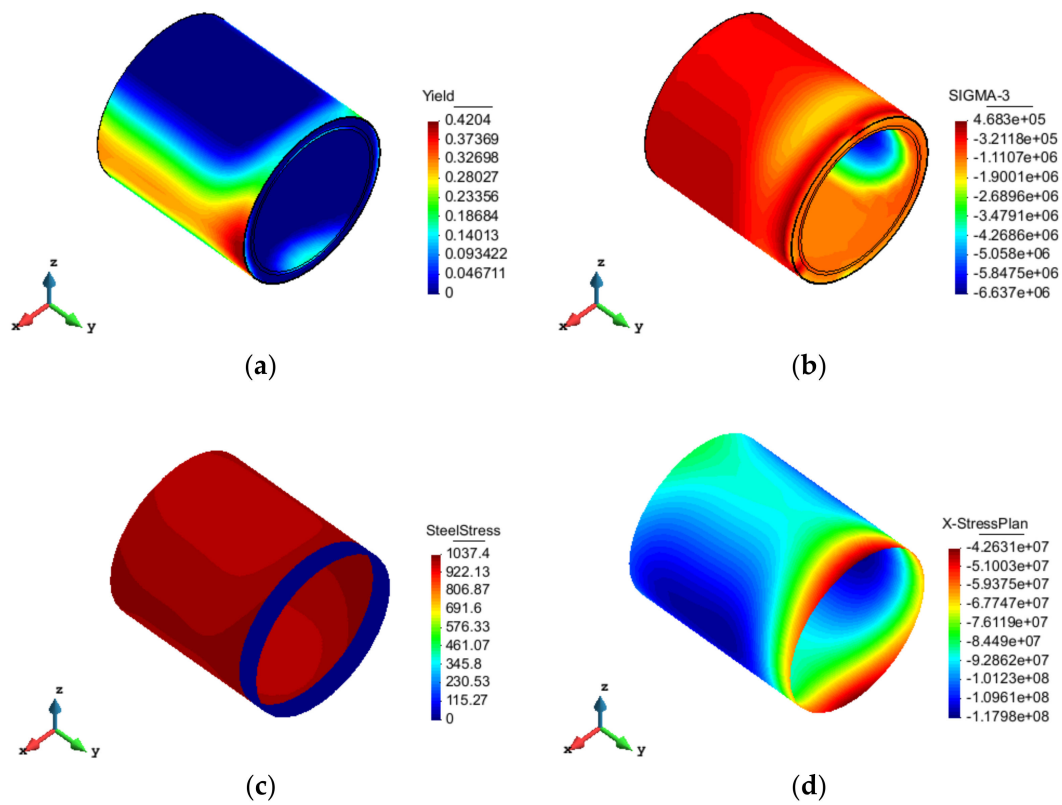


Figure 15. Nephogram of case 7: (a) damage, (b) 3rd principal stress, (c) wire stress, (d) cylinder stress.

5.2.3. Case 8: 10% Breakage Ratio and Breaking at Random Location

The obtained nephogram is shown in Figure 16.

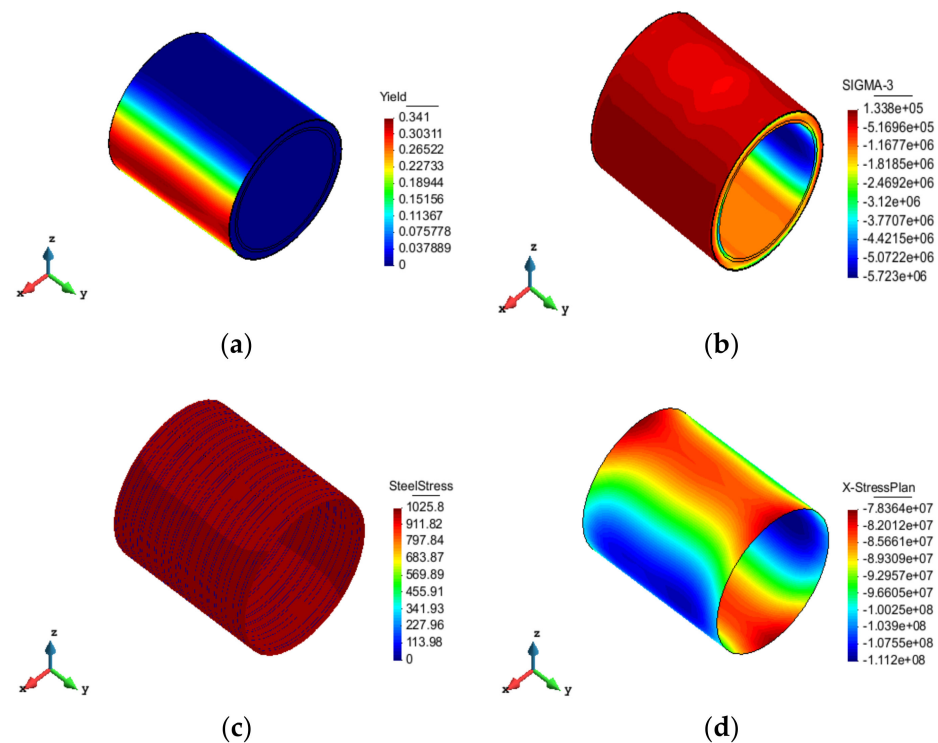


Figure 16. Nephogram of case 8: (a) damage, (b) 3rd principal stress, (c) wire stress, (d) cylinder stress.

As illustrated in Figure 16, the maximum damage value of mortar is 0.34, with an increment of 0.14 compared with the pre-breaking stage. At the springline, the damage to the outer concrete emerges and does not penetrate along the whole circumferential direction, exhibiting the maximum value of 0.16. A small amount of damage occurs at the invert and crown inside the pipe. The damage distribution in concrete and mortar is uniform along the axial direction. The maximum stress in the outer-layer wires approaches 1025.8 MPa with the increment of 11.0 MPa. At the same time, the third principal stress in the inner concrete and the stress in the cylinder increase. The damage to the outer concrete only expands, and the inner concrete is not damaged.

5.3. Summary of FEA Results

The FEA results of two pipes with 10% broken wire ratios are presented above. The other results are listed in the tables, with the results of pipe No.1 listed in Table 7, and those of pipe No.2 listed in Table 8. From the table, it can be seen that when the same number of wires are removed from the joint and middle of the pipe with the same breakage ratio, the area without pre-stressing wires at the joint is larger, so breaking at joint is more dangerous than that in the middle of the pipe [21]. For the double-layer broken pipe with a 10% breakage ratio, the outer concrete is circumferentially penetrated along the broken area in both assumptions of the outer layer or both layers at the joint, and the inner concrete also shows damage at the crown and invert, which does not occur in the middle breakage. As the ratio of wire breakage increases, the damage value and area of the outer concrete expands, penetration also occurs in the central wire breakage and damage occurs in the inner concrete. For the single-layer broken pipe with three wire break ratios, whether the wire is broken at the joint or in the middle, the outer concrete shows damage and circumferential penetration along the broken area, and the inner concrete also shows damage. Additionally, as the wire break rate increases, the concrete damage value and damage area further expand but still do not exceed their respective strength limits. Regardless of the single-layer or double-layer broken pipe, when breaks occur consistently in the middle or joint location, the wire stress and cylinder stress near the broken area further increase after the wire break, and the unevenness of the cylinder stress also worsens with the increase in the wire break rate, which may lead to the radial detachment of the cylinder and concrete, further leading to the deterioration of the overall service performance of the pipe.

Table 7. The main analyzing index value of pipe No.1.

Ratio	Maximum Stress of Cylinder (MPa)					Damage of Outer Concrete at Springline					Damage of Inner Concrete at Crown/Invert					Maximum Value of Wires (MPa)				
	Two Layers		Outer Layer		Random Location	Two Layers		Outer Layer		Random Location	Two Layers		Outer Layer		Random Location	Two Layers		Outer Layer		Random Location
	Middle	Joint	Middle	Joint		Middle	Joint	Middle	Joint		Middle	Joint	Middle	Joint		Middle	Joint	Middle	Joint	
10%	−118.9	−55.8	−125.3	−79.8	−138.9	0.29	0.36	0.24	0.30	0.22	0.07	0.30	0.03	0.20	0	1060.4	1076.4	1058.8	1058.9	1057.1
20%	−84.0	−18.8	−93.3	−78.5	−109.9	0.37	0.41	0.30	0.30	0.23	0.18	0.38	0.15	0.22	0	1081.4	1083.7	1062.0	1061.4	1060.8
30%	−59.0	−8.49	−84.0	−78.1	−99.8	0.42	0.50	0.31	0.43	0.23	0.24	0.39	0.17	0.23	0.10	1089.7	1089.8	1062.7	1061.5	1069.6

Table 8. The main analyzing index value of pipe No.2.

Ratio	Maximum Stress of Cylinder (MPa)			Damage of Outer Concrete at Springline			Damage of Inner Concrete at Crown/Invert			Maximum Value of Wires (MPa)			
	Joint		Random Location	Middle		Joint	Random Location		Middle	Joint		Random Location	
	Middle	Joint	Random Location	Middle	Joint	Random Location	Middle	Joint	Random Location	Middle	Joint	Random Location	
10%	−71.4	−42.6	−78.4	0.20	0.26	0.16	0.10	0.17	0	1031.1	1037.4	1025.8	
20%	−53.7	−25.6	−73.5	0.28	0.31	0.19	0.12	0.21	0	1040.6	1041.2	1029.8	
30%	−37.9	−21.7	−70.2	0.32	0.47	0.18	0.19	0.33	0.03	1044.9	1044.5	1032.5	

Under the three random wire breakage rates, for the double-layer pipe, the damage value of the outer concrete expands but does not penetrate, and the inner concrete does not show damage, but its tensile stress increases. The stress in the wire and cylinder do not increase much, and the overall stress uniformity of the cylinder is excellent. For the single-layer pipe, the outer layer of concrete damage does not penetrate along the circumferential direction, and the inner concrete does not develop damage in the 10% and 20% of the broken wire ratio scenario, but it occurs in the 30% ratio scenario.

The results of continuous and random breakage calculations are compared under three breakage ratios. For one pipe, the material damage values and area distribution, wire stress and cylinder stress under the random breakage assumption are smaller than the statistical values of the corresponding parameters under continuous breakage with the same breakage ratio. In addition, under the low breakage ratio, the outer concrete damage under continuous breakage basically penetrates along the circumferential direction, and the concrete and cylinder stress are more uniformly distributed. Since the distribution of random breakage locations is more discrete, the consequences caused by wire breakage are lesser than those under the assumption of continuous breakage. The bearing resistance of the double-layer pipe in the random and outer breakage assumptions is better than that of the single-layer pipe, while in the case of both double-layer breakage it is worse than that of the single-layer pipe. Although the double-layer pipe suffers more inner design water pressure, its integrity is stronger, because the double-layer wire is similar to the double insurance action.

6. Conclusions

A nonlinear FEA was developed to investigate the mechanical behavior of two typical ECPs with broken wires. We considered the actual production, installation and operation stages by dividing the analysis process into six or eight steps. The results were obtained under the hypothesis of broken wires with three ratios and different distributions when suffering under working water pressure and other loads. The main conclusions are summarized as below:

(1) We summarized the advantages and shortcomings of various pre-stressing simulation methods and then proposed an initial stress method based on a novel single-spring joint element method for FEA. The structural response results before wire breakage based on the specification method and the FEA model are basically consistent.

(2) The damage to the outer concrete increases and penetrates along the entire circumferential direction as the percentage of wire breakage grows at continuous wire breakage. Damage to the inner concrete begins to occur, and indicators such as wire stress, cylinder stress and distribution also deteriorate. The risk of breakage is higher in the joint than that in the middle location, and the breakage that occurs with continuous centralized distribution is more dangerous than breakage with random distribution. The double-layer pipe has better overall destruction resistance than that of the single-layer pipe under random breakage and outer continuous breakage conditions.

(3) A cylinder can be used as both an anti-seepage body to prevent seepage and safety reserve under wire breakage. If the tearing cylinder is regarded as the pipe burst indicator, the pipe will not immediately burst in the drafted broken ratios and assumptions. However, micro-cracks in concrete will lead to corrosive groundwater or chemical substances to further corrode intact wires and cylinder. At the same time, the unevenness of the cylinder will potentially lead to radial detachment with concrete. The combined effect will further result in the deterioration of the overall service performance.

Existing prototype tests of broken pipes are only performed for individual internal water load or three-edge bearing tests, and it is still very difficult to conduct test simulations of broken pipes in service states due to technical and economic reasons. At the same time, this paper does not further discuss more breakage ratio sensitivity analysis and ultimate internal water pressure calculation but focuses on research into the mechanical effects of the breakage location of two different pipe types. If we consider the mechanical

behavior of a broken pipe after concrete cracking, it will be related to the fine structure of concrete and crack path development problems, which will be further investigated in follow-up research.

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Article

Experimental Research on Dynamic Response of Layered Medium under Impact Load

Mingqing Liu ^{1,2,*} and Qinyu Gan ³
¹ College of Architecture and Energy Engineering, Wenzhou University of Technology, Wenzhou 325035, China

² Wenzhou Key Laboratory of Intelligent Lifeline Protection and Emergency Technology for Resilient City, Wenzhou University of Technology, Wenzhou 325035, China

³ College of Design and Art, Wenzhou University of Technology, Wenzhou 325035, China

* Correspondence: liumvsq@126.com

Abstract: Stress waves propagating through multiple parallel fractures are attenuated due to the multiple wave reflections and transmissions at the fractures. The dynamic response of a layered medium under an impact load is systematically studied by a model test in this paper. The effects of the impact energy, number of joints, thickness of the medium and joint layer, material properties of the cementation layer, and other factors of the dynamic stress propagation and attenuation of layered media are analyzed. Studies have shown that Sadovsky's empirical formula fits the attenuation law of vertical peak velocity well, and the obtained attenuation coefficient k and index b can be used for reflecting the dynamic response characteristics of layered media. The attenuation coefficient k is positively correlated with the impact height and negatively correlated with the plate thickness in a single-layer plate impact test. The thickness of the medium layer is the main factor affecting the vibration response of the layered medium in the impact test of the multi-layer slab. The medium thickness, the number of medium layers, and the number of joints have little influence. The different cementation materials and the change of cementation thickness will have a great influence on the dynamic response of layered media, when the cementation layer is filled with joint surfaces for the impact test of multi-layer plates.

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Keywords: layered medium; cementation layer; dynamic stress attenuation; fitting coefficient; parametric analysis

1. Introduction

The research on the dynamic properties of a layered medium has always been an important topic in geotechnical engineering, such as for the safety protection of a military underground structure, mine blasting or excavation blasting, the excavation of a metro tunnel or an underground pipe rack, the design of a roadbed or a pavement structure, or the vibration prediction of the blasting demolition of a structure. The propagation characteristics of stress waves in layered media need to be deeply studied and understood in these fields.

Generally, three methods are available for studies on wave propagation of a layered medium. One is the theoretical study from which the mechanism and the process of wave propagation can be revealed for some special geological cases. The experimental study is the second method, which is sometimes limited by the existing test techniques. The third is numerical modeling, which provides a convenient and economical approach, especially for complicated geological cases.

The most classical theoretical model is the displacement discontinuity model [1,2], which assumes that when waves propagate across a fracture, the stress field is continuous, but the displacement field is discontinuous due to the fracture deformation. On the basis of the displacement discontinuity model and Snell's law, propagation of wave incidence

across a planar linear slip interface was investigated by Schoenberg [3,4]. Many scholars have further extended the displacement discontinuous model. The effects of different joint quantities, joint geometric characteristics, stress wave incident conditions, and rock mass and joint media material deformation characteristics on the propagation characteristics of the stress wave in jointed media were considered. The wave propagation across parallel fractures and the permeation and reflection characteristics of joint surfaces were studied by Pyrak-Nolte [5,6] and Cook [7,8]. Chai S.B. [9] and Zhu [10] studied the wave transmission across jointed rock masses, where multiple intersecting joint sets exist. Goodman R.E. [11], Lu [12], Li [13], and Ju [14] considered the influence of the characteristics of the joint layer, such as joint stiffness, physical and mechanical properties of joint-filled media, joint surface boundary condition, and transfer condition. Pyrak-Nolte [5] and Gu [15,16] discussed the relationship between the transmittance and the reflection coefficient, the waveform conversion phenomenon, and the incident angle and derived the transfer equation of the incident angle. Cai [17] and Zhao [18–20] analyzed the effects of the linear and nonlinear deformation characteristics of the joint layer.

Although the wave propagation across different topography joints, such as single or groups of parallel joints and simple intersecting joints, have been considered in the existing theoretical research. However, it is difficult to accurately analyze and evaluate by theoretical methods in actual engineering, due to the complexity of joint distribution and the inaccuracy of the mechanics and boundary conditions. Therefore, many researchers have done a lot of work through the more intuitive research method of experiment. Generally, the Split Hopkinson Pressure Bar (SHPB) apparatus was adopted to investigate the wave propagation of a layered medium. Ju [14], Wang [21], Lu [22], and Li [23] studied the propagation characteristics of one-dimensional stress waves in layered media through the fractal theory and analyzed the transmittance and reflection properties at the joints and the effects of the fractal dimension and roughness of the joints. Similarly, based on SHPB test equipment, the influence of impact rate, strain rate, joint angle, joint thickness, and other parameters was studied by Li [23], Yang [24], Wei [25], and Challita G. [26]. In addition to the SHPB test system, Tian [27] and Sun [28] carried out some research with independently developed impact test equipment. Based on actual blasting engineering, Peng [29] and Wang [30] analyzed the wave attenuation of a natural layered foundation and improved the empirical formula of stress wave propagation.

For the numerical modeling, the same problem was studied with the many available tools, such as the finite element method (FEM), boundary element method (BEM), finite difference method (FDM), and discrete element method (DEM). In FEM and BEM, joints are modeled using “joint elements”, which simulate the interface discontinuity [31,32]. In FDM, joints are simulated using slide-lines [33]. Compared with the other methods, DEM has been recognized as a better alternative. The layered medium is treated as an assemblage of discrete blocks by fictitious or real joints, of which some are the contact interfaces between the distinct bodies [34]. In recent years, the universal distinct element code (UDEC), which is a 2D DEM-based numerical program, has been widely adopted to study wave propagation across the joints of layered media [35]. Brady [36] performed UDEC modeling on the wave attenuation across a single joint. Zhao et al. [37,38] carried out numerical studies on wave propagation across multiple non-linearly deformable joints with UDEC. Deng [39] performed a study aimed to verify the capability of 3DEC to model wave propagation across joints in 3D space. Zhu [40] developed a newly numerical code, the distinct lattice spring model (DLSM), to study normally incident wave propagation across a joint set.

However, no experimental work has been conducted to systemically study the dynamic stress propagation and attenuation of a layered medium with parallel joints. In this paper, a self-developed concrete-slab-impact testing system is used to study the dynamic properties of the layered medium through the impact test of single-layer slab, multi-layer slab with parallel joints, and multi-layer slab with cementation. Firstly, the attenuation parameters are obtained by fitting the peak particle velocity with the empirical formula.

Subsequently, parametric studies on impact energy, the number of joints, the thickness of the medium and joint layer, and the properties of the cementation layer are conducted. The dependence of wave propagation and attenuation on these parameters are discussed in detail.

2. Model Test Design

2.1. Specimen Dimension Determination

In order to eliminate the influence of the boundary effect, the numerical software LS-DYNA (960, 2001, ANSYS, Pittsburgh, PA, USA) was used to simulate the attenuation of peak particle velocity by modeling the hammer drop test on a three-layer concrete slab.

2.1.1. Calculation Model Establishment

The stress cloud diagram of the concrete slab was recorded to analyze the influence of boundary effect by applying the impact load of a 2.5 kg drop hammer on the surface, as shown in Figure 1. The SOLID164 element was used to build the concrete slab model, and the length of each slab is 4 m, the width is 1 m, and the thickness is 10 cm. The simulation in this paper adopts the projection boundary condition, which can simulate the effect of the wireless range in the finite range model. By applying artificial dampers to the boundary, the wave will be projected when it propagates to the boundary and then spreads to the infinite distance, as if it propagates in the infinite medium. The bottom concrete slab is fixed and restrained at the bottom. Surface-to-surface contact is set between each layer of the slab, and point-to-surface contact is set between the drop hammer and the upper layer of the slab.

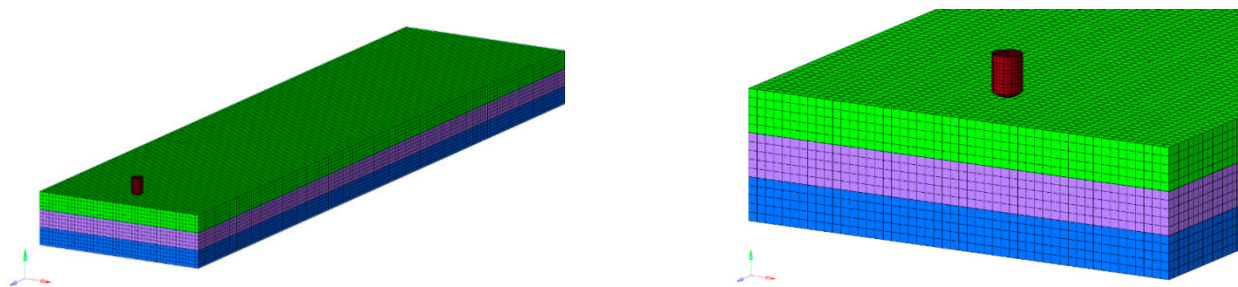


Figure 1. Calculation model of concrete slab impacted by drop hammer.

Surface-to-surface contact is adopted for the contact type between each layer of slabs, and this kind of contact is one of the most commonly used contact types. It is commonly used for contact problems between objects with an arbitrary shape and a large contact area, such as block sliding, cylinder sliding, etc. Point-to-surface contact is adopted for the contact type between the drop hammer and the uppermost concrete slab. A continuous cap model MAT_159 (MAT_CSCM_CONCRETE) was used for the concrete, and a follow-up hardening model MAT_003 (MAT_PLASTIC_KINEMATIC) was used for the drop hammer.

In the impact model of LS-DYNA, the material constitutive model of concrete is usually the continuous cap model MAT_159 (MAT_CSCM_CONCRETE). The model requires fewer parameters and can take into account the strain-rate effect, which is very suitable for media that are sensitive to compressibility. The main parameters required are a density of $2.35 \times 10^{-9} \text{ t/mm}^3$, a compressive strength of 23.5 MPa, and an aggregate size of 10 mm. The material constitutive model of the drop hammer adopts the commonly used nonlinear plastic dynamic hardening model MAT_003 (MAT_PLASTIC_KINEMATIC). This model takes into account the strain-rate effect of the material, and the isotropy and dynamic hardening can be selected by adjusting the hardening parameters between 0–1. The strain rate is considered by the Cowper–Symonds model, and the factors related to the strain rate are used to express the yield stress, which is very suitable for metal materials under an explosive impact load.

The parameters of the concrete and drop hammer materials are shown in Table 1. The transmission boundary is adopted for the boundary condition, and the initial impact velocity of the drop hammer is defined as 4.849 m/s.

Table 1. The parameters of materials.

Concrete Slab			Drop Hammer		
Density (kg/m ³)	Compressive Strength (MPa)	Aggregate Size (m)	Density (kg/m ³)	Elasticity Modulus (GPa)	Yield Strength (MPa)
2.35×10^3	23.5	0.01	8.32×10^3	200	400

2.1.2. Numerical Results Analysis

The direction stress cloud diagram of the concrete slab at different moments in the impact process is shown in Figure 2. The dynamic stress of the concrete slab shows gradual attenuation with the increase in impact distance. The horizontal influence range of the impact load is 2.5 m, and the stress response is very small, outside the range of 2.5 m. In the vertical direction, the dynamic stress shows an obvious jump attenuation across the joints, and a certain stress concentration appears at the interface between the layers. It can be seen that the joints have a significant effect on the dynamic propagation characteristics of the concrete slabs.

Figure 3 shows the vertical and radial peak particle velocity (PPV) values of each measurement point of the concrete surface in the horizontal direction. It can be seen that the attenuation of the vertical PPV is obviously greater than that of the radial PPV. The peak particle velocity tends to be stable when the measure point distance is greater than 2.5 m. Therefore, the final length of the concrete slab is determined to be 2.5 m.

2.2. Model Making

C30 fine aggregate concrete is selected as the concrete for model pouring, and the concrete ratio is water:cement:sand:gravel = 0.38:1.11:2.72, with 42.5 ordinary Portland cement used as the cement. Due to the small size of the model structure, melon seed chips are used as the stones, and the particle size is less than 20 mm.

Three kinds of cementitious materials with different elastic moduli are selected in this test, which are 1 cm EPE pearl cotton, 1 cm XPS extruded board, and 1 cm sand, respectively. The XPS extruded board and EPE pearl cotton are cut and pasted according to the width of the concrete slab and fixed on the concrete slab with adhesive tape to form a multi-layer slab.

Figures 4 and 5 show the process of concrete-slab-mold making, steel binding, and specimen casting. Firstly, the model boxes were made according to the size of the specimen, as shown in Table 2. Then, the release agent was painted on the inner wall of the model boxes, and the steel was tied. The concrete was poured into the model in batches and vibrated fully with a vibrator. Meanwhile, the standard cube concrete test blocks were made and cured under the same condition during the casting processing of the specimens. The average uniaxial compressive strength of the five groups of specimens at 28 days was 23.64 MPa.

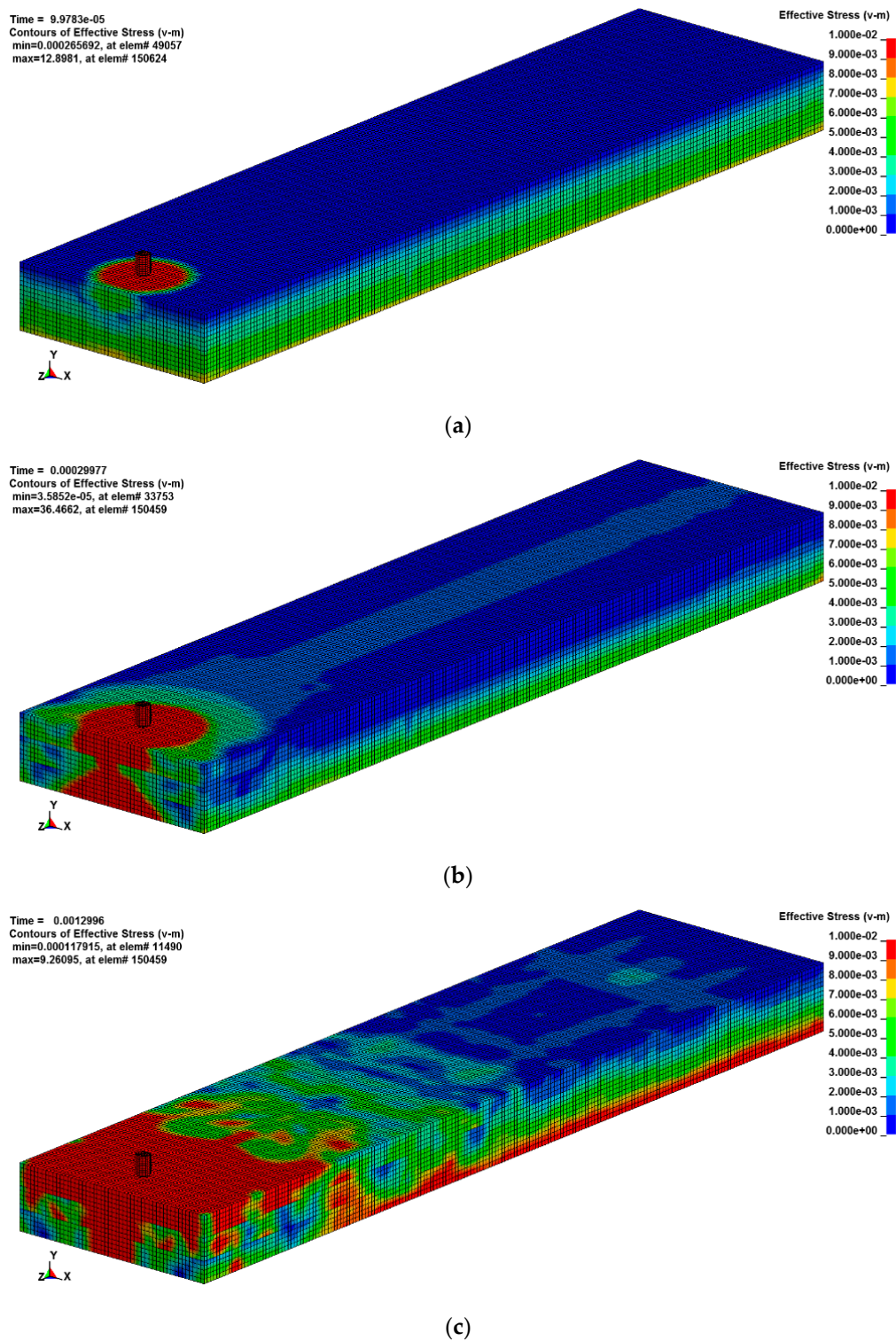


Figure 2. The stress cloud diagram at different moments: (a) $t = 9.983 \times 10^{-2}$ ms; (b) $t = 2.9977 \times 10^{-2}$ ms; (c) $t = 1.2996$ ms.

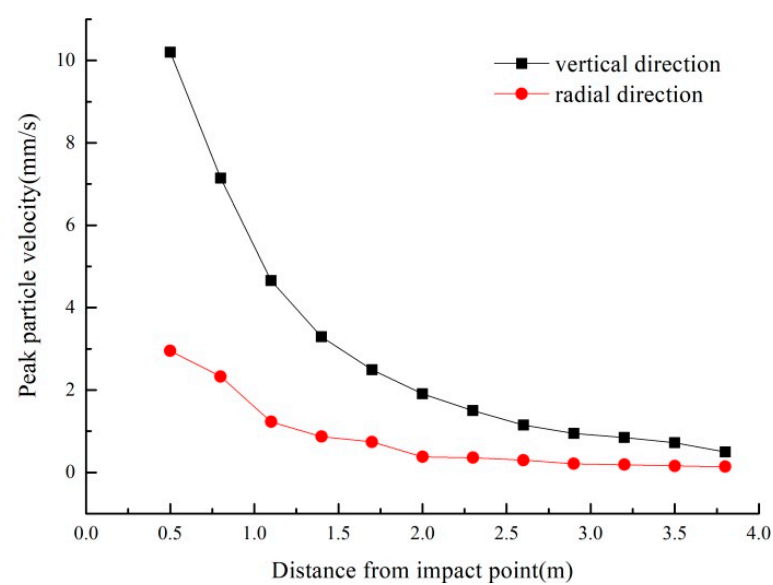


Figure 3. Attenuation of vertical and radial peak particle velocity with distance on concrete surface.



Figure 4. Mold making and steel binding.



Figure 5. Specimens casting.

Table 2. Type and size of concrete slab.

Type	Size (cm × cm × cm)	Number
B1	250 × 100 × 5	4
B2	250 × 100 × 10	4
B3	250 × 100 × 15	1
B4	250 × 100 × 20	1

2.3. Testing Process

This paper uses Canada's Minimate Pro4 vibration and overpressure monitor, which can be used to monitor the vibration velocity. The vibration sensors of the Minimate Pro4 vibration are arranged along the center line of the concrete surface, and the ranging is 0.3 m, 0.6 m, 0.9 m, 1.2 m, 1.5 m, 1.8 m, and 2.1 m, respectively, as shown in Figure 6. The concrete slab is placed on the ground freely without fixed constraint. In addition, the experimental acceleration of the fixed point was monitored. The 2.5 kg drop hammer was raised to the heights of 0.2 m, 0.4 m, 0.6 m, 0.8 m, 1.0 m, and 1.2 m, so that it can fall freely and impact the concrete slab. The JM5938 dynamic signal test and analysis system was used for signal acquisition, and its connection diagram is shown in Figure 7. Each group of tests was repeated three times. The larger discrete data will be eliminated, and the remaining data will be averaged to eliminate the error caused by the experimental contingency.

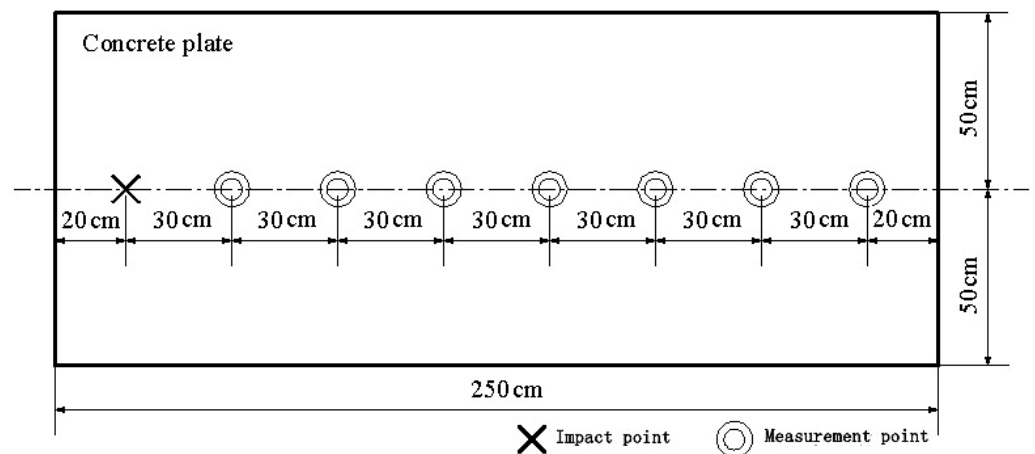


Figure 6. Schematic diagram of measurement point arrangement.

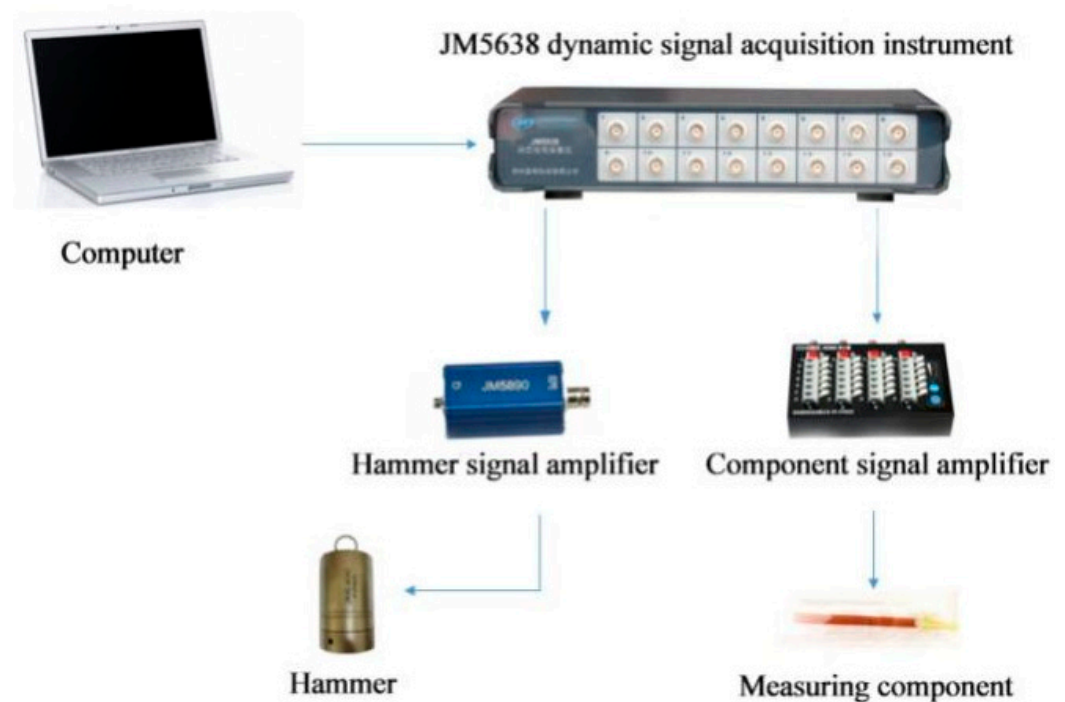


Figure 7. JM5938 dynamic signal test and analysis system.

3. Experimental Results Analysis

By changing the thickness, the number of layers, the types of the cementing layers, the thickness of the cementing layers, the stacking order of the slabs, and the material collocation of the cementing layers, the working conditions of the different layered medium models subjected to the impact load are simulated. The effects of the characteristics of the medium and the characteristics of the interlayer joint on the dynamic mechanical response of the layered medium under impact load were studied, by analyzing the vibration response and changing law of each measuring point under different working conditions.

There are many factors that affect the vibration strength of the medium, so it is difficult to obtain the specific mathematical function expression directly based on the theoretical analysis. Various kinds of empirical prediction formulas for the vibration strength from the experiments were obtained, combining dimensional analyses and empirical assumptions. It is generally recognized that the peak particle velocity shows the power decay with the increase in measuring distance. Sadovsky's empirical formula is widely used in vibration

prediction, and its expression is, where V_{peak} is the peak particle velocity, k is the attenuation coefficient, b is the attenuation index, and Z is the ranging.

3.1. Dynamic Response Analysis of Single-Layer Plate

Figure 8 shows the attenuation law of the vertical peak particle velocity of different thicknesses of concrete slabs. It can be seen that the value of attenuation index b fluctuates less under different impact heights (H), when the thickness of concrete slab is the same, and the value of attenuation coefficient k increases monotonically with the increase in impact height in the same condition. However, the value of the attenuation coefficient decreases with the increase in the thickness of the concrete slab, when the drop height remains unchanged. In order to further analyze the influence of the impact height and the thickness of the slab on the dynamic response, a sensitivity analysis of the attenuation coefficient and its influencing factors is carried out through linear regression, as shown in Figure 9. It is obvious that the attenuation coefficient and the impact height show a significant positive linear correlation under the same slab thickness, while the thickness of the concrete slab has a great influence on its dynamic stress response and attenuation law under the same impact height, and the attenuation coefficient k of the vertical peak particle velocity shows a negative correlation with the thickness of the concrete slab.

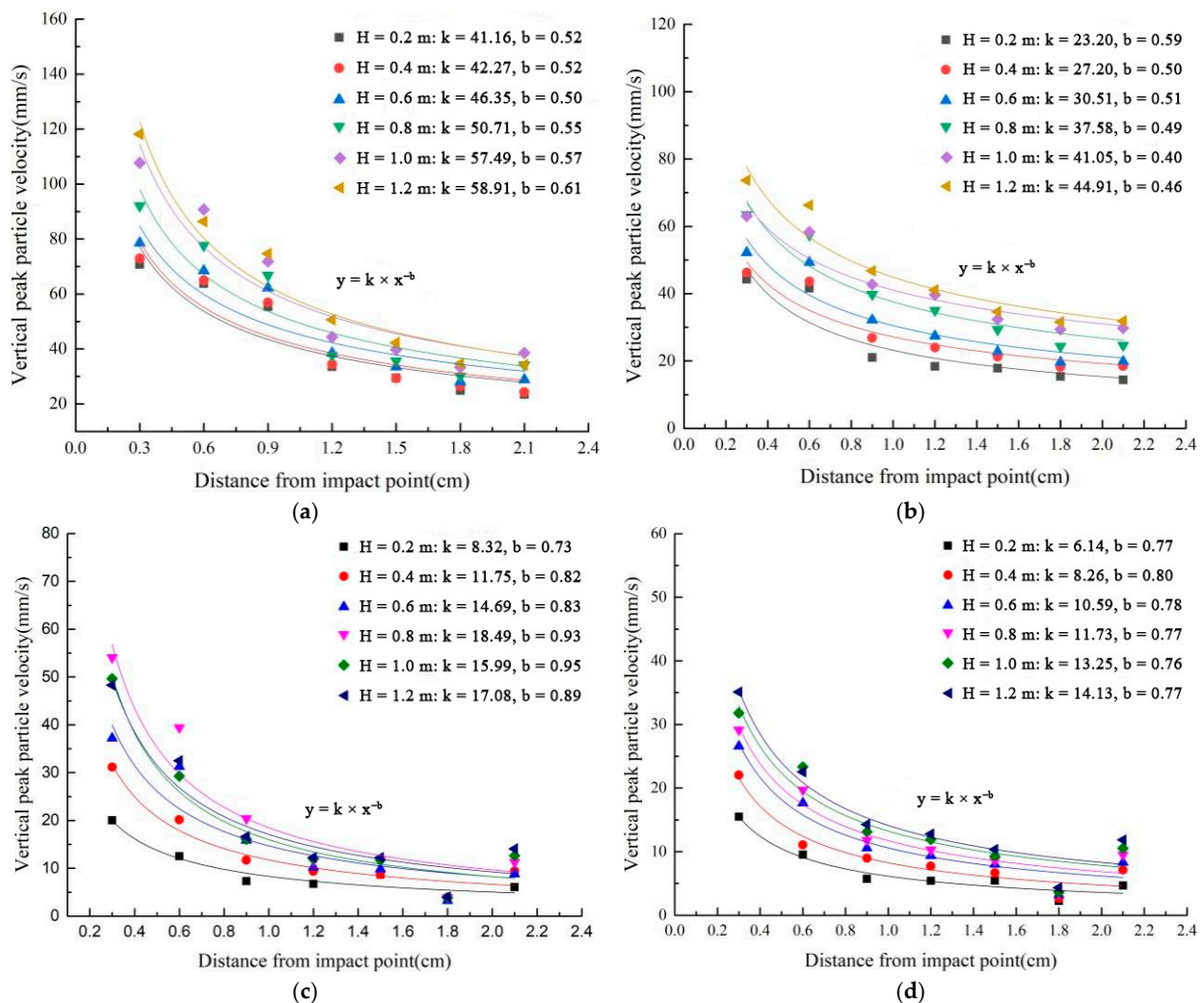


Figure 8. The attenuation law of the vertical peak particle velocity of different thicknesses of concrete slabs: (a) 5 cm; (b) 10 cm; (c) 15 cm; (d) 20 cm.

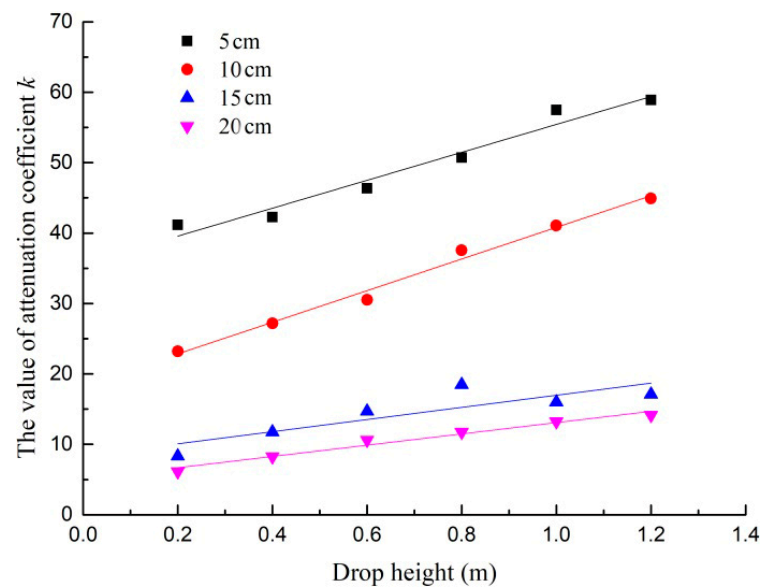


Figure 9. The change of the attenuation coefficient k of the vertical peak particle velocity with impact height under different thicknesses of concrete slabs.

3.2. Dynamic Response Analysis of Multi-Layer Plate

Compared with a single-layer plate, a multi-layer plate is stratified and discontinuous. The wave propagation becomes complicated due to inter-fracture wave reflection and transmission. Related research shows that the propagation of a stress wave in a layered medium is closely related to the properties of the joints, which are manifested in the coupling effect of the media and joint. The effects of the number and thickness of the layered medium (slab and joint) on the dynamic response of the layered medium are discussed under the same drop height.

Figure 10 shows the attenuation law of the vertical peak particle velocity under the same thickness of the layered medium and different numbers of slabs. Figure 11 shows the attenuation law of the vertical peak particle velocity under the same number but different thicknesses of slabs. Figure 12 shows the attenuation law of the vertical peak particle velocity under the same thickness but different numbers of slabs. Table 3 shows the fitting parameters based on Sadovsky's empirical formula. According to the test results, the numbers and thicknesses of the slabs have a significant effect on the dynamic response of a multi-layer plate with the same thickness. The value of the vertical peak particle velocity increases with a smaller slab thickness, and the attenuation law in the horizontal direction is more obvious. In other words, the attenuation parameters k and b increase with the decrease in the thickness of the slab. It is obvious that the experimental results show little difference from the change of the number of layers or the total thickness of the layered medium under the same slab thickness. In other words, the main influence factor of the dynamic response of the multi-layer plate is the layer thickness, so the influence of the number of layers and the total thickness can be neglected.

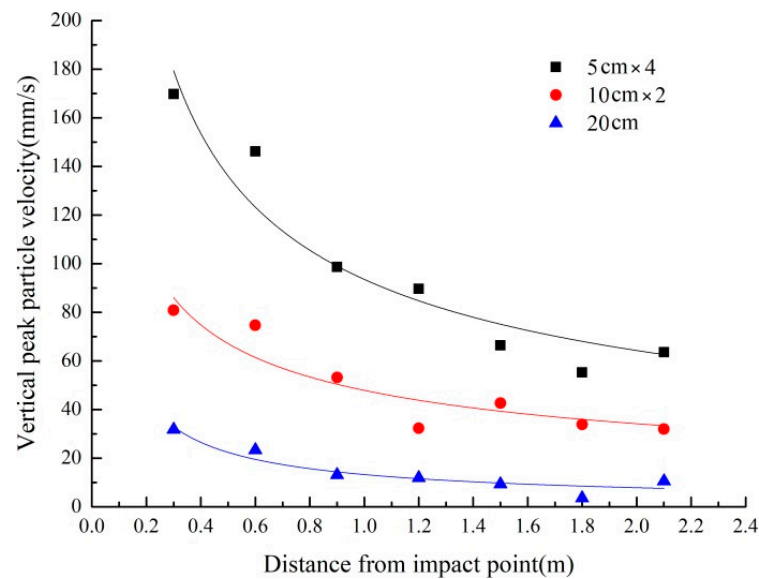


Figure 10. The impact test under the same thickness of layered medium and different number of slabs.

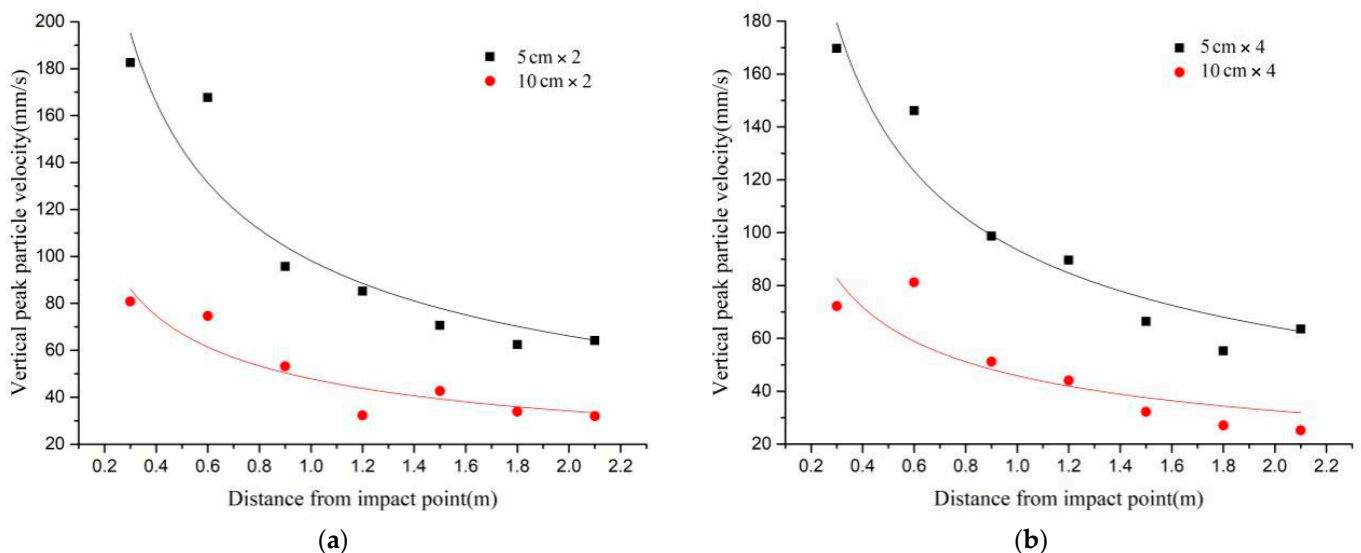


Figure 11. The impact test under the same number but different thicknesses of slabs: (a) 5 cm layer thickness double-layer plate and 10 cm layer thickness double-layer plate; (b) 5 cm layer thickness four-layer plate and 10 cm layer thickness four-layer plate.

3.3. Dynamic Response Analysis of Multi-Layer Plate Considering Cementing Joints

In the above test, the joint can be regarded as a two-dimensional plane without any thickness. However, the joint is generally presented as an interlayer, such as a clay thin interlayer, in the actual engineering. Not only the wave propagation across the joints but also the properties of the cementing joints, such as wave resistance, elastic modulus, interface roughness, contact area, cementing joint thickness, etc., need to be considered. So, the influence of the different filling materials and thicknesses of cementing joints on the dynamic response of the multi-layer plate are studied.

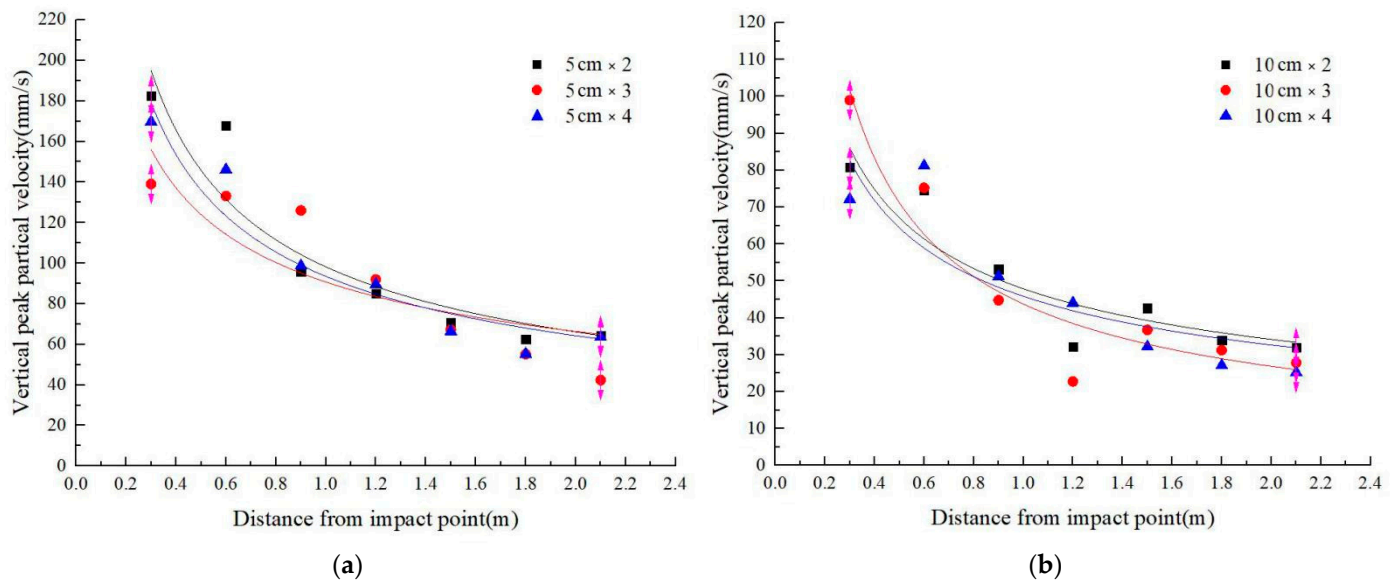


Figure 12. The impact test under the same thickness but different numbers of slabs: (a) 5 cm layer thickness multi-layer plate; (b) 10 cm layer thickness multi-layer plate.

Table 3. The fitting parameters of the vertical peak particle velocity attenuation curve under different test conditions.

Parameters	Single-Layer Plate			Double-Layer Plate		Triple-Layer Plate		Four-Layer Plate	
	10 cm	15 cm	20 cm	5 cm × 2	10 cm × 2	5 cm × 3	10 cm × 3	5 cm × 4	10 cm × 4
<i>k</i>	41.1	16.0	13.2	98.2	47.9	90.7	43.7	93.5	45.8
<i>b</i>	0.40	0.95	0.76	0.57	0.49	0.45	0.70	0.54	0.49

The filling materials of EPE, XPS, and sand are shown in Figure 13. Figure 14 gives the impact test results of the multi-layer plate, considering a cementing joint with different filling materials, as shown in Figure 13, their thicknesses are all 1 cm. Due to the diversity of the reflection and the transmission of the material, the value of the vertical peak particle velocity and its attenuation law are obviously different. When filled with sand and EPE, the attenuation law of the medium is close, but the value of the vertical peak particle velocity of the sand-filled multi-layer plate is larger. The postponed tendency is no longer apparent when XPS is selected as the filling material, and the value of the attenuation index is only 0.12, though the average value of the vertical peak particle velocity is the largest.

The effect of the cementing joint thickness on the dynamic response of the layered medium was investigated by the double-layer plate impact test. XPS was selected as the filling material of the cementing joint, and the thickness of XPS was set as 1 cm, 2 cm, and 3 cm. As shown in Figure 15, the dynamic response of the double-layer plate varies greatly with the change of the XPS slab's thickness. When the thickness of the XPS plate is 2 cm, the attenuation law of the vertical peak particle velocity in the horizontal direction is the most obvious, and the curve fitting degree is the highest. The fitting attenuation's indexes are only 0.12 and 0.29 when the thickness of the XPS plate is 1 cm and 3 cm, respectively. Apparently, the thickness of the cementation layer is the main factor affecting the dynamic response of the layered medium, but there is no regularity between the thickness and the attenuation parameters.

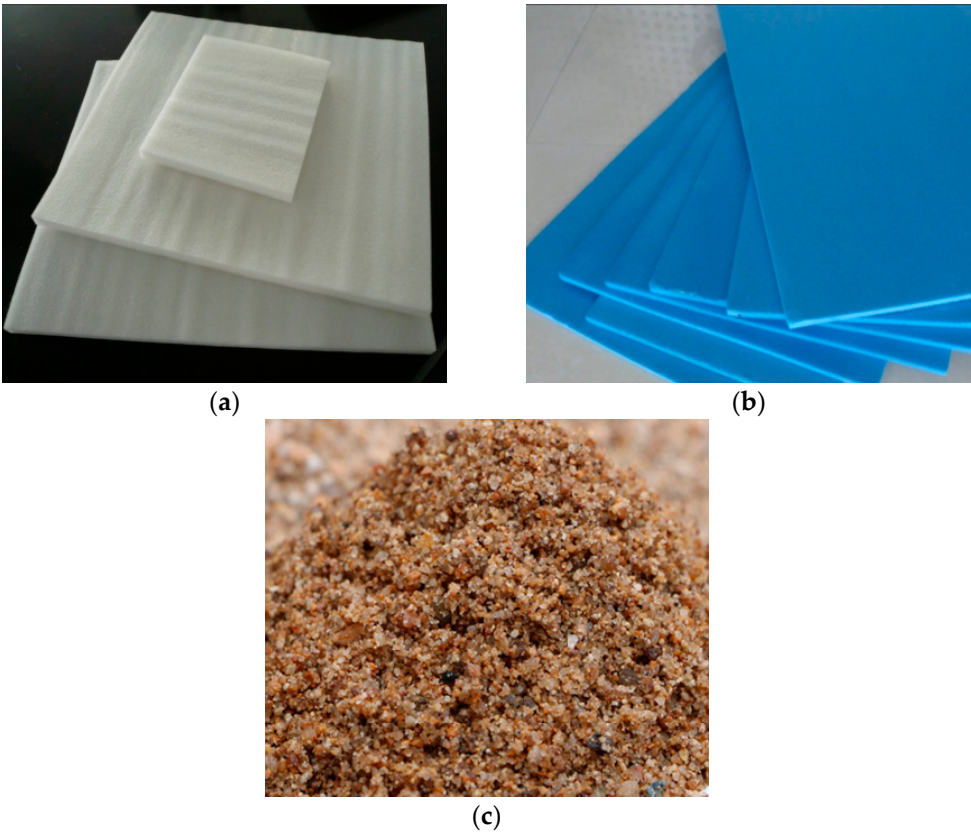


Figure 13. Different cementing joint filling materials: (a) EPE; (b) XPS; (c) sand.

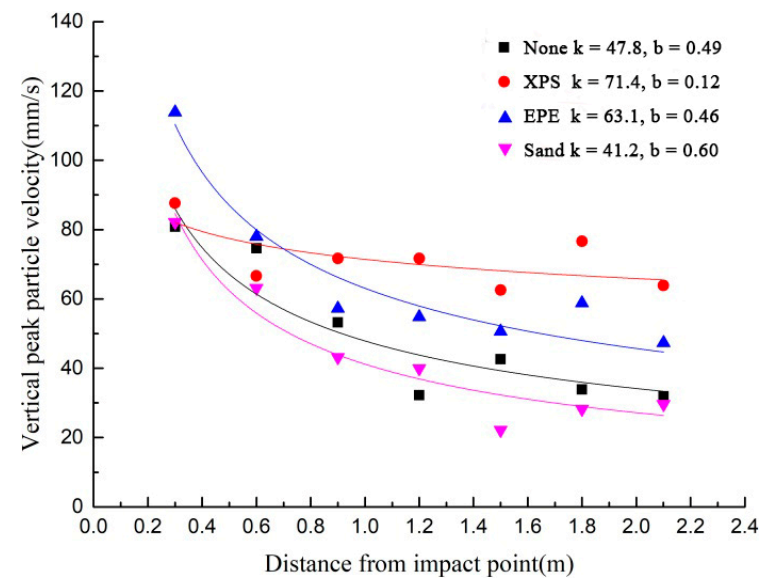


Figure 14. Dynamic response of double-layer plate with different filling materials.

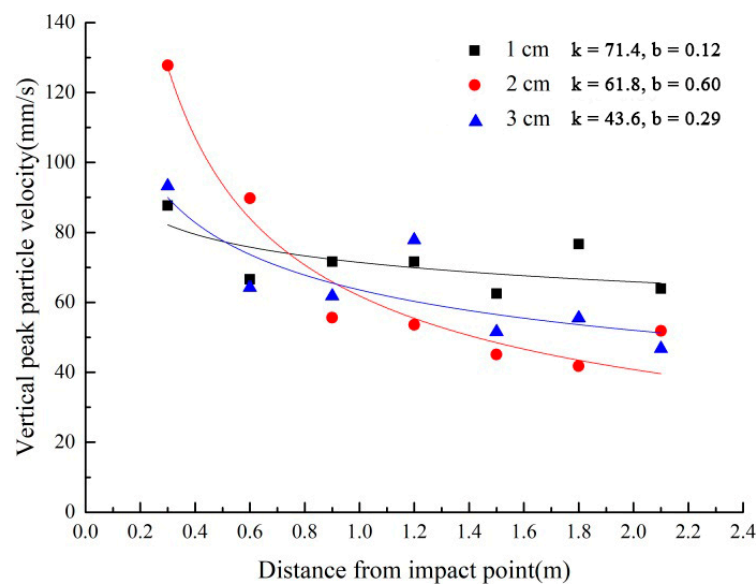


Figure 15. The dynamic response of double-layer plate with different cementing joint thicknesses.

4. Discussion and Conclusions

The influence factors of the dynamic response and attenuation law of the layered medium, such as impact energy, number and thickness of layers and joints, cementing joints materials, etc., are systematically investigated by impact tests of the single-layer plate and the multi-layer plate. The following conclusions are drawn:

- (1) Sadovsky's empirical formula can fit the attenuation law of the vertical peak particle velocity well, with the ranging in the horizontal direction, and the fitting attenuation's coefficient k and index b can directly reflect the characteristics of the wave attenuation. The attenuation coefficient k has a linear positive correlation with the impact height under the same plate thickness, while it is negatively correlated with the thickness of the concrete plate under the same impact height.
- (2) The thickness of the layer is the main factor affecting the dynamic response of the layered medium, so the number of plates and joints and the total thickness of the medium have little influence. The value of the vertical peak particle velocity is larger, and the attenuation law is more obvious, with an increase in the number of layers and a decrease in the thickness of layers under the same total thickness of the layered medium.
- (3) The dynamic response of the multi-layer plate, when considering the cementing joints filled with different materials, varies significantly, and the thickness of the cementing joint has a great influence on the wave propagation of the layered medium. However, the effects of the material and thickness are only presented apparently, so the internal connection and the mechanism between the media and the joint need to be discussed deeply. The properties of the cementing joints and their effects need to be conducted systematically in subsequent studies.

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Article

General Curve Model for Evaluating Mechanical Properties of Concrete at Different Ages

Qiuwei Yang ^{1,2}, Xinhao Wang ^{1,2}, Xi Peng ^{1,2} and Fengjiang Qin ^{3,4,*}

¹ School of Civil and Transportation Engineering, Ningbo University of Technology, Ningbo 315211, China; yangqiuwei@nbut.edu.cn (Q.Y.); wangxinhao0709@163.com (X.W.); pengxi@nbut.edu.cn (X.P.)

² Engineering Research Center of Industrial Construction in Civil Engineering of Zhejiang, Ningbo University of Technology, Ningbo 315211, China

³ Key Laboratory of New Technology for Construction of Cities in Mountain Area, Ministry of Education, Chongqing 400045, China

⁴ School of Civil Engineering, Chongqing University, Chongqing 400045, China

* Correspondence: qinfengjiang@cqu.edu.cn

Abstract: During the process of pouring and solidification of concrete, the compressive strength and elastic modulus of concrete exhibit dynamic growth patterns. The mechanical properties of concrete usually remain stable in the later stage (28 days after pouring). Studying appropriate curve models to accurately evaluate the changes in early mechanical properties of concrete has always been an important topic in the field of concrete materials. This work proposes a new dual parameter curve model for accurately evaluating the growth pattern of early compressive strength and elastic modulus of concrete. A comparative study was conducted between the proposed new curve model and existing curve models using 18 sets of experimental data from 10 literature sources. The research results indicate that the fitting average error and standard deviation of this new curve model are significantly smaller than the existing curve models. For some examples, the fitting error and standard deviation of the new model are only about 20%–30% of those of the existing models. The main advantages of this new curve model lie in two aspects. The first advantage is that this new curve model only contains two unknown parameters, so only a small amount of experimental data is required for data fitting and does not require complex mathematical operations. The second advantage is that this new curve model has a wide range of applications, which include compressive strength evaluation and elastic modulus evaluation and can also be extended to the evaluation of the mechanical properties of other materials similar to concrete.

Keywords: concrete; elastic modulus; compressive strength; two-parameter curve model; three-parameter curve model

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1. Introduction

Concrete is a widely used building material for residential buildings, factories, and bridges. The mechanical properties of concrete, such as compressive strength and elastic modulus, exhibit dynamic growth patterns after pouring and remain stable until 28 days later. Evaluating the early mechanical properties of concrete can effectively guide the early construction and design of concrete engineering [1–4], such as prestressed tensioning or formwork removal. For example, in the production of large-volume concrete for bridge box girders, steel reinforcement prestressing should be applied as early as possible to prevent concrete cracks. Therefore, it is necessary to accurately evaluate the compressive strength and elastic modulus of concrete at an early stage to determine the time for steel reinforcement tensioning [5]. In high-rise buildings and other projects that use a large amount of concrete, the cost of formwork engineering is very high. If the formwork can be removed as early as possible, it can accelerate the turnover of formwork and thus save construction costs. Thus, accurately evaluating the strength of concrete at early ages is also

very important for determining the time of formwork removal [6]. At present, there are two main methods for evaluating the early mechanical properties of concrete, namely machine learning-based evaluation methods and mathematical curve-based evaluation methods. Machine learning-based evaluation methods typically utilize large amounts of experimental data to train artificial neural network (ANN) models. The trained ANN models can then be used to evaluate the mechanical properties of concrete at various ages. Jaf et al. [7] used machine learning technology to evaluate the effects of silicon dioxide and calcium oxide in fly ash on the compressive strength of green concrete. Ahmed et al. [8] employed innovative machine learning techniques to predict the compressive strength of geopolymer concrete modified with nanoparticles. Trtnik et al. [9] analyzed the relationship between ultrasonic pulse velocity, static and dynamic Young's modulus, and shear modulus. Based on the experimental results, they demonstrated that the model constructed via artificial neural networks can estimate the compressive strength of concrete using only ultrasonic pulse velocity values and some mix ratio parameters of concrete. Yousif and Abdullah [10] found that water–cement ratio is the most important factor affecting the output of neural network models, and neural networks have strong potential as a tool for predicting concrete compressive strength. Duan et al. [11] constructed, trained, and tested a neural network model for predicting the compressive strength of recycled aggregate concrete using 146 sets of available data obtained from 16 different published literature sources. Nikoo et al. [12] used an artificial neural network model to calculate the compressive strength of concrete and optimized it using genetic algorithms. Khashman and Akpinar [13] designed an artificial neural network model for predicting the compressive strength of different concrete mixtures. Deng et al. [14] proposed a prediction model based on deep learning theory, which learns the deep features of concrete components via convolutional neural networks. The research results indicate that the prediction model based on deep learning has the advantages of high accuracy, high efficiency, and strong generalization ability and can be used as a new method for calculating the strength of recycled concrete. Feng et al. [15] proposed an intelligent prediction method for concrete compressive strength based on machine learning technology. Use adaptive boosting algorithm to predict the compressive strength of concrete. Muliauwan et al. [16] used three artificial intelligence methods: ANN, support vector machines (SVM), and linear regression (LR) to predict the compressive strength of concrete. Asteris and Mokos [17] studied the application of artificial neural networks in predicting the compressive strength of concrete in existing structures. Bka et al. [18] constructed three different neural network models using the Levenberg Marquardt (LM) algorithm to predict the compressive strength of recycled aggregate concrete. The results indicate that the prediction model for compressive strength of recycled aggregate concrete based on artificial neural networks has high accuracy and can be further used for mix ratio optimization design. Cui et al. [19] used seven machine learning algorithms to apply machine learning to predict the compressive strength of concrete. The experimental results indicate that the model has high prediction accuracy when the compressive strength is greater than 40 MPa. Kumar et al. [20] used machine learning algorithms such as Gaussian progressive regression (GPR), support vector machine regression (SVMR), ensemble learning (EL), and optimized GPR, SVMR, and EL to predict the compressive strength of lightweight concrete (LWC). Pakzad et al. [21] used machine learning (ML) and deep learning (DL) algorithms to predict the compressive strength (CS) of steel fiber-reinforced concrete (SFRC) containing hooked industrial steel fibers (ISF). Cünyet et al. [22] studied the performance of adaptive neural fuzzy inference system (ANFIS) in predicting the elastic modulus of ordinary concrete and high-strength concrete. The results indicate that their proposed ANFIS outperforms other models in terms of predictive ability. Demir [23] used collected data to establish, train, and test artificial neural network models and compared the predicted elastic modulus results with building codes and practical experience results. The results indicate that artificial neural networks have strong potential in predicting the elastic modulus of high-strength concrete and ordinary concrete. Ahmadi [24] further validated the method for predicting elastic modulus based on the

ANFIS model. The experimental comparison shows that the ANFIS model is superior to the other three nonlinear regression models. Han et al. [25] proposed a comprehensive machine learning (ML) model for predicting the elastic modulus of concrete prepared from recycled concrete aggregates. They compared the predictive performance of the integrated ML model with five commonly used ML models. The results indicate that compared to independent models, integrated ML models always produce more accurate predictions. Hasanzadeh et al. [26] utilized machine learning to predict the mechanical properties of basalt fiber-reinforced high-performance concrete by simulating the compressive strength and elastic modulus of concrete.

The second method is to evaluate the relationship between the mechanical properties of concrete and its age based on a certain function curve. The basic principle of this method is to obtain the functional equation between concrete mechanical parameters and age via data fitting based on early experimental data, and then calculate the concrete mechanical parameters at any time. The main advantage of this method is that it can be carried out with only a small amount of experimental data. At present, the commonly used concrete compressive strength–age curve models are the exponential model [27–29], the logarithmic model [30,31], the hyperbolic model [32,33], and the polynomial model [33,34]. Yang et al. [27] tested the compressive strength and elastic modulus of high-strength concrete within 28 days via experiments and proposed an exponential model for data fitting. Zhao et al. [28] measured the compressive strength and elastic modulus of C50 concrete at 1, 2, 3, 5, 7, 11, 14, 28, and 40 days and fitted the exponential curves of compressive strength and elastic modulus with these data. Yang et al. [29] combined regression fitting methods with exponential models to establish a prediction model for the relationship between the compressive strength and age of C60 concrete. Liu [30] analyzed the changes in compressive strength and elastic modulus of C50 concrete over time via experiments. Zhao [31] measured the compressive strength of concrete at 3, 7, 28, and 60 days via experiments and fitted the corresponding logarithmic function formula via data regression. Carino [32] and Viviani [33] fitted the changes in compressive strength of concrete over time using hyperbolic models, thereby establishing a prediction of compressive strength over age of concrete. Jin [34] used a polynomial model to fit the test data of early-age compressive strength and elastic modulus of high-strength concrete. Ling [35] identified the components that have an impact on the compressive strength of concrete by analyzing the composition of the concrete and built a prediction model for the compressive strength of mechanical sand concrete using a third-order nonlinear function. Jia et al. [36] obtained the compressive strength of ordinary Portland concrete at 1 to 28 days via concrete compressive strength experiments and fitted the obtained data using a cubic polynomial model to obtain a fitting curve of concrete compressive strength over time. Yeh [37] proposed a parametric trend regression and a four-parameter optimization method for predicting the compressive strength of concrete. Elaty [38] analyzed experimental data and revealed that a model can be constructed using two constants to predict the compressive strength of concrete at any age. Guo et al. [39] developed a time-dependent relationship model between concrete compressive strength and elastic modulus by referring to the existing curve models. Wang and Yue [40] analyzed the existing time-varying models of compressive strength, summarized the advantages and disadvantages of these models, and proposed further research directions.

Generally, although existing prediction models can predict the mechanical properties of concrete to a certain extent, they also have certain shortcomings in some aspects. The limitations of the ANN-based evaluation models lie in two aspects. Firstly, the accuracy of the ANN model is greatly affected by the number of neurons. However, there is currently no rigorous mathematical basis for determining the optimal number of neurons in the ANN model. Secondly, the ANN model can only be used for evaluation after being trained and matured. Usually, the training of the ANN model requires a lot of experimental data, which increases the cost of the experiment. The limitations of current curve regression models mainly lie in their low fitting accuracy and weak universality. For example, polynomial

models are inaccurate in predicting the mechanical properties of concrete in the later stage. Although logarithmic and exponential models can converge to a certain extent in the later stage, they cannot predict the mechanical properties of concrete well in the later stage. These types of prediction models do not perform well with data with high dispersion, resulting in low fitting accuracy. The three-parameter model has one more unknown fitting parameter than the two-parameter model, so more experimental data is needed to perform curve fitting accordingly. When there is little experimental data, the fitting accuracy of the three-parameter model may not be as good as that of the two-parameter model. In addition, the three-parameter model is more complex to calculate than the two-parameter model. In order to improve evaluation accuracy, it is necessary to overcome the shortcomings of existing curves and integrate the advantages of existing curves to study a new mathematical curve model for predicting the mechanical properties of concrete. The goal of this work is to develop a new dual parameter curve model that can more accurately evaluate the compressive strength and elastic modulus changes of concrete at early age compared to the existing curve models. Based on the evaluation results of the proposed model, it can more reliably guide the construction or mechanical calculation of concrete components at early age. To this end, this work first analyzes the advantages and disadvantages of the existing logarithmic model, exponential model, hyperbolic model, and polynomial model. Subsequently, this work develops a new curve model that overcomes the shortcomings of existing curve models, which can be used for the evaluation of concrete compressive strength at early ages as well as for the evaluation of elastic modulus at early ages. The improvements provided by the new model mainly lie in three aspects. The first is that the mathematical formula of the new model is concise, including only two unknown parameters, which is very convenient for engineering applications. The second is that the curves used by the new model are more in line with the changes in mechanical properties of concrete at early age, so the fitting accuracy of the new model is higher than that of existing models. Thirdly, the new model has better universality, can be applied to more types of concrete, and may also be applicable to other cementitious materials. This work focuses on the performance of the proposed new model in evaluating the compressive strength and elastic modulus of concrete at early age. Using 18 sets of experimental data from existing literature, a comparative study was conducted on the computational accuracy of the new model and existing models to illustrate the superiority of the proposed new model.

2. Mathematical Models for Predicting the Mechanical Properties of Concrete

As mentioned earlier, the prediction method of concrete mechanical properties based on some assumed mathematical curve is a simple and feasible method in engineering practice, because it only needs a few early test data for concrete. Figure 1 shows the operational process of this type of method in practice.

Obviously, the suitable mathematical curve is an important factor in accurately predicting the mechanical properties of concrete. The commonly used curve models can be divided into two-parameter models and three-parameter models based on the number of unknown parameters that need to be fitted in the curve equations. In this section, the common two-parameter and three-parameter models are reviewed, and the advantages and disadvantages of each model are briefly analyzed. Finally, a new universal two-parameter model is proposed to predict the mechanical properties of concrete at different ages. Without losing generality, in the following mathematical equations of curve models, t represents the time (age) after concrete pouring, and $z(t)$ represents the mechanical parameters (compressive strength or elastic modulus) of the concrete at time t .

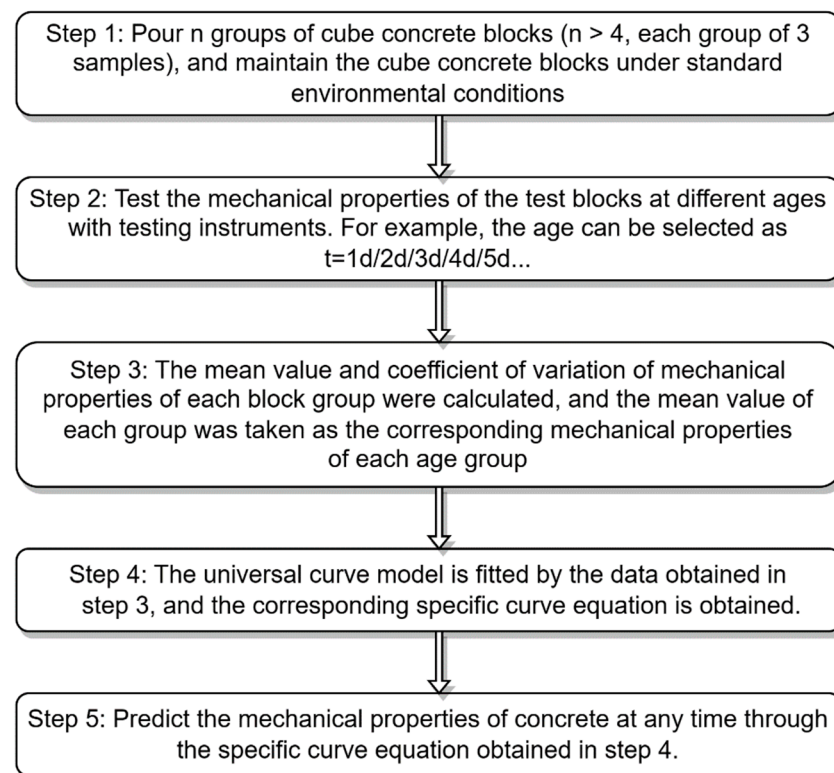


Figure 1. Flow chart for predicting mechanical properties of concrete.

2.1. Two-Parameter Curve Model

The commonly used two-parameter curve models include exponential model, logarithmic model, and hyperbolic model. The curve equation of exponential model is expressed by

$$z(t) = a \cdot e^{-b/t} \quad (1)$$

in which a and b represent two unknown non-negative parameters that need to be determined based on experimental data. As can be seen from Equation (1), the advantage of exponential model is that the exponential function increases monotonically, so with the increase of t , the value of $z(t)$ function will increase. This is consistent with the fact that the mechanical properties of concrete will increase with time in engineering practice. When $t \rightarrow \infty$, $z(t) \rightarrow a$, it is also consistent with the law that the mechanical properties of concrete will remain basically unchanged after a period of growth. However, the exponential model also has some shortcomings. The first drawback of this model is the lack of $t = 0$ point in the definition domain because t is the denominator of $\frac{b}{t}$ in Equation (1). In practical application, when $t = 0$, the mechanical property of concrete is 0. Therefore, it is important to note that the exponential model cannot include this point. In addition, because the equation of exponential model contains only two fitting parameters, the fitting accuracy may be low for experimental data with large dispersion, and it can easily produce large errors.

The curve equation of logarithmic model is expressed as:

$$z(t) = a + b \cdot \ln(t) \quad (2)$$

In Equation (2), a and b represent the two fitting parameters, which are obtained by combining partial experimental data with curve fitting. Comparing Equations (1) and (2), it can be seen that the logarithmic model can be seen as a modified equation obtained by taking the logarithms of both sides of the exponential model equation. Therefore, the advantages and disadvantages of the logarithmic model and the exponential model are

similar. The advantage of the logarithmic model is that the function is a monotonically increasing function, which is consistent with the law of concrete mechanical properties increasing over time in practical engineering. The disadvantage is that its definition domain also lacks the point $t = 0$. Meanwhile, due to its monotonic increasing function, when $t \rightarrow \infty$, although the growth rate slows down, the results do not converge, which is inconsistent with the actual growth law of concrete mechanical properties. Since this equation only contains two fitting parameters, the fitting accuracy of this equation may be low for data with large dispersion.

The curve equation of hyperbolic model is expressed as:

$$z(t) = \frac{t}{a + b \cdot t} \quad (3)$$

In the above formula, a and b are two fitting parameters of the hyperbolic model. The advantage of hyperbolic model compared with other models is that the predicted mechanical properties of concrete will gradually approach a fixed number with the increase of time, which is consistent with the fact that the mechanical properties of concrete remain basically unchanged in the later stage. The disadvantage of hyperbolic model is that the fitting accuracy of the model is not very good when dealing with some very discrete data.

2.2. Three-Parameter Curve Model

The common three-parameter models include polynomial model and hybrid curve model. The curve equation of polynomial model is expressed as

$$z(t) = a + b \cdot t + c \cdot t^2 \quad (4)$$

Equation (4) is a quadratic polynomial model, where a , b and c are the three fitting parameters of this model. Compared to the previous three prediction models, the advantage of the polynomial model is that its definition domain includes point $t = 0$, and the fitted function is also a monotonically increasing function, which is in line with the development laws of concrete mechanical properties in practical engineering. Meanwhile, due to the fact that the prediction model has three fitting parameters, its fitting accuracy is correspondingly more accurate than the logarithmic model, the exponential model, and the hyperbolic model with only two fitting parameters. The disadvantage of the polynomial model is that at $t \rightarrow \infty$, the growth rate of $z(t)$ will be too fast, which is different from the law in engineering practice that the mechanical properties of concrete will increase slowly or even remain basically unchanged in the later stage. The reason for the faster growth rate in the later stage of the polynomial model is the square term of time (i.e., $c \cdot t^2$) in the model. The longer the time, the faster the square term of time increases. Therefore, this prediction model is not suitable for predicting the later mechanical properties of concrete.

The curve equation of the hybrid curve model is expressed as

$$z(t) = at \ln\left(1 + \frac{1}{b + ct}\right) \quad (5)$$

Equation (5) is a mixed curve model, which includes a logarithmic model and a hyperbolic model. a , b and c are the three fitting parameters of this equation. The advantages of this model are that its definition domain includes point $t = 0$, the fitting function is also a monotonically increasing function, and the prediction model will eventually converge to a value, which is more in line with the development law of concrete mechanical properties in practical engineering. Meanwhile, due to the fact that the prediction model has three fitting parameters, its fitting accuracy is correspondingly higher than those of the logarithmic model and the hyperbolic model. However, although the three-parameter model has higher fitting accuracy than the two-parameter model, it is less convenient to use in practical engineering applications due to the need for an additional parameter for data fitting.

2.3. A Universal Two-Parameter Model

Based on the above analysis, a new mathematical model is proposed to accurately describe the changes in mechanical properties of concrete over time in practical engineering. The mathematical expression for the new dual parameter curve model is given as:

$$z(t) = \frac{a \cdot t}{t + 1} + b \cdot \arctan\left(\frac{t}{t + 1}\right) \quad (6)$$

In Equation (6), t and $z(t)$ represent the time and the corresponding mechanical parameters at time t , which are consistent with the previous definitions. The two unknown parameters a and b represent the fitting coefficients that need to be determined based on experimental data. The new model can be seen as a combination of hyperbolic function and arctangent function, and the accuracy of function fitting can be further improved by adding an item of arctangent function. The new model also inherits the advantage that its definition domain contains a point $t = 0$. At the same time, when t approaches infinity, $z(t)$ tends toward a certain value, which is in line with the fact that the mechanical properties of concrete in the later stage remain basically unchanged in practical engineering. This compensates for the defect of polynomial models that cannot converge when t approaches infinity, so this model can be used to predict the mechanical properties of concrete in the later stage. In addition, the prediction model only needs to fit two parameters, which is more convenient to use in practical engineering than the above three-parameter model. The physical meanings of parameters a and b are the weights of the hyperbolic function and the arctangent function in the sum function shown in Equation (6), respectively. From Equation (6), one obtains:

$$\lim_{t \rightarrow \infty} z(t) = a + b \cdot \frac{\pi}{4} \quad (7)$$

Therefore, the physical meaning of $a + b \cdot \frac{\pi}{4}$ is the stable mechanical parameter of concrete in the later stage, such as compressive strength or elastic modulus. Note that this new model only has two fitting parameters, a and b , that need to be determined via data fitting. When the new model is compared with the existing two-parameter models, it can be found that the new model is a combination of two types of curves (hyperbolic and arctangent curves), while the existing two-parameter models are all based on a single curve. The reason why the new model yields fewer errors compared to the existing models is that it introduces a new curve of the arctangent function, which is more in line with the growth law of mechanical properties of concrete at early age.

In summary, Table 1 presents the advantages/disadvantages of the above curve models for clarity. The improvement in fitting accuracy of the new model can be seen in the calculation results of multiple sets of experimental data in the next section.

Table 1. The advantages/disadvantages of various curve models.

Model	Number of Fitting Parameters	Definition Domain	Convergence of $z(t)$ When $t \rightarrow \infty$
Exponential model	2	Missing point with $t = 0$	Yes
Logarithmic model	2	Missing point with $t = 0$	No convergence
Hyperbolic model	2	Including point with $t = 0$	Yes
Polynomial model	3	Including point with $t = 0$	No convergence
Hybrid model	3	Including point with $t = 0$	Yes
New dual parameter model	2	Including point with $t = 0$	Yes

The next section describes the fitting parameter calculation method for the mathematical model described above. Firstly, a linear equation system of mechanical properties and age is established based on the measured data obtained from experiments, and then the corresponding fitting parameters are further calculated. Using n sets of experimental data, the linear system corresponding Equation (6) can be expressed as:

$$y = C \cdot x \quad (8)$$

$$y = \begin{Bmatrix} z(t_1) \\ \vdots \\ z(t_n) \end{Bmatrix}, \quad (9)$$

$$x = \begin{Bmatrix} a \\ b \end{Bmatrix} \quad (10)$$

$$C = \begin{bmatrix} \frac{t_1}{t_1+1} & \arctan(\frac{t_1}{t_1+1}) \\ \vdots & \vdots \\ \frac{t_n}{t_n+1} & \arctan(\frac{t_n}{t_n+1}) \end{bmatrix} \quad (11)$$

where $z(t_n)$ represents the mechanical parameter (elastic modulus or compressive strength) measured at time t_n . Clearly, the vector y and coefficient matrix C in Equation (8) can be obtained from the experimental data. Thus, the unknown fitting parameter vector x can be further calculated using the least square method. For this purpose, Equation (8) can be rewritten as

$$C^T y = (C^T C) \cdot x \quad (12)$$

From Equation (11), the solution of x is expressed as:

$$\hat{x} = (C^T C)^{-1} C^T y \quad (13)$$

The calculated $\hat{x}$ refers to the least squares estimate of x .

The fitted parameters obtained from the calculation can be used to obtain the specific curve equation of the prediction model, and the accuracy of the model prediction can be verified by comparing the difference between the predicted value and the experimental value. The difference between the predicted value and the experimental value is defined as:

$$\{\delta\} = |C \cdot \hat{x} - y| \quad (14)$$

where $\{\delta\} = (\delta_1, \dots, \delta_n)^T$ is called as the residual vector. Specifically, δ_i is the residual error between the i th experimental data and the predicted value. Average and standard deviation of the residual errors can be calculated by:

$$\bar{\delta} = \frac{\delta_1 + \dots + \delta_n}{n} \quad (15)$$

$$\sigma = \sqrt{\frac{\sum_{i=1}^n (\delta_i - \bar{\delta})^2}{n}} \quad (16)$$

where $\bar{\delta}$ is the average of the residual errors, σ is the corresponding standard deviation. The smaller the value of $\bar{\delta}$ and σ , the higher the fitting accuracy of the curve model.

3. Model Verification and Comparison via Concrete Experiment Data

The newly proposed evaluation model was validated using experimental data from references [28,30,34,36,39,41–45] (referred to as cases 1 to 11). The results were compared

with the fitting results of existing logarithmic, hyperbolic and polynomial models to demonstrate the superiority of the model.

3.1. Evaluation of Compressive Strength

Case 1. In reference [36], 32 groups of concrete specimens with a water–cement ratio of 0.56 and 50 mm × 50 mm × 150 mm were cured under standard curing conditions. The components of concrete materials are listed in Table 2. The experimental data for the compressive strength are shown in Table 3. The logarithmic, hyperbolic, polynomial, and new models are used to fit the experimental data, and the fitting curves are shown in Figure 2. The fitting average error and standard deviation of these models are shown in Table 4. The specific fitting equations for the four models are as follows:

$$z(t) = 10.2190 + 21.3746 \ln(t) \quad (17)$$

$$z(t) = \frac{t}{0.0822 + 0.0093 \cdot t} \quad (18)$$

$$z(t) = 17.6264 + 5.1571 \cdot t - 0.1102 \cdot t^2 \quad (19)$$

$$z(t) = 519.5090 \cdot \left(\frac{t}{t+1}\right) - 554.0748 \cdot \arctan\left(\frac{t}{t+1}\right) \quad (20)$$

Table 2. The components of concrete materials in case 1 (kg/m³).

Ordinary Portland Cement	Standard Sand	Water	Water–Cement Ratio
460	800	180	0.56

Table 3. Compressive strength of concrete blocks in case 1 (MPa).

Age (Unit: day)	Compressive Strength	Age (Unit: day)	Compressive Strength
1	4.26	15	70.56
2	23.83	16	71.31
3	36.42	17	68.29
4	43.28	18	66.04
5	45.85	19	73.79
6	44.91	20	70.81
7	54.21	21	76.95
8	57.99	22	76.65
9	58.75	23	68.26
10	62.26	24	78.46
11	60.33	25	75.66
12	65.72	26	79.92
13	68.72	27	79.73
14	68.90	28	85.39

According to the average error and standard deviation of the four models in Table 4, it can be seen that the average error and standard deviation of the new model are significantly smaller than those of the hyperbolic, polynomial, and logarithmic models. Figure 2 shows that compared to the logarithmic, polynomial, and hyperbolic models, the prediction curve of the new model is closer to the experimental test data. The results indicate that the new model has better data fitting ability than the logarithmic, polynomial, and hyperbolic models and is more accurate and reliable in evaluating the compressive strength of concrete.

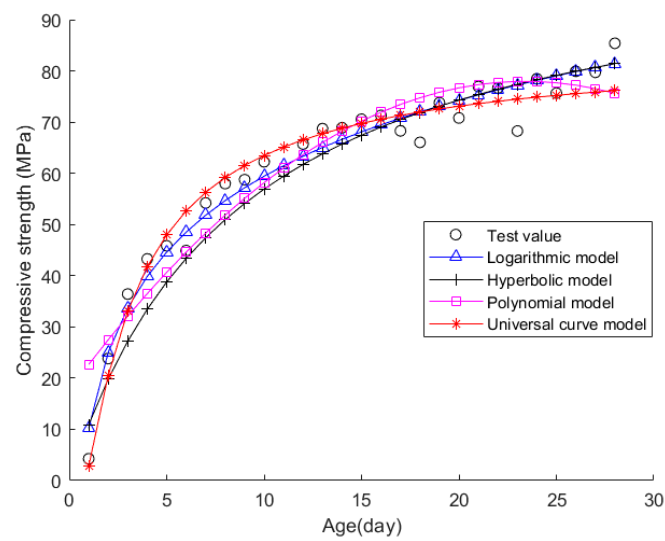


Figure 2. The fitting results of case 1.

Table 4. The fitting mean errors and standard deviations from the four models in case 1.

Index	Logarithmic Model	Hyperbolic Model	Polynomial Model	Universal Curve Model
Fitting mean error $\bar{\delta}$	1.4240	1.6451	4.3428	0.4712
Fitted standard deviation σ	0.2581	0.2895	0.8066	0.0669

Case 2. In reference [41], 14 groups of concrete specimens with 150 mm × 150 mm × 300 mm were manufactured and cured under standard and natural conditions. The components of concrete materials are listed in Table 5. The experimental data of compressive strength are shown in Table 6. Figures 3 and 4 illustrate the fitting curves for the four models. The fitting average errors and standard deviations for these models are shown in Tables 7 and 8. The specific fitting equations for the four models are as follows:

Table 5. The components of concrete materials in case 2 (kg/m³).

Cement	Fine Aggregate	Coarse Aggregate	Water	Water Reducer	Admixture
357	712	1115	123	8.82	63

Table 6. Compressive strength of concrete blocks in case 2 (MPa).

Age (Unit: hour)	Natural Conservation Condition	Standard Maintenance Condition
12	4.9	2.7
16	14.5	10.7
20	19.0	16.0
24	22.9	19.1
28	24.9	22.4
32	27.2	25.3
36	28.7	26.1
40	29.6	27.6
44	30.3	30.0
48	30.7	30.4
52	31.2	30.6
56	32.9	31.6
60	33.7	33.2
64	34.0	32.3

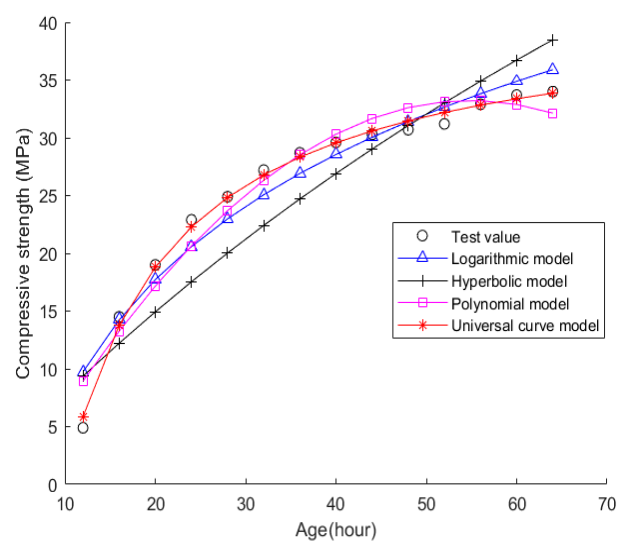


Figure 3. The fitting results for natural conservation conditions in case 2.

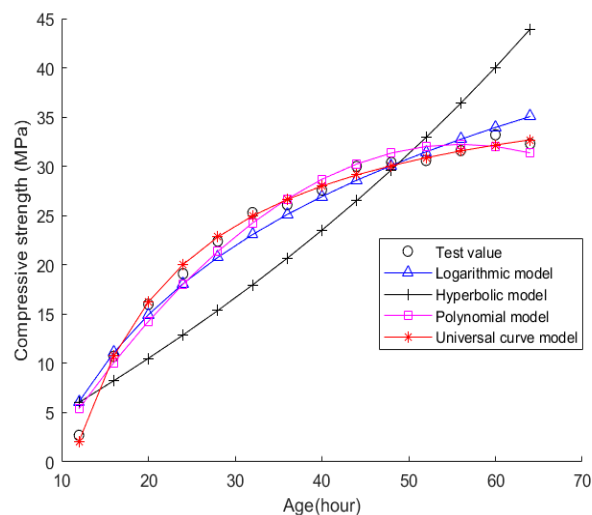


Figure 4. The fitting results for standard curing conditions in case 2.

Table 7. The fitting mean errors and standard deviations of the four models for natural conservation condition in case 2.

Index	Logarithmic Model	Hyperbolic Model	Polynomial Model	Universal Curve Model
Fitting mean error $\bar{\delta}$	1.0084	1.0436	0.8495	0.2083
Fitted standard deviation σ	0.2524	0.2214	0.2087	0.0497

Table 8. The fitting mean errors and standard deviations of the four models for standard maintenance condition in case 2.

Index	Logarithmic Model	Hyperbolic Model	Polynomial Model	Universal Curve Model
Fitting mean error $\bar{\delta}$	1.2761	1.5086	1.0147	0.2415
Fitted standard deviation σ	0.3257	0.2926	0.2571	0.0578

Natural conservation conditions:

$$z(t) = -29.0860 + 15.6257 \ln(t) \quad (21)$$

$$z(t) = \frac{t}{1.1927 + 0.0073 \cdot t} \quad (22)$$

$$z(t) = -6.5801 + 1.4516 \cdot t - 0.0132 \cdot t^2 \quad (23)$$

$$z(t) = 1283.1235 \cdot \left(\frac{t}{t+1}\right) - 1581.0830 \cdot \arctan\left(\frac{t}{t+1}\right) \quad (24)$$

Standard maintenance conditions:

$$z(t) = -36.8725 + 17.2964 \ln(t) \quad (25)$$

$$z(t) = \frac{t}{2.1067 - 0.0101 \cdot t} \quad (26)$$

$$z(t) = -11.2280 + 1.5518 \cdot t - 0.0138 \cdot t^2 \quad (27)$$

$$z(t) = 1411.1477 \cdot \left(\frac{t}{t+1}\right) - 1744.6763 \cdot \arctan\left(\frac{t}{t+1}\right) \quad (28)$$

Tables 7 and 8 show that the new model outperforms the hyperbolic, logarithmic, and polynomial models regarding fitting mean error and standard deviation. Figures 3 and 4 visually demonstrate that the fitting curve of the new model aligns more closely with the experimental data. These findings indicate that the new model is a better fit for the test data than the logarithmic, hyperbolic, and polynomial models.

Case 3. In reference [28], 30 sets of concrete specimens were manufactured using C50 grade concrete and cured under standard curing conditions. The components of the concrete are listed in Table 9. The tested compressive strength is shown in Table 10. The four fitting curves are presented in Figure 5 using these experimental data. The fitting average errors and standard deviations are shown in Table 11. The specific fitting equations for the four models are as follows:

$$z(t) = 27.2818 + 8.9263 \ln(t) \quad (29)$$

$$z(t) = \frac{t}{0.0332 + 0.0165 \cdot t} \quad (30)$$

$$z(t) = 30.1059 + 2.0246 \cdot t - 0.0340 \cdot t^2 \quad (31)$$

$$z(t) = 122.6500 \cdot \left(\frac{t}{t+1}\right) - 82.8490 \cdot \arctan\left(\frac{t}{t+1}\right) \quad (32)$$

Table 9. The components of concrete materials in case 3 (kg/m³).

Cement	Fine Aggregate	Coarse Aggregate	Water	Water Reducer	Admixture
400	382	1166	170	4.3	115

Table 10. Compressive strength of concrete specimens in case 3 (MPa).

Age (Unit: day)	Compressive Strength
1	24.0
2	32.1
3	39.6
5	45.7
7	46.8
11	47.6
14	49.9
28	56.8
40	58.4

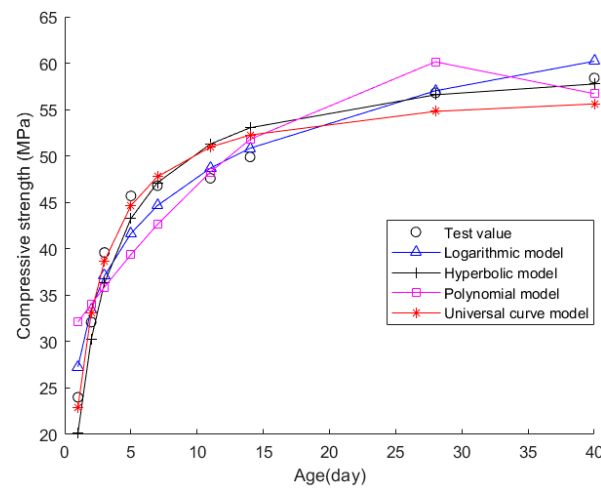


Figure 5. The fitting results of case 3.

Table 11. The fitting mean errors and standard deviations of case 3.

Index	Logarithmic Model	Hyperbolic Model	Polynomial Model	Universal Curve Model
Fitting mean error $\bar{\delta}$	0.1907	0.2214	0.3991	0.1231
Fitted standard deviation σ	0.0410	0.0492	0.0984	0.0163

Table 11 indicates that the new model has a smaller fitting mean error and standard deviation compared to the other models. The new model's average fitting error is approximately 35% lower than that of the logarithmic model, about 44% lower than that of the hyperbolic model, and about 35% lower than that of the polynomial model. The fitting standard error of the new model is roughly 40% for the logarithmic model, about 33% for the hyperbolic model, and about 16% for the polynomial model.

Case 4. In reference [42], 36 sets of 100 mm × 100 mm × 100 mm high-strength concrete specimens were manufactured and cured under standard curing conditions. The components of concrete materials are listed in Table 12. The experimental data of compressive strength are shown in Table 13, and the fitting curves are shown in Figure 6. The fitting average errors and standard deviations are shown in Table 14. The specific fitting equations for the four models are as follows:

$$z(t) = 71.7838 + 25.4840 \ln(t) \quad (33)$$

$$z(t) = \frac{t}{0.0061 + 0.0075 \cdot t} \quad (34)$$

$$z(t) = 48.6435 + 21.4047 \cdot t - 1.4696 \cdot t^2 \quad (35)$$

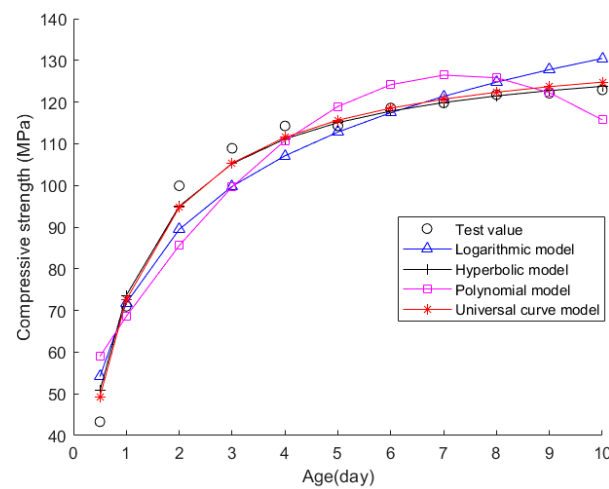
$$z(t) = 81.4696 \cdot \left(\frac{t}{t+1}\right) + 68.7367 \cdot \arctan\left(\frac{t}{t+1}\right) \quad (36)$$

Table 12. The components of concrete materials in case 4 (kg/m³).

Powder	Steel Fiber	Water
2130	160	196

Table 13. Compressive strength of concrete cubes in case 4 (Mpa).

Age (Unit: day)	Compressive Strength
0.5	43.303
1	71.012
2	99.915
3	108.909
4	114.274
5	114.300
6	118.566
7	119.790
8	121.774
9	122.116
10	122.919

**Figure 6.** The fitting results of case 4.**Table 14.** The fitting mean errors and standard deviations of case 4.

Index	Logarithmic Model	Hyperbolic Model	Polynomial Model	Universal Curve Model
Fitting mean error $\bar{\delta}$	0.3023	0.1885	0.4164	0.1564
Fitted standard deviation σ	0.0703	0.0495	0.1001	0.0388

Table 14 shows that the fitting mean error and standard deviation of the new model are significantly smaller than those of the hyperbolic, logarithmic, and polynomial models. This again indicates that the new model can more accurately evaluate the compressive strength of concrete.

Case 5. In reference [30], some concrete specimens with 150 mm × 150 mm × 300 mm were manufactured, and the experimental data of compressive strength are shown in Table 15. The fitting curves for the four models are shown in Figure 7, and the fitting average errors and standard deviations are shown in Table 16. The specific fitting equations for the four models are as follows:

$$z(t) = 33.3919 + 9.2854 \ln(t) \quad (37)$$

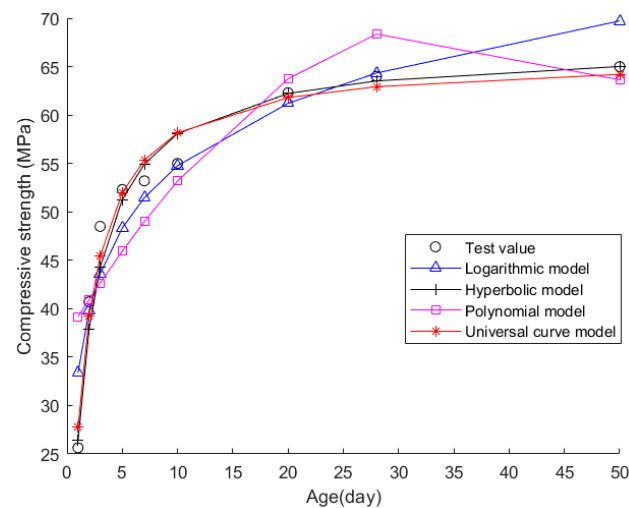
$$z(t) = \frac{t}{0.0229 + 0.0149 \cdot t} \quad (38)$$

$$z(t) = 37.3494 + 1.8490 \cdot t - 0.0265 \cdot t^2 \quad (39)$$

$$z(t) = 123.1825 \cdot \left(\frac{t}{t+1}\right) - 72.9417 \cdot \arctan\left(\frac{t}{t+1}\right) \quad (40)$$

Table 15. Compressive strength (MPa) of case 5.

Age (Unit: day)	Compressive Strength
1	25.63
2	40.71
3	48.50
5	52.30
7	53.20
10	55.00
20	62.30
28	64.00
50	65.00

**Figure 7.** The fitting results of case 5.**Table 16.** The fitting mean errors and standard deviations of case 5.

Index	Logarithmic Model	Hyperbolic Model	Polynomial Model	Universal Curve Model
Fitting mean error $\bar{\delta}$	0.3389	0.1337	0.5670	0.1332
Fitted standard deviation of error σ	0.0937	0.0309	0.1621	0.0276

From Table 16, it can be seen that the fitting accuracy of the new model is the highest. This indicates that the new model can more accurately evaluate the compressive strength of concrete.

3.2. Evaluation of Elastic Modulus

Case 6. In reference [30], Case 5 was followed as the experimental design scheme. Table 17 presents the experimental data of elastic modulus. The fitting curve, depicted in Figure 8, was obtained using four models. The average fitting error and standard deviation of the four models are stated in Table 18. The specific fitting equations for the four models are as follows:

$$z(t) = 29.5469 + 3.6042 \ln(t) \quad (41)$$

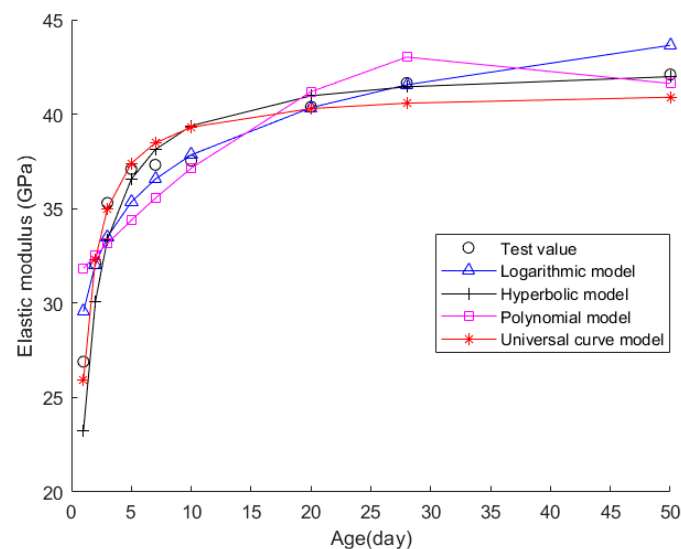
$$z(t) = \frac{t}{0.0196 + 0.0234 \cdot t} \quad (42)$$

$$z(t) = 31.1455 + 0.6973 \cdot t - 0.0098 \cdot t^2 \quad (43)$$

$$z(t) = -16.9105 \cdot \left(\frac{t}{t+1}\right) + 74.1154 \cdot \arctan\left(\frac{t}{t+1}\right) \quad (44)$$

Table 17. Elastic modulus (GPa) of case 6.

Age (Unit: day)	Elastic Modulus
1	26.90
2	32.10
3	35.30
5	37.10
7	37.32
10	37.54
20	40.39
28	41.65
50	42.10

**Figure 8.** The fitting results of elastic modulus in case 6.**Table 18.** The fitting mean errors and standard deviations for elastic modulus of case 6.

Index	Logarithmic Model	Hyperbolic Model	Polynomial Model	Universal Curve Model
Fitting mean error $\bar{\delta}$	0.1278	0.1701	0.2164	0.0787
Fitted standard deviation σ	0.0325	0.0425	0.0549	0.0158

According to the fitting average error and standard deviation of the four models in Table 18, it can be seen that the fitting average error and fitting standard error of the new model are smaller than those of the hyperbolic model, the logarithmic model, and the polynomial model. The average fitting error of the new model is about 61% of the logarithmic model, 46% of the hyperbolic model, and 36% of the polynomial model. Regarding fitting standard deviation, that of the new model is about 48% of that of the logarithmic model, about 37% of that of the hyperbolic model, and 28% of that of the polynomial model. The results indicate that the proposed new evaluation model is more accurate than the other three models in evaluating the elastic modulus of concrete.

Case 7. In reference [34], some concrete cube test blocks were manufactured with a design strength of 90 MPa. The mix proportions of concrete materials are listed in Table 19. The experimental data of elastic modulus are shown in Table 20, and the fitting curves

are shown in Figure 9. The average fitting error and standard deviation of the models are shown in Table 21. The specific fitting equations for the four models are as follows:

$$z(t) = -9.6390 + 8.1827 \ln(t) \quad (45)$$

$$z(t) = \frac{t}{0.8203 + 0.0278 \cdot t} \quad (46)$$

$$z(t) = 5.9162 + 0.4396 \cdot t - 0.0018 \cdot t^2 \quad (47)$$

$$z(t) = 1085.9326 \cdot \left(\frac{t}{t+1}\right) - 1341.0266 \cdot \arctan\left(\frac{t}{t+1}\right) \quad (48)$$

Table 19. The mix proportions of concrete materials in case 7.

Cement	Slag	Silica Fume	Water	Sand	Stone
0.5	0.4	0.1	0.23	1.2	1.8

Table 20. Elastic modulus of concrete cube in case 7 (GPa).

Age (Unit: hour)	Elastic Modulus
18	10.53
24	18.88
48	22.39
72	28.24
168	30.02

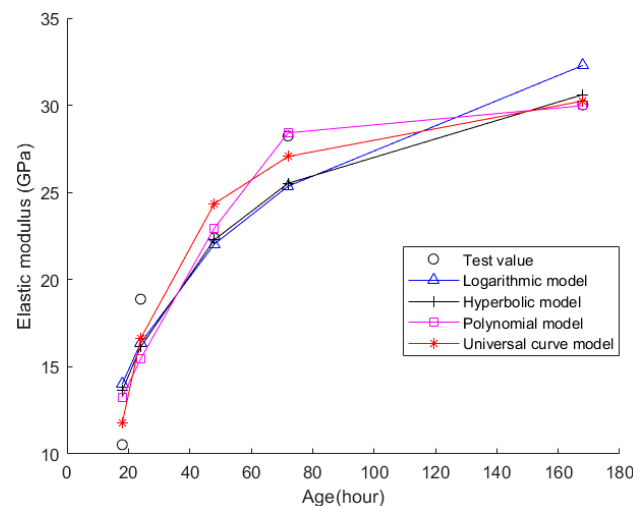


Figure 9. The fitting results of elastic modulus in case 7.

Table 21. The fitting mean errors and standard deviations of case 7.

Index	Logarithmic Model	Hyperbolic Model	Polynomial Model	Universal Curve Model
Fitting mean error $\bar{\delta}$	0.3788	0.3428	0.3174	0.1944
Fitted standard deviation σ	0.1194	0.1170	0.1182	0.0495

According to Table 21, the new model's fitting average error and standard deviation are smaller than those of the other models. It has been shown that the new model can more accurately evaluate the elastic modulus of concrete.

Case 8. The components of concrete materials are listed in Table 22. The experimental data of elastic modulus in reference [39] are shown in Table 23. The fitting curves of the

four models are shown in Figure 10, and the corresponding fitting errors are shown in Table 24. The specific fitting equations for the four models are as follows:

$$z(t) = 8.6594 + 13.6630 \ln(t) \quad (49)$$

$$z(t) = \frac{t}{0.0749 + 0.0175 \cdot t} \quad (50)$$

$$z(t) = 9.1545 + 4.8177 \cdot t - 0.1236 \cdot t^2 \quad (51)$$

$$z(t) = 344.8215 \cdot \left(\frac{t}{t+1}\right) - 367.0881 \cdot \arctan\left(\frac{t}{t+1}\right) \quad (52)$$

Table 22. The components of concrete materials in case 8 (kg/m³).

Cement	Fine Aggregate (Machine Made Sand)	Coarse Aggregate	Water	Water Reducer	Admixture
465.9	478.8	1069.9	125.8	8.2	54.8

Table 23. Elastic modulus of concrete cube in case 8 (GPa).

Age (Unit: day)	Elastic Modulus
2	13.25
3	21.86
4	27.76
5	29.90
6	38.29
7	41.33
14	46.93
28	48.02

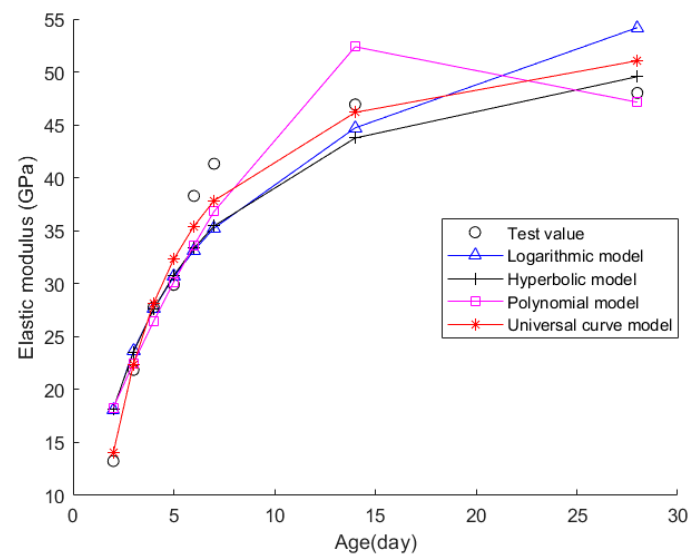


Figure 10. The fitting results of case 8.

Table 24. The fitting mean errors and standard deviations in case 8.

Index	Logarithmic Model	Hyperbolic Model	Polynomial Model	Universal Curve Model
Fitting mean error $\bar{\delta}$	0.4490	0.4344	0.4345	0.1667
Fitted standard deviation σ	0.1143	0.1177	0.1210	0.0293

Table 24 shows that the new model's average error and standard deviation are smaller than those of the hyperbolic, polynomial, and logarithmic models. This again indicates that the new model can better evaluate the elastic modulus of concrete than the existing models. Using Equation (7), the predicted long-term elastic modulus values in cases 6–8 are 41.30 MPa, 32.69 MPa, and 56.51 MPa, respectively. The coarse and fine aggregates in Cases 6 and 7 are both natural sand and gravel, while the fine aggregates in Case 8 are machine-made sand and the coarse aggregates are crushed stone. It can be observed that the type of aggregate has a significant impact on the elastic modulus of concrete, and the mechanism sand used in Case 8 may increase the elastic modulus. The research results of references [46,47] also show that the use of machine-made sand can improve the elastic modulus of concrete to a certain extent. Further research is needed in the future on the impact of aggregate types on elastic modulus of concrete.

3.3. High Strength Concrete and Lightweight Concrete

More types of concrete are further used to validate the new model to show its wider applicability and its practical implications. Cases 9 and 10 are high-strength concrete and Case 11 is lightweight concrete.

Case 9. In reference [43], a concrete mix design was developed by carefully combining different particle sizes of aggregates, setting the adjustment factor of cementitious materials, and determining parameters such as sand ratio, water–cement ratio, and dosage of admixtures. The relevant concrete material components for the three match ratios are listed in Table 25. The experimental data of compressive strength are shown in Table 26. Figures 11–13 present the fitting curves of the logarithmic, hyperbolic, polynomial, and new models for the three match ratios, respectively. In the Appendix A, Case 9 is also used as an example to elaborate in detail on the fitting process of the new model. The average fitting errors and standard deviations of these models are shown in Table 27. The specific fitting equations for the four models are as follows:

Table 25. The components of concrete materials in case 9 (kg/m³).

Match Ratio	Adjustment Coefficient	Cement	Mineral Admixtures	Microsphere	Silica Fume	Spall	Artificial-Sand	Admixture	Water
1	1.0	182	26	26	17	1203	985	6.2	55
2	1.2	218	31	31	20	1169	956	7.4	66
3	1.5	273	39	39	26	1117	914	9.3	83

Table 26. Compressive strength of concrete blocks in case 9 (MPa).

Age (Unit: day)	Compressive Strength		
	Match Ratio 1	Match Ratio 2	Match Ratio 3
1	11.5	17.8	26.2
3	23.8	31.5	43.6
7	28.8	40.0	56.8
28	33.3	46.2	60.5
60	37.8	50.3	66.9
90	42.3	57.5	71.6

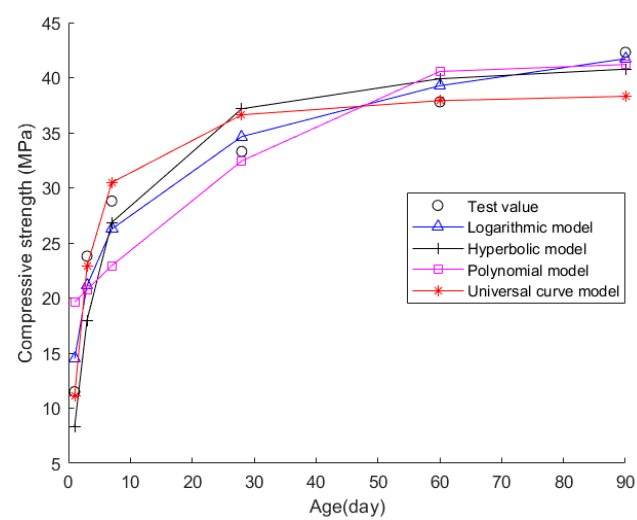


Figure 11. The fitting results for match ratio 1 in case 9.

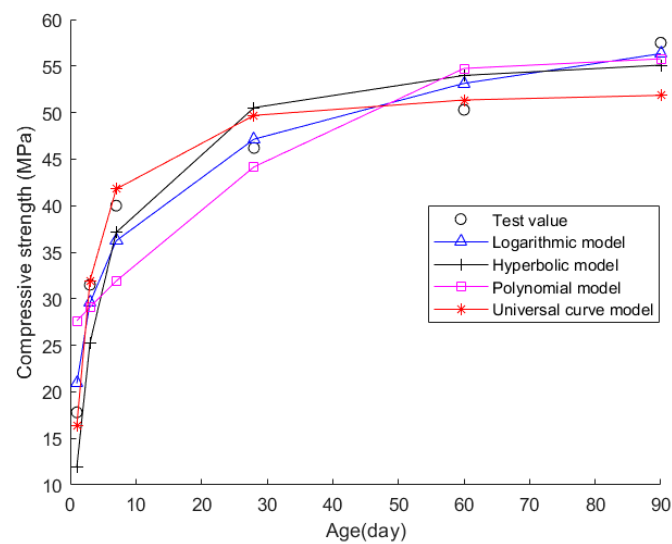


Figure 12. The fitting results for match ratio 2 in case 9.

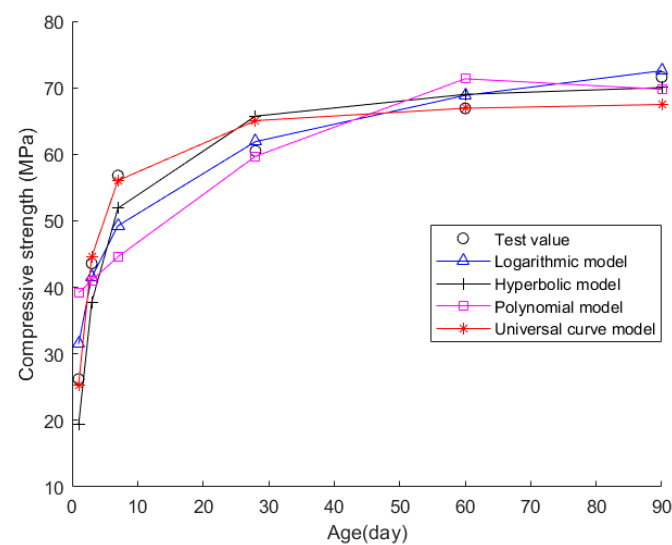


Figure 13. The fitting results for match ratio 3 in case 9.

Table 27. The fitting mean errors and standard deviations from the four models in case 9.

Match Ratio	Index	Logarithmic Model	Hyperbolic Model	Polynomial Model	Universal Curve Model
1	Fitting mean error $\bar{\delta}$	0.3013	0.3971	0.7499	0.1568
	Fitted standard deviation σ	0.0894	0.1024	0.2597	0.0380
2	Fitting mean error $\bar{\delta}$	0.2174	0.4132	0.6019	0.1561
	Fitted standard deviation σ	0.0579	0.1105	0.1989	0.0343
3	Fitting mean error $\bar{\delta}$	0.2500	0.3215	0.5478	0.1036
	Fitted standard deviation σ	0.0761	0.0879	0.1858	0.0281

For the first match ratio, one obtains

$$z(t) = 14.4787 + 6.0536 \ln(t) \quad (53)$$

$$z(t) = \frac{t}{0.0964 + 0.0235 \cdot t} \quad (54)$$

$$z(t) = 19.0340 + 0.5839 \cdot t - 0.0038 \cdot t^2 \quad (55)$$

$$z(t) = 131.9067 \cdot \left(\frac{t}{t+1}\right) - 118.1620 \cdot \arctan\left(\frac{t}{t+1}\right) \quad (56)$$

For the second match ratio, one obtains

$$z(t) = 20.9103 + 7.8711 \ln(t) \quad (57)$$

$$z(t) = \frac{t}{0.0667 + 0.0174 \cdot t} \quad (58)$$

$$z(t) = 26.8811 + 0.7510 \cdot t - 0.0048 \cdot t^2 \quad (59)$$

$$z(t) = 164.1922 \cdot \left(\frac{t}{t+1}\right) - 141.7374 \cdot \arctan\left(\frac{t}{t+1}\right) \quad (60)$$

For the third match ratio, one obtains

$$z(t) = 31.5073 + 9.1214 \ln(t) \quad (61)$$

$$z(t) = \frac{t}{0.0379 + 0.0139 \cdot t} \quad (62)$$

$$z(t) = 38.2221 + 0.9551 \cdot t - 0.0067 \cdot t^2 \quad (63)$$

$$z(t) = 167.9351 \cdot \left(\frac{t}{t+1}\right) - 126.4422 \cdot \arctan\left(\frac{t}{t+1}\right) \quad (64)$$

To quantify the improved accuracy over existing models, Table 28 shows the relative ratios of fitting error and standard deviation for each model, when the error and standard deviation of the polynomial model are set to 1.

From Table 28, it can be seen that the relative values of the fitting error and standard deviation of the new model are the smallest, which are approximately 20% of the values of the polynomial model. These results indicate that the new model has significantly improved fitting accuracy compared to existing models. The practical implications of these results mainly lie in the precise correspondence between compressive strength and time can be obtained via the fitting equation of the new model. For example, when using concrete to manufacture large-volume box beams, the prestressing force of the steel bars should be applied as early as possible in order to prevent concrete cracks. In engineering practice, it is generally required that the compressive strength of concrete reach at least 25 MPa before prestressing can be applied, which requires accurate estimation of the corresponding time

when the compressive strength reaches 25 MPa. This goal can be achieved via the specific fitting equation obtained from the new model. For concrete with the first match ratio, the time t corresponding to $z(t) = 25$ MPa obtained from the fitting Equation (56) of the new model is $t = 3.70$ d. For concrete with the second match ratio, the time t corresponding to $z(t) = 25$ MPa obtained from the fitting Equation (60) of the new model is $t = 1.85$ d. For concrete with the third match ratio, the time t corresponding to $z(t) = 25$ MPa obtained from the fitting Equation (64) of the new model is $t = 0.98$ d. These results can be used to guide prestressed construction in bridge engineering.

Table 28. The relative ratios of fitting error and standard deviation in case 9 (based on polynomial model).

Match Ratio	Relative Ratios	Logarithmic Model	Hyperbolic Model	Polynomial Model	Universal Curve Model
1	Fitting mean error	0.4018	0.5295	1.0000	0.2091
	Fitted standard deviation	0.3442	0.3943	1.0000	0.1463
2	Fitting mean error	0.3612	0.6865	1.0000	0.2593
	Fitted standard deviation	0.2911	0.5556	1.0000	0.1724
3	Fitting mean error	0.4564	0.5869	1.0000	0.1891
	Fitted standard deviation	0.4096	0.4731	1.0000	0.1512

Case 10. In reference [44], two types of high-strength concrete test blocks were designed: natural aggregate concrete and self-compacting aggregate concrete. These test blocks contained three different design strengths. The 100 mm × 100 mm × 100 mm concrete specimens were manufactured and tested. The mix proportions of concrete materials are listed in Table 29. The experimental data of their compressive strength are shown in Table 30. Figures 14–19 show the fitting curves of the four models, and Table 31 presents the fitting average errors and standard deviations. The specific fitting equations for the four models are as follows:

Table 29. Mix proportions of concrete in case 10 (kg/m³).

Number	Water	Cement	Fly Ash	Sand	Natural Aggregate	Water Reducer (%)	Water–cement Ratio (%)
NA-C-30	189.0	420	0	680.0	1110.0	0.07	45
NA-C-40	171.0	450	0	640.0	1138.0	0.33	38
NA-C-50	189.0	450	0	670.0	1090.0	0.10	42
NA-SCC-30	192.0	336	144	839.0	839.0	1.00	57
NA-SCC-40	176.8	364	156	826.6	826.0	1.20	49
NA-SCC-50	180.0	350	150	835.0	835.0	1.20	51

Table 30. Compressive strength of concrete blocks in case 10 (MPa).

Age (Unit: day)	Compressive Strength					
	NA-C-30	NA-C-40	NA-C-50	NA-SCC-30	NA-SCC-40	NA-SCC-50
7	21.67	29.43	29.75	19.37	24.87	30.73
28	44.17	50.71	53.40	35.33	49.40	53.20
56	45.70	52.75	55.47	40.53	55.60	55.75
90	47.22	53.86	54.72	40.57	57.91	57.13

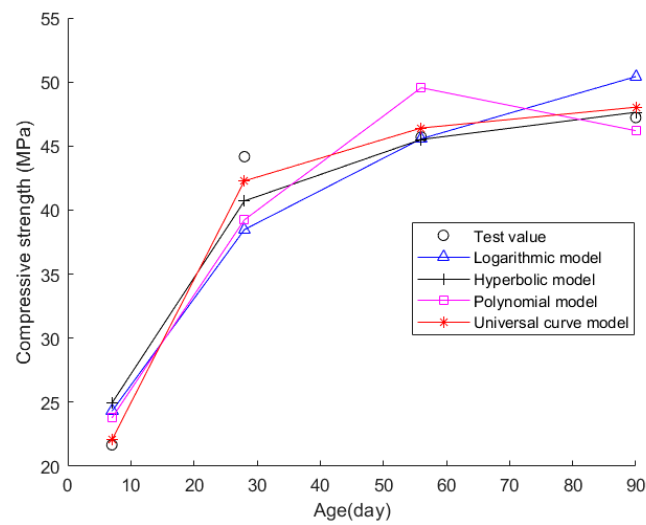


Figure 14. The fitting results of NA-C-30 in case 10.

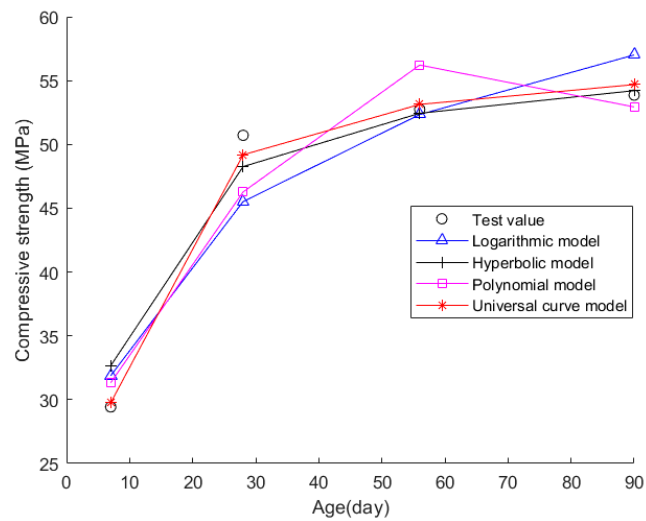


Figure 15. The fitting results of NA-C-40 in case 10.

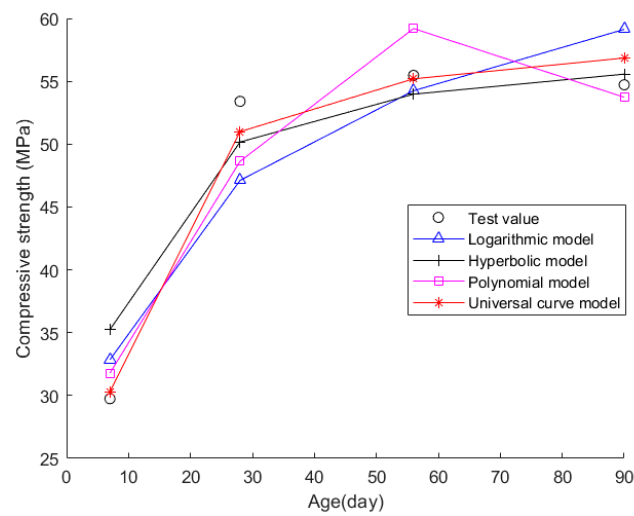


Figure 16. The fitting results of NA-C-50 in case 10.

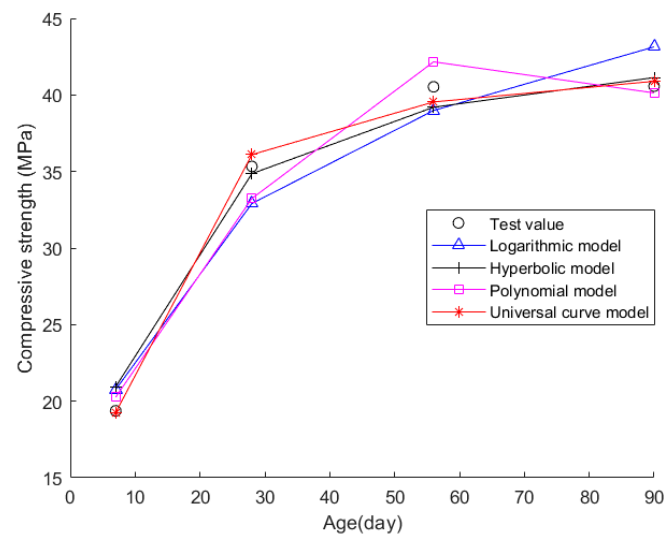


Figure 17. The fitting results of NA-SCC-30 in case 10.

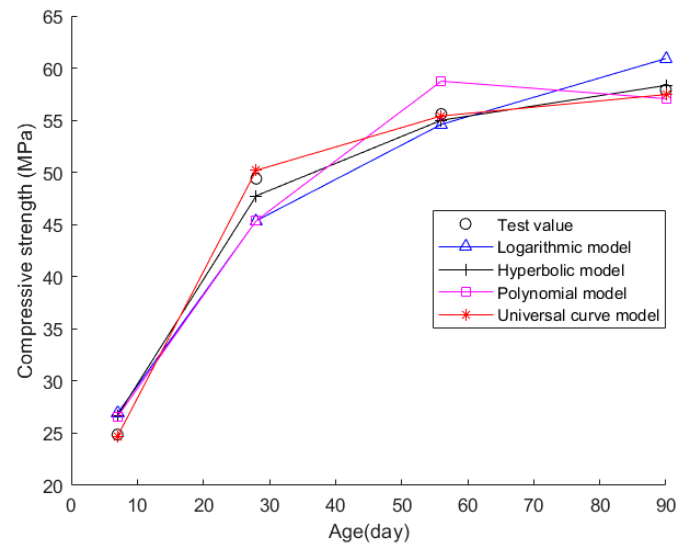


Figure 18. The fitting results of NA-SCC-40 in case 10.

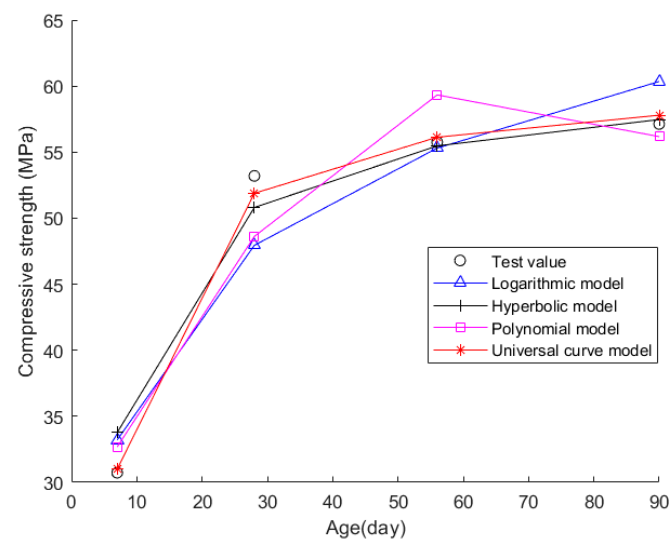


Figure 19. The fitting results of NA-SCC-50 in case 10.

Table 31. The fitting mean errors and standard deviations from the four models in case 10.

Number	Index	Logarithmic Model	Hyperbolic Model	Polynomial Model	Universal Curve Model
NA-C-30	Fitting mean error $\bar{\delta}$	0.1895	0.1700	0.1715	0.0517
	Fitted standard deviation σ	0.0583	0.0690	0.0395	0.0131
NA-C-40	Fitting mean error $\bar{\delta}$	0.1445	0.1201	0.1281	0.0362
	Fitted standard deviation σ	0.0409	0.0489	0.0296	0.0100
NA-C-50	Fitting mean error $\bar{\delta}$	0.1780	0.1970	0.1322	0.0627
	Fitted standard deviation σ	0.0423	0.0776	0.0301	0.0185
NA-SCC-30	Fitting mean error $\bar{\delta}$	0.1238	0.0899	0.0858	0.0343
	Fitted standard deviation σ	0.0153	0.0320	0.0205	0.0095
NA-SCC-40	Fitting mean error $\bar{\delta}$	0.1279	0.0811	0.1219	0.0195
	Fitted standard deviation σ	0.0303	0.0301	0.0291	0.0055
NA-SCC-50	Fitting mean error $\bar{\delta}$	0.1398	0.1091	0.1260	0.0295
	Fitted standard deviation σ	0.0394	0.0444	0.0293	0.0081

For natural aggregate concrete with a design strength of 30 MPa (NA-C-30), one obtains

$$z(t) = 4.3947 + 10.2281 \ln(t) \quad (65)$$

$$z(t) = \frac{t}{0.1451 + 0.0194 \cdot t} \quad (66)$$

$$z(t) = 17.1393 + 1.0002 \cdot t - 0.0075 \cdot t^2 \quad (67)$$

$$z(t) = 607.0926 \cdot \left(\frac{t}{t+1}\right) - 708.3099 \cdot \arctan\left(\frac{t}{t+1}\right) \quad (68)$$

For natural aggregate concrete with a design strength of 40 MPa (NA-C-40), one obtains

$$z(t) = 12.7301 + 9.8404 \ln(t) \quad (69)$$

$$z(t) = \frac{t}{0.0924 + 0.0174 \cdot t} \quad (70)$$

$$z(t) = 24.9121 + 0.9669 \cdot t - 0.0073 \cdot t^2 \quad (71)$$

$$z(t) = 564.4074 \cdot \left(\frac{t}{t+1}\right) - 645.6422 \cdot \arctan\left(\frac{t}{t+1}\right) \quad (72)$$

For natural aggregate concrete with a design strength of 50 MPa (NA-C-50), one obtains

$$z(t) = 12.8294 + 10.2890 \ln(t) \quad (73)$$

$$z(t) = \frac{t}{0.0787 + 0.0171 \cdot t} \quad (74)$$

$$z(t) = 24.4671 + 1.1061 \cdot t - 0.0087 \cdot t^2 \quad (75)$$

$$z(t) = 603.5390 \cdot \left(\frac{t}{t+1}\right) - 692.4944 \cdot \arctan\left(\frac{t}{t+1}\right) \quad (76)$$

For self-compacting aggregate concrete with a design strength of 30 MPa (NA-SCC-30), one obtains

$$z(t) = 3.7122 + 8.7625 \ln(t) \quad (77)$$

$$z(t) = \frac{t}{0.1778 + 0.0223 \cdot t} \quad (78)$$

$$z(t) = 14.7409 + 0.8316 \cdot t - 0.0061 \cdot t^2 \quad (79)$$

$$z(t) = 503.7077 \cdot \left(\frac{t}{t+1}\right) - 586.3501 \cdot \arctan\left(\frac{t}{t+1}\right) \quad (80)$$

For self-compacting aggregate concrete with a design strength of 40 MPa (NA-SCC-40), one obtains

$$z(t) = 0.9832 + 13.3191 \ln(t) \quad (81)$$

$$z(t) = \frac{t}{0.1544 + 0.0154 \cdot t} \quad (82)$$

$$z(t) = 18.6628 + 1.1919 \cdot t - 0.0085 \cdot t^2 \quad (83)$$

$$z(t) = 772.2900 \cdot \left(\frac{t}{t+1}\right) - 905.7117 \cdot \arctan\left(\frac{t}{t+1}\right) \quad (84)$$

For self-compacting aggregate concrete with a design strength of 50 MPa (NA-SCC-50), one obtains

$$z(t) = 12.5453 + 10.6228 \ln(t) \quad (85)$$

$$z(t) = \frac{t}{0.0926 + 0.0164 \cdot t} \quad (86)$$

$$z(t) = 25.8732 + 1.0270 \cdot t - 0.0077 \cdot t^2 \quad (87)$$

$$z(t) = 608.5387 \cdot \left(\frac{t}{t+1}\right) - 697.6181 \cdot \arctan\left(\frac{t}{t+1}\right) \quad (88)$$

To quantify improved accuracy over existing models, Table 32 shows the relative ratios of fitting error and standard deviation for each model, when the error and standard deviation of the hyperbolic model are set to 1.

Table 32. The relative ratios of fitting error and standard deviation in case 10 (based on hyperbolic model).

Number	Relative Ratios	Logarithmic Model	Hyperbolic Model	Polynomial Model	Universal Curve Model
NA-C-30	Fitting mean error	1.1147	1.0000	1.0088	0.3041
	Fitted standard deviation	0.8449	1.0000	0.5725	0.1899
NA-C-40	Fitting mean error	1.2032	1.0000	1.0666	0.3014
	Fitted standard deviation	0.8364	1.0000	0.6053	0.2045
NA-C-50	Fitting mean error	0.9036	1.0000	0.6711	0.3183
	Fitted standard deviation	0.5451	1.0000	0.3879	0.2384
NA-SCC-30	Fitting mean error	1.3771	1.0000	0.9544	0.3815
	Fitted standard deviation	0.4781	1.0000	0.6406	0.2969
NA-SCC-40	Fitting mean error	1.5771	1.0000	1.5031	0.2404
	Fitted standard deviation	1.0066	1.0000	0.9668	0.1827
NA-SCC-50	Fitting mean error	1.2814	1.0000	1.1549	0.2704
	Fitted standard deviation	0.8874	1.0000	0.6599	0.1824

From Table 32, one can observe that the relative values of the fitting error and standard deviation of the new model are the smallest, approximately 20%–30% of the values of the hyperbolic model. These results again indicate that the new model has significantly improved fitting accuracy compared to existing models. These results can also be used to accurately evaluate the corresponding time when the compressive strength of concrete will reach 25 MPa, in order to guide prestressed construction in practical engineering. Without loss of generality, for NA-C-50, the time t corresponding to $z(t) = 25$ MPa obtained

from the fitting Equation (76) of the new model is $t = 5.63$ d. For NA-SCC-50, the time t corresponding to $z(t) = 25$ MPa obtained from the fitting Equation (88) of the new model is $t = 5.50$ d.

In order to explore the relationship between the fitting parameters (a and b) of the new model and the water–cement ratio (w/c) of concrete, Figure 20 shows the variation curves of parameters a , b , and $a + b \cdot \frac{\pi}{4}$ with the water–cement ratio. From Figure 20, it can be seen that the variation patterns of the absolute values of parameters a and b are basically consistent. When the water–cement ratio is about 50%, the long-term strength (i.e., $a + b \cdot \frac{\pi}{4}$) of concrete is relatively great, up to 60 MPa.

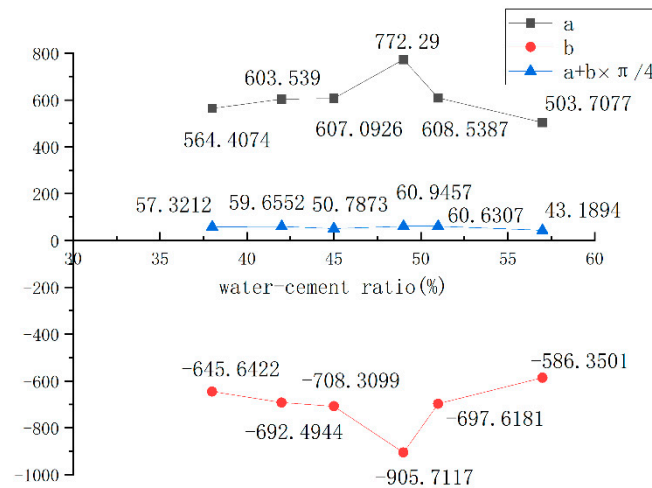


Figure 20. The variation curves of parameters a , b , and $a + b \cdot \frac{\pi}{4}$ with the water–cement ratio of case 10.

Case 11. In reference [45], some lightweight concrete specimens with dimensions of $100 \text{ mm} \times 100 \text{ mm} \times 100 \text{ mm}$ were manufactured using CL30 grade concrete, cured under standard curing conditions. The composition of the concrete can be found in Table 33. The test results of the compressive strength for the three batches are shown in Table 34. Figures 21–23 show the fitted curves of the four models based on experimental data. The fitting average error and standard deviation are listed in Table 35. The fitting equations for the four models are as follows:

Table 33. Mix proportions of concrete in case 11 (kg/m^3).

Cement	Water	Sand	Coarse Aggregate
386	244	705	591

Table 34. Compressive strength of concrete blocks in case 11 (MPa).

Age (Unit: day)	Compressive Strength		
	Batch 1	Batch 2	Batch 3
3	22.1	18.8	16.8
7	32.0	28.5	27.7
14	35.2	35.7	34.0
21	38.3	36.8	38.0
28	39.7	37.3	38.7

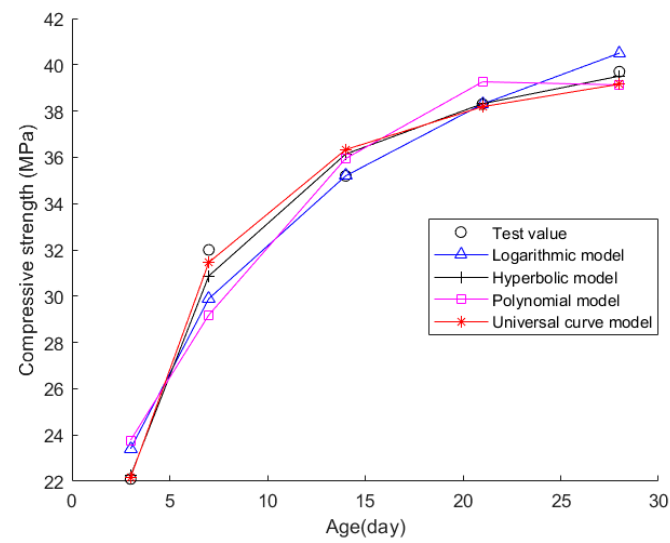


Figure 21. The fitting results of Batch 1 in case 11.

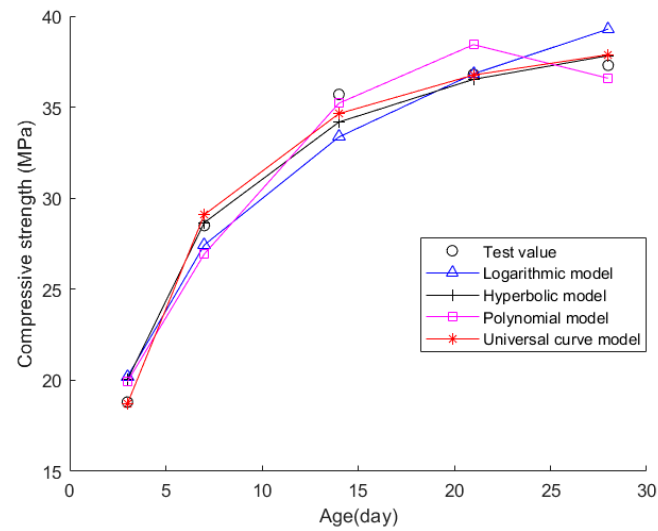


Figure 22. The fitting results of Batch 2 in case 11.

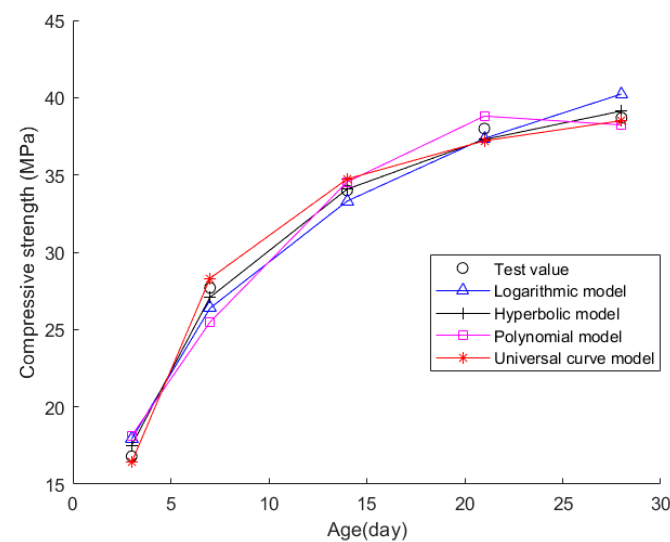


Figure 23. The fitting results of Batch 3 in case 11.

Table 35. The fitting mean errors and standard deviations of the four models in case 11.

Batch	Index	Logarithmic Model	Hyperbolic Model	Polynomial Model	Universal Curve Model
1	Fitting mean error $\bar{\delta}$	0.0908	0.0448	0.1215	0.0390
	Fitted standard deviation σ	0.0317	0.0153	0.0342	0.0121
2	Fitting mean error $\bar{\delta}$	0.1182	0.0798	0.956	0.0398
	Fitted standard deviation σ	0.0288	0.0261	0.0210	0.0116
3	Fitting mean error $\bar{\delta}$	0.0946	0.0523	0.1164	0.0441
	Fitted standard deviation σ	0.0204	0.0148	0.0347	0.0076

For batch 1, one obtains

$$z(t) = 15.0016 + 7.6525 \ln(t) \quad (89)$$

$$z(t) = \frac{t}{0.0660 + 0.0230 \cdot t} \quad (90)$$

$$z(t) = 18.9782 + 1.7046 \cdot t - 0.0352 \cdot t^2 \quad (91)$$

$$z(t) = 180.1339 \cdot \left(\frac{t}{t+1}\right) - 175.5051 \cdot \arctan\left(\frac{t}{t+1}\right) \quad (92)$$

For batch 2, one obtains

$$z(t) = 10.7864 + 8.5543 \ln(t) \quad (93)$$

$$z(t) = \frac{t}{0.0789 + 0.0236 \cdot t} \quad (94)$$

$$z(t) = 13.5875 + 2.2685 \cdot t - 0.0517 \cdot t^2 \quad (95)$$

$$z(t) = 220.5330 \cdot \left(\frac{t}{t+1}\right) - 227.9646 \cdot \arctan\left(\frac{t}{t+1}\right) \quad (96)$$

For batch 3, one obtains

$$z(t) = 6.9371 + 9.9926 \ln(t) \quad (97)$$

$$z(t) = \frac{t}{0.1061 + 0.0218 \cdot t} \quad (98)$$

$$z(t) = 11.5844 + 2.3295 \cdot t - 0.0492 \cdot t^2 \quad (99)$$

$$z(t) = 268.1057 \cdot \left(\frac{t}{t+1}\right) - 286.9670 \cdot \arctan\left(\frac{t}{t+1}\right) \quad (100)$$

To quantify the improved accuracy over existing models, Table 36 shows the relative ratios of fitting error and standard deviation for each model, when the error and standard deviation of the logarithmic model are set to 1.

From Table 36, one can observe that the relative values of the fitting error and standard deviation of the new model are the smallest, approximately 30%–40% of the values of the logarithmic model. These results indicate that the fitting equation obtained from the new model can more accurately evaluate the correspondence between compressive strength and time. The fitting equation of the new model can be used not only to guide prestressed construction, as in Cases 9 and 10, but also to guide the time for template removal. For example, if the template can be removed when the compressive strength reaches 15 MPa, the removal time for the first batch of concrete can be calculated from the fitting Equation (92) of the new model as $t = 1.73$ d. The removal time for the second batch of

concrete can be calculated from the fitting Equation (96) as $t = 2.31$ d. The removal time for the third batch of concrete can be calculated from the fitting Equation (100) as $t = 2.74$ d.

Table 36. The relative ratios of fitting error and standard deviation in case 11 (based on logarithmic model).

Batch	Relative Ratios	Logarithmic Model	Hyperbolic Model	Polynomial Model	Universal Curve Model
1	Fitting mean error	1.0000	0.4934	1.3381	0.4295
	Fitted standard deviation	1.0000	0.4826	1.0789	0.3817
2	Fitting mean error	1.0000	0.6751	8.0880	0.3367
	Fitted standard deviation	1.0000	0.9063	0.7292	0.4028
3	Fitting mean error	1.0000	0.5529	1.2304	0.4662
	Fitted standard deviation	1.0000	0.7255	1.7010	0.3725

4. Conclusions

This article proposes a new curve model for evaluating the mechanical properties of concrete. This model can evaluate the mechanical properties of concrete at different ages by fitting a small amount of experimental data. The model was validated using multiple sets of experimental data. Based on the calculation results, the following main conclusions can be drawn:

- Logarithmic models and exponential models have similar advantages and disadvantages. Both models are incremental functions, which align with the law of mechanical properties of concrete increasing with time in engineering practice, and both lack a point of $t = 0$. The hyperbolic model can gradually approach a fixed value as time increases, which is consistent with the fact that the mechanical properties of concrete remain unchanged in the later stage. However, when dealing with some very discrete data, the fitting accuracy of the logarithmic, exponential, and hyperbolic models is not very good. The advantage of the polynomial model is that its domain of definition includes the point $t = 0$. However, its fitting function is a monotonically increasing function. It cannot be used for the prediction of the mechanical properties of concrete in the later stage.
- The new model proposed in this article can be seen as a combination of the hyperbolic and arctangent functions. This model makes up for the deficiencies of the existing models and includes the point $t = 0$. While it tends toward infinity, the function of this new model can approach a specific value, which is in line with the fact that the mechanical properties of concrete remain unchanged in the later stages. In addition, the new model only needs to fit two parameters, making it easier to use in practical engineering than the three-parameter model.
- Based on the case studies of multiple sets of experimental data, the results show that the new model proposed in this article can evaluate concrete's compressive strength and elastic modulus. This model can evaluate the mechanical properties of concrete under different curing environments and concrete strength levels and has a broader range of applications in practical engineering.

It should be noted that this article mainly discusses the law of changes in the mechanical properties of concrete over time, and other factors that affect the mechanical properties of concrete are not fully considered. The proposed model cannot simultaneously establish the relationship between multiple factors and the mechanical properties of concrete like machine learning. However, the proposed model only requires a small amount of experimental data to establish the mathematical equation for evaluation. In the case of insufficient experimental data, the proposed model can be combined with ANNs to simulate and generate more data for the training of ANNs, thereby improving the generalization ability of the ANN model. Further research is needed in the future to verify the fitting and evaluation capabilities of this new model under more complex conditions. This new model may also be applicable to other cementitious materials besides concrete. In subsequent

studies, relevant experiments on other cementitious materials can be conducted to verify the feasibility of this dual parameter curve model.

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Appendix A

Without losing generality, Case 9 can be taken as an example to elaborate in detail on the fitting process of the new model. The fitting process of other models is similar to that of this new model. As mentioned earlier, the experimental data in Case 9 have been provided in Table 26. Next, the new model is used to fit the compressive strength data of concrete specimens made according to the first mix ratio in Case 9 at different ages. Using the data from the first column of Table 26, the matrix C can be calculated using Equation (11) as

$$C = \begin{bmatrix} 0.5000 & 0.4636 \\ 0.7500 & 0.6435 \\ 0.8750 & 0.7188 \\ 0.9655 & 0.7679 \\ 0.9836 & 0.7771 \\ 0.9890 & 0.7799 \end{bmatrix} \quad (A1)$$

Using the data from the second column of Table 26, the vector y can be obtained using Equation (9) as

$$y = \begin{pmatrix} 11.50 \\ 23.80 \\ 28.80 \\ 33.30 \\ 37.80 \\ 42.30 \end{pmatrix} \quad (A2)$$

Substituting Equations (A1) and (A2) into (8), the linear system corresponding to the new model can be obtained as

$$\begin{pmatrix} 11.50 \\ 23.80 \\ 28.80 \\ 33.30 \\ 37.80 \\ 42.30 \end{pmatrix} = \begin{bmatrix} 0.5000 & 0.4636 \\ 0.7500 & 0.6435 \\ 0.8750 & 0.7188 \\ 0.9655 & 0.7679 \\ 0.9836 & 0.7771 \\ 0.9890 & 0.7799 \end{bmatrix} \cdot x \quad (A3)$$

Multiplying both sides of Equation (A3) by C^T , as shown in Equation (12), one obtains

$$\begin{bmatrix} 159.9672 \\ 129.2835 \end{bmatrix} = \begin{bmatrix} 4.4560 & 3.6205 \\ 3.6205 & 2.9475 \end{bmatrix} \cdot x \quad (\text{A4})$$

From Equation (A4), the least squares solution of x can be obtained using Equation (13) as

$$\hat{x} = \begin{Bmatrix} 131.9067 \\ -118.1620 \end{Bmatrix} \quad (\text{A5})$$

From Equation (A5), the two fitting parameters of the new model are obtained as $a = 131.9067$ and $b = -118.1620$. Therefore, the specific fitting equation for the new model corresponding to the first mix ratio of Case 9 is

$$z(t) = 131.9067 \cdot \left(\frac{t}{t+1}\right) - 118.1620 \cdot \arctan\left(\frac{t}{t+1}\right) \quad (\text{A6})$$

The result shown in Equation (A6) is that shown in Equation (56).

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Article

Study on Flexural Strength of Interface between Full Lightweight Ceramsite Concrete and Ordinary Concrete

Hongbing Zhu ^{1,2,3,*}, Yu Xiao ¹, Xiu Li ⁴, Ye Wang ¹ and Siyu Wen ¹

¹ School of Urban Construction, Wuhan University of Science and Technology, Wuhan 430065, China; xiaoyu1572570451@163.com (Y.X.); iyewang@163.com (Y.W.); 18970520527@163.com (S.W.)

² High Performance Engineering Structure Research Institute, Wuhan University of Science and Technology, Wuhan 430065, China

³ Hubei Provincial Engineering Research Center of Urban Regeneration, Wuhan University of Science and Technology, Wuhan 430065, China

⁴ School of Transportation Engineering, Wuhan Technical College of Communications, Wuhan 430065, China; hubeilixiu@163.com

* Correspondence: zhuhongbing@wust.edu.cn

Abstract: The efficacy of full lightweight ceramsite concrete as a restorative material has been widely acknowledged, given its light weight, strength, and durability. However, the extent of its performance in repairing existing or old concrete remains uncertain. This study examined the reparation of flexural performance with full lightweight ceramsite concrete, using 14 different combinations of old and new concrete test blocks. The primary focus of the study was on investigating the flexural bond strength of the interface between the old and the new concrete. This included understanding the effects of the interfacial roughness, interfacial agent type, and concrete curing age of the concrete on the flexural strength. The test results showed that increasing the interface roughness from 0 mm to 5 mm resulted a restoration of the flexural strength of the sample by approximately 59%. Additionally, the flexural strength of the specimens was restored by 62%–78% of their original strength with the application of different types of interfacial agent. To rank the impact of these factors on the flexural strength, a univariate analysis of variance was conducted. This allowed us to establish a mathematical formula for calculating the flexural capacity of old and new concrete interfaces, taking the three aforementioned factors into account.

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Keywords: full lightweight ceramsite concrete; interfacial roughness; interfacial agent; curing age; flexural strength

1. Introduction

Numerous building structures annually encounter durability challenges due to adverse environmental conditions and external loads. For the safe operation of these structures, it is indispensable to repair and reinforce the whole or part of these structures. The patching and strengthening of concrete structures have thus emerged as crucial research topics in construction engineering [1,2]. Various bonding methods exist for concrete, most notably the sectional increase method [3] and the mechanical connection method [4]. The former, a widely adopted technique [5], is implemented by pouring new concrete onto existing concrete. However, this leads to the generation of new and old concrete interfaces, the most vulnerable parts of the repaired structure [6,7]. The mechanical properties of these interfaces critically influence the overall reliability of the structure, making their study an intriguing research subject.

In recent years, numerous experimental and theoretical studies have shed light on the factors affecting these interfaces' mechanical properties. Factors such as old concrete intensity [8], water content [9], new concrete intensity [10], the interval between new and old concrete pouring [11], interface roughness [12], surface-preparation methods [13], the

use of interface agents [14], and implanted steel bars [15] have all been identified as key determinants. In addition, the size of the bonding surface also has a certain impact on the bond strength. Some studies have found that the incorporation of fibers in concrete can reduce the size effect due to the bridging effect of fibers [16]. Advanced studies proposed that new and old concrete interfaces can be considered as concrete material 100 microns wide, with their mechanical strength determined by their microstructure [17,18]. Detailed microscopic studies further illuminated these findings.

Despite these insights, most studies have focused on the tensile and shear strengths of interfaces, while the knowledge on their flexural strength remains limited [19]. Given the inevitable cold joint defects in concrete structures due to on-site construction complexities, the cold joint (the interface between new and old concrete) significantly weakens the component's flexural performance [20]. Meanwhile, under the action of bending load, the concrete bonding interface produces a complex stress state. This is expressed as partial tensile stress at the bonded surface, and the remainder is subjected to compressive stress. The areas with the greatest tensile stress may produce cracks [21]. Consequently, considering the interface's flexural strength is essential [22]. Flexural testing is considered to be a more convenient and practical bonding test by some scholars [23]. On one hand, based on the results obtained by Madani [24] and Farzad [9], flexural strength is lower than tensile strength and shear strength. It requires a greater level of attention. On the other hand, the JC/T 2381-2016 [25] specification recommends describing the bonding strength of an interface based on its flexural strength, as it better represents the interface's carrying capacity under bending and stretching. Hence, the study of the flexural strength of the interface offers immense research value.

Full lightweight ceramsite concrete is a widely used artificial lightweight aggregate concrete, renowned for its light weight, high strength [26], superior fire resistance [27,28], strong seismic performance [29], and excellent durability [30]. It also presents substantial economic benefits. Previous studies suggested that under a constant water–cement ratio, the early autogenous shrinkage of ceramsite concrete decreases as the pre-wetting humidity of ceramsite increases, due to its water absorption and desorption capacity [31]. However, the addition of ceramsite with high water absorption or an excessive amount of ceramsite may have a negative impact on the mechanical properties of ceramsite concrete, such as the tensile strength, compressive strength, and elastic modulus [32,33]. Additionally, it is considered a promising new type of cover material, particularly for the repair and reinforcement of old concrete [11,12,14]. Despite these benefits, research on full lightweight ceramsite concrete interfaced with ordinary concrete, particularly in terms of its flexural strength, has remained largely unexplored. Based primarily on the flexural strength, the effect of repairing ordinary lightweight ceramsite concrete is unknown. The influence of various factors on the flexural strength of this type of interface remains unclear, and an applicable calculation formula is yet to be established. Therefore, a comprehensive exploration of the flexural strength of this type of interface is warranted.

This paper aims to systematically investigate the flexural strength of the interface between full lightweight ceramsite concrete and ordinary concrete. To achieve this goal, the first involved measuring the flexural strength of this interface via a bending test. Subsequently, the influence law and the degree of various factors, including interfacial roughness, the type of interfacial agent, and the concrete curing age, on this strength are revealed. Finally, a calculation model for the flexural strength of the interface is established. The intention of this research is to provide a scientific reference for a concrete-repair project that utilizes full lightweight ceramsite concrete as the repair material.

2. Bending-Strength Test of Bonding Surface

2.1. Concrete Raw Materials and Mix Ratio

Ordinary Portland cement, P.O.42.5, was adopted. Its density was 3.15 g/cm³ and its fineness was 6.5%. The mineral composition of cement clinker was C₃S, C₂S, C₃A, and C₃AF, and their contents were 45%, 25%, 12%, and 8%, respectively. The coarse aggregate

used for preparing old concrete was ordinary crushed stone, and the coarse aggregate used for preparing new concrete was 900-grade crushed stone shale ceramsite. The ceramsite particles were pre-wetted and fully drained before use. Subsequently, a surface covering measure was used. River sand was used as fine aggregate in the preparation of old concrete. Among the fine aggregates used in construction of new concrete is ceramsite sand. The physical characteristics of aggregate are shown in Table 1. Before the experiment, the fine aggregate was screened through a 4.75-mm square-hole sieve, and its relevant performance parameters met the specification requirements. High-performance polycarboxylate water reducers were used in the admixture. There were four types of interface agent: cement paste, silica-fume cement paste, polymer, and epoxy resin. The type of cement used in cement slurry was the same as that of concrete. In addition, the main component of silicon fume was SiO_2 , and its content was 96.74%. Its unit weight was 1600–1700 kg/m^3 and specific surface area was 20–28 g/m^2 . The technical characteristics of polymer interface agent are shown in Table 2. The mixing-water type was tap water.

Table 1. Physical characteristics of aggregates.

Types of Aggregate	Particle Size (mm)	Apparent Density (kg/m^3)
Crushed stone	5~20	1517
Shale ceramsite	5~20	1760
Ordinary river sand	0~5	2641
Ceramsite sand	0~5	1479

Table 2. Technical characteristics of polymer-interface agents.

Type of Interface Agent	State	Color	Shear Bond Strength (MPa)	Tensile Bond Strength (MPa)	Solidification Time (d)
Polymer	Liquid	Milky white	1.5	1.0	1
Epoxy resin	Liquid	Milky white with a little yellow	1.6	0.8	2

The JGJ 55-2011 [34] and JGJ/T 12-2019 [35] provided specifications for the mix-ratio tests. Smooth preparation of C45 ordinary concrete and LC50 lightweight ceramsite concrete was achieved. The mix proportions are shown in Tables 3 and 4. Their compressive strength is shown in Table 5. After measurement and calculation, it was found that the density of full lightweight ceramsite concrete was 1741 kg/m^3 , and its slump and gas content were 70 mm and 5.65%, respectively. The density of C45 ordinary concrete was 2417 kg/m^3 , and its slump and gas contents were 75 mm and 6.71%, respectively.

Table 3. Mix proportion of C45 ordinary concrete (kg/m^3).

Gravel	Sand	Cement	Water	Water Reducing Agent
1050	565	605	230	0

Table 4. Mix proportion of LC50 full lightweight ceramsite concrete (kg/m^3).

Ceramsite	Ceramsite Sand	Cement	Water	Water Reducing Agent
620	600	550	154	5.1

2.2. Experimental Design of Flexural Strength

Using lightweight ceramsite concrete as a repair material, we studied the extent of the repair's recovery of flexural performance. Simultaneously, the effects of interfacial roughness, interfacial agent, and curing age of existing concrete on the flexural strength of

the repaired interface were discussed. It was possible to produce 14 groups of specimens (C-3, J-1, and L-90 are the same group of test blocks) in this experiment (see Table 6). The dimension of each flexural specimen was 100 mm × 100 mm × 400 mm. First, the old concrete specimen (100 mm × 100 mm × 200 mm) was adopted from C45 ordinary concrete. In addition, the old concrete specimens were cured for 3 days, 7 days, 14 days, 28 days, and 90 days after demolding. In addition, their surfaces needed to be chiseled manually. Referring to the experimental design in the relevant research [14,36–38], the interface roughness was treated as 0 mm, 1 mm, 3 mm, and 5 mm, respectively. Subsequently, different interfacial agents were applied to the old concrete surface. Finally, one end of the 100 mm × 100 mm × 400 mm mold was placed. The LC50 full lightweight ceramsite concrete was poured into these molds for bonding to the C45 ordinary concrete specimen, as shown in Figure 1. After 28 days of standard curing (temperature 20 ± 2 °C, relative humidity above 95%), the new and old concrete specimens were successfully acquired.

Table 5. Compressive strength of C45 ordinary concrete and LC50 lightweight ceramsite concrete.

Curing Age (d)	C45 Ordinary Concrete				LC50 Lightweight Ceramsite Concrete			
	Sample 1	Sample 2	Sample 3	Average Value	Sample 1	Sample 2	Sample 3	Average Value
7	31.96	36.30	38.54	35.60	45.60	44.00	47.23	45.61
14	43.97	41.64	44.23	43.28	50.08	50.42	52.23	50.91
28	48.95	45.76	50.68	48.46	55.34	52.93	52.71	53.66



Figure 1. Bonding method of new and old concrete.

The bonding effect of new and old concrete was influenced by multiple factors. One of the most important signs of successful bonding was the excellent treatment of the bonding surface of new and old concrete. Therefore, the influence of roughness on the flexural performance of the interface between old and new concrete needs to be investigated. Using a sand-cone method, the roughness of the interface was measured, and the old concrete-base surface was treated manually with chisels. The roughness values of chiseling treatment were 0 mm, 1 mm, 3 mm, and 5 mm, respectively. The roughness of the bonding surface was calculated using Equation (1).

$$h = \frac{V}{S} \quad (1)$$

In the equation, h is the average depth of sand filling (mm), S is the surface area of the joint surface (mm²), and V is the volume of sand filling (mm³).

Table 6. Grouping and interfacial flexural strength of bonding specimens between full lightweight ceramsite concrete and existing concrete.

Group Number	Interfacial Agent Type	Interfacial Roughness (mm)	Existing Concrete Age (d)	Age of New Concrete (d)	Flexural Strength (MPa)			Objective
					Sample 1	Sample 2	Sample 3	
Z-45	-	-	28	-	4.79	4.89	4.90	As a control group, it was used to evaluate the effect of the repair
Z-50					5.11	5.25	5.03	
C-0					2.89	2.85	2.84	
C-1					3.17	3.10	3.15	
C-3					3.33	3.28	3.14	
C-5	Cement paste	3	90 + 28	28	3.20	3.11	3.26	Exploring the effect of interfacial roughness on the flexural strength of the interface
J-0					3.06	2.98	3.02	
J-1					3.33	3.28	3.14	
J-2					3.35	3.41	3.38	
J-3					3.59	3.50	3.53	
J-4	Polymer class Epoxy resin	3	90 + 28	28	3.79	3.75	3.80	Exploring the effect of interfacial agent types on the flexural strength of the interface
L-3					4.01	3.84	4.03	
L-7					3.87	3.96	3.84	
L-14					3.72	3.52	3.74	
L-28					3.43	3.26	3.45	
L-90	Cement paste	3	90 + 28	28	3.33	3.28	3.14	Exploring the influence of existing concrete's curing age on interfacial flexural strength

Note: Z-45 and Z-50 represent integral test specimens with strength grades of C45 and LC50, respectively. C-0, C-1, C-3, and C-5 represent test specimens with interfacial roughness of 0, 1, 3, and 5 mm, respectively. J-0, J-1, J-2, J-3, and J-4 represent test specimens without interfacial agent and coated with cement paste, silcrete slurry, polymer class, and epoxy resin interfacial agent, respectively. L-3, L-7, L-14, L-28 and L-90 represent the bonding test specimens of existing ordinary concrete with ages of 3, 7, 14, 28, and 90 days, respectively.

This experiment used a microcomputer-controlled electro-hydraulic servo universal testing machine to load. A four-point bending test was conducted to explore the influence of various factors on the flexural strength of prisms (100 mm × 100 mm × 400 mm). The loading speed of the testing machine was controlled at 0.05 MPa/s~0.08 MPa/s, and the specific loading mode is illustrated in Figure 2. The flexural strength of the interface between old and new concrete was calculated by using Equation (2).

$$f_f = \frac{Fl}{bh^2} \quad (2)$$

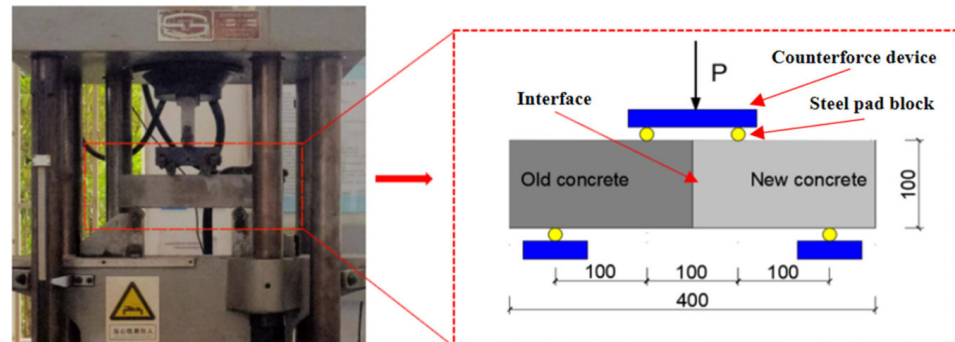


Figure 2. Bending test.

In the equation, f_f is the flexural strength of the concrete (MPa), F is the failure load of the specimen (N), l is the span between supports (mm), h is the cross-sectional height of the specimen (mm), and b is the cross-sectional width of the specimen (mm).

3. Results and Discussion

3.1. Test Phenomena and Strength

3.1.1. Experimental Phenomena of the Interface-Roughness Factor

According to Figure 3, the damage conditions of the interface at different roughness levels were different. Almost no sound was heard when the specimen with the interfacial roughness of 0 mm reached its ultimate load. Suddenly, the specimen broke at the point at which the new and old concrete interfaced. It can be observed from Figure 3a that the failure surface of the new and old concrete presented a smooth and complete state, and there was no aggregate attached to the fracture surface. When the specimen with the interfacial roughness of 1 mm reached the ultimate load, it made a slight cracking sound. Figure 3b shows the old concrete's fracture surface in black and white. The black part is mainly composed of ceramsite sand and shale ceramsite. This phenomenon showed that the interface between the new and old concrete had a bonding effect. The specimen with an interfacial roughness of 3 mm produced the sound of fine breakage during the loading process. It also produced the sound of clicking when the bearing capacity reached its limit. The specimen broke along the bonding surface of the new and old concrete. As shown in Figure 3c, more full lightweight ceramsite concrete was bonded on the failure surface of the old concrete. This shows that the new concrete and the old concrete combined very well. The failure state of the specimens with an interface roughness of 5 mm is displayed in Figure 3d. It can be seen that large areas of aggregate were exposed at the interface between old and new concrete.

3.1.2. Experimental Phenomena of Interfacial Agent Factors

The damage to the interface under different interface agents is indicated in Figure 4. The specimen without the interfacial agent did not produce a cracking sound accompanied by a uniform load increase. In contrast, the specimen coated with the interfacial agent generated a fine breaking sound. It was found that the specimen coated with the cement paste had more cement slurry on the fracture surface, as shown in Figure 4a,b. Figure 4c

shows that the fracture surface had obvious black silcrete slurry. In addition, a distinct white gummy substance was observed on the failure surface, as depicted in Figure 4d,e. Simultaneously, some broken gravel could be seen on the broken torus in Figure 4e.

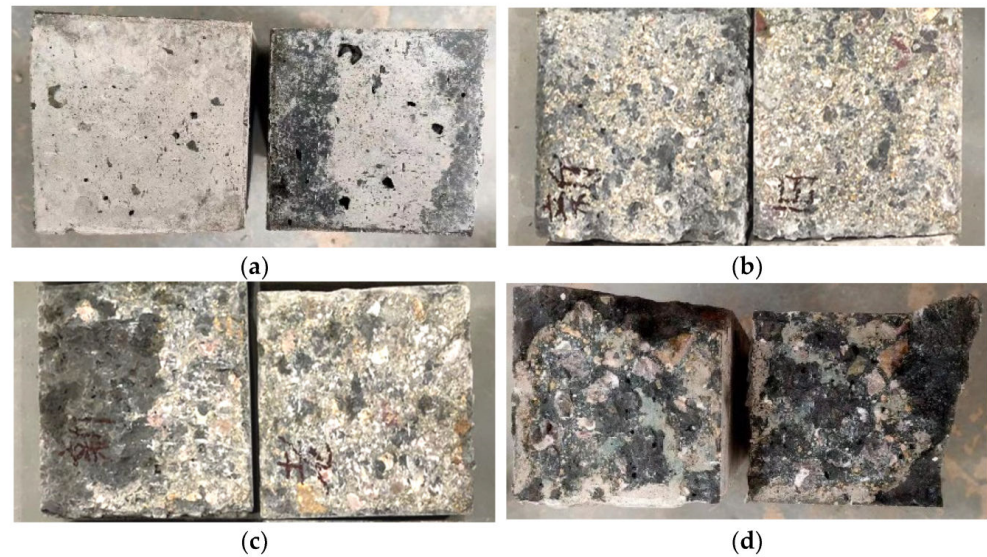


Figure 3. Failure states of specimens of different interfacial roughness levels: (a) failure state of a 0 mm-roughness specimen; (b) failure state of a 1 mm-roughness specimen; (c) failure state of a 3 mm-roughness specimen; (d) failure state of a 5 mm-roughness specimen.

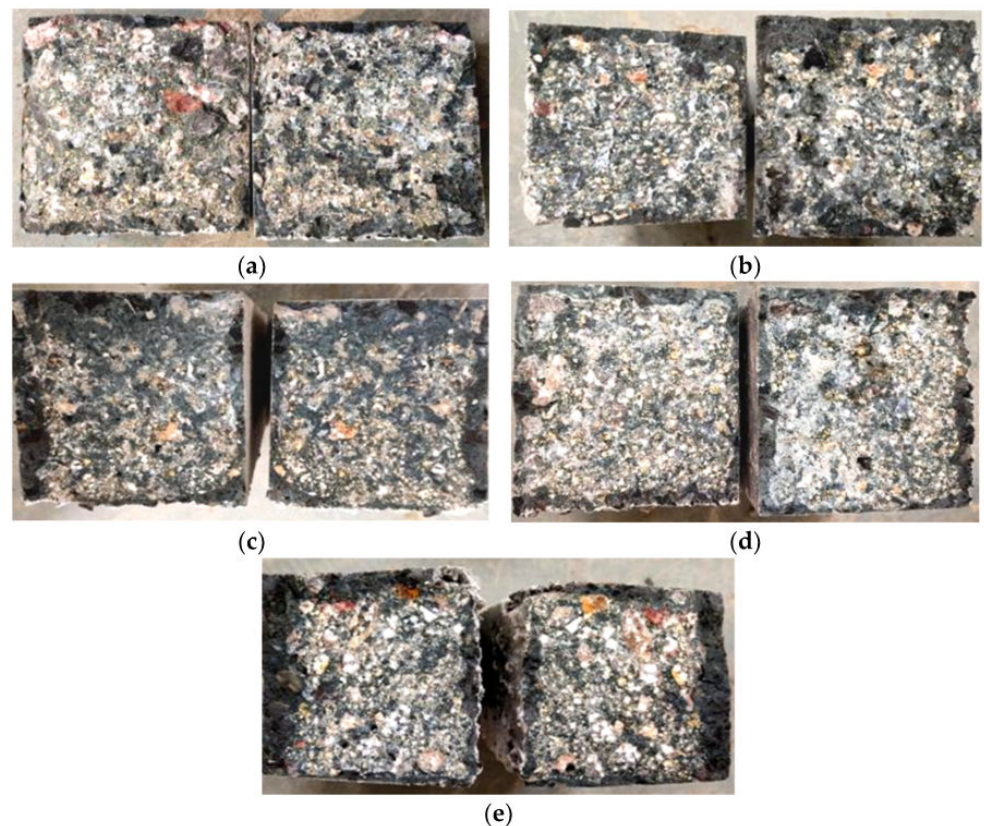


Figure 4. Failure states of specimens with different types of interfacial agent: (a) failure state of specimen without interfacial agents; (b) failure state of specimen with cement-paste interfacial agent; (c) failure state of specimen with the silica-fume agent; (d) failure state of specimen with epoxy-resin interfacial agent; (e) failure state of specimen with a polymer interfacial agent.

3.1.3. Experimental Phenomena of Age Factors in Existing Concrete

Figure 5 illustrates the failure conditions of specimens with different ages of concrete. The bonds tested differed significantly between the curing times of 3 days, 7 days, 14 days, and 28 days, 90 days. The fracture surfaces of the specimens were black and white, as shown in Figure 5a–c. In addition, a large area of aggregate was exposed at the interface when specimens aged 3 days were broken. This phenomenon showed that the interface between the new and the old concrete had a certain flexural performance at this time. The failure conditions of the specimens at 28 days and 90 days were essentially the same. Simultaneously, some aggregate was exposed at the interface, as shown in Figure 5d,e.

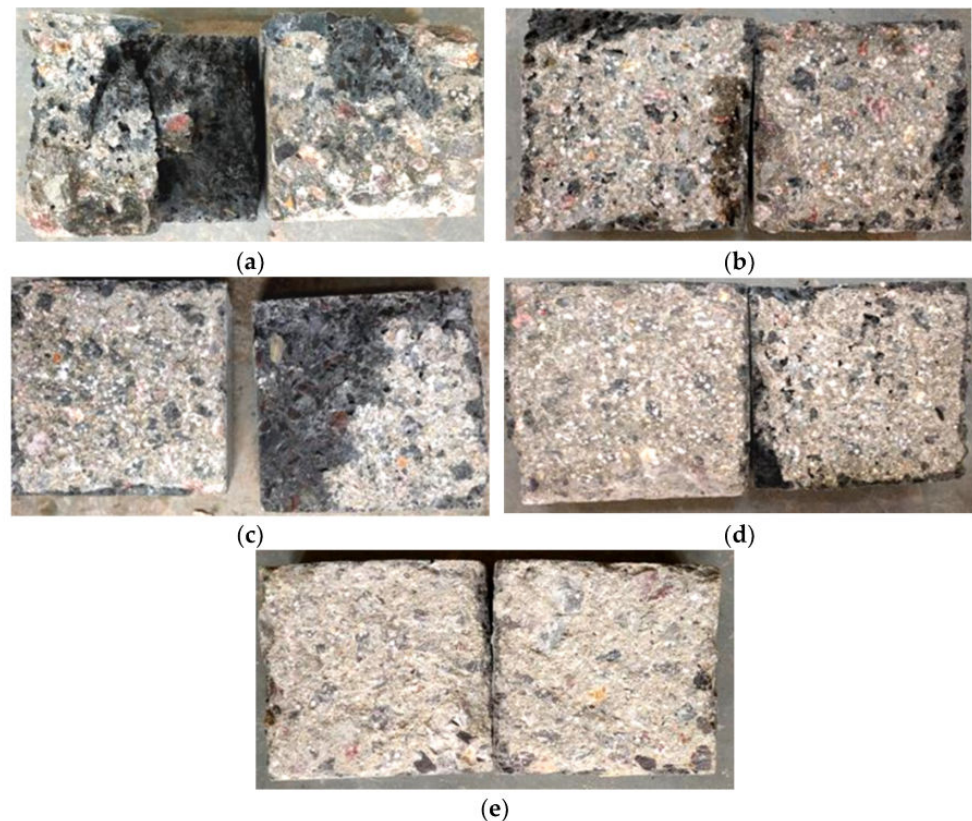


Figure 5. Failure states of specimens at different ages of existing concrete: (a) failure state of specimen at 3 days; (b) failure state of specimen at 7 days; (c) failure state of specimen at 14 days; (d) failure state of specimen at 28 days; (e) failure state of specimen at 90 days.

3.1.4. Test Results

The flexural strength was used as an index to analyze the effects of the interfacial roughness, the interfacial agent, and the age of the existing concrete on the flexural performance. The specimens in Table 6 were grouped and their flexural strengths are shown.

3.2. The Effect of Repair

By calculating the data in Table 6, the flexural strength of the LC50 full lightweight ceramsite concrete was found to be equivalent to that of the C50 ordinary concrete. Its strength grade was one level higher than that of the original structure. Therefore, LC50 lightweight concrete can be used as a repair material for C50 ordinary concrete, according to GB 50367-2013 [39]. The interface between the old and the new concrete was the most vulnerable part of the structure, so the flexural strength at the interface could represent the flexural strength of the repaired structure. The group Z-45 was undamaged old concrete, with an average bending strength of 4.86 MPa. The groups of C-0~C-5 and J-0~J-4 were specimens repaired by interfacial-roughness and interface agents, respectively. Their

flexural strengths ranged from 2.86 MPa to 3.96 MPa. It was seen from simple calculations that with the enhancement of the roughness, the flexural strength of the interface increased to varying degrees. It was possible to restore the flexural strength of the specimen to 59% of the undamaged specimen. The new and old concrete interfaces were coated with different interfacial agents. The flexural strength of the specimen was restored to 62%~78% of the undamaged strength.

3.3. Influencing Factors and Rules of Flexural Strength

3.3.1. Effect of Interfacial Roughness on Flexural Strength

It can be seen in Figure 6 that with the increase in the interfacial roughness, the flexural strength of the new and old concrete increased first and then decreased. The flexural strength of the interface reached its maximum when the interfacial roughness reached 3 mm. Moreover, a positive quadratic functional relationship between the two was observed through data fitting. The fitting relation is shown in Equation (3), and the R^2 is 0.93. It can be observed that, in Equation (3), when the roughness (average depth of the rough surface) is about 3 mm, the flexural strength reaches 3.31 MPa. Clearly, the interfacial roughness played an important role in the flexural strength of the interface. The reason for this phenomenon was that appropriate interfacial roughness can provide sufficient mechanical biting force to improve the flexural performance of the interface [40]. However, if the interfacial roughness is overtreated, microcracks occur in the near surface layer. In this way, the integrity of an existing concrete structure is damaged and the interfacial bond strength decreases [41]. As a result, in relevant repair works, the roughness of the interface between new and old concrete should be controlled reasonably.

$$f_f = -0.05h^2 + 0.3h + 2.86 \quad (3)$$

where f_f represents the flexural strength of the interface (MPa) and h represents the interfacial roughness (mm).

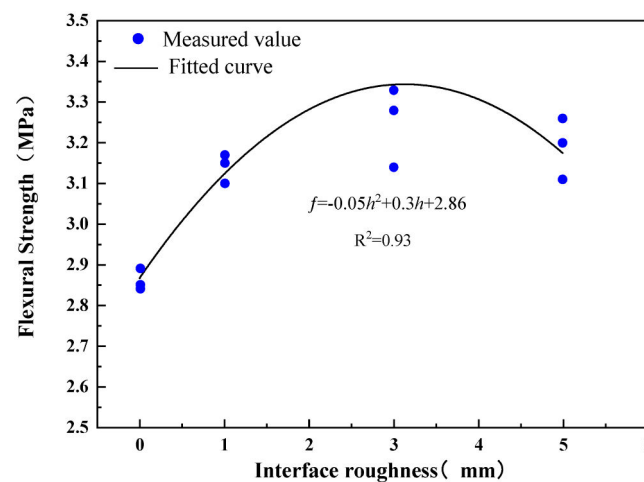


Figure 6. Impact of interfacial roughness on the flexural strength of interface.

3.3.2. Effect of Interfacial Agents on Flexural Strength

An investigation of the flexural strengths of specimens without and with various interfacial agents is presented in Figure 7. It can be observed that brushing the interfacial agent at the interface can effectively improve its flexural strength. The reason for this is that brushing the interfacial agent can improve the microstructure of the interfacial area, including the compactness, morphology, and distribution characteristics of hydration products [42]. Microlevel mechanical occlusion increases flexural strength at the interface by improving mechanical occlusion. In addition, the flexural strength of the interface was significantly improved after it was coated with an epoxy-resin interfacial agent. The use of

the specimen restored its flexural strength to 77% of its pre-damage value. The influence sequence of the interfacial agents was polymer-class interfacial agent, silica-fume interfacial agent, and cement-paste interfacial agent. This is because polymer-based organic interfacial agents form films and cover aggregates and cement particles with their surface chemistry. Polymers, which form the film, prevent micro-cracks from developing and enhance their adhesion to concrete [5]. This showed that brushing the interfacial agent was an effective way to enhance the interface flexural strength.

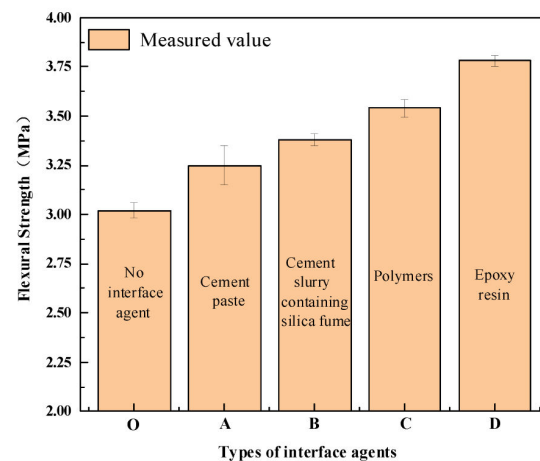


Figure 7. Impact of interfacial agents on the flexural strength of interface.

3.3.3. Effect of Curing Age on Flexural Strength of Existing Concrete

Figure 8 illustrates the effect of the differences in curing age of the existing concrete on the flexural strength of the interface between the new and the old concrete. It was found that the flexural strength of the existing concrete decreased rapidly when the curing age was less than 28 days. As the curing age continued to increase, the flexural strength decreased slowly with the growth of the curing age until it became stable. Due to the water membrane, the active ions in new concrete penetrate into the pores of existing concrete during the repair process. The existing concrete's pores are filled with reaction products when they react with cement. Eventually, a permeable layer is formed. However, with the increase in curing age, the existing concrete's porosity decreases gradually. Consequently, the permeable layer becomes thinner, reducing the adhesion between the new and the old concrete [43]. When the curing period exceeds 28 days, the concrete's strength stabilizes. It is rare for new concrete or old concrete to lose flexural strength with the aging process [44].

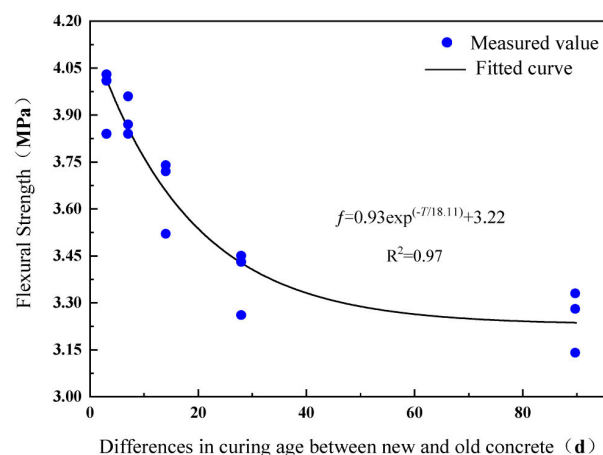


Figure 8. Impact of the differences in curing age of the existing concrete on the flexural strength of interface.